Development of a Performance-Based Mix Design for Porous Friction Courses

by

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Abstract

The development of a performance-based mix design procedure for Porous Friction Course (PFC) will help to mitigate some of the life cycle issues encountered with PFC pavements. PFC pavements are prone to raveling and cracking which lead to short service lives. A PFC is typically more expensive than a dense-graded mix due to required high quality aggregate materials, modified asphalt binder and higher asphalt binder contents. The use of PFC provides numerous safety benefits and also improves the noise quality of surrounding areas. Many agencies once used PFC for these reasons but have since halted its use due to performance issues. This study used laboratory performance tests to evaluate three PFC pavements that had good field performance (up to 18 years) and three PFC pavements that had poor performing field performance (less than 8 years).

This research study was composed of four parts. The first was to evaluate the six designs and determine if there was a distinguishable difference in performance results between the good and poor designs. The second part evaluated the use of increased P-200 content to provide more durable designs. The third section used asphalt binder modifiers to determine if mixture performance was affected. These designs had the stabilizing additive (fiber) removed from the design to evaluate if the binder modifiers could eliminate the need of the fiber as a stabilizing agent. The fourth and final part evaluated the strength of varying nominal maximum aggregate size (NMAS) mix designs at three different lift thicknesses to determine if the typical lift thickness (1.0 inch) of PFC pavements was adversely effecting performance of the designs.

A balanced mix design approach was selected for designing PFC pavements. Criteria and performance tests for durability, cracking and cohesiveness were selected. An increased P-200 content had a positive effect on almost all of the results, and it is recommended that the current P-200 gradation band be expanded. The binder modifications did not show total improvement but did provide some alternative design options. The NMAS to lift thickness ratio does not affect the durability of the mix.

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CHAPTER 1 – INTRODUCTION

BACKGROUND

Porous Friction Course (PFC) has been used in Europe and the United States for many decades. It is also known as Permeable European Mix (PEM), Open Graded Friction Course (OGFC) and Porous Asphalt (PA). PFCs are primarily used to improve safety by increasing the frictional properties of the pavement surface along with allowing surface water to drain through the pavement.

Despite the benefits, the use of PFC was diminished over the years due to durability and service life issues. A previous survey conducted by the National Center for Asphalt Technology (NCAT) showed that in 1998, 22 states had discontinued use of PFC (Kandhal P. S., 1998). A more recent survey conducted by NCAT as part of NCHRP Project 1-55, showed that out of 41 agencies (40 states and Puerto Rico) only half were using PFC mixes. Figure 1 depicts the results of the recent survey in regards to PFC usage. There has been little change in PFC usage by state agencies since the 1998 survey. The most recent survey showed that the agencies that did not use PFC felt that their designs were not adequate to maintain the expected performance life of PFC mixes. The primary distresses that reportedly caused premature failure are raveling and top-down cracking. Examples of raveling and cracking in a PFC mix can be seen in Figure 2.

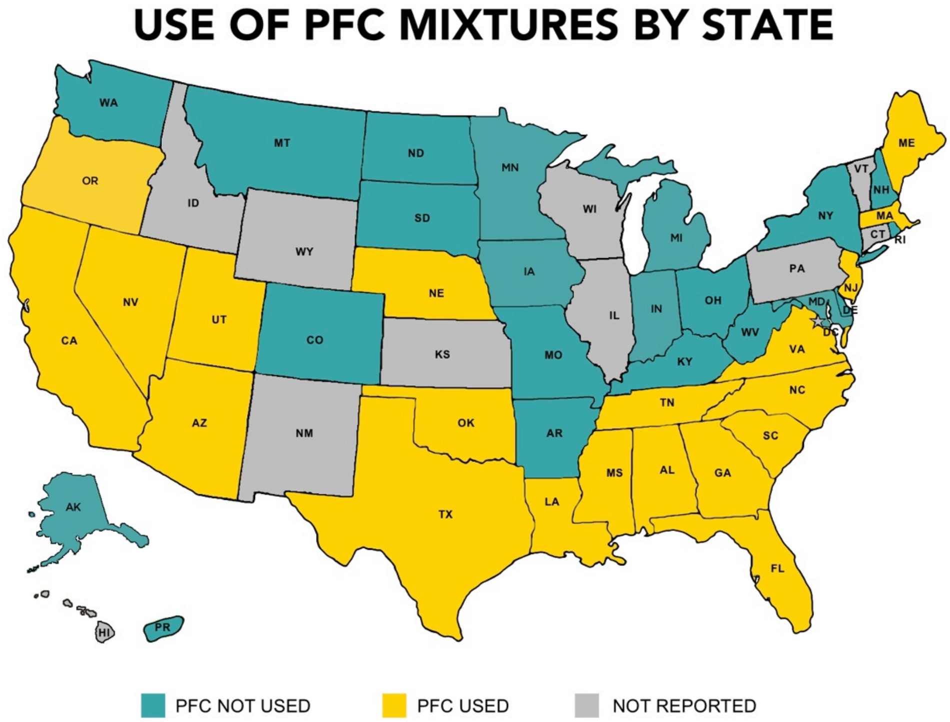


Figure 1 – 2015 Survey of PFC Usage

Top Down Cracking **Raveling**

Figure 2 – Primary Distresses Observed in PFC Mixes (NCAT, Fall 2014)

There are numerous factors that can lead to these types of distress. For example, the nominal maximum aggregate size (NMAS) and lift thickness play a part in raveling. Along with lift thickness, the amount of asphalt binder and air voids in the mix can influence a mix's susceptibility to raveling and cracking. The use of modified asphalts with specified minimum asphalt contents can help to prevent raveling and cracking, but there is still a need to address how to determine an optimum binder content. Currently, the glass pie plate drain-down method is still primarily used for determining optimum asphalt. This study will address durability and cracking issues during mix design and will also evaluate an array of performance tests to help determine what testing should be included in PFC mix design procedures.

OBJECTIVE AND SCOPE

The objective of this research is to develop a PFC design procedure that includes performance-based test procedures that address the types of distress commonly seen and is a viable procedure whether virgin aggregates, Reclaimed Asphalt Pavement (RAP) or Recycled Asphalt Shingles (RAS) are used in the mix design. Several laboratory tests will be used, and those that are most discriminating of successful performance will be selected for the design procedure. This research will include adjusting the asphalt, dust and fiber content of the mixes along with using lab performance tests to help determine what performance criteria should be defined in the specifications. Typical asphalt contents range from 5.5 – 7.0 percent and dust content ranges from 1-6 percent (Cooley, et al., 2009). A sample matrix with varying asphalt and dust contents will be tested for durability using an array of performance tests.

CHAPTER 2 – LITERATURE REVIEW

INTRODUCTION

The use of PFC mixes in the United States severely declined in the 1980s due to design and performance issues and was completely eliminated in some states. While conducting a literature review on PFC mixes, many of the reports and articles encountered were dated prior to this cessation. Some of these articles provided valuable background information, but they were based on prior mix design procedures and test methods. Hence, to better represent the currently used materials, mix design procedures and test methods, this literature review focuses more heavily on the post-cessation research.

BENEFITS OF POROUS MIXTURES

There are many benefits to PFC mixes with the majority of them being safety-related. The use of PFC to remove water from the surface provides good contact between tires and the pavement surface thus minimizing possible accidents and reducing traffic fatalities from occurring during rainy weather. Some of the benefits of PFC mixes include:

- Reduced risk of hydroplaning
- Increased friction resistance
- Reduced backsplash and spray from vehicle tires
- Reduced noise resulting from tire-pavement interaction
- Improved visibility of pavement markings

Depending on the required lift thickness, PFC pavements can also be economical because they can typically be placed in thinner lifts than dense-graded mixes (Kandhal P. , 2002).

Reduced Hydroplaning and Improved Friction

The risk of hydroplaning during a rain event is increased in low-lying areas or when rutting of a dense-graded mix has occurred. This surface water may cause a water film to form between the tire and the pavement, affecting the tire-pavement interface friction and the driver's ability to control the vehicle. A PFC surface allows water to drain through the surface and exit onto the shoulder (Figure 4). This limits the risk of hydroplaning and increases the friction resistance. By limiting the amount of water that is standing or flowing across the pavement surface, users are provided a safer traveling experience during rain events.

Nearly 6,000 people are killed and over 445,000 people are injured in weather-related crashes in the U.S. each year. The vast majority of most weather-related crashes, 73%, happen on wet pavement (Hamilton, 2016). A research study by the National Highway Traffic Safety Administration reported that the lifetime economic cost for each fatality was found to be $1.4 million. Therefore, any reduction in accidents and especially a reduction in traffic fatalities may have a dramatic impact on our society as a whole (NHTSA, 2014).

According to Huber (2000), many agencies have seen a decrease in wet pavement accidents on roadways with PFC mixes. A traffic study for Japan in 2010 showed that OGFC significantly reduced the number of fatalities during rainy weather in that country when results were compared to standard dense-graded mix (Figure 3).

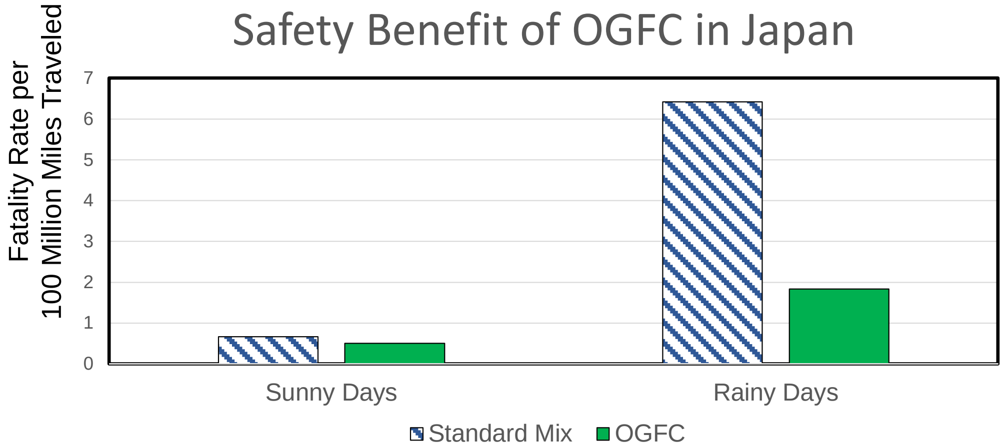


Figure 3 – Fatality Reduction on Rainy Days (Shimeno & T., 2010)

Figure 4 – Illustration of Water Transport on PFC and Dense-Graded Mixes (Porous Pavement)

Backsplash, Spray and Glare Reduction

Backsplash and spray from vehicles can diminish a driver's view of the paint striping and surrounding vehicles. A PFC allows the water to drain through the pavement and consequently reduces the effect of backsplash and spray significantly when compared to dense-graded mixes. A study conducted by the Texas Department of Transportation (TxDOT) compared a PFC and dense-graded mix in regards to backsplash and spray. The comparison showed dramatic changes (Figure 5).

Dense-Graded Mix **PFC Mixture**

Figure 5 – Backsplash Comparison Performed by TxDOT (Rand, 2004)

Huber (2000) stated that the United Kingdom reported a 90 – 95 percent reduction in backsplash and spray for PFC mixtures when compared to dense-graded mixes. As can be seen in Figure 5, the pavement markings on the PFC mixture have a higher degree of visibility during wet conditions when compared to the dense-graded mix. This is especially beneficial at night during wet weather. When a film of water is on the pavement surface, it can reflect a vehicle's headlights, and the glare can prevent the driver from following the pavement markings. The reflective glare from the water mitigates the reflective beads in the pavement markings and keeps the driver from being able to distinguish lane stripes.

Pavement Noise Reduction

While some highway noise comes from the vehicles, a large part of this noise comes from the pavement-tire interaction (Figure 6). This is especially true when the highway speed is above 45 miles per hour (mph). Metropolitan areas seem to have the most need for noise reduction due to the close proximity of businesses and homes to the highway. The most significant reason for the need of noise reduction is the quality of life of the population. The noise can become an annoyance to humans, which leads to negative impacts on the quality of life. It can also have an economic impact on real estate by keeping properties from being developed or sold (Donavan, 2007).

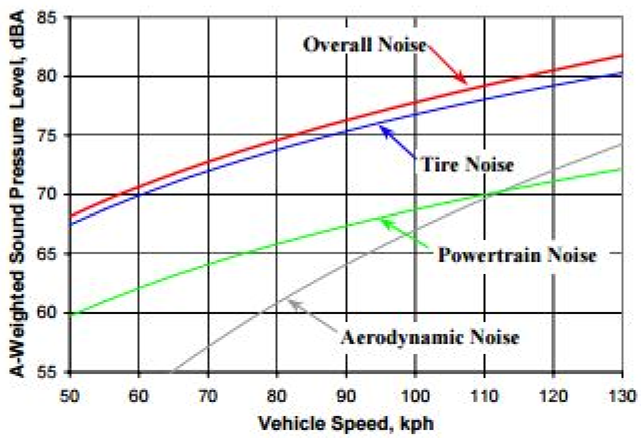


Figure 6 – Contribution of Highway Noise (Donavan, 2007)

There are a few different methods for mitigating the highway noise. One of the methods approved by the Federal Highway Administration (FHWA) is the use of noise barrier walls. While being somewhat effective, depending on distance from the source, these walls are extremely costly and often unsightly. Noise reduction is dependent on the distance, both horizontally and vertically, from the highway to the point source in question. A typical reduction of 5 decibels (dB(A)) is expected from a noise wall. A gap-graded, thin lift PFC can provide on average a reduction of 3 dB(A) (Bernhard & Wayson, 2004). Joint research conducted by NCAT, FHWA and several state agencies has shown that there can be as much as 1.5 dB(A) difference in noise between tire manufacturers alone.

MIX DESIGNS

The main function of a PFC mix is to remove water from the surface of the pavement. This helps prevent vehicles from hydroplaning and also eliminates backsplash and spray from the vehicles during rain events, allowing better visibility for all operators. A PFC pavement must be permeable enough to drain the water away from the surface and off the roadway while still providing adequate friction. An added benefit is the noise reduction achieved from the open design of the pavement. Current design of PFC mixtures requires four major components.

1. Suitable materials
2. An adequate design blend gradation
3. The optimum binder content
4. Evaluation of potential performance

These four components are critical in designing a PFC mix; however the degree of variability between agency practices allows for differences in the performance of these mixes. The focus of this study is to take an analytical approach to these differences and to develop performance tests and related thresholds that will help prevent current distresses such as premature raveling and top-down cracking. The following sections describe the four components in more detail.

Suitable Materials

PFC mixes consist primarily of coarse aggregate, fine aggregate, asphalt binder and stabilizing additives. In order to attain the high air void content required for PFC mixes, an open-graded aggregate gradation is required. This consists primarily of coarse aggregate so that the mix can maintain stone-on-stone contact by creating a stone skeleton of the coarse particles. The stone skeleton is essential because it provides the structure and strength of the mix. Aggregate mineralogy is not specified in national specifications because of limitations and availability of local aggregates. The importance of the aggregate properties and type is most notable in the stone skeleton. The coarse aggregate provides the stone-on-stone contact while the fine aggregate and stabilizing additives help to maintain the mix's stability. Flat and elongated particles can fracture during construction causing weakness and gaps in the stone skeleton, while non-durable stone can also break down during production and construction. A culmination of current aggregate property requirements can be found in Table 1. This table compares the requirements from ASTM D7064, *Standard Practice for Open-Graded Friction Course (OGFC) Mix Design*, and AASHTO PP77, *Standard Practice for Materials selection and Mixture Design of Permeable Friction Courses (PFCs)* (AASHTO, 2014). As stated previously, these requirements and recommendations may be altered by agencies if local materials cannot meet the minimum requirements.

Table 1 – Aggregate Requirements/Recommendations for PFC Mix Designs

Test Description	Method	ASTM 7064		AASHTO PP77	
		Min.	Max.	Min.	Max.
Coarse Aggregate					
Los Angeles Abrasion, % Loss	AASHTO T 96	-	30	-	30
Flat or Elongated, % (5 to 1)	ASTM D 4791	-	10	-	10
Soundness (5 Cycles), %					
• Sodium Sulfate	AASHTO T 104	-	-	-	10
• Magnesium Sulfate		-	-	-	15
Uncompacted Voids	AASHTO T 326, Method A	-	-	45	-
Fine Aggregate					
Soundness (5 Cycles), %					
• Sodium Sulfate	AASHTO T 104	-	-	-	10
• Magnesium Sulfate		-	-	-	15
Uncompacted Voids	AASHTO T 304 Method A	40	-	45	-
Sand Equivalency	AASHTO T 176	45	-	50	-

The use of modified binder has become common practice for most agencies due mostly to empirical results. The use of tire rubber, styrene-butadiene-styrene (SBS) and styrene-butadiene-rubber (SBR) as asphalt modifiers has proven to increase the durability of PFC mixtures by increasing the stiffness and ductility of the binder. The increased stiffness promotes increased film thicknesses while also preventing draindown of the mixture during production, transport and construction. Determining the optimum stiffness is important when choosing a modifier. Ruiz et al. suggested that an overly stiff binder will oxidize faster, which can lead to raveling issues prior to the expected design life (Ruiz, 1990). If modified appropriately, the binder may prevent short-term raveling that is caused by the shear forces between the tire-pavement interface (Molenaar, 2000).

Stabilizing additives are used to improve the durability of the mixture by preventing draindown and also by increasing the mixture's tensile strength (Pasetto, 2000). When draindown occurs during production and transportation of the PFC mixture, a significant amount of the asphalt binder is lost from the mix. This loss of binder can cause decreased durability, which may lead to premature raveling or cracking. Stabilizing additives, such as mineral and cellulose fiber, can help prevent draindown along with reinforcing the film thickness of the asphalt binder.

Design Gradation Selection

Selecting a design gradation for a PFC mixture is done by performing three trial gradations according to ASTM D7064. After suitable aggregate sources have been chosen, the optimization of the mix can begin by creating three designs that fall on the coarse limit, fine limit and in the middle of the recommended gradation range. The agencies surveyed by NCAT which currently use PFC mixes provided their gradation specification ranges for PFC mix designs. Table 2 summarizes the response to the survey question regarding the gradation specification ranges.

Table 2 – Gradation Specification Ranges for PFC Designs Currently Used by Agencies

State	¾ in. 19mm	½ in. 12.5mm	3/8 in. 9.5mm	No. 4 4.75mm	No. 8 or 10 2.36mm	No. 16 1.18mm	No. 30 or 40 0.6mm	No. 200 0.075mm
AL	100	85-100	55-65	10-25	5-10			2-4
AZ 1			100	30-45	4-8			0-2
AZ 2			100	31-46	5-9			0-3
CA 1			78-89	28-37	7-18			
CA 2		99-100		29-36	7-18			
FL	100	85-100	55-75	15-25	5-10			2-4
GA 1		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
LA 1		100	90-100	25-50	5-15			2-5
LA 2	100	85-100	55-75	10-25	5-10			2-4
MS		100	80-100	15-30	10-20			2-5
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	5-15			2-4
NE	100	95-100	40-80	15-35	5-12			0-3
NJ 1		100	89-100	30-50	5-15			2-5
NJ 2	100	85-100	35-60	10-25	5-10			2-5
NJ 3		100	85-100	20-40	5-10			2-4
NM		100	90-100	25-55	0-12		0-8	0-4
NV 1		100	90-100	35-55		5-18		0-4
NV 2		100	95-100	40-65		12-22		0-5
OR 1		99-100	90-100	22-40	5-15			1-5
OR 2	99-100	90-98		18-32	3-15			1-5
SC	100	85-100	55-75	15-25	5-10			0-4
TN	100	85-100	55-75	10-25	5-10			2-4
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
UT		100	90-100	35-45	14-20			2-4

The asphalt content selected for the trial designs is based on the combined aggregate bulk specific gravity (Table 3) (Cooley, et al., 2009) (AASHTO, 2014). Three specimens are prepared for each of the three trial designs and the voids in the coarse aggregate (VCA) and the air void content are used to determine which trial will be selected for design. The VCA is used to determine if the mix has stone-on-stone contact. The VCA of the mix (VCA_{MIX}) must be less than the VCA of the dry-rodded coarse aggregate (VCA_{DRC}) in order for the aggregate skeleton to have stone-on-stone contact. The survey showed that only one of the state agencies (Louisiana) uses VCA as part of the design procedure while the rest rely on historical gradations and performance. The optimum design gradation should be the one that meets the VCA requirement and has the largest air void content, as long as it meets the minimum requirement. According to ASTM D7064, the minimum accepted air void level is 18.0 percent. Table 4 lists the results of the survey in regards to state agency's air void requirements for PFC mix designs.

Table 3 – Minimum Asphalt Content for PFC Mix Designs (Cooley, et al., 2009)

Combined Aggregate Bulk Specific Gravity	Minimum Asphalt Content Based on Mass (%)
2.40	6.8
2.45	6.7
2.50	6.6
2.55	6.5
2.60	6.3
2.65	6.2
2.70	6.1
2.75	6.0
2.80	5.9
2.85	5.8
2.90	5.7
2.95	5.6
3.00	5.5

Table 4 – Air Void Requirements of PFC Mix Designs (Survey)

State	Aid Void Requirement
Alabama	Min. 12%
Georgia	18 – 20% for PFC 20 – 22% for PEM
Louisiana	18 – 26%
Maine	18 – 22%
Maryland	Min. 18%
Mississippi	Min. 15%
Nebraska	17 – 19%
New Jersey	Min. 15, 18 or 20% depending on mix
North Carolina	Min. 18%
Oklahoma	Min. 18%
Tennessee	Min. 20%
Texas	18 – 22%
Virginia	Min. 16%

Determining the Optimum Asphalt Binder Content

Once trial gradations have been completed and the design gradation has been selected, the optimum asphalt content needs to be determined. Additional specimens should be fabricated at three different asphalt contents. The asphalt contents should be in 0.5 percent increments above and below the trial asphalt content. As established in the previous section, the air void content must be greater than 18.0 percent for most agencies. According to ASTM D7064, the mix must also have a draindown percentage less than 0.3 percent, a tensile strength ratio (TSR) of 0.80 or greater and a $VCA_{MIX} \leq VCA_{DRC}$. Cooley et al. (2009) recommends selecting the optimum asphalt content based on the requirements in Table 5. Note that the TSR is decreased to 0.70 or greater and the VCA_{Mix} must be less than VCA_{DRC}. The VCA requirement in Table 5 indicates that the VCA_{MIX} must be less than VCA_{DRC}, not less than or equal too. This appears to be a mistake and should be less than or equal to. The text surrounding the table in NCHRP Report 640 states "When VCA_{MIX} is less than or equal to VCA_{DRC} stone-on-stone contact exists." Since the purpose of the VCA requirement is to ensure stone-on-stone contact this suggests that the table (Table 5) requirement for VCA may be in error. Previously, the optimum asphalt content for PFC mixes was selected based primarily on the pie-plate test. The pie plate method is a visual and subjective evaluation of the draindown of asphalt binder and several other agencies have found alternative methods (McDaniel, 2015) for determining the optimum asphalt content.

Table 5 – Requirements for Selecting Optimum Asphalt Content for PFC Mixtures (Cooley, et al., 2009)

Mix Property	Requirement
Asphalt Binder (%)	Table 3
Air Voids (%)	18 – 22
Cantabro Loss (%)	<15.0
VCA_{MIX} (%)	$<VCA_{DRC}$
Tensile Strength Ratio	≥0.70
Draindown at Production Temperature (%)	≤0.30

STATE OF THE PRACTICE

A PFC pavement provides many benefits over a dense-graded mix, but the obvious drawback of durability issues causes a shorter pavement performance life when compared to dense-graded mixtures. Timm et al. completed a study at the NCAT Test Track that concluded that the back-calculated structural number of OGFC was 0.15 (Timm, 2014). This shows that while permeable surface courses are useful for water drainage, splash prevention and noise mitigation, they provide only about a third of the structural capacity of a dense-graded mix. With little improvement to structural capacity, high asphalt contents and a shorter service life, PFC mixes must become more resilient if they are going to be accepted by more state agencies. The subsequent sections will delve into the current state of the practice for PFC mixes.

Selection of Materials

AGGREGATE CHARACTERISTICS

As part of the recent survey conducted by NCAT, agencies were asked to provide the type of aggregate they specified along with what they deemed as the most important aggregate properties when designing PFC mixes. The responses to the questionnaire can be seen graphically in Figure 7 and Figure 8. Granite and limestone are the predominant aggregate types while abrasion, polishing and flat and elongated particles are the principal aggregate properties being evaluated for PFC designs.

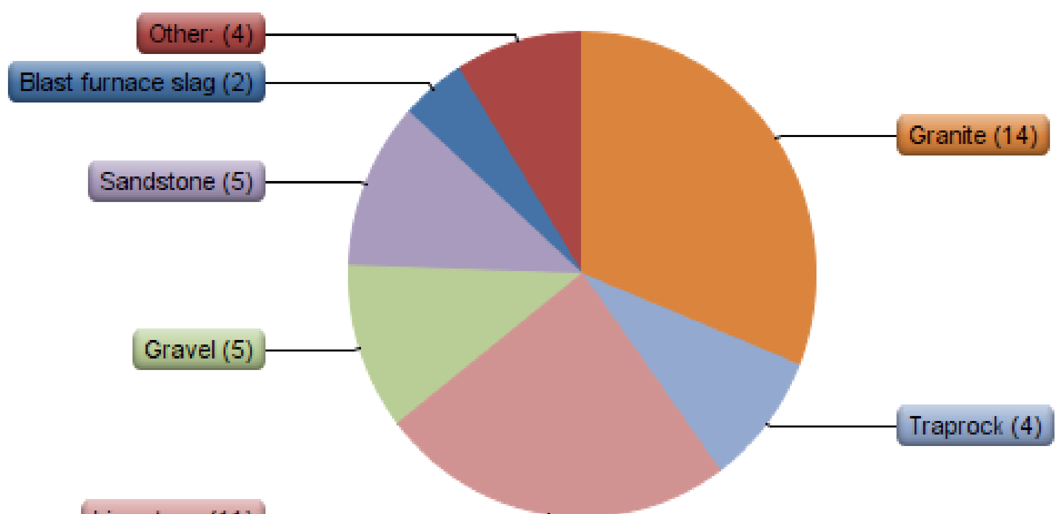


Figure 7 – Aggregate Type Specified by Agencies

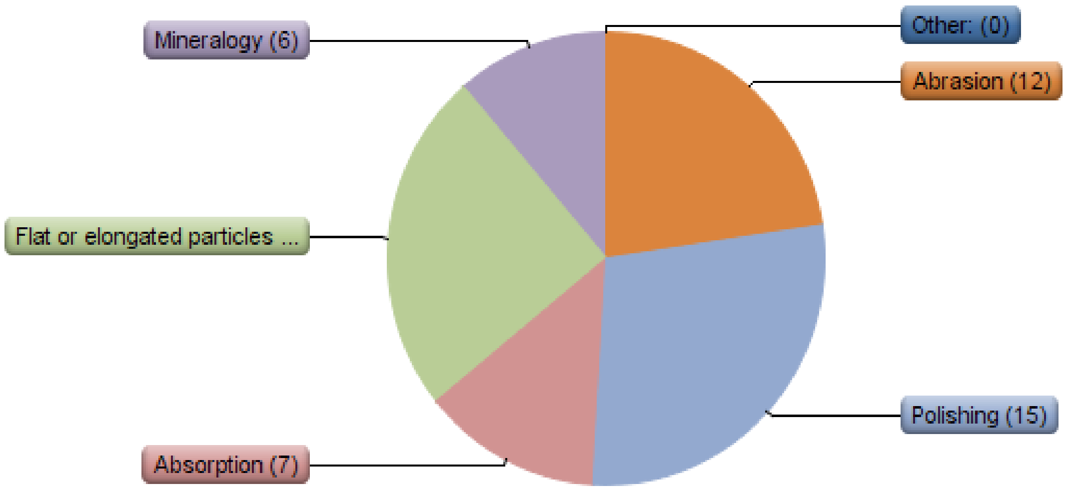


Figure 8 – Required Aggregate Properties for Use in PFC Designs

Aggregate characteristics that should be considered when designing PFC mixes are durability, polish resistance, angularity, cleanliness, abrasion resistance and absorption. The most important of these characteristics are the polish resistance and durability (Cooley, et al., 2009). The most common way to define durability and polish resistance is with the sulfate soundness test and the polish stone value (PSV). The PSV calculates the aggregate's ability to resist polishing. The minimum recommended PSV for porous mixes is 55 (German Asphalt Pavement Association, 2006). Europe considers this to be the most important criteria when designing porous mixes (Lefebvre, 1993). Spain recommended a minimum value of 45 while Great Britain recommends a minimum value of 60 (Bolzan, 2001). Attaining a value of greater than 50 may be difficult depending on locally available aggregates. New Zealand typically attains a value of 55-61 for its porous mix designs (Fletcher E., 2011) while South Africa recommends a value of greater than 50 (Masondo, 2001). The sulfate soundness test measures the durability of the aggregate in terms of weathering with regard to freeze-thaw cycles. The amount of acceptable loss for sulfate soundness is dependent on the agency. The state of Georgia allows up to 15 percent maximum loss (Watson D. E., 1998) while Oregon only allows a 12 percent loss (Huber, 2000).

Aggregate angularity and abrasion resistance are the next most important properties, with both being specified in ASTM D7064. Aggregate angularity, more commonly known as fractured face count, gives requirements for the number of fractured faces a stone particle must contain. If crushed gravel is used in the design, ASTM requires that 95 percent of the particles have two or more fractured faces. The criterion for two or more fractured faces ranges from 75 percent in Spain (Ruiz, 1990) to 100 percent in Florida (Huber, 2000). The most common way to test for aggregate abrasion resistance is through the Los Angeles (L.A.) Abrasion Tester. The L.A. Abrasion test determines the aggregate's resistance to crushing and degradation. The amount of allowed loss varies between agencies, but ranges from 12 percent to 50 percent loss allowed for the coarse aggregate (Alvarez, et al., 2006) (Watson D. E., 1998). According to ASTM and Kandhal et al. (2002), the current standard in the United States is a maximum allowable loss of 30 percent.

ASPHALT BINDER

PFC mixes have been used successfully with both modified and unmodified binders. The use of modified binders became more prevalent after research showed that modifying the binder could increase the life of the pavement and prevent draindown of the mix. The binders are graded according to the Superpave Performance Grading (PG) system in the United States, but some European countries still implement a penetration grading system. The use of modifiers, such as SBS, SBR and tire rubber, has significantly improved the mix performance of many different asphalt binders. The most common modifiers for PFC mixes are polymers and rubber. These modifiers improve the performance of the mix by increasing the modulus and elasticity of the asphalt. The most commonly used polymer is SBS (Kuennen, 2012). This elastic polymer soaks up the aromatics in the asphalt that creates more elastic recovery for the asphalt. This is known as a block polymer and is the result of polybutadiene and polystyrene forming chains, whose combination increases strength and flexibility of the asphalt. The use of crumb rubber modifiers (CRM), made from ground tire rubber (GTR), in asphalt has been around since the 1960's (Carlson, 1999). The most commonly used method for producing CRM is the crackermill process, which produces ground/torn particles ranging from 4.75 mm (No. 4 sieve) to 0.42 mm (No. 40 sieve) in size. It is typically added to the asphalt binder at a rate of 10 – 20 percent by weight of binder, using a "wet" process. The "wet" process blends the CRM into the binder at a temperature range of 300-400°F for 45 minutes to an hour. The reported effect of the CRM on the performance of mixes varies, but it is suggested that the CRM can offer more resistance to asphalt oxidation while mitigating rutting, and resisting thermal and reflective cracking (Willis J. R., 2013). Some results have shown a decrease in the permeability of PFC mixtures using CRM (Suresha, 2009). These reports did not indicate how the optimum binder calculation was performed. The purpose of using a CRM is for binder modification, not as binder replacement; therefore the 10 – 20 percent rubber is in addition to the optimum asphalt content. This may be one of the contributing factors for observing decreased permeability for CRM mixtures. Both an SBS and CRM modifier provide a stiffer asphalt film that leads to increased cohesion of the aggregate stone skeleton. This provides a more durable PFC mix.

In 2000, Huber reported that Britain used both modified and unmodified binders but required a 100-pen value. Italy and Spain used only modified binders requiring an 80/100 pen value (Huber, 2000). The Netherlands and Switzerland do not require modified binders, but Switzerland allows modified binders as an alternative to conventional binder (Alvarez A. E., 2007). Europe primarily uses polymer modification, SBS specifically, while South Africa uses both polymer and rubber in their PFR mixes (Huber, 2000).

While modified binders are beneficial to PFC designs, they are not always necessary. Consideration of the anticipated traffic volume and weather should be taken into account prior to designing the mix. The use of stabilizing agents, such as fibers, could replace the need for binder modifiers on low to medium traffic roads (Kandhal P. , 2002).

STABILIZING AGENTS

Stabilizing agents come in several forms. The most common of these are fibers. Fibers provide stability to the mix while also preventing draindown of the binder. There are many different types of fibers including cellulose, mineral, asbestos, polypropylene, acrylic, and glass fiber (Bennert & Cooley, 2014). Draindown is an issue in porous mixes because of the open-aggregate grading of the blend. The aggregate blend has little material passing the 4.75 mm (No. 4) sieve and a relatively low amount of P-200 material compared to dense-graded mixes. This results in a much lower aggregate surface area for PFC mixes, allowing for a thick coating of asphalt binder on the aggregate particles. A typical film thickness of a dense-graded mix is approximately 8 microns while a porous mix normally is around 30 microns (Watson D. E., 2004). New Zealand has had success with their porous mixes that typically have a film thickness of 10 microns (Fletcher E., 2011). This extra thick film can potentially drain down off the aggregate during production/construction and can cause a myriad of issues such as the loss of binder content in the mixture, flushing of asphalt in concentrated drain-down spots on the roadway, and excessive adherence of the mix to the truck bed. In 1998 a survey conducted by Kandhal and Mallick showed that only 19 percent of state agencies were using fiber in their PFC designs (Kandhal P. S., 1998). This number was significantly less than the 85 percent of agencies that were reported using fiber in 2009 according to Cooley et al. (2009). This percentage has not changed significantly since 2009, according to the most current survey indicating that 82 percent of the responding agencies use some type of stabilizing agent. Figure 9 depicts the results of the most current survey. Several agencies allow cellulose and mineral fiber.

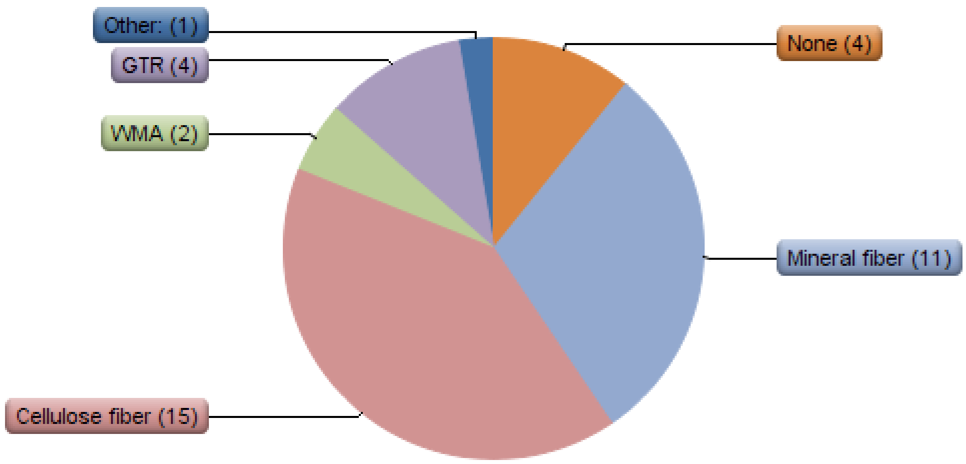


Figure 9 – Survey Response to Stabilizing Additives Used in PFC Mixes

The most common stabilizers used in the United States, Europe and Australia are cellulose and mineral fiber (Cooley, et al., 2009). They are typically added to the mix at a rate of 0.3 percent by total weight of the mixture, but can range from 0.2 to 0.5 percent. These values are recommended in the ASTM specification along with NCHRP Report 640 (Cooley, et al., 2009) (ASTM , 2013). Cellulose fibers are flora-based and can come in either pellet of loose form. The cellulose has high absorption so it can be an excellent method for maintaining high binder contents. Mineral fibers come in two forms: manufactured and naturally occurring. Asbestos, the only naturally occurring fiber used in asphalt, was used as mineral filler into the 1960's until its impact on people's health was discovered. The most common manufactured mineral fibers are mineral wool or rock wool. The fibers are formed by melting the minerals down and spinning the minerals until they form fibers, which is similar to the process of making cotton candy (McDaniel, 2015). Mineral fiber is not as absorbent as cellulose fiber and sometimes creates harsh mixes that are hard to rake and compact. Figure 10 illustrates how effective fibers are at preventing draindown.

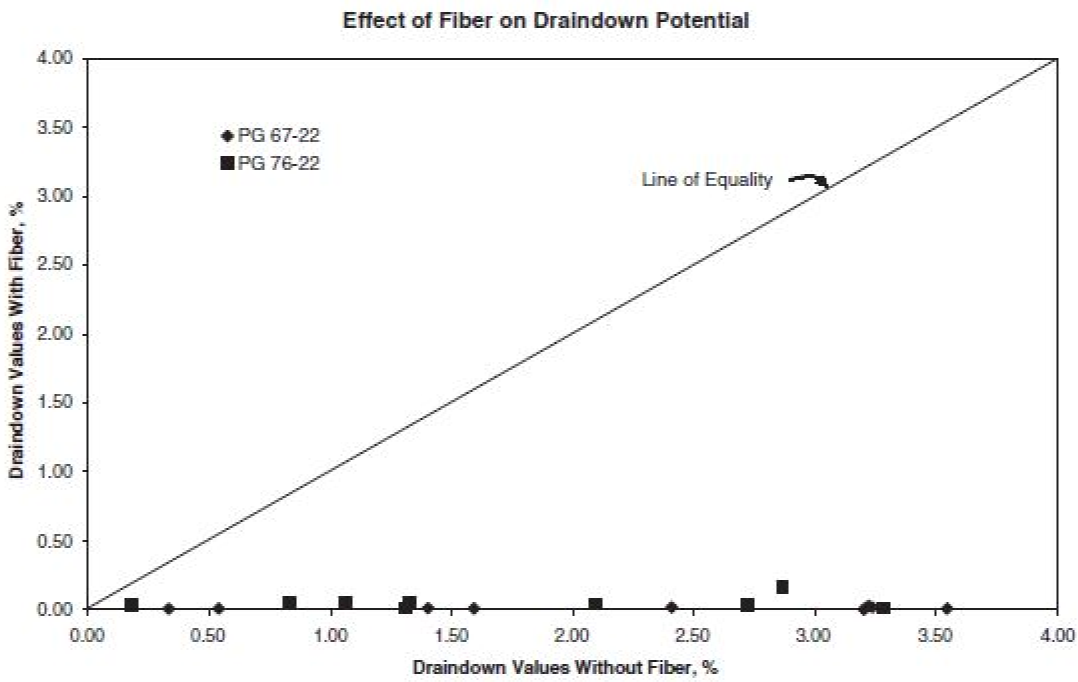


Figure 10 – Effect of Fibers on Draindown Potential of PFCs (Watson D. E., 2003)

FILLER/ANTI-STRIPPING AGENTS

Hydrated lime is used as a filler material by many agencies. The Netherlands uses a limestone filler but requires that at least 25% of it must be hydrated lime. Australia uses not only hydrated lime as a filler but also portland cement and limestone dust. Hydrated lime also doubles as an anti-strip agent to prevent moisture damage to the mixture. A recent survey showed that one of the contributing factors for PFC mixtures with greater than 12 years of service life was the use of 1.0 percent hydrated lime. FDOT has many PFC pavements that achieve in excess of 12 years of pavement life and attribute part of that to the use of hydrated lime. FDOT, having used both in previous works, indicates that hydrated lime performs better than liquid anti-strip additives in regards to overall pavement life. Cooley et al. (2009) reports that filler contents vary depending on the maximum aggregate size of the design. Italy provides the lower limit of 0 percent passing the 0.075 mm (No. 200) sieve while South Africa provides the upper limit by allowing as much as 8 percent (Cooley, et al., 2009). The recent survey shows that the responding agencies allow anywhere from 0 to 5 percent filler in the PFC mix designs.

DESIGN GRADATION SELECTION

With suitable materials selected, trial gradations with initial asphalt contents should be established. There is no nationally accepted gradation band for PFC mixes; however ASTM D7064 gives an “example” gradation. The *example* gradation in ASTM D7064 is the same *recommended* gradation (Table 6) in the NAPA Information Series 115 (Kandhal P. , 2002). Table 6 also shows the recommended gradation according to the FHWA Technical Advisory (FHWA, 1990).

Table 6 – Recommended Gradation for OGFC (Kandhal P. , 2002) (FHWA, 1990)

	Percent Passing	
Sieve (mm)	**NAPA**	**FHWA**
19.0	100	-
12.5	85 – 100	100
9.5	35 – 60	95 – 100
4.75	10 – 25	30 – 50
2.36	5 – 10	5 – 15
0.075	2 – 4	2 – 5

There are many different gradations used by agencies across the world. Some of these agencies define their mixtures according to nominal maximum aggregate size (NMAS) while others use a maximum aggregate size to define the mix type. The results of the Cooley et al. (2009) survey were converted to maximum aggregate size prior to summarizing them. According to this survey, the only agencies to have a 25.0 mm (1 inch) design were Oregon and Great Britain, while the only 12.5 mm (1/2 inch) designs were from Louisiana and Great Britain. All other responding agencies provided 19.0 mm (3/4 inch) designs. An illustration of these gradation bands for both the U.S. and International agencies can be seen in Figure 11 and Figure 12.

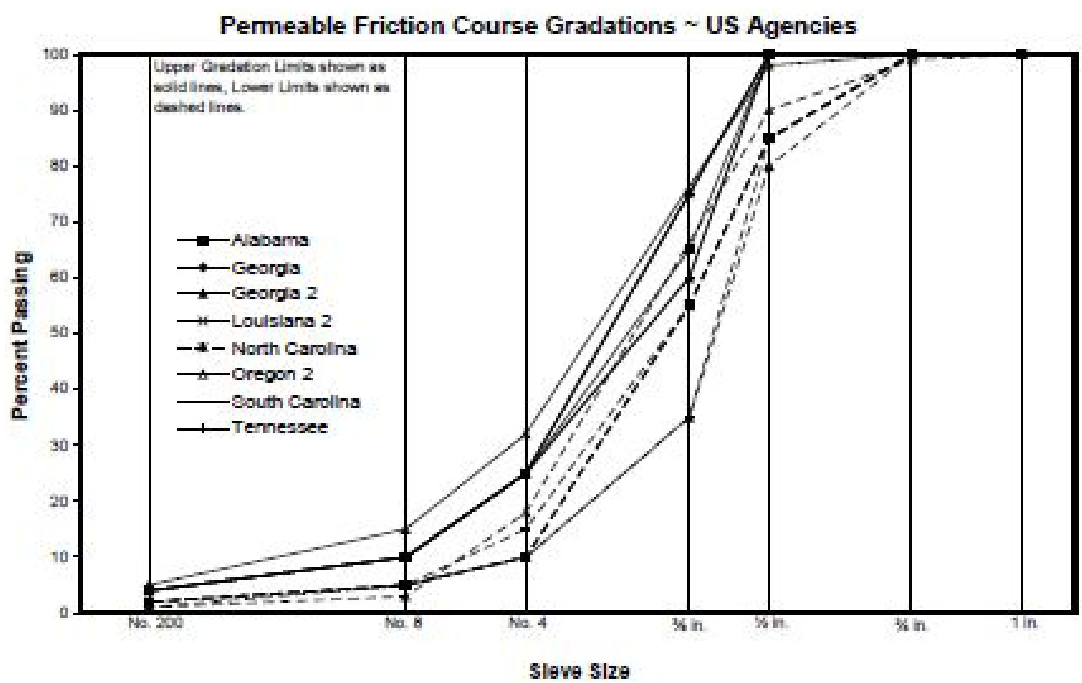


Figure 11 – 19.0 mm PFC Gradation Requirements from U.S. Agencies (Cooley, et al., 2009)

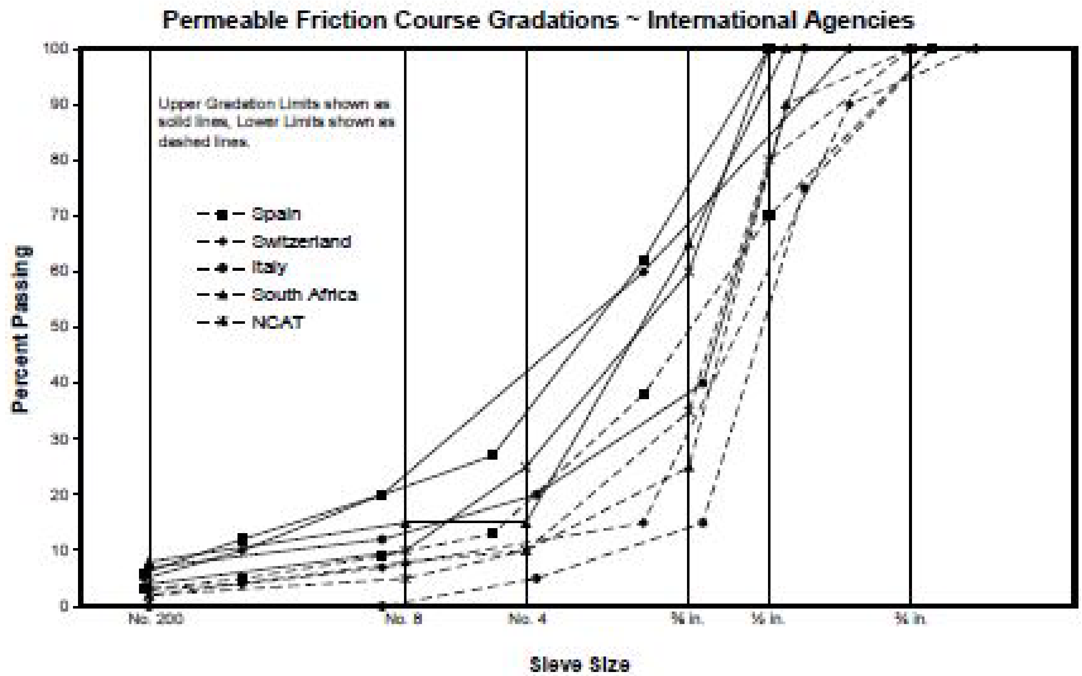


Figure 12 – 19.0 mm PFC Gradation Requirements from International Agencies (Cooley, et al., 2009)

The U.S. agencies primarily use a 19.0 mm maximum aggregate size design and the majority of them are gapped around the 4.75 mm (#4) sieve. The same can be seen for the international agencies in Figure 12. The gradations for Alabama, Georgia, Louisiana and South Carolina are almost identical. These similarities in the specifications can be attributed to the Watson et al. research conducted on PEMs in Georgia (Watson D. E., 1998). The gradation band recommended by NCAT's research in 2003 is also included in Figure 12 to show a comparison between the typical gradations used in the U.S. - and other countries. This comparison is well illustrated on the 9.5 mm sieve. Spain has an upper limit of approximately 75 percent passing the 9.5 mm sieve while Italy has a lower limit as low as 10 percent passing. The NCAT-recommended gradation band ranges from 35 to 60 percent passing the 9.5 mm sieve. The amount of filler allowed in the designs also varies significantly. According to this survey, all of the 19.0 mm U.S. designs range from 2 to 4 percent filler (same amount recommended by

NCAT). The international agencies range from a low band of 0 percent for Italy to a high band of 8 percent for South Africa.

Optimum Binder Content Selection

Trial asphalt contents, normally in 0.5 percent increments, are fabricated and then the properties of the mix are considered; however there is no specific procedure that requires particular properties of the mixture to be considered. While the ASTM standard merely suggests a minimum of 18.0% air voids and a draindown of less than 0.3 percent, it makes all other properties optional. The National Asphalt Pavement Association (NAPA) publication (Kandhal P. , 2002) has criteria for certain mixture properties in order to select the optimum binder content. A comparison of the properties can be found in Table 7. The minimum permeability requirement is based on research conduct by NCAT in 2000 by Mallick et al. (Mallick, 2000).

Table 7 – Optimum Asphalt Content Properties for PFC Mixes

Mix Property	**NCHRP 640**	**ASTM D7064**	**NAPA Series 115**
Air Voids (%)	18 – 22	≥18	≥18
Unaged Cantabro Loss (%)	≤15.0	**≤20.0**	≤20.0
VCA_{MIX} (%)	$<VCA_{DRC}$	$\leq VCA_{DRC}$	$\leq VCA_{DRC}$
Tensile Strength Ratio	≥0.70	≥0.80	≥0.80
Draindown at Production Temperature (%)	≤0.30	≤0.30	≤0.30
Permeability (m/day)	**100**	**100**	**100**

NOTE: Bold properties are optional

According to ASTM D7064, design specimens are to be compacted using a Superpave Gyratory Compactor (SGC) to a design level of 50 gyrations. This criterion was developed on recommendation from a previous NCAT study (Kandhal P. S., 1999). The study indicated that 50 gyrations in the SGC provided approximately the same amount of compaction as a 50 blow Marshall design (European PFC design).

A new design model (Figure 13) proposed by Bennert et al. (2014) considers a combination of draindown and Cantabro loss when selecting the optimum asphalt content. This allows the designer to select the optimum based on an acceptable range of binder contents. It does not take into account air void content, so this should be considered when making the final selection. South Africa has a similar process for determining the optimum asphalt content that includes air void content. The optimum is selected by averaging the larger asphalt content of the "minimum" values (durability and abrasion resistance) with the smaller asphalt content of the "maximum" values (air void content and draindown) (Masondo, 2001).

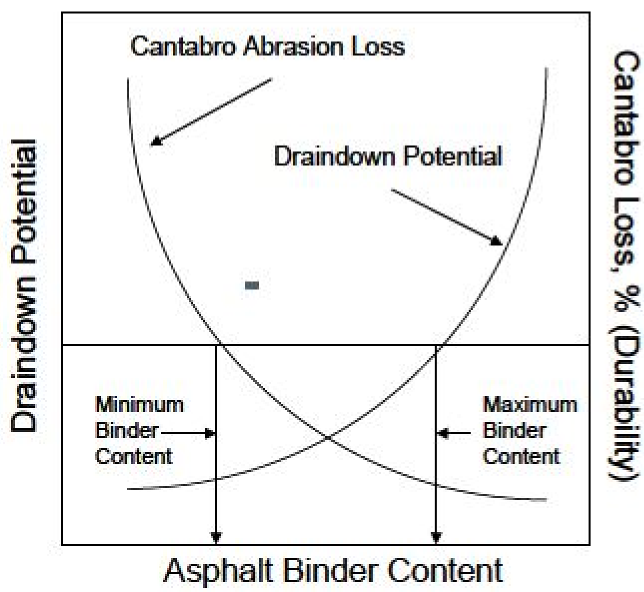


Figure 13 – Philosophy of Designing PFC Mixtures (Bennert & Cooley, 2014)

The durability of the mix is determined according to the Cantabro Abrasion Test. This method of testing was developed in Spain in the 1980's (Lefebvre, 1993). It is the most common method for determining the durability of PFC mixes. The relationship shows that as the asphalt content increases the durability of the mixture improves, but the risk of draindown is increased.

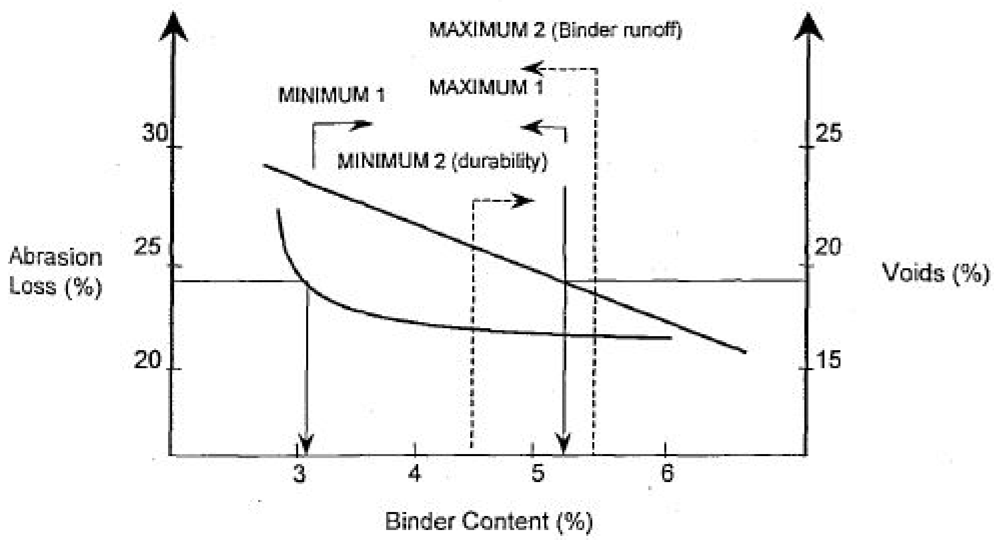


Figure 14 – South Africa Typical Graph for Determining Optimum Binder Content for Porous Mixtures (Masondo, 2001)

Draindown in the U.S. is typically performed according to ASTM D6390 or AASHTO T305; however there are several different methods for assessing the draindown potential for PFC mixes. Some agencies use the Pyrex bowl method better known as the "pie-plate test" for determining optimum asphalt content. This method is based on draindown, is subjective, and is only a visual test. Approximately 1,000 grams of uncompacted PFC mix is placed in an 8 – 9 inch Pyrex glass plate. The plate is then placed in an oven at 121°C (250°F) for 1 hour and a visual examination of the residual asphalt binder is conducted after the mix has cooled and the pie plate is inverted. This is done for all asphalt contents tested in the trials. Illustrations of the Pyrex bowl method can be found in Figure 15.

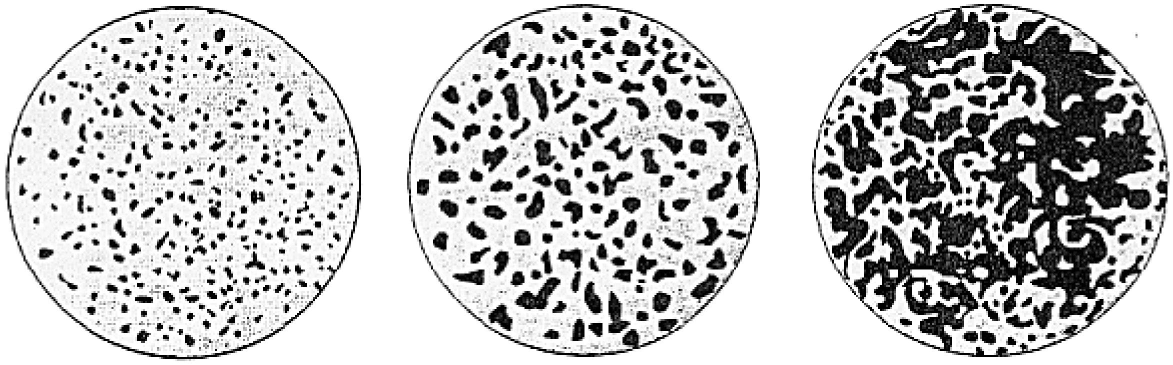


Figure 15 – Pyrex Bowl Method for Determining Draindown of PFC Mixtures

A recent survey showed that 19% of the state agencies still using PFC mixes are utilizing the pie-plate test for determining the optimum asphalt content. The Schellenberg drainage test is also another method that has been used in the past. A 1,000 gram sample of uncompacted PFC mix is placed in a glass beaker, which is then placed in an oven at 170°C (338°F) for 1 hour. The loose mix is then removed from the beaker and the asphalt residue is quantified. This method has a more measureable result, which makes it more valuable when determining draindown.

The most common method for determining draindown was developed by NCAT. A wire basket with uncompacted PFC mix is placed in an oven at 15°C (27°F) greater than the anticipated mixing temperature. The wire basket is placed on a container of known mass and after one hour the container is removed from the oven. The amount of material that passed through the wire basket and into the container is then quantified as a percentage of the total mass. The wire baskets were originally a 4.75 mm (No. 4) mesh, but subsequent research by Watson et al. (2003) showed that some intermediate-sized stone could pass through the mesh, allowing more than just asphalt binder to drain down onto the container. Based on that research, it was recommended that a 2.36 mm (No. 8) mesh be used for draindown testing instead. In addition to the mesh size change, amendments were made to the procedure in which any binder remaining on the basket after the one hour conditioning in oven should be considered part of the draindown percentage. Georgia DOT previously used both the pie-plate test and the Schellenberg method for determining draindown but have since moved to the draindown basket method. South Africa allows the designer to choose either the Schellenberg method or the draindown basket (Masondo, 2001). If significant draindown is occurring, fiber or the addition of a binder modifier will help to mitigate the draindown (Cooley, et al., 2009).

Construction and Maintenance of PFC Mixes

The main issues with PFC mixes that can be related to construction are raveling and delamination. The following factors are the main influences that lead to issues with PFC pavements during production and construction:

- Homogenous mix gradation and temperature
- Asphalt content
- Tack bond strength, rate and quality of application
- Layer thickness
- Mixing temperature during placement

According to Bennert et al. (2014), production and construction issues may be more responsible for raveling than the mix design properties. Inconsistent temperatures in the mix during construction can lead to both delamination and raveling. Delamination occurs when the bond between the underlying surface and the PFC is inadequate and causes a slip plane. A tack application is placed on the surface of the underlying layer so that the PFC can adhere. If the underlying layer is too cold or covered in dust the tack material may not adhere, causing the pavement to delaminate. The amount and type of tack material is also important. Since PFC mixtures are coarse-graded, there is less contact area between aggregate particles in the PFC and the underlying layer than for a dense-graded mix. It would therefore seem logical that the tack rate should be increased so that the contact area has the same tack bond strength as a dense-graded mix. Several studies have been conducted on the interface bond strength. An NCAT study in 2005 recommends a bond strength of 100 psi, when tested at 77°F, for newly constructed overlays (West, 2005). This study was primarily for dense-graded overlays but did include porous overlay data in the bond strength recommendation. By improving the bond of the two layers, the risk of delamination is diminished.

The rate at which the tack is applied is also a critical component. Figure 16 shows the tack rates provided by the agencies that responded to the survey. Most tack material is an emulsion. Emulsions consist of asphalt binder particles that are suspended in water. This allows the tack to be spread more evenly and allows it to be applied at lower temperatures for safety reasons. The percent of asphalt binder in the emulsion is known as the residual. Most application rates are based on the residual. There is a wide range of tack rates provided in the responses (0.02 – 0.15 gal/sy) depending on the type of tack material used. The "Other" category was used when agencies responded with tack rate ranges that were different than the options given in the survey. One example of this was South Carolina that provided a range of 0.05-0.15 gal/sy.

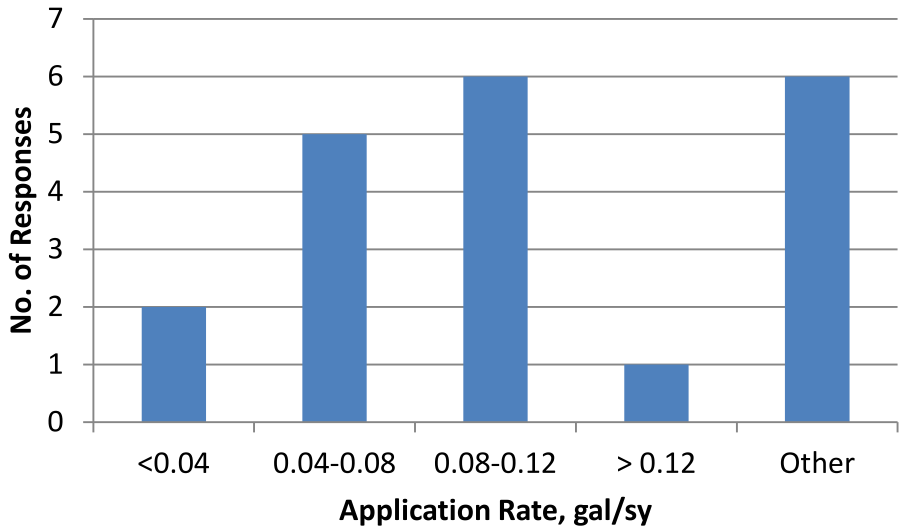


Figure 16 – NCAT Survey Response for Tack Application Rate

While raveling can be linked to the interface bond, it is also a durability issue that begins at the top of the pavement. Mix temperature is one of the biggest concerns when constructing PFC mixtures. Consistent mix temperatures and short haul times are critical for adequate placement. Due to the open structure, a PFC will cool faster than a standard dense-graded mix. This can be mitigated somewhat with the use of insulated truck beds and tarpaulins during transport; however initial production temperature, haul time and the ambient/pavement surface temperature are more critical. Great Britain specifies that from production until the mix is placed on the ground, no more than 3 hours can elapse (Alvarez, et al., 2006). The FHWA Technical Advisory recommended a limited haul distance of 40 miles and a travel time of less than 1 hour (FHWA, 1990). In order to mitigate the loss of heat in PFC mixtures during construction the following items have be considered:

1. Provide an adequate number of haul trucks so that there is no pause in construction. When the paver is required to wait on haul trucks due to a lack of mix, it stops which creates a cold transverse joint (Figure 17a).
2. Preheat the screed before the initial start-up at a transverse joint. A cold screed will pull some of the mix particles at the start-up transverse joint and will cause a lack of mix homogeneity. In dense-graded mix, the material can be raked to correct this issue; but raking a PFC, especially with modified binder is somewhat difficult.
3. Use a material transfer vehicle (MTV). A MTV is used to remix the asphalt mixture after it has been transported to the job site. This remixing should result in a homogenous mix temperature that will help eliminate cold spots in the asphalt mat.
4. Ensure adequate screed crown and temperature. Most pavers use multiple propane burners to heat the screed. These burners can go out during production and cause a cold spot in one section of the screed. It is important to provide proper screed crown in order to obtain a smooth finish. Due to the relatively thin lift thickness and a high proportion of coarse aggregate, failure to properly adjust the screed crown will cause the mix to pull and results in streaks in the mat (Figure 17b).

Cold Transverse Joint (a) Center of Paver Streak (b)

Figure 17 – Raveling of PFC Mixture Due to Construction Practices

PERFORMANCE TESTING

Moisture Susceptibility

The most recognized and widely used performance test for PFC mixes is a moisture susceptibility test. There are three different methods for determining moisture susceptibility of PFC mixes. The first is the modified Lottman method (AASHTO T283). The test uses the indirect tensile strength of the mix to calculate a tensile strength ratio. Due to the open void structure of PFC mixtures, it is not possible to saturate the specimens to a certain degree of saturation. Therefore, the specimens are placed under water, and a partial vacuum of 26 inches Hg (660.4 mm Hg) below atmospheric pressure is applied to the sample for 10 minutes. The specimen is then placed in a container and is kept submerged, in order to maintain saturation, during the freeze cycle. The specimens are frozen for a minimum of 16 hours and are then thawed in a hot water bath at 60 °C (140°F) for 24 hours. The specimens are then normalized in a water bath at the room temperature of 25°C (77°F) for 2 hours prior to testing. Some research has recommended using five freeze-thaw cycles prior to testing. In 2004, Watson et al. showed that there was no significant difference between 1, 3 and 5 freeze-thaw cycles (Cooley, et al., 2009). The second moisture susceptibility test is the boil test (ASTM D3625). The boil test requires a 250 gram uncompacted PFC sample be placed in a beaker of boiling water for 10 minutes. The sample is not to be conditioned and must be below 212°F and above 180°F prior to adding it to the boiling water. After the 10-minute boiling, the sample is removed from the beaker, and a visual inspection of the sample is performed to determine if any visual stripping of the aggregate has occurred. Texas and Georgia have used this method in the past, but its current use is primarily as a quality control tool.

The third and final option for evaluating moisture susceptibility is the wheel-tracking test. Cooley et al. (2000) did some initial testing with PFC beams by submerging the specimens in 60°C water bath overnight and testing the specimens for rut depth in the Asphalt Pavement Analyzer (APA) for 8,000 cycles (GDT-115). This process is similar to the Hamburg Wheel Tracking Test (HWTT), which also tests for moisture susceptibility of the mixture. The current standard for the HWTT (AASHTO T324) can be performed on beams or Superpave gyratory specimens, but it is tested at 50°C under a constant load of 158 lbs. Alvarez et al. suggested not including HWTT as part of mix design criteria due to its large range of coefficient of variance (0.02-0.57) (Alvarez A. E., 2007). Louisiana actually requires HWTT be performed as part of the design requirements (LADOTD, 2016). The requirement for a mixture with a PG 76-22m binder, states that the specimen must reach 5,000 passes prior to reaching 12.0 mm of rut depth.

Cantabro Abrasion Testing

According to the Cooley et al. (2009) survey, the most common way to determine the durability of a PFC is with the use of the Cantabro Abrasion Loss test. The test is typically performed at 25°C (77°F). Herrington et al. proposed that testing the specimens below 0°C may provide a better differentiation between mixes with different binder types. While maintaining a temperature lower than 25°C during testing would be difficult and expensive, he stated it would possibly be a better alternative when trying to compare different binder modifiers and types (Herrington, 2005). The British specification for Cantabro test requires a test temperature of 10°C (British Standards Institute, 2004). The Cantabro specimens can be classified as aged, unaged or moisture conditioned. Unaged specimens typically have a 25 percent maximum loss criterion but some other agencies such as Texas and Belgium have a tighter tolerance of 20 percent maximum loss. The aged specimens are placed in a forced draft oven for 7 days at 140°F prior to testing. The criterion for the aged specimens is more lenient with a maximum of 30 percent loss allowed. Some of the international agencies (Great Britain, South Africa, and Australia) require a moisture-conditioning period prior to testing. In the 2009 survey, Great Britain was the only agency that provided their aging protocol, which stated that the specimens were submerged for 24 hours at 140°F in a water bath prior to testing (Cooley, et al., 2009).

FIELD PERFORMANCE

Out of the 21 agencies that responded to the recent survey and are currently using PFC designs, the average service life for a PFC pavement is between 8 and 10 years. The distribution of the responses can be seen in Figure 18. Agencies that were achieving greater than 12 years of service life were asked what, if any, special consideration was given when designing and maintaining the pavement. Their responses are as follows:

- Eliminated the use of gravel and slag due to past performance issues
- All designs required stabilizers
- Replaced liquid anti-strip with hydrated lime
- Increased the NMAS from 9.5mm to 12.5mm and increased the lift thickness by ¾ of an inch

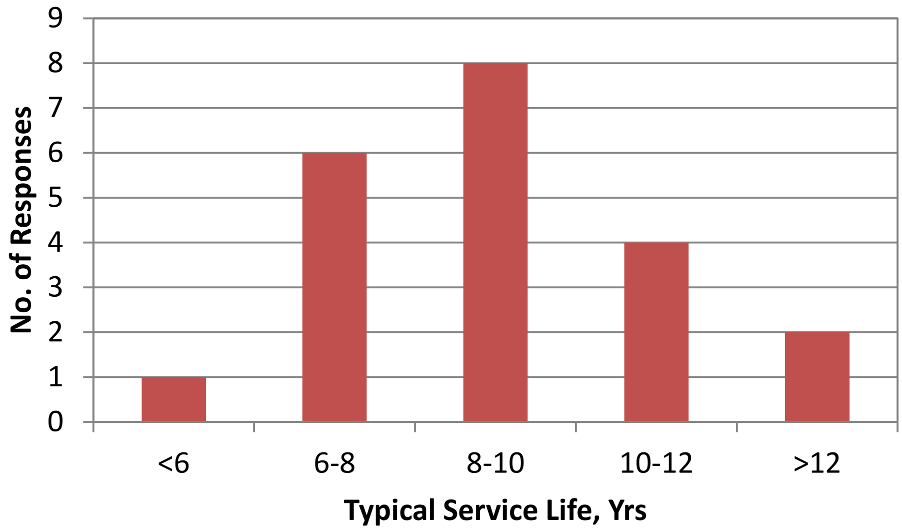


Figure 18 – Typical Service Life of PFC Pavements

The agencies were asked what primary distress was causing issues with their PFC pavements and the majority (76%) of responses claimed that raveling was the biggest issue. This is not only a local issue, but is also one of the main issues plaguing Europe's porous pavements. Van der Zwan (2011) stated that over 90 percent of their maintenance practices for porous pavements are in regard to raveling (Van der Zwan, 2011). Figure 19 shows the distribution of the different distress types from the most recent survey.

The agencies that are not currently using PFC designs were asked what improvements were needed in order for them to consider using a PFC. The majority of the responses requested improved durability (38%) along with safety and performance in colder climates (32%). Based on some of these responses, the lack of PFC use in the colder climates (as shown in Figure 1) seems to be primarily due to water freezing in the pavement and causing safety concerns.

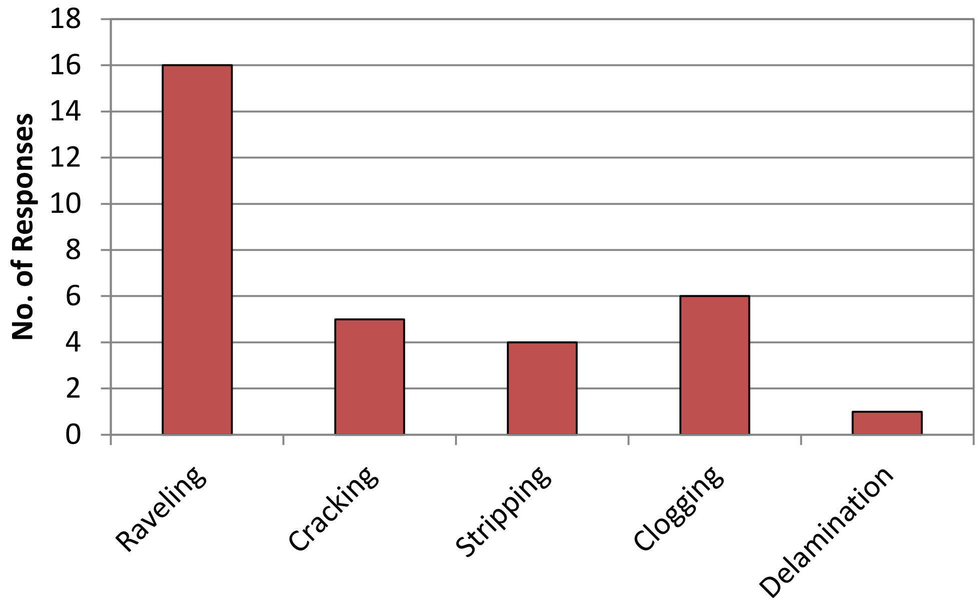


Figure 19 – Primary Distress Observed in PFC Pavement (Survey Response)

In the Netherlands over 80 percent of the roadways have some form of porous pavement. The Netherlands has a very harsh climate and is subject to many freeze/thaw cycles throughout the year yet still has adequate performing mixes. Mercado et al. states that even though the Netherlands has good success with porous pavements, they also have a standard maintenance practice of rehabilitating 5-7 percent of their PFC pavements annually. The design life of theses pavements exhibits a wide range of 5 – 18 years. The Netherlands uses a two-layer system that has a coarse-graded PFC on the lower lift and a finer-graded PFC on the surface. A fixed binder content of 4.5 – 5.5 is also utilized (Arambula-Mercado, 2016). Even though there is a set range for the binder content, the mix designs must still meet certain criteria when tested using a Cantabro abrasion test, semicircular bending test, indirect tensile strength and a rotating surface abrasion test (Ongel, 2007).

FACTORS AFFECTING PEFORMANCE OF PFC MIXES

In 1993, the Long Term Pavement Program (LTPP) developed a list of the most common distresses for hot mix asphalt (HMA). A comprehensive list of these distresses can be found in Table 8. PFC pavements primarily fail by raveling (Huber, 2000) with longitudinal cracking coming in second. Delamination is also seen in PFC pavements. The following sections discuss some factors that can potentially cause these distresses and other issues affecting the functional performance of PFC pavements.

Raveling

Raveling can be sorted into short-term and long-term raveling. Short-term raveling occurs on new construction due to the shearing force between tires and the pavement surface. Potential causes for this include, but are not limited to, placing the mix at an inadequate temperature or not properly compacting the mix that consequently prevents the creation of the stone skeleton needed to maintain the structural integrity of the pavement. California's design guide for OGFC also maintains that inadequate compaction can cause short-term raveling (Caltran, 2006). Other construction issues such as waiting on trucks or long haul distances can also play a part in the short-term raveling. If construction is on hold while waiting on trucks, it can cause differences in the temperature profile of the pavement, resulting in cold areas. Likewise, if there is a long haul distance, cold mix may be placed on joints or transition areas which will not attain adequate compaction due to lack of heat.

Table 8 – LTPP Defined Distresses for HMA Pavements (LTPP, 1993)

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

Long-term raveling is difficult to define; however it is the primary reason for the termination of a PFC service life. If short-term raveling does not occur, Pucher et al. (2004) states that OGFC pavements will deteriorate slowly for the first 5 to 10 years, but deterioration will significantly increase at this point, and raveling is the most commonly observed distress that causes this increase in deterioration. In early studies, prior to the use of modified binders, is was thought that over time gravity segregation of the binder and aggregate was causing the binder to drain down through the pavement consequently having less binder near the surface and allowing stone particles to be dislodged. Once stone particles are dislodged from the surface, other stones are dislodged at an increasing rate because there is a lack of support in the stone structure. This, accompanied with the aging (oxidation and hardening) of the binder, results in diminished service life (Molenaar, 2000). The hardening of the binder potentially causes loss in cohesive and adhesive bonds between the aggregate and binder. According to Nicholls et al. (2001), the oxidation and hardening of the binder may cause the pavement to become brittle at lower temperatures and therefore the strain from the traffic causes the binder-aggregate bond to fail. The indication is that micro-cracks form in the binder or mastic and over time form macro-cracks, which lead to the separation of binder and stone.

It is well known that given time and higher temperatures, asphalt binder can be self-healing (Garcia, 2012). Due to the severe problem with raveling, the Netherlands built a self-healing test section on Dutch highway A58 in December of 2010. The idea was to use the process of induction to keep macro-cracks from forming in the bond between the binder and aggregate. Steel wool was added as a stabilizing additive and also as an inductive agent. At the end of each winter the porous pavement was heated via induction energy as a form of preventative maintenance. This is still an on-going test, and no results could be found to determine what effect the induction process had on the potential of raveling.

Delamination

Delamination of PFC pavements is due largely to construction practices and tack rate. In the recent survey conducted by NCAT, tack coat rates as low as 0.02 gal/sy were reported. When placing a PFC mix over a dense-graded mix, it is imperative that a heavier tack coat be placed so that an adequate bond can be formed. Since there is less contact area for PFC mixes, a heavier tack coat is needed to compensate than for dense-graded mixes. In the survey conducted by Cooley et al. (2009), the tack coat rate ranged from 0.04 - 0.2 gal/sy. It was also noted that while most agencies specified an emulsion, some required a performance-grade binder instead. If the underlying layer is deemed permeable (>5 percent air voids (Alderson, 1996)), a slow setting emulsion should be placed at a rate of 0.05 – 0.10 gal/sy residual asphalt in order to seal the layer. British Columbia requires a tack rate of 0.17 gal/sy for sealing underlying layers (Bishop, 2001). Delamination can also be caused by moisture damage and may occur due to the mix being cold when placed on the receiving surface.

Top-Down Cracking

Cracking of PFC mixes is not as common as raveling or delamination, but it does occur. There is much debate about the type of cracking that is predominant in PFC mixes. Top-down, fatigue and reflective cracking are all possible, but the most common appears to be top-down cracking.

Top-down cracking is attributed to the shear stress applied to the pavement by tires. A schematic of tire pavement interaction can be found in Figure 20. Myers et al. (1999) stated that due to the relatively rigid tire wall and the structure of bias ply tires, the ribs of the tire cause an inward shear stress at the surface of the pavement by pulling the ribs into the center of the tire. Chen et al. (2012) concluded that the type of bond material at the interface between the PFC and underlying layer was critical in mitigating top down cracking. Using a Fracture Mechanics analysis, they suggested that the use of a thick polymer (SBS) modified tack increased the mixture's fracture resistance over conventional anionic slow setting emulsions (Chen, 2012).

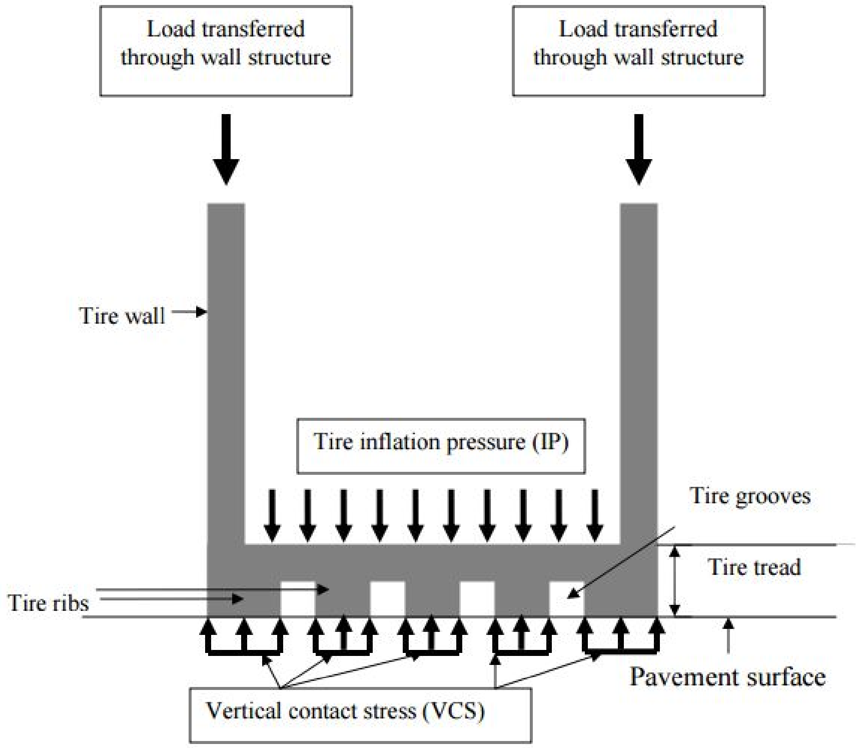


Figure 20 – Load Transfer Diagram of Tire-Pavement Interface (Baladi, 2003)

Loss of Permeability over Time

While PFC pavements are designed to reduce noise level, decrease the potential of hydroplaning, minimize splash-spray and improve friction values, this is only possible because of the high air void content in the mix. Because of this open void structure, PFC pavements become clogged with dust, silt and other debris over time. This debris causes a reduction in permeability of the pavement. An impermeable PFC cannot perform according to design, and it also loses some of the safety benefits accompanied with the use of a PFC. According to the recent survey conducted by NCAT, agencies reported design air void contents for PFC mixes ranging from 12 to 26 percent. These same agencies also report that little to no effort is being put into maintenance practices to address the issue of loss of permeability over time. The use of a standard maintenance practice is imperative in order to maintain the serviceability and function of a PFC. It has been recommended that a vacuum sweeper with a high pressure water system be used at least three times a year to prevent clogging of the pavement (Shirke & Shuler, 2008). The process of vacuum sweeping requires a large maintenance cost and can potentially damage the pavement. None of the agencies surveyed indicated that field permeability was being conducted to track the performance of the PFC mixtures. Some testing that was performed at the NCAT Pavement Test Track showed a decrease in permeability with traffic loading (Figure 21).

A large portion of the loss in permeability appears to happen directly after construction. This could be due to densification of the mix under traffic loading. Compaction methods for PFCs are dependent on the agency. Standard practice is to use steel wheel rollers only. Vibratory compaction can cause aggregate breakage and the goal of PFC compaction is not to achieve a density requirement, but to “seat” the aggregate in place. Pneumatic tire rollers are avoided because they tend to pick-up the mix during compaction. In the recent survey by NCAT only Nebraska monitored field density by use of cores being removed from the pavement during construction. Mississippi responded that a specific roller pattern was used based on historical results.

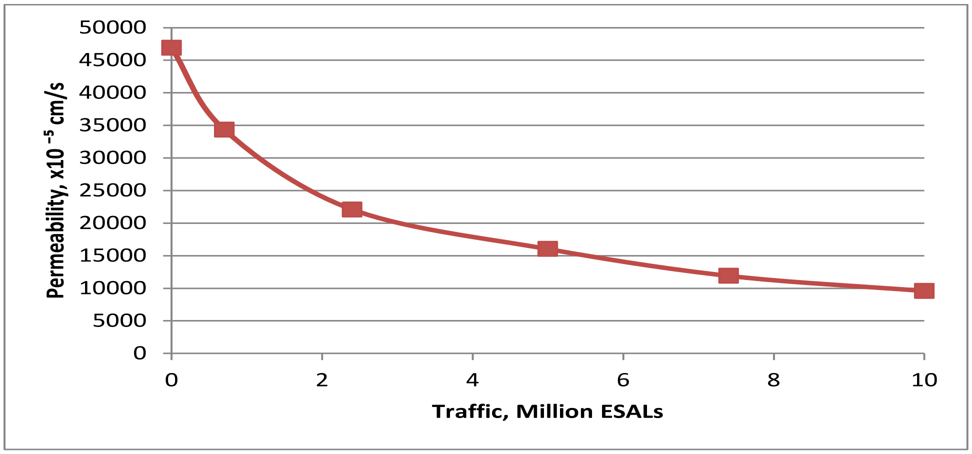


Figure 21 – Reduction in Permeability over Time with Traffic Loading at NCAT Test Track

Alvarez et al. (2009) recommends a field density requirement for PFC mixes in order to prevent over-compaction and non-uniform densification. It was stated that additional research would be required to develop appropriate and efficient methods for determining the density of the mix. One method for achieving this is the use of a field permeameter (Figure 22). There is a direct correlation between air voids and permeability of a mix (Brown E. R., 2004). A reasonable correlation between laboratory design air voids and field permeability was produced by NCAT from some sections of the test track (Figure 23). This was conducted directly after construction, prior to densification of the PFC mixture. However, according to this correlation, in order to achieve a permeability requirement of 100m/day the design air void content must be at least 15 percent. Argentina, Belgium and Japan specify that field permeability be tested at the time of construction as a quality control procedure. Similarly, Spain conducts field permeability testing as a method for determining the degree of compaction that has been attained during construction (Cooley, et al., 2009).

Figure 22 – NCAT Truck-Mounted Field Permeameter

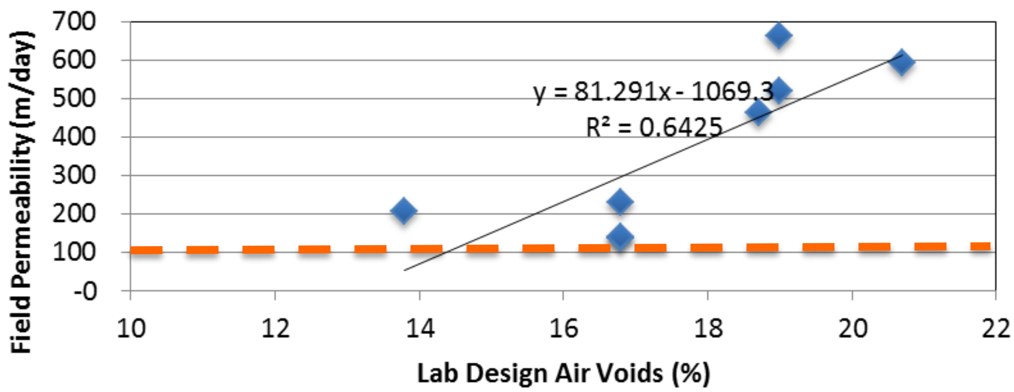


Figure 23 – Correlation between Lab Air Voids and Field Permeability - NCAT Test Track

The Danish Road Institute developed a method for determining the field permeability of the PFC, similar to the field permeameter in Figure 22, but with a much simpler process for determining if the pavement is clogged (Alvarez, et al., 2006). A specially designed tube is used to direct 10 cm of water into the pavement to assess the degree of clogging. Table 9 shows how the flow times correlate to the degree of clogging.

Table 9 – Danish Road Institute Field Permeameter Performance (Alvarez, et al., 2006)

TIME TO DRAIN 10 CM OF WATER FROM SPECIAL TUBE	PERFORMANCE EXPECTATION
Less than 30 Seconds	Highly Permeability (Expected of New PFC)
30 to 50 Seconds	Medium Permeability (Partially Clogged, but Can Be Cleaned)
Greater Than 75 Seconds	Low Permeability (Clogged, Cannot Be Cleaned)

Loss of Noise Reduction over Time

One of the primary reasons for the loss of noise reduction is clogging of the pores with dust, sand, silt and other types of debris. As noted in the previous section, there are methods for preventing and correcting clogging of the PFC, to a point.

A study conducted by NCAT measured the noise levels of 7 different PFC mixes across three states and compared that to the air voids of the pavement (Hanson, 2004). The air void range of the pavements was from 13 to 20 percent, which correlated to a 3 dB (A) range in noise results. The results of the study (Figure 24) show that as air void content decreases the noise level increases. This seems to indicate that if the PFC becomes clogged and the air void content decreases, there will be an increase in the noise level.

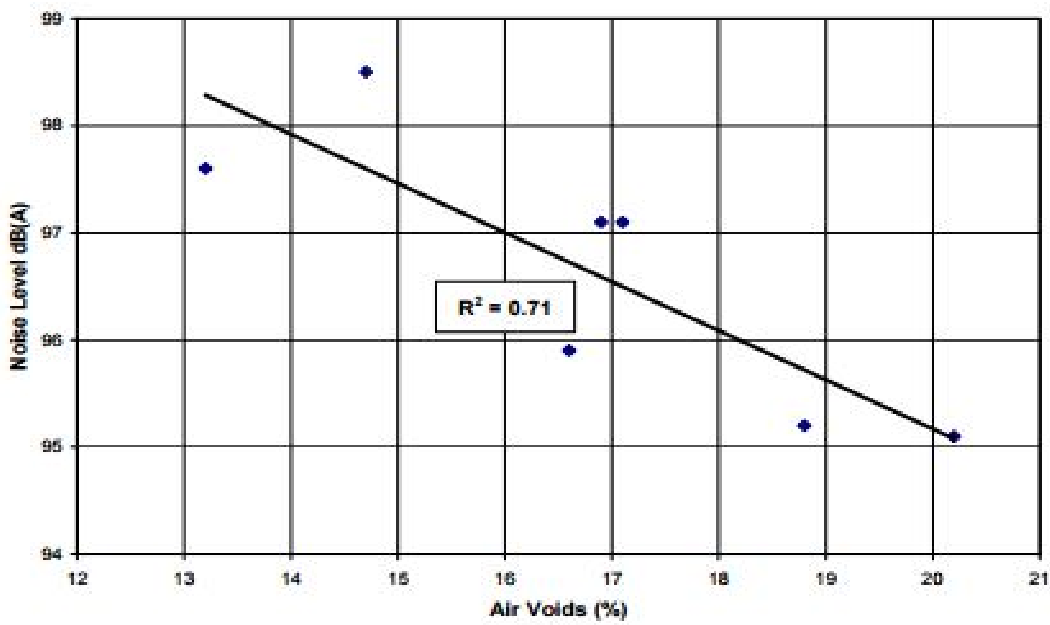
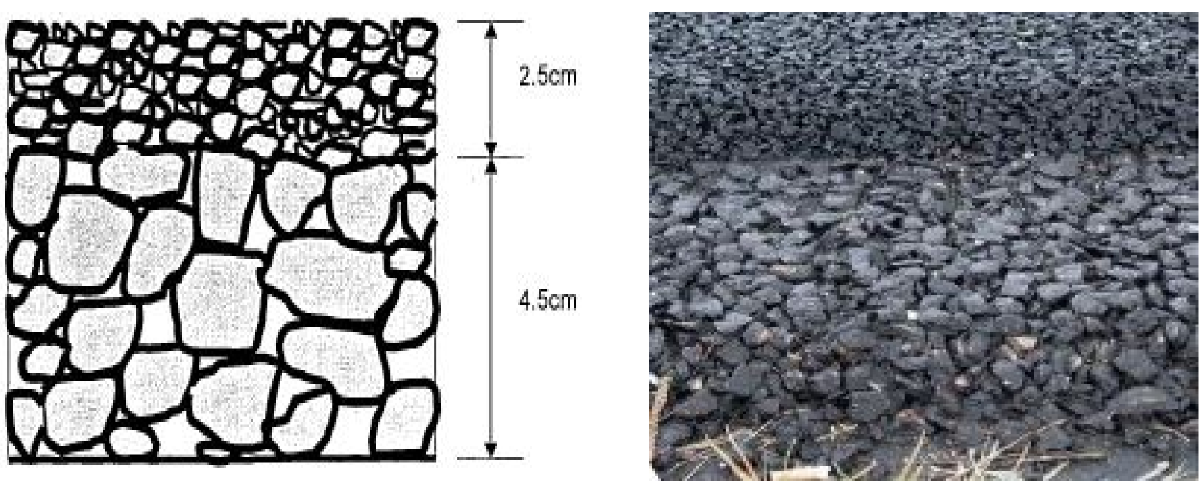


Figure 24 – Relationship of Tire-Pavement Noise to Air Void Content of PFC Mixtures (Hanson, 2004)

In another study conducted at the NCAT Test Track by Smit (2008), certain sections of the track were specifically designed with noise mitigation as the primary goal. It involved the testing of three different types of asphalt pavement: dense-graded mix, stone matrix asphalt (SMA) and PFC. The results of the study clearly indicated that PFC pavements decreased noise levels and that a thicker PFC section would provide a greater degree of reduction in noise. The maximum aggregate size of the mixture also appears to play a role in noise reduction potential.

Isenring et al. (2000) showed a reduced noise level with a smaller maximum aggregate size mixture, even after the pavement had been clogged.

In order to avoid clogging, many European countries and South Africa now use a twin-layer porous pavement (Twinlay). Figure 25 shows an illustration of Twinlay pavements in South Africa and The Netherlands. Both layers are PFC designs, but the top mixture has aggregate ranging in size from 4-8 mm, while the bottom mixture has particles ranging in size from 11-16 mm. The Twinlay approach can also minimize the effect of traffic noise. A study conducted at the NCAT test track during the 2006 test cycle used a Twinlay PFC that was laid simultaneously using an imported European paver (Figure 26). This section of the test track was the quietest surface tested for the entire 2-year (10 million ESALs) research cycle. This section was also the most drainable section for this cycle (Willis et al., 2009).



South Africa Illustration **Dutch Highway – The Netherlands**

Figure 25 – Examples of Porous Twinlay Pavements (Masondo, 2001) (Vejdirektoratet, 2012)

Figure 26 – Imported European Dual Paver for Twinlay PFC at NCAT Test Track

During the 2012 NCAT Test Track cycle, noise testing was conducted on a porous mix designed with SBS polymer and another one designed with GTR. The testing was conducted over a two-year period using the On-Board Sound Intensity (OBSI) procedure. The results for this testing can be found in Figure 27. It can be seen that as time increases the noise levels increase. This is mostly likely due to binder creep, densification of the pavement with increased traffic and clogging of the pores with debris.

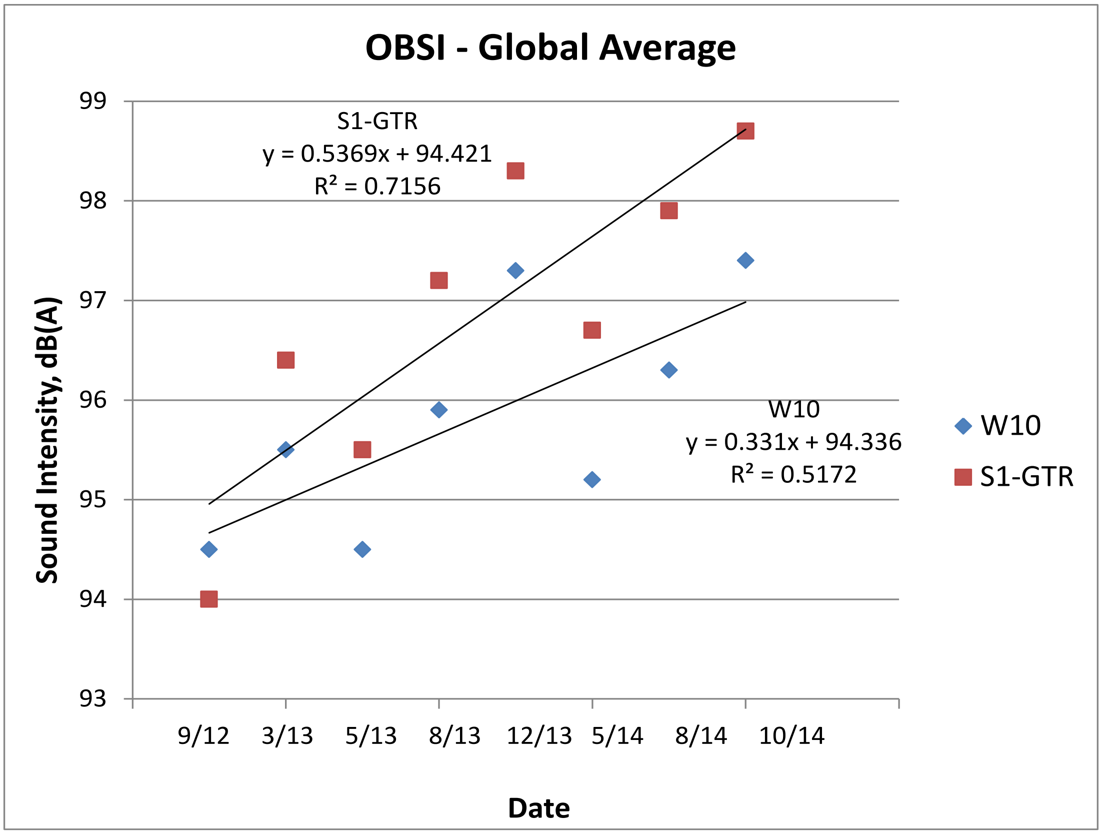


Figure 27 – Loss of Noise Abatement over Time and Traffic Loading

Issues Related to Cold Weather

Porous pavements in cold regions pose two major concerns, the first of which is the use of ice-control materials such as fine-graded sand and salt (Bernhard & Wayson, 2004). These materials can cause the pavement to clog and subsequently lose permeability and increase the noise level between the tire and pavement. This problem can be avoided by using brine or wetted salt, which will prevent freezing of any surface water with little potential for clogging. The recent survey showed that many state agencies do not use PFC mixtures because of problems encountered with snow and ice removal. One of the northern states, Maine, reported that vacuum-sweeping is conducted periodically in the winter months to keep the surface pores from clogging. In addition to the increased need for pavement maintenance in winter months, there are other financial costs associated with winter maintenance of PFC pavements. Due to the open nature of the pavement, a significant portion of the deicing materials settle into the PFC layer. This necessitates larger amounts of deicing material being used on the pavement, which leads to increased cost. France has ceased use of PFC mixes due to the increased winter maintenance cost of the additional 30 to 50 percent salt (Vejdirektoratet, 2012). Denmark uses special spreaders that apply a dry salt and brine simultaneously. While this process is more efficient, it still costs 10 to 20 percent more than winter operations for dense-graded mixes.

It was originally thought that since PFC pavements cool faster than dense-graded mix, the PFC would form frost and ice prior to the dense-graded mixtures and it would persist longer as well. Research conducted by Lebens et al. (2012) on MnRoad test sections, showed that snow and ice appeared to melt faster on the PFC pavement than on the standard dense-graded pavement (Lebens, 2012). No evidentiary support could be given to explain the cause of this phenomenon, and it was stated that more research would be needed on the topic.

Lift Thickness

The ratio of NMAS to lift thickness may be one of the factors leading to raveling. The recommended lift thickness to NMAS ratio for coarse-graded mixes and stone matrix asphalt (SMA) mixes is 4.0 (Brown E. R., 2004). The most common use of PFC mixes is for the removal of surface water. This dictates the thickness of the PFC mix based on the amount of expected rainfall and the amount of water storage needed (Cooley, et al., 2009). While most agencies require a specific thickness for PFC mixes, it is primarily based on past experience rather than a calculation of stormwater run-off. A design chart for PFC lift thickness can be seen in Figure 28. As shown in the figure, the chart takes into account cross-slope, permeability (k), rainfall intensity (I) and length of flow path (L) but does not include NMAS. The recent survey showed ranges in PFC thickness from 0.5 to 1.25 inches. A typical PFC thickness in the U.S. is less than

1.0 inch, while in Europe, the PFC thicknesses range from 1.0 to 2.0 inches (Cooley, et al., 2009). The 2-inch lift thickness is from the Netherlands and is dictated by the amount of expected rainfall.

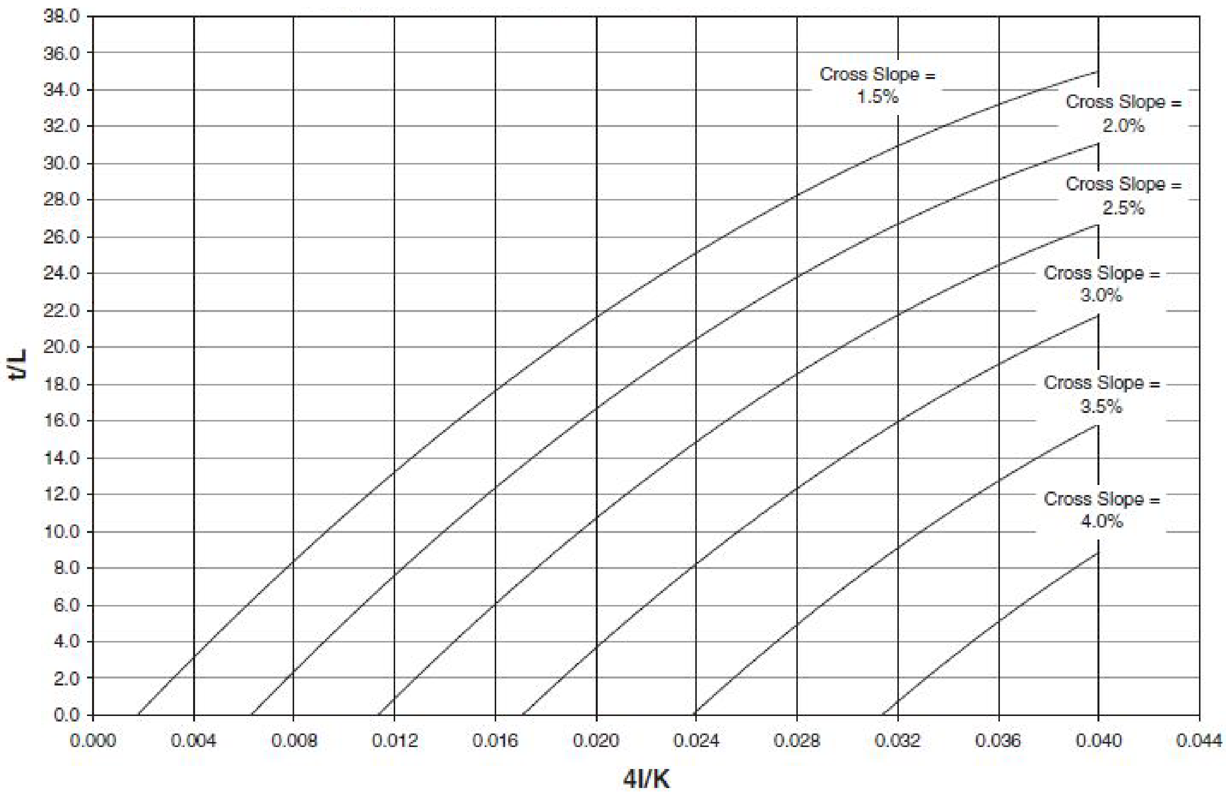


Figure 28 – Design Chart for PFC Lift Thickness (Cooley, et al., 2009)

Clemson University recently conducted a study that concluded that permeability and rainfall intensity had the greatest influence on PFC lift thickness selection, as long as the layer thickness was 2 times the maximum aggregate size. The study used the rainfall intensity at the 90th percentile and a minimum allowable permeability of 164 in/h and concluded that a PFC layer thickness should be between 1 and 1.25 inches thick for a single lane. Most highways that are paved with PFCs are two lanes, or more, in each direction excluding the shoulders. The pavement thickness for PFC pavements has a linear relationship with pavement width (Figure 29) (Putnam, 2012).

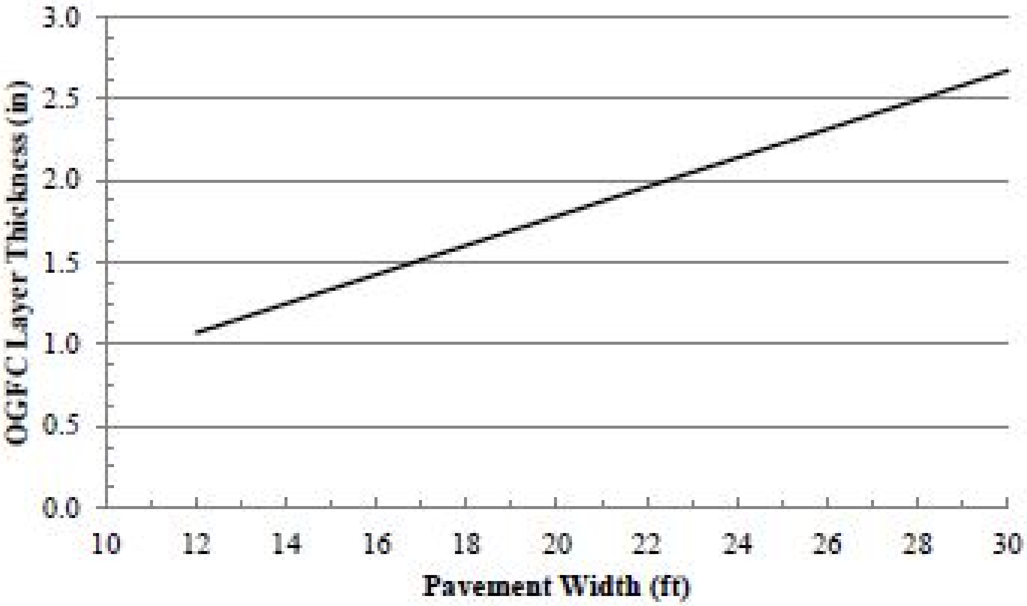


Figure 29 – Porous Pavement Lift Thickness based on Pavement Width (I=0.37 in./hr., K=164 in./hr., α=2.0%) (Putnam, 2012)

The NCAT Test Track demonstrated some differences in the performance of PFC mixes placed at different lift thicknesses (Watson D. , 2014). During the 2009 cycle, Sections S8 and N2 were surfaced with identical PFC mixtures with the only difference being the lift thickness. Section S8 was placed at 1.3 inches thick and performed well for the entire cycle, while section N2 was placed at 0.8 inches thick. N2 failed during the test cycle and had to be replaced. The thickness to NMAS ratio (t/NMAS) was 2.5 for S8 and 1.6 for N2. Additional research may show that increasing the lift thickness of a PFC may increase cohesion. The theory being that a larger lift thickness will promote more aggregate interlock. Another method for achieving better aggregate interlock may be to use a smaller NMAS mix design.

CHAPTER 3 – WORK PLAN

INTRODUCTION

The work plan was divided into two parts. Part 1 was to evaluate mix design parameters, performance tests and performance criteria that can be included in a performance-based specification. Part 2 was to further evaluate the effect of some specific mix components on the PFC mix performance parameters determined in Part 1. Six different mix designs were included in Part 1, and two of these mix designs were included in Part 2.

The six mix designs were selected based on their field performance and mineralogy. Three aggregate types (granite, limestone and traprock) were used in this study. Three poor performing mix designs were selected from Florida (Limestone), South Carolina (Granite) and Virginia (Traprock) while three good performing mix designs were selected from Florida (Limestone), Georgia (Granite) and New Jersey (Traprock). Based on agency comments, the *good* performing mixes have had services lives up to 18 years before being replaced while the *poor* performing mixes were replaced within 8 years.

In order to make findings from the study applicable to states that do not use hydrated lime as an anti-strip agent in PFC mixes, the research team agreed that a liquid anti-strip agent would be used for all mix designs evaluated in this study. In addition, for PFC mix designs with hydrated lime, it would be replaced by baghouse fines to maintain the total dust content. The liquid anti-strip agent used in all the mixes in this study is LOF 6500, an ArrMaz Custom Chemicals product, added to the binder before mixing at a dosage of 0.5 percent by weight of the binder.

Verification of PFC Mix Designs

The six mix designs were first verified in the laboratory before further testing was conducted. The job mix formulas (JMF) for each of the mixtures tested can be found in the Appendix. All of the aggregate specimens were taken from the same quarry referenced on the JMFs. Gradations of the aggregate materials sampled for this study were performed, and they were slightly different from those in the JMFs. This was not unexpected as some of these mix designs were done over 20 years ago. The variation in production or sampling may cause the difference between the historical and current gradations. To account for the difference, the aggregates were fractionated and then batched so that the aggregate gradations of the mixtures tested in this study were as close as possible to those in the respective mix designs. Specific gravity testing was conducted on all of the stockpiles as well so that the voids in mineral aggregate (VMA), voids in the coarse aggregate (VCA) and the film thickness could be calculated accurately.

PART 1 – DEVELOPMENT OF PERFORMANCE–BASED PFC MIX DESIGN

Currently, there are two national standard mix design procedures for PFC—ASTM D7064 and AASHTO PP 77. These procedures are similar to each other and include requirements for selecting materials, design gradation, optimum binder content and performance testing for draindown, raveling, permeability and moisture susceptibility. In Part 1, a performance-based mix design procedure was to be developed based on these procedures.

Table 10 shows important parameters in four areas, including selecting materials, determining the aggregate structure, determining the optimum asphalt content, and conducting performance tests, for a performance-based PFC mix design procedure. These important parameters were initially considered as a baseline for producing a good-performing PFC mix design and were therefore determined for each of the six mix designs. Results of Part 1 would be analyzed to determine which parameters and associated criteria could be used to distinguish PFC mix performance in the field. The performance tests were conducted based on the following test procedures in Part 1.

- AASHTO T305 – Draindown Characteristics in Uncompacted Asphalt Mixtures
- AASHTO T283 (modified) – Tensile Strength Ratio
- AASHTO T324 – Hamburg Wheel Track Testing
- ISSA TB 100 – Wet Track Abrasion Test
- Tex-248-f – Overlay Test
- AASHTO TP108 – Cantabro Loss
- Illinois Test Procedure 405 - Determining the Fracture Potential of Asphalt Mixtures Using the Illinois Flexibility Index Test (I-FIT)

Table 10 – Expected Critical Factors for a PFC Performance-Based Mix Design

• **Materials Selection**	• **Optimum AC**
• LA Abrasion: ≤ 30% • FAA: ≥ 45% • F&E: ≤ 10% @ 5:1 • SE: ≥ 50 • PG: 1 to 2 grades higher • Antistrip: liquid • Fiber: cellulose	• N_{DES}: 50 gyrations • %Va: ≥15% • %Pb: min. 6% ($VCA_{mix} \leq VCA_{drc}$) • % VMA: minimum point on curve • Film Thickness: ≥ 24 µm • Passes all performance requirements below
• **Gradation Optimization**	• **Performance Requirements**

• 3 trial gradations
• #200: min. 2%, max. 8%
• $VCA_{mix} \leq VCA_{drc}$
	• Draindown: max. 0.3% (2.36 mm wire basket)
	• Raveling:
	  • Cantabro loss (max.): 20% (unconditioned) & 30% conditioned (freeze-thaw)
	  • Wet Track Abrasion: Criteria not yet developed
	• Stripping (TSR): ≥0.7 ; psi: ≥50
	Hamburg Stripping Inflection Point: ≥5,000 cycles (after freeze-thaw conditioning)
	• Permeability: (min. 100 m/day)
	• Cracking:
	  • Overlay test: Min. 200 cycles
	  • Semi-Circular Bend: FI of 8.0 or greater

PART 2 - OPTIMIZING PERFORMANCE-BASED MIX DESIGN PROCEDURE FOR PFC

The experiment in Part 2 was to evaluate the contribution of filler, binder modification, fiber, and thickness-to-NMAS ratio to the mix resistance to raveling and cracking. Part 2 included three experiments. A description of each experiment follows.

Experiment 1 – Effect of Added Dust

The original approach was to use the Georgia *good* granite mix design and alter the dust content by adding 3.0 percent and 6.0 percent dust to the existing mix design to determine if durability could be improved. The stockpile percentages were altered for both of these options to keep the blend gradation as close as possible to the JMF. After some preliminary testing on design specimens, at varying asphalt contents, it was concluded that 3.0 and 6.0 percent added dust decreased the air void content below acceptable limits. The 6.0 percent added dust produced an average permeability value of 11 meters/day and an average air void content of 11.4 percent. Since these results were unacceptable for a PFC mix design, the added dust content was changed to 2.0 and 4.0 percent. The 4.0 percent added dust produced an average permeability value of 66 meters/day and an average air void content of 14.3 percent. While this was deemed close to acceptable based on the anticipated 15 percent minimum air void content, it was decided that this mix was already practically optimized and was only slightly improved when 2.0 percent dust was added to increase durability. For this reason the South Carolina *poor* granite mix design was included in the testing plan. The South Carolina design has the same mineralogy as the Georgia design and a similar blend gradation. This additional testing allowed for the comparison of a *good* and *poor* mix with added dust. From the testing performed with the 6.0 percent added dust, it was decided that the maximum added dust should be 4.0 percent. Data supporting these decisions and explanations can be found in Chapter 5. The testing plan for the granite designs with 2.0 and 4.0 percent added dust is shown in Table 11. Performance tests, shown in Table 11, were conducted in this experiment to determine the mix resistance to raveling and cracking.

Table 11 – Testing Plan for Experiment 1

<table>
<tr><th>Mix Design</th><th>Added BHF</th><th>Cellulose Fiber</th><th>Binder</th><th>Performance Test</th></tr>
<tr><td rowspan="2">Georgia Granite “Good”</td><td>2%</td><td rowspan="2">0.4%</td><td rowspan="4">PG 76-22 (SBS)</td><td rowspan="4">➢ Tensile Strength Ratio
➢ Cantabro
➢ Hamburg
➢ I-FIT SCB
➢ OT
➢ Permeability
➢ Draindown
➢ Wet Track Abrasion</td></tr>
<tr><td>4%</td></tr>
<tr><td rowspan="2">South Carolina Granite “Poor”</td><td>2%</td><td rowspan="2">0.3%</td></tr>
<tr><td>4%</td></tr>
</table>

Experiment 2 – Evaluation of Binder Modification

This experiment was designed to determine what effect binder, and its modifications, had on the performance of the mix. The test plan for this experiment is shown in Table 12. The Georgia *good* mix design was used for this part of the experiment and was tested without adding more dust to the mix. The PG 76-22, modified with SBS, was tested with and without cellulose fiber (0.4%) while the other two binder grades were tested without fiber. This was done to determine if fibers were necessary with modified binders. Several performance tests as shown in Table 12 were conducted in this experiment to evaluate the mix resistance to raveling and cracking.

Table 12 – Testing Plan for Experiment 2

Mix Design	Binder	Cellulose Fiber	Performance Test
Georgia Granite "Good"	PG 76-22 (SBS)	0.0% & 0.4%	➢ Tensile Strength Ratio ➢ Cantabro ➢ Hamburg ➢ I-FIT SCB ➢ OT ➢ Permeability ➢ Draindown ➢ Wet Track Abrasion
	PG 76-22 (Ground Tire Rubber)	0.0%	
	PG 82-22 (Highly Modified)	0.0%	

The PG 76-22 binder modified with Ground Tire Rubber (GTR) was blended in the lab prior to mixing the performance specimens. A PG 67-22 produced by Ergon Inc. was used as the base grade and a minus 30 mesh GTR was added at a rate of 12.0 percent by weight of the virgin binder. A heating mantle was used to keep the binder at the appropriate temperature while blending the GTR. A high shear paddle mixer was used at a rate of 700 RPM when adding the GTR. The GTR was added to the virgin binder over the course of 2 minutes and then the binder was continually blended for an additional 30 minutes at 1,000 rpm. Care was taken to keep the binder from exceeding a temperature of 325°F while blending. The blended binder was divided into quart cans after blending due to concern that the GTR may settle in a larger container. Even after dividing the binder into quart cans, the binder was stirred with a glass rod prior to mixing every specimen to ensure it was a representative sample of the modified binder.

The highly modified PG 82-22 (HiMA) was prepared by Kraton Polymers US, LLC. It was recommended by Kraton to start with a base binder that had a low temperature grade of -28 and modify it with SBS to achieve a final grade of PG 82-22. It has approximately 7.5 percent SBS polymer, which is roughly double the rate of polymer used in a typical PG 76-22 binder.

Experiment 3 – Effect of Lift Thickness to NMAS Ratio

The purpose of this testing was to determine what effect lift thickness to NMAS had on the performance of the mix. Table 13 shows the testing plan for Experiment 3. Three mixes with different NMASs were evaluated. Two of the designs were PFC mixes, and the other was a dense-graded mix. The first PFC mix was the Georgia 12.5-mm granite mixture from Part 1, and the second PFC mix was a 9.5-mm PFC mixture tested in the 2012 Test Track. The dense-graded mix was a 4.75 mm mix design from another study at NCAT. The mix designs for each of these mixes can be found in the Appendix.

Table 13 – Testing Plan for Experiment 3

Mix Design	NMAS	Layer Thickness	Performance Test
Georgia Granite PFC	12.5 mm	1) 0.75 in. 2) 2.5 in. 3) 2.5 x NMAS	Splitting Tensile Strength
NCAT Test Track Section E9-1A PFC	9.5 mm		
Dense Graded Mix – Lee Road 159	4.75 mm		

To determine the effect of lift thickness to NMAS on the performance of the mix, specimens were compacted in a gyratory compactor to their design height for the PFC mixtures and to a height of 95 mm for the dense-graded mixture. They were then cut into 3 different lift thicknesses and were saturated, frozen and thawed (AASHTO T283) prior to testing them for indirect tensile strength. The 4.75 mm dense-graded mix was chosen to determine the sensitivity of the test results due to change in layer thickness. If the tensile strength of the dense graded mix is not affected by the varying specimen thicknesses, then this test plan will show that the difference in tensile strengths of the porous mixes may be due to aggregate structure and cohesion and not lift thickness. If the different lift thicknesses for the dense-graded mix show varying tensile strengths then this strategy may not be applicable in evaluating the durability of the mix in terms of the relationship of lift thickness-to-NMAS.

CHAPTER 4 – METHODOLOGY

INTRODUCTION

In order to have a good performing design, the PFC mixture must have a high enough air void content in order to achieve the permeability required for water to drain through and away from the pavement surface. Along with performance tests to evaluate durability and cracking concerns, testing was conducted to determine volumetric properties and permeability of the mixtures. The methods and processes followed for this testing are discussed in the following sections.

VOLUMETRIC ANALYSIS

The volumetric properties were calculated based on data collected at the NCAT laboratory. The specific gravities of the aggregates were determined according to AASHTO T84 and T85. The PFC specimens were fabricated on a Superpave Gyratory Compactor with a compaction effort of 50 gyrations. The gyration level of 50 is recommended in both ASTM D7064 and AASHTO PP 77. All specimens were fabricated in this manner, even though some performance specimens require certain specimen heights. By fabricating all the specimens to a design gyration this allowed all of the specimens to have the same level of compaction effort and consequently close to the same design air void content. The specimen air voids were determined according to ASTM D6752, *Bulk Specific Gravity and Density of Compacted Bituminous Mixtures Using Automatic Vacuum Sealing Method* (Equation 1).

$$G_{mb} = \frac{W}{W_b + W - W_{bs,w} - \frac{W_b}{CF}} \qquad \text{Equation 1}$$

Where:

W = the mass of the specimen in air (g)
W_b = the mass of the bag in air (g)
$W_{bs.w}$ =the mass of the sealed bag and the specimen in water (g)
CF = bag correction factor

This method was used because of the accuracy it provides for specimen density. The voids in mineral aggregate (VMA) and voids in coarse aggregate (VCA) were calculated for each mix to determine if these factors showed any variation between the *good* and *poor* mixes. The equations used for calculating VMA, VCA_{DRY} and VCA_{MIX} are shown below.

$$VMA = 100 - \frac{G_{mb} * P_s}{G_{sb}} \qquad \text{Equation 2}$$

$$VCA_{DRC} = \frac{G_{ca}\gamma_w - \gamma_s}{G_{ca}\gamma_w} * 100 \qquad \text{Equation 3}$$

$$VCA_{MIX} = 100 - \left(\frac{G_{mb} * P_{ca}}{G_{ca}}\right) \qquad \text{Equation 4}$$

Where:

VMA = voids in mineral aggregate
VCA_{DRC} = voids in coarse aggregate in dry-rodded condition
VCA_{DRC} = voids in coarse aggregate of the compacted mixture
G_{sb} = combined bulk specific gravity of the total aggregate
P_s = percent of aggregate in the mixture
γ_s = unit weight of the coarse aggregate fraction in the dry-rodded condition (kg/m^3)

γ_w = unit weight of water (998 kg/m^3)
G_{ca}= bulk specific gravity of the coarse aggregate
P_{ca} = percent of coarse aggregate in the mixture
G_{mb} = bulk specific gravity of the compacted mixture
G_{ca} = bulk specific gravity of the coarse aggregate

The film thickness of each design was also calculated to determine if film thickness should be a design consideration. A minimum film thickness of 24.0 microns was originally discussed; however, after some trials, it was determined that this was an unrealistic threshold based on specimen performance.

CANTABRO TESTING

Cantabro testing is used to determine the durability of a mix in relation to the asphalt binder content and grade. It is primarily used for evaluating PFC mixes but has more recently been used to evaluate other asphalt mixes. The test method followed for this testing was AASHTO TP108-14, *Standard Method of Test for Determining the Abrasion Loss of Asphalt Mixture Specimens.* According to ASTM D7064, an acceptable amount of loss for unaged specimens is 20%, while 30% is allowed for aged specimens. All design specimens for this testing were unaged. Design specimens are individually placed in the Los Angeles Abrasion machine, without the steel charges, and tested for 300 revolutions at a rate of 30 to 33 revolutions per minute. The loose material is then discarded and the final specimen weight is recorded. The percent loss is then calculated for each specimen according to Equation 5.

$$CL = \frac{A-B}{A} * 100 \qquad \text{Equation 5}$$

Where:

CL = Cantabro Loss, %
A = Initial weight of test specimen
B = Final weight of test specimen

DRAINDOWN

Draindown occurs when asphalt binder drains from the aggregate particles in the PFC mixture and settles in the bottom of the silo, transfer vehicles and construction equipment. This is due to several factors, the greatest of these being the exclusion of a stabilizing agent to hold the thick binder film in place. Cellulose or mineral fibers are the most common stabilizing agents and are very effective in preventing draindown. The degree of stiffness of the binder and the gradation of the mix are also part of the draindown cause. Typically, mixing temperatures at the plants are 35-50°F greater than the recommended compaction temperature so that the mixture will coat completely and will not lose too much heat prior to reaching the job site. A softer asphalt binder and a coarse gradation typically have a greater draindown potential. The draindown testing was conducted according to AASHTO T305, *Draindown Characteristics in Uncompacted Asphalt Mixtures,* using a 2.36mm mesh sieve for the draindown baskets. Samples using the PG 76-22 with SBS and the PG 76-22 with 12% GTR were tested at 320° and 347°F. The samples using PG 82-22 (HiMA) were tested at 340° and 367°F. According to the specification, the lower test temperature should be equivalent to the production temperature and the higher test temperature should be 15°C (27°F) above the production temperature. This accounts for the anticipated fluctuation in production temperature. The samples were conditioned in the basket over a pie plate for 1 hour. The maximum recommended amount of draindown allowed is 0.3 percent. The draindown is recorded as the percent of material that is on the pie pan after the 1-hour conditioning.

WET TRACK ABRASION TEST

The wet track abrasion test was originally designed for determining the wearing potential of slurry treatments under wet conditions while modeling abrasive traffic. The setup (Figure 30) was modified to test the PFC mix. The weighted rubber hose abrades the submerged specimen and potentially causes aggregate particles to lose their cohesive bond. Slab specimens were fabricated using a kneading compactor (Figure 31) at the NCAT laboratory and 10 inch cores were removed from the slabs for testing. The slab's target height was fixed at 1.0 inch to emulate in-place PFC pavements.

Figure 30 – Wet Track Abrasion Equipment (Equipment)

Figure 31 – Kneading Compactor for 20 x20 inch slabs (Tran, 2012)

HAMBURG WHEEL TRACKING TEST

The purpose of the Hamburg wheel-track test (HWTT) was to determine the susceptibility of the mixtures to stripping and rutting. All of the specimens were fabricated and tested according to AASHTO T324, *Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA).* All of the mixes were tested on a Cox Hamburg Wheel Tracker produced by Cox and Sons Inc. Six specimens were fabricated for each design so that statistical analysis could be performed on all of the mixtures. The machine can only test four specimens at a time, so the extra sets of specimens were tested separately. The specimens were subject to a load of 158 ± 1 lbs produced by a solid steel wheel and extra weights. The specimens were submerged and conditioned in a 50°C water bath for 30 minutes prior to testing. The water bath maintained the 50°C temperature for duration of the testing (20,000 passes). All data output was recorded by the computer using the linear variable differential transformer (LVDT) attached to each arm and was analyzed to determine the stripping point and moisture susceptibility of the mix. The stripping inflection point (SIP) of the mix was determined by incorporating tangents to the secondary and tertiary sections of the graph. The SIP is the value where the tangents intersect. An example of calculating the SIP can be found in Figure 32.

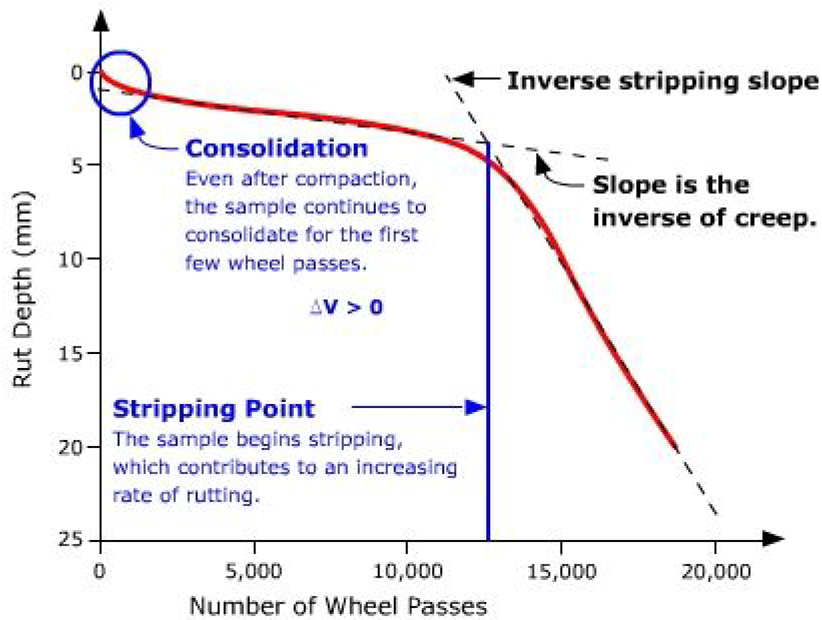


Figure 32 – Example Hamburg SIP Determination (Consortium, 2011)

There is currently no nationally accepted criterion for the maximum allowable rutting depth. Many states specify their own criteria based on performance grade of the virgin binder in the mixture. The most widely accepted criteria are from the Texas Department of Transportation (TxDOT). All of the mix designs with the exception of the Part 2 mix with PG 82-22 were produced with a PG 76-22. The TxDOT criterion for a dense-graded mix with PG 76-xx is < 12.5 mm of rutting at 20,000 passes (TxDOT, 2006). There is a TxDOT specification specifically for PFC mixes, but it only has criteria for fine-graded PFC mixes (PFC-F). All of the mixes tested in this study are classified as coarse-graded PFC mixes (PFC-C). Analyzing Hamburg data can be onerous when comparing different mix designs, due to the wide variability in results. The most concise way to report the data is to report the passes to failure and give a pass-fail status to the mix based on binder grade. A combined graphical depiction of the rut depth data for all the mix designs is also an effective method for comparing a mixture's performance.

TENSILE STRENGTH RATIO

The tensile strength ratio (TSR) test was conducted on each mix design. This test was conducted according to AASHTO T283, *Resistance of Compacted Bituminous Mixture to Moisture Induced Damage,* with slight modifications to accommodate PFC mixes. The modifications were recommended in the ASTM D7064 test procedure. The specimens were compacted to the design gyration level and height instead of the target height in the procedure of 95 mm. While this differs from the specification, the height of the specimens is included in the final calculations so this change is accounted for in the final results. The weight of the design specimens was altered slightly for these specimens to target a height of 110 to 115mm in order to ensure that the specimens fit inside the breaking head. The specimens were saturated at 26 in Hg (660.4 mm Hg) below atmospheric pressure for 10 minutes and then the saturated specimens were frozen in plastic concrete cylinder molds. The specimens were kept submerged under water while freezing to keep the interior voids filled with water. The rest of the test procedure was followed according to the specification. The specimens were tested for indirect tensile (IDT) strength on a Marshall Stability press at a rate of 2 inches per minute. The IDT strength of the mixes was determined by using the peak load recorded on the device and the specimen dimensions. The ratio of the conditioned and unconditioned IDT strengths must be at least 80% according ASTM D7064.

PERMEABILITY

The permeability of the specimens was tested according to FM 5-565, *Florida Method of Test for Measurement of Water Permeability of Compacted Asphalt Paving Mixtures.* The falling head permeability apparatus for 6-inch specimens along with the large diameter (6.985cm) graduated cylinder were used for this testing to determining the coefficient of permeability (k). Minimum permeability requirements vary by agency, with Mississippi requiring as low as 30 meter/day (Putnam, 2012). The majority of the states surveyed by NCAT responded that they had no permeability requirements. Research conducted by NCAT in 1999 recommended a minimum permeability of 100 meter/day (Kandhal P. S., 1999). While permeability testing is optional according to ASTM D7064, it also recommends a rate of 100 meter/day. If the main purpose of the PFC is to reduce noise, the recommended minimum permeability requirement is 60 meter/day (Alvarez, et al., 2006). The European standard requires a permeability range of 8.6 to 346 meter/day (Ongel, 2007).

The specimens were submerged in a container and allowed to soak a minimum of 1 hour prior to testing in order to condition the specimens. A minimum of three specimens was tested for each mix design. In order to provide accurate results, three consecutive runs had to be within 4.0 percent of each other. The formula for calculating the permeability of each specimen can be seen in Equation 6.

$$k = \frac{a*L}{A*t} * \ln\left(\frac{h_1}{h_2}\right) * t_c \qquad \text{Equation 6}$$

Where:

k = coefficient of permeability
a = area of the testing pipe
L = length of the specimen
A = testing area of the specimen
t = testing duration
h_1 = initial height of water
h_2 = final height of water
t_c = temperature correction for the water

CRACKING

As cracking was one of the primary distresses observed in PFC mixtures, the Texas overlay test (OT) and the Illinois Flexibility Index Test (I-FIT) were performed on each mix and the subsequent altered designs. The overlay test was used to determine the fatigue or reflective cracking potential of the mixes while the I-FIT test is used at intermediate temperatures to determine the fracture resistance of the mix. The Texas specification, Tex-248-F, *Test Procedure for Overlay Test,* was used for the overlay testing and the Illinois Test Procedure 405, *Determining the Fracture Potential of Asphalt Mixtures Using the Illinois Flexibility Index Test (I-FIT)* was used for the I-FIT testing. While the I-FIT has a provisional AASHTO standard, AASHTO TP124-16, *Determining the Fracture Potential of Asphalt Mixtures Using Semicircular Bend Geometry (SCB) at Intermediate Temperature*, the OT test has no AASHTO or ASTM specification. However, the OT test is widely used and requested for reflective cracking testing. The I-FIT test is relatively new to the industry but is making significant advancements and currently has applied for an AASHTO provisional standard. The I-FIT test is also known as a Semi-Circular Bend (SCB) test. There are currently two intermediate temperature SCB tests that are being explored as viable options for determining mixture cracking potential. For this study, the I-FIT method was chosen over the Louisiana Transportation Research Center (LTRC) method due to a high coefficient of variance (COV) observed when testing SCB specimens according to the LTRC method.

Texas Overlay Test

The overlay test was performed on an IPC Global Asphalt Mixture Performance Tester (AMPT) according to Tex-248-F. The notes regarding PFC mixes were followed with the exception of measuring the density of the trimmed specimens. In Note 8, the procedure recommends not measuring the density of the trimmed PFC specimens; however, based on past research, it is known that specimens of dense-graded mixes typically lose 0.5 – 1.0 percent air voids. It was therefore decided to check the specimens after trimming using the Corelok method to determine the amount of air void loss in the specimens. The amount of air void loss was different for each mix design, but the average loss for the PFC mixes tested for Part 1 was 2.1 percent. Prior to determining the bulk specific gravity of the specimens, they were vacuum dried to be sure all moisture was removed.

The cut specimens were glued to two metal plates with a gap of 4.2 mm between the plates and tested at a constant temperature of 25°±0.5°C. The 4.2 mm gap is a modification to the original Tex-248-f specifications. The updated specification (February 2014) changed the gap from 2.0 to 4.2 mm. Figure 33 still shows the 2.0 mm gap from the older version of the specification. The specimens were tested using the controlled displacement mode at a rate of 0.1 Hz. The testing is designed to terminate when the specimens reach 93% reduction of the maximum load or to be tested for 1,200 cycles. In some cases, the 93% reduction did not take place prior to the 1,200 cycles and the test was allowed to run for 2,000 cycles to determine the failure point. Some specimens still did not fail prior to 2,000 cycles, and the data were extrapolated to predict the cycles at which the specimen would have reached the 93% reduction.

One test specimen was cut from each gyratory compacted specimen. The specimen was trimmed to dimensions of 150 mm long by 76 mm wide by 38 mm high. A model of the Overlay Tester with a specimen glued to the plates can be seen in Figure 33. This is the original concept of the overlay tester and it has since been modified so that it can be tested in the AMPT with a conversion kit provided by the manufacturer.

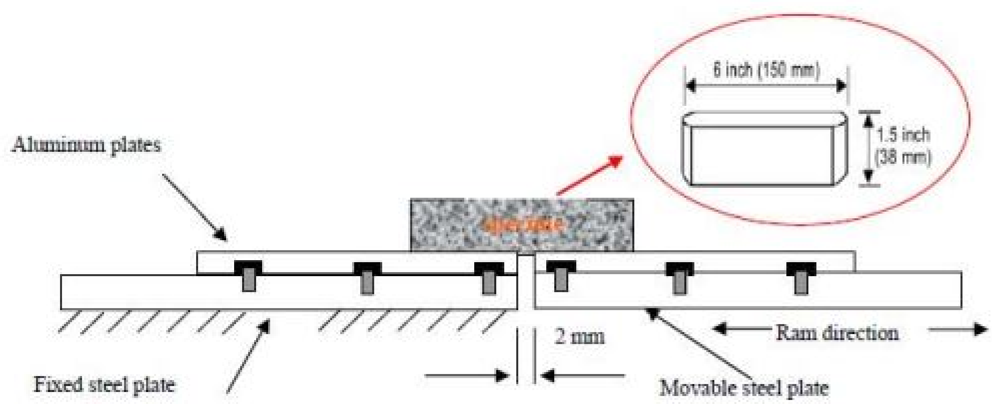


Figure 33 – Model of Texas Overlay Tester (F. Zhou & Scullion, 2007)

In the AMPT, the top plate remains fixed while the bottom plate applies the load to the specimen. An LVTD attached to the back of the conversion kit measures the cyclic saw-tooth load. The test uses a constant maximum opening displacement (MOD) of 0.635 mm (0.025 in) when applying the load to the bottom plate.

Illinois SCB Test (I-FIT)

The Illinois SCB test (I-FIT) was designed to determine the fracture resistance of asphalt at intermediate temperatures (77°F). The relevant parameters, including fracture energy (G_f) and Flexibility Index (FI), are used to predict the fracture resistance. The G_f is the energy required to create a crack in the surface of the mix (IDOT, 2016) whereas the FI provides a way to categorize and identify brittle and stiff mixtures. With increasing amounts of reclaimed asphalt (RAP) and recycled asphalt shingles (RAS) being incorporated into mixes, it is becoming necessary to develop an efficient method of determining a mixture's resistance to cracking. The PFC mixtures tested for this had no RAP or RAS added to the mixes with the exception of the Virginia design with traprock, which only had 5% RAP. FI was primarily meant for use in comparing and ranking mix designs with similar design parameters. It is the intention of this study to use the FI to rank the PFC mixes from the least to most brittle and see if this coincides with the mix performance in the field. The G_f is part of the calculation for determining the FI but alone can indicate the mixture's potential for damage resistance. The relationship between the G_f of the mix and the capacity of the mix to withstand stress is proportional. The current standard (Illinois Test Procedure 405) notes that this testing is only applicable to mixes with a NMAS of 19.0 mm or less.

The test specimens are trimmed from design specimens at a width of 50.0 mm with a tolerance of ±1.0 mm. The specimens are then cut in half so that the final specimen is a semicircular specimen with a thickness of 50 mm. A representation of the trimmed I-FIT specimen can be seen in Figure 34.

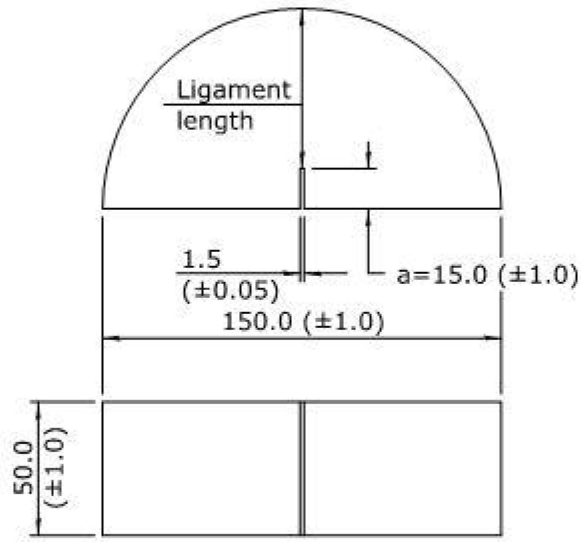


Figure 34 – I-FIT Test Specimen Illustration (IDOT, 2016)

While Figure 34 shows a notch tolerance of ±0.05mm the updated specification now specifies a tolerance of ±0.1mm. This is difficult to achieve due to the amount of vibration that occurs in the saw blade when trimming the specimens. A smaller blade width (1.2 mm) accounts for the vibration and allows the notch width to be within specification. The specimens are conditioned in an environmental chamber for 2 ± 0.5 hours at 25 ± 0.5°C prior to testing. The specimens are placed under a seating load of 0.1 kN prior to beginning the test. After the seating load is obtained, the test applies a load at rate of 50mm/min which is maintained until the specimen fractures and the recorded load falls below the initial seating load of 0.1 kN. The load line displacement (LLD), and corresponding load data, must be recorded for the entire duration of the test in order to accurately analyze the FI of the mix. The load and LLD results are used to calculate the post-peak slope (m), strength, critical displacement (u_1), G_f, and FI.

The calculation of the FI begins by determining the Work of Fracture (W_f), which is the area under the load vs LLD curve. A depiction of the load vs LLD curve can be seen in Figure 35. The W_f is used to calculate the G_f according to Equation 7 and 8.

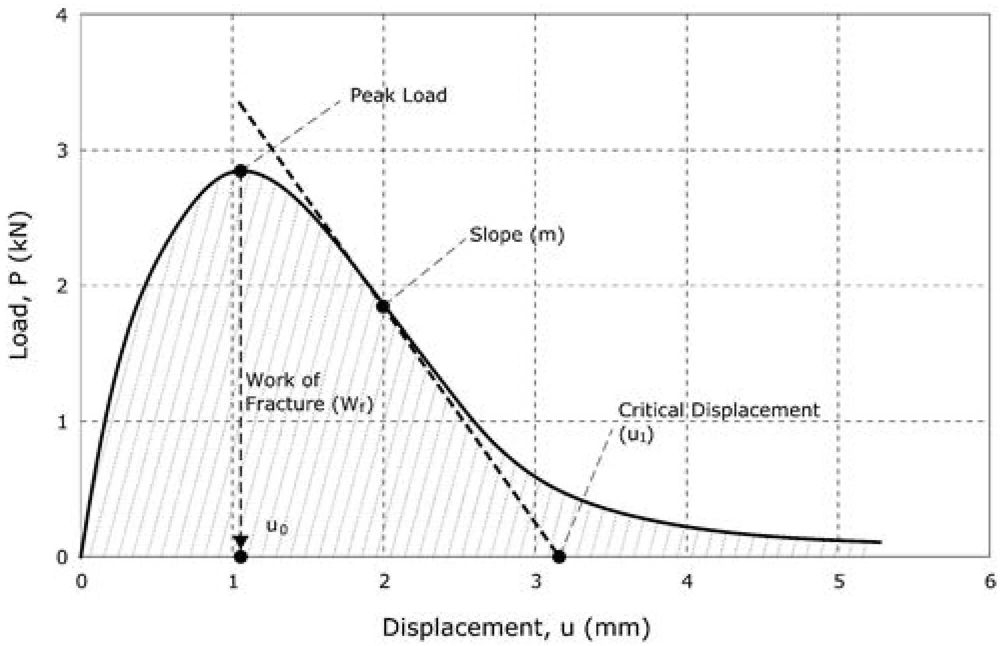


Figure 35 – Example of Load vs Load Line Displacement (LLD) Curve for I-FIT (IDOT, 2016)

$$G_f = \frac{W_f}{Area_{lig}} \qquad \text{Equation 7}$$

$$Area_{lig} = (r - a) * t \qquad \text{Equation 8}$$

Where:

G_f = fracture energy (Joules/m^2)
W_f = work of fracture (Joules)
P = load (kN)
u = displacement (mm)
$Area_{lig}$ = ligament area (mm^2)
r = specimen radius (mm)
a = notch length (mm)
t = specimen thickness (mm)
m = post-peak slope (kN/mm)

LABORATORY CONDITIONING OF SPECIMENS

The recommended aging procedures for the specimens were based on each test method. ASTM D7064 recommends short-term aging specimens according to AASHTO R30 for laboratory compacted specimens. This procedure was followed for the Cantabro and permeability test specimens. For all other performance test specimens, with the exception of the TSR testing, the *Short Term Conditioning for Mixture Mechanical Property Testing* (AASHTO R30) was used. This required that the specimens be conditioned for 4 hours at 135°C (275°F), while being stirred every hour, and then compacted.

The OT test procedure states in Note 3, “Cure warm-mix asphalt (WMA) mixtures at 275°F for 4 hr ± 5 min. before molding,” and requires all other laboratory mixed specimens be conditioned for 2 hours at the appropriate compaction temperature. The HWTT test procedure states that the mix must be conditioned according to AASHTO R30 *Short Term Conditioning for Mixture Mechanical Property Testing.* The I-FIT testing recommends fabricating the specimens according AASHTO T312, which then directs the user to AASHTO R30. AASHTO R30 has mixture conditioning methods for:

1. Volumetric Mixture Design
2. Mixture Mechanical Property Testing – Short Term
3. Mixture Mechanical Property Testing – Long Term

The performance testing for I-FIT is a mechanical property test, so it is assumed that option 2 or 3 should be chosen. *Short term* aging simulates the effect that production and construction has on the mix, while *long term* aging simulates the aging expected of the mix after 7-10 years post-construction. Since the objective of the research is to develop a performance-based specification, it was decided that all performance tests be fabricated according to the short term aging in option 2. A summary of the aging procedures can be found in Table 14.

Table 14 – Aging Procedures for Work Plan Testing

	Aging Requirements	
Test Method	**Specified**	**Performed**
Cantabro (Tx-245-f)	*HMA* - *2* h ± 5 min at 150°C *WMA* - *4* h ± 5 min at 135°C	*2* h ± 5 min at 150°C[1]
Permeability (FM 5-565)	Not Provided	*2* h ± 5 min at 150°C[1]
TSR (AASHTO T283 modified)	16 h ± 1 h at 60 ± 3°C → 2 h ± 10 min at Comp Temp	16 h ± 1 h at 60 ± 3°C → 2 h ± 10 min at 150°C[1]
HWTT (AASHTO T324)	4 h ± 5 min at 135 ± 3°C	4 h ± 5 min at 135 ± 3°C
OT (Tx-248-f)	*HMA* - *2* h ± 5 min at 150°C *WMA* - *4* h ± 5 min at 135°C	4 h ± 5 min at 135 ± 3°C
SCB (Illinois TP 405)	AASHTO T312→R30	4 h ± 5 min at 135 ± 3°C

[1]All mixes were compacted at 150°C with the exception of the PG 82-22 specimens (160°C)

CHAPTER 5 – PART 1: EVALUATION OF MIX DESIGNS

INTRODUCTION

Three aggregate types, including Limestone, Traprock and Granite, were evaluated in Part 1. For each aggregate, one *good* mix design and one *poor* mix design were selected per agency recommendations. Table 15 shows properties of the original mix designs recommended by the Florida Department of Transportation (FDOT), New Jersey Department of Transportation (NJDOT), Virginia Department of Transportation (VDOT), Georgia Department of Transportation (GDOT), and South Carolina Department of Transportation (SCDOT) with a few exceptions mentioned afterward.

Table 15 – Original Mix Design Components

Mixture Source	Florida	Florida	New Jersey	Virginia	Georgia	South Carolina
Mixture Designation	*Good*	*Poor*	*Good*	*Poor*	*Good*	*Poor*
Aggregate Mineralogy	Limestone		Traprock		Granite	
Asphalt Type	PG 67-22	PG 76-22				
Binder Modifier	12% - #30 GTR	2.5% SBS Polymer				
Anti-strip	0.5% LOF 6500 by weight of binder					
Fiber, %	0.4	0.4	0.3	0.3	0.4	0.3
Asphalt Content, %	7.1	6.0	6.0	5.8	6.0	6.0
Total P-200, %	0.9	0.9	3.9	2.6	2.0	1.7

The following changes were made to these mix designs for evaluation in this study:

- Two JMFs were in the mix design recommended by NJDOT. One used GTR while the other used SBS. The stockpile materials sampled for this study were for the SBS design. The GTR JMF included 0.3 percent fiber in the design, but the SBS design showed no fiber addition. Fiber at a rate of 0.3 percent was added to the New Jersey SBS design for this study.

- The mixtures recommended by GDOT and SCDOT contained hydrated lime. This was replaced, in terms of filler, with baghouse fines and, in terms of an anti-stripping agent, with liquid anti-strip.
- Georgia mixture (designed in 1995) used an AC20 binder. The Virginia mixture used a PG 82-22 RM. PG 76-22 asphalt binder was used for all designs.
- Virginia used a Pavebond Lite anti-strip, and Florida required a liquid anti-strip but did not specify what type in the JMF. Liquid anti-strip, LOF 6500, was used at a dosage rate of 0.5% by weight of binder for all designs.

Testing for gradations, specific gravity and dry-rodded unit weight was conducted on the aggregates. Some of the stockpile gradations did not match the JMF gradations reported. This was not unexpected since some of the designs were over 20 years old. The stockpile percentages were adjusted to match the original JMF blend gradation. Each adjusted gradation was verified to be within the broadband gradation limit, and that its differences from the original JMF blend gradation were within the agency's mix production tolerance. A comparison of the original JMF and adjusted blends is shown in Table 16 for the *good* blends and in Table 17 for the *poor* blends.

Table 16 – "Good" Mixture Gradations and Asphalt Contents

Percent Passing Sieve	**Florida "Good"**		**Florida Limits**	**Georgia "Good"**		**Georgia Limits**	**New Jersey "Good"**		**New Jersey Limits**
	JMF	NCAT		JMF	NCAT		JMF	NCAT	
25.0 mm, 1"	100	100		100	100				
19.0 mm, 3/4"	100	100	100	100	100	100			
12.5 mm, 1/2"	90	88	85 - 100	92	96	85 - 100	100	100	100
9.5 mm, 3/8"	66	68	55 - 75	66	66	55 – 75	92	91	80 – 100
4.75 mm, #4	24	26	15 - 25	25	21	15 – 25	34	36	30 – 50
2.36 mm, #8	10	8	5 - 10	8	8	5 – 10	13	11	5 – 15
1.18 mm, #16	8	6		5	6		8	8	5 – 10
0.600 mm, #30	7	5		4	5		6	6	
0.300 mm, #50	6	3		3	4		5	5	
0.150 mm, #100	5	2		2	3		4	5	
0.075 mm, #200	3.5	0.9	2 - 4	1.5	2.0	2 - 4	3.0	3.9	2 - 5
AC, %	7.1	7.1		6.0	6.0	5.75-7.25	6.0	6.0	

Table 17 – "Poor" Mixture Gradations and Asphalt Contents

Percent Passing Sieve	Florida "Poor"		Florida Limits	Virginia "Poor"		Virginia Limits	South Carolina "Poor"		South Carolina Limits
	JMF	NCAT		JMF	NCAT		JMF	NCAT	
25.0 mm, 1"	100	100		100	100		100	100	
19.0 mm, 3/4"	100	100	100	100	100	100	100	100	100
12.5 mm, 1/2"	93	88	85 - 100	100	100	100	95	95	89 - 100
9.5 mm, 3/8"	69	68	55 - 75	86	87	85 - 100	70	74	63 - 75
4.75 mm, #4	23	26	15 - 25	21	25	20 - 40	21	21	15 - 25
2.36 mm, #8	9	8	5 - 10	9	8	5 - 10	8	8	5 - 10
1.18 mm, #16	5	6			6			5	
0.600 mm, #30	4	5			5		5	3	
0.300 mm, #50	3	3			4			3	
0.150 mm, #100	3	2			3		5	2	
0.075 mm, #200	3.0	0.9	2 - 4	2.5	2.6	2 - 4	2.2	1.7	0 - 4
AC, %	6.0	6.3		5.8	5.8	5.59 – 6.01	6.0	6.0	5.64 - 6.36

There were two differences between the Florida original mix designs. The Florida *good* design used a GTR modified binder and had 5 percent aggregate screenings in the design while the *poor* design used an SBS modified binder and had no screenings. The coarse aggregate sampled for the Florida designs had less than 3 percent passing the No. 8 sieve. It was decided that a small amount of screenings would need to be added to the *poor* design in order to match the gradation of the original *poor* blend; therefore the only real difference in the adjusted blends was the difference in binder content and binder type. Instead of using the 6.0 percent asphalt content stated on the JMF, it was decided to use the same asphalt content as the *good* design (7.1

percent) and to subtract out the amount of GTR (12 percent). This changed the optimum binder content to 6.34 percent. With these changes, the mixtures were compared based solely on the difference in binder type.

Specific gravity values were not provided on the JMF for some of the stockpiles. The specific gravity testing conducted for this project was primarily for the calculation of VMA, VCA and film thickness. The aggregate blend bulk specific gravity (G_{sb}) values used for this study can be found in Table 18.

Table 18 – Bulk Specific Gravity of Aggregate Blends

Mix Design	Aggregate Mineralogy	Blend Bulk Specific Gravity (Gsb)
Georgia	Granite	2.625
South Carolina	Granite	2.615
Florida	Limestone	2.410
Florida	Limestone	2.410
Virginia	Traprock	2.943
New Jersey	Traprock	2.936

RESULTS AND DISCUSSION

Cantabro testing was performed on both conditioned and unconditioned specimens to determine if there was any statistical difference in the results based on the conditioning process. This was a preliminary step to determine if the specimens should be conditioned for the remainder of the experiment. The conditioned specimens were conditioned using the same vacuum saturation and freeze-thaw method as for TSR conditioning (AASHTO T 283).

After conditioning, the specimens were air-dried for several days and then placed in a Core-Dry device to remove any trapped moisture. Specimens were produced at the optimum binder content and also at ± 1.0 percent. The conditioned vs unconditioned comparison was only conducted at the optimum asphalt content. A minimum of 3 specimens were tested for each design point. A two sample t-test was performed on each of the mix designs to determine if the conditioning had any significant effect on the Cantabro loss. The p-value for each of these comparisons (Table 19) showed that, with the exception of the Florida *poor* mixture, there was no statistical difference between the conditioned and unconditioned specimens. The average results for the conditioned and unconditioned specimens (Figure 36) showed that the conditioned specimens had less loss in most cases. This is most likely due to the steric hardening that the binder incurs from the hot water bath. With there being almost no statistical difference between the conditioned and unconditioned samples, all following Cantabro testing was performed on unconditioned specimens.

Table 19 – Cantabro Conditioned vs Unconditioned T-Test Results (α=0.05)

Mix Design	Designation	P-Value	Difference
Georgia	Good	0.426	Insignificant
South Carolina	Poor	0.818	Insignificant
Florida	Good	0.756	Insignificant
Florida	Poor	0.019	Significant
Virginia	Poor	0.126	Insignificant
New Jersey	Good	0.480	Insignificant

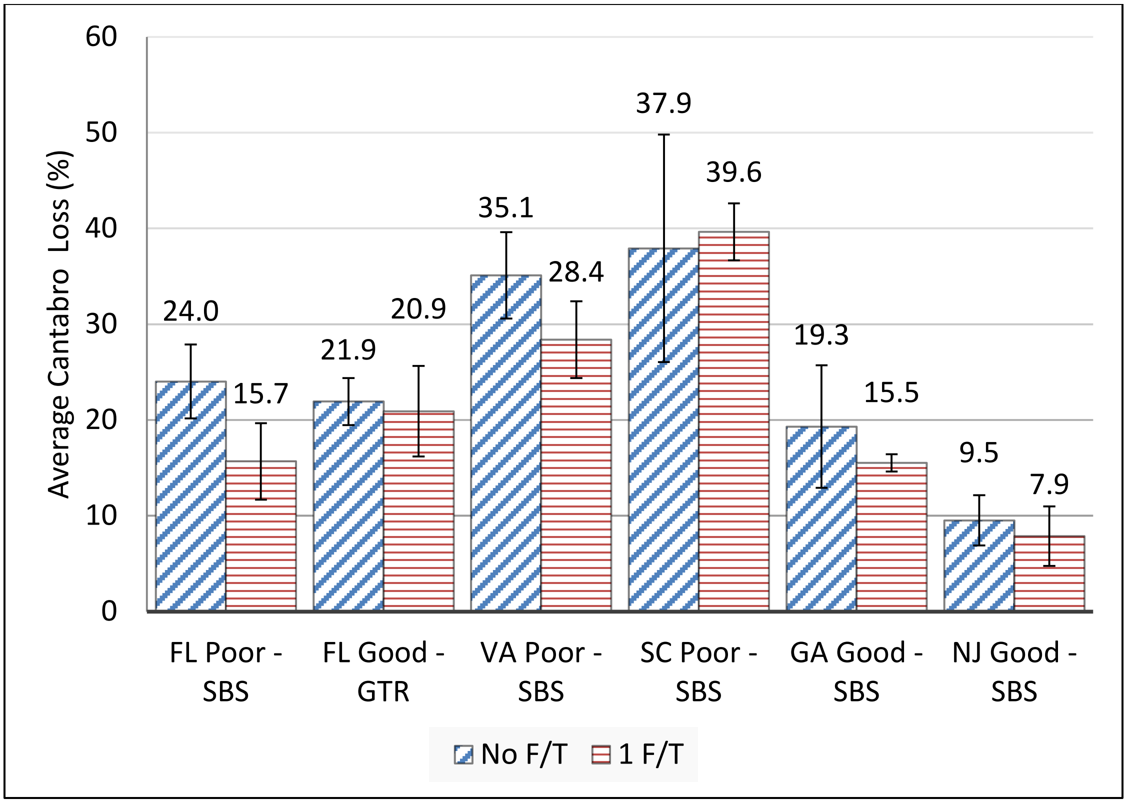


Figure 36 – Part 1 Conditioned vs Unconditioned Cantabro Loss Results

A summary of the average results from all of the unconditioned Cantabro testing can be seen in Table 20 and a graphical depiction of the data can be seen in Figure 37. When considering only the design asphalt contents, the good performing mixes from New Jersey and Georgia mixtures were the only specimens to pass the ASTM recommended 20 percent maximum loss criterion. If the AASHTO criterion (15 percent) is applied, only the New Jersey design passes. A relationship of specimen air voids to Cantabro loss was observed when considering all of the Cantabro data (Figure 38A). An exponential trend line was fitted to the data and a goodness of fit (R^2) value of 0.32 was observed for all of the data; however when the data are separated by NMAS, the trend line provides a much better fit. These results are represented in B), C) and D) of Figure 38.

Table 20 –Summary of Unconditioned Cantabro Results

Mix ID	AC Content (%)	Average Air Voids (%)	Cantabro Loss (%)		
			Average	St Dev	COV (%)
FL Poor - SBS	5.3	19.4	35.4	6.3	17.8
FL Poor - SBS	6.3	17.7	24.0	3.8	16.0
FL Poor - SBS	7.3	16.0	15.2	2.6	16.7
FL Good - GTR	6.1	19.6	38.8	1.9	4.9
FL Good - GTR	7.1	17.1	21.9	2.5	11.2
FL Good - GTR	8.1	15.3	16.4	2.9	17.4
VA Poor - SBS	4.8	23.5	46.9	1.9	4.1
VA Poor - SBS	5.8	21.8	35.1	4.5	12.8
VA Poor - SBS	6.8	18.9	21.6	1.5	7.2
SC Poor - SBS	5.0	23.6	57.3	4.7	8.2
SC Poor - SBS	6.0	22.2	37.9	11.9	31.3
SC Poor - SBS	7.0	20.6	26.8	7.1	26.5
GA Good - SBS	5.0	17.5	25.4	1.9	7.6
GA Good - SBS	6.0	15.7	19.3	6.4	33.4
GA Good - SBS	7.0	12.5	12.8	3.2	24.7
NJ Good - SBS	5.0	21.9	19.7	3.9	20.0
NJ Good - SBS	6.0	19.0	9.5	2.6	27.6
NJ Good - SBS	7.0	17.2	4.3	1.2	27.2

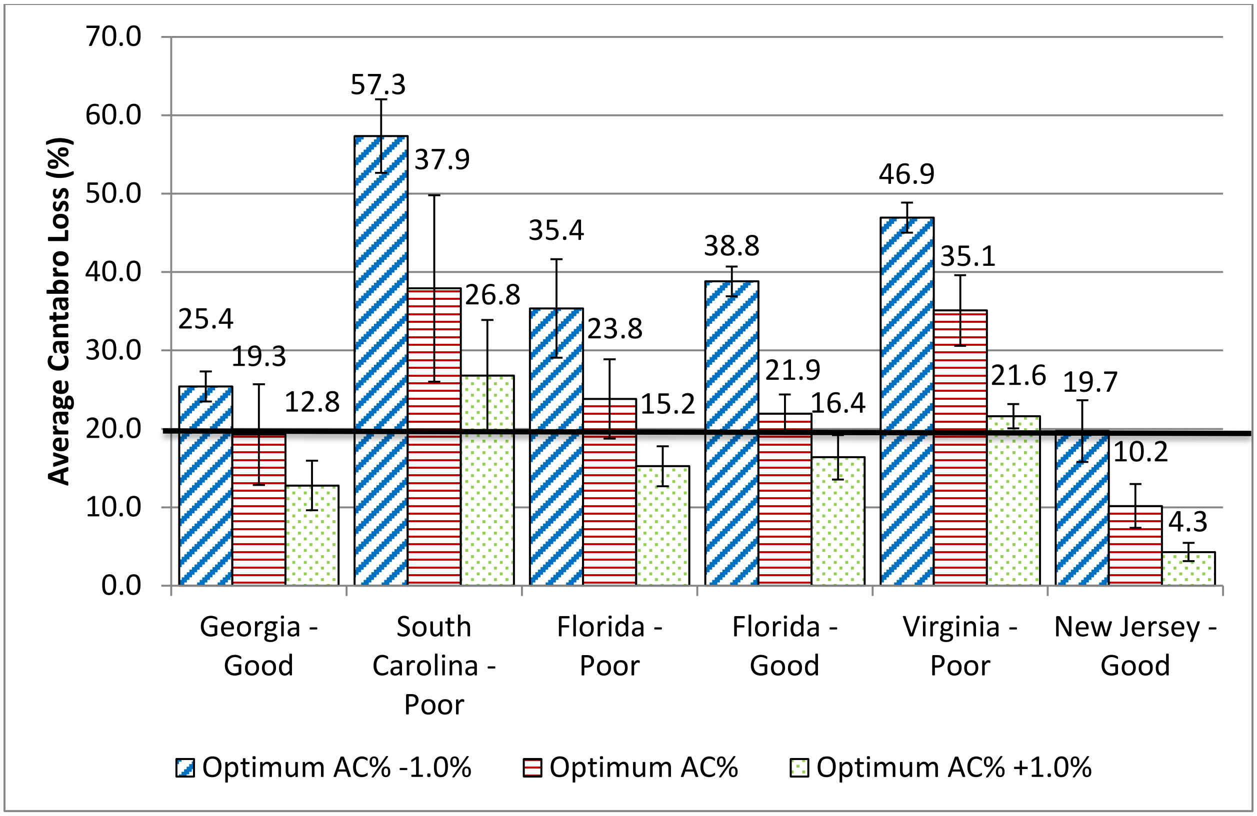
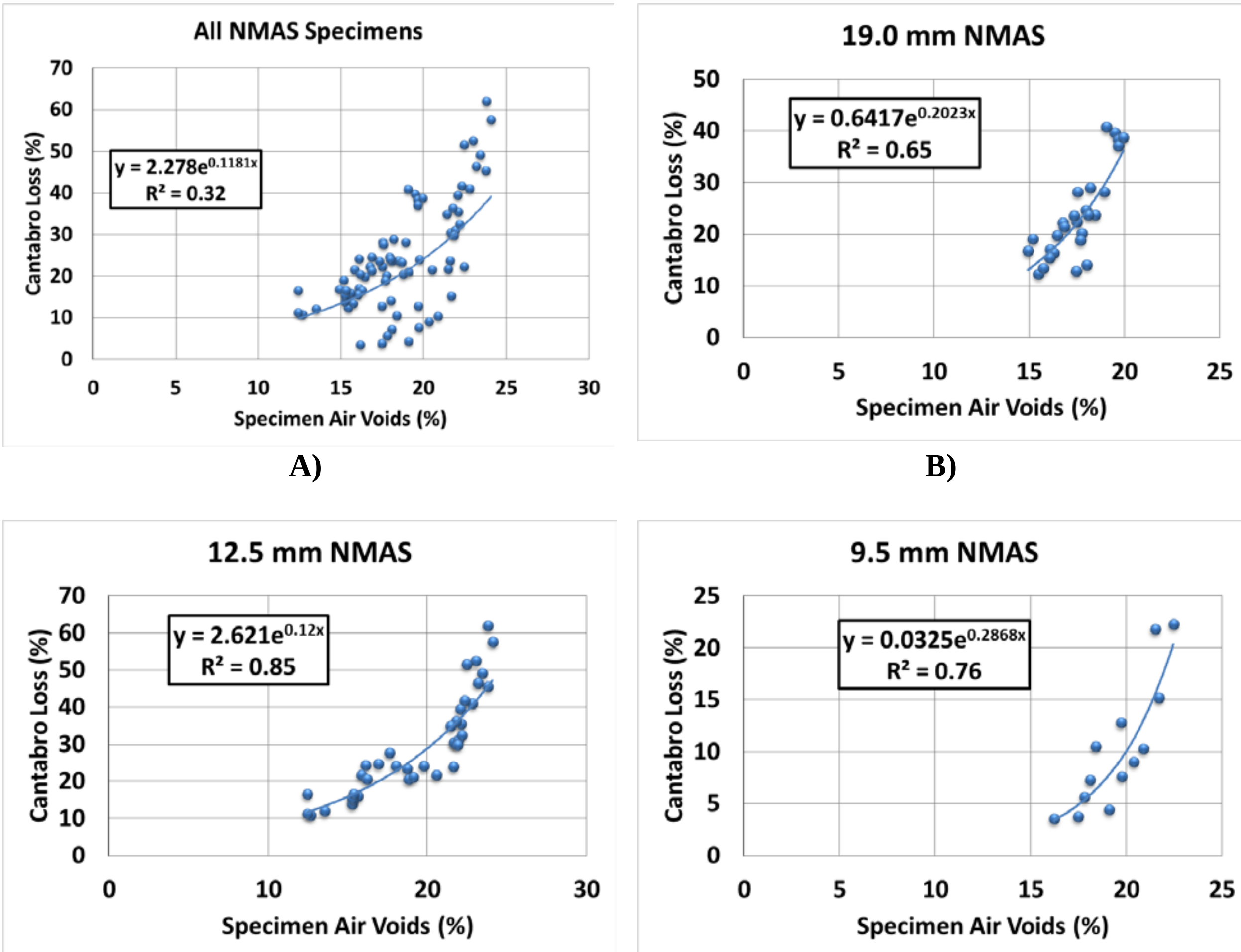


Figure 37 – Unconditioned Cantabro Results for Part 1



C) D)

Figure 38 – Air Voids vs Cantabro Loss Relationship

An ANOVA analysis ($\alpha = 0.05$) was conducted using a Tukey-Kramer grouping to determine whether statistical differences were observed between the mixture designs (Table 21). This was performed using Minitab 16 software. The analysis shows results for Cantabro loss at the optimum asphalt content for each mixture. The Georgia and South Carolina designs (Granite) were not grouped together which shows that they were statistically different designs in regards to percent Cantabro loss. The New Jersey and Virginia designs (Traprock) were also not grouped together showing they were statistically different as well. This is an important point to note since these mixtures share aggregate mineralogies.

Table 21 – ANOVA Analysis - Unconditioned Cantabro Loss at Optimum AC

Mix ID	Cantabro Loss, %		
	N	**Mean**	**Grouping**
South Carolina - Poor	3	37.9	A
Virginia - Poor	3	35.1	A B
Florida - Poor	3	23.8	A B C
Florida - Good	3	21.9	A B C
Georgia - Good	3	19.3	B C
New Jersey - Good	3	10.2	C

The volumetric properties calculated for this study were based on the Cantabro specimens since Cantabro specimens were the only data sets that were tested at multiple asphalt contents. A summary of the volumetric properties can be found in Table 22. Each mixture had extra specimens fabricated at the optimum asphalt content for permeability and conditioned Cantabro testing. Three specimens were fabricated for the other asphalt contents.

Table 22 – Mixture Properties for Part 1

Mix ID	Number of Specimens	Total AC (%)	Avg. Va (%)	Avg. VMA	Avg. VCA_{MIX}/VCA_{DRC}	Avg. Film Thickness (microns)
Florida - Poor	3	5.3	19.4	26.0	1.07	25.5
Florida - Poor	9	6.3	17.7	26.2	1.07	32.5
Florida - Poor	3	7.3	16.0	26.6	1.08	41.0
Florida - Good	3	6.1	19.6	26.8	1.08	28.3
Florida - Good	6	7.1	17.1	26.4	1.07	35.9
Florida - Good	3	8.1	15.3	26.6	1.08	43.7
Virginia - Poor	3	4.8	23.5	32.4	1.17	20.5
Virginia - Poor	6	5.8	21.8	32.9	1.18	25.4
Virginia - Poor	3	6.8	18.9	32.2	1.17	30.5
South Carolina - Poor	3	5.0	23.6	31.9	1.08	28.0
South Carolina - Poor	6	6.0	22.2	32.3	1.09	34.7
South Carolina - Poor	3	7.0	20.6	32.6	1.09	41.2
Georgia - Good	3	5.0	17.5	26.3	1.00	21.9
Georgia - Good	9	6.0	15.4	26.3	1.00	27.1
Georgia - Good	3	7.0	12.5	25.7	0.99	32.4
New Jersey - Good	3	5.0	21.9	31.3	1.28	15.1
New Jersey - Good	7	6.0	19.5	31.1	1.27	18.6
New Jersey - Good	3	7.0	17.2	31.0	1.27	22.2

The Corelok air voids were calculated for every specimen fabricated for this study. The average value for each mixture and its variations were summarized. Air voids from Part 1 of the study can be found in a graphical depiction in Figure 39. All of the mix designs exceeded the anticipated minimum requirement of 15.0 percent air voids. However, both the ASTM and AASHTO specifications recommend a minimum air void content of 18.0 percent. The AASHTO specification also has a maximum limit of 22.0 percent. If these limits are used for design criteria, only the Virginia and New Jersey designs pass. An 18.0 percent design air void target may be more beneficial if it is determined that air void content has a direct correlation to mixture performance.

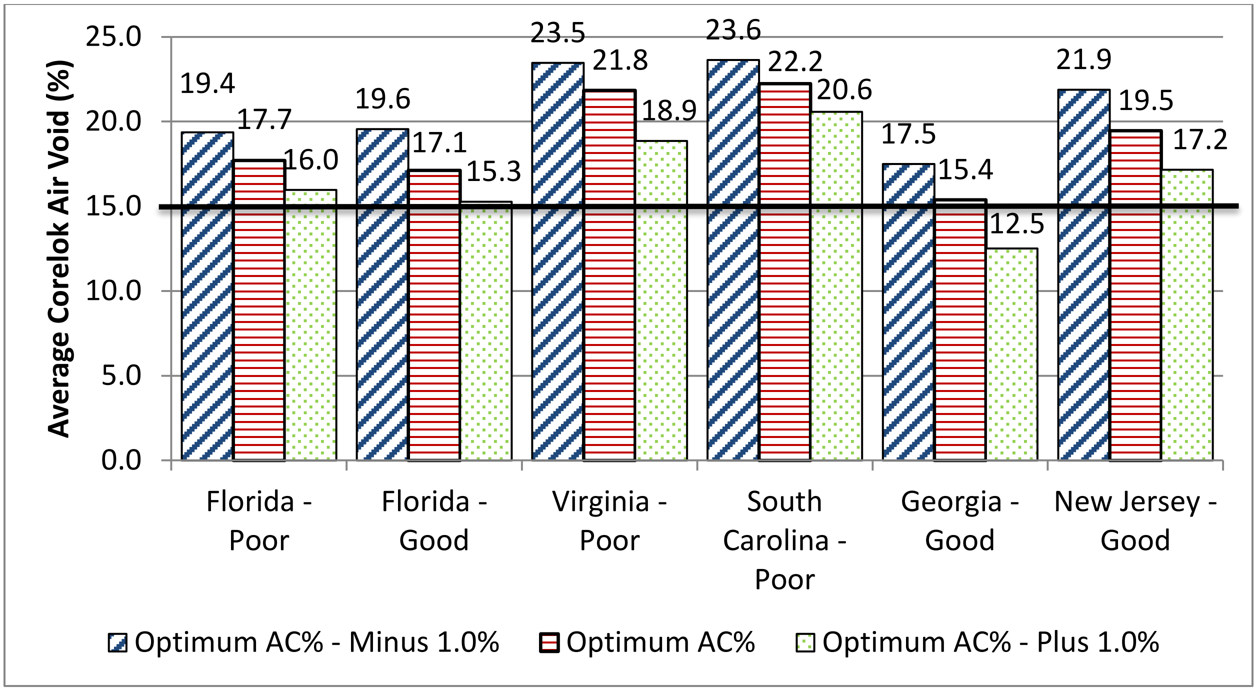


Figure 39 – Part 1 Air Void Content Using the Corelok Method

The original expectations for some of these properties appeared to have minimal effect on the performance of the mixtures. The anticipated minimum film thickness requirement of 24 microns did not correspond to the mixture's field performance. In Figure 40 it can be seen that the New Jersey *good* mix has a film thickness of 18.6 microns, while the South Carolina *poor* mix has a film thickness of 34.7 microns. This may indicate that film thickness is not critical when designing a good performing PFC.

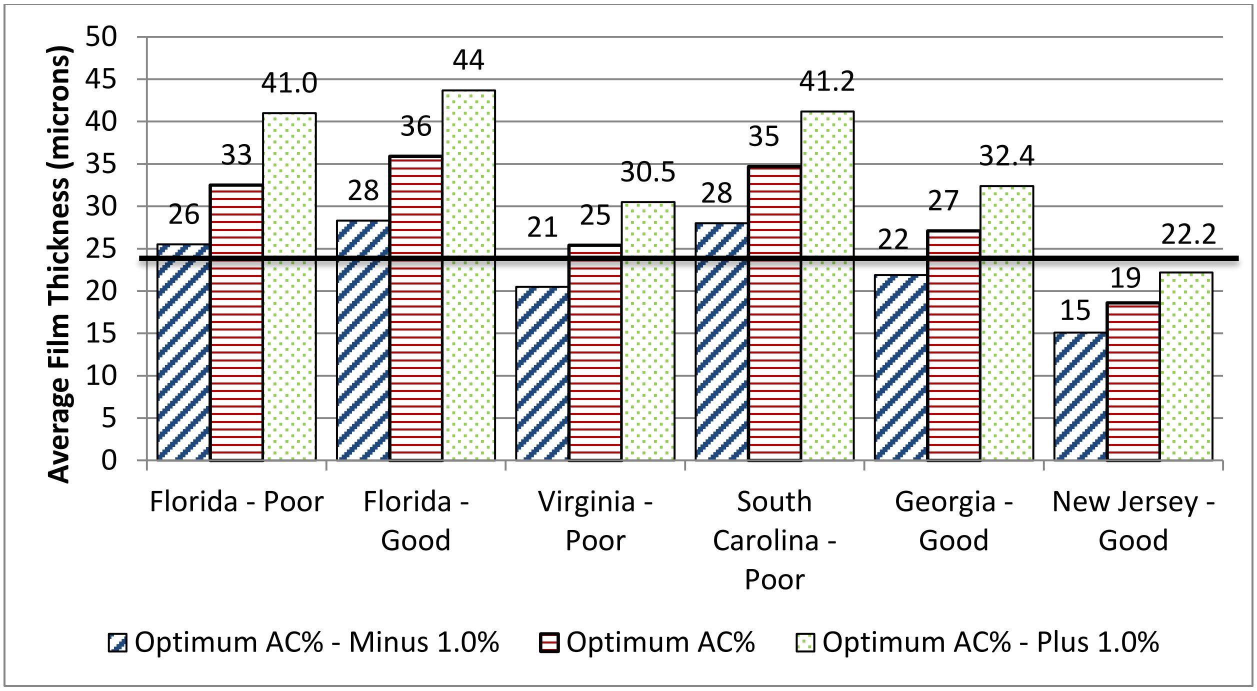


Figure 40 – Part 1 Film Thickness Results

Determining the VCA of the mix and aggregate can help to determine if the design has stone-on-stone contact. The ratio of VCA_{MIX} to VCA_{DRC} (VCA_{MIX}/VCA_{DRC}) should be less than or equal to 1.00 in order for stone-on-stone contact of the coarse aggregate particles to occur. Most agencies do not require this for design although it is specified in both the AASHTO and ASTM OGFC design procedures. It was anticipated that this would be a critical design factor for PFC designs since this should be indicative of the mixture's strength. Of the designs tested for this study, only New Jersey and Virginia require the calculation of VCA for design purposes. The following figure (Figure 41) shows that the mixtures evaluated for this study all had values of 1.00 or greater. As discussed in the Literature Review, if the criteria is in fact $VCA_{MIX} \leq VCA_{DRC}$, and not $VCA_{MIX} < VCA_{DRC}$, then the Georgia mix is the only design to pass the VCA requirement. Since New Jersey and Virginia require the VCA calculation to be incorporated in their designs, the failing VCA ratios seemed to be in error. Further investigation and research was performed to determine if this data were in error or if the mix designs did in fact all have ratios of 1.00 or greater.

The original concept for stone-on-stone contact to create an aggregate skeleton was defined in research conducted for NCHRP Project 9-8, which was research on stone matrix asphalt (SMA). This research was published under NCHRP Report 425 and recommended the VCA concept for use in SMA design. In this report the coarse aggregate in the VCA_{DRC} calculations is defined as the total aggregate blend material retained on the #4 sieve for a 12.5 mm, 19 mm and 25 mm NMAS mixtures. For the 9.5 mm NMAS mixtures the coarse aggregate is defined as the aggregate blend material retained on the 2.36 mm (No. 8) sieve (Brown E. L., 1999). The original VCA_{DRC} calculation for this PFC study was based on ASTM D7064 and AASHTO PP77. ASTM defines the coarse aggregate as material retained on the 4.75 mmm (No.4) sieve and AASHTO does not define the coarse aggregate. In order to determine what effect changing the definition of coarse aggregate would have on the mixture's VCA_{DRC}, the procedure recommended in NCHRP Report 425 was conducted on the New Jersey mixture. Since the New Jersey design is a 9.5 NMAS mix this seemed appropriate. The results provide a decrease in the VCA ratio to a value of 0.92, which passes the recommended criterion. Upon observing what significant affect changing the definition of coarse aggregate had on the VCA results, further investigation was conducted on the VCA test procedure. Watson et al. (2004) conducted research on the VCA technique to ensure that method was suitable for use in OGFC pavements. Digital imaging techniques were used to determine if stone-on-stone contact was occurring in several mix designs. The report concluded that the use of VCA was valid; however there were instances where the VCA ratio (using the No. 4 sieve to define coarse aggregate) was greater than 1.00 but X-ray images showed stone-on-stone contact. For these mixtures, redefining the coarse aggregate by using the No. 8 sieve, in some cases, gave a passing VCA ratio. The recommendation was to determine the critical breakpoint sieve and use that sieve to define the coarse aggregate. The critical breakpoint sieve is defined as the finest sieve size for which at least 10 percent of the total aggregate is retained (Watson D. E., 2004). In NCHRP Report 640, Cooley et al. also defines the coarse aggregate by using the breakpoint sieve. If the breakpoint sieve method is used for this study, all of the designs would define coarse aggregate as material retained on the No. 8 sieve. In order to ensure a comprehensive investigation into the VCA ratio, this method was also performed. These results can be seen in Figure 42. Using the breakpoint sieve method, all of the designs passed the VCA criterion. The breakpoint sieve for all designs was the No. 8 sieve. It was anticipated that these designs would pass due to the fact that the percent coarse aggregate ranged from 83.1 (New Jersey) to 86.5 (South Carolina) percent.

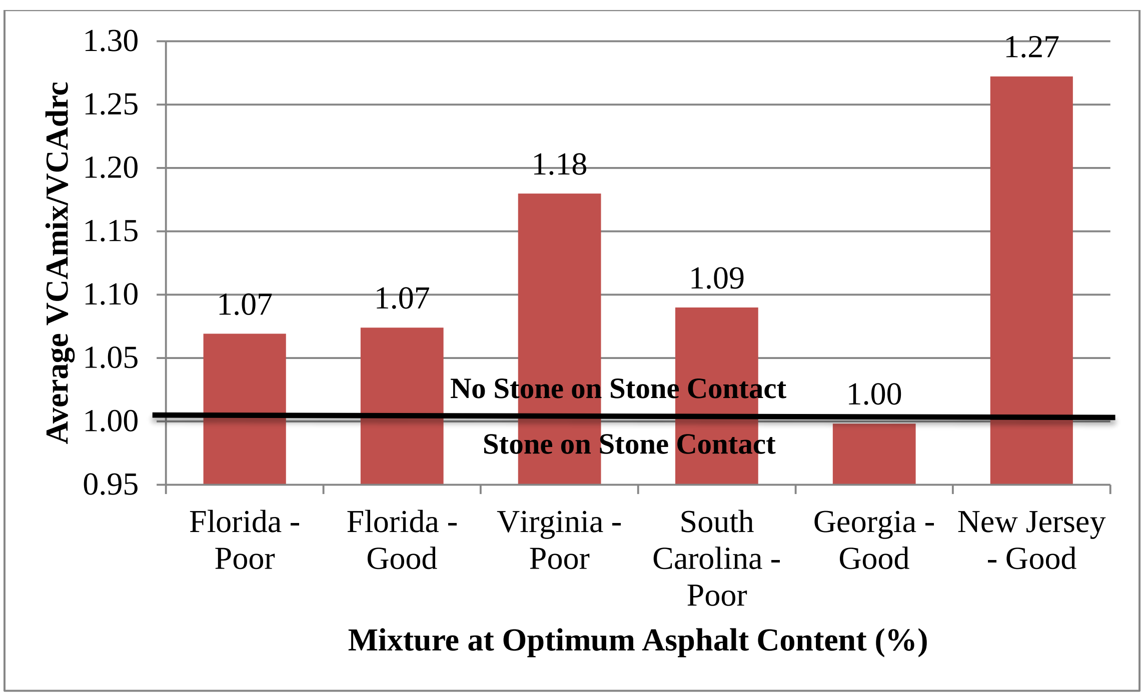


Figure 41 – Part 1 VCA Ratio Using the No. 4 Sieve to Define Coarse Aggregate

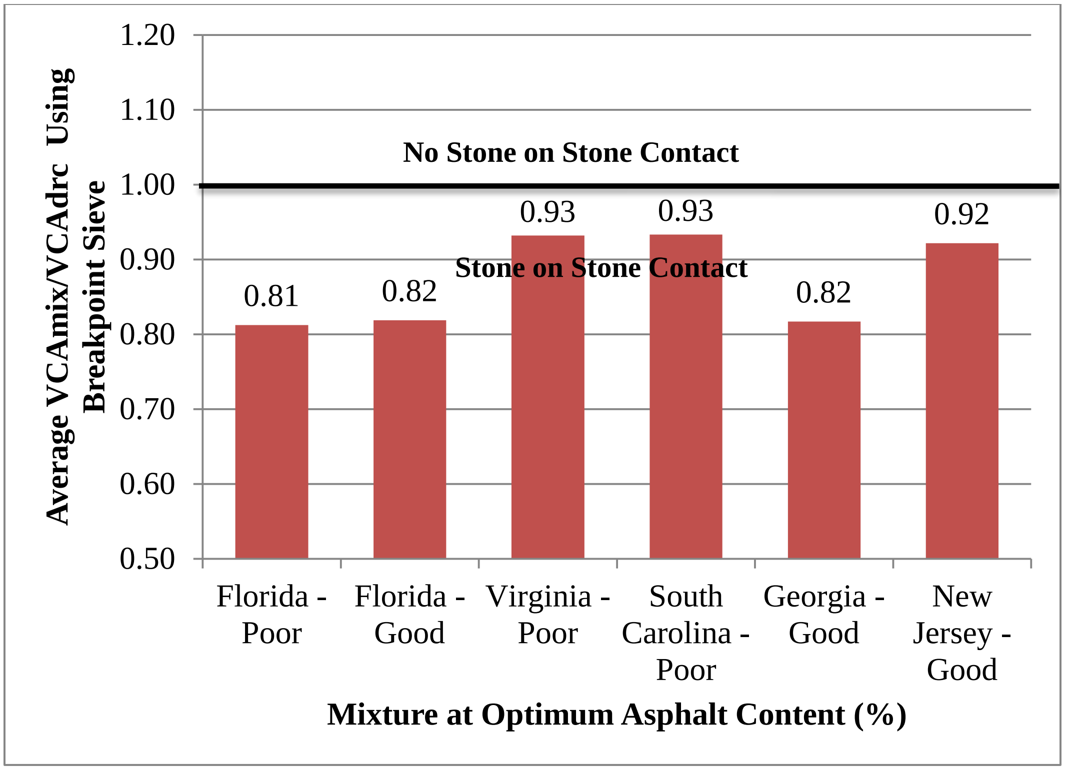


Figure 42 – Part 1 VCA Ratio Using Breakpoint Sieve (No. 8) to Define Coarse Aggregate

The VMA of each mixture changed very little for the varying asphalt contents. Typical VMA data over a range of asphalt contents has a vertical parabolic shape. As can be seen in Figure 43, the "curves" are basically non-existent. As shown previously in Equation 2, the G_{mb} and the percent stone in the mixture are the changing factors. PFC specimens show relative little change in G_{mb} with varying asphalt contents. The reason that differences in air void content are observed is due primarily to the change in G_{mm}. This small change in G_{mb} is sometimes so small that there is no noticeable change between asphalt contents. In Figure 43 and Table 22 it can be seen that the Virginia mix has a higher VMA at the optimum asphalt content than it does at the other asphalt contents. This is due to the average G_{mb} of the optimum asphalt content (5.8%) showing little change from the lower asphalt content (4.8%). This variability is not unexpected when fabricating PFC specimens. A change in VMA of less than 1.0 percent for a design shows no relative change to the mixture. This seems to indicate that PFC mixes are not sensitive to change in asphalt content in regards to VMA.

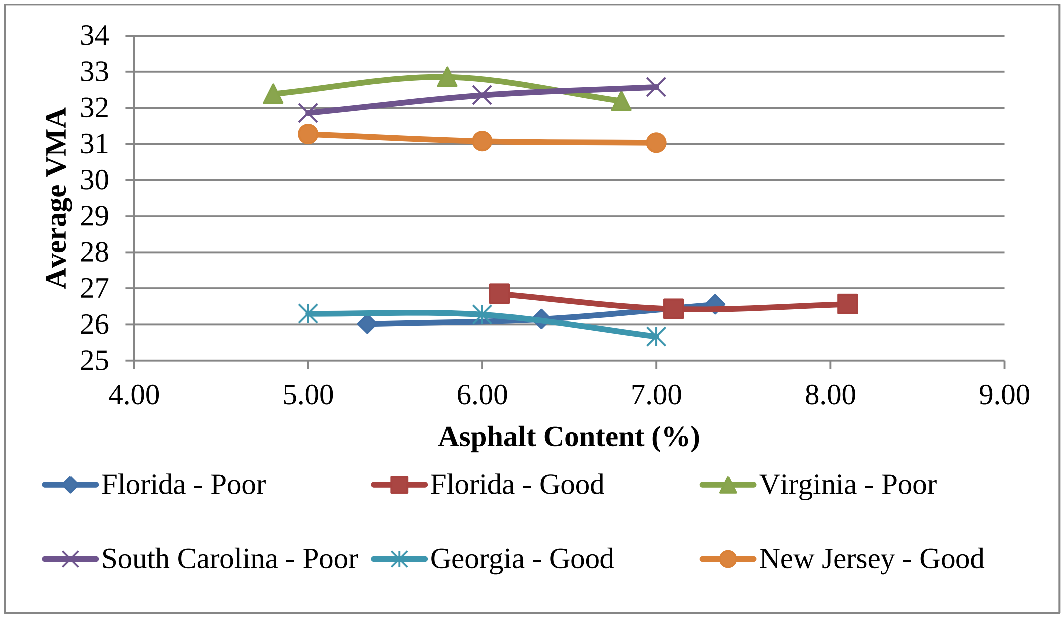


Figure 43 – VMA Curves for Part 1 Design Data

The permeability testing showed a direct correlation between the permeability and air void content of the specimens (Figure 44). A summary of the data (Table 23) shows that two of the *poor* mixtures have higher air voids and corresponding higher permeability values. The New Jersey design has good performance, a high air void content and a high permeability rate. This may be due to the NMAS of the mixture. The design requirements in the ASTM and AASHTO specifications state that the mixtures must have a minimum air void content of 18.0 percent. In Figure 44 the optimum air void content associated with the recommended permeability rate of 100 meters/day is close to 17.0 percent. Two of the *good* designs (Georgia and Florida GTR) had a permeability rate lower than the recommended 100 meters/day criterion. Since these mixtures had such good performance in the field, the recommended permeability rate may need to be re-evaluated. With Mississippi specifying as low as 35 meters/day, a compromise of 50 – 60 meters/day may be a better criterion. The lowest average rate recorded for this part of the study was 77 meters/day. However, both Georgia and Florida good performing mixes had individual permeability values as low as 69 m/day. Based on this information, and a minimum recommended air voids of 15 percent, a minimum permeability value of 50 m/day is recommended.

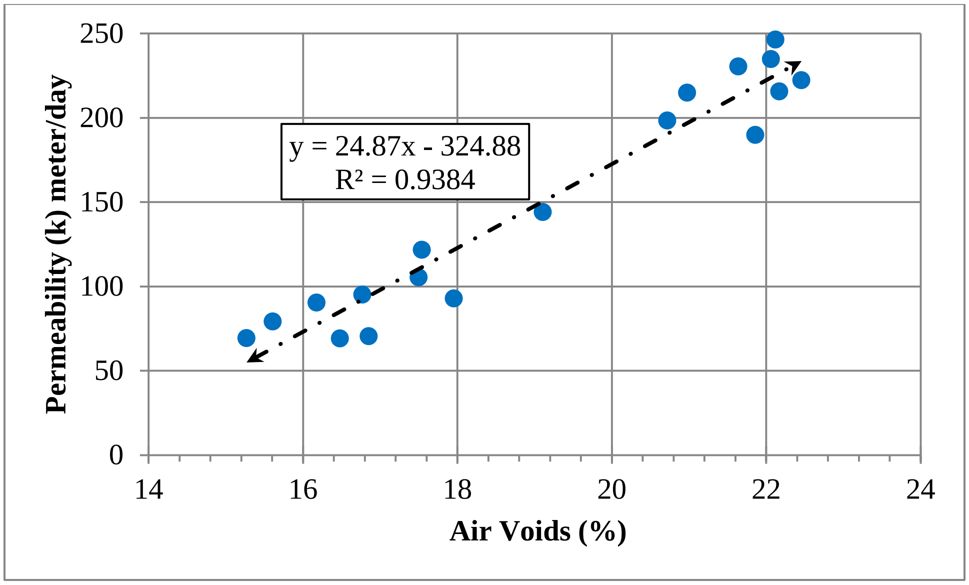


Figure 44 – Part 1 - Permeability to Air Void Correlation

Table 23 – Part 1 Permeability Data Summary

Mix ID	Total AC (%)	Fiber (%)	Average Air Voids (%)	Permeability (k) meters/day		
				Average	St Dev	COV(%)
Georgia -Good	6.0	0.4	15.7	80	10.5	13.1
Florida - Good	7.1	0.4	17.1	77	13.3	17.2
New Jersey -Good	6.0	0.3	20.3	186	37.0	19.9
South Carolina - Poor	6.0	0.3	22.2	209	17.1	8.2
Florida -Poor	6.3	0.4	17.3	107	13.4	12.4
Virginia SBS	5.8	0.3	21.9	237	8.2	3.5

The draindown testing was performed on all of the JMF designs using a 2.36mm (#8) mesh basket. All of the designs used a final grade PG 76-22 binder. The mixing temperature range for this binder was 320-330°F. In order to assure that draindown was not going to occur, the higher end of the range (330°F) was used for the mixing temperature (lower test temperature). The amount of fiber content varied between 0.3 and 0.4 percent for this testing based on the mix design provided by the agency. The results in Table 24 show that no draindown occurred for any of the designs.

Table 24 – Part 1 Draindown Results Using a 2.36mm (#8) Mesh Basket

Mix ID	Total AC (%)	Fiber (%)	Draindown (%)	
			Test Temp, °F	
			330	357
FL Poor - SBS	6.3	0.4	0.0	0.0
FL Good - GTR	7.1	0.4	0.0	0.0
VA Poor - SBS	5.8	0.3	0.0	0.0
SC Poor - SBS	6.0	0.3	0.0	0.0
GA Good - SBS	6.0	0.4	0.0	0.0
NJ Good - SBS	6.0	0.3	0.0	0.0

The HWTT data for this testing proved to be difficult to analyze. The mixtures had a wide range in terms of performance and made it impractical to perform statistical analysis on the data. The simplest and most comprehensive way to explain the performance of these specimens was through a graphical depiction (Figure 45). The following figure shows that most of the mixtures performed reasonably well, with the exception of South Carolina's granite mix design. There were 3 sets of data (6 specimens) fabricated for each mix design. The rut depth of the specimens was recorded every 200 passes and 3 sets of data were averaged to form the graphs depicted in Figure 45.

Table 25 shows the variability in the 3 sets of data as well as the pertinent mix design information. It should be noted that while the “Greatest Rut Depth Recorded” is being reported this is a slight misnomer. The HWTT machine records rut data until the LVDT reaches its maximum limit. The South Carolina design reached this limit early on in the testing. The specimens failed so quickly that the maximum rut depth was reached prior to the machine reaching 4,000 passes. So while the average maximum rut depth recorded for the South Carolina mix is reported as 15.85 mm, it is not wholly representative of the sample’s performance because that value does not show how poorly the specimens performed. The graphs along with the accompanying data in the table provide a comprehensive view of the HWTT.

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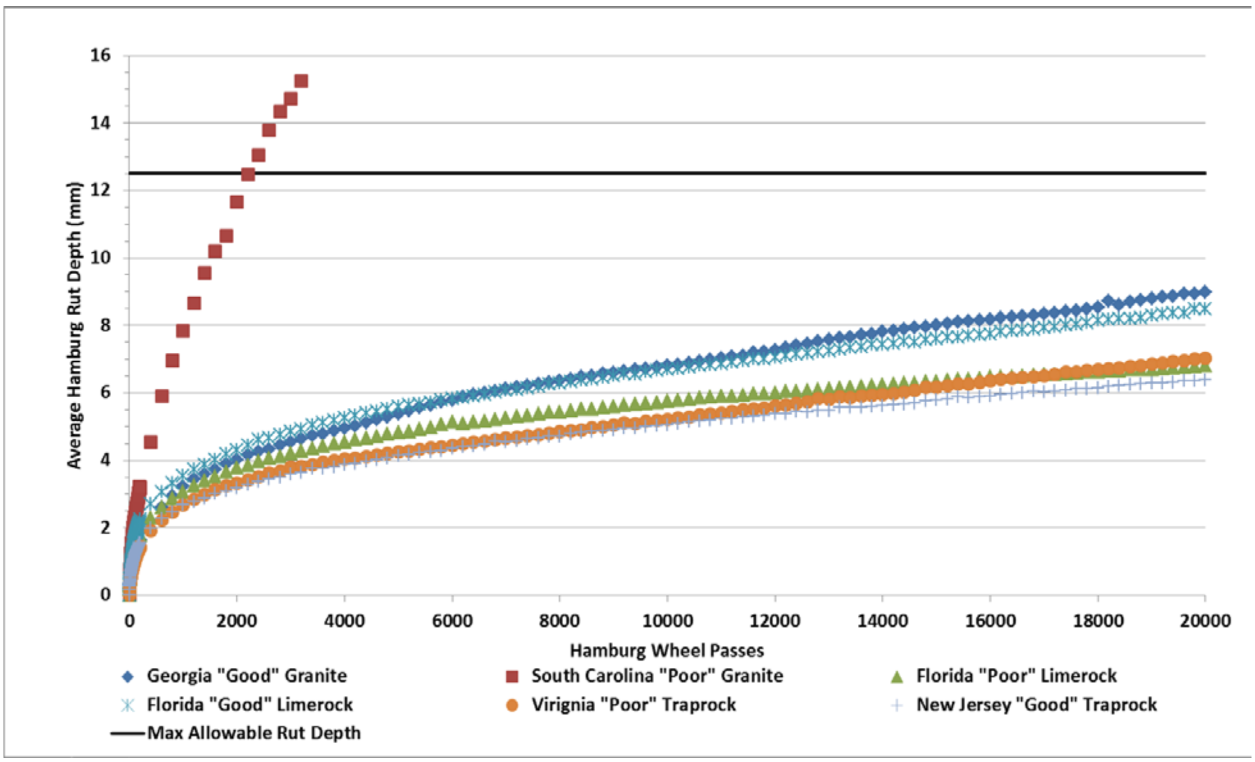


Figure 45 – Part 1 – Hamburg Wheel-Track Test Results

Table 25 – Part 1 – HWTT Summary

Mix ID	Total AC (%)	Total P-200 (%)	Fiber (%)	Avg. Air Voids (%)	Greatest Rut Depth Recorded (mm)		
					Average, mm	St Dev, mm	COV(%)
Georgia -Good	6.0	2.0	0.4	14.3	8.99	2.88	32.0
South Carolina -Poor	6.0	1.7	0.3	21.7	15.85	1.84	11.6
Florida - Poor	6.3	0.9	0.4	17.9	6.81	0.26	3.8
Florida - Good	7.1	0.9	0.4	17.2	8.47	1.04	12.2
Virginia - Poor	5.8	2.6	0.3	22.2	7.04	0.51	7.2
New Jersey - Good	6.0	3.9	0.3	19.7	6.39	1.17	18.3

The moisture susceptibility testing for Part 1 provided both indirect tensile strengths (ITS) of the conditioned and unconditioned specimens along with the TSR for each mixture. All of the designs met the expected minimum criterion of 0.70. The ASTM specification requires a TSR of 0.80 or better. If the ASTM criterion was applied to these designs, the Florida *poor* and the Georgia *good* mixtures would fail. This could have been due to the change in type or amount of anti-stripping agent used. The Georgia mix was one of the designs that originally had hydrated lime but was removed for this study. Even though these two mixes failed the TSR according to ASTM, they have 2 of the highest unconditioned ITSs. The Georgia design had the second highest conditioned ITS for this part of the study. The ITS of the mixtures is essential because it directly relates to the asphalt's potential for cracking. So while it is important to note that 2 of the mixtures failed TSR according to one specification, it is also equally important to take into account the unconditioned ITS. The TSR can be improved most of the time by increasing the amount of liquid anti-strip or by switching to hydrated lime, but the ITS is based on the mixture properties and the corresponding adhesion of mixture components.

Table 26 – Part 1 – Moisture Susceptibility Testing Summary

Mix ID	Total AC (%)	Fiber (%)	Avg. Specimen Air Voids (%)		Avg. ITS (psi)		TSR
			Conditioned	Unconditioned	Conditioned	Unconditioned	
FL - Poor	6.3	0.4	17.2	21.0	52.8	72.5	0.73
FL - Good	7.1	0.4	17.6	15.2	54.0	50.1	1.08
VA - Poor	5.8	0.3	20.8	18.2	53.2	59.5	0.89
SC - Poor	6.0	0.3	21.2	21.2	36.7	45.2	0.81
GA - Good	6.0	0.4	13.9	14.0	57.7	74.3	0.78
NJ - Good	6.0	0.3	18.2	18.2	64.5	76.2	0.85

An ANOVA analysis was conducted to determine if any of the mixtures were significantly different. Tukey-Kramer statistical groupings were also included in the analysis to determine which mixtures were significantly different on a mix by mix basis. Table 27 shows the results of the analysis based on the ITS of each mixture. The data were grouped into two separate sets for the analysis: Conditioned and Unconditioned. Means that do not share a letter are significantly different. When looking at the unconditioned strengths, it can be seen that there is a large gap between the mean strength of the mixtures. The New Jersey, Georgia and Florida-*poor* designs have strengths greater than 70 psi while the Virginia, Florida-*good* and South Carolina designs have strengths of less than 60 psi. This distinct split between the unconditioned ITSs seems to indicate the need for a specified minimum ITS in the design procedure. A minimum value of 70 psi seems to correlate well to the good performing mixtures. The Florida *poor* and *good* designs appear to be swapped, when analyzing these results based on field performance; however the only difference between these designs was the binder modifier and total asphalt content. The GTR design had an asphalt content of 7.1 percent which may have had a lubricating effect on the sample when it was broken in indirect tension.

Table 27 – Part 1 – ANOVA Statistical Comparisons of ITS (α=0.05)

Mix ID	Conditioned			Unconditioned		
	N	**Mean**	**Grouping**	**N**	**Mean**	**Grouping**
New Jersey - Good	3	64.5	A	3	76.2	A
Georgia - Good	3	57.7	A B	3	74.3	A B
Florida - Poor	3	54.0	B	3	72.5	A B
Virginia - Poor	3	53.2	B	3	59.5	B C
Florida - Good	3	52.8	B	3	50.1	C
South Carolina - Poor	3	36.7	C	3	45.2	C

When running the OT test, results are normally extremely variable. There were a few results in the OT testing for this study that seemed to be outliers, so the use of ASTM E178, *Standard Practice for Dealing with Outlying Observations*, was implemented. The standard is used to test the statistical significance of the results from a study to determine if an "outlier" is present within the data set. The equation (Equation 9) uses the average, standard deviation and the number of observations to compare to a confidence interval value. The confidence interval value is provided in a table in the standard. A one-sided test with a confidence interval of 90 percent was chosen for this evaluation.

$$T_n = \frac{(X_n - \bar{X})}{S} \qquad \text{Equation 9}$$

Where:

T_n = Test criterion
X_n =Anticipated Outlier
$\bar{X}$ = Arithmetic average of all *n* values
S = Estimate of the population standard deviation based on the sample data

There was a single outlier in 3 of the specimen sets. Since there were at least 4 samples tested per mix design, this left at least 3 specimens so that statistical analysis could be performed. The cycles to failure was based on a 93 percent load reduction (Tx-248-f). The test terminates once the specimens reach a 93 percent load reduction from the peak load or 1,000 cycles. Some of the specimens went the full 1,000 cycles and never reached the 93 percent reduction prior to the test terminating. For these specimens the data was extrapolated to determine the number of cycles it would have taken to reach to the 93 percent load reduction. A summary of the data sets and their properties can be found in Table 28, while a figure depicting the cycles to failure can be found in Figure 46. The coefficient of variance (COV) for OT testing is normally extremely high (approximately 50 percent). The average COV for this testing was 20.5 percent, which indicates that these data results show less variability than typical OT specimens. This may be due in part to the ASTM outlier specification being implemented.

Table 28 – Part 1 – Overlay Tester Summary

Mix ID	*Replicates*	*Average Air Voids (%)*	*Average Peak Load (kN)*	*Cycles to Failure*		
				Average	*St Dev*	*COV (%)*
FL - Poor	3	18.7	2.093	370	57	15.4
FL - Good	3	17.8	1.731	67	2	2.3
VA - Poor	4	19.2	1.818	1,291	431	33.4
SC - Poor	6	19.2	1.798	1,491	388	26.0
GA - Good	3	12.8	2.621	583	166	28.4
NJ - Good	4	18.5	1.993	1,866	324	17.3

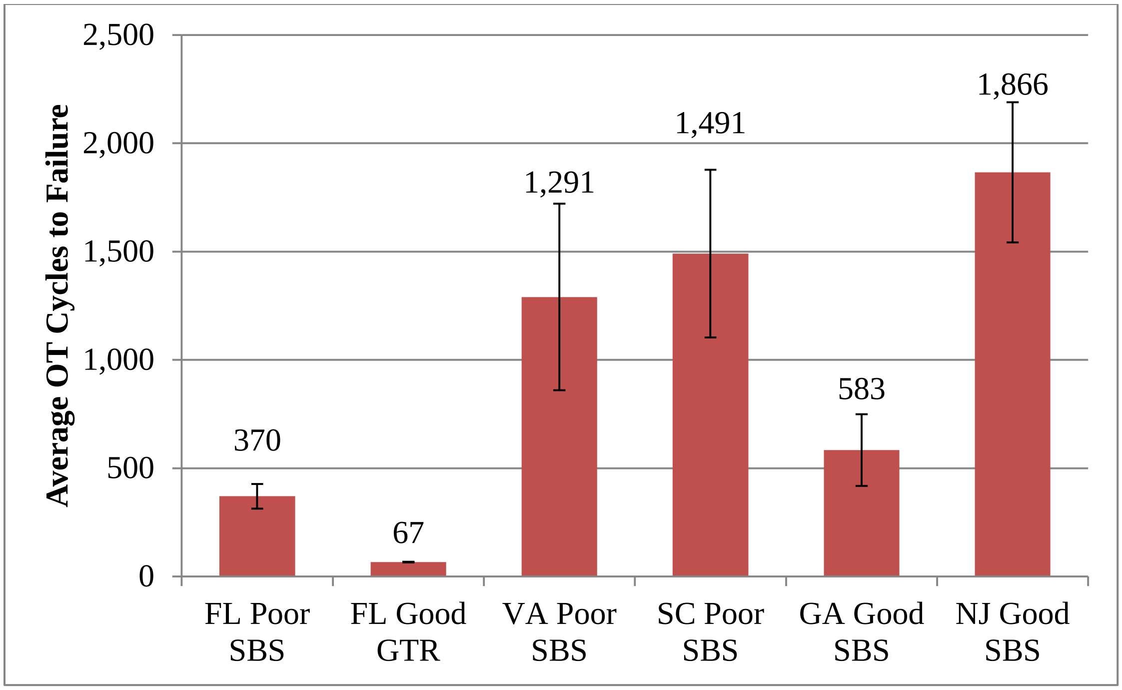


Figure 46 – Part 1 Overlay Tester Results

Using a one-way ANOVA analysis (α=0.05), it was determined that the mixtures were statistically different. The grouping according to the Tukey-Kramer method can be found in Table 29. The model fit was good (R^2=84.22%) which is most likely due to the COV being low for this testing. OT testing normally does not provide a good model fit. A clear difference between the mixtures can be observed when looking at the means. The New Jersey, South Carolina and Virginia designs all had to be extrapolated because they exceeded the 1,000 cycle test limit. This can be seen in the groupings as well, since they are grouped separately from the remaining 3 designs. The Florida *good* design performed extremely poorly (mean cycle to failure of 67). It is not certain why this was the case especially when this mixture performed well in the field.

Table 29 – Part 1 – ANOVA Statistical Comparisons of OT Results (α=0.05)

Mix ID	*N*	*Mean*	*Grouping*		
New Jersey - Good	4	1866	A		
South Carolina - Poor	6	1491	A		
Virginia - Poor	4	1291	A	B	
Georgia - Good	3	583		B	C
Florida - Poor	3	370			C
Florida - Good	3	67			C

$p<0.001$
$R^2 = 84\%$

The SCB test performed according to the Illinois Flexibility Index Test (I-FIT) procedure was used to determine the mixture susceptibility to intermediate temperature cracking. While most dense-graded mixtures have a FI ranging from 0 to 20, the PFC mixtures were much larger due to the large slope, post-peak. Since most of the testing performed to-date has been primarily on dense-graded mixtures with varying amounts of asphalt binder replacement, the use of the FI to distinguish the *good* and *poor* mixes for this project will be subjective. The peak load, G_f, and FI were all analyzed in order to see which property would provide the best model for distinguishing the mixtures. Figure 47through Figure 49 show a comparison of the mixtures based on peak load, G_f and FI. The determination of possible outliers was also performed for this part of the testing: and based on the results, three specimens were determined to be outliers and were removed from the test data (1 specimen each from Georgia, Florida-Good and New Jersey) prior to performing any analysis.

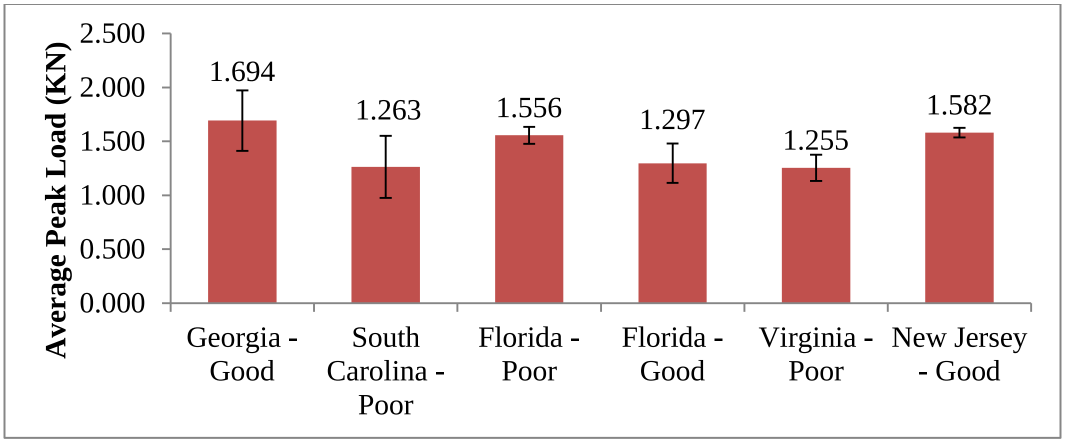


Figure 47 – Part 1 I-FIT – Average Peak Load

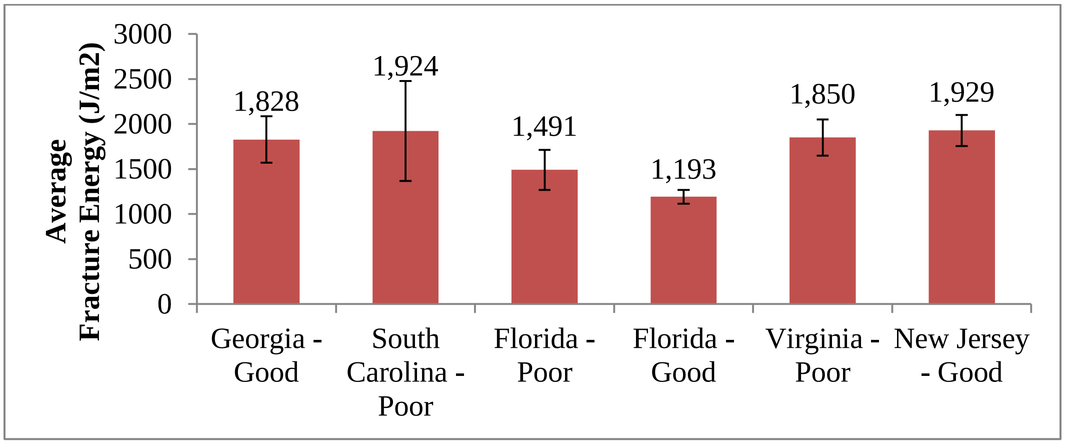


Figure 48 – Part 1 I-FIT – Average Fracture Energy (G_f)

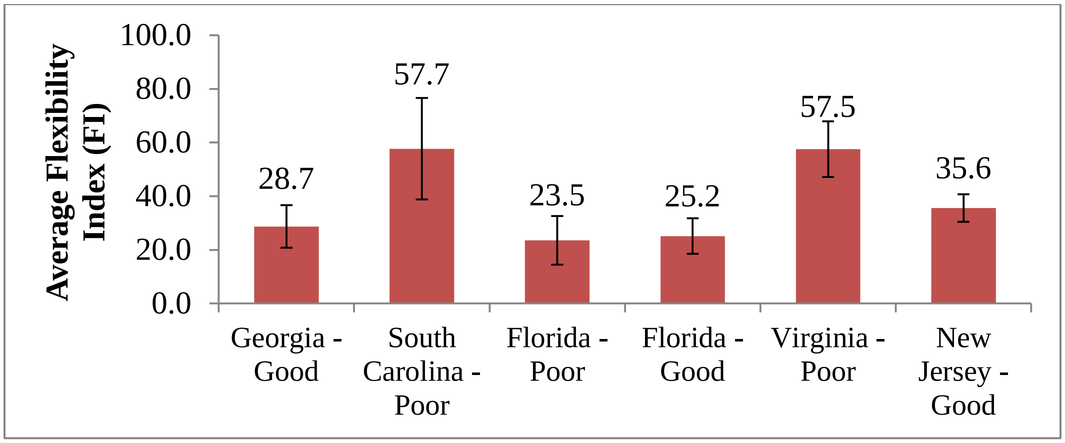


Figure 49 – Part 1 I-FIT – Average Flexibility Index (FI)

Using the Minitab software, an ANOVA statistical analysis with Tukey-Kramer grouping was performed on the peak load, G_f and FI. While all of the analyses showed significant differences between the mixtures, the FI analysis provided the best fit with an R^2 = 71.41%. The South Carolina and Virginia designs were statistically different from the other designs. Since FI is an indication of the mixture's resistance to cracking, these results were not unexpected. Both of these designs also performed well in regards to the OT testing. In terms of G_f, both of the Florida designs were grouped together. The G_f of these specimens was significantly less than that of the other mixtures tested.

Table 30 – Part 1 I-FIT ANOVA Analysis for Peak Load

Mix ID	Peak Load (KN)			
	N	Mean	Grouping	
Georgia - Good	5	1.694	A	
New Jersey - Good	5	1.582	A	B
Florida - Poor	6	1.556	A	B
Florida - Good	5	1.297		B
South Carolina - Poor	4	1.263		B
Virginia - Poor	6	1.255		B

p =0.001
R^2 = 54%

Table 31 – Part 1 I-FIT ANOVA Analysis for Fracture Energy

Mix ID	Fracture Energy (J/m^2)			
	N	Mean	Grouping	
New Jersey - Good	5	1929	A	
South Carolina - Poor	4	1924	A	
Virginia - Poor	6	1850	A	
Georgia - Good	5	1828	A	
Florida - Poor	6	1491	A	B
Florida - Good	5	1193		B

p =0.001
R^2 = 56%

Table 32 – Part 1 I-FIT ANOVA Analysis for Flexibility Index

Mix ID	Flexibility Index		
	N	Mean	Grouping
South Carolina - Poor	4	57.7	A
Virginia - Poor	6	57.5	A
New Jersey - Good	5	35.6	B
Georgia - Good	5	28.7	B
Florida - Good	5	25.2	B
Florida - Poor	6	23.5	B

$p<0.001$
$R^2 = 71\%$

The Wet Track Abrasion Test was originally performed according to the ISSA TB-100 test procedure and the specimens were submerged in water and tested for 5.25 minutes. This produced no visible wear to the specimen so the testing time was increased to 30 minutes. After the 30 minute test there was still no visible wear on the specimen; and after drying the specimen and reweighing it, it was noted that there was still no abrasion loss (Figure 50). The rubber hose had begun to abrade during the 30 minute test cycle so it was decided that this procedure would not be valid for determining the durability of the PFC designs. The purpose of this test was to determine the cohesiveness of the mixtures by abrading them with a rubber hose. An alternative test was chosen to replace the Wet Track Abrasion test. An experiment using a modified version of the I-FIT cracking test was conducted to measure the amount of shear force needed to break the cohesive bond of the mixture.

Sample 146 - Prior to Testing **Sample 146 – After 30 Minutes of Abrasion**

Figure 50 – Wet Track Abrasion Sample Showing No Abrasion for PFC Mixture

The I-FIT specimen was fabricated according to the test procedure, but the notch was not cut into the specimen. This allowed the crack to form at the path of least resistance. The data analysis method was the same as the original I-FIT procedure; however since it is a modification to the procedure, the normal criteria may not apply. This was solely used to try and differentiate the *good* from the *poor* mixtures. There were 3 outliers removed from the data prior to performing the analysis (1 from Georgia, Virginia and Florida poor). The average peak load (Figure 51) recorded for this test shows a difference in the mixtures. When looking at the results of the ANOVA analysis (Table 33) it can be seen that the Georgia, Florida-*poor* and the New Jersey mixtures are significantly different from the other mixtures. The model fit is good (82.53%) and there is a distinct numerical separation of the means. Virginia and South Carolina are statistically different designs while Florida *good* was not statistically different from either. A possible minimum peak load of 2.750 KN may be able to differentiate *good* from *poor* mixtures.

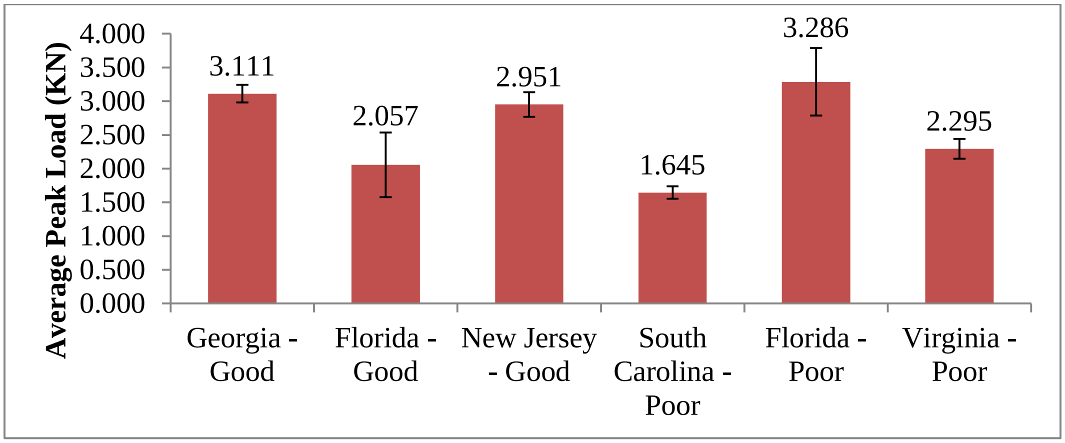


Figure 51 – Part 1 I-FIT No-Notch Peak Load Chart

Table 33 – ANOVA Analysis for Peak Load of Part 1 No-Notch I-FIT Specimens

Mix ID	**Peak Load**		
	N	**Mean**	**Grouping**
Florida - Poor	5	3.286	A
Georgia - Good	5	3.111	A
New Jersey - Good	6	2.951	A
Virginia - Poor	5	2.295	B
Florida - Good	6	2.057	B C
South Carolina - Poor	6	1.645	C

p<0.001
R^2 = 83%

The G_f of the designs did not provide as much separation of the mixtures as the peak load, but the Florida *good* design was statistically different form all of the other designs. The New Jersey design with a G_f of 3871 J/m^2 was statistically different from the South Carolina and Florida *good* designs. The results of this testing can be found in Figure 52 with the corresponding analysis in the following table (Table 34).

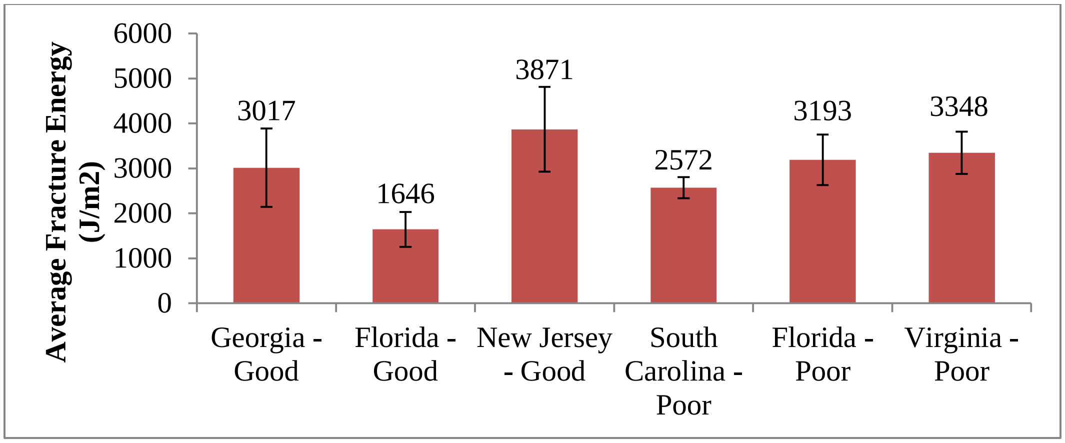


Figure 52 – Part 1 No-Notch I-FIT Fracture Energy Chart

Table 34 – ANOVA Analysis for Fracture Energy of Part 1 No-Notch I-FIT Specimens

Mix ID	Fracture Energy		
	N	Mean	Grouping
New Jersey - Good	6	3871	A
Virginia - Poor	5	3348	A B
Florida - Poor	5	3193	A B
Georgia - Good	5	3017	A B
South Carolina - Poor	6	2572	B C
Florida - Good	6	1646	C

p<*0.001*
R^2 = 62%

The FI of the designs trended the same as the notched I-FIT specimens. The results (Figure 53) showed that South Carolina and Virginia still had the highest FI while the Florida designs had the lowest. Georgia and both of the Florida designs were statistically different from the other designs, while South Carolina was statistically different from all of the designs (Table 35).

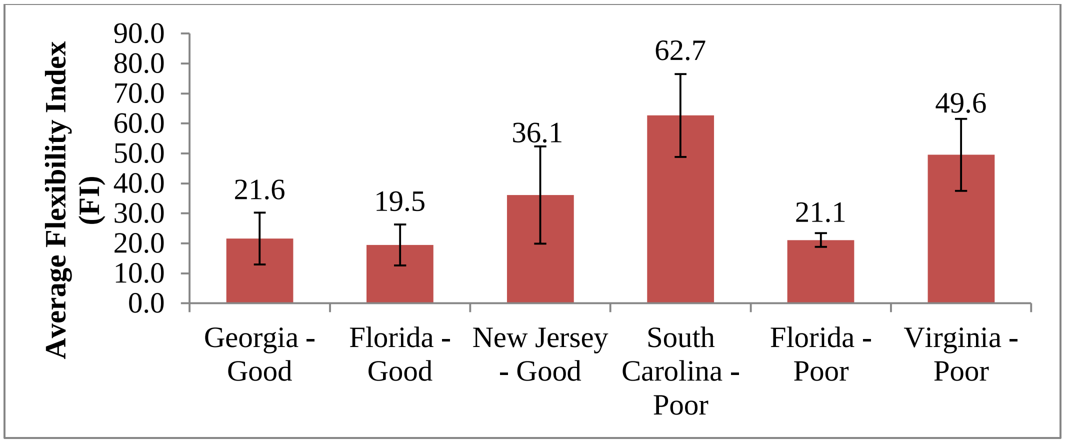


Figure 53 – Part 1 No-Notch I-FIT Flexibility Index Chart

Table 35 – ANOVA Analysis for Flexibility Index of Part 1 No-Notch I-FIT Specimens

Mix ID	Flexibility Index				
	N	Mean	Grouping		
South Carolina - Poor	6	62.7	A		
Virginia - Poor	5	49.6	A	B	
New Jersey - Good	6	36.1		B	C
Georgia - Good	5	21.6			C
Florida - Poor	5	21.1			C
Florida - Good	6	19.5			C

p<0.001
R^2 = 72%

A comparison of the notch and no-notch I-FIT data was conducted to determine if the no-notch modification was a valid method for determining cohesion of the mixture or if the results were not significantly different from the notched data. The peak load, G_f and FI were analyzed for each mix design and the notch and no-notch results were compared using a t-test. Equal variance was assumed and an α of 0.05 was used for a confidence interval of 95 percent. Graphical comparisons of these properties can be seen in Figure 54 through Figure 56 and a summary table with the results (p-value) of the t-tests can be found in

Table 36. All of the designs when analyzed for peak load were significantly different. All of the designs except Florida *poor* were also significantly different for the G_f. The Florida *good* mix was close (0.059) which is suggestive of it being statistically different but conclusive results most likely cannot be drawn from this value. All of the FI comparisons show that none of the designs are affected by the notch in the specimens. The FI data for each group are not statistically different. Since peak load and almost all of the fracture energy comparisons are statistically different, these properties will be used for comparison of the remaining mixtures for this study. FI for the no-notch specimens will not be part of the analysis conducted on the mixtures in the following sections.

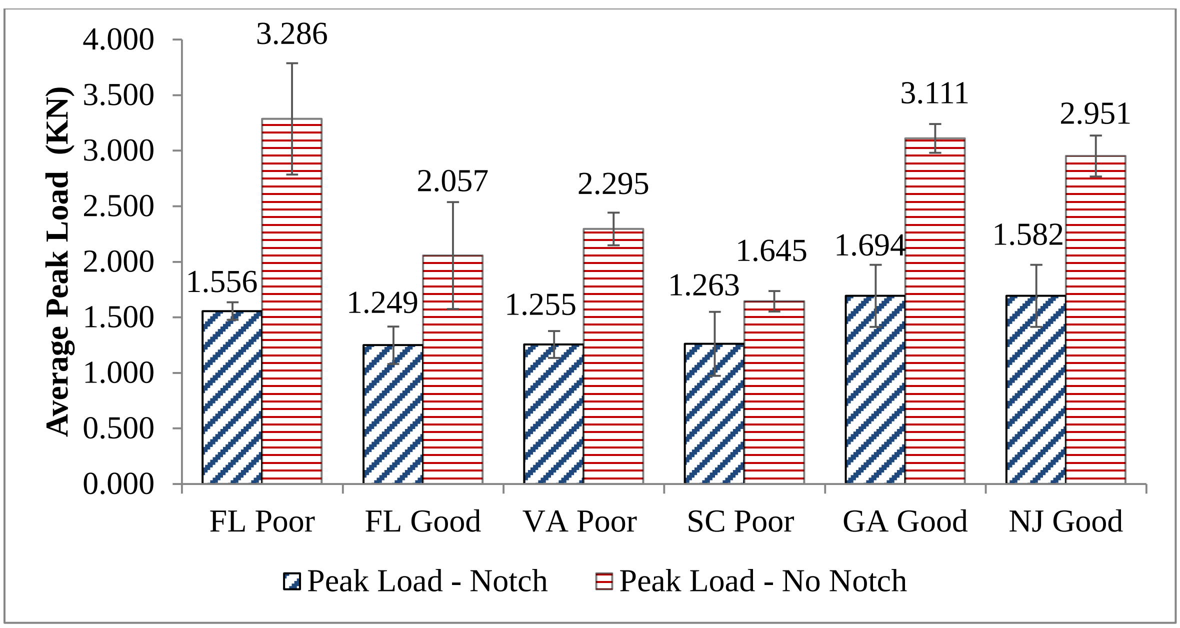


Figure 54 – I-FIT Notch vs No-Notch Comparison for Peak Load

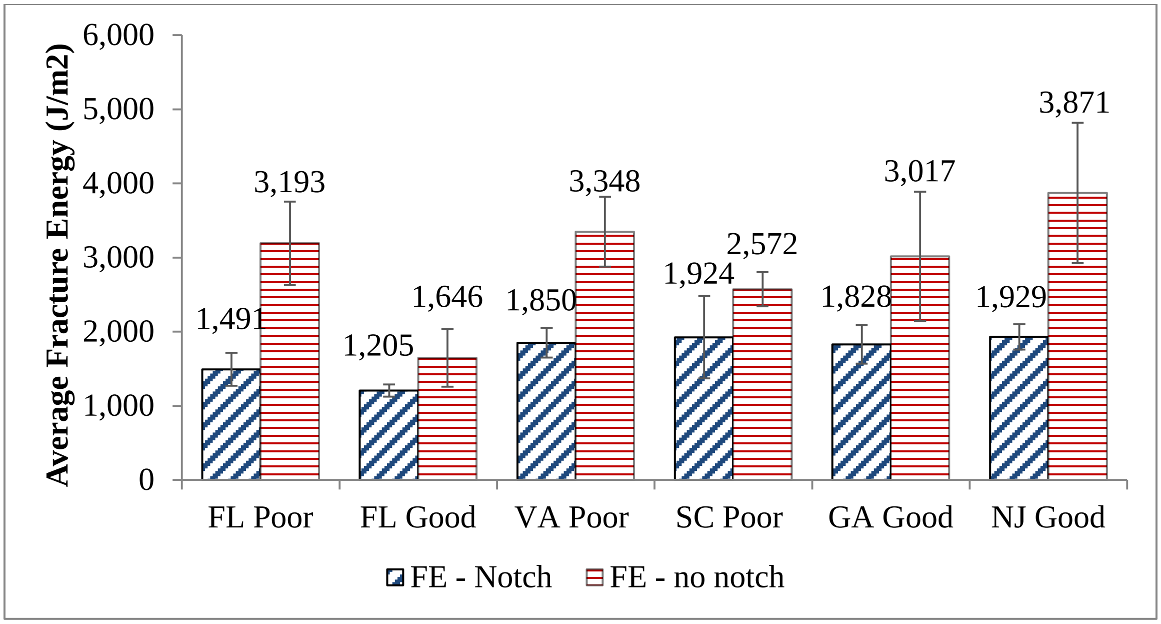


Figure 55 – I-FIT Notch vs No-Notch Comparison for Fracture Energy

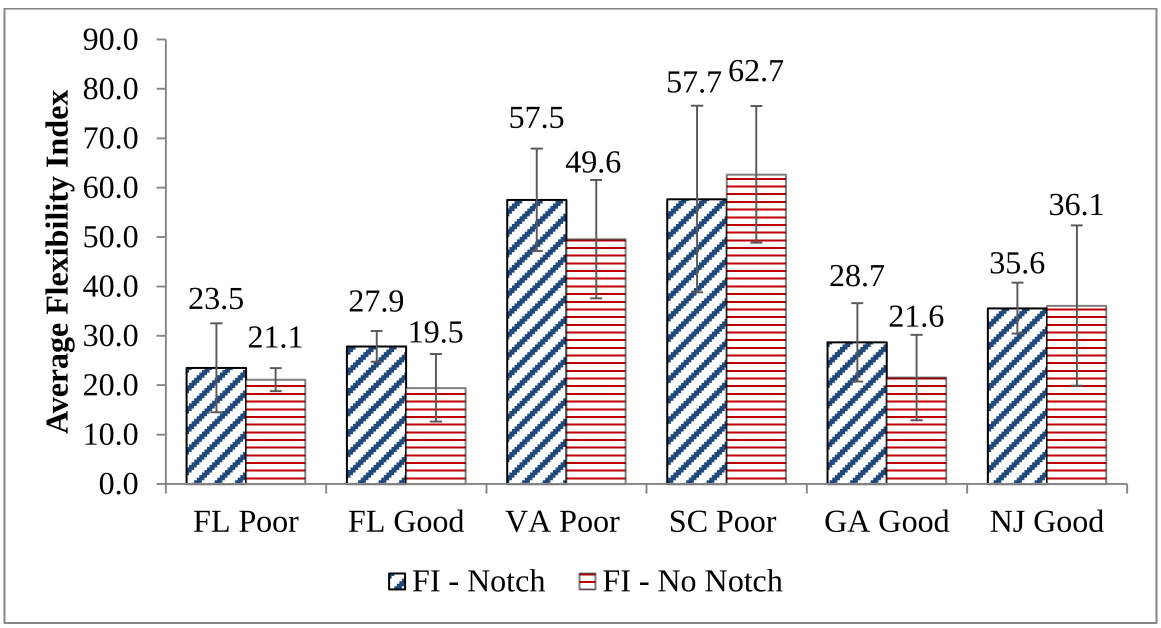


Figure 56 – I-FIT Notch vs No-Notch Comparison for Flexibility Index

Table 36 – Significance of I-FIT Notch vs No-Notch Specimens

Mix ID	P-value		
	Peak Load	G_f	FI
FL Poor SBS	0.000	0.000	0.575
FL Good GTR	0.015	0.059	0.052
VA Poor SBS	0.000	0.000	0.268
SC Poor SBS	0.013	0.032	0.641
GA Good SBS	0.000	0.019	0.212
NJ Good SBS	0.000	0.002	0.949

CHAPTER 6 – PART 2

EXPERIMENT 1: EFFECT OF INCREASED P-200 CONTENT

Introduction

As stated earlier in the Work Plan section, the original plan was to modify the Georgia design by adding baghouse fines (BHF) to the mixture and therefore increase the P-200 material. The expectation was that the binder and increased dust would create a mastic and provide a more durable mixture. The Georgia mix design, already having a design air void content of 15.4 percent, seemed to have little room for additional filler if the samples for this part of the study adhered to the expected minimum air void content of 15.0 percent. The initial addition of 3.0 and 6.0 percent BHF showed a slight improvement to performance but reduced air void content below the minimum of 15.0 percent. The 6.0 percent addition of BHF appeared to provide an insignificant amount of benefit to the mixture; therefore after deciding to swap to the South Carolina mix design, the decision was made to cut back the addition of BHFs to the rate of 2.0 and 4.0 percent. The Georgia and South Carolina designs show marked differences in field performance but have the same mineralogy, gradation, optimum asphalt content, binder type and approximately the same fiber content. This led to a discussion about deducing what mixture properties led to the difference in field performance and what performance test could be used to distinguish these properties. It was decided to test both the Georgia and the South Carolina mix designs for this evaluation. The Georgia design, while already at the expected minimum allowable air void content, was used as a comparison to see how the added BHF would affect both a *good* and *poor* performing mix design. The mix design components for this part of the testing can be found in Table 37. The original Georgia and South Carolina design data are included in this section for comparison purposes. They have been labeled Georgia Control and South Carolina Control.

Table 37 – Experiment 1 Mix Design Components

Mixture Type	Georgia "Good"			South Carolina "Poor"		
Mixture Designation	Control	+2%BHF	+4%BHF	Control	+2%BHF	+4%BHF
Aggregate Mineralogy	Granite					
Asphalt Type	PG 76-22					
Binder Modifier	2.5% SBS Polymer					
Anti-strip	0.5% LOF 6500 by weight of binder					
Fiber, %	0.4			0.3		
Asphalt Content, %	6.0					
Total P-200, %	2.0	3.9	6.0	1.7	3.7	5.6

The percentages of the stockpiles were altered to attempt to keep the mixture gradation blend equal to the original NCAT verification design. As can be seen in Table 38 and

Table 39, the +#4 sieves stayed relatively consistent, while the finer sieves changed more drastically with the increasing amount of BHF. This was unavoidable due to the lack of material passing the #8 sieve. An increase in the P-200 material caused the other fine sieves to shift by approximately the same percentage.

Table 38 – Experiment 1 Georgia Mix Design Alterations

Percent Passing Sieve	Georgia Mix Design Alterations						Georgia Gradation Limits
	JMF	NCAT	+2%BHF	+3%BHF	+4%BHF	+6%BHF	
25.0 mm, 1"	100	100	100	100	100	100	
19.0 mm, 3/4"	100	100	100	100	100	100	100
12.5 mm, 1/2"	92	96	96	96	96	96	85 - 100
9.5 mm, 3/8"	66	66	66	68	66	66	55 - 75
4.75 mm, #4	25	21	22	21	21	23	15 - 25
2.36 mm, #8	8	8	10	7	8	11	5 - 10
1.18 mm, #16	5	6	8	7	8	10	
0.600 mm, #30	4	5	7	6	7	10	
0.300 mm, #50	3	4	6	6	7	9	
0.150 mm, #100	2	3	5	6	7	9	
0.075 mm, #200	1.5	2.0	3.9	4.9	6.0	8.2	2 - 4

Table 39 – Experiment 1 South Carolina Mix Design Alterations

Percent Passing Sieve	South Carolina Mix Design Alterations				South Carolina Gradation Limits
	JMF	NCAT	+2%BHF	+4%BHF	
25.0 mm, 1"	100	100	100	100	
19.0 mm, 3/4"	100	100	100	100	100
12.5 mm, 1/2"	95	95	95	95	89 - 100
9.5 mm, 3/8"	70	74	75	75	63 - 75
4.75 mm, #4	21	21	22	22	15 - 25
2.36 mm, #8	8	8	9	10	5 - 10
1.18 mm, #16		5	6	8	
0.600 mm, #30	5	3	5	7	
0.300 mm, #50		3	4	6	
0.150 mm, #100	5	2	4	6	
0.075 mm, #200	2.2	1.7	3.7	5.6	0 - 4

Results and Discussion

It was anticipated that the added BHF would decrease the air voids in the mix. It was also expected that the film thickness would decrease, however the projection was that the mixture performance would improve enough to mitigate any loss in film thickness or air void content. This may help determine if film thickness and air void content are critical design components, and if so, at what threshold level. A summary of the mixture properties can be found in Table 40. The VCA ratio is based on the breakpoint sieve method.

Table 40 – Summary of Mixture Properties with Increased P-200

Mix ID	Total AC (%)	Total P-200 (%)	Average Air Voids (%)	Average VMA	Average VCA_{MIX}/VCA_{DRC}	Film Thickness (microns)
GA Control	5.0	2.0	17.5	26.3	0.82	21.9
GA Control	6.0	2.0	15.4	26.6	0.82	27.1
GA Control	7.0	2.0	12.5	25.7	0.80	32.4
GA +2BHF	5.0	3.9	15.3	24.1	0.80	13.9
GA +2BHF	6.0	3.9	12.8	23.9	0.80	17.3
GA +2BHF	7.0	3.9	10.4	23.7	0.79	20.7
GA +3BHF	6.0	5.0	14.6	25.1	0.79	15.4
GA +4BHF	5.0	6.0	16.1	25.1	0.80	11.3
GA +4BHF	6.0	6.0	13.1	24.4	0.79	13.9
GA +4BHF	7.0	6.0	11.0	24.5	0.79	16.7
GA +6BHF	6.0	8.2	11.3	22.3	0.79	10.1
SC Control	5.0	1.7	23.6	31.9	0.92	28.0
SC Control	6.0	1.7	22.2	32.3	0.93	34.7
SC Control	7.0	1.7	20.6	32.6	0.94	41.2
SC +2BHF	5.0	3.7	22.3	30.4	0.91	15.9
SC +2BHF	6.0	3.7	20.7	30.8	0.92	19.8
SC +2BHF	7.0	3.7	18.3	30.4	0.91	23.8
SC +4BHF	5.0	5.6	21.3	29.8	0.90	12.0
SC +4BHF	6.0	5.6	19.3	29.8	0.91	14.8
SC +4BHF	7.0	5.6	17.6	30.1	0.91	17.8

The CoreLok air voids of the mixtures (Figure 57) did decrease to a point with added BHF. The Georgia design, which was already near the minimum threshold, did not show any additional decrease in air voids between the 2.0 and 4.0 percent added BHF. The South Carolina design showed an incremental decrease in the air voids with the increased dust content. This consistent decrease is most likely due to the amount of extra room available in the design as indicated from the higher VMA values in Table 40. The film thickness of the designs did significantly decrease with the added BHF (Figure 58). This was anticipated because the surface area of the P-200 is a significant part of the total surface area calculation. The original designs for both the Georgia and South Carolina mixtures had film thicknesses greater than the expected requirement of 24.0 microns. The added BHF dropped all of the modified designs below that point.

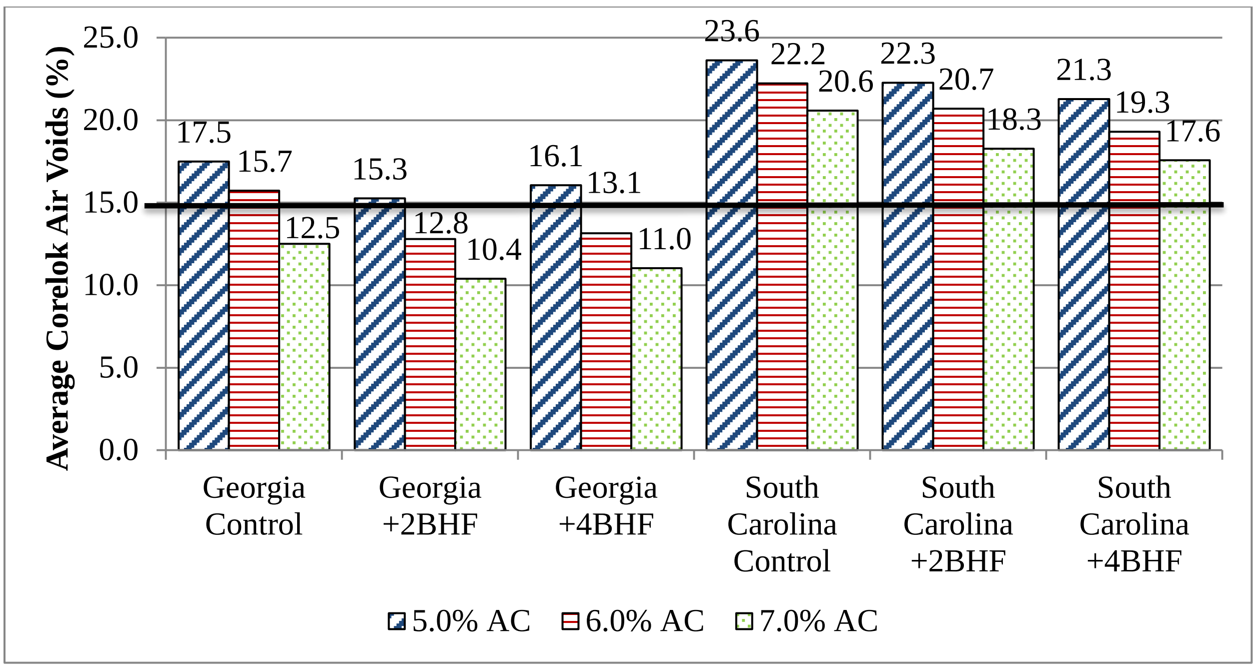


Figure 57 – Experiment 1 CoreLok Air Voids Summary

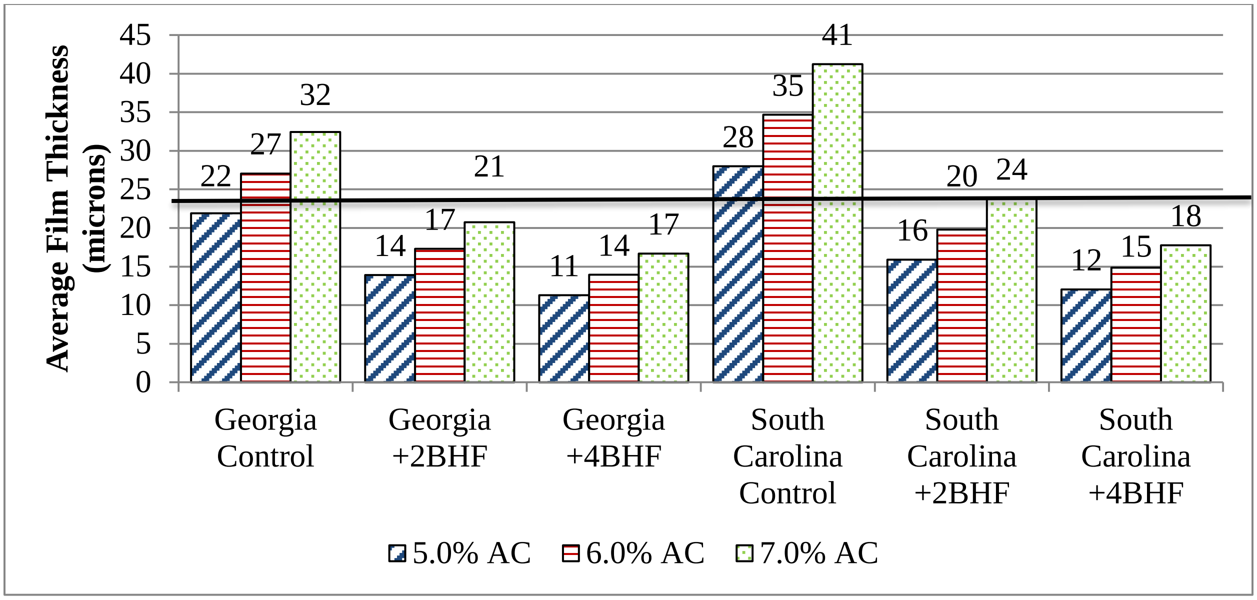


Figure 58 – Experiment 1 Film Thickness Summary

The Cantabro test demonstrated that an increase in P-200 material could improve the durability of the mixture. A graphical depiction of the data can be seen in Figure 59 and a summary of all the data can be seen in Table 41. The added 3.0 and 6.0 percent BHF data are represented in the table but are absent from the figure due to only fabricating specimens at the optimum asphalt content. As anticipated, the Cantabro loss decreased with increased asphalt. The Cantabro loss also decreased with the increased BHF. The South Carolina design showed significant improvement with the initial 2.0 percent addition BHF. Looking at the bar chart in Figure 59, the data shows that increasing the P-200 material by 2.0 percent provides more durability to the design than by increasing the asphalt binder content by 1.0%. The South Carolina Control at 7.0% asphalt content has a percent loss of 26.8, while the South Carolina +2BHF design at the optimum asphalt of content of 6.0% shows a percent loss of 18.9. If the study shows that increased P-200 does more for durability than asphalt binder without sacrificing a significant amount of air voids and permeability, this could greatly benefit the industry. This could decrease the already expensive cost of PFC designs and make them more economical. As shown in Figure 59, the addition of 2% BHF at 5% AC reduced Cantabro loss to values that were the equivalent of adding 1.0% more AC.

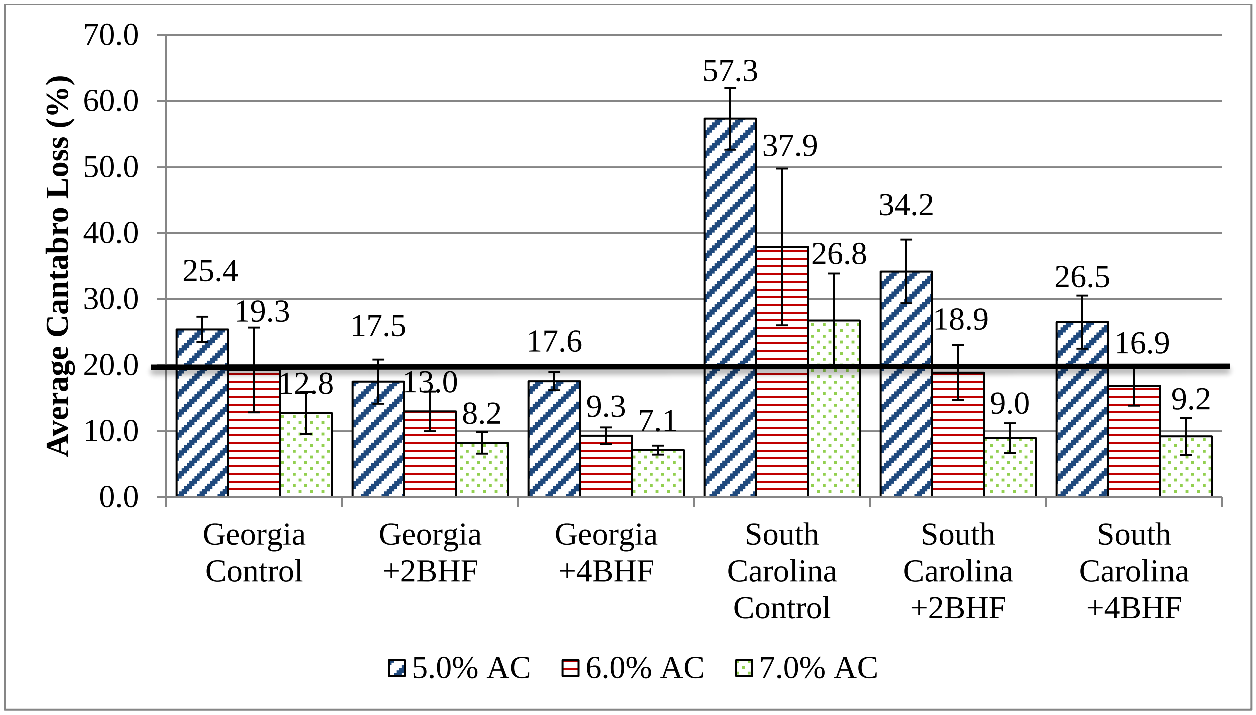


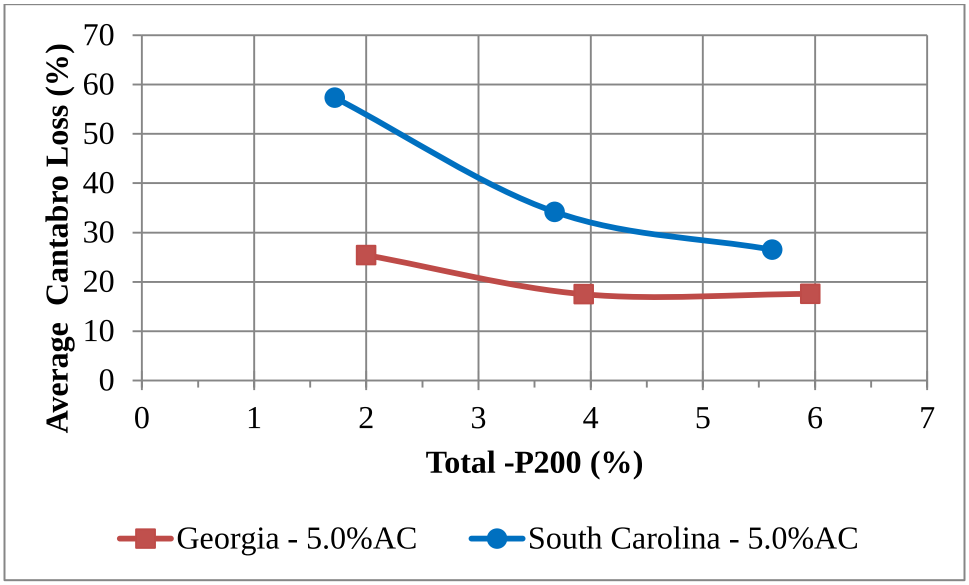
Figure 59 – Experiment 1 Cantabro Loss Summary

Table 41 – Experiment 1 Cantabro Results

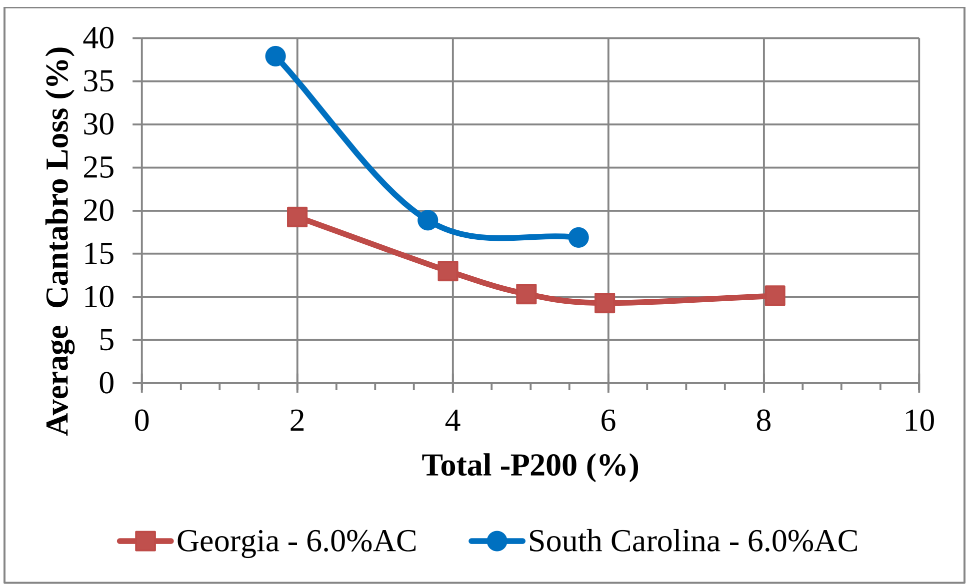
Mix ID	Total AC (%)	Total P-200 (%)	Fiber (%)	Average Air Voids (%)	Cantabro Loss (%)		
					Average	St Dev	COV(%)
GA Control	5.0	2.0	0.4	17.5	25.4	1.9	7.6
GA Control	6.0	2.0	0.4	15.2	19.3	6.4	33.4
GA Control	7.0	2.0	0.4	12.5	12.8	3.2	24.7
GA +2BHF	5.0	3.9	0.4	15.3	17.5	3.3	19.1
GA +2BHF	6.0	3.9	0.4	13.5	13.0	3.0	23.2
GA +2BHF	7.0	3.9	0.4	10.4	8.2	1.7	20.1
GA +3BHF	6.0	5.0	0.4	14.4	10.3	3.5	34.2
GA +4BHF	5.0	6.0	0.4	16.1	17.6	1.4	7.8
GA +4BHF	6.0	6.0	0.4	13.1	9.3	1.3	13.5
GA +4BHF	7.0	6.0	0.4	11.0	7.1	0.7	9.3
GA +6BHF	6.0	8.2	0.4	11.4	10.1	1.0	9.9
SC Control	5.0	1.7	0.3	23.6	57.3	4.7	8.2
SC Control	6.0	1.7	0.3	22.2	37.9	11.9	31.3
SC Control	7.0	1.7	0.3	20.6	26.8	7.1	26.5
SC +2BHF	5.0	3.7	0.3	22.3	34.2	4.8	14.1
SC +2BHF	6.0	3.7	0.3	20.8	18.9	4.2	22.2
SC +2BHF	7.0	3.7	0.3	18.3	9.0	2.2	25.1
SC +4BHF	5.0	5.6	0.3	21.3	26.5	4.0	15.1
SC +4BHF	6.0	5.6	0.3	19.4	16.9	3.0	17.8
SC +4BHF	7.0	5.6	0.3	17.6	9.2	2.8	30.3

The average Cantabro data was plotted to try and determine how to optimize the amount of P-200 material. The data for each asphalt content are represented in separate graphs but all of the data can be found in Figure 60. The data for the Georgia design at optimum asphalt content (Figure 60B) shows 5 data points because it includes the added 3.0 and 6.0 percent BHF. This graph, having more data points, may be more accurate than the other graphs with only 3 points. All of the data indicate that an optimum P-200 content may be somewhere between 4.5 and 5.5

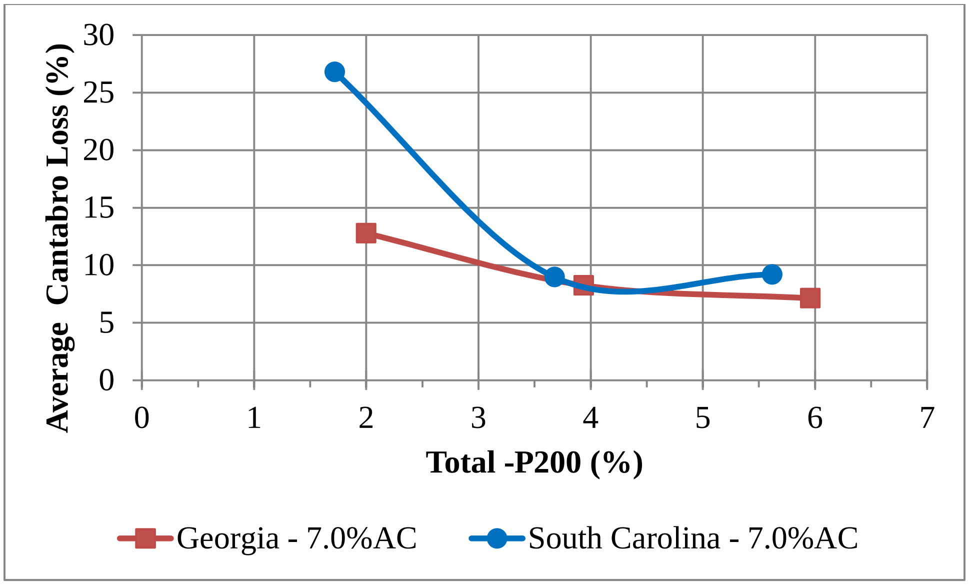
percent. After a P-200 of 5.5, the effects of the increased dust have a negligible effect on the durability of the mix.



A)



B)



C)

Figure 60 – Effect of Increased P-200 on Cantabro Loss

An ANOVA analysis with Tukey-Kramer groupings showed that the South Carolina mix with any percentage of added BHF was statistically different from the South Carolina JMF Control (Table 42B). The Georgia design showed improvement, as can be seen by the means, but the only statistical difference was between the Control and the added 6.0 percent BHF (Table 42A). It is important to note that the Georgia Control passed the expected 20.0 percent Cantabro loss criterion, while the South Carolina Control did not. The South Carolina Control improved by over 20 percent with the addition of 2.0 percent BHF, but it only improved an additional 2 percent when BHF was increased to 4.0 percent. This, along with the Georgia analysis, shows that increasing the P-200 material will help the durability of all the mixtures, but it may have a greater effect on mixtures with a larger initial Cantabro loss.

Table 42 – ANOVA (α=0.05) Analysis of Cantabro Loss with Increased P-200

A)

Mix ID	Georgia			
	N	Mean	Grouping	
Georgia Control	3	19.3	A	
Georgia +2.0BHF	3	13.0	A	B
Georgia +3.0BHF	3	10.3	A	B
Georgia +4.0BHF	3	10.1	A	B
Georgia +6.0BHF	3	9.3		B

p =0.039
R^2 = 60%

B)

Mix ID	South Carolina			
	N	Mean	Grouping	
South Carolina Control	3	39.0	A	
South Carolina +2.0BHF	3	18.9		B
South Carolina +4.0BHF	3	16.9		B

p<0.001
R^2 = 71%

As discussed in Part 1, there appears to be a direct correlation between air void content and permeability of the PFC specimens. The increased P-200 causes the air void content of the specimens to decrease; therefore it was expected that the permeability of the specimens would also decrease. The Georgia 3.0 and 6.0 percent added BHF were included in both the summary (Table 43) and the graph (Figure 61). The South Carolina design showed little decrease in permeability with increased P-200. This is most likely due to the initial air void content of the mixture being relatively high. The Georgia design which had an initial permeability value less than the recommended 100 m/day showed a decrease in permeability with increased P-200. The air void content of the mixtures directly reflects this as well. The only other criterion for permeability testing is the 35 m/day provided by Mississippi in their survey response. If that criterion is applied, only the Georgia design with an added 6.0 percent BHF fails.

Table 43 – Permeability Summary with Increased P-200

Mix ID	Total AC (%)	Total P-200 (%)	Fiber (%)	Average Air Voids (%)	Permeability (k) meter/day		
					Average	St Dev	COV(%)
Georgia Control	6.0	2.0	0.4	15.7	80	10.5	13.1
Georgia +2BHF	6.0	3.9	0.4	13.4	43	1.6	3.6
Georgia +3BHF	6.0	5.0	0.4	14.5	51	17.4	34.3
Georgia +4BHF	6.0	6.0	0.4	13.2	38	7.4	19.5
Georgia +6BHF	6.0	8.2	0.4	11.4	11	3.7	33.0
South Carolina Control	6.0	1.7	0.3	22.2	209	17.1	8.2
South Carolina +2BHF	6.0	3.7	0.3	20.6	222	17.3	7.8
South Carolina +4BHF	6.0	5.6	0.3	19.2	196	17.1	8.7

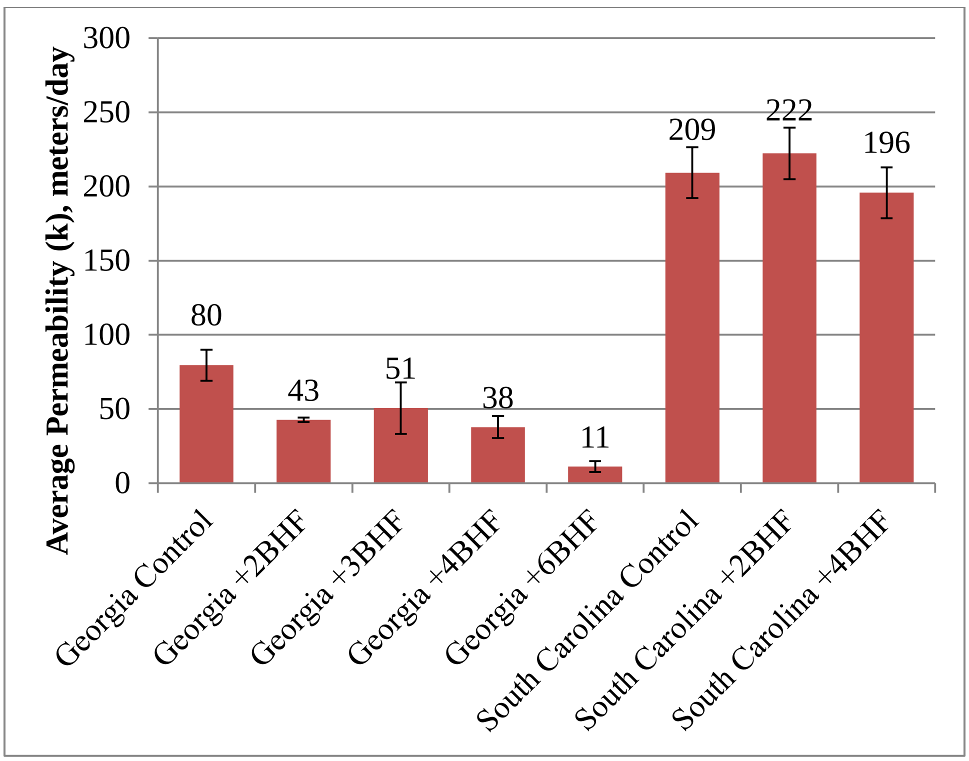


Figure 61 – Increased P-200 Permeability Results

The HWTT summary (Table 44) for this section shows trends that are similar to the Cantabro results. The South Carolina Control mixture failed around 2,200 passes but when the P-

200 content was increased with 2.0 percent BHF, the performance improved and the mixture did not fail until 12,600 passes. The 4.0 percent added BHF specimens showed improvement over both designs and reached 19,400 passes before reaching the failure criterion. This +4BHF design came close to passing the TxDOT criterion of 20,000 passes. As can be seen in Figure 62 and Figure 63, the samples incurred primary (initial consolidation) and secondary (constant strain) deformation, but did not reach tertiary (shear deformation). Without tertiary deformation there is no inflection point for these mixtures. The samples only exhibited densification and no change due to shear was observed. The Georgia design showed marked improvement with the addition of 2.0 percent BHF but no additional improvement was observed with the 4.0 percent BHF specimens.

Table 44 – HWTT Summary Results for Increased P-200

Mix ID	Total AC (%)	Total P-200 (%)	Fiber (%)	Average Air Voids (%)	Greatest Rut Depth Recorded		
					Average, mm	St Dev, mm	COV(%)
GA Control	6.0	2.0	0.4	14.3	8.99	2.88	32.0
GA +2BHF	6.0	3.9	0.4	13.7	5.54	0.62	11.3
GA +4BHF	6.0	6.0	0.4	13.1	5.36	0.67	12.4
SC Control	6.0	1.7	0.3	21.7	15.85	1.84	11.6
SC +2BHF	6.0	3.7	0.3	20.8	15.14	4.64	30.6
SC +4BHF	6.0	5.6	0.3	20.4	12.81	1.99	15.5

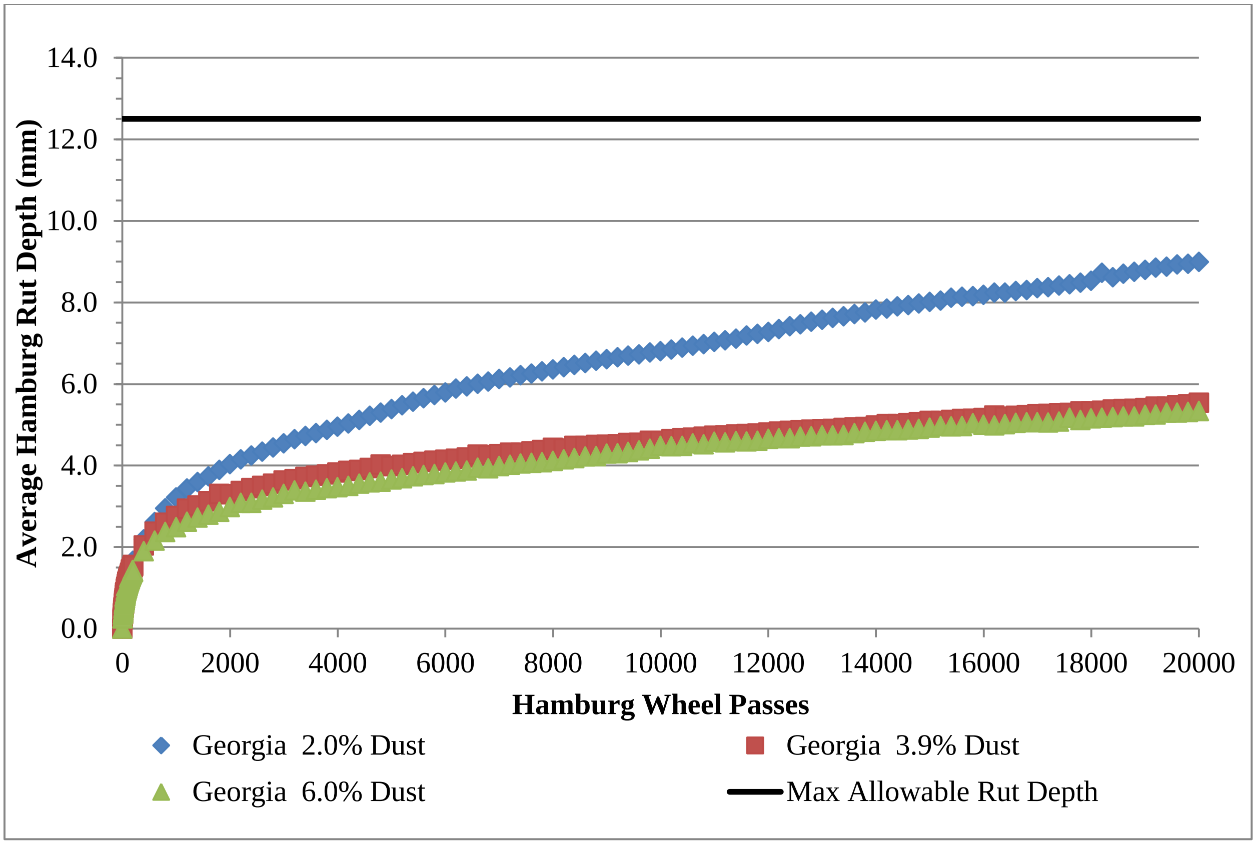


Figure 62 – Georgia HWTT Results with Increased P-200

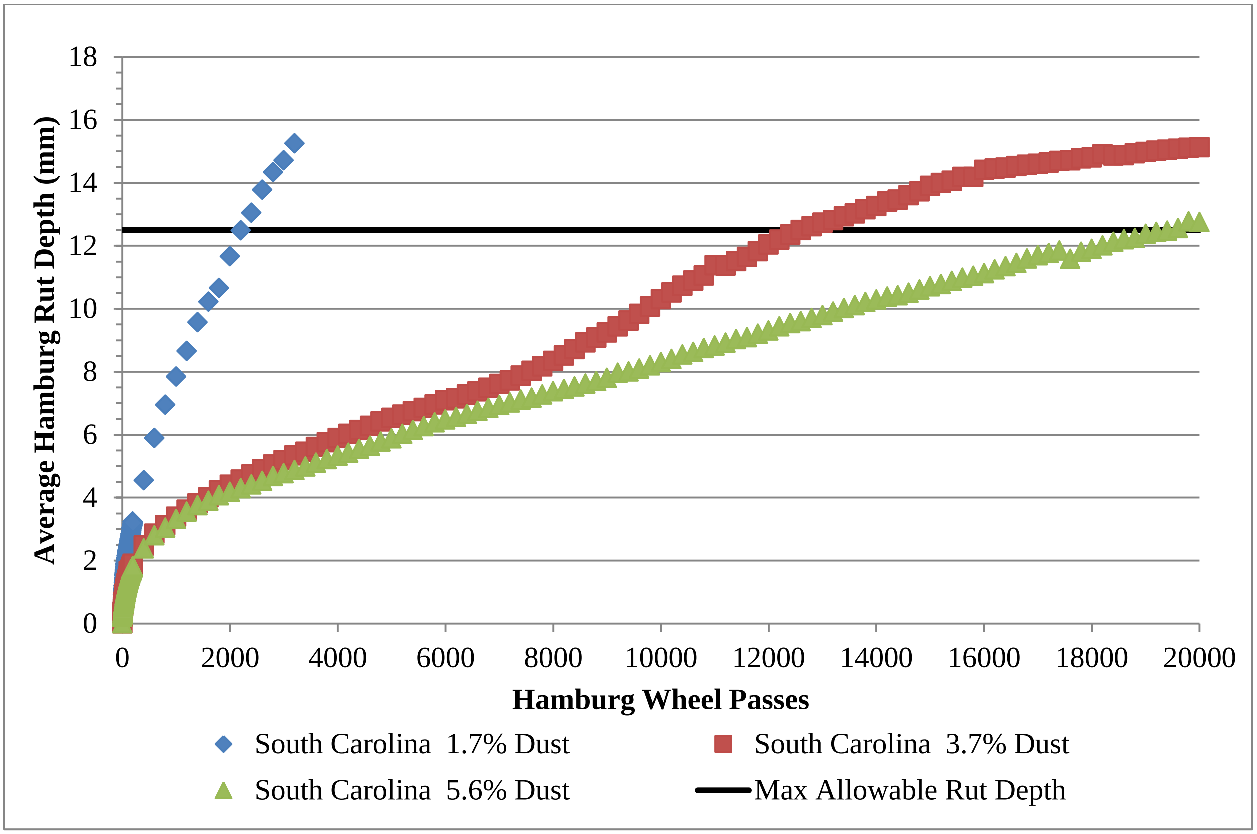


Figure 63 – South Carolina HWTT Results with Increased P-200

The testing for TSR in this experiment was critical. While increasing the P-200 can potentially create a mastic and consequently a stronger mixture, it also creates less free binder. Free binder is the extra binder remaining after the aggregate has been coated and binder absorption has taken place. The P-200 absorbs more of the free binder, leaving less to coat the coarse material. This can potentially lead to moisture susceptibility issues. The expectation was that the extra P-200 would increase the ITS of the mixtures and hopefully not decrease the TSR. If the ITS was increased significantly and the TSR did fall below the recommended criteria, it was thought that the anti-strip dosage could be increased or hydrated lime could be added to the mixture to ensure passing results. The improvement in performance of the mixture should offset the additional cost of the change in dosage or type of anti-strip. The summary for this testing can be found in Table 45. The unconditioned tensile strength of each mixture with increased P-200 improved over the control for each design. As shown in Table 45, the conditioned strength of the mix improved over 40% when mix with 4% BHF is compared to the conditioned values of the control. The TSR value increased with the Georgia design but decreased with the South Carolina design. The South Carolina conditioned strengths did not increase with the same magnitude as the unconditioned strengths.

Table 45 – Tensile Strength Ratio Summary for Increased P-200

Mix ID	Total AC (%)	Total P-200,%	Average Air Voids (%)		Average ITS (psi)		TSR
			Conditioned	Unconditioned	Conditioned	Unconditioned	
GA Control	6.0	2.0	13.9	14.0	57.6	74.3	0.78
GA +2BHF	6.0	3.9	13.3	13.4	86.4	100.3	0.86
GA +4BHF	6.0	6.0	14.3	14.3	82.4	99.8	0.83
SC Control	6.0	1.7	21.2	21.2	36.8	45.2	0.81
SC +2BHF	6.0	3.7	20.6	20.5	38.8	62.0	0.63
SC +4BHF	6.0	5.6	19.0	18.9	54.4	77.3	0.70

The ANOVA analysis was conducted on both the South Carolina and Georgia designs. It was initially performed separately, but the results for this testing were so significantly different that combining the results showed the same results. The results from the Tukey-Kramer grouping can be found in Table 46. Both the conditioned and unconditioned results showed good fit with a significant difference between the mixtures. The Georgia mix designs in regards to ITS with the additional 2.0 BHF, showed some improvement over the control, but looking at the means it can be seen that the 4.0 BHF showed no improvement over the 2.0 BHF. The South Carolina Control was significantly different from both the 2.0 and 4.0 BHF for the unconditioned strengths. The 2.0 BHF was not significantly different from the South Carolina Control in the conditioned strengths. As seen in Table 45 the South Carolina added BHF shows a decrease for the TSR results. This lower conditioned value for the 2.0 BHF caused the TSR value to be low and also caused it to be grouped with the Control.

Table 46 – ANOVA Analysis for ITS of Increased P-200 (α=0.05)

Mix ID	Conditioned			Unconditioned		
	N	Mean	Grouping	N	Mean	Grouping
Georgia +2BHF	3	86.4	A	3	100.3	A
Georgia +4BHF	3	82.4	A	3	99.8	A
Georgia Control	3	57.6	B	3	74.3	B
South Carolina +4BHF	3	54.4	B	3	77.3	B
South Carolina +2BHF	3	38.8	C	3	62.0	B
South Carolina Control	3	36.8	C	3	45.2	C

Conditioned: $p<0.001$, $R^2 = 98\%$

Unconditioned: $p<0.001$, $R^2 = 94\%$

The OT results for the increased P-200 specimens showed a relative increase in performance when compared to the Control specimens. The summary of the results can be seen in Table 47 and a graphical depiction is shown in Figure 64. The South Carolina designs all terminated prior to reaching the 93 percent load reduction and therefore had to be extrapolated. An outlier in both the Georgia Control and Georgia +2BHF designs was observed. These 2 outliers were removed from the data set prior to analysis. The Georgia +4BHF had no outliers but did show a large COV (49%). The peak load for each design decreased with increase P-200, while cycles to failure showed an increase with the increased P-200. The South Carolina +2BHF showed more extrapolated improvement than the +4BHF design, but the Georgia design improved with each increase of the P-200.

Table 47 – OT Summary for Increased P-200

Mix ID	*Replicates*	*Average Air Voids (%)*	*Average Peak Load (kN)*	*Cycles to Failure*		
				Average	*St Dev*	*COV (%)*
SC Control	6	19.2	1.798	1491	388	26.0
SC +2BHF	4	17.3	1.659	2335	480	20.6
SC +4BHF	4	22.5	1.638	1662	423	25.4
GA Control	3	12.8	2.621	583	166	28.4
GA +2BHF	3	11.0	2.456	682	177	25.9
GA +4BHF	4	13.4	1.998	941	464	49.3

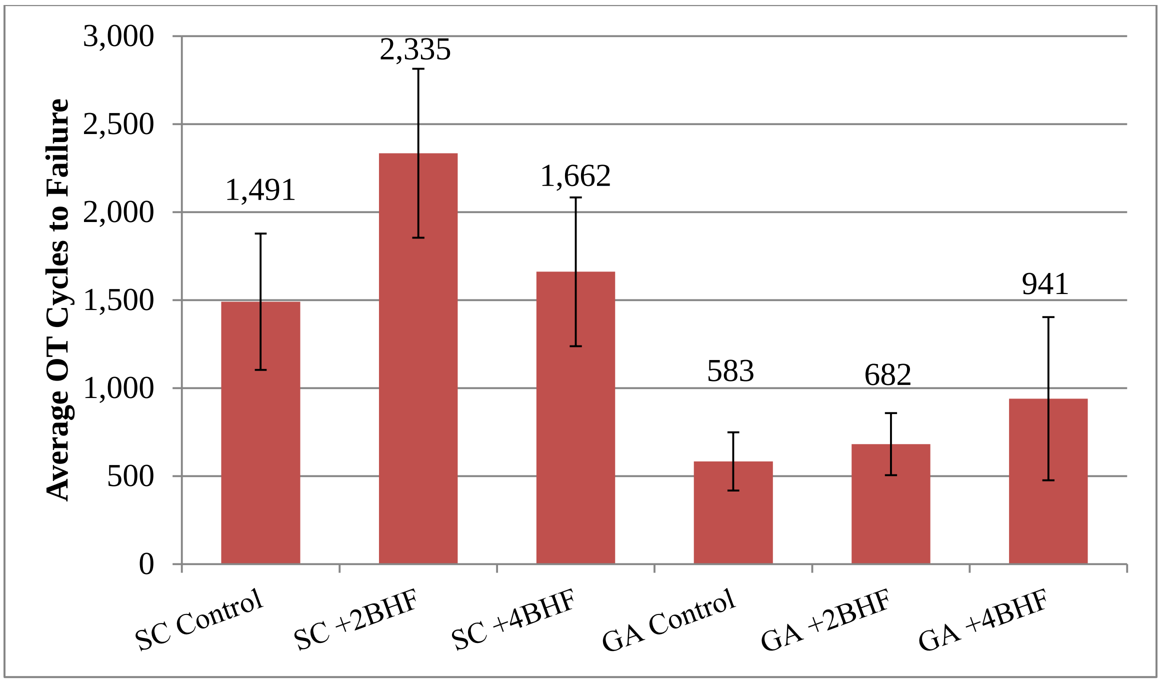


Figure 64 – OT Results for Increased P-200

The ANOVA analysis for the cycles to failure (Table 48) showed that the South Carolina +2BHF was significantly improved over the Control. The South Carolina +4BHF was not different from the Control or the +2BHF designs. The Georgia designs showed numerical improvement with increased P-200 but there was not a statistical difference between the designs when analyzing cycles to failure. In regards to peak load, an almost inverse relationship is observed (Table 49). The Georgia Control is statistically different from the Georgia +4BHF design while the Georgia +2BHF design shows no difference. The South Carolina design showed some difference in peak load but there was no significant improvement observed with the change in the P-200.

Table 48 – ANOVA Analysis for OT Cycles to Failure for Increased P-200 Specimens

Mix ID	Cycles to Failure					
	N	Mean	Grouping			
SC +2BHF	4	2335	A			
SC +4BHF	4	1662	A	B		
SC Control	6	1491		B	C	
GA +4BHF	4	941		B	C	D
GA +2BHF	3	682			C	D
GA Control	3	583				D

$p<0.001$
$R^2 = 75\%$

Table 49 – ANOVA Analysis for OT Peak Load for Increased P-200 Specimens

Mix ID	Peak Load				
	N	Mean	Grouping		
GA Control	3	2.621	A		
GA +2BHF	3	2.456	A	B	
GA +4BHF	4	1.998		B	C
SC Control	6	1.798			C
SC +2BHF	4	1.659			C
SC +4BHF	4	1.638			C

$p<0.001$
$R^2 = 73\%$

Draindown was not evident for this experiment, nor was it expected. The only thing that changed in the designs for this part of the study was the addition of BHF, which would only decrease the amount of free binder. These results were as expected.

Table 50 – Draindown Results for Increased P-200

Mix ID	Total AC (%)	Fiber (%)	Total P-200 (%)	Draindown (%)	
				Test Temp, °F	
				330	357
Georgia Control	6.0	0.4	2.0	0.0	0.0
Georgia +2BHF	6.0	0.4	3.9	0.0	0.0
Georgia +4BHF	6.0	0.4	6.0	0.0	0.0
South Carolina Control	6.0	0.3	1.7	0.0	0.0
South Carolina +2BHF	6.0	0.3	3.7	0.0	0.0
South Carolina +4BHF	6.0	0.3	5.6	0.0	0.0

The I-FIT testing for the mix designs with the increased P-200 can be found in Figure 65 through Figure 67. As previously analyzed, the peak load, G_f and FI of each mix was calculated and summarized. The peak load (Figure 65) appears to increase with increased P-200. As seen in other testing, there appears to be negligible benefit between the Georgia 2.0 and 4.0 BHF. The G_f of the mixtures (Figure 66) are relatively uniform regardless of mix design or addition of BHF. This may indicate that the mixture strength is increased with the additional P-200, but the amount of energy required to fracture the specimen is not affected. The FI (Figure 67) of the mixtures did not trend as expected. The FI is an indication of cracking potential, therefore with increased P-200 it was expected that the FI would decrease due to the decrease in free binder. The South Carolina mix design did show a decrease in FI with increased P-200, however the Georgia design showed a more parabolic trend (similar to the Cantabro results).

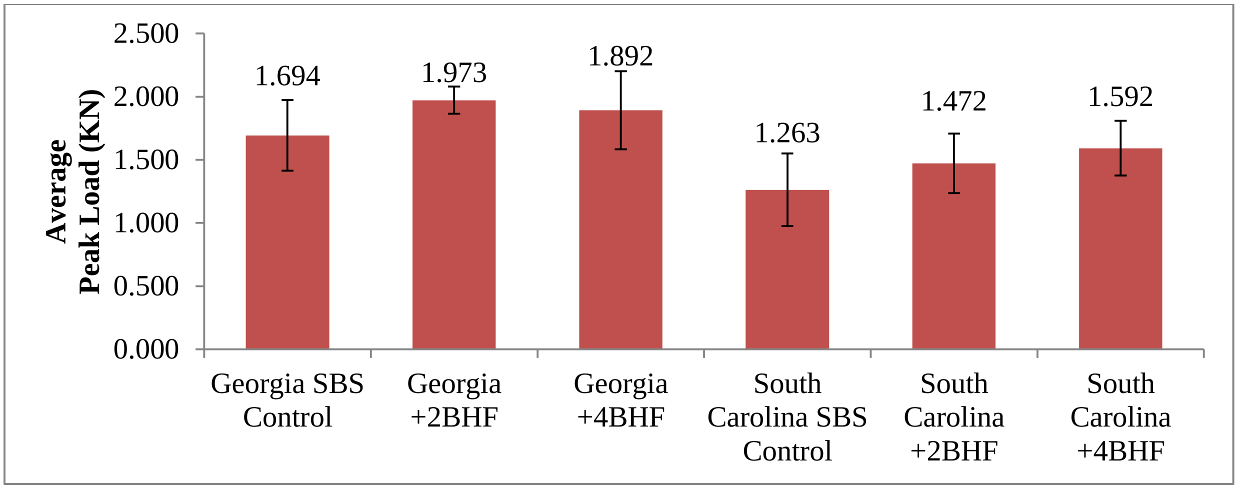


Figure 65 – I-FIT Peak Load Results for Increased P-200

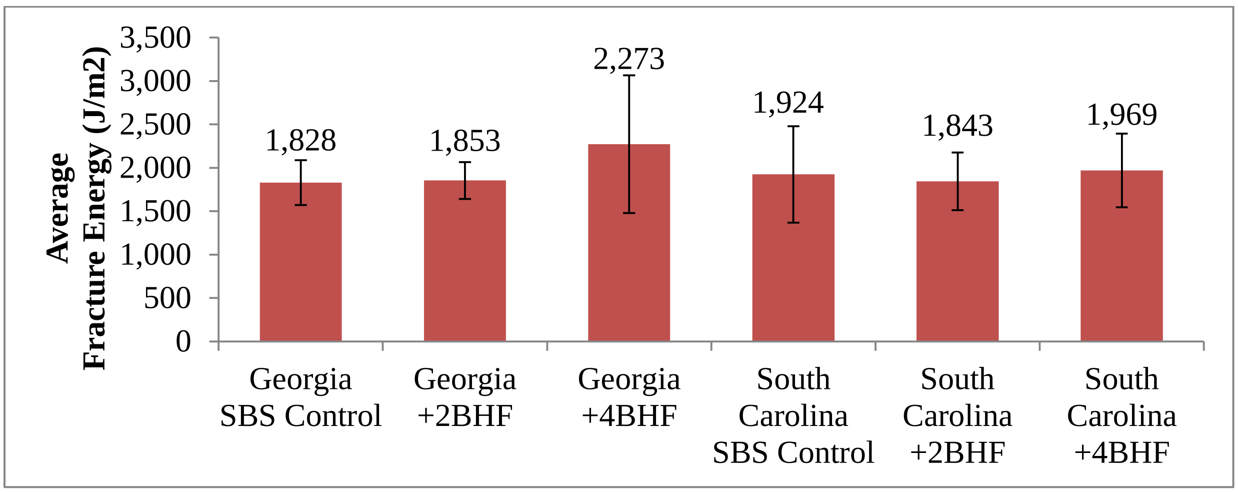


Figure 66 – I-FIT Fracture Energy Results for Increased P-200

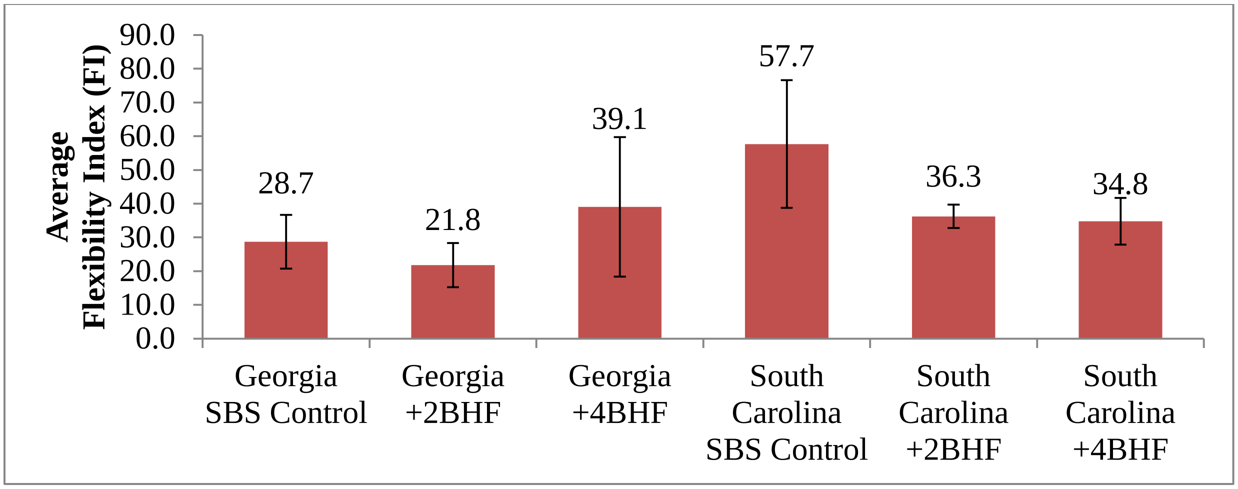


Figure 67 – I-FIT Flexibility Index Results for Increased P-200

The ANOVA analysis (α=0.05) showed that some of the mixtures were statistically different when comparing peak load and FI (Table 51). The peak load analysis shows that the increase in P-200 material did not statistically change the Georgia or the South Carolina results. The Georgia design with the additional 2.0 percent BHF was statistically different from both the South Carolina Control and 2.0 percent additional BHF designs. This indicates that even though adding the BHF to the South Carolina design did not change the peak load when compared to the Control, the addition of the 4.0 percent BHF changed it enough to allow it to be grouped with Georgia +2BHF design. While the numerical difference in peak load for the South Carolina designs was not large enough to be significant, it did make a noticeable change when analyzed with a larger data set. This numerical increase in peak load is important to note, if the peak load becomes part of the criteria for designing PFC. The G_f results of the mixtures were not statistically different. This can be seen in Table 52 with a p-value of 0.636 and a single Tukey-Kramer grouping. This analysis had a poor fit with a R^2 of only 12.1 percent. The FI analysis showed that the South Carolina Control was statistically different from the Georgia Control and +2BHF designs. It was expected that the Georgia and South Carolina designs would be statistically different; however the Georgia +4BHF had the second highest FI of the mixes. This indicates that it has the same resistance to cracking as the South Carolina Control. There is a large numerical difference between these designs but they are not statistically different. As stated earlier, the Georgia data did not trend as expected.

Table 51 – ANOVA Analysis for I-FIT Peak Load with Increased P-200

Mix ID	Peak Load		
	N	**Mean**	**Grouping**
Georgia +2BHF	5	1.973	A
Georgia +4BHF	6	1.892	A B
Georgia Control	5	1.694	A B C
South Carolina +4BHF	6	1.592	A B C
South Carolina +2BHF	5	1.472	B C
South Carolina Control	4	1.263	C

p =0.002
R^2 = 52%

Table 52 – ANOVA Analysis for I-FIT Fracture Energy with Increased P-200

Mix ID	Fracture Energy		
	N	**Mean**	**Grouping**
Georgia +4BHF	6	2273	A
South Carolina +4BHF	6	1969	A
South Carolina Control	4	1924	A
Georgia +2BHF	5	1854	A
South Carolina +2BHF	5	1843	A
Georgia Control	5	1828	A

p =0.636
R^2 = 12%

Table 53 – ANOVA Analysis for I-FIT Flexibility Index with Increased P-200

Mix ID	Flexibility Index			
	N	Mean	Grouping	
South Carolina Control	4	57.7	A	
Georgia +4BHF	6	39.1	A	B
South Carolina +2BHF	5	36.3	A	B
South Carolina +4BHF	6	34.8	A	B
Georgia Control	5	28.7		B
Georgia +2BHF	5	21.8		B

p =0.007
R^2 = 45%

The no-notch I-FIT testing showed that with increasing P-200 there was a significant increase in the peak load required to fracture the specimens (Figure 68). The force required to break the specimens was increased, with increased P-200, for both the Georgia and South Carolina designs. The trend of increased P-200 and peak load was linear for each of the designs. There were 2 outliers removed from the data (1 each from Georgia Control and Georgia +2BHF) prior to the analysis.

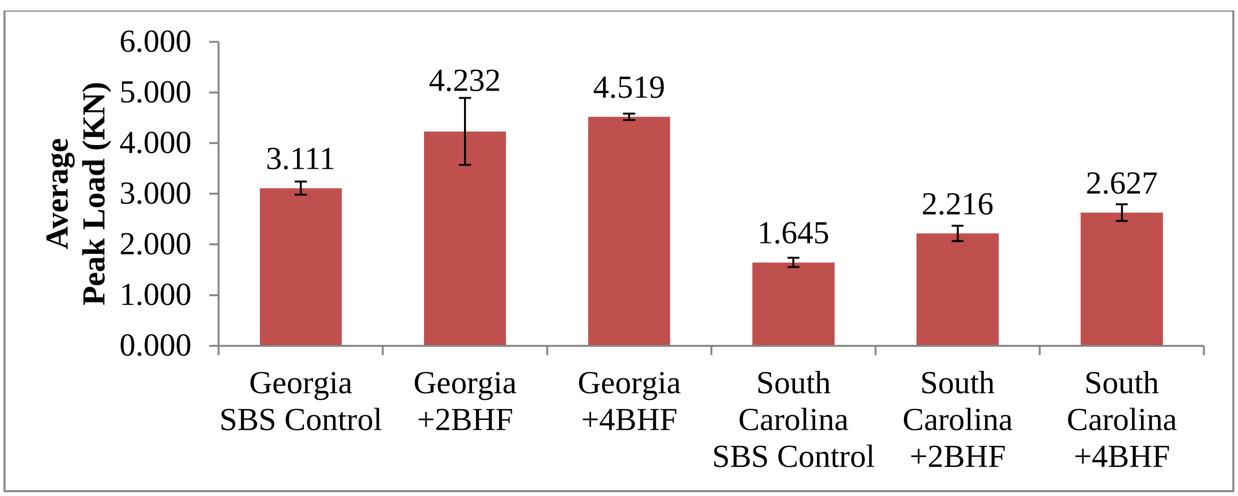


Figure 68 – No-Notch I-FIT Peak Load Results with Increased P-200

The ANOVA analysis (α=0.05) in regards to peak load (Table 54) showed that the Georgia designs with added BHF exhibited statistical improvement over the Georgia Control. The South Carolina Control did not share a grouping with any of the designs and was therefore statistically different from all the other mixtures tested for this experiment. The South Carolina designs with added BHF also exhibited statistical improvement over the South Carolina Control. There was continuous numerical improvement for each design with added BHF; but the 2.0 and 4.0 BHF designs, for both Georgia and South Carolina, were not statistically different.

Table 54 – ANOVA Analysis for No-Notch I-FIT Peak Load with Increased P-200

Mix ID	Peak Load		
	N	**Mean**	**Grouping**
Georgia +4BHF	6	4.519	A
Georgia +2BHF	5	4.232	A
Georgia Control	5	3.111	B
South Carolina +4BHF	6	2.627	B C
South Carolina +2BHF	6	2.216	C
South Carolina Control	6	1.645	D

$p<0.001$
$R^2 = 95\%$

The G_f of both the South Carolina and Georgia designs showed no statistical improvement by increasing the P-200 for each mixture. There was a continuous numerical improvement for each design, which may indicate that if the volumetric properties and permeability will allow more P-200, the mixture may be improved. The Georgia design could not be increased; however the South Carolina design had room to increase the P-200 material even more than the additional 4.0 percent BHF.

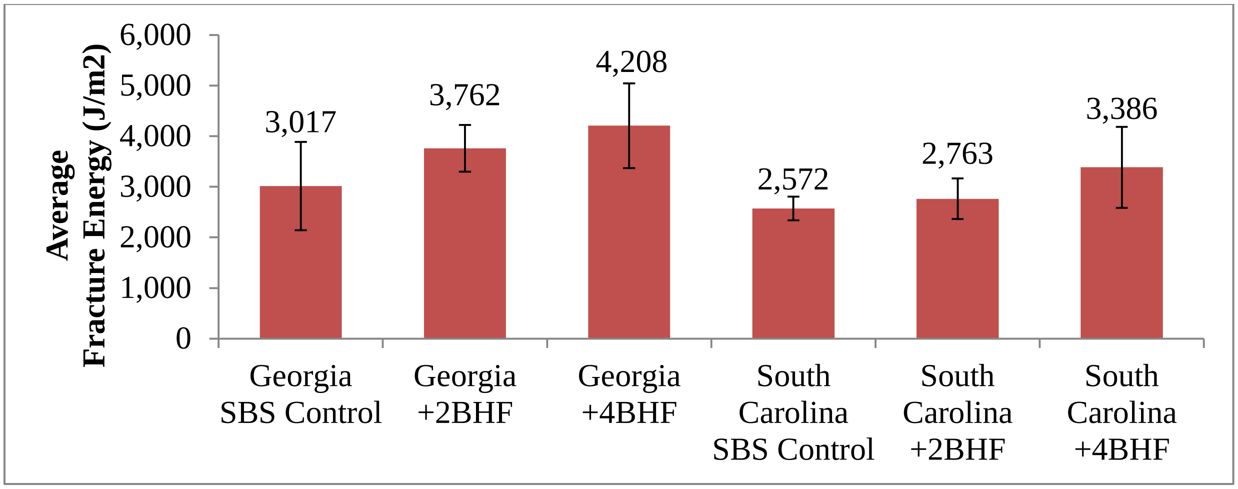


Figure 69 - No-Notch I-FIT Fracture Energy Results with Increased P-200

Table 55 - ANOVA Analysis for No-Notch I-FIT Fracture Energy with Increased P-200

Mix ID	**Fracture Energy**			
	N	**Mean**	**Grouping**	
Georgia +4BHF	6	4208	A	
Georgia +2BHF	5	3762	A	B
South Carolina +4BHF	6	3386	A	B
Georgia Control	5	3017	A	B
South Carolina +2BHF	6	2763		B
South Carolina Control	6	2572		B

p =0.001
R^2 = 49%

EXPERIMENT 2: EFFECT OF BINDER MODIFICATION

Introduction

This section involves the differences in the Georgia mix design performance when binder modification and fiber content are altered. The expectation was that a change in the binder modification would provide insight into the resistance of PFC mixture to raveling and cracking. The Georgia design at the optimum asphalt content was chosen for this part of the study. The fiber was omitted from this part of the study as well for dual purposes: (1) to determine if with binder modification, fiber would still be needed to prevent draindown, and (2) to see if the fiber had an impact on the raveling or cracking potential of the mixture. The mix design components for this part of the study can be found in Table 56. The PG 76-22 (SBS) was the same binder used for Part 1 and the GTR binder was the same as the binder used for the Florida *good* mix in Part 1. The PG 82-22 (HiMA) was originally a PG 88-28 that was modified with a high dosage of SBS polymer. In order to ensure that the asphalt binder was the expected performance grade, samples of each binder were tested according to AASHTO M320, *Standard Specification for Performance-Graded Asphalt Binder*. The results from this testing can be seen in Table 57. As can be seen in the table, all three asphalt binders graded as expected.

Table 56 – Mix Design Components for Georgia with Binder Modifications

Mixture Type	Georgia "Good"		
Aggregate Mineralogy	Granite		
Asphalt Type	PG 76-22	PG 67-22	PG 82-22
Binder Modifier	2.5% SBS	12% -#30 GTR	7.5% SBS
Anti-strip	0.5% LOF 6500 by weight of binder		
Fiber, %	0.0		
Asphalt Content, %	6.0		
P-200, %	2.0	2.0	2.0

Table 57 – Asphalt Binder Grade Summary (AASHTO M320)

Expected Binder Grade	Blending	Additive	Continuous Grade	PG Grade
PG 76-22	Manufacturer	SBS	77.1 - 25.4	76 - 22
PG 76-22	NCAT Lab	GTR	81.0 - 23.8	76 - 22
PG 82-22	Manufacturer	SBS	87.4 - 26.8	82 - 22

Results and Discussion

The original Georgia design data are included in this section for comparison purposes. They have been labeled Georgia Control. The mixture properties for this section are based on the Cantabro data. The summary of these properties can be found in Table 58. The GTR design had a lower air void content compared to the other mixtures due to the increased asphalt content. The SBS design without fiber had a higher air void content than the Control design with fiber. The film thicknesses of the specimens were relatively uniform. The small deviations are due to the variation in mixture G_{mb} and G_{mm}. The GTR design had a larger film thickness due to the higher asphalt content. The VCA ratio is based on the breakpoint sieve method.

Table 58 - Mixture Properties Summary with Binder Modification

Mix ID	Total AC (%)	Fiber (%)	Average Air Voids (%)	Average VMA	Average VCA_{MIX}/VCA_{DRC}	Film Thickness
Georgia Control	6.0	0.4	15.4	26.6	0.82	27.1
Georgia PG 76-22 SBS	6.0	0.0	16.3	26.5	0.81	25.2
Georgia PG 76-22 GTR	6.7	0.0	13.9	25.9	0.80	28.2
Georgia PG 82-22 SBS	6.0	0.0	14.9	25.5	0.79	26.1

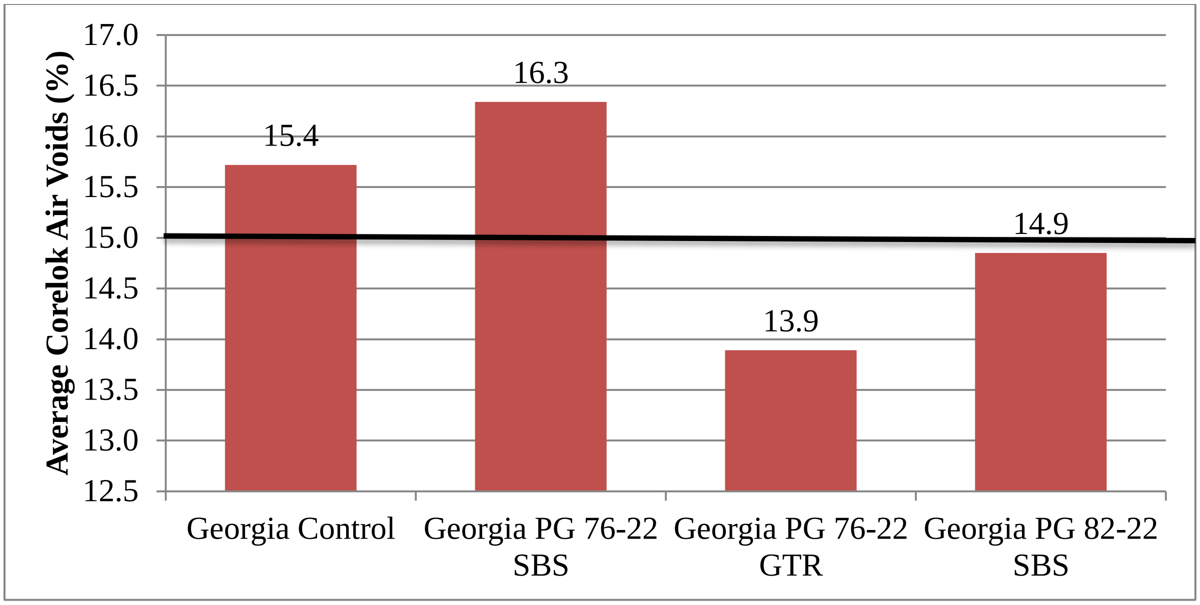


Figure 70 – Corelok Air Voids with Binder Modification

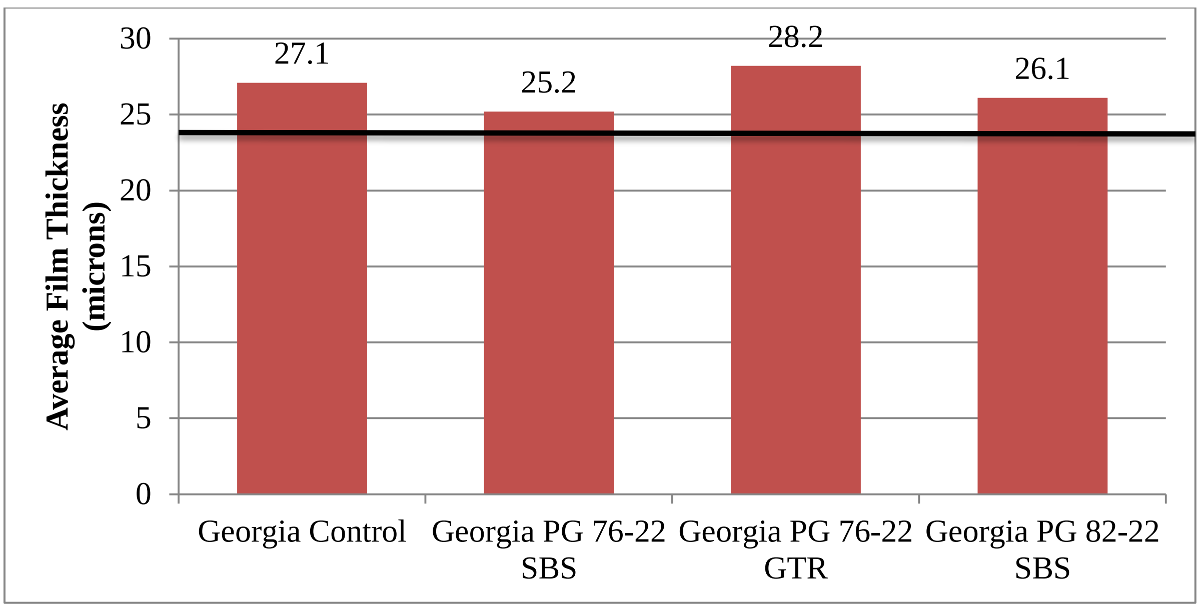


Figure 71 – Film Thickness with Binder Modification

The Cantabro data showed that the designs without fiber performed better than the design with fiber. The summary table (Table 59) shows that the SBS and GTR samples had similar percent loss while the HiMA design had negligible loss. There were 3 specimens tested for each design so the sample size is adequate; however there is a high COV of the SBS design so that any conclusions regarding Cantabro loss for that binder may be misleading.. The almost negligible loss for the HiMA design was not unexpected due to the large amount of polymer in the binder. It was expected that the HiMA design would perform well for both durability and crack resistance.

Table 59 – Summary of Cantabro Loss with Binder Modifications

Mix ID	Total AC (%)	Fiber (%)	Average Air Voids (%)	Cantabro Loss (%)		
				Average	St Dev	COV(%)
Georgia Control	6.0	0.4	15.2	19.3	6.4	33.4
Georgia PG 76-22 SBS	6.0	0.0	16.1	12.3	6.2	50.3
Georgia PG 76-22 GTR	6.7	0.0	12.8	12.1	0.8	7.0
Georgia PG 82-22 SBS	6.0	0.0	14.4	4.7	1.1	23.2

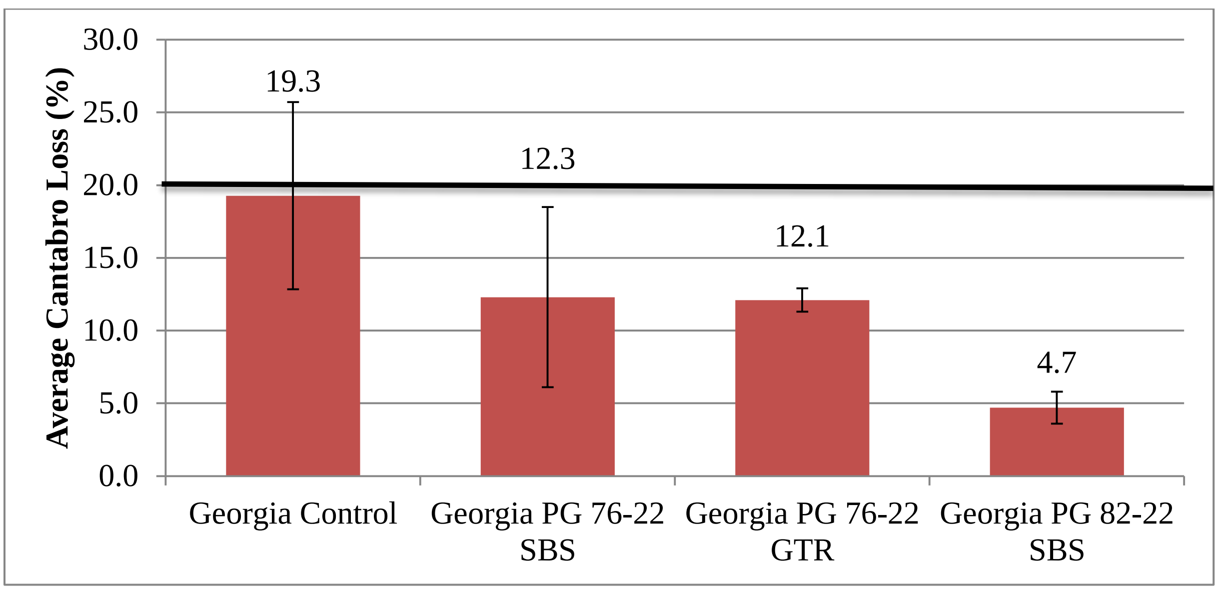


Figure 72 – Cantabro Loss for Binder Modification Designs

The ANOVA analysis showed that some of the mixtures were statistically different with a good model fit of 66.22 percent. The Georgia Control was statistically different from the HiMA design. All of the designs passed the 20.0 percent maximum loss criterion. Even though the SBS and GTR designs are not statistically different from the Control, there is numerical improvement over the Control.

Table 60 – ANOVA Analysis for Cantabro Loss with Binder Modifications

Mix ID	Cantabro Loss (%)			
	N	Mean	Grouping	
Georgia Control	3	19.3	A	
Georgia PG 76-22 SBS	3	12.3	A	B
Georgia PG 76-22 GTR	3	12.1	A	B
Georgia PG 82-22 SBS	3	4.7		B

p =0.027
R^2 = 66%

The permeability summary can be seen in Table 61, while a graphical depiction of the results is shown in Figure 73. The Control and SBS had similar permeability results which may indicate that the fiber had no effect on the permeability of the specimens. The permeability of the GTR and HiMA designs considerably decreased over the Control. The GTR design had a higher asphalt content which may explain the decrease in permeability, but the only difference in the HiMA design was the increase in the polymer dosage. The HiMA has approximately 3 times the amount of polymer as the SBS design. This increase in polymer did not decrease the air void content of the design but did significantly affect the permeability.

Table 61 – Permeability Summary with Binder Modification

Mix ID	Asphalt Content (%)	Fiber (%)	Average Air Voids (%)	Permeability (k) meters/day		
				Average	St Dev	COV(%)
Georgia Control	6.0	0.4	15.7	80	10.5	13.1
Georgia PG 76-22 SBS	6.0	0.0	16.6	79	12.2	15.3
Georgia PG 76-22 GTR	6.7	0.0	13.3	33	14.1	42.5
Georgia PG 82-22 SBS	6.0	0.0	15.1	37	9.8	26.6

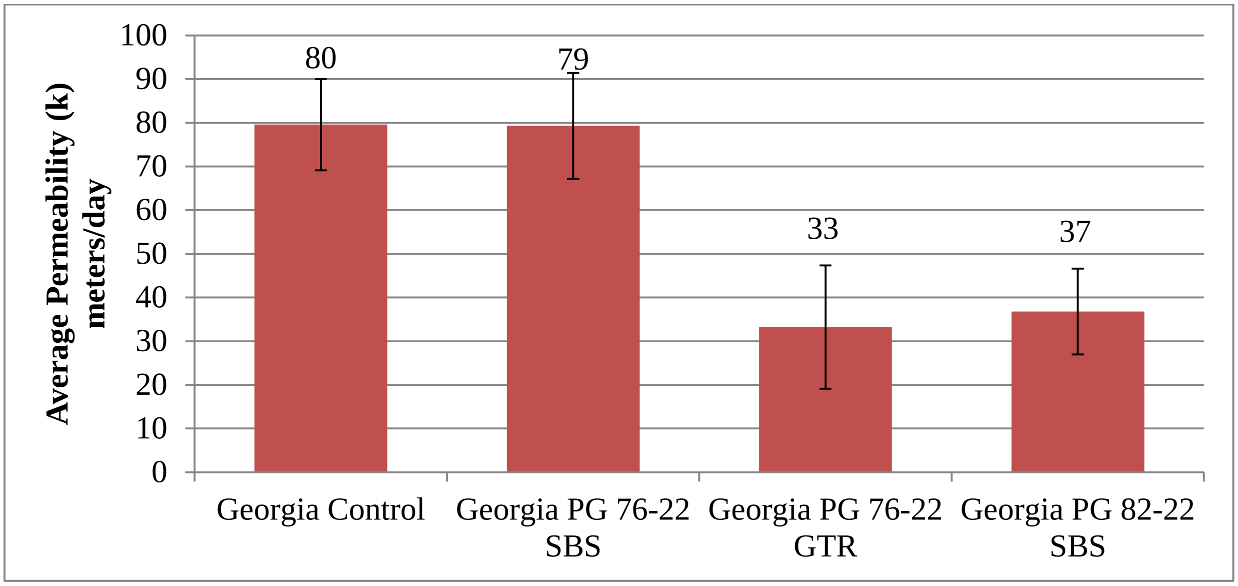


Figure 73 – Permeability Results for Binder Modification

The HWTT results (Figure 74) show that the SBS and GTR designs had a greater rut depth than the Control design. The COV, seen in Table 62, for the SBS and Control designs are relatively high. This may account for the small difference in the measured rut depth. The HiMA design had the smallest rut depth and the lowest COV. The HiMA design has the stiffest asphalt binder so these results were expected. The Control performed better than the SBS design; but given the variability for these designs, it is likely that the differences are within normal testing variability.

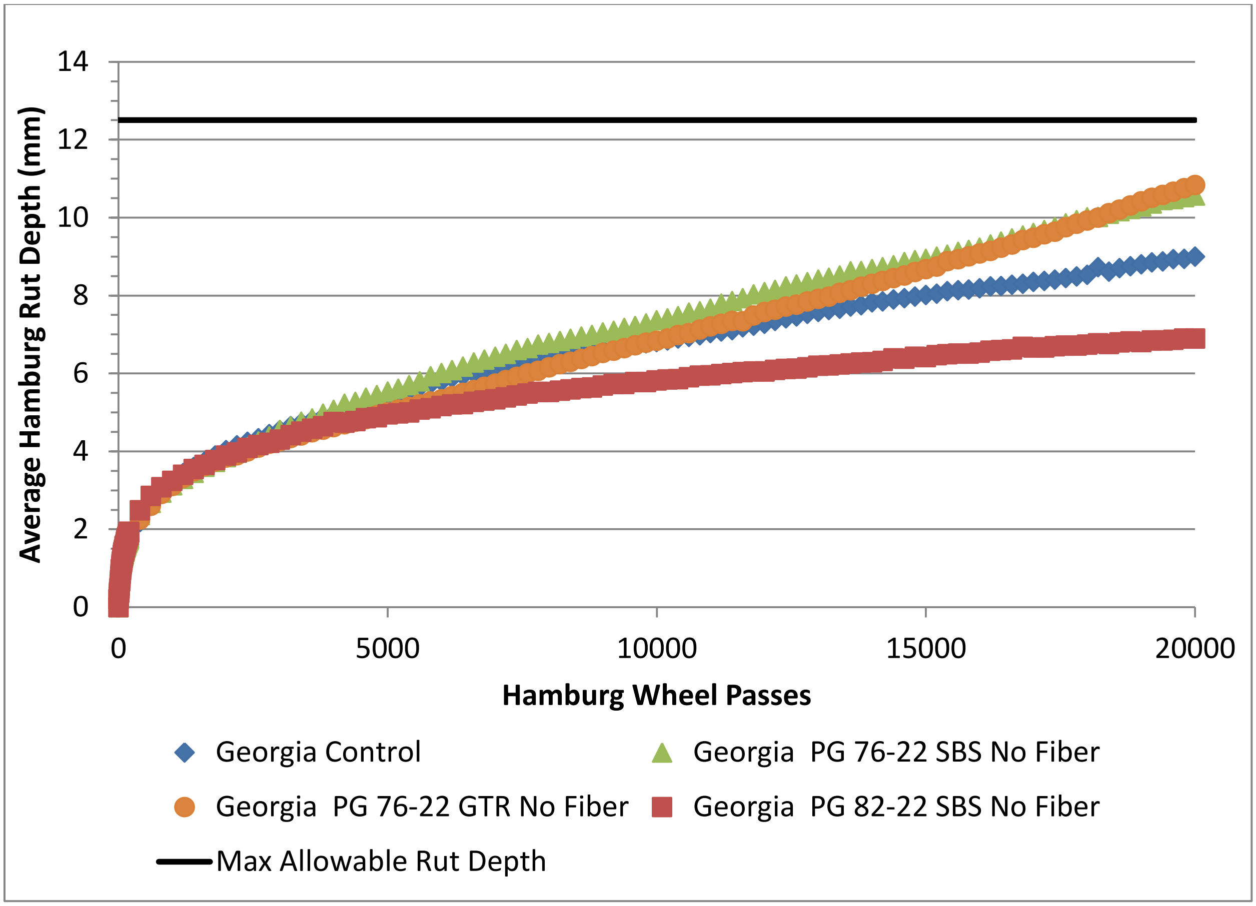


Figure 74 – HWTT Results for Binder Modification

Table 62 – HWTT Summary with Binder Modification

Mix ID	Total AC (%)	Fiber (%)	Average Air Voids (%)	Greatest Rut Depth Recorded (mm)		
				Average, mm	St Dev, mm	COV(%)
Georgia Control	6.0	0.4	14.3	8.99	2.88	32.0
Georgia PG 76-22 SBS	6.0	0.0	17.3	10.56	3.09	29.2
Georgia PG 76-22 GTR	6.7	0.0	13.5	10.84	1.64	15.1
Georgia PG 82-22 SBS	6.0	0.0	15.5	6.89	0.36	5.2

The TSR summary results are presented in Table 63. The unconditioned strengths dropped slightly for all of the designs with binder modifications, but according to Table 64 there is not a statistical difference between the unconditioned strengths. There is a difference between the conditioned strengths. The Control design was statistically different from the SBS and HiMA designs. The SBS and HiMA designs showed a decrease in both the unconditioned and conditioned strengths when compared to the Control. The GTR sample and the HiMA designs passed the ASTM requirement of 0.80 while all of the designs passed the AASHTO criterion of 0.70. The GTR design showed almost no moisture damage. This is important to note since the Florida GTR design from Part 1 also showed no sign of moisture damage. Therefore, the use of GTR in a PFC design may provide resistance to moisture damage. Additional laboratory testing with and without anti-strip agents would be able to determine if this is correct.

Table 63 – TSR Summary Results with Binder Modification

Mix ID	Total AC (%)	Fiber (%)	Average Air Voids (%)		Average ITS (psi)		TSR
			Conditioned	Unconditioned	Conditioned	Unconditioned	
GA Control	6.0	0.4	13.9	14.0	57.6	74.3	0.78
GA PG 76-22 SBS	6.0	0.0	15.9	16.0	48.8	61.6	0.79
GA PG 76-22 GTR	6.7	0.0	14.6	14.8	66.8	70.2	0.95
GA PG 82-22 SBS	6.0	0.0	13.4	13.4	56.0	63.3	0.88

Table 64 – ANOVA Analysis for ITS with Binder Modification

Mix ID	Conditioned ITS				Unconditioned ITS		
	N	Mean	Grouping		N	Mean	Grouping
Georgia Control	3	57.6	A		3	74.3	A
Georgia PG 76-22 GTR	3	66.8	A	B	3	70.2	A
Georgia PG 82-22 SBS	3	56.0		B	3	60.3	A
Georgia PG 76-22 SBS	3	48.8		B	3	61.6	A

p =0.002
R^2 = 82%

p=0.346
R^2 = 32%

The OT results for this part of the study can be seen in Table 65. The SBS and HiMA designs did not reach the 93 percent loss reduction prior to reaching 1,000 cycles so the data had to be extrapolated. The GTR design has a large COV (73.7%) while the other designs had reasonable COVs. The GTR design ranged from 112 to786 cycles to failure for this testing. There were no outliers observed in the GTR design, but there was a single outlier observed in the Control and SBS designs. Both of these outliers were removed prior to performing any analysis.

Table 65 – OT Summary for Binder Modification Designs

Mix ID	*Replicates*	*Average Air Voids (%)*	*Average Peak Load (kN)*	*Cycles to Failure*		
				Average	*St Dev*	*COV (%)*
Georgia Control	3	12.8	2.621	583	166	28.4
Georgia PG 76-22 SBS	3	13.3	2.129	2137	104	4.9
Georgia PG 76-22 GTR	4	11.6	2.977	388	286	73.7
Georgia PG 82-22 SBS	4	12.3	1.243	2877	551	19.1

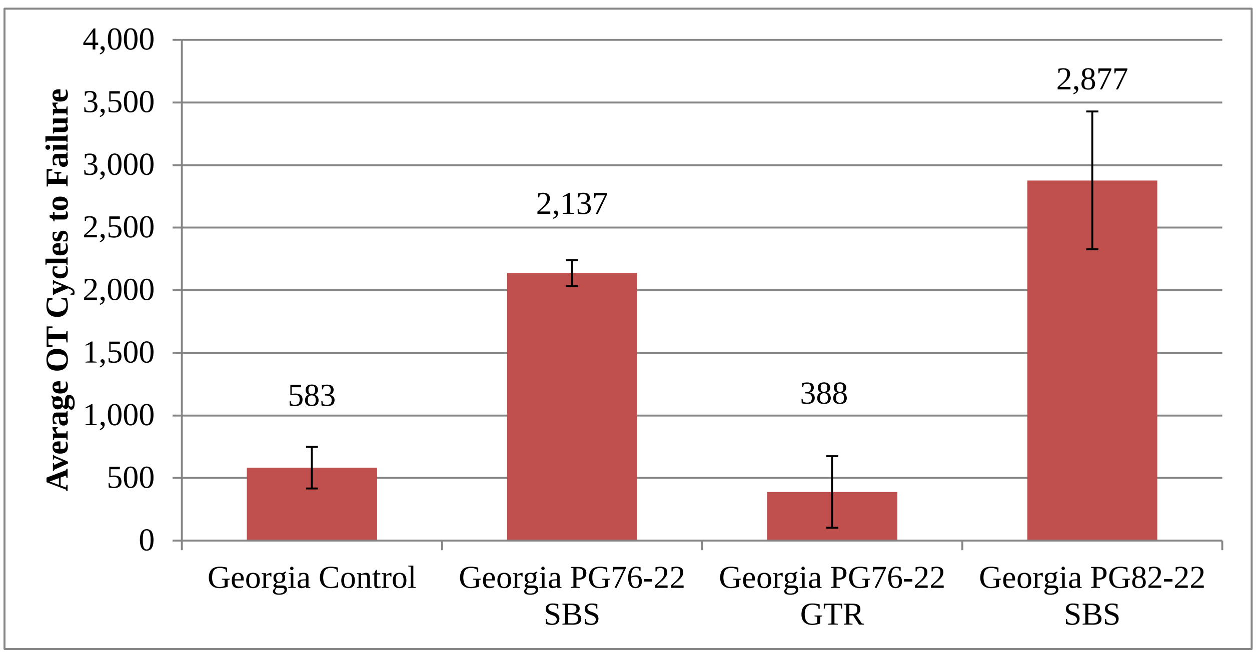


Figure 75 – OT Results for Binder Modification Designs

An ANOVA was conducted on both the peak load and cycles to failure data. Both sets of analysis have goodness of fit values greater than 90 percent which indicates that the models show a good fit with the data. The cycles to failure data (Table 66) shows that the Control and GTR designs are statistically different from the HiMA and SBS designs. This was expected since the HiMA and SBS designs did not fail prior to the 1,000 cycle cut-off and the data had to be extrapolated. The HiMA and the SBS designs are modified only with polymer and perform well according to the OT. The SBS design without fiber performed 3.5 times better than the design with fiber (Control). This may indicate that the fiber causes the mastic to stiffen and reduce the elasticity of the mix in regards to the OT.

Table 66 – ANOVA Analysis for OT Cycles to Failure Results with Binder Modification

Mix ID	Cycles to Failure		
	N	Mean	Grouping
Georgia PG 82-22 SBS	4	2877	A
Georgia PG 76-22 SBS	3	2137	A
Georgia Control	3	583	B
Georgia PG 76-22 GTR	4	388	B

$p<0.001$
$R^2 = 93\%$

The peak load analysis (Table 67) showed that the HiMA design was statistically different from the other designs. The HiMA design had the lowest peak load recorded out of the 4 designs but had the highest cycles to failure, while the GTR design had the highest peak load and the lowest recorded cycles to failure. This seems to suggest that the performance difference may be related to elasticity of the designs. The HiMA design has a significant amount of polymer which provides greater elastic recovery. This may indicate that the elasticity of the GTR is significantly less than that of the polymer.

Table 67 – ANOVA Analysis for OT Peak Load Results with Binder Modification

Mix ID	Peak Load		
	N	Mean	Grouping
Georgia PG 76-22 GTR	4	2.977	A
Georgia Control	3	2.621	A B
Georgia PG 76-22 SBS	3	2.129	B
Georgia PG 82-22 SBS	4	1.243	C

$p<0.001$
R^2 = 93%

The draindown results for this part of the study (Table 68) were the most pertinent because the stabilizing agent in the JMF (fiber) had been removed from these designs. The purpose was to see if the binder modifications would negate the need for fiber as a draindown solution while still providing a good performing mixture. All of the samples were tested at 330 and 357°F except for the HiMA design. The HiMA design required a mixing temperature of 340°F; therefore the test temperatures for the HiMA design were 340 and 367°F. The HiMA design tested at 367°F was the only sample to fail the 0.3 percent maximum draindown criterion. The samples tested at 357°F for the SBS and 340°F for the HiMA designs were close to failing. The GTR design showed almost no draindown. This may indicate that the use of GTR negates the need for fiber when incorporated in PFC mixtures. It should be noted that during fabrication of specimens for the SBS and HiMA designs, draindown was observed in the aging pans for the Cantabro and permeability specimens that were compacted at their recommended compaction temperatures (300° and 320°F). The performance samples that were aged and compacted at 275°F showed minimal draindown in the aging pans for the SBS and HiMA designs.

Table 68 – Draindown Results for Binder Modification Designs

Mix ID	Total AC (%)	Fiber (%)	Draindown (%)			
			Test Temp, °F			
			330	357	340	367
Georgia Control	6.0	0.4	0.0	0.0		
Georgia PG 76-22 SBS	6.0	0.0	0.0	0.3		
Georgia PG 76-22 GTR	6.7	0.0	0.0	0.0		
Georgia PG 82-22 SBS	6.0	0.0			0.3	1.0

The I-FIT testing with the binder modification results can be seen in the following figures. There was one outlier that was removed from the GTR design set, while the Control set also had one outlier that was removed in the Part 1 analysis. In regards to peak load, the GTR design was statistically different from the SBS and HiMA designs but not from the Control (Table 69). The G_f results showed that even though there was a numerical increase with the binder modifications over the Control, the designs were not statistically different. The HiMA design showed an obvious increase in FI over the other designs (Figure 78). It was statistically different from the other mix designs and the model had a goodness of fit of 57.43 percent (Table 71). The HiMA with the high polymer content provides more elasticity to the mixture, allowing some recovery to occur after the peak load and fracture has transpired.

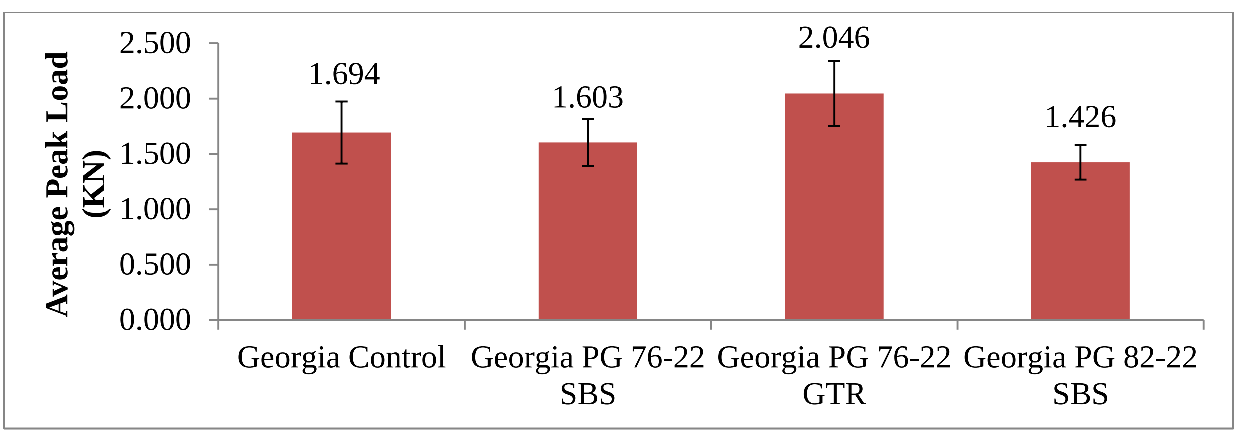


Figure 76 – I-FIT Peak Load Results for Binder Modification Designs

Table 69 – ANOVA Analysis for I-FIT Peak Load for Binder Modification Designs

Mix ID	Peak Load (KN)		
	N	Mean	Grouping
Georgia PG 76-22 GTR	5	2.046	A
Georgia Control	5	1.694	A B
Georgia PG 76-22 SBS	6	1.603	B
Georgia PG 82-22 SBS	6	1.426	B

p =0.003
R^2 = 52%

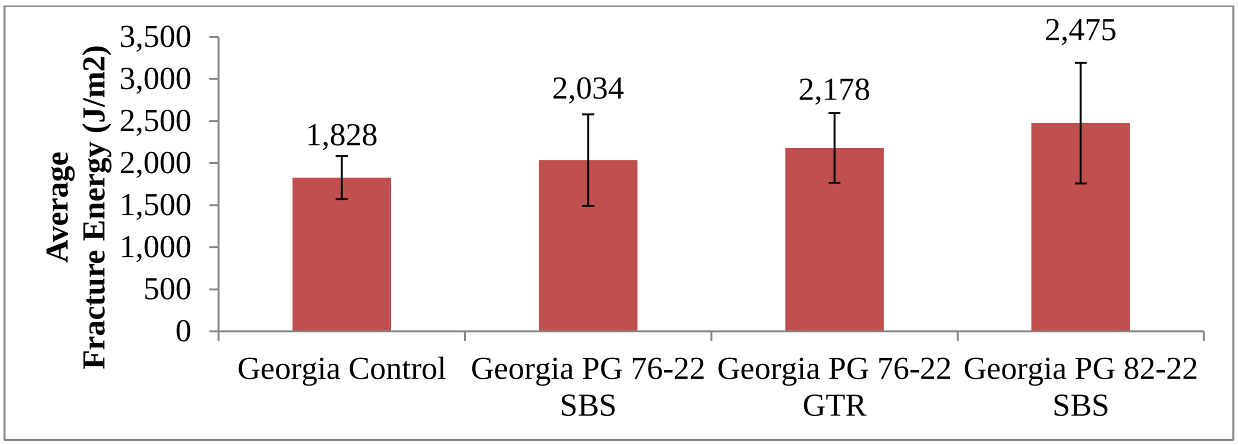


Figure 77 – I-FIT Fracture Energy for Binder Modification Designs

Table 70 – ANOVA Analysis for I-FIT Fracture Energy with Binder Modifications

Mix ID	Fracture Energy (J/m^2)		
	N	Mean	Grouping
Georgia PG 82-22 SBS	6	2475	A
Georgia PG 76-22 GTR	5	2179	A
Georgia PG 76-22 SBS	6	2034	A
Georgia Control	5	1828	A

p =0.253
R^2 = 20%

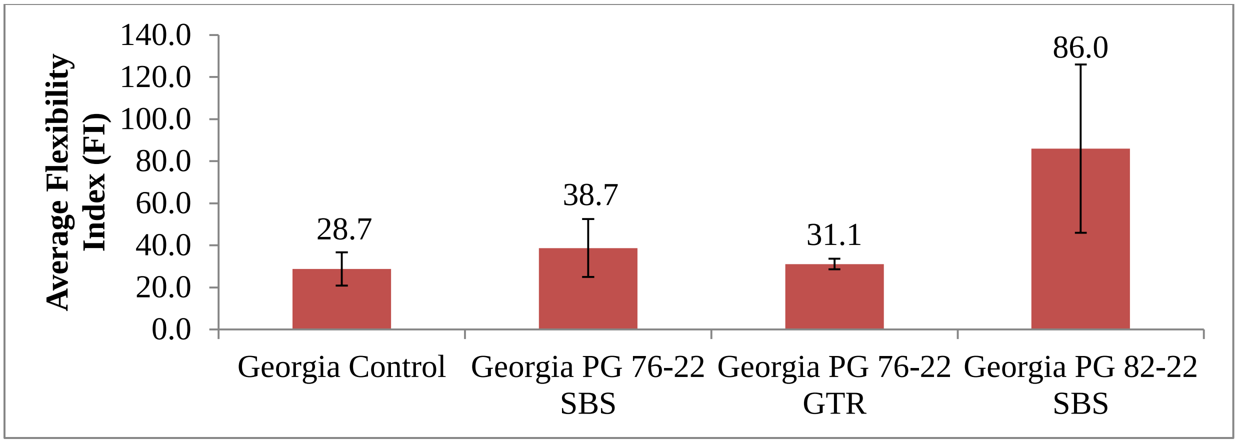


Figure 78 – I-FIT Flexibility Index for Binder Modification Designs

Table 71 – ANOVA Analysis for I-FIT Flexibility Index with Binder Modification

Mix ID	Flexibility Index		
	N	Mean	Grouping
Georgia PG 82-22 SBS	6	86.0	A
Georgia PG 76-22 SBS	6	38.7	B
Georgia PG 76-22 GTR	5	31.1	B
Georgia Control	5	28.7	B

p =0.001
R^2 = 57%

The No-Notch I-FIT specimens showed an increase in the peak load for the SBS design when compared to the Control. There was a single outlier removed from the Control and GTR designs prior to performing the analysis. The HiMA design was statistically different from the other designs with only an average peak load of 2.413 KN. The other average peak loads were greater than 3.00 KN. These results show the same trend as the OT results in regards to the peak load.

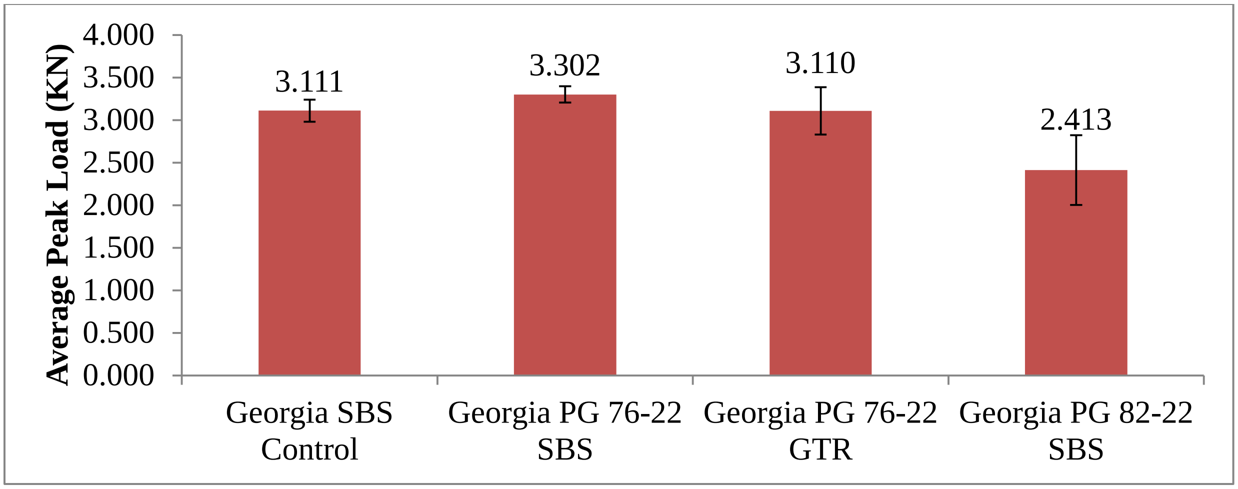


Figure 79 – No-Notch I-FIT Peak Load with Binder Modifications

Table 72 – ANOVA Analysis for No-Notch I-FIT Peak Load with Binder Modifications

Mix ID	**Peak Load (KN)**		
	N	**Mean**	**Grouping**
Georgia PG 76-22 SBS	6	3.302	A
Georgia Control	5	3.111	A
Georgia PG 76-22 GTR	5	3.110	A
Georgia PG 82-22 SBS	6	2.413	B

$p<0.001$
$R^2 = 68\%$

The G_f results show that the GTR and HiMA designs are statistically different. The G_f of the HiMA design is higher than all of the other the other designs but the reported G_f may be lower than the actual. Three of the HiMA samples hit the backstop on the I-FIT machine and never terminated. The specimens never reach the 0.1 KN cut-off therefore the machine had to be manually stopped. Since the specimens did not reach the 0.1 KN cut-off, all of the area under the curve was not attainable. The data was trimmed at its lowest point after the peak load for the G_f calculations.

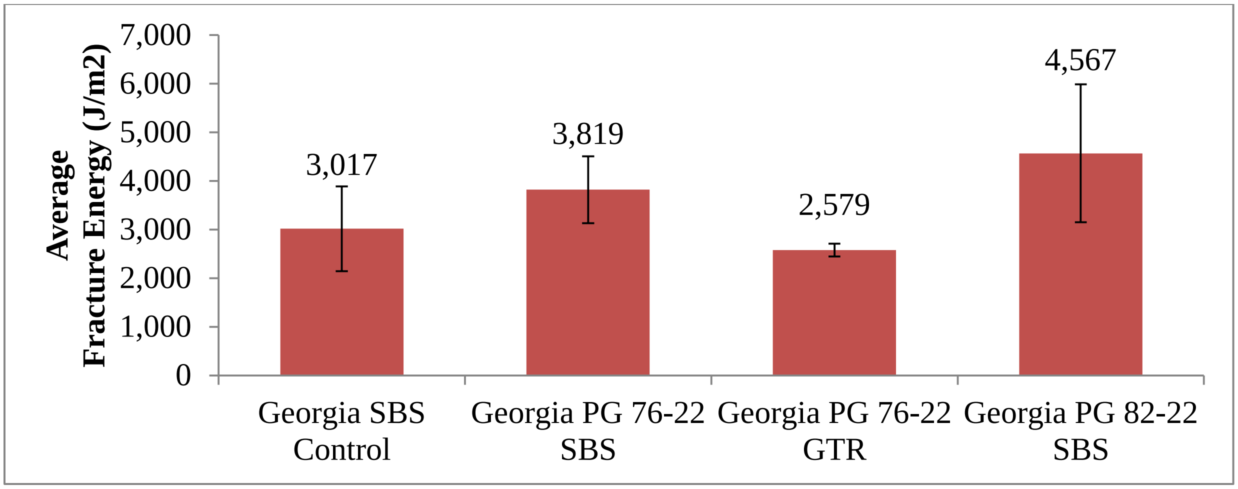


Figure 80 - No-Notch I-FIT Fracture Energy with Binder Modifications

Table 73 - ANOVA Analysis for No-Notch I-FIT Fracture Energy with Binder Modifications

Mix ID	**Fracture Energy (J/m^2)**			
	N	**Mean**	**Grouping**	
Georgia PG 82-22 SBS	6	4567	A	
Georgia PG 76-22 SBS	6	3819	A	B
Georgia Control	5	3017	A	B
Georgia PG 76-22 GTR	5	2579		B

p =0.011
R^2 = 45%

EXPERIMENT 3: NMAS TO LIFT THICKNESS RATIO

Introduction

The determination of splitting tensile strength (or ITS) at varying lift thicknesses was performed to determine if the NMAS of the mixture had an effect on ITS. This testing was important because PFC mixtures are often placed less than one-inch thick. Therefore, the standard dimensions of TSR samples for dense-graded mixes may not apply to PFC mixes, or may not be representative of field conditions for PFC mixes. Testing a 4.75 mm dense-graded mixture along with a 9.5 mm and 12.5 mm PFC mixture allowed for a comparison to be conducted. The 4.75 mm dense-graded mix was selected because it is often placed less than one-inch thick also. If ITS of the 4.75 mm mix is not effected by variation in layer thickness, it would be reasonable to assume the same may be true for PFC mixes. The 4.75 mm samples were prepared from mix produced for Lee County Road 159 test sections, and the 9.5 mm PFC was prepared from mix samples for NCAT Test Track section E9-1a. The test thicknesses of 2.5 inches, 0.75 inches and 2.5 x NMAS were chosen for the evaluation. The 2.5 inch specimens were cut from standard design specimens for the PFC mixtures and from a 95 mm tall specimen for the dense-graded mixture. The 4.75 mm dense-graded mixture specimens were fabricated according to AASHTO T 283. The specimens fabricated for the thicknesses of 0.75 inches and 2.5 x NMAS were fabricated the same as the 2.5 inch thick specimens; however, in order to conform to the AASHTO T283 specification, a 4.0 inch diameter specimen was cored from these specimens. Specimens tested according to AASHTO T283 must be 6.0 inches in diameter and greater than 2.5 inches thick. If the specimen thickness is less than 2.5 inches, the specimen diameter must be 4.0 inches. The specimens for this part of the study were conditioned in the same manner as the TSRs from the previous sections.

Results and Discussion

The specimens were all saturated and subjected to one freeze-thaw cycle prior to testing. Three specimens were tested at each thickness for each of the different NMAS mixtures. A summary of the results can be seen in Table 74. The dense-graded mixture had a larger ITS than both the PFC mixtures. An ANOVA analysis ($\alpha=0.05$) was conducted on the ITS results from each mix design to determine if ITS for the varying lift thicknesses were statistically different.

According to Table 75 the lift thickness had no effect on the 4.75mm ITS. The goodness of fit was only 40.53 percent for this analysis. The ANOVA analysis conducted on the 9.5mm (Table 76) and 12.5mm (Table 77) PFC mixtures also showed that the ITS of the varying lift thicknesses were not significantly different. The analysis for the 12.5mm mixture had a p-value of 0.071. This is close to the distinguishing value of 0.05. According to the *Statistical Sleuth* (Ramsey, 2002), a p-value of 0.05 to 0.10 is suggestive but inconclusive.

Table 74 – Summary Results for ITS Based on Lift Thickness

Mix ID	NMAS (mm)	AC (%)	Specimen Diameter, in	Specimen Thickness, in	Specimen Air Voids (%)	Avg S_T (psi)	Std Dev S_t (psi)	COV S_T
LR 159 2.5xNMAS	4.75	6.2	4.0	0.45	4.8	175	2.2	1.2
LR 159 0.75inch	4.75	6.2	4.0	0.79	6.8	183	10.0	5.5
LR 159 2.5 inch	4.75	6.2	6.0	2.50	6.4	187	8.0	4.3
E9-1a 2.5xNMAS	9.5	6.0	4.0	0.93	13.6	95	3.2	3.3
E9-1a 0.75 inch	9.5	6.0	4.0	0.79	14.8	83	6.4	7.7
E9-1a 2.5 inch	9.5	6.0	6.0	2.51	16.9	89	13.5	15.3
GA 2.5xNMAS	12.5	6.0	4.0	1.22	11.4	95	7.9	8.3
GA 0.75 inch	12.5	6.0	4.0	0.73	10.1	83	11.4	13.7
GA 2.5 inch	12.5	6.0	6.0	2.53	12.9	75	3.3	4.3

Table 75 – ANOVA Analysis of ITS for the 4.75 mm Dense-Graded Mixture

Source	DF	SS	MS	F	p
Mix ID	2	231.0	115.5	2.04	0.210
Error	6	338.9	56.5		
Total	8	569.9			

S = 7.515

R^2 = 41%

Table 76 – ANOVA Analysis of ITS for the 9.5mm PFC Design

Source	DF	SS	MS	F	p
Mix ID	2	209.1	104.5	1.34	0.330
Error	6	468.4	78.1		
Total	8	677.5			

S = 8.835

R^2 = 31%

Table 77 – ANOVA Analysis of ITS for the 12.5mm Georgia PFC Design

Source	DF	SS	MS	F	p
Mix ID	2	578.1	289.1	4.25	0.071
Error	6	408.1	68		
Total	8	986.2			

S = 8.247

R^2 = 59%

CHAPTER 7 – CONCLUSIONS AND RECOMMENDATIONS

The development of a performance-based mix design procedure for PFC mixtures should include performance testing that evaluates mixture properties relating to the two observed distresses, including raveling and cracking. This may be best accomplished by a balanced mix design approach with both durability and cracking performance criteria.

At the beginning of the study, a list of performance tests and initial performance criteria were specified (Table 10). This study showed that some of these tests and mixture properties were not valid for use in the mix design procedure. The 6 mix designs that were verified and tested for this study were compared to determine if there was a discernable difference in the results and properties for the *good* and *poor* designs. The average results and mixture properties were summarized and arranged in ascending order to see if there was a clear break between the designs and also to see where the initial estimated design requirement for each performance test fell within the data set. The following table shows all of the verification design data (Table 78).

Table 78 – Summary of Mixture Properties and Performance

Mix ID	Criterion			Results	Recommended Design Requirement
	AASHTO	ASTM	Study		
Unconditioned Cantabro, % Loss					
New Jersey - Good	15%	20%	20%	10.2	20 maximum
Georgia - Good				19.3	
Florida - Good				21.9	
Florida - Poor				23.8	
Virginia - Poor				35.1	
South Carolina - Poor				37.9	
Air Void Content, %					
Georgia - Good	18-22%	18%	15%	15.4	15 -22
Florida - Good				17.1	
Florida - Poor				17.7	
New Jersey - Good				19.5	
Virginia - Poor				21.8	
South Carolina - Poor				22.2	
Film Thickness, microns					
New Jersey - Good	Not Defined	Not Defined	24	18.6	Not Applicable
Virginia - Poor				25.4	
Georgia - Good				27.1	
Florida - Poor				32.5	
South Carolina - Poor				34.7	
Florida - Good				35.9	

VCA Ratio Using #4 Sieve as Breakpoint					
Georgia - Good	1.00	1.00	1.00	1.00	Not Applicable
Florida - Good				1.07	
Florida - Poor				1.07	
South Carolina - Poor				1.09	
Virginia - Poor				1.18	
New Jersey - Good				1.27	
VCA Ratio Using #8 Sieve as Breakpoint					
Florida - Poor	1.00	1.00	1.00	0.81	Not Applicable
Georgia - Good				0.82	
Florida - Good				0.82	
New Jersey - Good				0.92	
South Carolina - Poor				0.93	
Virginia - Poor				0.93	
VMA, %					
Florida - Poor	Not Defined	Not Defined	Minimum Point on Curve	26.2	Not Applicable
Georgia - Good				26.3	
Florida - Good				26.4	
New Jersey - Good				31.1	
South Carolina - Poor				32.3	
Virginia - Poor				32.9	
Permeability, meters/day					
Florida - Good	100	100	100	77	50 minimum
Georgia - Good				80	
Florida - Poor				107	
New Jersey - Good				186	
South Carolina - Poor				209	
Virginia - Poor				237	

Draindown using #8 Mesh Basket at High Test Temperature, %					
Georgia - Good	0.3%	0.3%	0.3%	0.0	0.3%
New Jersey - Good				0.0	
Florida - Good				0.0	
Florida - Poor				0.0	
South Carolina - Poor				0.0	
Virginia - Poor				0.0	
HWTT - Greatest Rut Depth Recorded, mm					
New Jersey - Good	Not Defined	Not Defined	12.5	6.39	12.5mm Max for 20,000 passes for PG 76-22
Florida - Poor				6.81	
Virginia - Poor				7.04	
Florida - Good				8.47	
Georgia - Good				8.99	
South Carolina - Poor				15.85	
Conditioned Indirect Tensile Strengths, psi					
South Carolina - Poor	Not Defined	Not Defined	50	36.7	50 minimum
Florida - Poor				52.8	
Virginia - Poor				53.2	
Florida - Good				54.0	
Georgia - Good				57.7	
New Jersey - Good				64.5	
Unconditioned Indirect Tensile Strengths, psi					
South Carolina - Poor	Not Defined	Not Defined	Not Defined	45.2	70 minimum
Florida - Good				50.1	
Virginia - Poor				59.5	
Florida - Poor				72.5	
Georgia - Good				74.3	
New Jersey - Good				76.2	

Tensile Strength Ratio (TSR)					
Florida - Poor	0.70	0.80	0.70	0.73	0.70 minimum
Georgia - Good				0.78	
South Carolina - Poor				0.81	
New Jersey - Good				0.85	
Virginia - Poor				0.89	
Florida - Good				1.08	
Overlay Tester, Cycles to Failure					
Florida - Good	Not Defined	Not Defined	200	67	Not Applicable
Florida - Poor				370	
Georgia - Good				583	
Virginia - Poor				1,291	
South Carolina - Poor				1,491	
New Jersey - Good				1,866	
SCB I-FIT Flexibility Index					
Florida - Poor	Not Defined	Not Defined	8.0	23.5	25.0 minimum
Florida - Good				25.2	
Georgia - Good				28.7	
New Jersey - Good				35.6	
Virginia - Poor				57.5	
South Carolina - Poor				57.7	
No-Notch SCB I-FIT Peak Load, KN					
South Carolina - Poor	Not Defined	Not Defined	Not Defined	1.645	2.750 minimum
Florida - Good				2.057	
Virginia - Poor				2.295	
New Jersey - Good				2.951	
Georgia - Good				3.111	
Florida - Poor				3.286	

The most common requirement for a PFC design is a minimum air void content. The AASHTO standard recommends a design air void range of 18-22 percent while ASTM requires a minimum design air void content of 18 percent. The initial estimation for this study was a minimum design air void level of 15 percent. The air voids for this study showed that three of the designs, the Georgia and the two Florida designs, had a design air void content below 18 percent; however, all of the designs were above the initial estimation of a 15 percent minimum. The South Carolina design with an air void content of 22.2 percent was outside the acceptable range according to the AASHTO standard. Based on the permeability results shown in Figure 44, the minimum amount of air void content needed to achieve a permeability rate of 100 meters/day is approximately 17.0 percent. If the permeability rate of 100 meters/day is essential, a minimum design air void content of 17.0 percent will be necessary. The permeability data showed that 2 of the *good* mix designs had rates of 80 or less. There is a direct correlation between air voids and permeability rates, so it is not surprising that two of the designs failed the current recommended permeability rate. With these two *good* designs having good field performance, this may indicate that the rate of 100 meters/day may be higher than would be needed. If a permeability rate of 50 meters/day is used, then a corresponding minimum design air void content of 15.0 percent would be required. Based on the data analyzed for this study it is recommended that the design range of 15-22 percent air void content and minimum permeability rate of 50 meters/day be used for PFC design.

Based on the results from this study, it was determined that film thickness, VMA and VCA ratio were not necessary for designing PFC mixtures. It was initially estimated that a minimum film thickness of 24 microns would be needed to provide a good performing PFC design. The AASHTO and ASTM standards do not specify a film thickness calculation for design purposes. All of the designs, with the exception of New Jersey, have film thicknesses greater than the estimated 24 microns. The good performing New Jersey design had a film thickness of 18.6 microns while the poor performing South Carolina design had a film thickness of 34.7 microns. This range in film thickness values along with having a good performing mix fail seems to indicate that film thickness is not as vital to the performance of PFC pavements as originally thought. The VMA for each design was conducted across three asphalt contents, and a VMA curve was plotted. The initial estimation for this study was that designing at the bottom of the VMA curve may be beneficial to mixture performance, as it is in dense-graded mixtures. There were distinct differences in the designs according to the calculated VMA, but the differences were not a split between good and poor performing designs. The Florida designs and the Georgia design showed VMA values of approximately 26 while the New Jersey, South Carolina and Virginia designs showed VMAs of approximately 32. This large difference seems to indicate that recommending a minimum VMA value would not be beneficial. In addition, the VMA curve for each design was practically non-existent. The VMA values across a range of asphalt contents for each design showed no relative change. The change in G_{mb} for some of the specimens was so minuscule that even some of the data points created an inverse parabolic "curve". Designing at the bottom of the VMA curve for PFC mixtures will not provide any benefit to the mixture performance.

The VCA ratio for the designs varied but did not show a distinct difference between the *good* and *poor* designs. Initially all of the designs had a ratio of 1.00 or greater, but after further investigation it was decided to perform the testing according to an alternative method. The ASTM standard states that the coarse aggregate in the mixture is defined as the material retained on the No. 4 sieve. This method was initially performed and resulted in failing VCA ratios for five of the six designs. The proportion of coarse aggregate, according to AASHTO, is defined by the breakpoint sieve. The breakpoint sieve is the finest sieve with at least 10 percent material retained. The AASHTO method was performed using the breakpoint sieve to define the coarse aggregate. Using this method all of the VCA ratios passed the requirement of 1.00 or less. There was a distinct split between the designs, and again it was the same split as it was for the VMA calculations. The Florida designs and Georgia had approximate values of 0.82 and the New Jersey, Virginia and South Carolina designs were approximately 0.92. If VCA ratio is to be used in design, it should be stated that the coarse aggregate is defined by the breakpoint sieve which is the finest sieve to have at least 10 percent aggregate retained. Due to the small portion of fine aggregate in the PFC designs, the VCA ratio showed no significant difference in the data. It is not recommended that VCA be used for design purposes.

The draindown results showed that none of the designs exhibited draindown. This is most likely due to the fact that all the designs had cellulose fiber added to the mixture.

The performance testing conducted for determining mix durability (Cantabro and HWTT) was fairly successful in distinguishing between the *good* from the *poor* designs. The Cantabro criterion for the ASTM standard is 20 percent loss for unaged specimens. The AASHTO standard requires a maximum of 15 percent loss. The initial estimation for this study was a maximum of 20 percent loss. According to the initial estimate and the ASTM standard, only the Georgia and New Jersey designs passed the 20 percent maximum loss criterion. The Georgia design (19.3%) failed the AASHTO criterion. The Florida *good* design was close to passing the ASTM standard with a percent loss of 21.9 percent. The HWTT results showed little variability in the results for each design, with the average COV being only 14.2 percent. The South Carolina design was the only design to fail the Texas criterion of a maximum 12.5mm rut depth before

20,000 passes. All of the other designs had an approximate average of 7.5 mm of rut depth. The HWTT did not differentiate between all of the *good* and *poor* mixtures. It did however screen the poor performing South Carolina design. For a balanced mix design approach it is recommended that the HWTT, using the current TxDOT criteria, be used for designing PFC mixtures. The Cantabro test with a maximum of 20 percent loss for unaged specimens is also recommended for determining the durability of the designs in regards to raveling susceptibility.

Determining the durability of the designs should also be evaluated in terms of cohesiveness. The ITS is a good indication of a mixture's strength in terms of cohesiveness. Thus, it along with a modified I-FIT test was used to determine if a *good* mixture's cohesiveness could be differentiated from a *poor* mixture's cohesiveness. The ITS of a mixture is calculated as part of the modified AASHTO T283 test procedure. For this study, it was estimated that a minimum conditioned ITS of 50 psi should be required for design while no initial unconditioned limit was set. All of the designs except South Carolina had conditioned strengths greater than 50 psi. Since most of the designs met the initial conditioned strength criterion the unconditioned strengths were also analyzed to determine is a minimum unconditioned strength criterion should also be required. The South Carolina, Florida *good* and Virginia designs had unconditioned ITS values of less than 60 while the Georgia, New Jersey and Florida *poor* designs had unconditioned ITS values over 70. This discernable difference between these groups was also observed in the modified No-Notch I-FIT testing. The peak load for the South Carolina, Florida *good* and Virginia designs was less than 2.300 KN while the Georgia, New Jersey and Florida *poor* designs had peak loads greater than 2.900 KN. Both of these tests show the same trends and groupings in regards to the mixture's cohesiveness; therefore, whichever test is easier to perform should be recommended for design purposes. Determining a mixture's moisture susceptibility according to AASHTO T283 is a vital part of PFC design, so the most efficient method will be determining a mixture's cohesiveness in terms of conditioned and unconditioned ITS. It is recommended that a minimum value of 50 psi for conditioned strength (or a minimum of 70 psi for unconditioned strength) be chosen. If the peak load of the No-Notch I-FIT test is chosen to determine a mixture's cohesiveness, then a minimum value of 2.750 KN is recommended. The use of TSR alone as a predictor of performance appears to be insufficient since the Virginia design out-performed the New Jersey and Georgia designs. It is recommended that the 0.70 criterion for TSR be kept but the minimum ITS should be specified as part of the procedure.

The OT and I-FIT testing were used to evaluate the design's susceptibility to cracking. Neither of these procedures is currently included in the AASHTO or ASTM design procedure. The OT test is included in the Texas design procedure, but it only provides a criterion for fine-graded PFC designs. The criterion is a minimum of 300 cycles while the initial estimation for this study was a minimum of 200 cycles. All of the designs met this criterion with the exception of the Florida *good* design (67 cycles). The same split that was seen for the VMA and VCA ratio was observed in the OT test results as well. There was a distinct split between the designs. The Florida designs and the Georgia design had less than 600 cycles to failure while the New Jersey, Virginia and South Carolina designs had 1,200 or more cycles to failure. The FI calculated from the I-FIT testing results showed the same split in data, with the lower group averaging 25 and the higher group averaging approximately 45. The recommendation is to use the I-FIT test to determine the cracking susceptibility of the designs. While the OT also showed similar trends, the I-FIT test ranks the designs in the expected order, while the OT shows a disconnect between the *good* and *poor* designs. It was expected that the lower strength mixes would perform better in terms of cracking resistance, but it was deemed more important to design with strength and cohesiveness in mind to prevent raveling. If the I-FIT test is recommended, a minimum FI of 25 is recommended for design.

The following table (Table 79) provides the recommended criteria for a performance-based PFC mix design.

Table 79 – Recommended Criteria for Performance-Based PFC Design

Property	Criteria
Unconditioned Cantabro Loss, %	20 max
Air Void Content, %	15 -22
Permeability, meters/day	50 min
Draindown at Production Temperature, %	0.30 max
Hamburg Wheel Tracking Test, mm (PG 76-22 @ 20,000 passes)	12.5mm max
Tensile Strength Ratio	0.70 min
Conditioned Indirect Tensile Strengths, psi	50 min
Unconditioned Indirect Tensile Strengths, psi[1]	70 min
No-Notch SCB I-FIT Peak Load, KN[1]*(Optional)*	2.750 min
SCB I-FIT Flexibility Index	25.0 min

[1] Only one of these is required for determining cohesiveness

The evaluation of the increased P-200 specimens showed marked improvement in terms of durability and cohesiveness of the designs. There were conflicting data for the performance tests that assess cracking susceptibility. A summary of the data for each test is provided in Table 80. The data are arranged in ascending order and the performance for each design is based on the average improvement over the Control design. The increase in P-200 material had a negative effect on the air void content and permeability rates, but provided improvement for all other testing. The I-FIT results showed some improvement in FI for the Georgia design with +4BHF, but all other increased P-200 designs showed a decrease in FI. It is unclear why this was the case for the FI and not so for the OT. The OT data for this part of the study unexpectedly showed an increase in cycles to failure with increased P-200 for both the Georgia and South Carolina designs. It was anticipated that the increased P-200 would reduce the amount of effective binder and therefore reduce the cracking resistance. The results from Part 1 indicated that the OT and I-FIT test were equivocal when trying to determine a mixture's cracking potential. The OT and I-FIT test for this part of the study did not show the same trends. The OT did not provide the expected results, while the I-FIT test showed the expected trend, with the exception of the Georgia +4BHF. For the HWTT test, the data in this section shows how effective increased P-200 content is at increasing the durability of the designs. The Georgia and South Carolina data for the Cantabro and HWTT testing showed improvement over the Control. The ITS and No-Notch I-FIT specimens also show a significant increase in performance over the Control. This indicates that an increase in P-200 should improve durability, and possibly crack resistance; it also will increase the cohesiveness of the mixtures. While the TSR did not show improvement for the designs with increased P-200 content, the unconditioned and conditioned ITS were improved over the Control. This reinforces the recommendation that the ITS criterion be included when determining the TSR. As shown in Figure 60, the South Carolina and Georgia design showed increased improvement in terms of percent Cantabro loss until around a P-200 content of approximately 6.0 percent (although some mixture showed a trend of improvement up to around 8 percent P-200). At this point the, percent loss either showed negligible improvement or a small increase in percent loss. Typically, agencies specify a P-200 maximum of either 4 or 5 percent for PFC designs (Table 2). The AASHTO design procedure specifies 0 to 4 percent, and ASTM specifies 2 to 4 percent P-200. Based on this study, the gradation band for the No. 200

sieve should be revised to 2-8 percent. As shown in the Literature Review section, there are currently other international agencies that allow up to 8 percent. This research seems to indicate that specifying a gradation band of 2-8 percent will provide more durable mixes.

Table 80 – Summary of Mixture Property Improvement with Increased P-200

Mix ID	Recommended Design Requirement	Results	Improvement Over Control
Unconditioned Cantabro, % Loss			
Georgia +4BHF	20	9.3	Yes
Georgia +2BHF		13.0	Yes
Georgia Control		19.3	
South Carolina +4BHF		16.9	Yes
South Carolina +2BHF		18.9	Yes
South Carolina Control		37.9	
Air Void Content, %			
Georgia +2BHF	15 - 22	12.8	No
Georgia +4BHF		13.1	No
Georgia Control		15.4	
South Carolina +4BHF		19.3	No
South Carolina +2BHF		20.7	No
South Carolina Control		22.2	

Film Thickness, microns			
Georgia +4BHF	Not Applicable	13.9	No
Georgia +2BHF		17.3	No
Georgia Control		27.1	
South Carolina +4BHF		14.8	No
South Carolina +2BHF		19.8	No
South Carolina Control		34.7	
VCA Ratio Using #4 Sieve as Breakpoint			
Georgia +4BHF	Not Applicable	0.97	Yes
Georgia +2BHF		0.98	Yes
Georgia Control		1.00	
South Carolina +4BHF		1.06	Yes
South Carolina +2BHF		1.07	Yes
South Carolina Control		1.09	
VCA Ratio Using #8 Sieve as Breakpoint			
Georgia +4BHF	Not Applicable	0.79	Yes
Georgia +2BHF		0.80	Yes
Georgia Control		0.82	
South Carolina +4BHF		0.91	Yes
South Carolina +2BHF		0.92	Yes
South Carolina Control		0.93	
VMA, %			
Georgia +2BHF	Not Applicable	23.9	No
Georgia +4BHF		24.4	No
Georgia Control		26.6	
South Carolina +4BHF		29.8	No
South Carolina +2BHF		30.8	No
South Carolina Control		32.3	

Permeability, meters/day			
Georgia +4BHF	50	38	No
Georgia +2BHF		43	No
Georgia Control		80	
South Carolina +4BHF		196	No
South Carolina Control		209	
South Carolina +2BHF		222	Yes
Draindown using #8 Mesh Basket at High Test Temperature, %			
Georgia Control	0.3%	0.0	
Georgia +2BHF		0.0	No
Georgia +4BHF		0.0	No
South Carolina Control		0.0	
South Carolina +2BHF		0.0	No
South Carolina +4BHF		0.0	No
HWTT - Greatest Rut Depth Recorded, mm			
Georgia +4BHF	12.5mm Max for 20,000 passes for PG 76-22	5.36	Yes
Georgia +2BHF		5.54	Yes
Georgia Control		8.99	
South Carolina +4BHF		12.81	Yes
South Carolina +2BHF		15.14	Yes
South Carolina Control		15.85	
Conditioned Indirect Tensile Strengths, psi			
Georgia Control	50 min	57.6	
Georgia +4BHF		82.4	Yes
Georgia +2BHF		86.4	Yes
South Carolina Control		36.8	
South Carolina +2BHF		38.8	Yes
South Carolina +4BHF		54.4	Yes

Unconditioned Indirect Tensile Strengths, psi			
Georgia Control	70 min	74.3	
Georgia +4BHF		99.8	Yes
Georgia +2BHF		100.3	Yes
South Carolina Control		45.2	
South Carolina +2BHF		62.0	Yes
South Carolina +4BHF		77.3	Yes
Tensile Strength Ratio (TSR)			
Georgia Control	0.70	0.78	
Georgia +4BHF		0.83	Yes
Georgia +2BHF		0.86	Yes
South Carolina +2BHF		0.63	No
South Carolina +4BHF		0.70	No
South Carolina Control		0.81	
Overlay Tester, Cycles to Failure			
Georgia Control	Not Applicable	583	
Georgia +2BHF		682	Yes
Georgia +4BHF		941	Yes
South Carolina Control		1,491	
South Carolina +4BHF		1,662	Yes
South Carolina +2BHF		2,335	Yes
SCB I-FIT Flexibility Index			
Georgia +2BHF	25.0	21.8	No
Georgia Control		28.7	
Georgia +4BHF		39.1	Yes
South Carolina +4BHF		34.8	No
South Carolina +2BHF		36.3	No
South Carolina Control		57.7	

No-Notch SCB I-FIT Peak Load, KN			
Georgia Control	2.750	3.111	
Georgia +2BHF		4.232	Yes
Georgia +4BHF		4.519	Yes
South Carolina Control		1.645	
South Carolina +2BHF		2.216	Yes
South Carolina +4BHF		2.627	Yes

The binder modification designs were expected to show that the need for fiber may be negated with the use of a binder modifier. This, along with an anticipated increase in performance, could prove beneficial to the industry. The effect of binder modification had mixed effects on the mixture performance when compared to the Georgia Control. A summary of the data for all of this testing can be seen in Table 81. The data are arranged in ascending order and the performance for each design is based on improvement over the Control design. The Cantabro data showed improvement for all of the modified designs, but the HWTT only improved for the HiMA design. The air voids of the designs decreased for the HiMA and GTR but increased slightly for the SBS design. The permeability rates of the SBS design did not change, but the GTR and HiMA had significant decreases in permeability. The decrease in air voids and permeability for the GTR design was expected due to the higher asphalt content of the mix. The VMA changed so little that the change observed is most likely due to variability in specimen fabrication. While the VCA ratio did improve, this is due to absence of the fiber. The fiber is part of the mix and is therefore part of the calculation when determining percent coarse aggregate.

The draindown for this section was the most critical evaluation. With the removal of the fiber (stabilizing agent) the mixture relied solely on the binder modification to prevent draindown. The only design to fail the draindown test was the HiMA design at the high test temperature (367°F). The SBS design showed evident draindown at the high temperature but remained within the recommended criteria. The FI data showed increased performance for all of the modified designs, and the OT performance was better for the HiMA and SBS design but not for the GTR. The GTR sample had worse performance than the Control, which seems to trend with the Florida *good* design (GTR modified) performing so poorly compared to the Florida *poor* design (SBS modified). GTR modification proved effective at combatting draindown of the design but did not improve the performance results when compared to the Control. One important thing to note is the increase in performance of the SBS design without fiber over the Control mixture with fiber in regards to cracking performance. The fiber in the design may be reducing the elastic recovery of the mixture and therefore decreasing the design's cracking resistance. The use of GTR as a binder modification will prove effective in terms of draindown and moisture susceptibility, but based on this data it may be more prone to cracking than polymer modified designs. The HiMA design, while providing improvement in most of the performance tests, decreased the permeability of the mix while also failing the draindown criteria. With the HiMA design failing the draindown criteria, the addition of fiber or an alternative stabilizing agent will be needed. The SBS mixture was the same design as the Control mixture with the exception of the fiber. As can be seen in Table 81, the SBS design improved over the Control in regards to durability (Cantabro) and crack resistance (OT and FI) but decreased in terms of cohesiveness (ITS and No-Notch I-FIT). This seems to indicate that using fiber as a stabilizing additive decreases the elasticity of the binder. In addition to the decrease in cohesiveness, the SBS design was borderline failing the draindown criteria. If measures were implemented to keep mixture temperatures consistent through the production process, a lower mixing temperature for PFC designs could mitigate draindown. This would allow fiber to be omitted from the mix and therefore create a better performing PFC with limited draindown. There is an alternative approach to the draindown test that includes the amount of material retained on the mesh basket as part of the draindown percentage. This alternative approach is similar to the Schellenberg method mentioned in the Literature Review. This alternative approach was used along with the regular draindown testing for this section to see if there was a significant difference in the amount of draindown recorded. The results showed that the mesh basket retained a large amount of asphalt binder and therefore increased the "percent draindown" of the designs. The GTR and Control did not change significantly but the SBS and HiMA designs had significantly more recorded draindown with this alternative approach. This seems to indicate that fiber provides a significant benefit for mixtures that have some recorded draindown from the AASHTO procedure. Further studies should be conducted to determine if this alternative approach would benefit the industry and if it should be included in the AASHTO procedure.

Table 81 - Summary of Mixture Properties and Performance with Binder Modifications

Mix ID	Recommended Design Requirement	Results	Improvement Over Control
Unconditioned Cantabro, % Loss			
Georgia PG 82-22 SBS	20	4.7	Yes
Georgia PG 76-22 GTR		12.1	Yes
Georgia PG 76-22 SBS		12.3	Yes
Georgia Control		19.3	
Air Void Content, %			
Georgia PG 76-22 GTR	15 - 22	13.9	No
Georgia PG 82-22 SBS		14.9	No
Georgia Control		15.4	
Georgia PG 76-22 SBS		16.3	Yes
Film Thickness, microns			
Georgia PG 76-22 SBS	Not Applicable	25.2	No
Georgia PG 82-22 SBS		26.1	No
Georgia Control		27.1	
Georgia PG 76-22 GTR		28.2	Yes
VCA Ratio Using #4 Sieve as Breakpoint			
Georgia PG 76-22 GTR	Not Applicable	0.99	No
Georgia PG 82-22 SBS		0.99	No
Georgia Control		1.00	
Georgia PG 76-22 SBS		1.00	No
VCA Ratio Using #8 Sieve as Breakpoint			
Georgia PG 82-22 SBS	Not Applicable	0.79	Yes
Georgia PG 76-22 GTR		0.80	Yes
Georgia PG 76-22 SBS		0.81	Yes
Georgia Control		0.82	

VMA, %			
Georgia PG 82-22 SBS	Not Applicable	25.5	No
Georgia PG 76-22 GTR		25.9	No
Georgia PG 76-22 SBS		26.5	No
Georgia Control		26.6	
Permeability, meters/day			
Georgia PG 76-22 GTR	50	33	No
Georgia PG 82-22 SBS		37	No
Georgia PG 76-22 SBS		79	No
Georgia Control		80	
Draindown using #8 Mesh Basket at High Test Temperature, %			
Georgia Control	0.3%	0.0	
Georgia PG 76-22 GTR		0.0	No
Georgia PG 76-22 SBS		0.3	No
Georgia PG 82-22 SBS		1.0	No
HWTT - Greatest Rut Depth Recorded, mm			
Georgia PG 82-22 SBS	12.5mm Max for 20,000 passes for PG 76-22	6.89	Yes
Georgia Control		8.99	
Georgia PG 76-22 SBS		10.56	No
Georgia PG 76-22 GTR		10.84	No
Conditioned Indirect Tensile Strengths, psi			
Georgia PG 76-22 SBS	50 min	48.8	No
Georgia PG 82-22 SBS		56.0	No
Georgia Control		57.6	
Georgia PG 76-22 GTR		66.8	Yes

Unconditioned Indirect Tensile Strengths, psi			
Georgia PG 76-22 SBS	70 min	61.6	No
Georgia PG 82-22 SBS		63.3	No
Georgia PG 76-22 GTR		70.2	No
Georgia Control		74.3	
Tensile Strength Ratio (TSR)			
Georgia Control	0.70	0.78	
Georgia PG 76-22 SBS		0.79	Yes
Georgia PG 82-22 SBS		0.88	Yes
Georgia PG 76-22 GTR		0.95	Yes
Overlay Tester, Cycles to Failure			
Georgia PG 76-22 GTR	Not Applicable	388	No
Georgia Control		583	
Georgia PG 76-22 SBS		2,137	Yes
Georgia PG 82-22 SBS		2,877	Yes
SCB I-FIT Flexibility Index			
Georgia Control	25.0	28.7	
Georgia PG 76-22 GTR		31.1	Yes
Georgia PG 76-22 SBS		38.7	Yes
Georgia PG 82-22 SBS		86.0	Yes
No-Notch SCB I-FIT Peak Load, KN			
Georgia PG 82-22 SBS	2.750	2.413	No
Georgia PG 76-22 GTR		3.110	No
Georgia Control		3.111	
Georgia PG 76-22 SBS		3.302	Yes

The lift thickness evaluation proved that the thin lifts in which most PFC pavements are placed do not have an effect on the durability of the mixture. The ITS of the 4.75 mm dense-graded mixture was significantly larger than the ITS of the PFC designs. However, the varying lift thicknesses were not statistically different. The 9.5 mm and 12.5 mm PFC designs also showed no statistical difference in ITS when comparing the varying lift thicknesses.

The following points provide some of the more important facts discovered during this study:

- Variations in asphalt content had little to no effect on mixture VMA (Table 22 and Figure 43).
- A Cantabro loss of 20% appears to be a reasonable threshold for mix design (Figure 37).
- Film thickness does not seem to be a critical property for performance of PFC (Table 22 and Figure 40).
- VCA testing is not recommended but if it is tested, VCA_{MIX} should be ≤ VCA_{DRC}. For VCA calculations, use breakpoint sieve rather than designating a specific sieve for all mixes. Recommend breakpoint sieve be defined as the finest sieve for which there is at least 10% of total aggregate retained.
- Permeability is directly related to air voids.
- Need to reevaluate 100 m/day permeability criteria. Recommend a value of approximately 50 meters/day.
- OT results do not rank mixes according to field performance as some mixes failed due to raveling before the occurrence of cracking.
- Adding the initial 2% BHF reduced Cantabro loss the equivalent of adding 1% more AC for both the South Carolina and Georgia designs (Figure 59).
- Adding 4% BHF showed little to no improvement in mix performance except that conditioned ITS was increased by more than 40% (Table 45).
- Results show P-200 optimized between 4-6% with some mix showing improvement trend up to about 8 percent. Recommend revising P-200 gradation range to 2-8 percent passing.

- GTR improved resistance to moisture damage (Table 63) and mitigated draindown.
- Use of HiMA reduced Cantabro loss and improved rutting resistance (Figure 72 and Figure 74) but reduced permeability (Figure 73).
- Elasticity of GTR mixes may be less than SBS mixes (Table 66).
- Recommend minimum Va of 15% based on Corelok, or minimum of 17% based on dimensional methods (based on graph from our data shown below in Figure 81).

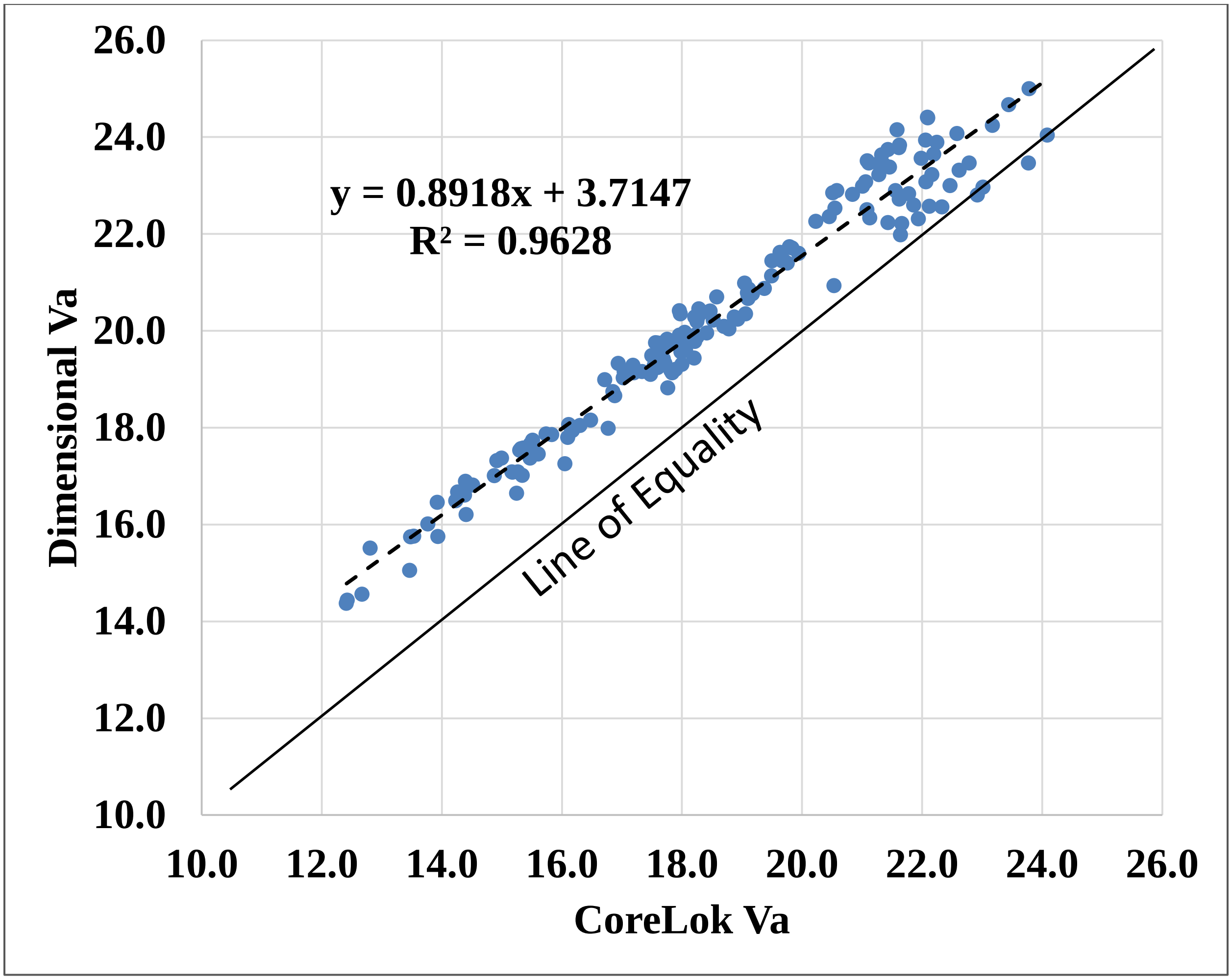


Figure 81 – Air Void Correlation for CoreLok and Dimensional Analysis Methods

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APPENDIX

Date:9-12-95

Department Of Transportation - State Of Georgia
Asphaltic Concrete Design Report

Mix Type: D-MOD   Mix I.D. No: 102-DM1

This design is approved for use contingent upon approval by the Engineer of a Job Mix Formula.
A change in materials properties or unacceptable field performance may invalidate this design.

	Materials	Size,Grade Type Code	% Used With Lime	% Used W.out Lime	Group	Source Number	Source Name And Location
		007	59	0	IIA	102C	- Blue Circle Aggregates,Inc. Buford,GA
		089	35	0	IIA	102C	- Blue Circle Aggregates,Inc. Buford,GA
	Aggregate	W10	5	0	IIA	102C	- Blue Circle Aggregates,Inc. Buford,GA
			0	0			-
			0	0			-
			0	0			-
[1]	Asph. Cement	AC20				0033	- Plant Improvement Co. Gwinnett Co.,Ga
	Hydr. Lime		1.0				- Approved Source
	Additive			0.0			

% AC	Theor. Sp.Gr.	Actual Sp.Gr.	% Air Voids	Mix Density	% Vma	% Aggr. Voids Filled	Stab. Lbs.	Flow (.01 In.)
			50 Blows Hydrated Lime					
0.0	0.000	0.000	0.0	0.0	0.0	0.0	0	0.0
0.0	0.000	0.000	0.0	0.0	0.0	0.0	0	0.0
0.0	0.000	0.000	0.0	0.0	0.0	0.0	0	0.0
0.0	0.000	0.000	0.0	0.0	0.0	0.0	0	0.0
0.0	0.000	0.000	0.0	0.0	0.0	0.0	0	0.0
			75 Blows Hydrated Lime					
0.0	0.000	0.000	0.0	0.0	0.0	0.0	0	0.0
0.0	0.000	0.000	0.0	0.0	0.0	0.0	0	0.0
0.0	0.000	0.000	0.0	0.0	0.0	0.0	0	0.0
0.0	0.000	0.000	0.0	0.0	0.0	0.0	0	0.0
			Additive Only					
0.0	0.000	0.000	0.0	0.0	0.0	0.0	0	0.0
0.0	0.000	0.000	0.0	0.0	0.0	0.0	0	0.0
0.0	0.000	0.000	0.0	0.0	0.0	0.0	0	0.0
0.0	0.000	0.000	0.0	0.0	0.0	0.0	0	0.0

Aggregate Gradations

Agg. Size	007	089	W10				Lime	Comb. Grad.
1 1/2	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0
3/4	100	0	0	0	0	0	0	100
1/2	87	0	0	0	0	0	0	92
3/8	42	100	0	0	0	0	0	66
4	2	52	100	0	0	0	0	25
8	2	7	75	0	0	0	0	8
16	0	4	48	0	0	0	0	5
30	0	3	32	0	0	0	0	4
50	0	2	22	0	0	0	0	3
100	0	2	7	0	0	0	100	2
200	0	1	3	0	0	0	100	2

Diametral Tensile Splitting

Property	Lime	Liquid
Conditioned Psi	0.0	0.0
Control Psi	0.0	0.0
Retained Stab(%)	0.0	0.0

Job Mix Formula Criteria

With H Lime	Optimum AC %	Film Thick
	6.00	33.35

Aggr. Eff. Gravity 2.630

Note [1] Asphalt Cement to be modified with a Thermoplastic Polymer to meet Section 820.02.
[2] Mineral Fiber meeting requirements of Section 819.02C shall be added at 0.4% of total mix.

Harry H. McGaughey

Harry H. McGaughey
State Bituminous Construction Engineer

2.148B

PLANT LOCATION: Augusta		MIX DESIGN LAB NO.: APAC04		JOB MIX NO.: **E0171**	
TYPE MIX: Open Graded Friction Course		DATE APPROVED: 03/12/04			
CONTRACTOR: APAC-Southeast, Inc.		DATE OF LAST REV.: 03/12/04		NO. OF REVISIONS: 0	
.TROL METHOD: QA		DATE VOID: 03/12/06			

	Source of Aggregate	Type of Aggregate	% Agg.	Gsb
1	Martin Marietta @ Augusta	#7 Stone	91	2.66
2	Martin Marietta @ Augusta	#89 Stone	8	2.68
3	Tenn Luttrell, Luttrell, TN	Lime	1	2.68
4				
5				
6				
7				

SIEVE	GRADATION 1	2	3	4	5	6	7	COMB. GRAD.	TARGET	LIMITS
1 1/2" / 37.5 mm										
1" / 25.0 mm										
3/4" / 19.0 mm	100	100	100					100	100	100.0
1/2" / 12.5 mm	94	100	100					95	95	89.0 - 100.0
3/8" / 9.5 mm	67	98	100					70	69	63.0 - 75.0
#4 / 4.75 mm	17	60	100					21	20	15.0 - 25.0
#8 / 2.36 mm	6	16	100					8	7	5.0 - 10.0
#30 / 0.60 mm	4.2	4	100					5		
00 / 0.150 mm	3.8	3	100					4.7		
#200 / 0.075 mm	1.2	1.4	100					2.20	2.0	0.00 - 4.00
OPTIMUM BINDER CONTENT,%									6.0	5.64 - 6.36

JOB MIX

		DATE
PREPARED BY	ACS	3-12-04
REVIEWED BY	CAS/CWH	3/17/04
APPROVED BY	[illegible]	3/22/04

PERCENT BINDER				
MARSHALL STABILITY, N				
FLOW IN 0.25 mm				
MAXIMUM SPECIFIC GRAVITY				
BULK SPECIFIC GRAVITY				
% AIR VOIDS IN TOTAL MIX				
% V. M. A.				
% VOIDS FILLED				

EFFEC. SPECIFIC GRAVITY: NA TSR(%): NA WET TS:(kPa) NA

GRADE OF BINDER: PG 76-22

DESIGN DUST TO ASPHALT RATIO: NA

6.0 % Asphalt recommended with permissible variation of: 0.36 This mix is satisfactory and meets SCDOT specification for use in Open Graded Friction Course

REMARKS: Verified mix 0.3% by weight used of Interfibe - Cellulose Fiber QA Spec.s

.der Film Thickness (Microns)= NA

Form TL-127 (Rev. 11/09)

VIRGINIA DEPARTMENT OF TRANSPORTATION
MATERIALS DIVISION
STATEMENT OF ASPHALT CONCRETE OR CENTRAL-MIX AGGREGATE JOB-MIX FORMULA

Submit to the District Administrator, Virginia Department of Transportation. The Materials Division must be notified by the contractor before work is begun using the submitted mix design. Once approved, this job-mix design may be used for all Department projects for the type of mix shown below.

New Mix: YES ☐ NO ☒

Contractor Design Mix No. ______ **Design Lab No.** C-2

Date 1-18-12 **Job Mix ID No.** 9002-2011-84 **Calendar Year:** 2012 **TSR Test No.** ______

Type Mix / Size Aggregate Porous Friction Course 9.5 PG 82-22 RM (Quiet Pavement)

Producer Name & Plant Location Superior Paving Corp. Leesburg **Phone** 703-729-0633

Materials	Job Mix Phase A	B*	C		Kind	Source
Aggregate	89			%	#8	Luck Stone Leesburg
Aggregate				%		
Bag House	1			%	Plant Breakdown	Superior Paving Leesburg
Mineral Filler				%		
Screening				%		
RAP	10			%	Processed	Superior Paving Leesburg
Asphalt Cement	6.40			%	PG 82-22 RM	Blacklidge Greer SC
Tack					EM-50TT	Seaboard Baltimore, MD
Additives:						
Anti Strip	.3				Pavebond Lite	Rohm/Haas Cincinnati OH
Fiber	.3				Cellulose	Hi-Tech Asp. Sol .Mechanicsville

*** All Asphaltic Materials to Produce This Mix Must be Certified by VAAP***

Job-Mix Sieves	Job Mix Phase Total % Passing: Lab JMF A	Production JMF B*	Tolerance % + or -	Acceptance Range Average of 8 Test(s) A	B	End of Year Average C	Design/Spec. Range
3/4"	100		0	100			100
1/2"	100		0	100			100
3/8"	86		2.8	83.2-88.8			85-100
#4	21		2.8	18.2-23.8			20-40
#8	9		2.8	6.2-11.8			5-10
#200	2.5		.7	1.8-3.2			2-4
Asphalt (%)	6.4		.21	6.19-6.61			Min. 6.0
VTM	18.7						Min. 16
Rice (Gmm)	2.650		50 gyrations				
Compacted Unit Wt							

Lay Down Temperatures	290-340 °F	Muffle Furnace Correction Factor:	.52
		Field Correction Factor ($G_{se} - G_{sb}$):	
Lab Compaction Temperatures	310-320 °F	Pill Weight:	4450
		SMA Mixes	*50 Gyration*
		VCA_{DRC}:	42.2
Producer Certification Technician's Signature	Daniel Poole	G_{CA}:	2.958

MATERIALS DIVISION USE ONLY – TO BE COMPLETED UPON CONTRACTOR SUBMISSION OF PART B

Remarks Apprem 2011 Need Test Strip

Nominal Max. Size Aggregate 9.5mm **Application Rates: Min.** ______ lb/yd² (kg/m²) **Max.** ______ lb/yd² (kg/m²)

Checked By:

Approved tentatively subject to the production of material meeting all other applicable requirements of the specification.
*** Note: Part B 'Production JMF' and corresponding Material percentages will be filled out by the Contractor upon receipt of the additional requirements of the Contractor within the first lot.**

Copies:				
State Materials Engineer	Approved By ✓	Part A:	[signature] For D. Shields	Date: 4-10-12
District Materials Engineer / Project Inspector	Approved By	Part B:		Date:
Sub-Contractor and/or Producer	Approved By	Part C:		Date:

CORRECTED COPY CORRECTED COPY

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

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

Contractor: Community Asphalt Corp. Address: 14005 N.W. 186th Street, Hialeah, FL 33018

Phone No. (561) 790-6467 Fax No. (561) 790-1093 E-mail: ttrueblood@ctlabs.net

Submitted By: Todd B. Trueblood Type Mix: FC-5 Intended Use of Mix: Friction Course

TYPE MATERIAL	F.D.O.T. CODE	PRODUCER	PIT NO.	DATE SAMPLED
1. S-1-A Stone	41	White Rock Quarries	87-339	08 / 25 / 1997
2. S-1-B Stone	51	White Rock Quarries	87-339	08 / 25 / 1997
3. Asphalt Screenings	22	White Rock Quarries	87-339	08 / 25 / 1997
4. ARB-12	336-AR			
5.				
6.				

PERCENTAGE BY WEIGHT TOTAL AGGREGATE PASSING SIEVES

Blend Number	50% 1	45% 2	5% 3	4	5	6	JOB MIX FORMULA	CONTROL POINTS	RESTRICTED ZONE
3/4" 19.0mm	100	100	100				100	100	
1/2" 12.5mm	80	100	100				90	85 - 100	
3/8" 9.5mm	38	94	100				66	55 - 75	
No. 4 4.75mm	6	35	100				24	15 - 25	
No. 8 2.36mm	3	10	82				10	5 - 10	
No. 16 1.18mm	3	4	57				8		
No. 30 600μm	2	3	36				7		
No. 50 300μm	2	2	20				6		
No. 100 150μm	1	2	9				5		
No. 200 75μm	1.0	1.0	2.0				3.5	2 - 4	
G_{sb}	2.407	2.412	2.527				2.415		

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

JMF reflects aggregate changes expected during production.

SP 05-3979A (FC-5)

Transferred from SP 03-2470A (FC-5)

CORRECTED COPY due to incorrect binder shown.

Director, State Materials Office: Thomas O. Malerk

Original document retained at the State Materials Office

Effective Date: 02 / 23 / 2005

Expiration Date: 02 / 23 / 2008

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

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION

STATEMENT OF SOURCE OF MATERIALS AND JOB MIX FORMULA FOR BITUMINOUS CONCRETE

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

Contractor APAC-Southeast, Inc., Southern Florida Division Address P.O. Box 2579, Sarasota, FL 34230

Phone No. (941) 483-3329 Fax No. (941) 486-0170 E-mail dlsheppard@ashland.com

Submitted By Linda Sheppard Type Mix FC-5 Intended Use of Mix Friction Course

TYPE MATERIAL	F.D.O.T. CODE	PRODUCER	PIT NO.	DATE SAMPLED
1. S-1-A Stone	41	White Rock Quarries	87-339	02 / 07 / 2003
2. S-1-B Stone	53	White Rock Quarries	87-339	02 / 07 / 2003
3. PG 76-22	916-PG			
4.				
5.				
6.				
7.				

PERCENTAGE BY WEIGHT TOTAL AGGREGATE PASSING SIEVES

SIEVE SIZE	Blend	50%	50%					JOB MIX	CONTROL	
	Number	1	2	3	4	5	6	FORMULA	POINTS	
	3/4" 19.0mm	100	100					100	100	
	1/2" 12.5mm	85	100					93	85 - 100	
	3/8" 9.5mm	46	92					69	55 - 75	
	No. 4 4.75mm	10	35					23	15 - 25	
	No. 8 2.36mm	7	10					9	5 - 10	
	No. 16 1.18mm	5	5					5		
	No. 30 600μm	4	4					4		
	No. 50 300μm	3	3					3		
	No. 100 150μm	3	3					3		
	No. 200 75μm	1.0	1.0					3.0	2 - 4	
	G_{sb}	2.407	2.416					2.411		

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

JMF reflects aggregate changes expected during production

SPM 05-3987B (FC-5)

SPM 05-3987A revised to reflect change binder content.

Director, Office of Materials

Effective Date 08 / 31 / 2006

Expiration Date 02 / 03 / 2008

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

New Jersey Department of Transportation

HMA Mix Design

12/21/2015

Region: Central

Mix ID#	C02DC0737VIR
Mix Type	HMA,OPEN GR FRICTION CRS, ASPHALT RUBBER
Producer	TRAP ROCK INDUSTRIES - KINGSTON, NJ (HMA PLANT)
Mix Temp. (F)	320
Compaction Temp. (F)	310
Effective Date	3/4/2014
Expiration Date	12/31/2049
Verification Type	Lab Verification
Designer	Michael Jopko

SIEVE SIZE Inch	mm	Job Mix Formula	Broadband min.	max.	Production Tolerances min.	max.	Tests Performed	Test Results	Test Criteria min.	max.
2	50	100.0	100	100			%Air Voids (Va)	19.64		
1 1/2	37.5	100.0	100	100			%VMA			
1	25	100.0	100	100			%VFA			
3/4	19	100.0	100	100			Dust/Asphalt Ratio			
1/2	12.5	100.0	85	100			Drain Down			
3/8	9.5	92.2	35	60	86.7	97.7	VCA - Mix			<VCA dry
No.4	4.75	33.9	10	25	28.4	39.4	VCA - dry			
No.8	2.36	12.9	5	10	8.4	17.4	Max. Sp.Grav. (Gmm)	2.688		
No.16	1.18	8.4	0	100			Bulk Sp.Grav. (Gmb)	2.160		
No.30	0.6	6.4	0	100			% Gmm @ N Max			
No.50	0.3	5.2	0	100			Sp. Grav. of Binder (GB)			
No.100	0.15	3.7	0	100			Sp. Grav. of Agg. Blend (Gsb)	2.937		
No.200	0.075	3.0	2	5	1	5	Moist.Sensitivity TSR	94		
Virgin Binder Content		6.0					lbs./Square Yard/Inch	101.13		
							% Gmm @ N Design	75.8		
							Ignition Oven Agg. Correction Factor CFI		0.15 @ 538 Degrees C	
							% Absorbed AC	0.10		

COMPONENT MATERIALS	TOTAL MIX %	COMPONENTS - PRODUCER & LOCATION
AGGREGATES, STONE SAND, UNWASHED	12.0	TRAP ROCK INDUSTRIES - KINGSTON, NJ (AGGREGATES)
AGGREGATES, COARSE, *8, BROKEN STONE	88.0	TRAP ROCK INDUSTRIES - KINGSTON, NJ (AGGREGATES)
ASPHALT, BINDER, GRADE 76-22	6.0	AXEON SPECIALTY PRODUCTS, LLC - PAULSBORO, NJ

Remarks:

9/20/2013

Quadrant: L
Section: 21
Sublot: 1

Laboratory Diary

General Description of Mix and Materials

Design Method:	Super
Compactive Effort:	75 gyrations
Binder Performance Grade:	76-22
Modifier Type:	SBS
Aggregate Type:	Lms/Sand
Design Gradation Type:	ARZ

Avg. Lab Properties of Plant Produced Mix

Sieve Size	Target	QC
25 mm (1"):	100	100
19 mm (3/4"):	100	100
12.5 mm (1/2"):	100	100
9.5 mm (3/8"):	100	100
4.75 mm (#4):	99	98
2.36 mm (#8):	76	74
1.18 mm (#16):	53	53
0.60 mm (#30):	36	34
0.30 mm (#50):	23	18
0.15 mm (#100):	15	12
0.075 mm (#200):	11.5	9.4
Binder Content (Pb):	6.1	6.2
Eff. Binder Content (Pbe):	5.6	5.7
Dust-to-Binder Ratio:	2.0	1.6
Rice Gravity (Gmm):	2.441	2.449
Avg. Bulk Gravity (Gmb):	2.343	2.359
Avg Air Voids (Va):	4.0	3.7
Agg. Bulk Gravity (Gsb):	2.647	2.662
Avg VMA:	16.9	16.8
Avg. VFA:	76	78

Construction Diary

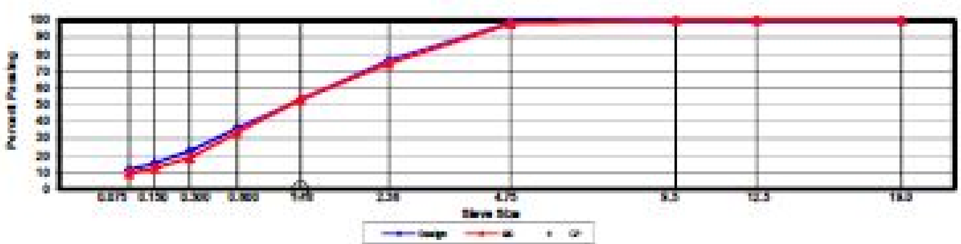
Relevant Conditions for Construction

Completion Date:	August 13, 2012
24 Hour High Temperature (F):	89
24 Hour Low Temperature (F):	61
24 Hour Rainfall (in):	0.00
Planned Sublot Lift Thickness (in):	0.8
Paving Machine:	Blaw Knox

Plant Configuration and Placement Details

Component	% Setting
Binder Content (Plant Setting)	6.3
820 Calera Limestone	69.0
Shorter Coarse Sand	30.0
Hyd Lime	1.0

As-Built Sublot Lift Thickness (in):	NA
Total Thickness of All 2012 Sublots (in):	0.8
Approx. Underlying HMA Thickness (in):	5.6
Type of Tack Coat Utilized:	NTSS-1HM
Undiluted Target Tack Rate (gal/sy):	0.06
Approx. Avg. Temperature at Plant (F):	325
Avg. Measured Mat Compaction:	95.6%



General Notes:
1) Mixes are referenced by quadrant (E=East, N=North, W=West, S=South, L=Lee Rd 159), section number, and sublot (top=1);
2) SMA and OGFC refer to stone matrix asphalt and open-graded friction course, respectively; and
3) Mixes not containing hydrated lime were run with either Gripper X antistrip or Evotherm Q1 warm mix additive at a 0.5% rate

NCAT 2012 Test Track Section E9-1a JMF

Percent Passing Sieve	Comb. Blend
25.0 mm, 1"	100.0
19.0 mm, 3/4"	100.0
12.5 mm, 1/2"	97.1
9.5 mm, 3/8"	82.7
4.75 mm, #4	25.4
2.36 mm, #8	7.0
1.18 mm, #16	5.4
0.600 mm, #30	4.4
0.300 mm, #50	3.6
0.150 mm, #100	2.8
0.075 mm, #200	2.0

Gmm:	2.463
AC%:	6.0
Air Voids:	17.1%
Gsb:	2.639
Gyrations:	50
Compaction Temp, °F:	320
Binder Grade:	PG 76-22

Stockpile	% Blend
Granite M10's	6
Granite 89's	50
Granite 78's	44