EVALUATION AND OPTIMIZATION OF FUNCTIONALITY AND DURABILITY OF POROUS GRADED ASPHALTS

A Thesis

by

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ABSTRACT

Porous Graded Asphalts (PGA) are thin layers utilized on top of conventional pavements that are characterized for containing high air void contents (AV) (usually larger than 18%) which results in high drainability; this provides safety conditions since it reduces hydroplaning, splash and spray and glare at night and improves skid resistance and the visibility of pavement markings along with some environmental and driving enhancements. An ideal PGA should achieve a balance between functionality (i.e., the capacity of maintaining the beneficial properties throughout the service life) and durability (i.e., the resistance to distress and/or failure); however, some transportation agencies have reported highly functional PGA mixtures with early failures, or highly durable mixtures that do not achieve long-term beneficial properties.

In order to overcome this problem, an assessment of the durability and functionality of four different PGA materials was performed. Durability was evaluated via Cantabro loss test, Hamburg Wheel Track test (HWTT), Semi Circular Bending test (SCB) and Indirect Tensile Strength/ Tensile Strength Ratio test (IDT/TSR). Functionality was evaluated via permeability test with TXDOT, NCAT and FDOT permeameters and noise absorption was evaluated using the Impedance Tube test. Additionally, construction and maintenance specifications were revised in order to identify key procedures to guarantee adequate functionality and durability of PGA.

It was found that the functionality of the mixture is best represented by the permeability measurements which depend crucially on the AV, thickness and type of permeameter utilized. Durability of the mixture was better represented by the Cantabro loss test, HWTT and SCB are not recommended; the Cantabro loss depends significantly on the AV, type of conditioning protocol and thickness. It was found that increasing the AV increases the permeability, which is beneficial for functionality, but at the same time the Cantabro resistance decreases, which translates to a greater potential for abrasion related regardless of

the thickness or type of material. Hence it is suggested to do an optimization and include it in PGA specifications with both minimum and maximum AV thresholds guaranteeing functional mixtures that perform adequately over the service life.

DEDICATION

This work is dedicated to my parents, Nohra and Eduardo, who have always encouraged and supported me on following my dreams. The kindness, love, support and formation they have given me were essential on every step I have taken and made me the strong and determined person I am now.

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NOMENCLATURE

AASHTO	American Association of State Highway and Transportation Officials
AR	Asphalt Rubber
ARC	Asphalt Roller Compactor
ASTM	American Society for Testing and Materials
AV	Air Void
BPN	British Pendulum Number
CA	California
CPB	Controlled By-Pass Method
CPX	Close Proximity Method
CTM	Circular Texture Meter
DFT	Dynamic Friction Tester
DGHMA	Dense Graded Hot Mix Asphalt
DOT	Department of Transportation
FDOT	Florida Department of Transportation
FL	Florida
FN	Frictional Number
GA	Georgia

GSPKWY	Garden State Parkway
HVB	High-Viscosity Binders
HWTT	Hamburg Wheel Tracking Test
IDT	Indirect Tensile Strength Test
IFI	International Friction Index
IRI	International Roughness Index
LKC	Linear Kneading Compactor
LMLC	Lab Mix Lab Compacted
LTPP	Long-Term Pavement Performance
ME	Maine
NAPA	National Asphalt Pavement Association
NCAT	National Center of Asphalt Technology
NDS	Netherlands
NJ	New Jersey
NM	New Mexico
NV	Nevada
OBC	Optimum Binder Content
OBSI	On-Board Sound Intensity

OGFC	Open Graded Friction Courses
PCC	Portland Cement Concrete
PFC	Permeable Friction Courses
PGA	Porous Graded Asphalt
PMPC	Plant Mix and Plant Compacted
RPMLC	Reheated Plant Mix and Lab Compacted
RQI	Ride Quality Index
SBS	Styrene-Butadiene-Styrene
SCB	Semi Circular Bending Test
SGC	Superpave Gyratory Compactor
SHRP	Strategic Highway Research Program
SMA	Stone Matrix Asphalt
TLPGA	Two-Layer Porous Graded Asphalt
TSR	Tensile Strength Ratio
TTI	Texas A&M Transportation Institute
TX	Texas
TxDOT	Texas Department of Transportation
UT	Utah

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1. INTRODUCTION*

Porous Graded Asphalt (PGA) surface courses are thin layers utilized on the top of conventional pavements that are used to obtain environmental, safety and frictional benefits (Burns Cooley Dennis Inc., 2009; Putman, 2012). PGAs are asphalt mixtures characterized by more open aggregate gradations, stiffer binders, higher binder contents and higher air void contents (AV) (i.e. usually between 15% and 20%) compared to conventional Dense Graded Hot Mix Asphalt (DGHMA). There are different terms used for referring to PGAs; in the United States (US), PGAs are commonly known as Open Graded Friction Courses (OGFC) or Permeable Friction Courses (PFC), while in Europe and Asia the most common term for this material is Porous Asphalt (PA). This study will use the general term PGA to refer to this type of mixture.

Below a PGA course, there is always an impermeable layer which, in most cases, is DGHMA. The large quantity of interconnected AV in PGAs permits water to penetrate the structure and to drain laterally at the interface with the subsequent layer to the edge of the pavement structure, as shown in Figure 1. This is the main difference between PGA and permeable pavements, which are also pervious in the layers underneath the surface.

* Part of the data reported in this chapter is reprinted with permission from "Mix design, performance and maintenance of Permeable Friction Courses (PFC) in the United States: State of the Art" by Hernandez-Saenz, Maria A.; Caro, Silvia; Arambula, Edith and Epps Martin, Amy, 2016. *Construction and Building Materials.* Vol. 111, Pages 358-367, Copyright [2016] by Elsevier.

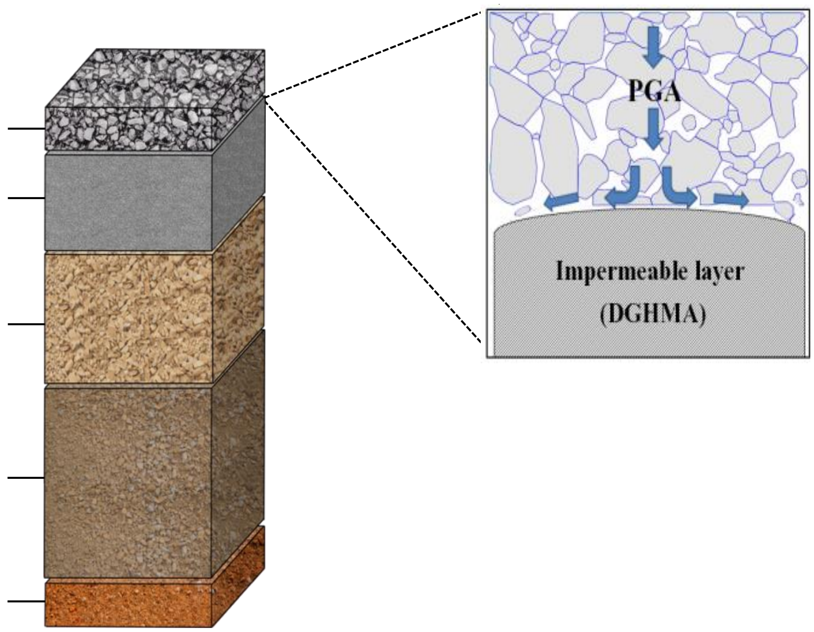


Figure 1. Typical pavement structure with a PGA surface course.

High AV in PGA result in highly permeable mixtures, which effectively reduce hydroplaning, even in conditions near saturation (Dell'Acqua, De Luca, & Lamberti, 2011), diminish splash and spray by 90-95% as compared to DGHMA mixtures and thus improve visibility by 2.7 to 3.0 times (Nicholls, 1997; Rungruangvirojn & Kanitpong, 2010). In addition, PGA decreases glare at night, improves road marking visibility (Lefebvre, 1993) and enhances the friction of the pavement surface, especially in wet weather (Adam & Shah, 1974; Brunner, 1975; Huddleston, Zhou H., & R.G., 1993). These safety advantages are the main reason for using PGA in the US.

Other important reasons for using PGAs are their environmentally related benefits. The use of PGA results in a 3 and 6 dB reduction in the level of noise generated by passing vehicles compared to DGHMA (Bennert, Fee, Sheehy, Jumikis, & Sauber, 2005; Freitas, Pereira, de Picado-Santos, & Santos, 2009; Kandhal, 2004; Nordic Road & Transport Research, 1994), and between 5.5 and 10.5 dB reduction as compared to Portland Cement Concrete (PCC) surface layers (Kandhal, 2004). Also, it has been found that the total suspended solids and lead present in the water runoff from PGA decreases around 90% in comparison to the water runoff that is generated through DGHMA (Eck & Klenzendorf, 2011; Keafott, Barrett, & Malina Jr., 2005). These environmental benefits are the main motivation for using PGA in Europe (Hernandez-Saenz, Caro, Arambula, & Epps Martin, 2016).

Despite their multiple benefits, there are also important challenges related to the use and maintenance of PGA mixtures. Corrective maintenance activities are difficult to implement since this type of mixture cannot be patched in large areas due to the negative consequences on permeability (Cooley Jr. et al., 2009). However, the principal concern is winter maintenance since PGA freezes sooner and for longer periods compared to DGHMA (Fay & Akin, 2013). Besides, PGA also requires higher application rates and more frequent application of de-icers, and it does not permit the use of salt or sand that are commonly used to improve the frictional properties of the pavement during cold weather events, as both products tend to clog the open AV structure (Fay & Akin, 2013; Yildirim, Dossey, Fults, Tahmoressi, & Trevino, 2007).

Other challenges of PGA mixtures pertain to durability (i.e., the resistance to distress and/or failure) and functionality (i.e., the capacity of maintaining the beneficial properties throughout the service life of the pavement). The principal difficulty related to durability is the appearance and evolution of raveling, which consists of the loss of aggregates at the surface of the pavement due to the repeated abrasion caused by traffic (Arambula, 2014a). The principal problem related to functionality is clogging, which occurs when sediments are deposited within the open AV structure, generating a reduction in permeability and noise reduction capability (Cooley Jr. et al., 2009; Huber, 2000a; Kandhal, 2002).

An ideal PGA should achieve a balance between functionality and durability; however, some transportation agencies have reported highly functional PGA mixtures with early failures, or highly durable mixtures that do not achieve long-term beneficial properties. These problems are the motivation for a comprehensive characterization and optimization of functionality and durability of PGA in which both characteristics are fully assessed individually and then related to each other. The ultimate goal of this project is to present solutions that meet current challenges and future needs related to the use of PGA surface courses. The objective of this study is to overcome some of the current performance challenges of using PGA mixtures in the performance field via functionality and durability analysis. The idea is to ensure adequate pavement performance and longer service life and thus encourage utilization of this material nationwide.

In terms of durability the tests performed and described in this document include Cantabro Test (with seven different types of conditionings), Hamburg Wheel Test (HWTT), Semi Circular Bending Test (SCB) and Indirect Tensile Strength (IDT) (with two different types of conditionings). In terms of functionality, the properties assessed were permeability through falling head permeameter tests and noise absorption through the impedance tube tests. Materials from four different places of the US were utilized to optimize two main parameters: thickness of PGA layer and AV. Specific objectives of this study include evaluating the impact of AV and thickness in both functionality and durability, identifying key tests for assessing these properties and correlating laboratory and field data.

In this context, the document presented next presents the research work done to achieve this objective. Specifically, the initial part of this document presents a comprehensive literature review of all the relevant information related to the use PGA, followed by the detailed description of the experimental design. Next, the results are presented first through a functionality analysis and then through a durability analysis. After this, a chapter describing important construction and maintenance considerations regarding the use PGA is presented. Finally, the last section summarizes the main findings, conclusions and recommendations obtained from this study.

2. LITERATURE REVIEW

2.1 History and Use of PGA

2.1.1 History

PGA appeared initially as a product of experimentation with plant mix seal coats during the 1940s (Cooley Jr. et al., 2009). Plant mix seal coats were developed as an alternative for providing friction resistance to the surface of asphalt pavements, replacing the use of seal coats or chip seals due to their frequent problems, which included bleeding (i.e., film of asphalt appearing on the surface), raveling, loose stone, and short service life (Kandhal, 2002). California decided to produce plant mix seal coats with higher asphalt binder contents and a smaller nominal maximum aggregate size than typically used in seal coats and instead of applying the binder and spreading the aggregate in the field as for typical seal coats, the new product was mixed in a conventional DGHMA plant and placed as a thin layer at the surface of the pavement. Besides improving the skid resistance of the pavement, the plant mix seal coats also reduced noise and improved performance and ride quality (Kandhal, 2002). Thus, California was the pioneer in the use of PGA mixtures, followed by some other western states.

However, the use of PGA layers did not become widespread until the 1970s due to a program initiated by the Federal Highway Administration (FHWA) to improve the skid resistance of the national road network infrastructure, in which plant mix seal coats were recommended (Kandhal, 2002). During the same decade, the material received the name of OGFC, and in the 1980s the first mix design procedure for these mixtures was published by the FHWA (FHWA, 1980). Between the 1970s and 1980s, several agencies reported problems related to draindown, sudden failures and raveling and delamination, especially due to construction issues (Cooley Jr. et al., 2009). These problems motivated an important number of states to discontinue its use. Nevertheless, other states, such as Texas, Oregon and Georgia started improving the associated design, construction and maintenance practices to obtain a better quality material (Nielsen, 2006). These proposed changes included the modification of the binder with polymers and the use of additives to reduce draindown (Putman, 2012).

The use of PGA in Europe started in the 1950s when the Property Service Agency imported the technology from the US to the United Kingdom to use it in military airfields with the primary objective of reducing hydroplaning (Khalid and Walsh, 1995). During the next decade, the material was transferred to highway pavements and based on the results of some additional research, the asphalt binder content was increased and some other additives were included. Due to the reported advantages related to these materials, PGA was introduced in other European countries like France, in 1976, and the Netherlands, in the early 1980s (Nielsen, 2006). To date several European countries, including Italy, France, Belgium, Austria, Denmark, Sweden, Switzerland, the Netherlands, the United Kingdom and Germany, report the use of PGA (Cooley Jr. et al., 2009; Kandhal, 2002). Other countries, like China, Japan, Canada, Argentina and Australia, also report the application of this type of surface layer (Dennis, 2009; Shackel, 2010; Takahashi, 2013; Yu et al., 2014).

2.1.2 Use in the US

The use of PGA in the US has been variable through the years. A survey reported by the National Center of Asphalt Technology (NCAT) in 2015, that included responses of 40 states plus Puerto Rico, show that 21 of the States were using the material (Jenks, 2015). The results of this survey are presented in Figure 2.

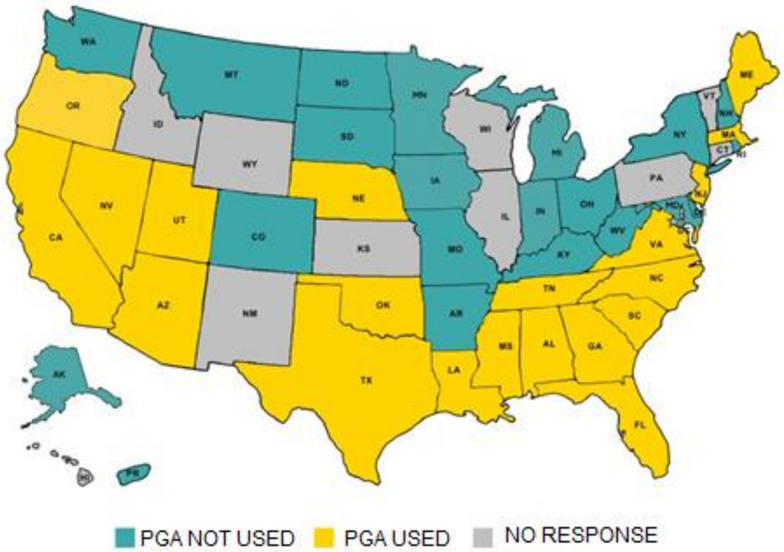


Figure 2. Current use of PGA according to NCAT (modified after Jenks, 2015)

If the state responded that they were using the material, the corresponding specifications and standards were requested; otherwise they were questioned about the main reasons for not using it. A total of 44 states responded and 25 states were not using PGA while the other 19 states were using the material. This information is summarized in Figure 3. The map obtained by NCAT, and the one prepared in this study provide evidence that there is a clear division in the use of PGA between the northern and southern states of the US.

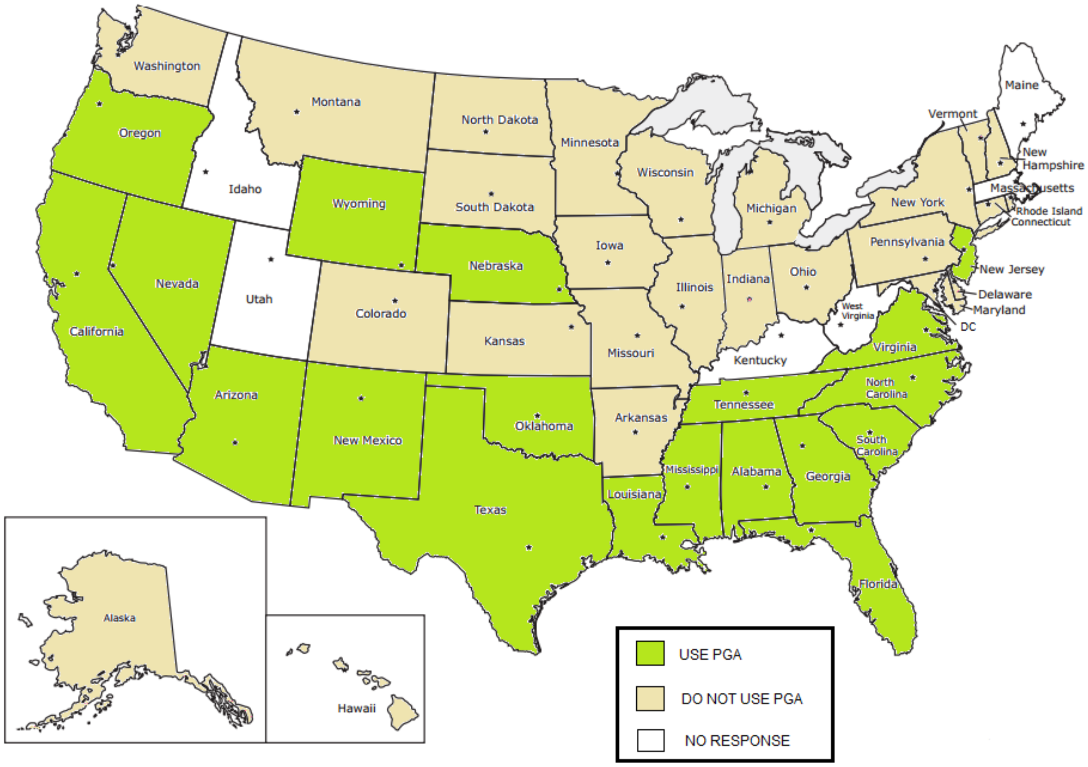


Figure 3. Map of states in US using PFC.

Based on the responses obtained from direct communications and the information found through a survey conducted by Huber in 2000 (Huber 2000), another map was developed with the objective of classifying the states that currently reported that they are not using PGAs in three different categories: a) the ones that have never used PGA, b) the ones that discontinued the use of PGA, and c) the ones that are currently testing or making specifications for this type of material. This map is presented in Figure 4, and it illustrates that the majority of these states have used PGA but discontinued it at some point in time for a variety of different reasons. Only Alaska, Indiana and New Hampshire have never used this type of surface layer. Maryland reported that they were in the process of generating the standard specifications for the design, construction and maintenance related to this material, while Hawaii and Kansas are currently testing the performance of the mixture to decide whether or not its use is considered appropriate in their states.

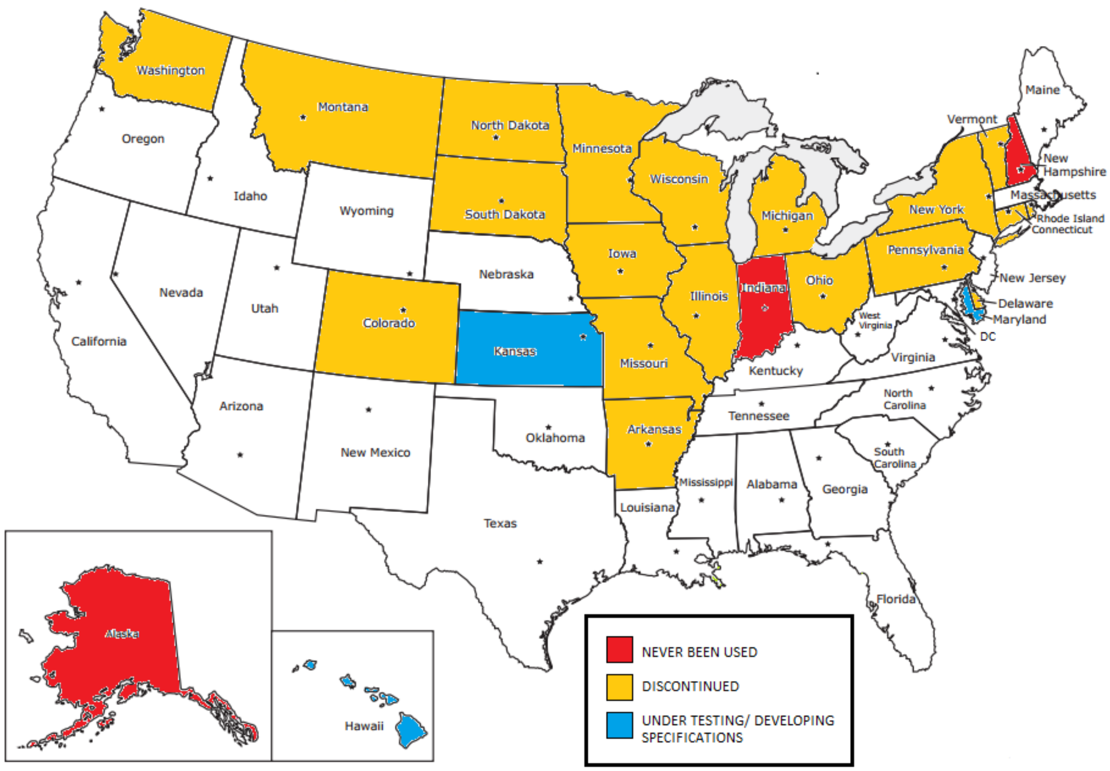


Figure 4. Map that illustrates status of states that are not using PGA currently.

Some of the states that discontinued the used of PGA stated that their reason was the cold winters with freeze-thaw conditions present in most of the northern states, which resulted in demanding maintenance issues and significant problems with durability. Other reasons reported included difficulty in controlling clogging, premature failures (e.g., delamination and raveling), and general unsatisfactory performance. The responses of some of the agencies are summarized in Table 1.

Table 1. Principal reasons for not using PGA in some states in the US.

State	Reasons for discontinuing the use of PGA
Connecticut	• *"Ice clinging to the pavement during winter storms.* • *Clogging of the surface texture with sand and debris.* • *Rapid delamination.* • *Sensitivity to construction operations that occurred during adverse weather conditions (such as colder temperatures) that led to poor pavement durability/raveling.*"
Delaware	• *"Maintenance issues".*
Hawaii	• *"It did not perform well".*
Iowa	• *"Due to the maintenance involved in keeping the porous pavement from clogging (by accumulation of sand/grit and deicers used in Iowa's winter)* • *Potential pavement damage resulting from subsequent freeze-thaw conditions".*
Missouri	• *"Because of our freezing wet weather climate".*
Montana	• *"The structures have not performed well".*
Ohio	• *"It takes an extra-large amount of salt for de-icing, which becomes prohibitive in a climate such as ours".*
Pennsylvania	• *"These surface treatments are not conducive to the weather that we receive in our state".*
South Dakota	• *"Very cold winters and an asphalt mixture that would retain moisture would have a very short lifespan due to the effects of multiple freeze/thaw cycles in the pavement layers".*
Vermont	• *"Our experience with some premature failures led us to discontinue its use".*

Various other studies have been conducted to determine the use of PGA in the US. Smith (1992) reported the results of a survey conducted in 1988 among 47 states. In total, 55% of these states reported using PGA at that time. Later, in 2000, Huber conducted a survey that included the response of 42 states, where 55% of them reported that they were not using PGA at the time (Huber, 2000). These data illustrate that in the late 1980s some states discontinued the use of PGAs. Since then, similar surveys have continued to highlight this trend. For example, in 2008, Cooley Jr. et al. obtained responses from 32 states, and 56% of them reported that they were not using PGA (Cooley Jr. et al. 2008). Figure 5 summarizes the data collected from these studies, including the information obtained in this project.

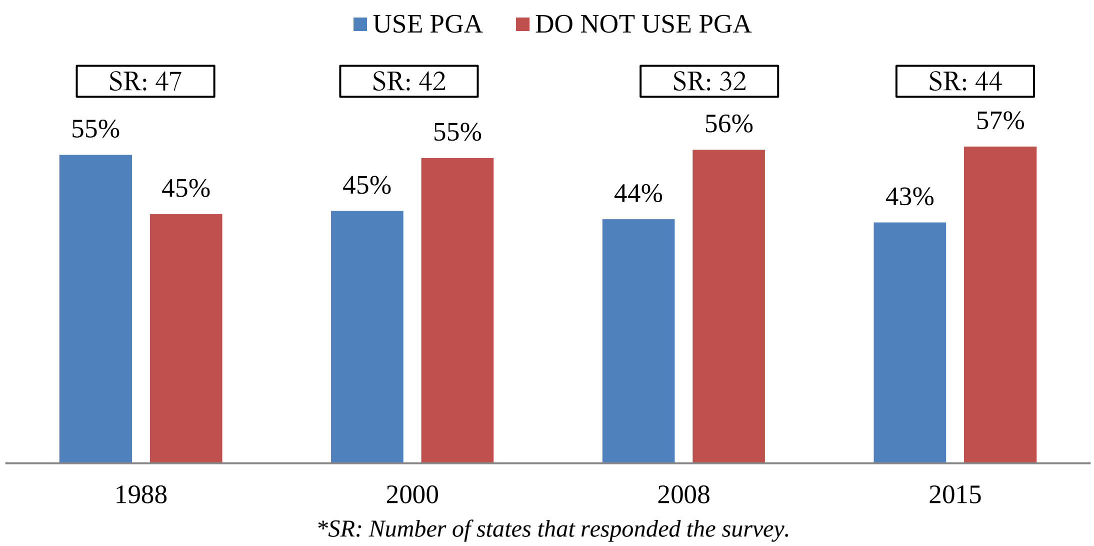


Figure 5. Evolution of the use of PGA Surface Courses in the US

PGA is getting spread worldwide due to its beneficial characteristics but it is at the same time starting to lose some other users in specific locations due its failures. To better understand the dynamics on the use of PGA it is important to understand in depth the advantages and disadvantages of the material. The following section addresses this.

2.2 Advantages

PGA presents several advantages that can be classified in three main groups: 1) safety improvements, 2) driving experience enhancements and 3) environmental benefits. According to the information available in the literature, it is possible to conclude that in the US PGA is mostly used for safety benefits, whiles in Europe the environmental advantages, especially those related to noise reduction, are the main reason for its use. This section describes the principal advantages of PGA by comparing their performance against that of the most common mixture used for surface layers: DGHMA.

2.2.1 Safety Improvements

In terms of safety, PGAs are effective in controlling both hydroplaning and spray and splash phenomena, as well as in enhancing road marking visibility and friction resistance. This results in overall reduction of wet weather accidents. Each of these enhancements is explained in detail in this section.

2.2.1.1 Reduction of Hydroplaning

Hydroplaning or aquaplaning occurs when there is enough water to form a film or layer between the tire of the vehicle and the road surface (Lefebvre, 1993). This layer does not allow a proper contact of the tire with the surface of the pavement, causing extremely low friction between the surface and the tires (Kurtus, 2008). When this occurs, hydroplaning disables the driver to stop, turn and/or to have a complete control of the vehicle (Cooley Jr. et al., 2009).

Hydroplaning occurs principally when there is heavy rain or storms (Kandhal, 2002). However, due to the high air voids content of PGA, under these situations water permeates through the mixture, avoiding the formation of a film of water on the surface of the pavement. Even when the PGA starts approaching to a saturation state due to strong or long rainfalls, the pressure under the tires gets dissipated through the porous structure because of the macrotexture that characterizes these materials (Dell'Acqua et al., 2011).

2.2.1.1.1 Reduction of Splash and Spray

Splash and spray occurs during rain events when water stays on the surface of the pavement forming pools and the vehicles pass over them. Splash is associated with large water particles, while spray, also known as mist, is associated with very fine water particles (Khalid and Perez, 1996). Both conditions significantly reduce visibility, raising the risk of an accident. In fact, several authors describe that the visibility reduction is more severe than the one caused by the presence of fog, because the airborne particles produced in splash and spray are larger and have more density compared with common fog particles (Huber, 2000; Khalid and Perez, 1996). As an example, Figure 6 illustrates the phenomena of splash and spray on a conventional DGHMA surface (right) and PGA surface (left) on an urban interstate highway in Texas.

Porous Graded Asphalt

Regular dense-graded surface

Figure 6. Splash and Spray in an urban interstate in San Antonio, Texas (modified after Arambula et al., 2013).

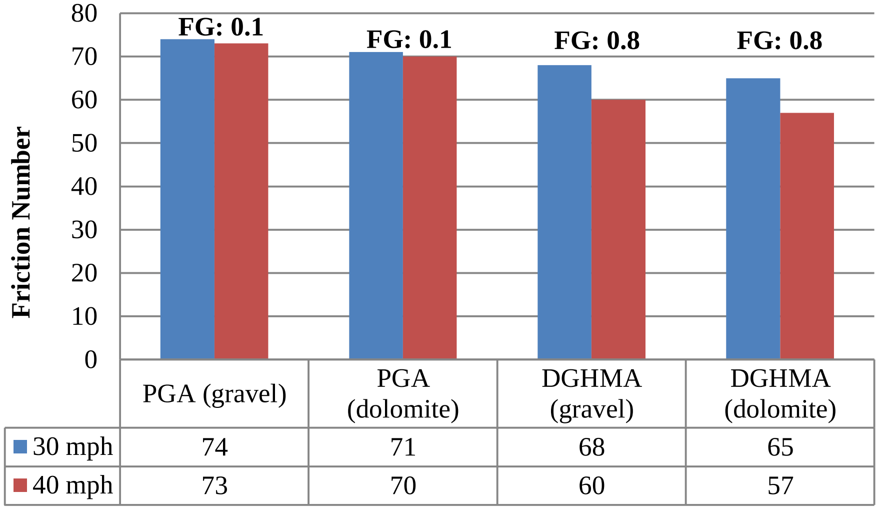
The drainage properties of PGA are responsible for controlling and reducing splash and spray because water cannot stay on the surface of the road during a rain event. Nicholls (1997) reported that the use of PGA in Europe was related to a reduction of 90%- 95% in splash and spray as compared to DGHMA. Similarly, in 2010 a study in Thailand measured the impact of splash and spray on visibility reduction using two different methods for both PGA and regular DGHMA courses (Rungruangvirojn and Kanitpong, 2010). The results obtained by applying the first method showed that at 80 km/h and high water level, DGHMA reduced visibility by 55% while PGA reduced it by only 28%. The results of second method showed that when a heavy truck passes, vehicles on the adjacent lane lose visibility 2.7-3.0 times more on a DGHMA than on a PGA surface.

2.2.1.2 Improvement of Glare at Night and Marking Visibility

Another advantage related to the use of PGA is the reduction of glare of the headlights, and the enhancement of pavement marking perception. According to Cooley Jr. et al. (2009), the reflection of light on smooth surfaces, like DGHMA, tends to resemble that of a mirror, especially on wet surfaces. On the contrary, the macrotexture of PGA diffuses the reflection of light both in dark and luminous conditions, improving the overall visibility of road markings, especially under wet conditions (Lefebvre, 1993).

2.2.1.3 Improvement of Frictional Resistance

Several authors have indicated that PGA improves the friction properties of the pavement, principally in wet conditions. Kandhal (2002) conducted a comprehensive summary of different studies conducted in North America and Europe whose objective was to quantify the friction surface properties of these surfaces by means of the frictional number (FN) and the friction gradient (i.e., the rate of change of the FN per mile divided by the change of speed). One of the studies included in this summary was a report by the Pennsylvania DOT in which the FN and the friction gradient were measured on two types of PGA and two types of DGHMA that were fabricated using two different types of aggregate (Brunner, 1975). The results, which are summarized in Figure 7, showed that the PGA layers offered more friction than those constructed with DGHMA, and that the difference in the friction properties between both surfaces was more evident at higher speeds. Additionally, the results suggested that the friction gradients were lower in the PGA and that the mean friction on this type of surface did not change significantly with an increase in speed, allowing vehicles to drive faster but safely. Similar results were also reported by other DOTs like Oregon and Louisiana (Adam and Shah, 1974; Huddleston et al., 1993).



**FG : friction gradient*

Figure 7. Friction Number and Friction Gradient of Pennsylvania DOT Report (modified after Brunner, 1975).

2.2.1.4 Overall Reduction of Accidents

The combined advantageous characteristics obtained with the use PGA have resulted in a reduction of wet weather related accidents worldwide. Takahashi (2013), for example, studied the impact of porous asphalts in 213 rain accident-prone sites all over Japan. In this context, it is noteworthy to mention that Japan started using PGA in 1998 partially due to the amount of reported vehicle accidents. The results from this study showed that after one year of installation of the PGA course, the amount of accidents decreased by 85% compared to the previous year, where the surface of the road was a regular DGHMA. In 2013 a similar experiment was performed on different roads in Louisiana (King et al., 2013). In one of the sections analyzed, the data indicated that the use of PGA was related to a 100% reduction in fatalities and 76% reduction in wet weather related accidents. Also, Smit and Prozzi (2013) evaluated several road sections in Texas in the period 2003-2011 using data from the Crash Records Information System, in which they verified that PGA indeed reduces the number of accidents, injuries and fatalities, especially under wet weather conditions.

2.2.2 Driving Experience Enhancements

The driver experience enhancements refer to the advantages perceived by the driver while utilizing roads with PGA surfaces. These benefits include higher average speeds and traffic capacity, improvement of the smoothness condition of the pavement and reduction of fuel consumption.

2.2.2.1 Higher Average Speed and Traffic Capacity

As mentioned previously, PGA reduces splash and spray and hydroplaning in wet weather conditions, and it improves visibility both day and night. These conditions provide drivers with the confidence of having complete control of their vehicles. This situation does not only reduce fatigue and stress conditions in the drivers but it also affords roads with PGA higher mean speed values in rain events compared to those with regular DGHMA surfaces. Therefore, this increase in confidence is associated with higher net highway speed values and, consequently, to an increase in traffic capacity in wet weather scenarios (Cooley Jr. et al., 2009).

2.2.2.2 Improvement of the Road Smoothness Condition

Several authors have mentioned that PGA can increase smoothness properties (Bennert et al., 2005; Bolzan et al., 2001; Khalid and Perez, 1996); however, there are limited reported data in the literature that supports this assertion. Bennert et al. (2005) conducted a study in New Jersey whose objective was to analyze and compare various properties of different thin-lift HMA with different ages and materials. These properties included a ride quality assessment that was evaluated through two parameters: 1) the Ride Quality Index (RQI) and 2) the

International Roughness Index (IRI). The RQI was measured by means of an automatic road analyzer that uses accelerometers to register the vertical acceleration of the rear axle due to the longitudinal road profile. This information is used to obtain roughness data that is then related to the users' opinion through a panel study. The RQI value is classified in 5 categories: 1) very good (4.0-5.0), 2) good (3.0-4.0), 3) fair (2.0-3.0), 4) poor (1.0-2.0) and 5) very poor (0.0-1.0). The IRI was quantified using the information captured by two lasers mounted in the Automatic Road Analyzer (ARAN) that measured the distance between the vehicle and the surface of the road. The results show that the modified PGA and the PGA with rubber reported 'good' and 'very good' values of RQI, and they produced IRI values lower than 122, which classified them as roads with small level of roughness (Bennert et al., 2005). In addition, several other authors have also reported that there is practically no rutting occurring in pavements with PGA (Isenring et al., 1990; Rogge, 2002; Takahashi and Partl, 2001), which also improves smoothness and driving comfort (Cooley Jr. et al., 2009).

2.2.2.3 Reduction of Fuel Consumption

Experiments conducted in the WesTrack experiment in Reno, Nevada, demonstrated that driving on smoother surfaces can reduce fuel consumption to 4.5% compared to a rough pavement (Mitchell, 2000). Khalid and Perez (1996) mention that for PGA mixtures, fuel consumption could be reduced by about 2%. Moreover, due to the high macrotexture of PGAs, tire distresses are reduced, resulting in a decrease of the tire wear rate (Khalid and Walsh, 1995; Lefebvre, 1993).

2.2.3 Environmental Benefits

As mentioned previously, there are important environmental benefits associated with the use of PGA. In the search for a more sustainable road infrastructure, most agencies have found in these benefits a strong justification to promote the use of these materials. This section explains how PGA is related to both noise reduction and a better quality of the runoff generated in pavements with these materials.

2.2.3.1 Noise Reduction

Undoubtedly, noise reduction has been recognized worldwide as one of the principal benefits provided by PGA courses. The Netherlands, for example, has defined noise reduction as the main reason for using PGAs in their roadways (J. T. van der Zwan, 2011).

Noise generated by the interaction between the traffic and the pavement is considered a source of environmental pollution that affects people's health, comfort and life quality (Kandhal, 2004). The high-frequency noise caused by traffic on pavements results from a phenomena described as "air pumping", which occurs when air is forced away in front of the contact area between the tire and the road and, due to the movement of the tires, it is sucked in behind (Kandhal, 2002). PGA mixtures attack noise generation by pumping the air down into the pavement, and by absorbing it instead of "reflecting" it, as occurs in regular DGHMA (Kandhal, 2002).

In 2004, Kandhal published a report where he synthetized studies dealing with noise reduction experiences reported by several agencies and countries in Europe and North America. Figure 8 and Figure 9 illustrate these results, which are expressed as noise reduction ranges of PGA compared with regular DGHMA and Portland Cement Concrete (PCC) (Kandhal, 2004). The minimum reduction value reported corresponds to 3dB, which in perspective is the equivalent of reducing the traffic volume by half (Nordic Road and Transport Research, 1994).

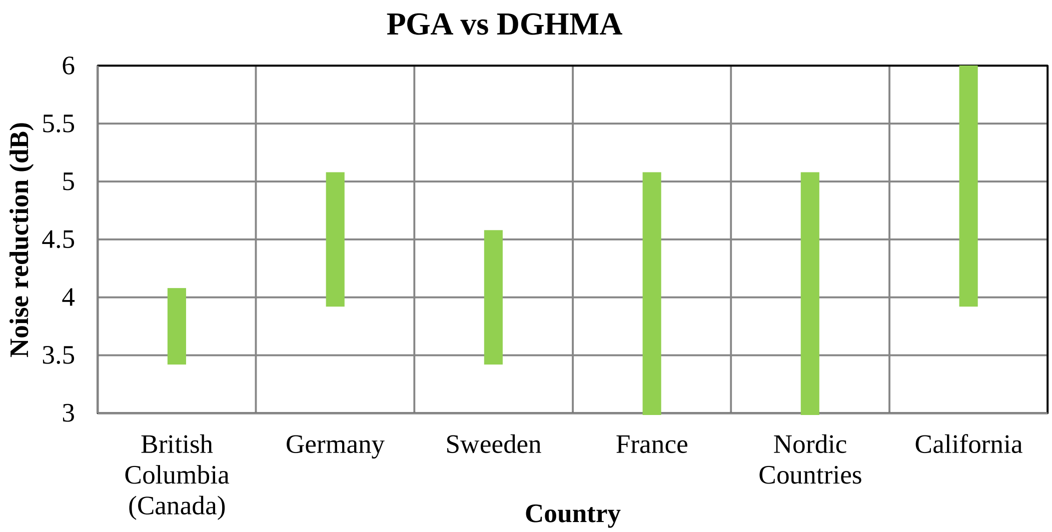


Figure 8. Noise reduction of PGA compared to DGHMA (modified after Kandhal, 2004)

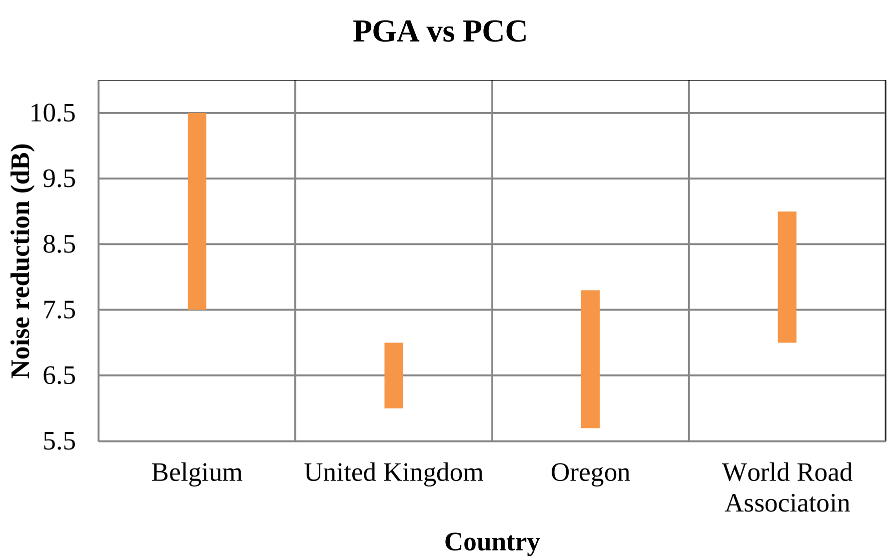


Figure 9. Noise Reduction of PGA compared to PCC (modified after Kandhal, 2004)

These results presented did not specify the method of measuring the noise reduction, since the data is based on agencies' general experiences. However, results presented subsequently do specify the method of measurement. There are three main methods used currently to measure noise reduction: 1) Close Proximity Method (CPX), 2) Controlled By-Pass Method (CPB) and 3) On-Board Sound Intensity (OBSI). The CPX consists of placing microphones inside acoustical chambers (in order to eliminate the traffic noise) near the tire-pavement interface to directly measure the tire-pavement noise (Bennert et al., 2005). The CPB measures the peak noise level of each individual vehicle from the undisturbed traffic with the microphone fixed at 7.5 m from the center of the lane (Freitas, Paulo, Bento Coehlo, & Pereira, 2008). Finally OBSI is similar to the CPX method since it also places microphones on the tire, however the configuration of the microphones is more complex and is always measured with a standardized tire at 60 mph (Rasmussen, Sohaney, & Wiegand, 2011). OBSI and CPX results may be similar, but the CPB method is different and its results cannot be compared with the others.

Freitas et al. (2009) quantified the noise generated on DGHMA and PGA surface layers, both on dry and wet surfaces using the CPB method. One of the main conclusions of this study was that the existence of water on the surface of the pavement having DGHMA considerably increases traffic noise. The results indicated that in wet conditions, the pavement with a DGHMA surface was 3.6 dB noisier at 110 km/h, and 2.9 dB noisier at 80 km/h, as compared to a pavement with PGA (Freitas et al., 2009).

The use of specific materials in the production of PGA has also been observed to contribute to noise reduction. In a report presented by the Texas DOT (2003) a reduction from 85 dB to 71 dB in noise in a reinforced concrete pavement was reported before and after using PGA with asphalt rubber (AR). This translates to a reduction of 14 dB compared to an average of 7dB, as illustrated in Figure 9. In another study, Bennert et al. (2005) compared noise levels of pavements with different surface materials, including PGA and AR-PGA, at three different speeds (55 mph, 60 mph and 66 mph). Noise was measured using the CPX. Overall, the results proved that all PGA structures (in the table presented as AR-OGFC and NJ MOGFC) resulted in lower values of noise compared to PCC, Stone Matrix Asphalt (SMA) and DGHMA. Also, structures with PGA provided intermediate values of noise gradient, or dB per mph.

To improve noise reduction, a modification to regular PGA that consists of using two layers of this material has been proposed by Van Bochove (1996). This structure, called the 'two-layer porous graded asphalt'(TLPGA), has a bottom layer of coarse aggregates and a top layer of fine aggregates, both of porous asphalt mix, as presented in Figure 10. According to van der Zwan (2011), these structures—that have been used at the surface in 4% of the highways in the Netherlands—have an average noise reduction of 6 dB, which doubles the reference reduction of 3dB of conventional PGA. Despite the environmental benefits, the use of TLPGA has been associated with significant performance-related issues. These surfaces tend to present a shorter service life, with higher life-cycle costs (J. T. van der Zwan, 2011).

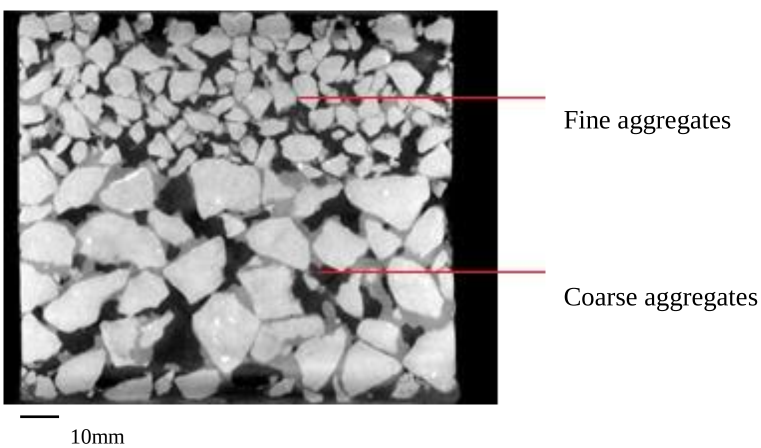


Figure 10. Cross section of two-layer porous asphalt. Modified from BAM Federal Institute for Materials Research and Testing (Recknagel and Altkrüger, n.d.).

PGA constitute a feasible option to achieve noise reduction requirements. Although there are other ways to reduce noise in road infrastructure projects, such as the classical use of noise barriers, a Danish study proved that the use of PGA is from 2.5 to 4.5 times more efficient at reducing noise than these noise barriers, based on a unit cost basis analysis (Newcomb and Scofield, 2004). Additionally, these materials help providing better views and landscapes without affecting living quality in urban areas.

2.2.3.2 Runoff Quality

Another environmentally-related advantage associated with the use of PGA is better quality of runoff generated from this type of surface. Research has shown that water quality from PGA is as good or better than that generated from a vegetated filter (Keafott et al., 2005). In 2011, Eck and Klenzendorf presented a paper with the results of a multi-year research study that had the objective of evaluating water quality and other hydraulic aspects of PGA (Eck and Klenzendorf, 2011). They studied water quality in three sites in Texas and obtained significant changes in pollutant contents of PGA runoff in comparison to pavements with regular DGHMA surfaces. The results showed that the total suspended solids decreased by 91%, on average, as well as the total lead. Furthermore, other constituents such as total copper and total phosphorus diminished, on average, by 57% and 63%, respectively.

Recently, some other studies are focusing on developing ways of using PGA as an in situ treatment technique to remove copper and zinc from highways runoff. For instance, Gang et al. (2016) utilized PGA with five different types of adsorbents additives added in the air voids of the mix, and quantified the adsorption capacity for copper and zinc removal. It was found that with most of these products, PGA had high adsorption capacity and could reduce metal concentration in a very effective way, positioning this technique as a potential field application for these environmental benefits (Gang, Khattak, Ahmed, & Rizvi, 2016).

2.3 Disadvantages

Despite the multiple benefits, there are also important disadvantages related to the use and maintenance of PGA. Table 1 listed the main reasons reported by several states in the US to discontinue the use of PGA. Among the most common motives are the poor performance of the material—that leads to a shorter service life due premature failures—, a reduction of the functionality of the layer due to clogging; and some difficulties related to maintenance activities, especially during wintertime. These reasons, which constitute the principal disadvantages that have been associated with the use of PGA, are described in detail in this section. It should be noted that there are several research groups currently working on the development of methodologies and strategies to overcome some of these difficulties. Therefore, the technologies related with the use and maintenance of PGA are expected to provide better tools in the short and long terms, which could result in an overall increase in the use of these materials worldwide.

2.3.1 Performance Difficulties

2.3.1.1 Reduction in Functionality

Functionality refers to the capacity of maintaining the beneficial properties of a component or system through its service life. The two principal concerns reported in the literature regarding the functionality of PGA are the decrease in permeability and the reduction of the noise control capacity of the material during the life of the pavement. These two difficulties are a direct consequence of clogging, which is one of the major and most studied problems of PGA. Clogging occurs when sediments are deposited on the pavement surface, leaving the structure impermeable, similar to a regular DGHMA (Martin et al., 2013). Clogging not only reduces the benefits related to the permeable capacity of the structure (e.g. hydroplaning, splash and spray, etc.), but it also increases the potential of moisture damage of the underlying pavement layers (Root, 2009). Consequently, several authors have recommended to avoid the use of PGA near quarries, farms or any place with potential for debris (Van Heystraeten and Moraux, 1990).

2.3.1.2 Reduction in Durability

Durability in this context refers to the resistance of the material or layer to support distress and/or failure. The principal durability problem reported with the use of PGA is raveling (Cooley Jr. et al., 2009; Huber, 2000; Kandhal, 2002). As mentioned previously, raveling is a degradation phenomenon defined as the loss of aggregates in the surface due to the repeated abrasion caused by vehicles, and is aggravated due to the presence of moisture (Arambula, 2014a). The principal difficulty with raveling is that it progresses fast and leads to the initiation of other distresses, reducing the service life and serviceability level of the pavement (Alvarez et al., 2006; Huber, 2000). Some other distresses such as delamination and rutting due to studded tires (i.e., tires with chains used during winter) have also been reported for PGA surfaces.

2.3.2 Increased Costs

Another disadvantage related to PGA is the high costs associated with the construction and maintenance of this surface layer. Research has shown that OGFC is 30%-40% more expensive than traditional DGHMA (X. Chen, Zhu, Dong, & Huang, 2016). In terms of construction, the cost per area-depth of PGA is lower than one presented in DGHMA by almost 50%, which reflects the greater thicknesses normally present in DGHMA (Smith, 1992), however the cost per ton is meaningfully higher than for DGHMA layers due, among other causes, to the higher binder content—which is usually modified—, and the special aggregate gradation requirements (Root, 2009). According to Root (2009), Arizona has reported a 38% increase in cost per ton of normal PGA in comparison to DGHMA, while Wyoming and Georgia have reported an increase up to 23% and in the range of 30-35 %, respectively. Also, modified PGA generated an increase of 57% in Georgia and 81% in Arizona in the cost of the material in comparison to traditional DGHMA.

Maintenance of PGA usually requires additional procedures and materials in comparison to regular DGHMA, especially during wintertime, which represents additional costs during the service life of the pavement. In spite of this, the gradual increase in the use of PGA worldwide seems to prove that the benefits related to its use compensate these extra costs.

2.3.3 Maintenance Requirements and Costs

In comparison to regular DHMA surfaces, one disadvantage of PGA is the impossibility of conducting corrective maintenance on these layers since it cannot be patched in big areas due to problems with the continuity of the permeability of the layer (Cooley Jr. et al., 2009). However, the principal concern, and one of the main reasons explaining why northern states discontinued the use of PGA is the challenge of preserving the integrity of the layer during winter; i.e., the absence of strategies or the inefficiency of existing methodologies.

One of the principal problems of winter maintenance is that there is not a consensus about which is the 'best' set of procedures to conduct these activities, as observed in the variety of specifications reported by different countries and states in the US. Within this context, in 2013, Fay and Akin published a comprehensive research report for winter control on PGA, and they were able to identify the principal disadvantages of PGA during winter conditions (Fay and Akin, 2013). These disadvantages are summarized in Table 2.

Table 2. Disadvantages of PGA during winter conditions (modified after Fay and Akin, 2013).

Item	Description
Freezing	Freezes sooner and for longer periods.
Deicers	Requires higher application rates, and more frequent applications for a longer duration.
Sanding	Its use is not recommended to use for improving friction because of repercussions in clogging.
Icing	High possibility of icing in adjacent zones with DGHMA.
Snow and Ice	Stick sooner and remain longer.
Anti-icing (salting)	Is not as beneficial because salt penetrates the structure.
Snowplows	Can cause raveling and gouging.

Thus winter maintenance of PGA requires greater quantities of de-icing products, which represents higher maintenance costs. Additionally, conventional winter maintenance for regular DGHMA surfaces, such as sand and salt, can cause clogging problems. Moreover, snow represents a main problem since it remains longer and the mechanisms to remove it, such as snowplow, can cause additional distresses (Yildirim et al. , 2006).

Once the general characteristics, advantages and disadvantages of PGA are understood, it is necessary to understand the performance of the material. The following section present a comprehensive explanation of the PGA performance assessed both in terms of functionality and durability.

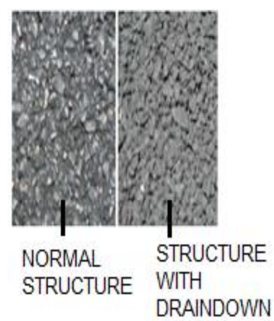
2.4 PGA Performance

This section describes aspects related to the performance of PGA, including both functionality and durability. Functionality is the ability of the layer to maintain its beneficial properties through time, while durability refers to the resistance of the mixture to distresses and failures.

2.4.1 PGA Functionality

Functionality refers to the ability of the PGA mixture to maintain its beneficial properties through time, especially in terms of permeability and noise reduction. Clogging has been determined to be the major problem in terms of functionality. Table 3 describes the factors affecting the functional performance of this material, including its potential causes and consequences.

Table 3. Principal Functionality Issues in PGA (Burns Cooley Dennis Inc., 2009; Hernandez-Saenz, Caro, Arámbula-Mercado, & Epps Martin, 2016).

Problem	Description	Causes	Effects	Picture
Clogging	Deposit of sediments on the pavement surface or migration of the binder to the pavement surface occupying the air voids	Dust, debris and other particles deposited by vehicles, wind or storm water Fat spots because of fibers agglomerations Fine aggregate gradation Excessive binder content Suction of soft binder created by traffic during hot weather	Loss of drainability Increase of noise levels Reduction of safety properties	
Aggregate crushing	Fracture of coarse aggregates in the mixture	Excessive compaction Poor aggregate quality	Loss of drainability Rutting Appearance of early failures	
Draindown	Migration of the binder by gravity and segregation of the aggregates from the binder film	Excessive binder content High mix production temperatures Lack of stabilizing additives	Loss of drainability Raveling	

These problems have significant repercussions in the performance of PGA, especially in terms of the loss of drainability/permeability which is the characteristic that provides most of the benefits (e.g. reduction of splash, spray and hydroplaning). The following sections describe in detail permeability, noise reduction and friction properties through the life cycle of PGA, with special emphasis on the effects of clogging in modifying these properties.

2.4.1.1 Permeability

Permeability refers to the ability of a porous material to allow fluids to pass through it. In the case of PGA, the most important fluid is water (Klinkenberg, 1941). Several studies have focused on studying the causes of clogging and its impact on permeability. Suresha et al. (2010) studied clogging and de-clogging in mixtures with different gradations, binder contents and clogging materials, using the falling head permeability concept. They determined that an important factor in assuring appropriate permeability levels through the service life of the material is the initial permeability for which the mixtures are designed. The authors suggested in this study that PGA should have an initial permeability of more than 100 m/day in order to guarantee proper performance through the service life.

Ranieri et al. (2010) conducted a similar study, in which they analyzed different factors influencing permeability in PGA during a year. The results obtained suggested that the increment of binder content is related with a reduction in permeability and porosity properties of the mixture. Additionally, the authors determined that the air void content of the mixture is directly correlated with the magnitude of the vertical permeability, as expected, and that this property is practically null when the air void content is smaller than 14% and when the porosity is smaller than 13% (Ranieri et al., 2010). In a different study, it was found that the binder type has little effect on the initial permeability of PGA, however with time, the decreasing rate of the permeability depends on it (J Chen, Lee, & Lin, 2016).

In terms of the impact of aggregate gradation on permeability, a different study demonstrated that there is a close relation between the aggregate size of the mixture for which 15% of the gradation is finer (D_{15}), and the permeability of the material (Martin et al., 2013). The results also showed that the permeability of the mixture increase with an increase in the value of D_{15}.

Martin et al. (2013) simulated in the laboratory clogging and de-clogging in a PGA and TLPGA by measuring permeability with the falling-head concept. To simulate clogging, soil dissolved in water was spread over the top of the surface of each specimen, and for the unclogging process, a conventional vacuum cleaner was used in order to reproduce an actual maintenance process conducted in real PGA pavements. The authors concluded that clogging occurs mainly in an early stage of the life of PGA, after which the structure clogs occurs more slowly, as presented in Figure 11.

In addition, the results specified that PGA mixtures were more susceptible to clogging after a few clog and unclog cycles. When comparing the two selected structures, they determined that even though it took more cycles to clog the TLPGA, it is less durable in the field as compared with a regular PGA. Clogging was also found to be more severe at higher temperatures.

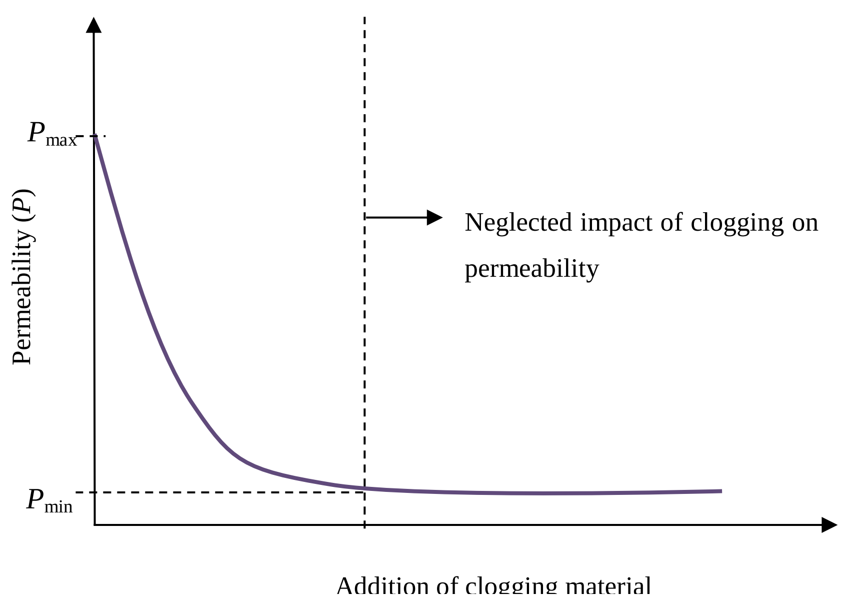


Figure 11. General tendency of permeability reduction with the addition of clogging material

Even though various maintenance processes exist to control clogging (e.g. fire hose or high pressure cleaning), Martin et al. (2013) demonstrated that after performing de-clogging cycles using a vacuum cleaner—which is the most popular technique used in Europe—only approximately 69% of the initial permeability was restored. In other words, these results suggest that permeability is permanently affected by clogging. This occurs because some sediment get into the internal void structure and cannot be removed with most de-clogging techniques.

Some studies have suggested that clogging could be controlled trough a 'self-cleaning' mechanism that consists of a pumping-suction effect that is produced by the tires of the vehicles when circulating at high speeds. Thus, high speeds and high volume characteristics are recommended when considering installing PGAs (Cooley Jr. et al., 2009; Newcomb & Scofield, 2004; Van Heystraeten & Moraux, 1990). Besides the speed and volume of traffic, it has also been recommended to use PGA with high air void contents and a large maximum aggregate side to prevent clogging (Isenring et al., 1990). Once constructed, clogging can be controlled through rigorous, frequent and specialized maintenance procedures that include the use of a fire hose, a high pressure cleaning or a special cleaning vehicle invented in Europe that sucks dirt (Huber, 2000).

The performance life of permeability (i.e., the time for which the PGA maintains its permeability properties in a level that guarantees proper functionality) has been reported to be highly variable and dependent on the characteristics to which the road is exposed. Isenring et al. (1990) reported that although the permeability in PGA materials could be preserved for more than five years without maintenance, under certain critical scenarios the layer could become almost impervious in a one-year period. Khalid and Perez (1996) reported a drainage functional life of a PGA mixture to be nine years in Spain for roads with medium traffic and seven years for layers submitted to heavy traffic and less than 20% air voids. In the province of Jiangsu in China, pavements with PGA have been reported to have satisfactory permeability values for a period of five years (Yu et al., 2014), while in Taiwan PGA maintain permeability for more than 8 years (J Chen et al., 2016).

2.4.1.2 Noise Reduction

Besides affecting the overall permeability of the mixture, clogging also impacts the noise reduction capacity of PGA materials. Anderson et al. (2013) produced a comprehensive report in which they analyzed the main advantages and disadvantages associated with the use of PGA in the state of Washington, to determine if the material was a viable option to control noise pollution. The study consisted of selecting several pavement segments to compare the noise level for over two years of three different types of mixtures: PGA modified with crumb rubber (PGA-AR), PGA modified with Styrene-Butadiene-Styrene (SBS) (PGA-SBS) and regular DGHMA. Some of the results obtained are summarized in Table 4. These results suggest that, in the initial measurements, the PGA-AR and the PGA-SBS materials presented an average reduction of 3.8 dBA and 3.2 dBA, respectively, in comparison to the DGHMA layer. However, in the final measurement, which was conducted after more than 40 months since the construction of the pavement, the average noise levels of the PGA materials were slightly under or even above the values reported for DGHMA surfaces. This observation showed that after certain period of time, the effectiveness of PGA to control and reduce noise decreases in the absence of correct maintenance.

Table 4. Increases in average noise levels (modified after Anderson et al., 2013)

Project	Reading	Average Noise Level		
		PGA-AR (dBA)	PGA-SBS (dBA)	DGHMA (dBA)
I-5 Lynnwood	Initial	95.1	96.0	99.4
	Final	103.3	102.2	103.5
	Increase over 48 months	**8.2**	**6.2**	**4.1**
SR 520 Medina	Initial	96.1	97.8	99.8
	Final	105.1	104.1	104.6
	Increase over 48 months	**9.0**	**6.3**	**4.8**
I-405 Bellevue	Initial	97.4	96.8	100.9
	Final	104.6	104.0	105.3
	Increase over 44 months	**7.2**	**7.2**	**4.4**
Average increase for all projects		**8.1**	**6.6**	**4.4**

PGA-AR = Porous Grades Asphalt modified with crumb rubber from recycled tires.

PGA-SBS=Porous Grades Asphalt modified with Styrene Butadiene Styrene.

DGHMA = Dense Graded Hot Mix Asphalt

Anderson et al. (2013) also analyzed the relationship between the average noise level reported on the pavement segments and the corresponding seasons of the year, since Washington has very cold winters with considerable snow and icing. Some of the results are presented in Figure 12, and they indicate that winter conditions worsen the acoustic performance of the PGA compared to that of regular DGHMA. This is a consequence of using studded tires, and of the increase in moisture damage generated by the existence of freeze-thawing cycles. As demonstrated in this study, Washington is an example of some difficulties associated with the functionality of PGA materials under hard winter conditions. Also, this situation supports the importance of developing research to identify and implement effective maintenance strategies to preserve the beneficial properties of PGA through the winter.

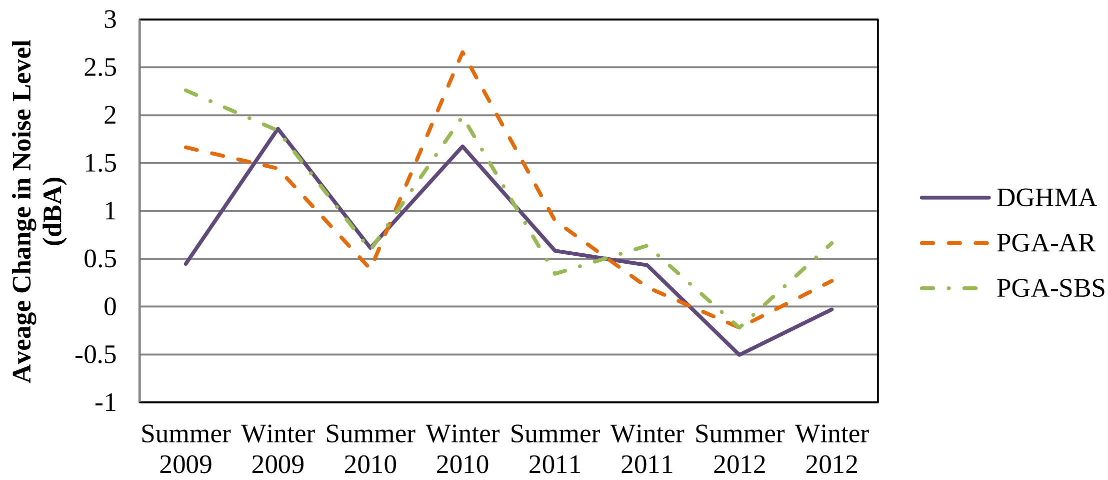


Figure 12. Seasonal change in noise levels at Washington Project I-405 Bellevue (modified after Anderson et al., 2013).

Isenring et al. (1990) determined that there is a close relationship between the permeability and the noise reduction capacity of PGA; however the macrotexture was the most determinant factor affecting the noise reduction capacity. In the same study, a relationship between the thickness of the layer and the gradation of the mixture was also determined. A PGA layer of 2 or more in (50.8 mm) thick was observed to potentially absorb more sound, while mixtures with smaller maximum aggregate sizes provided higher noise reduction values.

Similar to permeability, the performance life of the noise reduction capacity of PGAs is variable. Huber (2000) indicated that PGA mixtures perform well for 5 or more years, similar to what has been reported by Yu et al. (2014), who found a satisfactory performance for 5 years in the province of Jiangsu in China. In a different field study, Raaberg et al. (2001) found that in a case study on the island Zealand in Denmark the noise reduction capacity of a PGA pavement was significantly reduced after 7 years of service.

2.4.1.3 Texture and Friction

According to Cooley Jr. et al. (2009), there are two main aspects that determine the frictional properties of PGA: 1) permeability, and 2) texture. As explained previously, permeability and texture increase skid resistant and prevent hydroplaning. Mcdaniel et al. (2004) studied the friction resistance of pavements constructed in a field trial with three different surface layers: PGA, Stone Matrix Asphalt (SMA) and DGHMA, using three mechanisms: 1) Dynamic Friction Tester (DFT), 2) Circular Texture Meter (CTM), and 3) International Friction Index (IFI). The results, summarized in Table 5, show a larger average texture depth (i.e. the mean average vertical height differences of the texture) in the PGA as compared to the texture observed in the SMA and the DGHMA. In fact, the mean profile depth measured through the CTM for the PGA was 4.5 greater than the one for the DGHMA. In conclusion, the high texture of the PGA resulted in an improvement of the frictional properties of the PGA.

Table 5. Friction parameters of different pavement types (Mcdaniel et al., 2004)

Surface Type	Friction Measure		
	CTM Mean Profile Depth (mm)	DFT number at 20 km/h	Friction at a slip speed of 60 km/h (IFI)
PGA	1.37	0.51	0.36
SMA	1.17	0.37	0.28
DGHMA	0.3	0.52	0.19

The role of surface texture on the friction properties of the layer could be studied with more detail by separating macro- and micro-effects. In this context, macrotexture refers to the overall pavement surface texture. As reported by Martin et al. (2013), the macrotexture of the surface of the pavement is strongly related to the aggregate gradation of the mixture, especially with the percentage of material passing the No. 4 sieve and D_{15}. The authors noted that those mixtures with higher percentages of material passing the No.4 sieve had lower macrotexture levels, while the mixtures with higher values of D_{15} provided larger values of macrotexture.

Microtexture refers to the fine scale texture of the aggregates themselves, and this property has also been referenced to be an important factor in determining the frictional properties of PGA materials (Cooley Jr. et al., 2009). Microtexture depends on the general characteristics of the aggregate and the thickness of the asphalt film on the aggregates (Padmos, 2002). Yu et al. (2014) quantified the microtexture of pavement surfaces composed of PGA, SMA and DGHMA materials through the British Pendulum Test. The result of this test is the British Pendulum Number (BPN), which reflects the general level of friction of the layer. The results proved that even though the BPN values for the PGA, SMA and HDGMA layers were very similar, the PGA presented the highest values (i.e., for the truck lane the average BPN value was 74.7, 70.2 and 69.9, respectively), demonstrating that this material does present better friction properties.

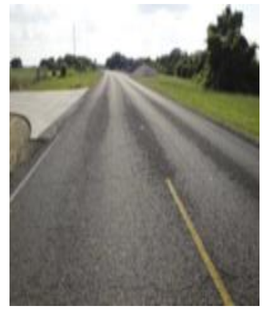
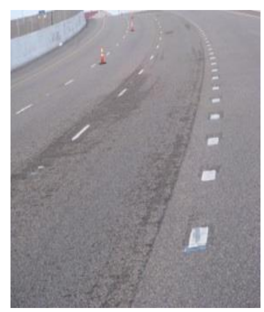
According to Isenring et al. (1990), PGA layers maintain their adequate frictional characteristics through their service life even after they become clogged. Fay and Akin (2013) stated that even during winter conditions, the friction values of PGA are generally the same or even better than the ones reported for DGHMA surfaces.

2.4.2 Durability

2.4.2.1 Common Distresses

As reported previously, the most common damage in a PGA structure is raveling. However, other distresses such as delamination, rutting due to studded tires and cracking, are also commonly mentioned. Based on this information, Table 6 summarizes and describes these distresses, their potential causes and frequency of appearance.

Table 6. Typical distresses of PGA.

Distress	Description	Causes	Frequency	Example
Raveling	Loss of aggregates at the surface of the pavement. (Arambula, 2014a)	Repeated abrasion by traffic Presence of moisture Low binder content Oxidation of the film over time Cooling of the mixture at placement Excessive compaction Asphalt draindown Asphalt aging Early stop-and-go traffic	Very common	
Delamination	Sections of the surface layer that have come loose from the pavement. (Road Science, 2015)	Inadequate tack coat application Excessive cooling in placement Moisture susceptibility	Common	
Rutting	The traditional rutting caused for plastic deformation is rare. The type of rutting appearing is usually described as raveling within the wheel paths. (Cooley Jr. et al., 2009)	Studded tire wire	Not a major issue	
Cracking	Longitudinal or transverse cracks usually at joints.	Reflective cracking / inadequate structure	Not a major issue	

In terms of raveling, some recent research has demonstrated that it is mainly a fracture Mode I process, highly influenced by the volumetric properties of the mixtures and loading conditions (Manrique-Sanchez, Caro, & Arámbula-Mercado, 2016). Molenaar and Molenaar (2000) have explained that there are two different forms of raveling: 1) the short-term raveling that happens in new surfaces due to the high level of shear forces between the tire and the pavement, and 2) the long-term raveling that occurs because of the separation of the binder from the aggregates, partially due to draindown problems during the production and placement of the mixture. To reduce short-term raveling it is important to guarantee proper construction processes, especially during the placement and compaction of the mixture. In terms of long-term raveling, modified asphalts or fibers are utilized to prevent long-term draindown due to gravity (Molenaar and Molenaar, 2000); these products promote stronger adhesive and cohesive properties within the microstructure of the mixture, which in turn reduces the possibilities of aggregate dislodgment (Huurman et al., 2010).

Another main reason causing raveling and decreasing the durability of PGA materials is binder aging. Alvarez et al. (2006) present a comprehensive summary of the importance of aging in PGA design. Oxidative aging hardens the binder, which makes the material more brittle and more susceptible to fracture or cracking (Lin et al., 1995; Liu et al., 1996). Due to their open structure, PGA mixtures have more exposure to air and water, resulting in accelerated oxidative hardening processes (Cooley Jr. et al., 2009). Other reasons for the high oxidation rates observed in PGA cited by Alvarez et al. (2006) are the high temperatures to which the surface is exposed in comparison to the temperatures existing deeper in the pavement structure (i.e., higher temperatures cause an increase in oxidation rates according to most oxidation chemical kinetics models reported in the literature, (Liu et al., 1996)), the high permeability and the thick binder films.

Another potential cause for raveling reported in the literature is associated with the quality of existing PGA mix design procedures and the volumetric properties of the material (Huber, 2000; Kandhal, 2002). In 2009, Miradi et al. utilized artificial intelligence to model PGA performance (Miradi et al., 2009). The authors constructed two models to predict raveling during the initial 5 and 8 years of service of the pavement using visual data collected in a number of sections for a period of 10 years and applying artificial neural network theory. The results showed that raveling is mostly influenced by the asphalt content of the mixture and its total air void content, but other parameters such as the percentage of coarse aggregates and the number of cold days also proved to influence both the appearance and magnitude of this distress. In terms of the asphalt content, the authors found that mixtures with less than 4% by weight of the total mixture were related to inadequate performance. In terms of air void content, it was demonstrated that raveling increased with an increase in the total air void content of the mixture (Miradi et al., 2009).

Other numerical modeling of raveling (Arambula, 2014b) have also reviewed the role of different parameters of the mixture in the potential development of raveling, as well as the role of some relevant external conditions (i.e., load and environment). Table 7 presents a summary presented by Arambula (2014b) regarding the importance of various parameters in the promotion of raveling based on existing numerical models of this phenomenon.

Table 7. Relevant parameters in promoting raveling based on previous numerical modeling efforts (modified after Arambula, 2014b)

Parameter	Importance on promoting raveling				Reference
	Low	Medium-low	Medium	High	
Diameter of particles				X	(Mo et al., 2007)
Mastic film thickness (binder content)				X	(Mo et al., 2007)
Air void content				X	(Mo et al., 2007)
Presence of water			X		(Mo et al., 2011, 2014)
Temperature		X			(Huurman et al., 2010b)
Asphalt type				X	(Mo et al., 2008; Mo et al., 2011, 2014; Mo et al., 2010)
Contact angle of adjacent aggregates - condition of particle skeleton- particle packing				X	(Mo et al., 2007, 2008)

In terms of mitigation methods for raveling, researchers mention the efficiency of polymer-modified mixtures (J Chen et al., 2016; Jian-shiuh Chen, Chen, & Liao, 2013; Shirini & Imaninasab, 2016; Suresha, Varghese, & Shankar, 2009). Additionally, it has also been suggested to use high-viscosity binders (HVB), which are binders with extremely high kinetic viscosity at 60° C (i.e. >300,000 Pa.s) and softening point (>100º), to improve the overall performance of PGA (Ma, Li, Cui, & Ni, 2016). Also, the use of fibers and lime has been related to higher raveling resistance (Hassan & Al-Jabri, 2005; Ma et al., 2016).

New technology is also emerging in order to combat raveling, specifically the use of high frequency induction curing. The process basically consists of adding a steel wire mesh/ Hot Dip Galvanized Wire (GI) wire to a PGA to make the road electrically conductive and suitable for induction heating. When microcracks are going to occur in the asphalt mastic (i.e. bitumen and fines), the wire mesh/GI is heated externally to cure the binder and prevent the formation of major cracks and thus raveling (Neogi, Sadhu, & Banerjee, 2016).

Additionally, it has been suggested to reconsider the test procedures used to simulate long term-aging of the mixtures in the laboratory, since the time currently utilized does not represent the real aging impact on the performance of the mixture through time (Alvarez et al., 2006).

2.4.2.2 Service Life

Service life refers to the time between the moment at which the PGA mixture is placed and compacted and the time when the mixture needs to be replaced or rehabilitated (Cooley Jr. et al., 2009). The service life of PGA is highly variable, but it can range from 7 to 12 years. Table 8 presents a summary of PGA service life information that was obtained from the current literature review. In terms of the deterioration rate, Cooley Jr. et al. (2009) and Pucher et al. (2004) used existing data to conclude that the deterioration rate of PGA is slow during the initial 5-10 years but it significantly increases after this period.

Table 8. Typical service life of PGA.

Country	**Mixture**	**Service Life (years)**	**Reference**
United States	PGA	8 or more (in more than 70% of the states)	(Mallick et al., 2000)
	PGA	6-10 (in more than 45% of the states)	(Yildirim et al., 2007)
	PGA	8-10	(Watson and Tran, 2015)
The Netherlands	PGA	12 (average)	(Voskuilen et al., 2004)
	Two-layer PGA	8 (average)	(Hofman et al., 2005)
Japan	PGA	8-10	(Takahashi, 2013)
Spain	PGA	10 (average)	(Ruiz et al., 1990)
United Kingdom	PGA	7-10	(Nicholls, 2001a)

There are many factors determining PGA service life, including traffic conditions and the type of binder used. Nicholls (2001a) determined that the typical service life of PGA in the United Kingdom was typically up to 12 years in roads with traffic below 4,000 commercial vehicles per lane per day. Other authors have associated the total service life to the type of binder used in the mixture. The use of modified binders has been reported as necessary in several surveys performed in Europe and the US to guarantee longer service lives (Cooley Jr. et al., 2009; Huber, 2000). An example of this is the use of PGA modified with crumb rubber (AR-PGA). The use of this material in pavements in Arizona has reduced reflective cracking and has permitted to PGA service lives to reach up to 13 years (Huber, 2000; Way, 2003).

This section presented a comprehensive evaluation of the performance of the mixture; however, it is also necessary to study what are mix design considerations, in terms of materials and procedures that must be taken into account when designing a PGA layer. The following section covers this research.

2.5 Mix Design Considerations

This section presents relevant mix design considerations for PGA mixtures. Initially, the criteria for materials selection is described, followed by a presentation of some of the most commonly aggregate gradations and methods used for determining the Optimum Binder Content (OBC). Finally, some of the tests used for characterizing the mixtures are presented. Each section includes specifications of several states and agencies in the US, and of some countries in Europe and Asia.Table 9 presents the references from where the specifications described in these sections were obtained.

Table 9. References of the specifications and standards utilized in section 2.5

Agency / State / Country	Reference	Agency / State / Country	Reference
Alabama	(Alabama Department of Transportation, 1999, 2012)	Oregon	(Oregon Department of Transportation, 2008)
Arizona	(Arizona Department of Transportation, 2008)	South Carolina	(South Carolina Department of Transportation, 2007)
California	(California Department of Transportation, 2006, 2012)	Tennessee	(Tennessee Department of Transportation, 2015)
Wyoming	(Wyoming Department of Trasnportation, 2010)	Texas	(Texas Department of Transportation, 2004)
Florida	(Florida Department of Transportation, 2010, 2014a)	Virginia	(Virginia Department of Transportation, 2012)
Georgia	(Georgia Department of Transportation, 1989)	Louisiana	(Louisiana Department of Transportation, 2013)
Mississippi	(Mississippi Department of Transportation, 2005, 2014)	NAPA / NCAT*	(Kandhal, 2002)
Nebraska	(Nebraska Department of Transportation, 2015)	FHWA	(Federal Highway Administration, 1990)
Nevada	(Nevada Department of Transportation, 2014)	ASTM**	(ASTM, 2013a)
New Jersey	(New Jersey Department of Transportation, 2007)	Spain	(Asociación Española de Normalización y Certificación, 2007)
New Mexico	(New Mexico Department of Transportation, 2012, 2014)	United Kingdom	(British Standards, 2008)
North Carolina	(North Carolina Department of transportation, 2012)	Germany	(Deutsches Institut für Normung, 2006)
Oklahoma	(Oklahoma Department of Transportation, 2009)	China	(Yu et al., 2014)

* National Asphalt Pavement Association / National Center for Asphalt Technology

**American Society for Testing and Materials

2.5.1 Materials Selection

The initial stage in the design of a PGA material is the selection of materials, which consists of defining the source and type of aggregates, asphalt binder, and stabilizing additives or modifiers that will be used as part of the mixture. Certainly, the materials properties of the individual constitutive phases of the mixture are essential to assure a proper performance of the mixture, and are usually described in detail in the specifications that were herein reviewed, as explained next.

2.5.1.1 Aggregates

Cooley Jr. et al. (2009) conducted a survey among several DOTs in the US, in which they were asked to rank from the most to the least important the characteristics of the aggregates in PGA mixtures that are required for guaranteeing a good performance. According to the results, the most important characteristics were polish resistance and durability; in a second level of importance were reported angularity, abrasion resistance, particle shape and cleanliness; and the least important characteristic according to the surveyed DOTs was aggregate absorption.

Although polish resistance was reported to be one of the most important requirements of aggregates for PGA mixtures in the US, none of the revised specifications have a precise requirement regarding this aspect. In Europe, aggregate polish resistance is also considered important, and the Polish Stone Value is usually specified as a requirement (Lefebvre, 1993; Ruiz et al., 1990). In Europe, aggregate polish resistance is also considered important, and the Polish Stone Value is usually specified as a requirement (ASTM, 2009a).

Aggregate durability is usually measured through the soundness of the coarse aggregates. This parameter is commonly measured immersing and saturating the aggregates in sulfate, following the procedure described in the American Association of State Highway and Transportation Officials (AASHTO) standard 104 (AASHTO, 2011). China, Nevada and Oregon specify a maximum soundness loss of 12%, while Virginia, South Carolina and North Carolina permit values equal or smaller than 15%. Texas and Wyoming require maximum values of 20%, while Tennessee has the strictest requirement with a maximum soundness loss of 9%.

Aggregate abrasion resistance is an important characteristic to guarantee the resistance to crushing and degradation through the service life of the material. In terms of abrasion resistance, the most common test used worldwide is Los Angeles Abrasion that is specified in the ASTM C 131 standard (ASTM, 2003). This test consists of introducing the aggregates in a rotating steel drum containing a specified number of steel spheres. The test is usually conducted at 500 revolutions, although some states specified it at 100 revolutions. At 500 revolutions, the maximum percentage of abrasion loss has been specified as 30% in Oklahoma, Oregon, Tennessee, ASTM (D7064M-08) and NAPA/NCAT. Other states, such as Virginia, Arizona, Wyoming, California and the general design methodologies specified by the FHWA specified a maximum loss of 40%, while New Jersey and South Carolina specify a maximum abrasion loss of 50% and 52%, respectively, which are the most permissible values among all specification reviewed. At 100 revolutions, the maximum abrasion loss values requirements were 9% in Arizona and 12% in California.

In terms of the angularity, this property is usually determined using the number of fractured faces of the coarse fraction of the aggregates. According to Kandhal (2002), the most commonly test utilized to determine this parameter is the ASTM D 5821 (ASTM, 2006), which visually determines the characteristics and fractured faces in a previously known mass of washed aggregates. Almost every standard reviewed specifies this property in terms of the percent of the granular material that has 2 or more fractured faces, and the percent of the aggregates with 1 or more fractures faces. For 2 or more fractured faces, values in the specifications range between 75% and 90% for most agencies, while for 1 or more fractured faces values range between 75% and 100%. Table 10 summarizes the values of the specifications presenting this requirement for 19 different Agencies in the US.

Table 10. Aggregate Angularity Requirements for PGA Mixtures

State / Agency	Min % of 2 or more fractured faces	Min % of 1 or more fractured faces
AZ	85	92
CA	75	90
LA	ns	90
MS	ns	90
NE	90	95
NV	ns	90
NM	ns	75
NC	90	95
OK	95	100
OR	75	90
SC	ns	90
TN	90	100
TX	ns	90
VA	90	100
WA	ns	85
WY	90	95
ASTM	90	95
NAPA /NCAT	90	100
FHWA	75	90

*ns= not specified

Particle shape is another important morphological property of aggregates, and it is usually evaluated by means of two parameters: 1) the flakiness index, and 2) the flat and elongated index. The method used to determine these properties is described in ASTM D 4791 (ASTM, 2005), and it consist in determining the ratios of width to thickness, length to thickness and length to width of individual coarse aggregate particles. The flakiness index is the percentage by weight of a sample of particles whose least dimension (i.e. thickness) is less than three-fifths of its mean dimension. This parameter is only specified by Arizona, which permits a maximum value of 25%. The flat and elongation index is defined as the percentage by total weigh of a set of aggregates having a ratio of width to thickness or length to width greater than a specific value. The requirements are commonly expressed in terms of as a 5:1 ratio or a 3:1 ratio. For the 5:1 ratio, all the states that have specifications for PGA aggregates properties (i.e., Nebraska, New Jersey, North Carolina, Oklahoma, Oregon, Texas, Virginia and ASTM (D7064M-08)), define a maximum permitted value of 10% of the total weight of the aggregate sample. For the 3:1 ratio, maximum required values of 20% by total weight of the sample are specified by Mississippi, Tennessee and the NAPA/NCAT, and of 25% are specified by Louisiana and Arizona.

The cleanliness of the aggregates is evaluated through the sand equivalent test, which aims to determine the amount of silt, clay contamination, or clay-size aggregate particles in the fine portion of the aggregates, as described in ASTM D 2419 (ASTM, 2009b). According to this standard, this is a rapid method conducted in the field to determine the changes in the quality of the aggregates during construction. The term "sand equivalent" expresses the concept that most fine aggregates are mixtures of desirable coarse particles (e.g., sand) and generally undesirable clay or plastic fines and dust. Only 6 states (i.e., Arizona, North Carolina, Oregon, Louisiana, and Wyoming) and the ASTM (D7064M-08) specified sand equivalent values requirements. All these agencies specify a minimum sand equivalent of 45% by weight, except for Arizona that requires a minimum of 55%.

Even though in the survey conducted by Cooley Jr. et al. (2009) most agencies specified that absorption was the least important characteristic of the aggregates in a PGA mixture, it can deeply affect the performance of the material (Kandhal, 2002). If the aggregates are highly absorptive, there could be an important reduction of the effective asphalt content, which is the responsible for binding the individual particles, increasing the potential rate of deterioration of the layer (Kandhal, 2002). To regulate this property, the parameter used is water absorption, which is determined by saturating the aggregates with water and then weighing them; then, the aggregates are dried in the oven and reweighed; the difference between these two values divided into the dry weight is defined as the absorption capacity of the material. These methodologies are specified in the ASTM C 127 (ASTM, 2015). China, New Jersey, Virginia and the guidelines provided by NAPA/NCAT, require a maximum water absorption of 2%, while Arizona define a maximum value of 2.5%, and Nevada of 4%.

All the aggregate properties previously described are summarized in the APPENDIX B.

2.5.1.2 Asphalt Binder

Similar to DGHMA, the selection of the asphalt binder for PGA depends on different factors, including traffic, weather, and expected performance (Kandhal, 2002). Around the world, asphalt binders are commonly classified based on their penetration, a test specified through the ASTM D 5 (ASTM, 2013b), their viscosity, a test specified in ASTM D2171 (ASTM, 2010), or their Superpave Performance Grading (PG), specified in the AASHTO PP6 and AASHTO MP1. In general, stiffer binders are used in PGAs to prevent draindown and short-term raveling (Molenaar and Molenaar, 2000). According to Kandhal (2002) the PG of the asphalt binder used in a PGA is generally two grades stiffer than that normally used in a specific zone according to the Superpave specifications. However, it is important to guarantee the use of a binder that is manageable during construction.

Asphalt binders in Europe are usually classified based on their penetration. The general European normative of porous asphalt, which is currently used in countries like Spain, Germany, Italy, Belgium, and the United Kingdom, specifies binders with penetration values between 35/50 to 240/330 (1/10 mm). In the US, there is a wide range of asphalt binders' PG that could be used in the fabrication of PGA mixtures, as listed in Table 11. Binder modification typically includes the use of polymers or rubber. The most common polymers include the Styrene Butadiene Styrene (SBS), the Ethylene Vinyl Acetate (EVA) and the Styrene Butadiene Rubber (SBR). The rubber used for asphalt binder modification is obtained after a process of recycling the discarded tires of vehicles and trucks. Most of the PGA mixtures commonly used in the US use asphalt binders modified with one or both of these products. For example, in New Mexico the "Rubberized Open Graded Friction Course" contents a minimum of 10% of recycled rubber and a minimum 2% of SBS by weight, as described in their Standard Specifications for Highway and Bridge Construction.

Table 11. Binders Used in PGA Surface Courses in the United States

State	Binder used for PFC	State	Binder used for PFC
AZ	PG 54-22/ AR-ACFC (20 percent ground tire crumb rubber by weight of the binder content).	*NM*	PG 70-28+, PG 70-28R**+
AL	PG 76-22	*NC*	PG 76-22, PG 64-22
CA	Depends on the weather; one of the most common is PG 58-34 and rubberized RHMA PG 64-16	*SC*	PG 76-22
FL	PG 76-22, ARB-12, ARB-5	*TX*	PG 76-XX (low temperature found with SuperPave PG procedure), AR meeting Type I or Type II requirements
MS	PG 76-22	*VA*	PG 70-28
NJ	PG 64E-22, PG64-22R**, PG 58-28R**		

** Letter "R" refers to asphalt rubber

2.5.1.3 Additives and Other Products

As it has been mentioned before, the higher binder content of PGAs results in thicker asphalt films, which are associated with a higher potential of the mixture to develop draindown. To reduce this phenomenon, two types of additives are generally utilized: 1) cellulose fibers, and 2) mineral fibers. There are other types of fibers (e.g., textile fibers) that have also proved to reduce draindown (Hassan and Al-Jabri, 2005). The review of the majority of PGA-related specifications in US permits to conclude that they permit to select either cellulose or mineral fibers to control draindown and to help assuring the proper performance of the mixture. The first fiber is commonly utilized at a dosage rate of 0.4% by total weight of the mixture, while the second is commonly applied at a dosage rate of 0.3% by total weight of the mixture. Fibers are usually added to the mixture in a loose or petellized form, either in the pugmill or in the weigh hopper, manually or automatically.

Other products that are commonly used in PGAs are anti-stripping agents. The most common product used for this purpose is hydrated lime, which is usually used as a filler at a rate of 1%-1.5% by total weight of the aggregate (Hassan and Al-Jabri, 2005). Besides being an effective anti-stripping, this material also reduces hardening of the binder, thus increasing the overall durability of the mixture (Alvarez et al., 2006). Additionally, there are liquid anti-strip additives that are usually used in mixtures that contain limestone aggregate. Some states such as California and Louisiana suggest a minimum rate of dosage of 0.5% of these liquid products by total weight of the binder.

2.5.2 Aggregate Gradation for PGA Mixtures

After selecting and fulfilling the properties required for the individual components of the mixture, the selection of the aggregate gradation of the mixture is the following stage in the design process of a PGA mixture. For PGAs, the two main volumetric properties that should be guaranteed are the amount of stone-stone contact, in order to minimize rutting, and a final high percent of voids content, in order to guarantee the proper functionality of the mixture (Kandhal, 2002). The first property is directly related to the gradation of the mixture, and the majority of the countries, states and agencies have already established upper and lower limits that are permitted for PGAs. These gradations are described and compared in the following sections.

2.5.2.1 Aggregate Gradations in the US

The review of the specifications and standards of different states and agencies resulted in an ample gradation range for the design of PGA mixtures. These gradations are specified as envelopes through the definition of upper and lower limits. In order to compare some of the characteristics of these gradations, the maximum aggregate size of the envelope (i.e., the smallest sieve of both the upper and lower limits, through which 100 percent of the aggregate sample particles passes) is going to be analyzed. Since several states have more than one gradation envelope (e.g., Georgia has two gradations, California three and Texas four), the comparison among the different gradations was conducted using the largest maximum aggregate among all possibilities. This analyzes permitted to conclude that the maximum aggregate size of the PGA mixtures in the US can be classified between 3/8 in (9.51 mm), ½ in (12.5 mm) and 1 in (25.4 mm). Table 12 shows the classification of the states and agencies in these three groups; from where it can be observed that the majority of the entities permit a maximum aggregate size of ½ in.

Table 12. Maximum Aggregate Size of PGA gradations.

Maximum aggregate size		
Sieve 3/8" (9.51 mm)	**Sieve 1/2" (12.5 mm)**	**Sieve 1" (25.4 mm)**
• Arizona. • Nevada*. • New Mexico. • Oklahoma. • Wyoming. • FHWA.	• Alabama. • Florida. • Georgia*. • Mississippi*. • Nebraska. • New Jersey*. • North Carolina*. • South Carolina. • Tennessee. • Texas*. • Virginia*. • Louisiana. • NAPA/NCAT. • ASTM	• California*. • Oregon*.

*The states marked with an asterisk had two or more gradation envelopes. For comparison purposes, the one with the largest maximum aggregate was selected.

The gradation envelopes of the entities mentioned above are presented in APPENDIX C. Although there is a wide range of gradations for PGAs in the US, there are some characteristics that are shared by most specifications. For example, most gradations seem to be gapped between the sieve 3/8 in (9.51 mm) and No. 4 (4.76 mm) since there is an absence of intermediate aggregate sizes. Also, the material passing the sieve No. 200 (i.e., mineral filler) ranges between 0% and a maximum of 8%. Finally, typical values of the coefficients of uniformity and curvature for these gradations are 4 and 1, respectively.

2.5.2.2 Aggregate Gradation in Other Countries

The European Normative EN 13108-7 that is currently used in countries like Spain, Germany and the United Kingdom specifies the gradation of the PGA. However, the process is different than in US since this normative provides more autonomy for selecting some gradation characteristics, such as the maximum nominal size. The process initiates by selecting one of the two following set sieves:

- Basic sieve set plus set 1: 4 mm, 5.6 mm, 8 mm, 11.2 mm, 16 mm, and 22.4 mm.
- Basic sieve set plus set 2: 4 mm, 6.3 mm, 8 mm, 10 mm, 12.5 mm, 14 mm, 16 mm and 20 mm.

It is not allowed to combine sieves sizes from sets 1 and 2. A fine sieve between 1mm, 0.5 mm, 0.25 mm and 0.125 mm, should be also selected. The overall limits of the expected gradation should meet the criteria specified in Table 13; where *D* is a sieve from the sieve set selected that includes one or two sieves between *D* and 2 mm.

Table 13. Overall limits of target composition (modified after British Standards, 2008)

Sieve (mm)	% Passing by mass
1.4 *D**	100
D	90-100
2	5-25
0.063	2-10

* If this number is not an exact sieve of the ISO 565/R 20, approximate this value to the next nearest sieve in the set.

Besides of these overall limits, the normative also specifies the width of the gradation envelope. Table 14 presents the ranges between the maximum and minimum values for the selected gradation envelope; the normative specifies that a single value can be selected between these limits.

Table 14. Ranges between maximum and minimum values for the selected grading envelope (British Standards, 2008).

Sieve (mm)	Ranges (% by mass)	
	Smallest range	**Widest range**
Optional sieves between D and 2	10	20
2	0	7
Optional sieves between 2 and 0.063	4	15
0.063	1	5

During the current literature review, it was also possible to determine the gradation requirements for PGA mixtures in the province of Jiangsu in China. In this province, the maximum aggregate size permitted–based on the same definition than in the US (section 2.5.2.1) – is 0.53 in (13.2 mm), which is similar to the maximum aggregate size of ½ in (12.5) that is commonly specified in the US.

2.5.3 Optimum Binder Content (OBC) Methods

After selecting the materials and the aggregate source and gradation, the design of the mixture is conducted to determine the optimum binder content or OBC. PGAs usually have asphalt contents between 5% and 8% by total weight of the mixture, but the procedures used to determine this percentage are highly variable among agencies. Kline (2010) and Putman (2012) made a comprehensive research in which they compared PGA mix design methods utilized in US. From this document and the research conducted, the different PGA design methods have been classified into four main groups: 1) those that use compacted specimens, 2) those that are based on the absorption of the predominant aggregate fraction, 3) those that are based on a visual determination technique, and 4) those that combine some of the first three methods.

2.5.3.1 Design Methods Based on Compacted Specimens

This design method evaluates certain properties of the mixtures on compacted specimens. Mixtures are prepared using a range of asphalt binder content (between 3 and 5 different contents) with a set of three or four specimens or replicates per content. After mixing, the material is compacted using the (SGC), the Marshall methodology or other methods or devices specified by a specific agency. The following volumetric properties are determined from the compacted specimens:

- Maximum Specific gravity (G_{mm}) (this property is performed in the loose mixture not in the compacted specimens).
- Bulk Specific Gravity (G_{mb}).
- Effective Specific Gravity of the aggregate (G_{se}).
- Air Voids (V_a).
- Voids Filled with Asphalt (*VFA*).
- Voids in the Mineral Aggregate (*VMA*).
- Voids in the dry-rodded coarse aggregate fraction of the job mix formula aggregate skeleton (VCA_{drc}).
- Voids in the coarse aggregate fraction of the compacted mix (VCA_{mix}).

After completing the volumetric analysis, different tests are performed to determine the properties of the PGA mixture. The most common properties include permeability, draindown potential, abrasion loss on aged (i.e. specimens that have been conditioned to simulate aging in the binder of the mixture) and unaged specimens through the Cantabro test, stone-on-stone contact, and moisture damage sensibility (typically through the Tensile Strength Ratio (TSR)). The asphalt binder content (P_b) that fulfills specific volumetric and performance properties requirements defined by each state, country or agency, is the one selected for the PGA mixture.

According to the reviewed conducted as part of this report, from the 20 states that currently use PGA, 13 of them uses this design method and this method is also proposed in the design guidelines provided by ASTM and NAPA/NCAT. Table 15 summarizes the parameters required in each state or agency within this methodology.

The ASTM mix design method also provides some advice for the cases where some of the requirements are not fulfilled. To reduce draindown values, for example, they recommend reducing the asphalt binder content or to change the type of stabilizer used. If the air voids are under the specification, it is advised to reduce the asphalt binder content. In terms of abrasion, if the unaged loss is higher than the required value, it is recommended to increase the asphalt binder content. Also, if the loss of the aged specimens is larger than the specification, this methodology suggest increasing the asphalt binder content and to modify the selected additives.

Table 15. Requirements for mix design methods based on compacted specimens.

State	Number of binder contents used	Superpave Compaction (No. Gyrations)	Air void (%)	Binder content (%)	Permeability (Min. m/day)	Draindown (Max. %)	Abrasion loss aged (Max. %)	Abrasion loss unaged (Max. %)	TSR (Min. %)	Stone-stone contact (VCA_{mix}^{*})
MS	4 (intervals of ± 0.5%)	50	15	-	30	0.3	40	30	-	$< VCA_{drc}$
NE	4		18±1	5.8 - 6.8		0.3				
NM	4 (intervals of ± 0.5%)	50	≥18	≥ 5.5	92	0.3			80	$< VCA_{drc}$
OK		50				0.2				
OR	3 (4.5%, 5.5 % and 6.5%)	50	13.5-16						80	
TN	3 (intervals of ± 0.5%)	50	≥18		100	0.3	30	20		
TX	3 (increment intervals of 0.5%)	50		≥ 6				20		
VA		50	≥16			0.3		20	80	$< VCA_{drc}^{**}$
LA		50	18-24			0.3			80	
NAPA/NCAT	3 (intervals of ± 0.5%)	50	≥18		100	0.3	30	20		
ASTM	3 (intervals of ± 0.5%)		≥18		100	0.3	30	20		

*VCA_{mix}: Voids in Coarse Aggregate (i.e., voids in the coarse aggregate fraction of the mixture).

**VCA_{drc}: Voids in Coarse Aggregate in a the dry-rodded coarse aggregate (i.e., volume in between the coarse aggregate skeleton of the mixture corresponding to the final job mix formula).

Recently, The National Center of Asphalt Technology (NCAT) finished the project NCHRP 01-55: Performance-Based Mix Design of Porous Friction Courses related to the improvement of PGA mix design (D. Watson, Tran, Rodezno, & James, 2016). The objective of the project is to mitigate many of the life cycle issues encountered with PFC pavements. This study used laboratory performance tests to evaluate three PGA pavements that had good field performance (up to 18 years) and three PGA pavements that had poor performing field performance (less than 8 years). A balanced mix design approach which utilizes compacted specimens was selected for designing PGA pavements. Criteria and performance tests for durability, cracking and cohesiveness were selected and are presented in Table 16.

Table 16. NCAT mix design requirements.

Property	Requirement
Air voids (%)	15 to 22
Abrasion loss (%)	20 max
Shear Stress (psi)	75 min
Permeability (m/day)	Meet agency criteria (50 min recommended)
Conditioned Tensile strength (psi)	50 min
Unconditioned Tensile strength (psi)	70 min
Tensile strength ratio (TSR)	0.70 min
Draindown (%)	0.30 max
Hamburg Wheel Tracker (cycles before reaching 12.5 mm (0.5 in.) rut depth)	PG 64 or lower; ≥ 10,000 passes, PG 70, ≥15,000 passes, PG 76 or higher, ≥ 20,000 passes

2.5.3.2 Design Methods Based on the Determination of Absorption of the Predominant Aggregate Fraction

The oil absorption method is the one specified by the FHWA in their guidelines from 1990. It was verified that it is currently used by Alabama and, according to Putman (2012), it is also used in Arizona and Wyoming. The general procedure starts by determining the surface capacity of the predominant aggregate fraction (which is usually that passing the sieve 3/8 and retained in the sieve No. 4). Then, it is necessary to separate and oven dry 100 gr of the material. After that, the material (aggregates without binder) is placed in a metal funnel with a wire mesh at the bottom, usually like that of the sieve No. 10. The funnel that contains the aggregates is completely immersed in the Society of Automotive Engineers S.A.E. No. 10 lubricant oil, typically during 5 minutes at air temperature. After this, the material is let to drain for 2 additional minutes. Afterwards, the funnel and the mixture are placed in the oven during 15 minutes at a temperature close to 140°F. Finally, the sample is poured into a tared pan, where it is let to cool down and weighed again. With the apparent specific gravity and the difference between the weighs before and after the submersion in oil, the Percent of Oil Retained (POR) is calculated. Once the POR value has been computed, a surface constant value (K_c) is determined and used in a specific empiric formula to obtain the design OBC value.

2.5.3.3 Design Method Based on Visual Determination

This category of mixture design was detected to be in current use in three states: Florida, Nevada and South Carolina. The process consists of preparing between 1000 gr and 1200 gr of mixture at different asphalt contents. The mixtures are placed into clear Pyrex dishes or "pie plates". The dishes containing the mixtures are placed in the oven for about 1 or 2 hours at the mixing temperature, which is approximately 320°F. Then, the dishes are retired from the oven and the material is letting to cool down at room temperature. The plate with the mixture is inverted and the bottom surface of the dish is inspected. The optimum asphalt content must show sufficient bonding between the plate and the mix without evidencing too much drainage. Figure 13 was obtained from the Florida DOT standard specifications, and it perfectly exemplifies the visual determination method.

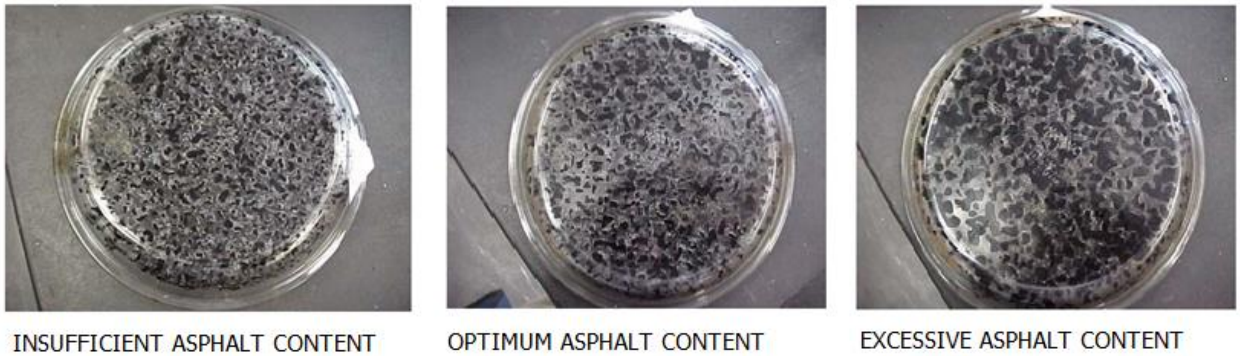


Figure 13. Example of the visual determination method (modified after Florida Department of Transportation, 2014).

California uses the same draindown principle to determine the OBC. However, they place the specimens in extraction thimbles instead of letting them in the oven, and they quantify the draindown by mass instead of determining it visually. In this case, the thimbles are subjected to a compaction of a 4kg mass on top mixture. The mixture is weighed before and after the extraction process in the thimbles, in order to calculate the total mass of asphalt binder that has been drained. The amount of drained asphalt is plotted against the asphalt content, and the point in which the drainage gets a value of 4 gr corresponds to the OBC.

2.5.3.4 Combination of the Three Methods

Georgia and New Jersey use a combination of the three design methods previously described. Georgia uses the compacted specimens' method with 3 asphalt contents at 0.5% intervals, and they determine the OBC by selecting the asphalt content that produces the minimum value in a graph of VMA vs. P_b. New Jersey uses the same procedure but requires 5 binder contents. For the absorption determination method both states use the procedure described using the S.A.E No. 10 oil. Finally, for the visual determination method, New Jersey uses 3 binder contents while Georgia uses 4 contents to visually determine the OBC as specified by this method. The final OBC for both states results from averaging the individual OBC values that are obtained from each one of the three methods. Similar to the other states, both states suggest performing moisture susceptibility, abrasion and permeability tests on the mixture, once the OBC has been determined.

2.5.3.5 OBC Methods Proposed by Other Countries and Guidelines

The European normative EN 13108-7 (currently utilized in Spain, United Kingdom and Germany, among other countries) does not specify a single methodology to determine the OBC. The specification lets the designers to decide a minimum binder content between 3% and 7%. This binder content is multiplied by a correction factor, α, which is determined with Equation 1, where ρ_a is the apparent particle density in megagrams per cubic meter:

$$\alpha = \frac{2.650}{\rho_a} \qquad \text{(Equation 1)}$$

Another design methodology was proposed by Khalid and Walsh (1995) in the United Kingdom. The design procedure consists on evaluating the properties presented in Table 17 in mixtures with different asphalt contents. Then, the ranges of binder contents that satisfy each property are overlapped and the mid-point is identified and taken as the OBC of the mixture.

Table 17. Current mix design method (modified after Khalid and Walsh, 1995) .

Binder Content	Mixture Property	Procedure
Maximum	Binder draindown	Binder drainage test.
Maximum	Voids content	Volumetric measurement.
Maximum	Voids structure	Falling-head permeability test.
Minimum	Elastic Stiffness	Repeated Load Indirect Tensile Test (RLIT)
Minimum	Retained Stiffness	Soaked RLIT
Minimum	Durability/ adhesiveness	Cantabro

The procedure to design the mixture plays a very important role on the PGA performance on the mixture. Nevertheless the pavement design considerations, and more specifically the determination of the thickness of the PGA layer has an important role too but has not been center of PGA research. The following section summarizes the literature found on the pavement design considerations when using PGA.

2.6 Pavement Design Considerations

PGA is usually conceived to have no structural capacity and, hence, this layer is not often considered in the pavement design process. Thus, only the safety and environmentally related advantages are utilized when conducting benefit/cost analysis, and the potential profits in strengthening the pavement structure are not taken into account (Wang et al. , 2013). California, for example, refers to PGA as a “sacrificial surface course” used to extract water and enhance the service life of the underlying pavement (CalTrans, 2006).

In this regard, Oregon is probably the state of the US that has more experience in having considered or studied the structural properties of PGAs. This state conducted deflection testing and analysis, and they determined that the deflection reduction of PGA was comparable to that of a DGHMA of a similar thickness (Scott et al., 2000). Consequently, the state uses the same structural coefficient for a PGA layer as that of a DGHMA material as part of the pavement design methodology. Comparisons in the mechanical response of pavements with PGAs, in which the structural properties are considered the same for PGA and DGHMA, have been reported in Spain and the United Kingdom as well (Khalid and Perez, 1996).

According to Bolzan et al. (2001), the resilient modulus of a PGA was found to be approximately 60 percent of that of conventional DGHMA mixtures. Van der Zwan et al. (1990) indicated that the dynamic modulus of common PGAs is about 70 to 80 percent of the DGHMA, while Van Heystraeten and Moraux (1990) reported that PGA provided 73 to 79 percent of the structural capacity of a typical DGHMA. Wang et al. (2013) analyzed the impact of a PGA layer in a pavement structure using the mechanistic–empirical pavement design method, and the study revealed that this layer reduces the tensile strain at the bottom of the concrete asphalt layer and the top of sub-grade soil, which are the two performance parameters that are correlated with fatigue and permanent deformation in this method, thus suggesting an increase in the performance life of the pavement. They determined that the effect of one unit thickness of OGFC in a warm climate (Gainesville, Florida) is approximately equivalent to two-third unit thickness of DGHMA(Wang et al., 2013). In some other study, Timm et al. evaluated the structural coefficient of PGA from deflection data collected with the Falling Weight Deflectometer (FWD) apparatus. Results found determined that PGA has a structural coefficient of 0.15 and that there is a 12% in required pavement thickness to achieve the same structural number as a DGHMA(Timm H & Vargas-Nordcbeck, 2012).

There are no formal methods for determining the typical thickness of PGA layers. Nevertheless, some authors such as Ranieri (2002) and Cooley Jr. et al. (2008) have proposed some methodologies based on the hydraulic conductivity and the rainfall intensity of the zone of the project. Also, some pavement design software as FPS 21 developed by the Texas Department of Transportation (TxDOT), include this type of mixture with their corresponding modulus and Poisson ratio and permit the user to include it as another layer of the pavement structure. However, generally speaking, agencies have standard thickness values of PGA layers that have been defined based on the experience. These values range between 19.05 mm (0.75 in) and 76.2 mm (3 in); a summary of typical thickness layer values used for PGAs worldwide is presented in Table 18.

Table 18. Typical thickness of PGAs.

State/ Country	Layer Thickness	Reference
Switzerland	28 - 50 mm (1.10 -1.97 in)	(Isenring et al., 1990)
Spain	40 mm (1.57 in)	(Ruiz et al., 1990)
Belgium	40 mm (1.57 in)	(Van Heystraeten and Moraux, 1990)
Germany	40 mm (1.57 in)	(Stotz and Krauth, 1994)
Netherlands	25-55 mm (1-2 in)	Survey presented in section 2.1.3
California	1.2-1.8 in (30.5-45.7 mm)	Survey presented in section 2.1.3
Mississippi	1 - 1.25 in (25.4-31.75 mm)	(Mississippi Department of Transportation, 2014)
New Jersey	2 - 1.25 in (25.4-31.75 mm)	Survey presented in section 2.1.3
Oregon	50 mm (1.97 in)	(Moore et al., 2001)
Argentina	51 mm (1.97 in)	(Comision Permanente del Asfalto, 2015)
Maine	1 - 3 in (25.4-76.2 mm)	Survey presented in section 2.1.3
Nevada	0.75 in (19.05 mm)	Survey presented in section 2.1.3
New Mexico	0.625 in (15.88 mm)	Survey presented in section 2.1.3
Georgia	0.75-1.25 in (19.05-31.75 mm)	Survey presented in section 2.1.3

In addition, some recent research has identified the importance of taking into account not only the thickness but also the width of the pavement for determining and optimizing permeability and skid of the road. Specifically, a late research used artificial neural network concepts to model the water-fil thickness and skid resistance of pavements with PGA and multiple lanes. They found that the skid resistance of an outer lane is always lower than an inner lane and that the wider the width of a pavement from its central crown to the outermost lane, the larger is the difference between the skid resistance in the innermost and the outermost lanes (L. Zhang, Fwa, Ong, & Chu, 2016).

Finally, the literature review permitted to conclude that there are important challenges associated to the current use of PGAs. The development of more scientifically sounded mix design methods, better quality assurance practices, and the development of standardized maintenance procedures specially in winter conditions are, probably, the most relevant aspects to advance towards more durable PGA layers.

3. EXPERIMENTAL DESIGN

This chapter aims to describe the experimental design proposed for this study. The first section of the chapter described the materials utilized in the study and its principal characteristics. Then, a general overview of all the experimental design with the detail combination of types of specimens, types of tests and types of materials is presented. After this, the procedures for fabricate the specimens are presented; finally, the functionality and durability tests utilized in this study are described.

3.1 Materials

The materials that were used to fulfill the objectives of the study were obtained from four different field projects to represent the four environmental zones proposed by the Strategic Highway Research Program (SHRP)-Long-Term Pavement Performance (LTPP) (i.e. wet-freeze, dry-freeze, dry-no freeze and wet-no freeze) shown in Figure 14.

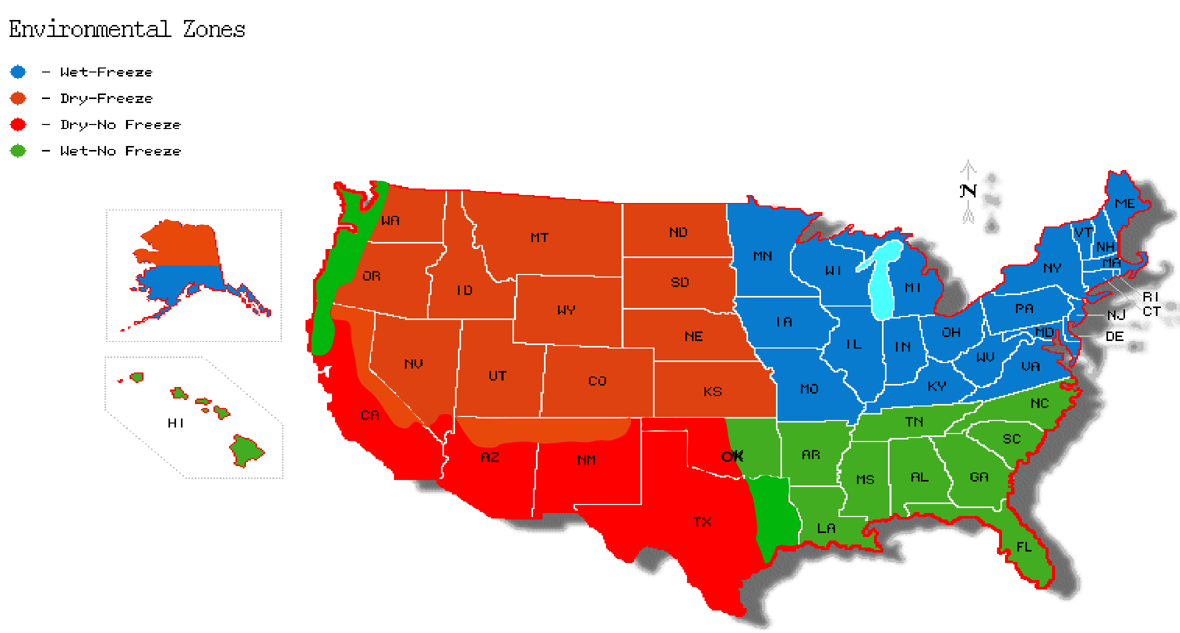


Figure 14. SHRP-LTPP environmental zones (modified after Hadley, 1994).

The first project was in the wet-no freeze zone in the state of Florida. This project consisted of a 30-mile long stretch road located on IH 95 in Broward and Miami-Dade counties in Florida (FL). During construction of this and every field project, personnel were onsite coordinating the material and field core collection, preparing onsite specimens, taking field measurements and documenting all construction activities. The detailed construction report for the Florida project is presented in APPENDIX D. The second project was in the state of Utah (UT) in the dry-freeze zone. This project was 4 miles long and is located south of downtown Salt Lake City. The detailed construction report for this project is presented in APPENDIX E. The third project in the wet-freeze zone is located in the south end of the Garden State Parkway (GSPKWY) in Cape May County, New Jersey (NJ). This paving job was part of a larger project of improving Interchanges 9-11 of GSPKWY undertaken by New Jersey Turnpike Authority. The detailed construction report for this project is presented in APPENDIX F. The fourth and final project is located on the Texas A&M University campus in Bryan, Texas (TX) in the dry-no freeze zone. This project is a test pad section constructed by a research group at the Texas A&M Transportation Institute (TTI) to evaluate the impact resistance of delineators when mounted on concrete and asphalt surfaces, one of which was PGA. Details of the construction of this project are presented in APPENDIX G.

Taking into account that the materials come from different parts of the country and were supplied under different specifications, there are some important differences in their characteristics that must be underscored. In terms of the gradations, the mixes from Florida and Texas tend to be coarser than the ones in New Jersey and Utah (Figure 15). The nominal maximum aggregate size for the materials from Texas and Florida is 3/4” (19 mm), while for New Jersey and Utah is 3/8” (9.5 mm).

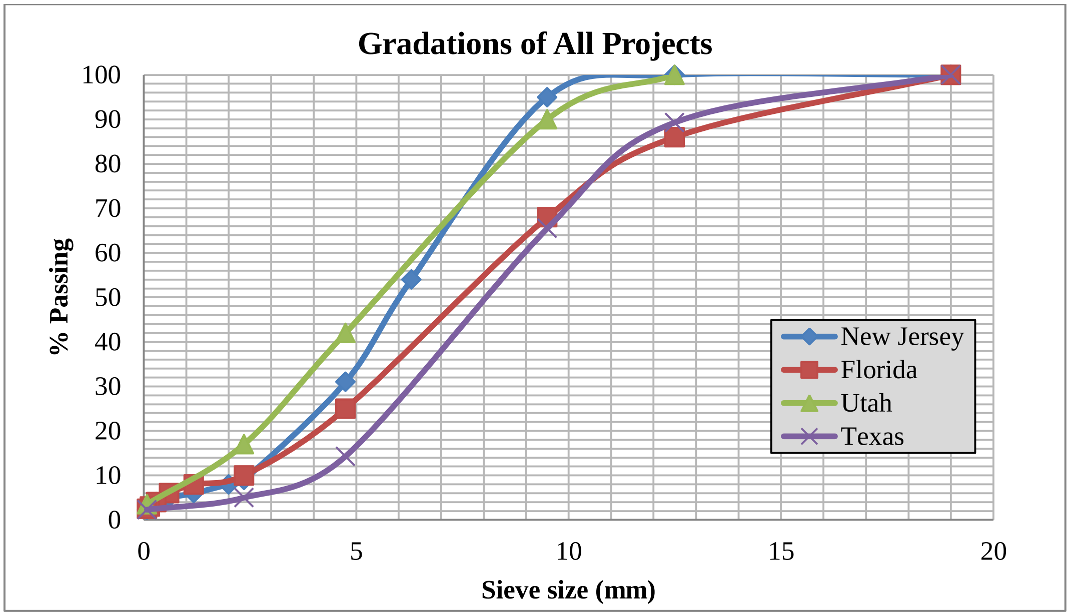


Figure 15. Aggregate Gradations.

The other volumetric characteristics compared were binder type, % binder, % fibers and thickness of the PGA layer. These data are summarized in Table 19. New Jersey has the highest % binder (i.e. 8%), followed by Florida (i.e. 6.50%), Texas (i.e. 6.10%) and Utah (5.9%). Utah also has the binder with the lowest PG at high and low temperatures. Only Florida and Texas contain fibers, and the range of thicknesses for the mixtures varies between 0.75 in and 2 in. All the detailed mix designs are presented in each of the construction reports presented in APPENDICES D-G.

Table 19. Characteristics of the mixtures utilized in the study.

Characteristic	New Jersey	Florida	Utah	Texas
Binder Type	PG 64-22 AR	PG 76-22	PG 58-28	PG 76-22
% Binder	8%	6.50%	5.90%	6.10%
% Fibers	0	0.40%	0	0.30%
Thickness (in)	2 (51 mm)	0.75 (19.05 mm)	1.5 (38.1 mm)	0.75(19.05 mm)

3.2 General Experimental Design

From each field project, two different types of materials were collected: 1) the raw aggregates, binder, and additives separately which were mixed and compacted later in the laboratory as Laboratory Mixed-Laboratory Compacted (LMLC) specimens; and 2) the material delivered from the plant that was placed in the field and also compacted in the laboratory as Reheated Plant Mixed-Laboratory Compacted (RPMLC) specimens. Two other types of samples were also collected:1) field cores extracted directly from the pavement after construction and 2) Plant Mixed-Plant Compacted (PMPC) specimens that were compacted from the plant mix in the field in a portable laboratory

The main objective of this study is to optimize the durability and functionality of PGA. To evaluate functionality, two main characteristics were assessed: permeability and noise absorption. Permeability was assessed with three different permeameters (i.e. TxDOT permeameter, NCAT permeameter and Florida Department of Transportation (FDOT) permeameter) while noise absorption was evaluated with the impedance tube. In terms of durability, abrasion resistance (Cantabro test), rutting potential (Hamburg Wheel Test), fracture susceptibility (Semi Circular Bending test) and moisture susceptibility (Indirect Tensile Test) were evaluated. These tests mentioned require different specimen sizes. In general three main types of specimens were used: 1) SuperPave Gyratory Compacted (SGC) specimens with a height of 4.5 in (114.3 mm) (i.e. SGC4.5”); 2) SGC specimens with a height of 2.5 in (63.5 mm) (i.e. SGC2.5”) and 3) slabs of 20x20 in (508x508 mm). In addition, cores were also extracted from the slabs to produce the fourth type of specimen called Slab Cores (i.e. SC).

From the literature review it was identified that there are two main characteristics that optimize the functionality and durability of PGA: the thickness of the layer and the AV of the mixture. In order to determine the impact of these two characteristics, two AV were analyzed with most of the tests. The two AV correspond to the original AV that the mix was designed for (i.e. Design AV) and the actual AV found in the field (i.e. Construction AV). In terms of the thickness, three thicknesses which covered the range found in the literature were evaluated: 0.75 in (19.05 mm), 1.5 in (38.1 mm) and 2.5 in (63.5 mm). For each of the thicknesses, the two AV were analyzed. The thickness variation was only included for the slab specimens, while the AV variation was included for both the slabs and SGC4.5” specimens.

A summary of the combinations of these characteristics (i.e. type of material, type of test and type of specimens) utilized for the laboratory experiment in this study is shown in Table 20 along with the number of specimens for each type of material, test and specimen and for each field project.

Table 20. Summary of type of materials, type of specimen, and specimen quantity for each type of test for each field project.

Type of Material	Type of Test	Type of specimen	No. of Specimens at Design AV	No. of Specimens Construction AV	No. of Specimens with Thickness 0.75 in *	No. of Specimens with Thickness 1.75 in *	No. of Specimens with Thickness 2.5 in (for both AV)*	Total No. of Specimens
LMLC	Cantabro test (x7 conditioning protocols)	SGC 4.5"	21	21				42
PMPC	Hamburg Wheel Test	SGC 2.5"	4					4
PMPC	Semi Circular Bending Test	SGC 2.5"	2					2
PMPC	Indirect Tensile Strength	SGC 2.5"	6					6
RPMLC	Noise Absorption	Slabs	6	6	4	4	4	12
RPMLC	Permeability (NCAT, TXDOT)	Slabs	6	6	4	4	4	12
RPMLC	Cantabro test (x6 conditioning protocols)	Slab Cores	54	54	36	36	36	108
RPMLC	Permeability (FDOT)	Slab Cores	9	9	6	6	6	18

* This number of specimens corresponds to both Design (50% of them) and Construction AV (50 % of them). For instance, there are 2 specimens of 0.75 in for noise absorption with Design AV and 2 for Construction AV.

In addition, the study has a component of field testing. The field testing consisted of both measurements made directly in the field and also the extraction of cores and subsequent evaluation in the laboratory. The field testing performed consisted of permeability measurements with TxDOT and NCAT permeameters soon after construction and after one year in service, while on the cores the principal properties measured were air voids and permeability with the FDOT permeameter after construction and after one year in service. To summarize the information provided in this section, Figure 16 provides a general flow chart of the general experimental design. The following sections of this chapter describe the processes for the specimen fabrication and each of the tests performed in this study.

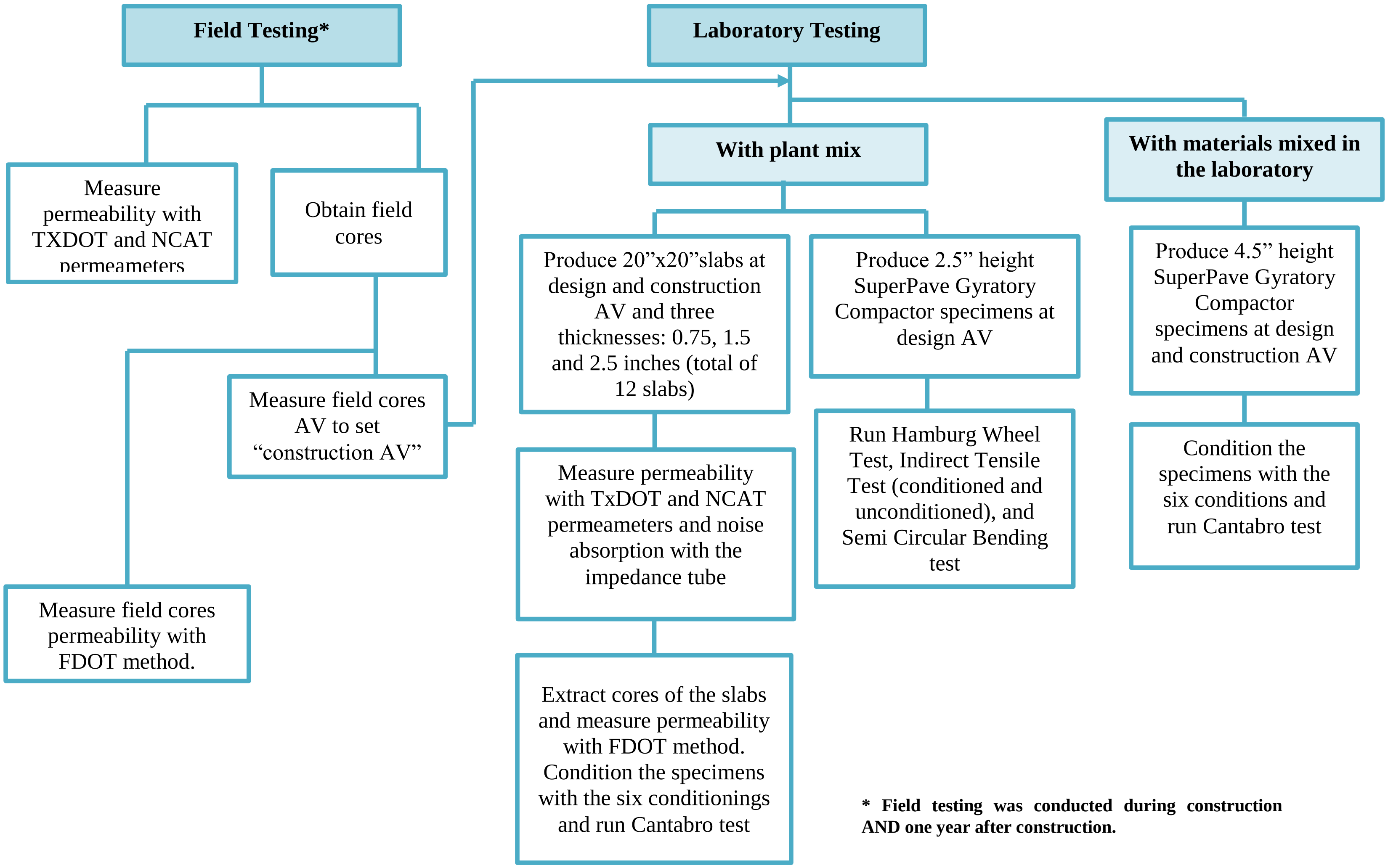
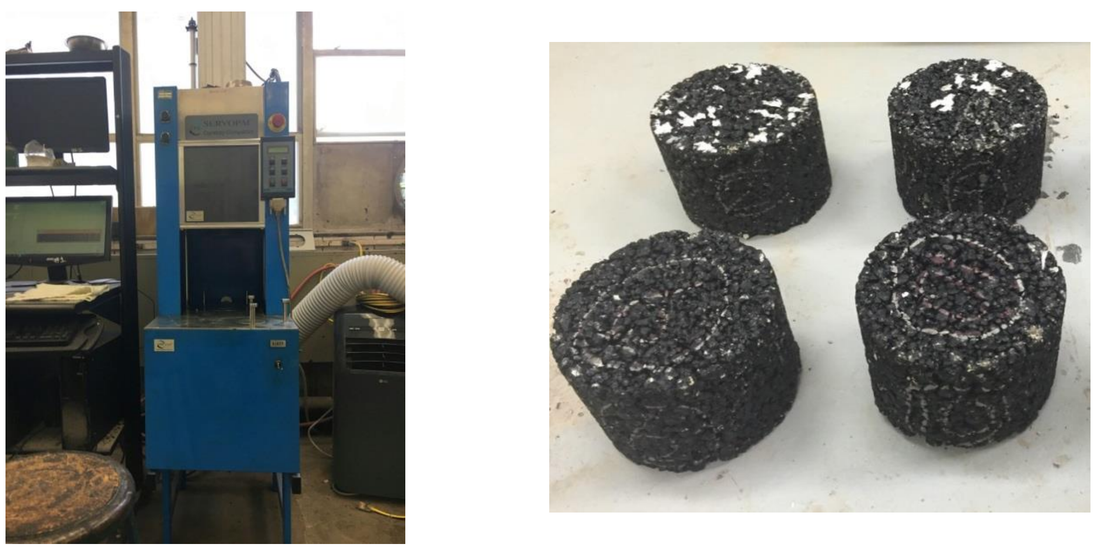


Figure 16. General experimental design.

3.3 Specimen Fabrication

3.3.1 SGC Specimens

Before the fabrication of the different types of specimens, a wet sieve analysis of the mixture was performed according to AASHTO T11(AASHTO, 2005). The preparation of the SGC specimens started by mixing the source of aggregates in the proportions specified in the mix design, and then sieving the combined material in order to obtain the individual aggregate sizes. Once the material was sieved, batches with the gradation adjusted based on the wet sieve analysis were prepared and then mixed with the hot binder and the additives. Then the specimens were molded in the SGC to the specific height desired. After compaction, the AV of the specimens was measured to verify that the target was met within a ±1% tolerance. A picture of the SGC and the final product after compaction are presented in Figure 17.



(a) **(b)**

Figure 17. (a) SGC equipment and (b) final product after compaction cylinders of 4.5 in height.

3.3.2 Slab Specimens

Slabs were prepared using the RPLMC material and a linear kneading compactor (LKC). This machine utilizes a series of heated steel plates that are vertically aligned on top of the loose plant mix, and a steel roller that compresses the material into a slab of constant predetermined thickness (Federal Highway Administration, 2000). The process started by heating the mix until it was loose and workable for compacting and also heating the plates of the compactor. The linear compactor was prepared by putting some wax paper in the bottom and preheating it for two hours. When the compactor, the plates and the mixtures were all preheated; the material was poured into the compactor as shown in Figure 18(a). The material was spread out such that it was as equally distributed as possible. Immediately after this, the plates were put one by one on top of the mixture as shown in Figure 18(b). Once all the plates were inserted (Figure 18(c) and (d)), the compaction process started. As shown in Figure 18(d), the plates were at first uneven on the outside of the mold, so the compaction proceeded until all the plates were even with the mold. Then, the machine was stopped and the plates were taken off. The slab was left to cool for at least 5 hours, and then it was unmolded. The final slab is shown in Figure 18(f).

(a)

(b)

(c)

(d)

(e)

(f)

Figure 18. (a) Pouring of the mixture in the compactor, (b) Colocation of the plates in the compactor, (c) Final set-up before compacting (d) Plates completely into the mold (e) Compacting process (f) Final product: slab.

During the period of compaction, there was a malfunction in the LKC, which due to the age of the equipment was not able to be resolved. The laboratory acquired a new asphalt roller compactor (ARC), which molds slabs through a roller foot with multiple linear rotating compactors simulating the compaction of a steel wheel roller. After an analysis and comparison of the two equipment through results from different tests (i.e. Cantabro, AV and permeability), it was determined that it was safe to assume that producing slabs with the ARC compactor will not be detrimental to the objectives of this study. The comparison analysis and the specifications for this equipment are presented in APPENDIX H.

3.3.3 Coring and Drilling

To extract the cores from the slabs, a Hilti coring drill was utilized as shown in Figure 19(a). A total of nine cores were extracted per slab with the slabs compacted with the LKC and six from the ones compacted by the ARC (extra slabs were made with the ARC to get to same number of cores as from those with the LKC). An example of the core extraction and final product is presented in Figure 19(b).

(a)

(b)

Figure 19. (a) Coring with the Hilti coring drill and (b) cores extracted from the slabs.

3.4 Functionality Tests

3.4.1 Noise Absorption Measurements

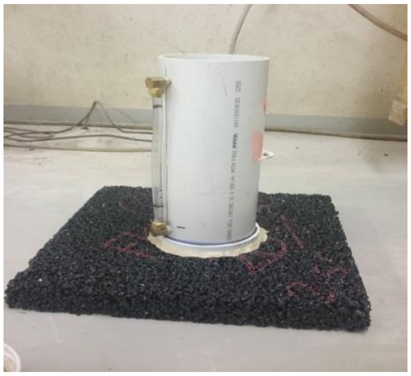
The noise absorption test was performed in the laboratory according to ASTM E1050 using an impedance tube such as the one shown in Figure 20(ASTM, 2012). In this test, plane waves are produced in the tube using a broad band signal from a noise source, while sound pressures are measured simultaneously at two locations spaced on the side wall of the tube.

Figure 20. Impedance tube test setup.

3.4.2 Permeability Measurements

Three different apparatus were used for measuring permeability. All three methods follow the falling head permeability principles, but there are some variations in the geometry of the equipment and the type of the specimens used in each methodology. The TxDOT permeameter test is performed according to Tex-246-F and employs a cylindrical PVC tube equipped with a pipette as shown in Figure 21(a) (Texas Department of Transportation, 2009). The NCAT permeameter is a transparent plastic device with different cross sectional areas as presented in Figure 21(b) (D. E. Watson, Cooley, Moore, & Williams, 2004).

Both the NCAT and TxDOT permeameters are placed and sealed on top of a flat compacted surface before the beginning of the test. Permeability tests with the FDOT apparatus are done in accordance with FDOT FM 5-565 using 6.0 inch diameter specimens as shown in Figure 21(c). Besides differences in the specimen geometry, the FDOT apparatus utilizes specimens whose sides have been sealed with Vaseline; and the equipment also applies pressure to the sides of the specimen with a latex membrane that surrounds the specimen to prevent lateral water flow (Florida Department of Transportation, 2014b).



(a)

(b)

(c)

Figure 21. (a) FDOT apparatus; (b) TXDOT permeameter; and (c) NCAT permeameter;

The general calculation of the coefficient of permeability for the falling head permeability tests is done utilizing Equation 2.

$$\boldsymbol{k} = \frac{aL}{At}\ln\left(\frac{h_1}{h_2}\right) \qquad \text{(Equation 2)}$$

Where,

k = coefficient of permeability (cm/s);

a= inside cross sectional area of the buret (cm^2);

L= thickness of the test specimen (cm);

A = cross sectional area of the test specimen (cm^2);

t = elapsed time between h_1 and h_2 (s);

h_1 = initial head across the test specimen (cm); and

h_2 = final head across the test specimen (cm).

3.5 Durability Tests

3.5.1 Cantabro Test

In the Cantabro test, initially proposed in Barcelona (Spain) and specified in ASTM C131 (ASTM, 2003), the PGA mixture was first compacted. In Texas, two replicates of 5.9 in diameter and 4.5±0.2 in height, compacted in the SGC at 50 gyrations are required. The mass of the specimen before the test was recorded as W_1. Then, the specimen was placed in the Loss Angeles abrasion machine without the steel balls. The machine is operated at 77 ± 2 °F (25 ± 1 °C) at a speed of 30-33 rpm for 300 revolutions. After this, the specimen is reweighed and this value is recorded as W_2. The setup of the machine and specimens before and after the test are presented in see Figure 22. The abrasion loss was calculated as presented in Equation 3:

$$Abrasion\ loss\ (\%) = \frac{W1-W2}{W2} * 100 \qquad \text{(Equation 3)}$$

The Cantabro Abrasion Loss was calculated for both aged and unaged specimens. In order to age the mixture, the AASHTO PP2-01 specifies putting the specimen in the oven at 185 °F

(85°C) for 120 hours, and then cooling it down at 77°F (25°C) for 4 hours before testing (AASHTO, 2001).

Figure 22. Los Angeles abrasion machine, Cantabro loss test setup.

An important problem found in the literature is that the general conditioning protocols performed currently (i.e. unaged and aged) for the Cantabro test do not include an analysis of moisture or low temperature damage. To explore these effects, seven conditioning protocols were used in this study:

1. *Unaged dry*: without any conditioning, considered as the control.
2. *Unaged dry-freeze*: placing the samples in an environmental chamber at 32°F (0°C) for 24 hours and testing immediately.
3. *Aged*: placing the specimens inside an environmental chamber for 7 days at 140°F (60°C), and then stabilizing at room temperature for 24 hours.

4. *Unaged wet-freeze*: following the procedure indicated in AASHTO T283 with 10-min vacuum saturation and one freeze/thaw cycle (Figure 23) while the specimen is submerged in water and drying.

(b)

(a)

Figure 23. (a) Samples in freeze cycle and (b) thawing cycle according to AASHTO T283.

5. *Unaged wet-hot:* submerging the unaged specimens for 24 hours in a circulating water bath at a constant temperature of 140°F (60°C) and drying.
6. *Unaged moisture induced stress tester (MIST):* subjecting the specimen to cyclic stress and pore pressure at 140°F (60°C) using the MIST machine (Figure 24) at a pressure of 40 psi and 1000 load cycles and drying.

Figure 24. MIST machine.

7. *Aged wet-freeze:* placing the specimens inside an environmental chamber for 7 days at 140°F (60°C), stabilizing at room temperature for 24 hours, following the procedure indicated in AASHTO T283 with 10-min vacuum saturation and one freeze/thaw cycle while the specimen is submerged in water and drying.

The conditioning protocols utilized allowed for assessment of the response of the mixture in its original state, after moisture damage with and without freezing (i.e., conditioning with three different methodologies) and at low temperature and after aging when the asphalt is more brittle and susceptible to raveling.

For the some of the seven conditioning protocols (i.e. aged, unaged wet-hot, unaged wet-freeze and aged wet-freeze) in some of the materials, it was necessary adapt a brace to prevent collapse of the mixture in the specimen while permitting homogeneous conditioning throughout the sides of the specimens. The brace was fabricated with wire mesh and tightened using an adjustable clamp as shown in Figure 25.

Figure 25. Brace for preventing collapse of the mixture in some conditioning protocols.

3.5.2 Indirect Tensile Strength (IDT) and Tensile Strength Ratio (TSR) Test

The most common method used to quantify the moisture susceptibility of PGA mixtures is the same as that used for regular DGHMA: the modified Lottman test as specified in AASHTO T283 (AASHTO, 2003). The procedure consisted of preparing at least eight compacted cylindrical specimens 6 in (150 mm) in diameter and 2.5 in (63.5 mm) in height. The specimens were separated into two subsets of at least three specimens each, so that the average of the air voids content is approximately the same in both subsets. One of the subsets was called the *dry* subset, and the other was called the *wet* subset. The *dry* specimens were stored at 77 ± 2 °F (25 ± 1 °C) before testing. For the *wet* subset, specimens were submitted to a vacuum saturation process conducted in order to initiate the conditioning process by quantifying their final saturation state. Then, the specimens were immersed in freeze water at approximately 0 °F (-18°C) for 24 hours. Finally, the specimens were placed in warm water (77°F) for an additional two hours. Once the conditioning process was over, both subsets of specimens were submitted to a tensile strength loading test, as illustrated in Figure 26. The average tensile strength of the wet subset of specimens divided by the average tensile strength of the dry subset is the defined as the Tensile Strength Ratio (TSR) which was the parameter used to indicate moisture susceptibility.

Figure 26. IDT test setup.

3.5.3 Hamburg Wheel Test

The Hamburg Wheel Tracking Test (HWTT) was used to evaluate rutting and stripping potential. The test consisted of tracking a 158 lb (71.7 kg) loaded steel wheel back and forth directly on a mixture specimen submerged in water at 50ºC (122 °F) as shown in Figure 27. The current standard for the HWTT is AASHTO T324 (AASHTO, 2016b), and in this case Super Pave Gyratory compacted specimens were used.

Figure 27. Hamburg Wheel Test Setup.

Two methodologies were used: AASHTO method and TAMU method. The AASHTO method is the one described in the standard AASHTO T324 (AASHTO, 2016b) while the TAMU method is a novel methodology proposed at Texas A&M University and detailed explained in the paper *Novel Method for Moisture Susceptibility and Rutting Evaluation Using Hamburg Wheel Tracking Test* (Yin et al., 2014).

3.5.4 Semi-Circular Bending(SCB) Test

The Semi-Circular Bend (SCB) or I-FIT test is a relatively new test performed on a semicircular asphalt mixture specimen with the flat side on two rollers that are covered with a friction reducing material. The load is applied along the vertical diameter of the specimen, and the load and load line displacement are measured during the entire duration of the test. The test has a provisional AASHTO standard: AASHTO TP124-16 (AASHTO, 2016a). A setup of the test is presented in Figure 28. The output of the SCB test is a load versus displacement curve. The calculation methodologies are described in detail in Chapter 5.

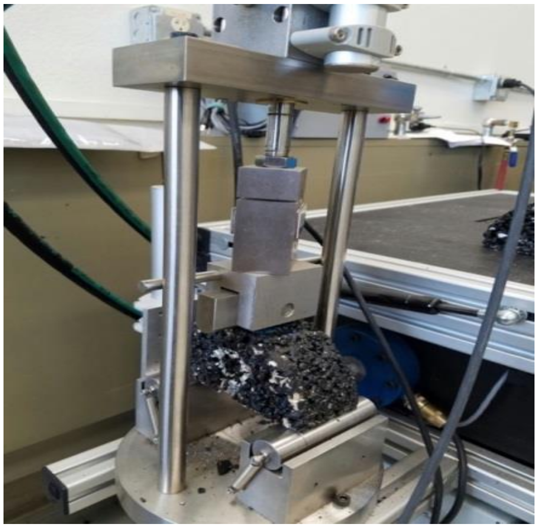


Figure 28. SCB test setup.

4. FUNCTIONALITY DATA AND RESULTS ANALYSIS

This chapter presents the results and discussion related to the functionality evaluation of the PGA mixtures in slabs and slab core specimens as described in Chapter 3. The materials utilized for this analysis were from Florida, New Jersey and Texas. The material from Utah could not be utilized because the slabs fell apart after compaction as shown in Figure 29. The functionality analysis was divided into two main components: permeability and noise reduction. The results are presented in this chapter.

Figure 29. Slab compacted with material from Utah.

4.1 Permeability Analysis

In terms of permeability; the impact of the type of permeameter, thickness, AV, type of mixture, aging state and type of measurement were analyzed independently. The results are presented in the following sections.

4.1.1 Type of Permeameter

The permeability analysis was completed using three different apparatuses: the TxDOT, NCAT and FDOT permeameters. Permeability with the TxDOT and NCAT permeameters was measured on compacted slabs, while the FDOT apparatus was utilized for cores extracted from these same slabs. Figure 30, Figure 31 and Figure 32 present the coefficients of permeability for the six structures evaluated (i.e. three thicknesses at two AV each) for the materials from Florida, New Jersey and Texas, respectively. The bars represent the average of the eighteen measurements (i.e. nine per slab, two slabs per structure) performed per structure, and the error bars represent ± one standard deviation from the average value.

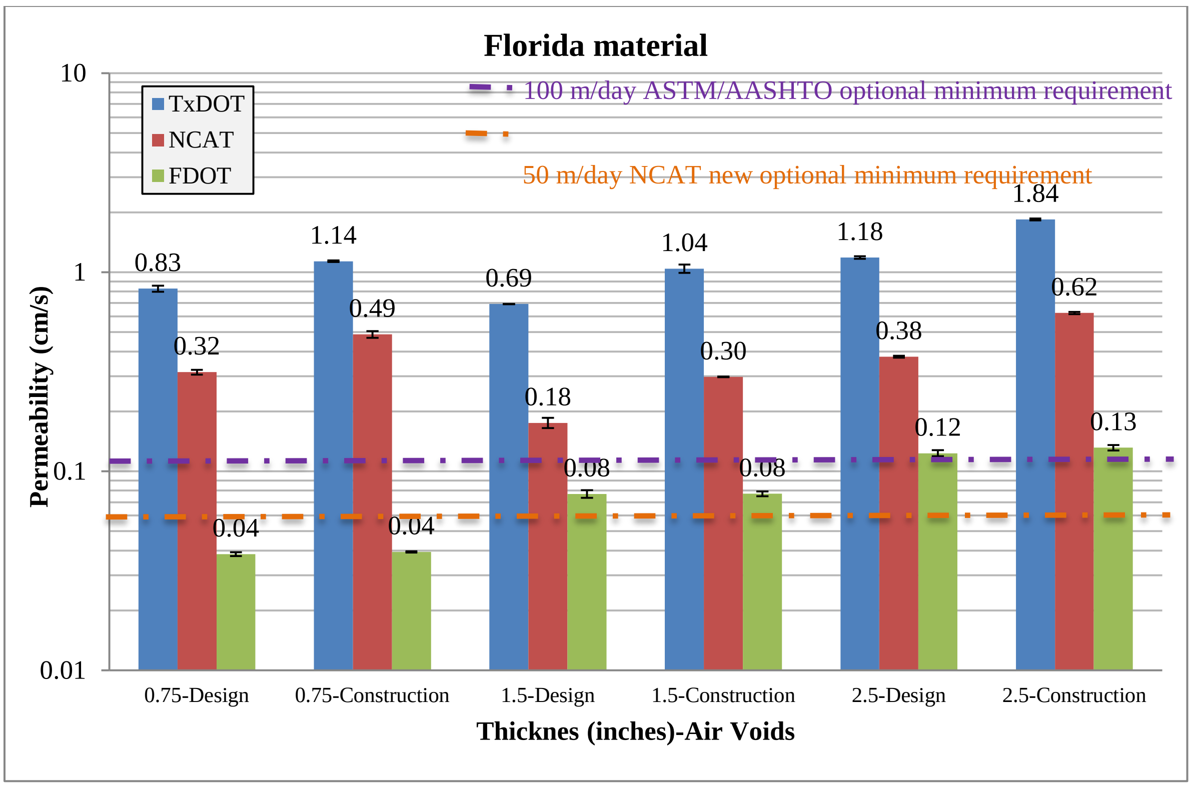


Figure 30. Type of permeameter vs permeability analysis for Florida material.

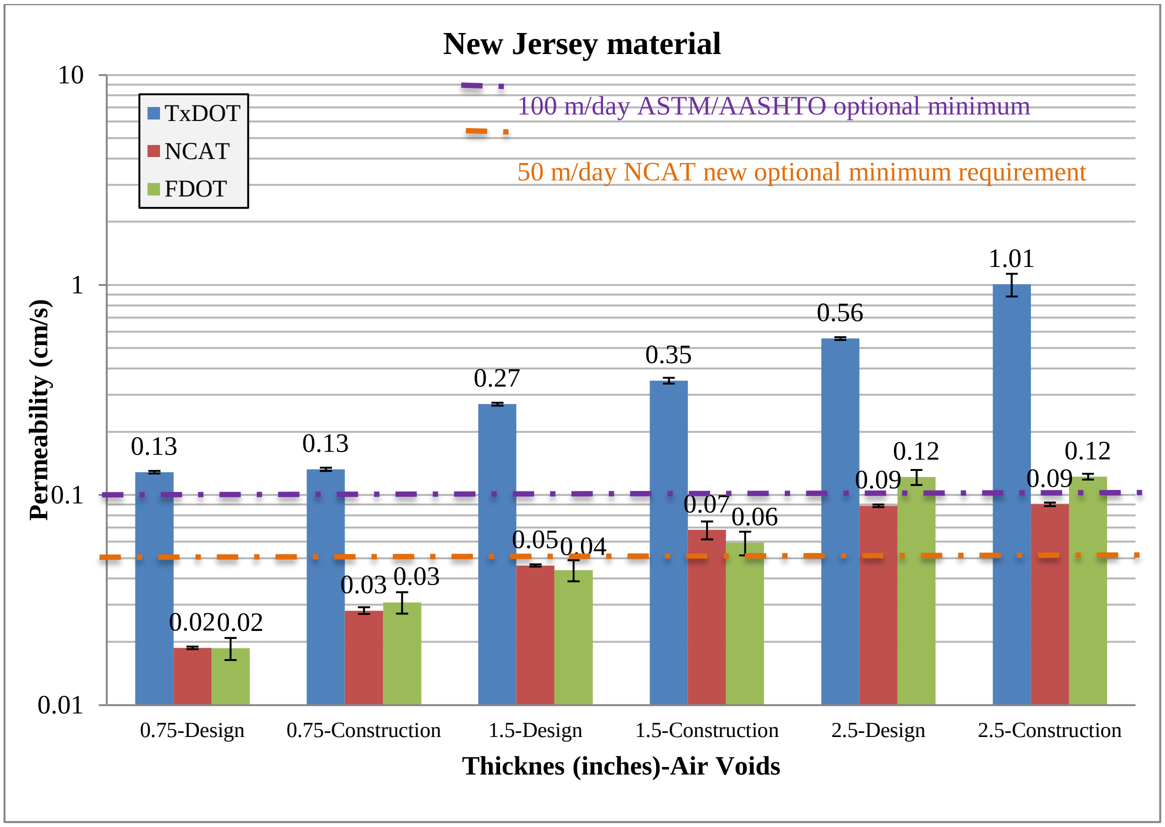


Figure 31. Type of permeameter vs permeability analysis for New Jersey material.

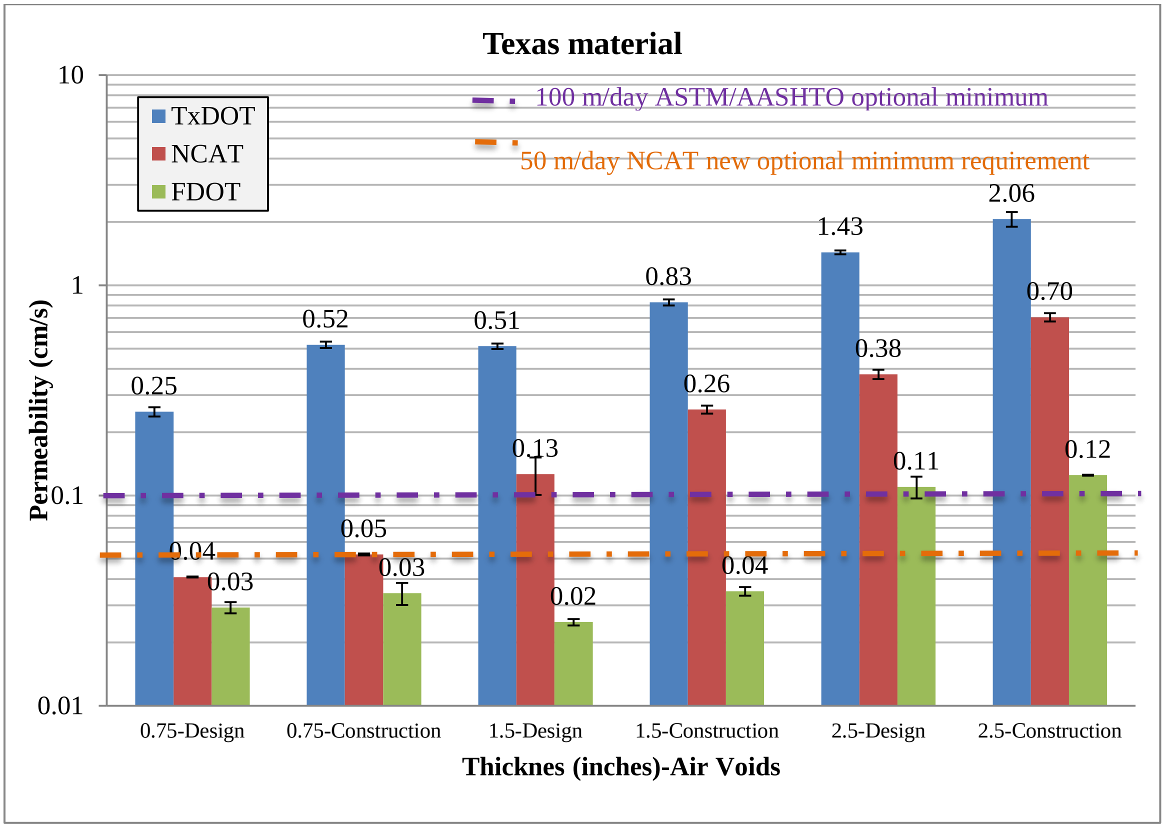


Figure 32. Type of permeameter vs permeability analysis for Texas material.

There is a wide range of values for coefficients of permeability of PGA reported in the literature, since this value depends on several factors including the mix design and the test method utilized. In general, the coefficients of permeability of PGA are reported to be higher than 0.030 cm/s (30 m/day) (Alvarez, Martin, & Estakhri, 2011; Hernandez-Saenz, Caro, Arambula, et al., 2016; Masad, Al-Omari, & Lytton, 2006). In this study, the type of permeameter had a statistically significant effect on the coefficient of permeability at $\alpha=0.05$. The complete statistical analysis is presented in APPENDIX I. The TxDOT permeameter lead to the highest values of permeability on the slabs regardless of thickness, AV or type of mixture. In general, the coefficient of permeability measured with the TxDOT permeameter is the highest, followed by the NCAT permeameter and then the FDOT permeameter.

In this study, the coefficient of permeability obtained with the TxDOT permeameter was on average 72% higher than that obtained with the NCAT permeameter, and 92% higher than that obtained with the FDOT apparatus. Additionally, the NCAT coefficient of permeability was 70% higher than the values obtained with the FDOT apparatus. The relationship between these three types of permeameters was relatively consistent; however, it is evident that the coefficients themselves were different from each other. The low permeabilities obtained with the FDOT permeameter could be attributed to a lower "effective diameter" for the specimens coated with Vaseline and surrounded by the membrane (i.e., not really 6-in diameter as input in the permeability equations).

The national standards for mix design of PGA in ASTM D7064 and AASHTO PP 77 include an optional minimum permeability requirement for PGA of 100 m/day but do not specify the type of permeameter that should be used to perform the measurement or the specimen dimensions (Hernandez-Saenz, Caro, Arambula, et al., 2016). The new mix methodology proposed by NCAT and described in Chapter 2 (see Table 26) proposes a new requirement of 50 m/day but again does not specify the test method either. The results presented in Figure 30, Figure 31 and Figure 32 indicate the importance of setting a threshold for each test since structures that pass the requirement with one apparatus fail with the other. For instance, with the Florida material the six structures meet the recommended 100 m/day and 50m/day threshold when the TxDOT and NCAT permeameters are used to obtain the coefficient of permeability, yet none of the structures achieved the threshold with the FDOT apparatus for 100m/day and only the thicker structures (i.e. 2.5 in and 1.5 in) fulfill the 50 m/day requirement. This observation is consistent with a previous study (D. E. Watson et al., 2004).

From this study, the 100 m/day requirement may be applied with the TXDOT apparatus; but with the NCAT and FDOT permeameters, the new NCAT requirement of 50 m/day is more reasonable and achievable while still guaranteeing adequate drainability of the pavement. In terms of the variability of the tests, the three permeameters have low coefficients of variation that are on average 3.2%, 3.9% and 6.4% for the TXDOT, NCAT and FDOT permeameters, respectively. There is not a definite answer on which type of permeameter is better, as all of them are functional and capture the sensitivity to different parameters such as AV and thickness. In order to make a selection, the main characteristics and differences in the methodologies with the three types of apparatus are summarized in Table 21.

Table 21. Differences between types of permeameters.

Characteristic	TXDOT	NCAT	FDOT
Hydraulic principle	Falling head	Falling head	Falling head
Geometry	Cylindrical (one cross sectional area)	Cylindrical (two different cross sectional areas for entrance and exit of water)	Cylindrical (one cross sectional area for entrance of the water, another cross sectional area for penetration the water in the specimen and another cross sectional area for exit of the water)
Material	PVC	Plastic	Plastic, metal and latex
Type of specimen	Flat surface/slab	Flat surface/slab	Cylindrical specimen/core
Sealing	Plumbers putty around the base	Plumbers putty in the base plus weights on top	Specimen sealed with Vaseline and pressurized to the sides with latex membrane
Type of measurement	Mainly field, in lab on slabs	Mainly field, in lab on slabs	Mainly lab, for field is necessary to take cores
Coefficient of Variability (from the experiments in this study)	3.2%	3.9%	6.4%

4.1.2 Thickness

Three different thicknesses were selected in order to analyze the impact of the wide range of thicknesses used for PGA layers worldwide on permeability. The three thicknesses selected were 0.75 in, 1.5 in and 2.5 in; and the permeability was analyzed with the TXDOT, NCAT and FDOT permeameters. Figure 33, Figure 34 and Figure 35 present the results of the permeability versus thickness for each of these types of permeameters, respectively.

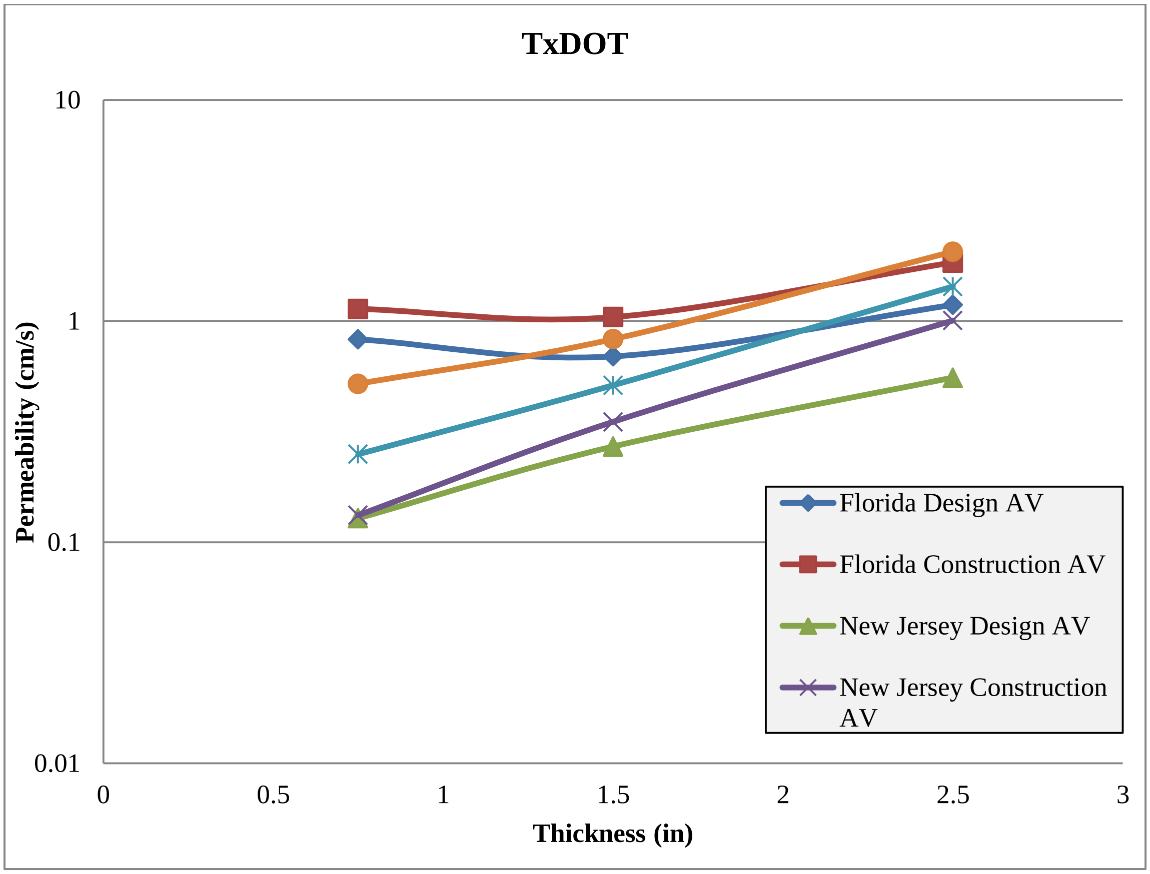


Figure 33. Thickness vs Permeability for TXDOT permeameter measurements.

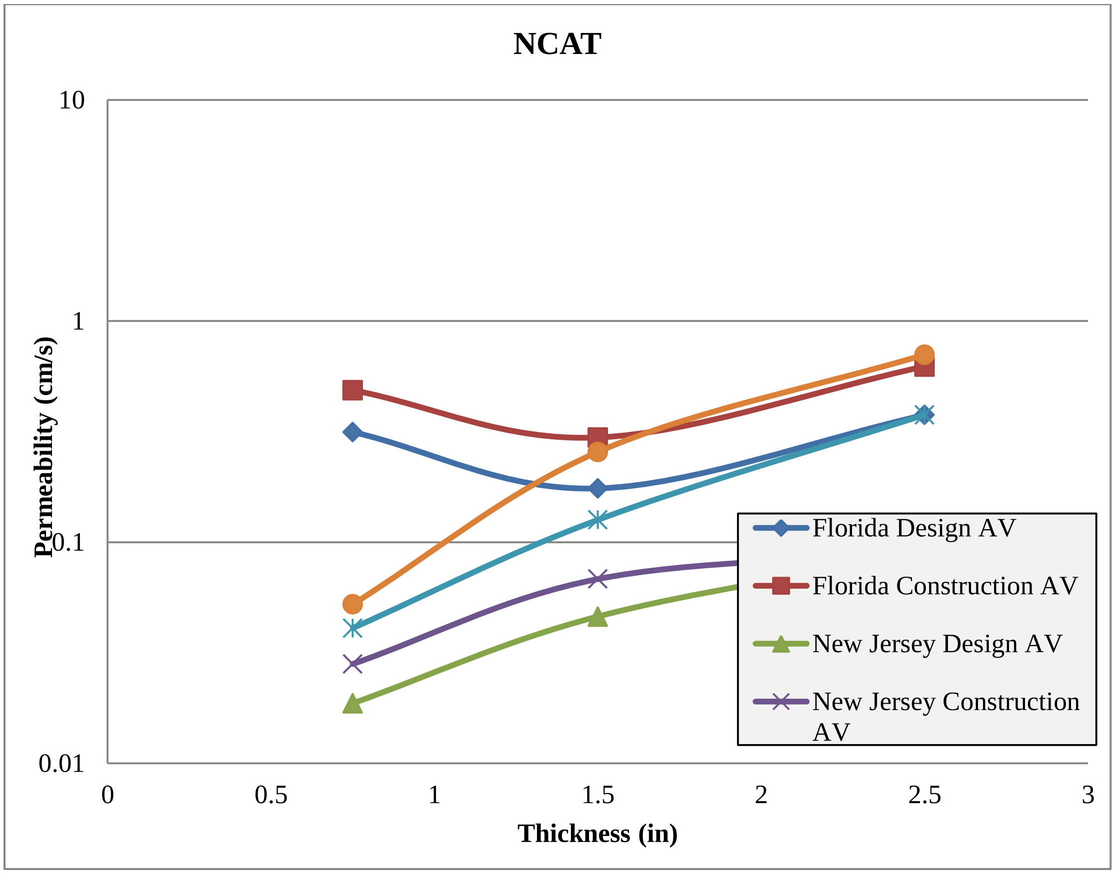


Figure 34. Thickness vs Permeability for NCAT permeameter measurements.

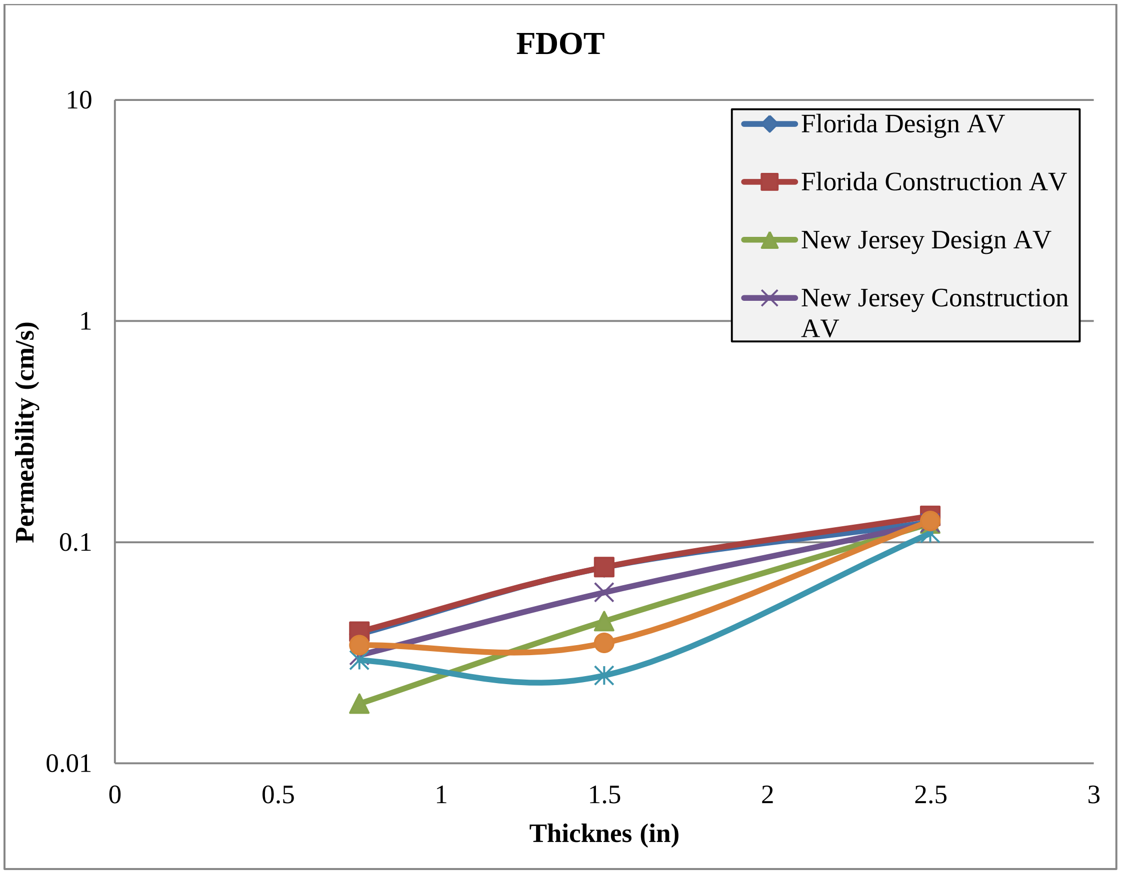


Figure 35. Thickness vs Permeability for FDOT permeameter measurements.

With respect to thickness, this parameter had a statistically significant effect on the coefficient of permeability at α=0.05. The complete statistical analysis is presented in APPENDIX I. The thickest structure (i.e. 2.5 in) had the highest coefficient of permeability as compared to the thinner structures for all three types of permeameters with every type of material. However, there was not as clear a relationship between the coefficient of permeability and thickness for the 1.5 in and 0.75 in structures. For the New Jersey and Texas material, the 1.5 in structures had higher permeability than the 0.75 in ones for all three permeameters. However, for the Florida material this behavior was only captured with the FDOT permeameter, while the TXDOT and NCAT permeameters suggest that the 0.75 in structure had higher permeabilities as compared with the 1.5 in structure. Yet, the differences for the TXDOT permeameter were not statistically significant. Hence, in general terms the thicker the PGA layers, the higher the permeability. It is important to underscore that the change between 0.75 in and 1.5 in is not as dramatic as the one between 1.5 in and 2.5 in. The changes are on average 34% and 46%, respectively. This leads to the conclusion that that even though the thickness of the slab could make an important difference in the permeability of PGA it is also important to account for the cost and the durability of the material and get an optimized design balancing these parameters.

4.1.3 Air Voids

It is known and has been reported that the AV found in PGA after construction are usually greater than the AV established in the initial design of these mixtures (Hernandez-Saenz, Arámbula-Mercado, & Epps Martin, 2017). Even though this difference is inherent to the construction of DGHMA pavements in general, with PGA this problem is exacerbated first because there are no strict QA/QC regulations for checking the AV of this type of mixtures, and second because this type of mixture has higher AV. In order to analyze the effect of this common AV change in the functionality of PGA, permeability measurements were conducted with three type of permeameters in 0.75 in, 1.5 in and 2.5 in structures at both construction and design AV levels. Figure 36, Figure 37 and Figure 38 present the results of the AV versus permeability analysis using the TXDOT, NCAT and FDOT permeameters, respectively. The graph presents the lines tied by each pair of points for all the structures; the equation of each line of the average of all measurements is also presented to have a general idea of the magnitude of the increment.

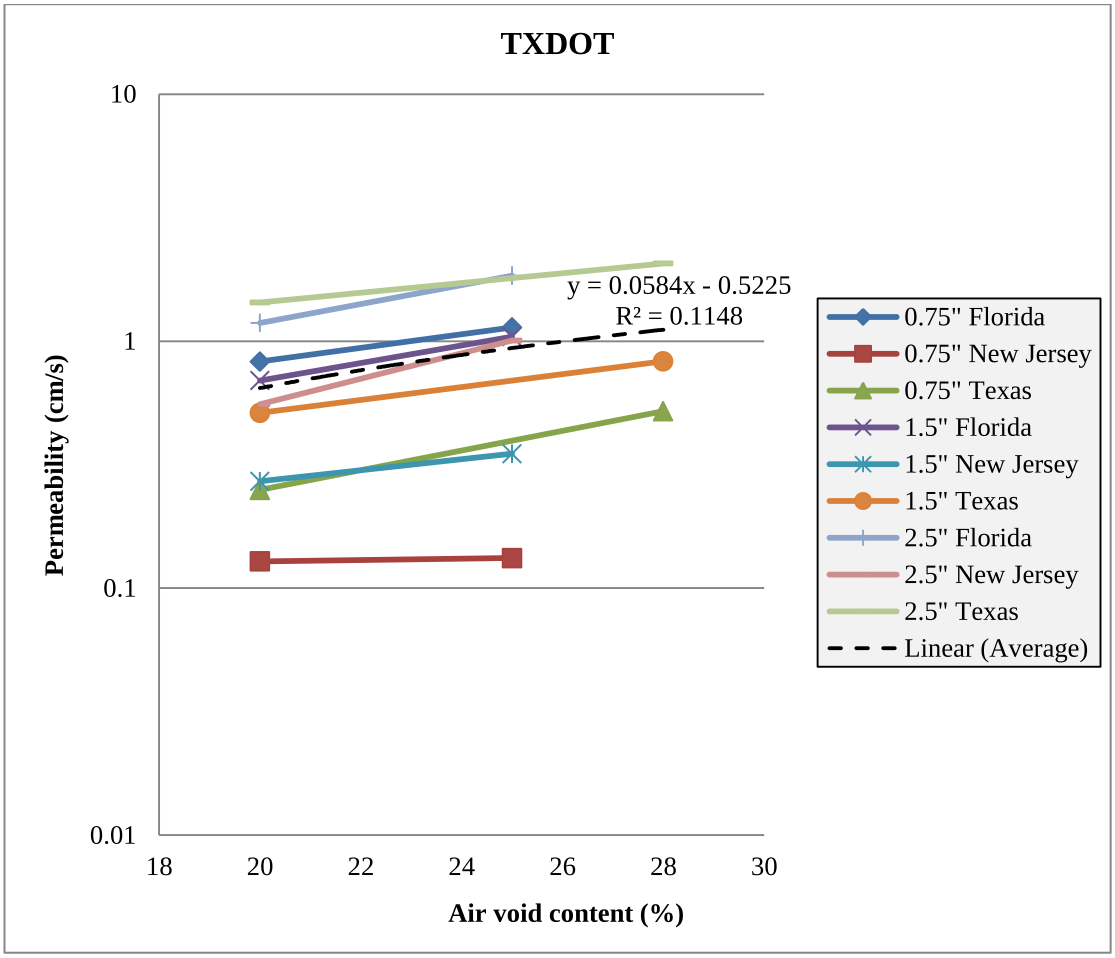


Figure 36. Permeability vs AV with TXDOT permeameter.

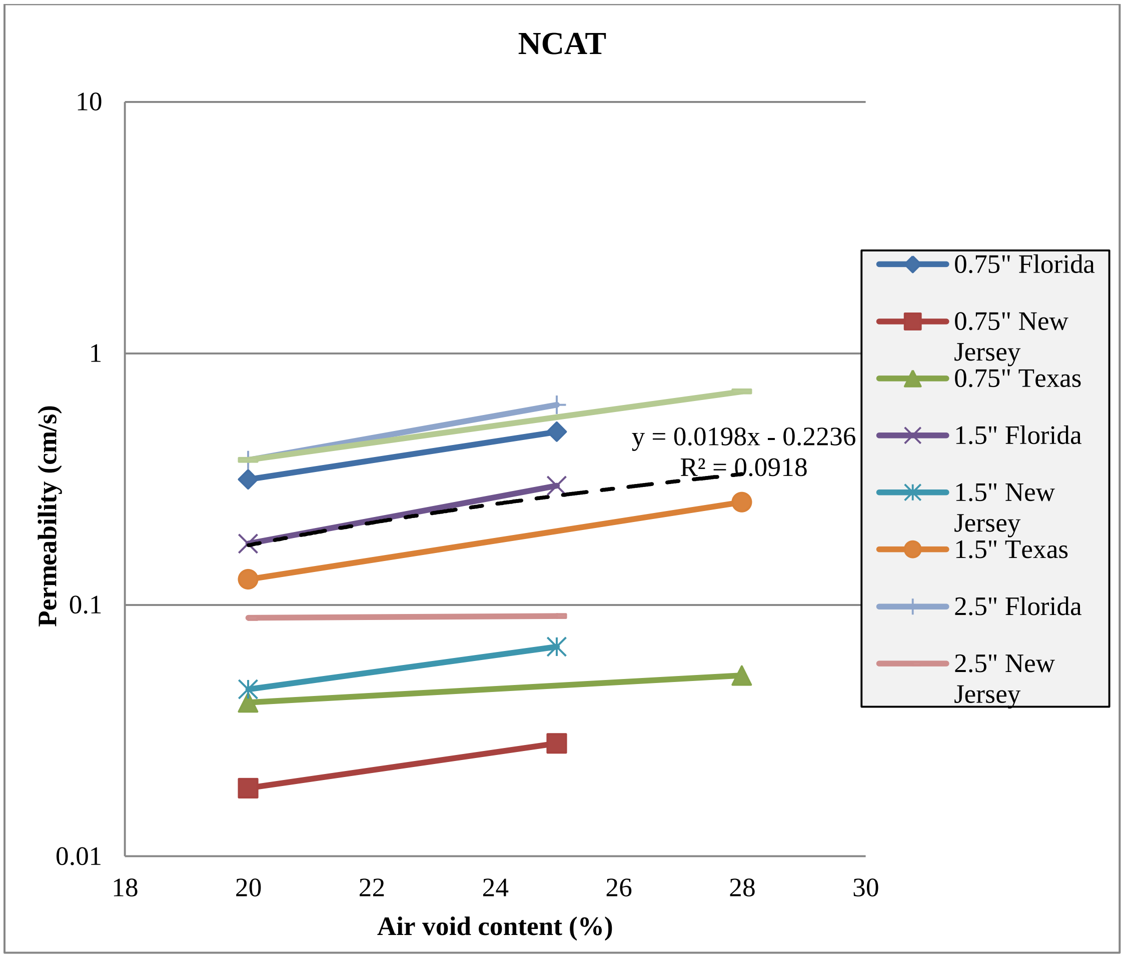


Figure 37. Permeability vs AV with NCAT permeameter.

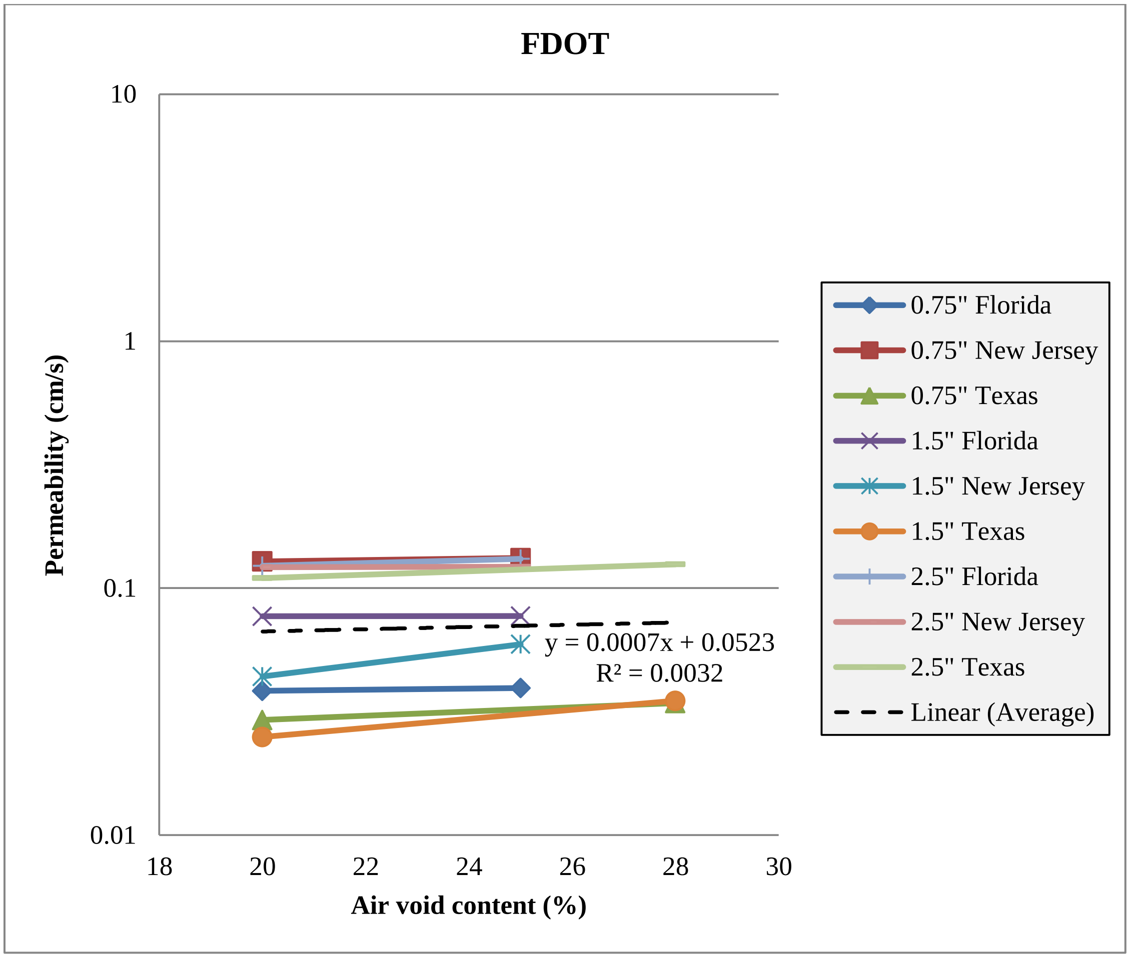


Figure 38. Permeability vs AV with FDOT permeameter.

From the data it was determined that the AV had a statistically significant effect on the coefficient of permeability at α=0.05. The complete statistical analysis is presented in APPENDIX I. Construction AV (i.e. larger AV) led to larger permeability values regardless of the type of permeameter, thickness and material. The measurements with the TXDOT permeameter captured an increase of 32% in permeability when augmenting the AV from design to construction, while this increase was 37% and 15% for the NCAT and FDOT permeameters, respectively. This could be an advantage for the functionality of PGA; however, the durability of the mixture is likely affected with high AV. The permeability test captured the changes in AV; hence, it would be desirable to not only have a minimum permeability threshold, but also a maximum. This type of provision could aid in preventing early durability issues caused by the increase in AV from design to construction and at the same time guarantee adequate functionality.

4.1.4 Type of Material

As explained in Chapter 3, four types of materials found in real PGA projects covering the four SHRP climatic zones (i.e. wet-freeze, wet-no freeze, dry freeze and dry no-freeze) were obtained and utilized in this study. The four materials utilized were from Utah, Florida, New Jersey and Texas. As mentioned previously, the material from Utah was not utilized for the functionality analysis since slabs could not be fabricated. The three materials utilized for this study have different binder contents, binder types and additives as described in Chapter 3. In order to analyze if the impact of different types of materials was captured by the permeability tests, Figure 39, Figure 40 and Figure 41 present the coefficients of permeability calculated for the three materials with the TXDOT, NCAT and FDOT permeameters, respectively.

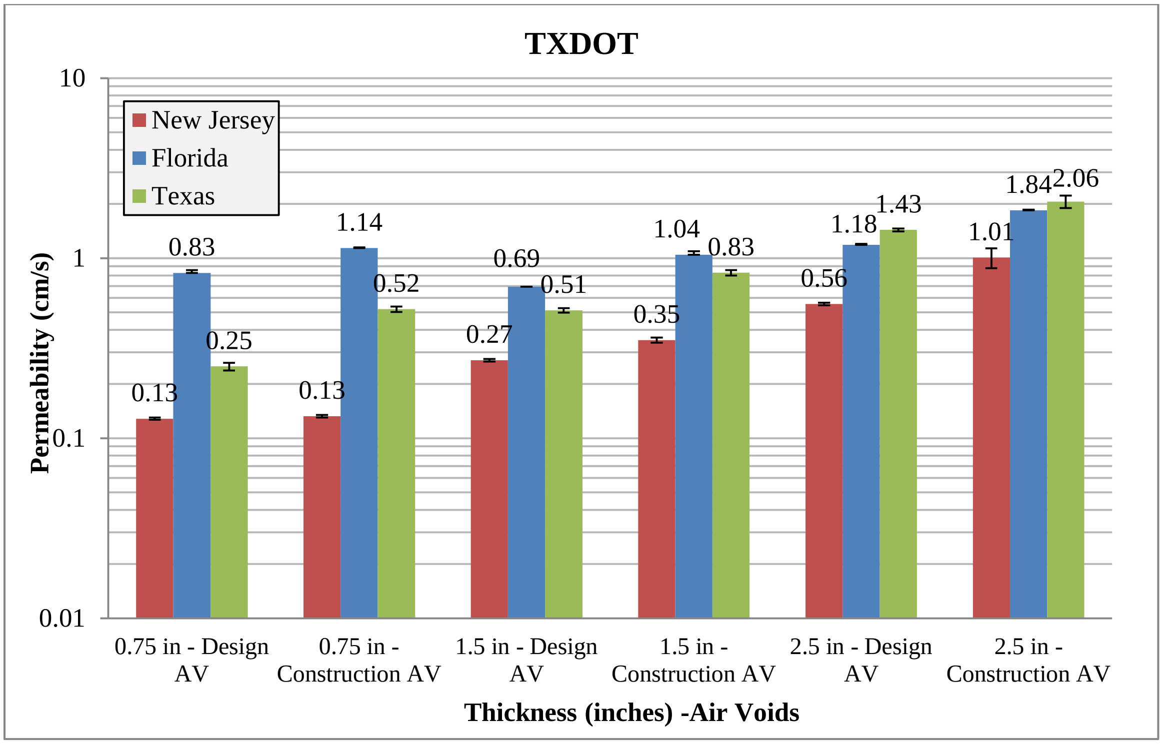


Figure 39. Impact of the type of material in permeability for TXDOT measurements.

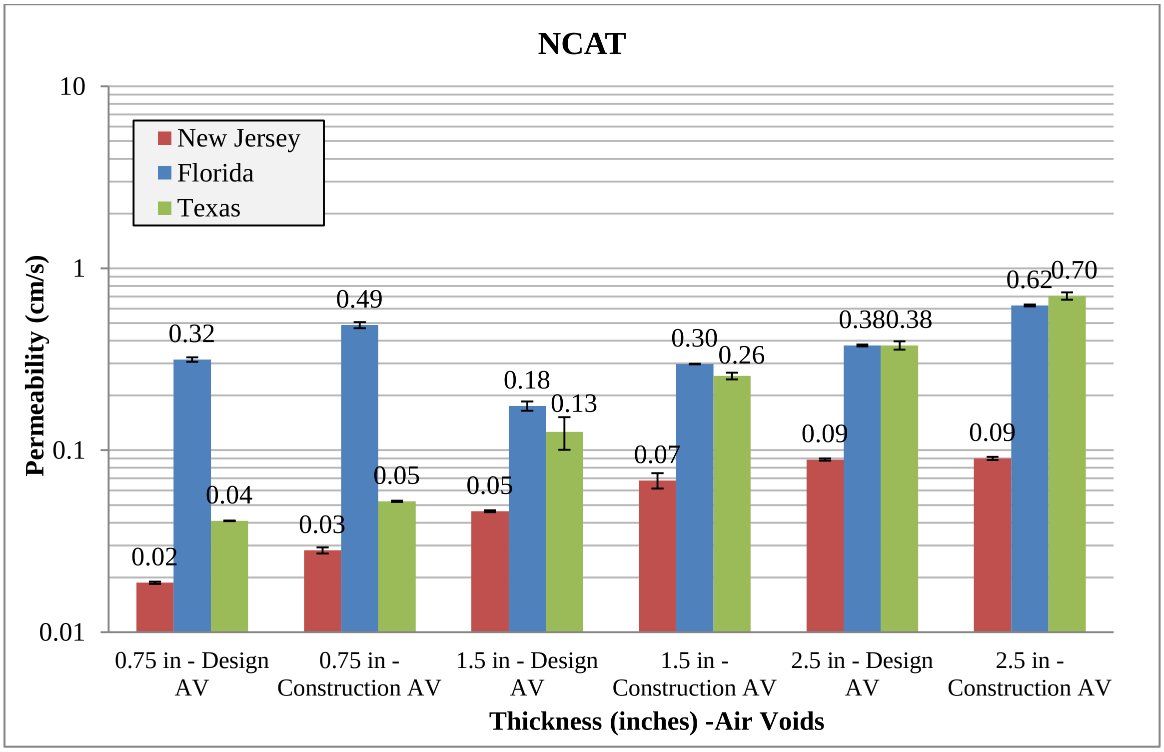


Figure 40. Impact of the type of material in permeability for NCAT measurements.

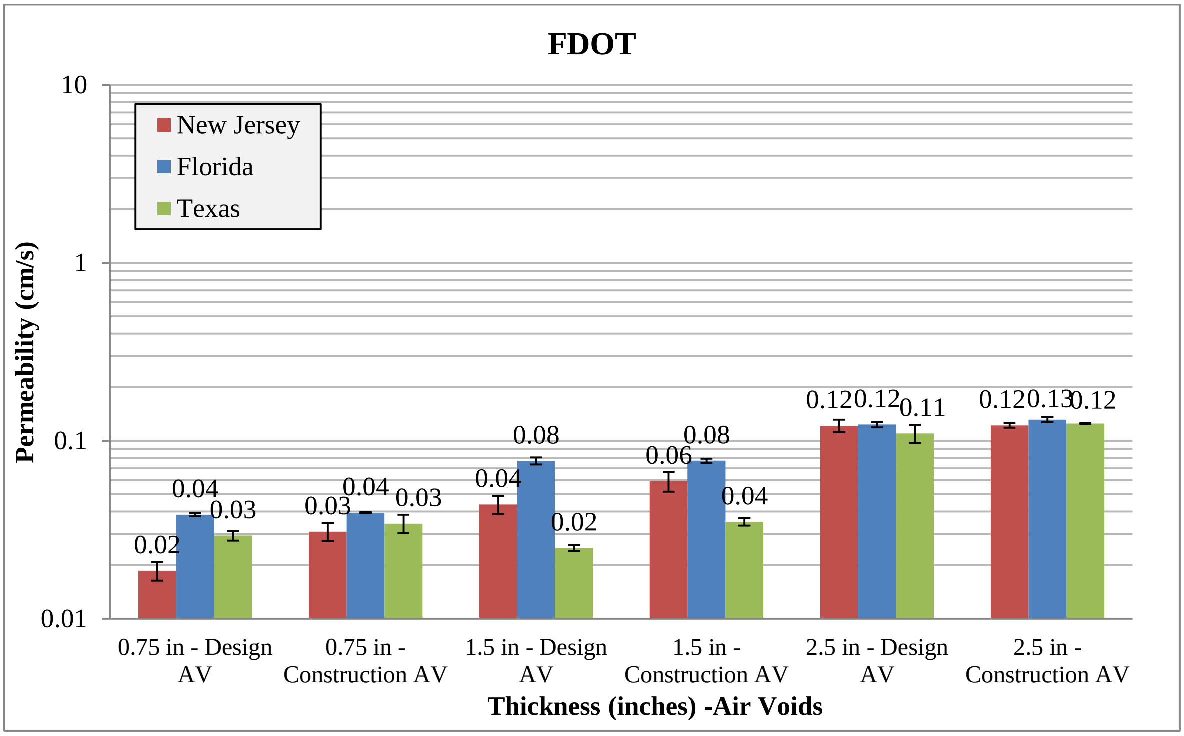


Figure 41. Impact of the type of material in permeability for FDOT measurements.

Based on the statistical analysis, the type of material had an effect on the permeability coefficient and was considered statistically significant at $\alpha=0.05$. The complete statistical analysis is presented in APPENDIX I. In general Florida structures had the highest permeability, followed by Texas and then New Jersey, regardless of the type of permeameter or AV. On average the Florida material has permeability coefficients 21% higher than Texas and 66% higher than New Jersey; also, Texas has permeability coefficients 58% higher than New Jersey. Since there are several factors that vary between mixtures (i.e. binder content, binder type, aggregate source and additives), the specific reason for the change of permeability could not be defined. However, it is noteworthy that the New Jersey mixture is the one that has the lowest permeability and is also the only one that contains asphalt rubber, which according to the literature reduces permeability (Lu & Harvey T, 2011; Shirini &

Imaninasab, 2016). Hence, the permeability test utilized captured a substantial effect of different types of materials on the functionality of PGA.

4.1.5 Laboratory vs. Field

Besides the laboratory measurements performed on the slabs, permeability field measurements were obtained at the construction sites. The TXDOT and NCAT permeameters were carried to the field site, and the permeability was measured directly on the road on which the PGA layer was placed. For the FDOT permeameter analysis, cores were extracted from the field and then taken to the laboratory for performing measurements. As explained previously, in the laboratory these measurements were performed on slabs and slab cores with three different thicknesses (i.e. 0.75 in, 1.5 in and 2.5 in) and two AV (i.e. design and construction) for each type of material. In order to compare the laboratory and field measurements, data from the field at the construction AV were tied to the closest thickness of the laboratory measurements at the construction AV. For instance, for the Florida material, the PGA layer in the field is 0.75 in, so the data from the 0.75 in slabs at construction AV was utilized. Figure 42 summarizes this field versus laboratory data.

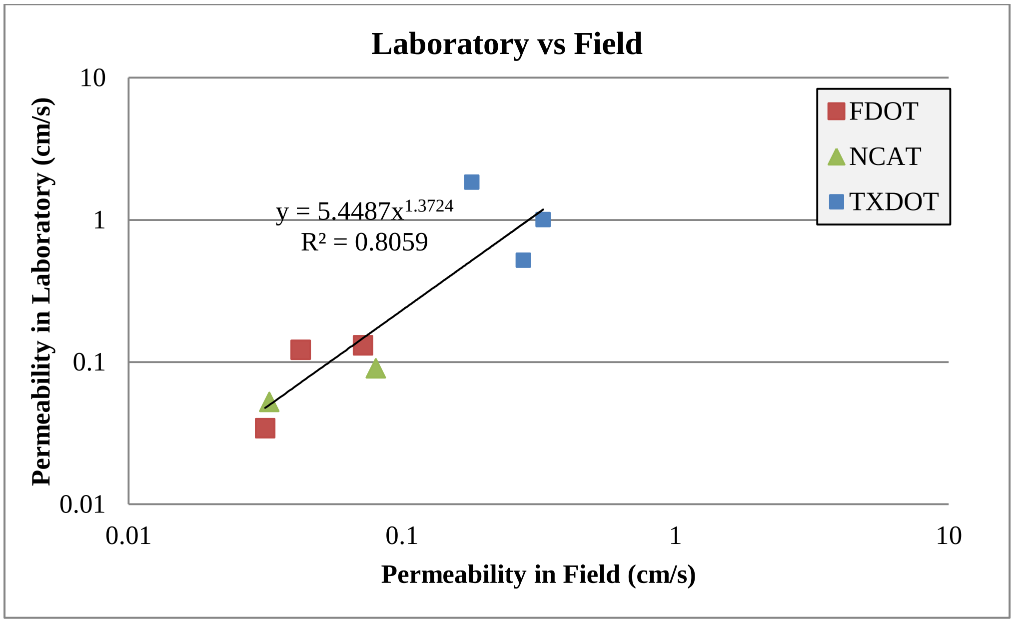


Figure 42. Laboratory vs field permeability.

In general the laboratory data had higher values of permeability regardless of the type of permeameter or material. On average laboratory measurements were 46% higher than field measurements. However, the size of this effect and its statistical significance depended on the type of permeameter and material. Specifically, the difference between laboratory and field permeability was largest when using the TXDOT permeameter; Also, the difference was statistically significant between TxDOT and NCAT permeameters, but not with FDOT. The complete statistical analysis is presented in APPENDIX J. The type of measurement (i.e. laboratory or field) had an impact on the coefficient of permeability at α=0.05; and the differences could be attributed to the difference in compaction of the specimens, and hence in the interconnected voids of the mixture which are crucial when analyzing permeability. Thus permeability requirements should also specify if the threshold is for laboratory or field measurements, because the type of measurement significantly changes the values.

4.1.6 Aging State

As explained in previously, permeability tests using the TxDOT and NCAT permeameters were also performed in the field at the time of construction and also approximately one year (i.e., 14 months) after construction. Field cores obtained at those same times were used in the laboratory to measure permeability with the FDOT apparatus. The two materials that have the data to the date are the ones from Florida and Utah, which were the two first projects constructed. The comparison between the field permeability for the three test methods at the initial time of construction and one year later is presented in Figure 43. The bars in the graph represent the average of the 24 measurements for the field, while the error bars represent +/- one standard deviation from the average value. No permeability measurements were performed in the field at construction for the Florida material with the NCAT permeameter because the equipment was not available at the time.

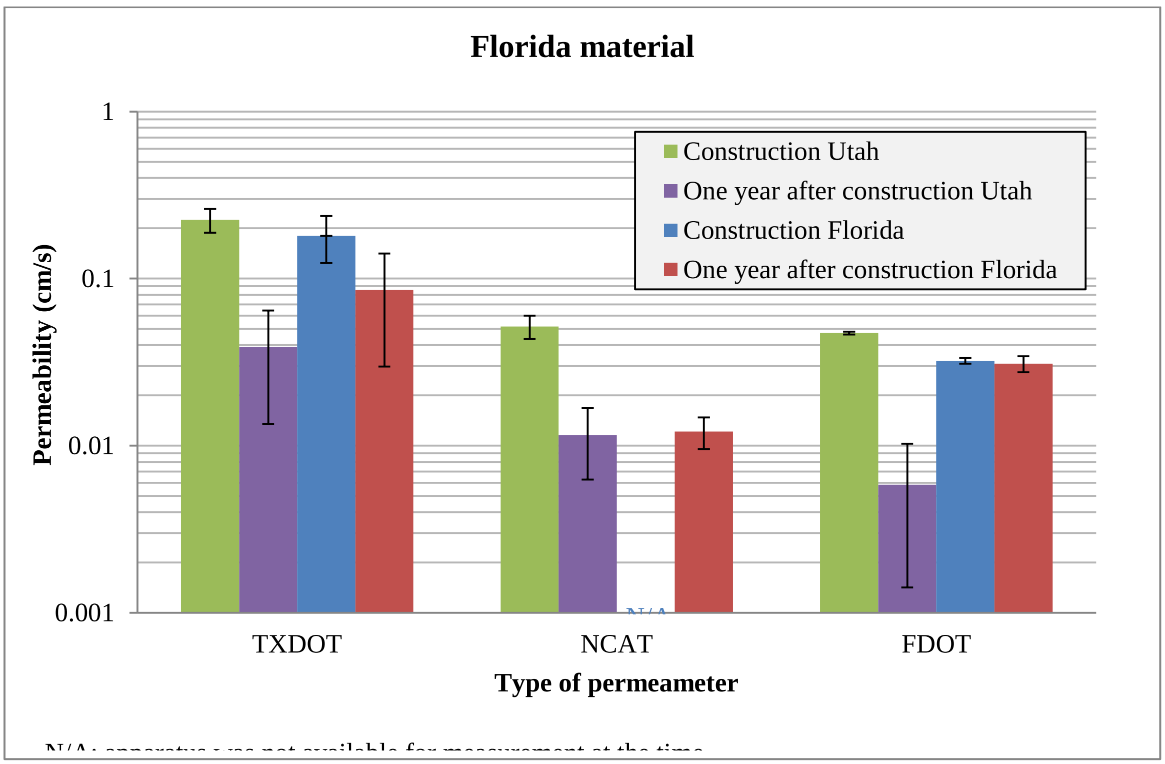


Figure 43. Field permeability of Utah and Florida material over time.

Permeability after one year of construction was smaller as compared to that at the construction time while the variability of them increases with time. However, the reduction in permeability depends on the type of permeameter and more importantly, the material. For both materials, there was a reduction of permeability that was statistically significant over time (full statistical analysis provided in APPENDIX K); however, the reductions were higher for the Utah material which had 83%, 78% and 88% less permeability with the TXDOT, NCAT and FDOT permeameters, respectively. For the Florida material there was a statistically significant reduction of 53% using the TXDOT permeameter; however, with the FDOT permeameter this change was not significant. The fact that permeability is reduced one year after construction could be attributed to the densification of the mixture by traffic or clogging of the AV structure in the mixture (Cooley Jr. et al., 2009). The fact that the permeability test could capture the effect of problems like clogging suggests the possibility of using this test for assessment.

4.2 Noise Absorption

The noise reduction capacity of the material was evaluated with the impedance tube test through the noise absorption coefficients which are evaluated in a range of frequencies from 400 Hz to 1,600 Hz. These coefficients can range from zero to one; where zero indicates the material reflects all the noise incident upon it, while one indicates the material absorbs all the generated noise (C. S. Y. Lee & Fleming, 1996). Previous studies have reported that for DGHMA, the noise absorption coefficient ranges between 0.1 and 0.2; while for PGA this number varies between 0.4 to 0.7 (Hanson, James, & Nesmith, 2004). Two common methodologies for analyzing the impedance tube results were found in the literature: 1) comparing qualitatively the frequency versus sound absorption curves (Crocker & Li, 2005) and 2) averaging all sound absorption coefficients at all frequencies (Lu & Harvey T, 2011). In order to assess the first analysis methodology, Figure 44, Figure 45 and Figure 46summarize the noise absorption coefficients for slabs with three thicknesses at both design and construction AV for the Florida, New Jersey and Texas materials for the range of frequencies tested.

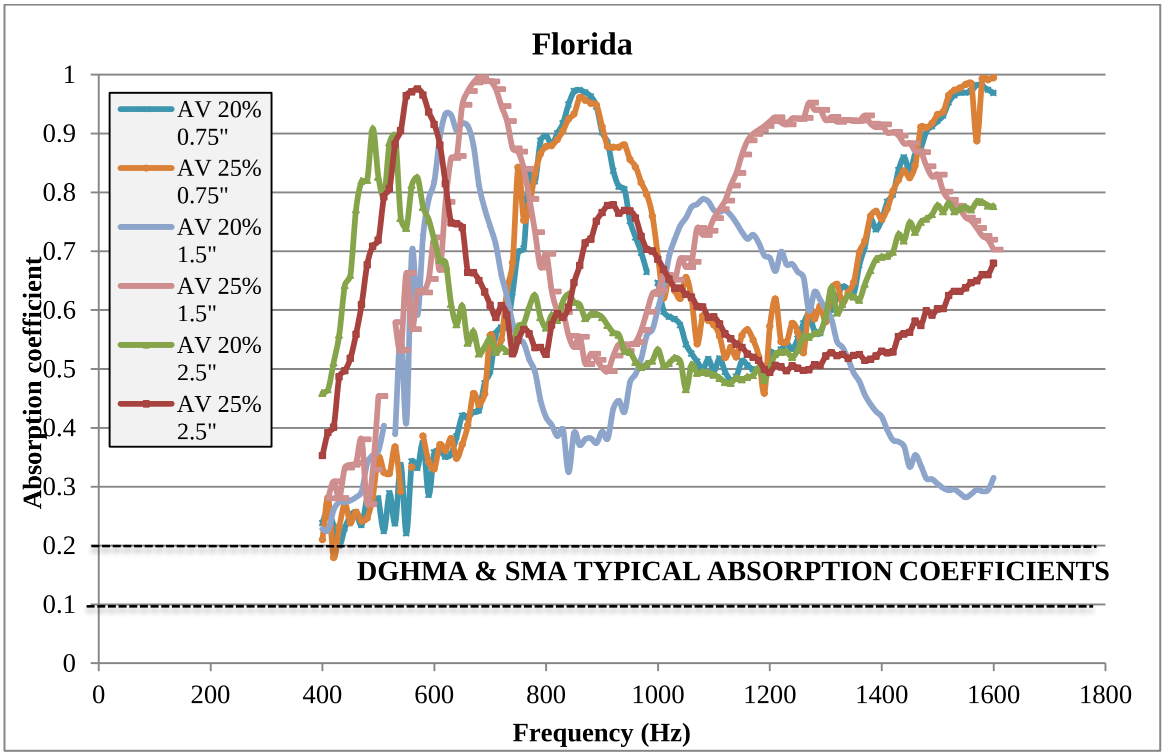


Figure 44. Noise absorption coefficient vs. frequency Florida material.

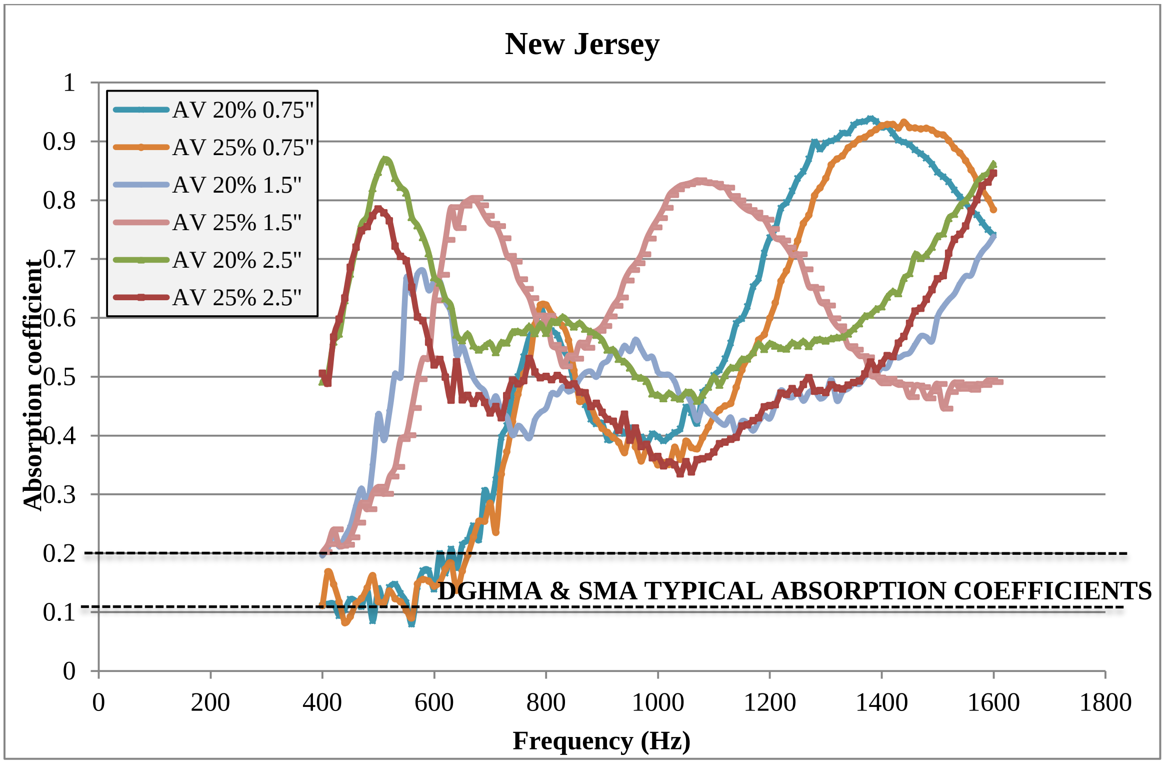


Figure 45. Noise absorption coefficient vs. frequency New Jersey material.

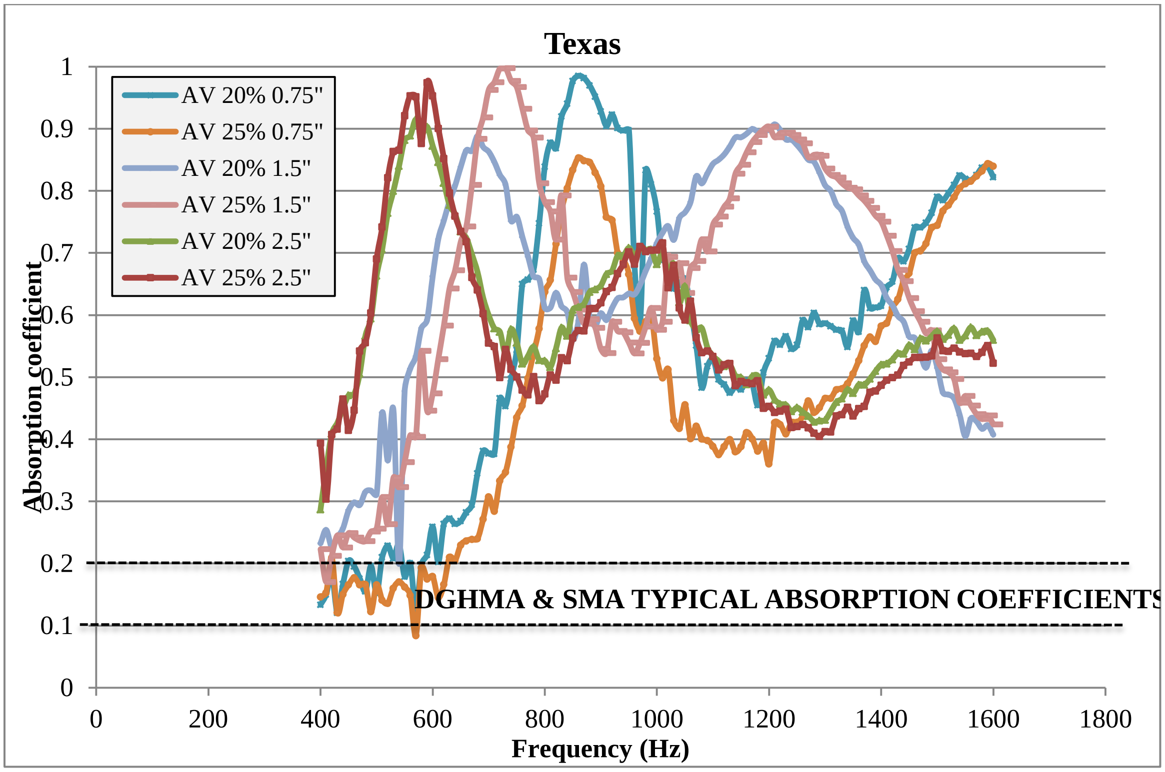


Figure 46. Noise absorption coefficient vs. frequency Texas material.

The average noise absorption coefficient for all slabs ranged between 0.45 and 0.65, which is within the typical range reported in the literature for PGA mixtures and represents enhanced noise reduction characteristics as compared to DGHMA. When comparing the results for each thickness, it is apparent that the noise absorption coefficient peaks (i.e., highest value) moved to higher frequencies as the thickness of the layer diminished, which is consistent with previous studies (Crocker & Li, 2005; Shen, Wu, & Du, 2009). Also, the thicker layers (i.e., 1.5 in and 2.5 in) were more significantly affected by changes in AV as shown by the upward shift of the curves with 25% AV. In order to further analyze the effect of thickness and AV on the noise absorption coefficients, the second methodology of analysis (i.e. average over the frequencies) was utilized as discussed subsequently.

4.2.1 Thickness

In order to evaluate specifically the effect of thickness on the noise absorption coefficient, Figure 47 presents the average noise absorption coefficient over all frequencies versus the thickness for the three materials analyzed at both design and construction AV.

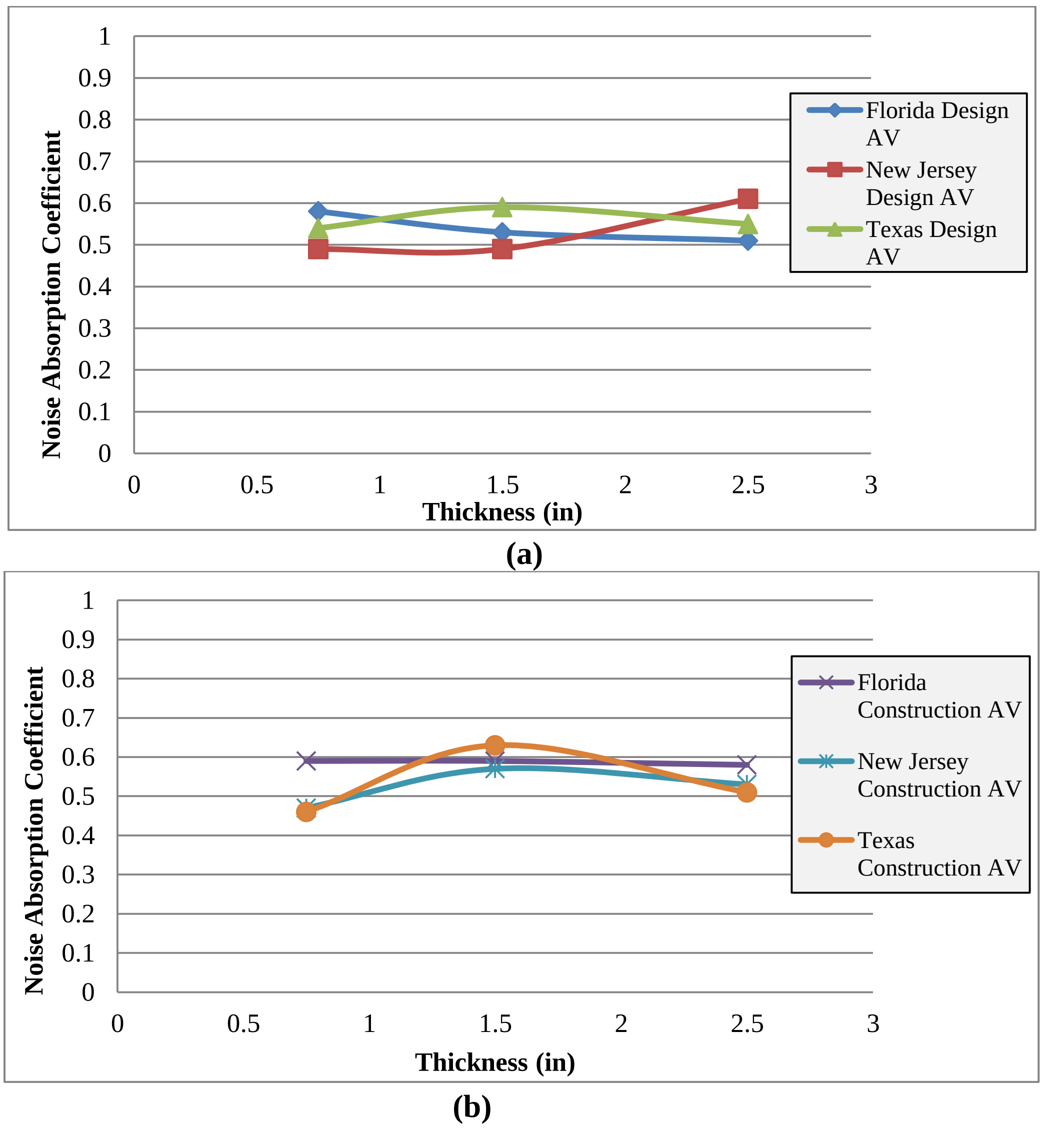


Figure 47. Noise absorption coefficient vs thickness for (a) design AV and (b) construction AV.

Based on a statistical analysis of the data (APPENDIX L), thickness did not have a statistically significant effect on noise reduction for the design AV; while for the construction AV, the 1.5 in structure had the highest noise absorption; however, this difference was at most 17%. In terms of materials for Florida and New Jersey, thickness did not have a statistically significant effect on the noise absorption; while for Texas, the noise absorption for the 1.5 in thickness was significantly higher than that for the 0.75 in structure. In general, there is no clear trend when comparing the average noise absorption coefficient values with respect to the thickness of the slab and most of the differences were not statistically different. Thus, the noise absorption coefficient is not particularly sensitive to changes in thickness.

4.2.2 Air Voids

In order to analyze the impact of AV on the noise absorption coefficient, the impedance tube was utilized for slabs of three different thicknesses (i.e. 0.75 in, 1.5 in and 2.5 in) each at design and construction AV. Figure 48, Figure 49 and Figure 50 summarize the data for the Florida, New Jersey and Texas materials, respectively.

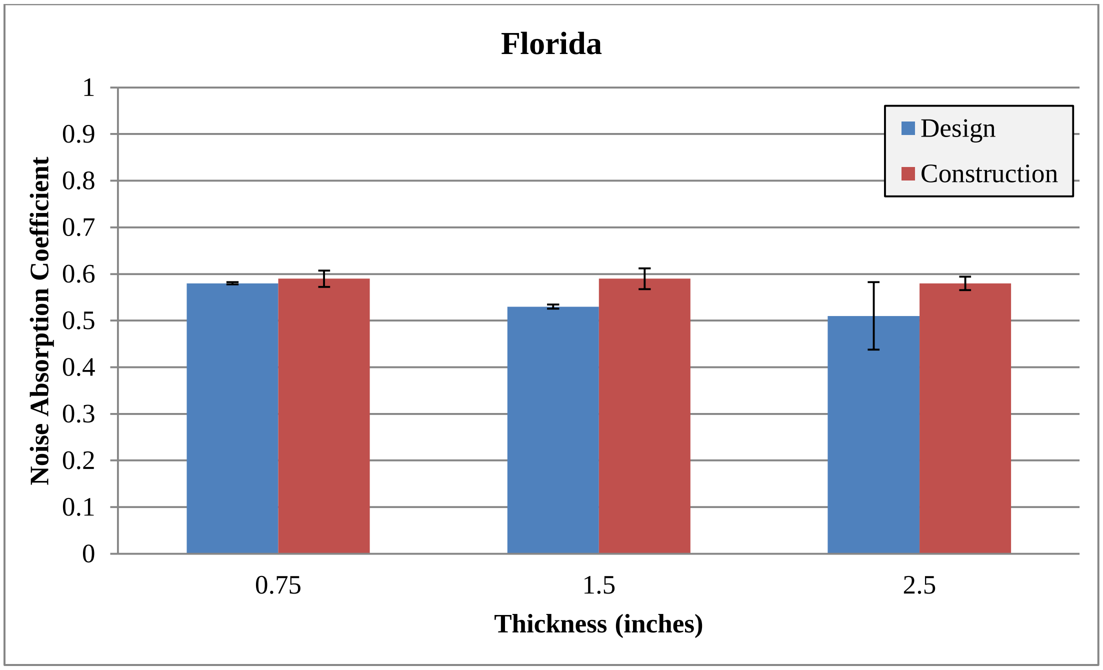


Figure 48. AV impact on noise absorption coefficient for Florida material.

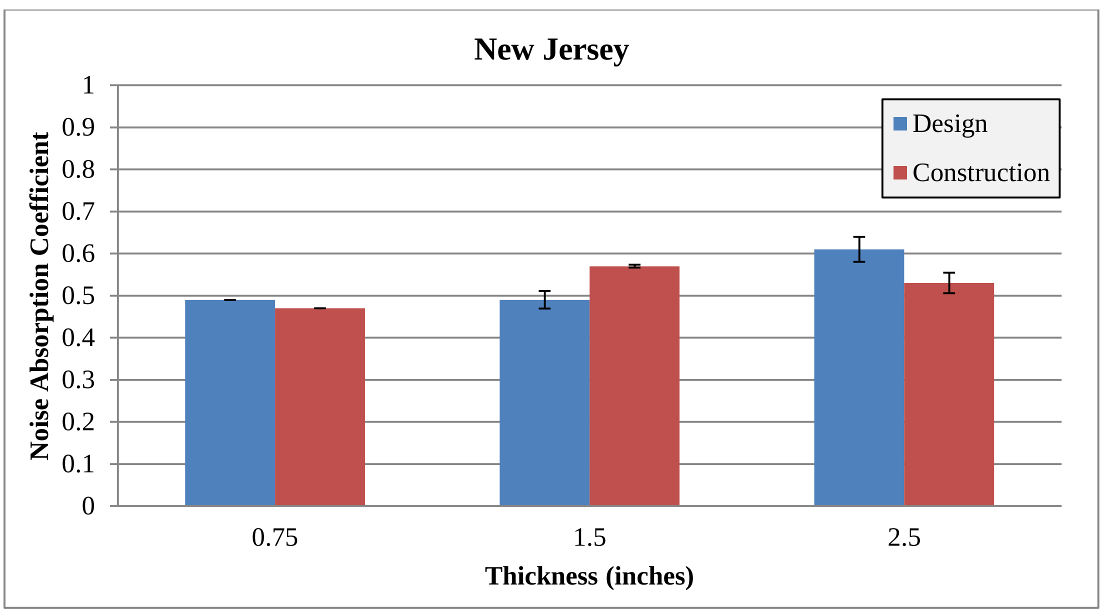


Figure 49. AV impact on noise absorption coefficient for Florida material.

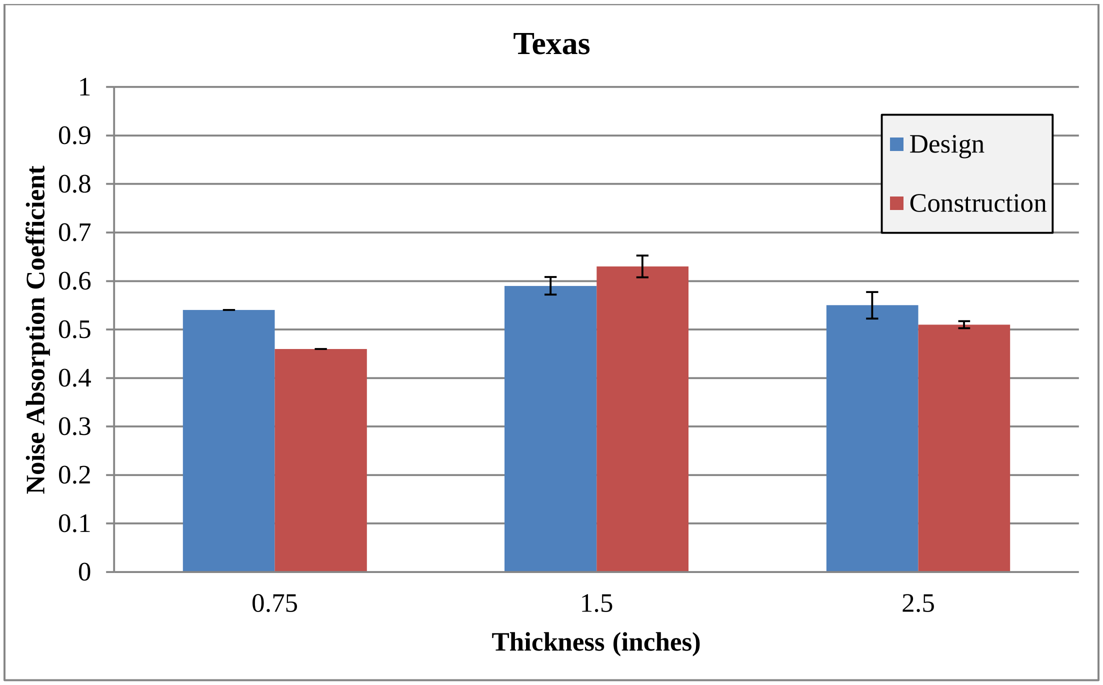


Figure 50. AV impact on noise absorption coefficient for Florida material.

A statistical analysis was performed and the results are presented in APPENDIX . When the thickness was 1.5 in, the construction AV lead to higher noise absorption coefficients compared to the design AV; however, for other thicknesses the effect of AV was opposite although the difference was not statistically significant. Also, construction AV lead to a statistically significant higher value of noise absorption coefficient compared to design AV with the Florida material. For materials from Texas and New Jersey, however, the effect of AV was opposite although the difference was not statistically significant. Hence, it was determined that for the noise reduction there was not a clear and statistically significant relationship with AV. However, the noise absorption coefficients for all of the thicknesses evaluated was within the typical range for PGA mixtures, which represents enhanced noise reduction characteristics as compared to DGHMA, corroborating what has been reported previously in the literature.

4.3 Conclusions of Functionality Analysis

The functionality of PGA with three different pavement layer thicknesses at two AV was evaluated via noise absorption and permeability using plant mixture collected from field projects in Florida, New Jersey and Texas. Permeability was evaluated with portable permeameters developed by TxDOT and NCAT, as well as with an FDOT laboratory apparatus. The permeameters were used on laboratory-compacted slabs and in the field soon after the field project was constructed and after one year in-service. The FDOT apparatus was used on field cores acquired soon after construction and one year after construction and also on cores extracted from the laboratory-compacted slabs. Noise absorption was evaluated using the impedance tube on the laboratory-compacted slabs. The following points summarize the main findings related to functionality:

- The coefficient of permeability obtained with the TxDOT permeameter was on average 72% higher than that obtained with the NCAT permeameter, and 92% higher than that obtained with the FDOT apparatus. Additionally, the NCAT coefficient of permeability was 70% higher than the values obtained with the FDOT apparatus. The relationship between these three permeability methods was relatively consistent; however, the development of permeability requirements for each type of equipment needs further evaluation.
- Thickness of the PGA layer had a statistically significant effect on the coefficients of permeability. In general, the thicker the PGA layers, the higher the permeability. The change between 0.75 in and 1.5 in was less dramatic as that between 1.5 in and 2.5 in. Some life cycle cost analysis could be performed to analyze the benefit cost of changing the thickness of the layer.
- The AV had a statistically significant effect on the coefficient of permeability at $\alpha=0.05$. This could be an advantage for the functionality of PGA; however, the durability of the mixture is likely affected with high AV. Both minimum and maximum permeability thresholds are recommended. This could aid in preventing early durability issues caused

by the increase in AV from design to construction and at the same time guaranteeing adequate functionality.

- In general Florida structures had the highest permeability value, followed by those in Texas and then New Jersey, regardless of the type of permeameter or AV.
- Laboratory data exhibited higher values of permeability regardless of the type of permeameter or material as compared to field data. On average, laboratory measurements were 46% higher than field measurements.
- Permeability field measurements decreased significantly one year after construction, while the variability of the measurements increased. This could be an indication of densification of the mixture under the influence of traffic or the clogging of the interconnected AV structure.
- The average noise absorption coefficients varied for all structures between 0.45 and 0.65 and did not change significantly with the thickness of the PGA slabs or the AV. All structures analyzed had enhanced noise reduction characteristics as compared to DGHMA.
- Since both functionality and durability of PGA mixtures should be considered to achieve balanced performance, the coefficient of permeability is recommended as the primary indicator of functionality. Data that best captured the influences of thickness, AV and type of material was provided by the TXDOT apparatus, so these data were utilized in subsequent analyses.

5. DURABILITY DATA AND RESULTS ANALYSIS

This chapter presents the results and discussion related to the durability assessment of the PGA mixtures described in Chapter 3. The Cantabro loss test with different conditioning protocols was performed for both SGC and slab cores specimens at both design and construction AV, while the HWTT, SCB and IDT/TSR were run on PMPC specimens obtained from field cores. Four materials (Florida, New Jersey, Texas and Utah) were characterized for the durability analysis; but as for the functionality evaluation, the Utah material was not utilized with slabs since they could not be fabricated.

5.1 Effect of Type of Compaction on AV

For the durability analysis, both SGC specimens and slab cores were used. Since the compaction methods for these methods are different (SGC vs. linear kneading compactor), the variance in AV of the final specimens was quantified before analyzing the durability data. Figure 51 and Figure 52 present the AV of the specimens obtained from each type of compaction for the design and construction AV, respectively. The bars represent the average of several measurements, while the black line represents ± one standard deviation from the average.

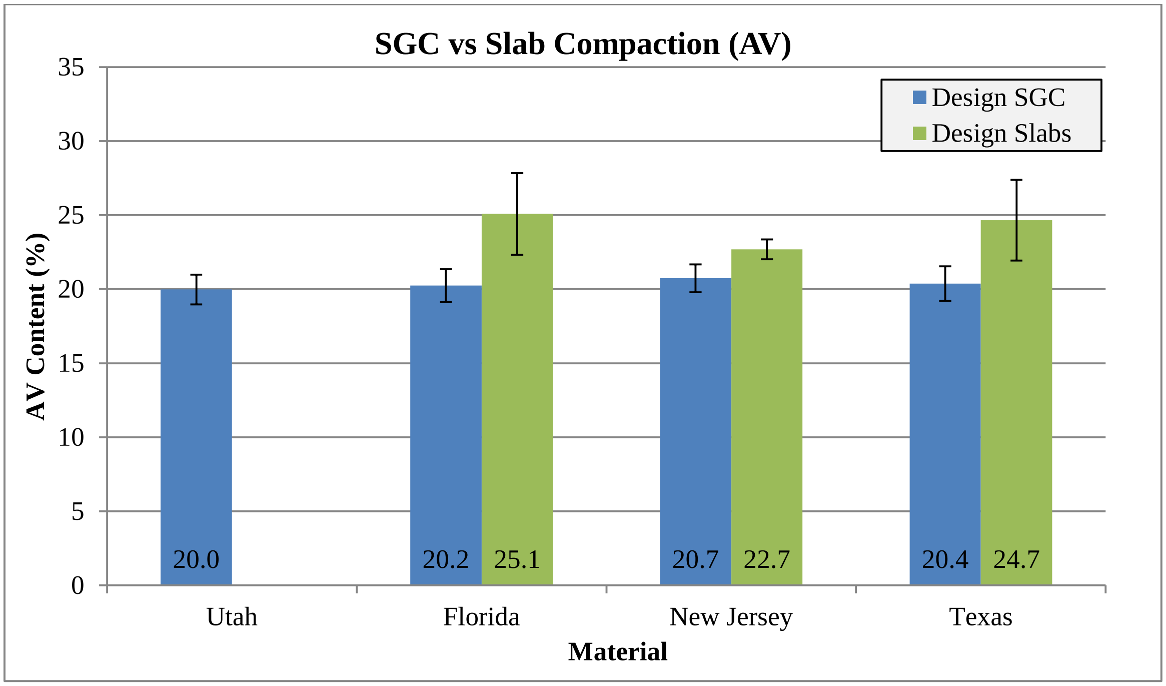


Figure 51. AV vs type of compaction (Design AV).

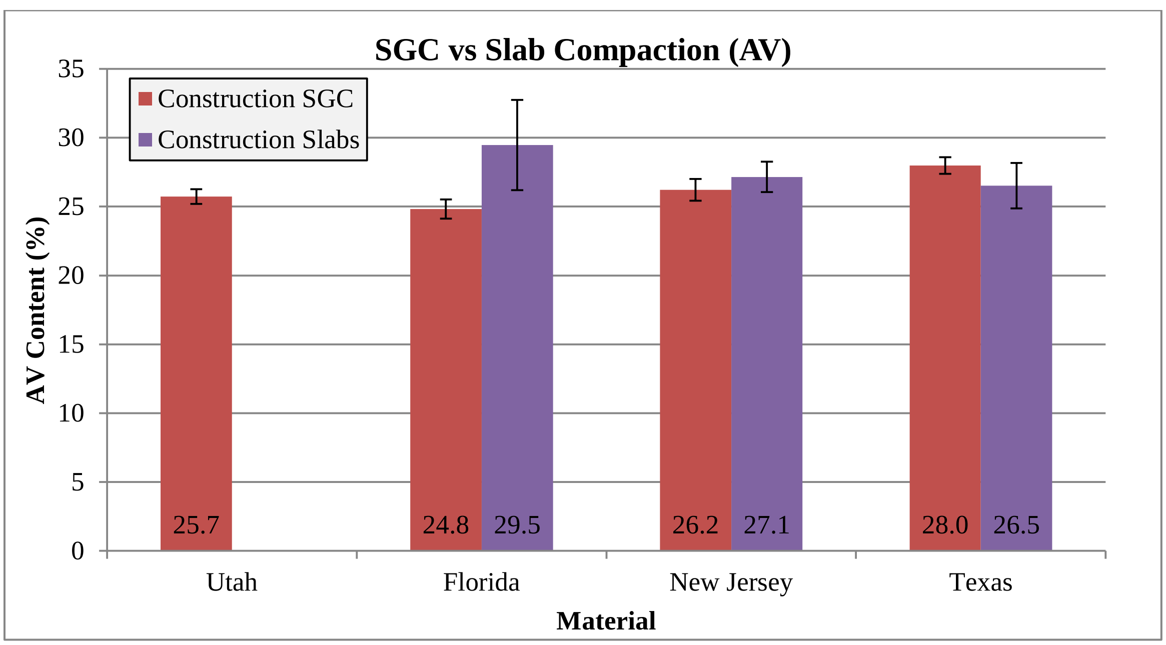


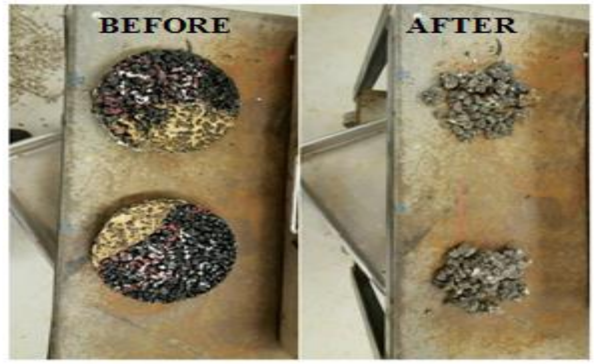
Figure 52. AV vs type of compaction (Construction AV).

A complete statistical analysis was performed as described in APPENDIX M, and it was identified that the type of compaction did have a statistically significant effect on the AV of the final specimens at $\alpha=0.05$. In general, the AV in the slab cores were statistically greater than those for SGC specimens. In addition, the variability of these measurements between the samples is higher for the slab cores as compared to those for the SGC specimens. This suggests that the SGC produces specimens with more consistent total AV and distribution of the interconnected voids in each specimen as compared to specimens cut from laboratory slabs. The slab cores, however, better simulate the field because they replicate the actual thicknesses of the PGA layers and because they have a similar compaction method to full-scale pavement construction. In the field, the construction AV are usually higher than the design AV and there is variability between the AV in the center and at the edges of the pavement. Both of these phenomena are also reflected with the slab compaction method.

5.2 Cantabro Test Analysis

The Cantabro loss test is one of the most utilized procedures to assess the durability of PGA mixtures worldwide (Hernandez-Saenz, Caro, Arambula, et al., 2016). This test consists of subjecting cylindrical specimens to 300 revolutions in the Los Angeles abrasion machine to measure the breakdown and durability of the mixture. The test plan for the Cantabro test included both SGC specimens which are 4.5 in height and 6 in diameter and slab cores with three different heights (0.75 in, 1.5 in and 2.5 in) and two AV (i.e. design and construction). The specimens were submitted to different conditioning protocols including: unaged dry, aged, unaged dry-freeze, unaged MIST, unaged wet-freeze, unaged wet-hot and aged wet-freeze. The thinner 0.75 inch extracted cores were not able to withstand the Cantabro loss test, so two other abrasion experiments listed in Table 22 were attempted for this type of specimens without success. Hence, the Cantabro analysis of the slab cores was performed on only the 1.5 and 2.5 in specimens.

Table 22. Abrasion Tests Performed on 0.75 inch Slab Cores.

Test	Results
• Cantabro loss Test. 	• Specimens were destroyed. 
• Ball Mill Method (TX 116E). 	• Specimens were broken, but the abrasion phenomenon was not replicated. 
• Rotating-Cutter Method (ASTM C944). 	• The abrasion of the aggregate but not the mixture was replicated. 

The analysis performed with the Cantabro results determined the impact of AV, type of conditioning protocol, thickness and type of material on the abrasion properties as indicated by Cantabro loss percentage. The following sections analyze each of the parameters individually for both SGC specimens and slab cores.

5.2.1 Air Voids

The volumetric properties, especially the AV of PGA play a very important role defining both the durability and functionality of the mixture. This section aims to quantify the effect of the design AV and the actual construction AV on the Cantabro loss properties.

5.2.1.1 SGC Specimens

Figure 53, Figure 54, Figure 55 and Figure 56 illustrate the difference between the Cantabro loss percentage at construction AV and design AV for each type of conditioning protocol for the Florida, New Jersey, Texas and Utah materials, respectively. The complete statistical analysis is presented in APPENDIX N.

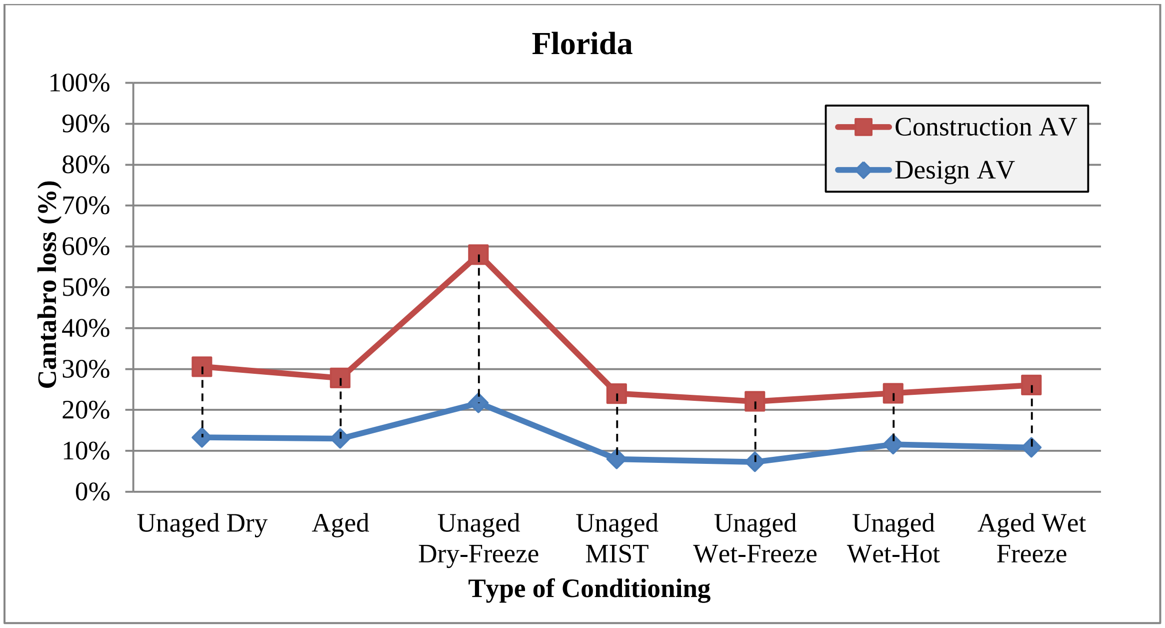


Figure 53. Effect of AV on Cantabro loss Florida material SGC specimens.

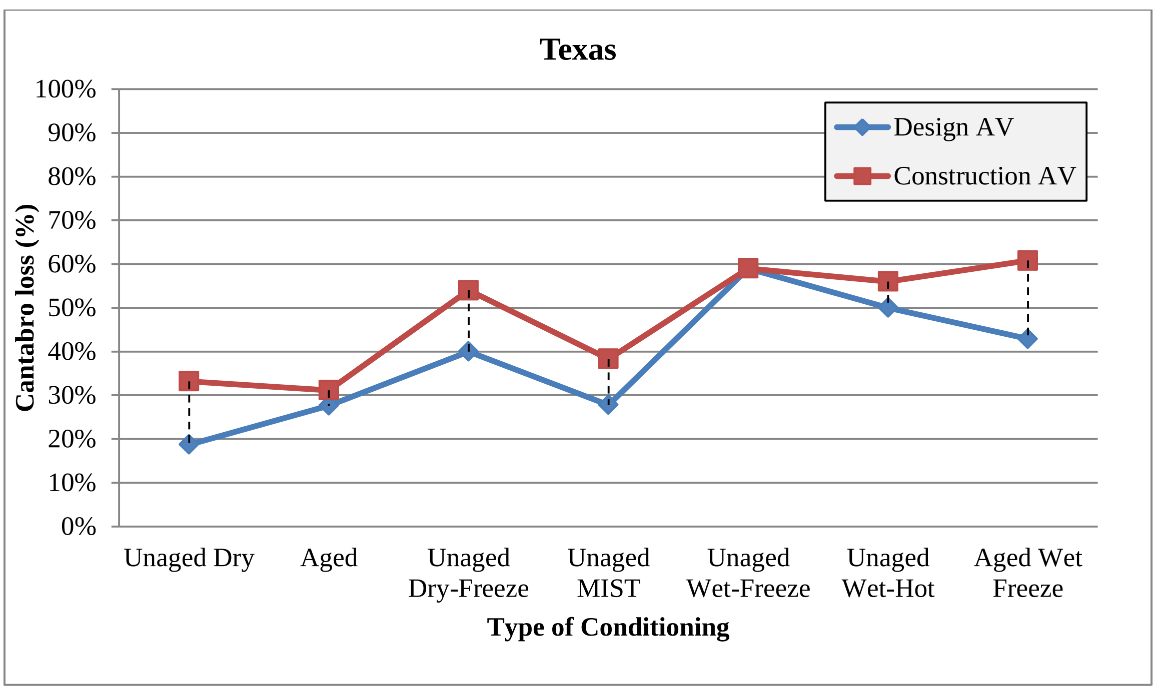


Figure 54. Effect of AV on Cantabro loss Texas material SGC specimens.

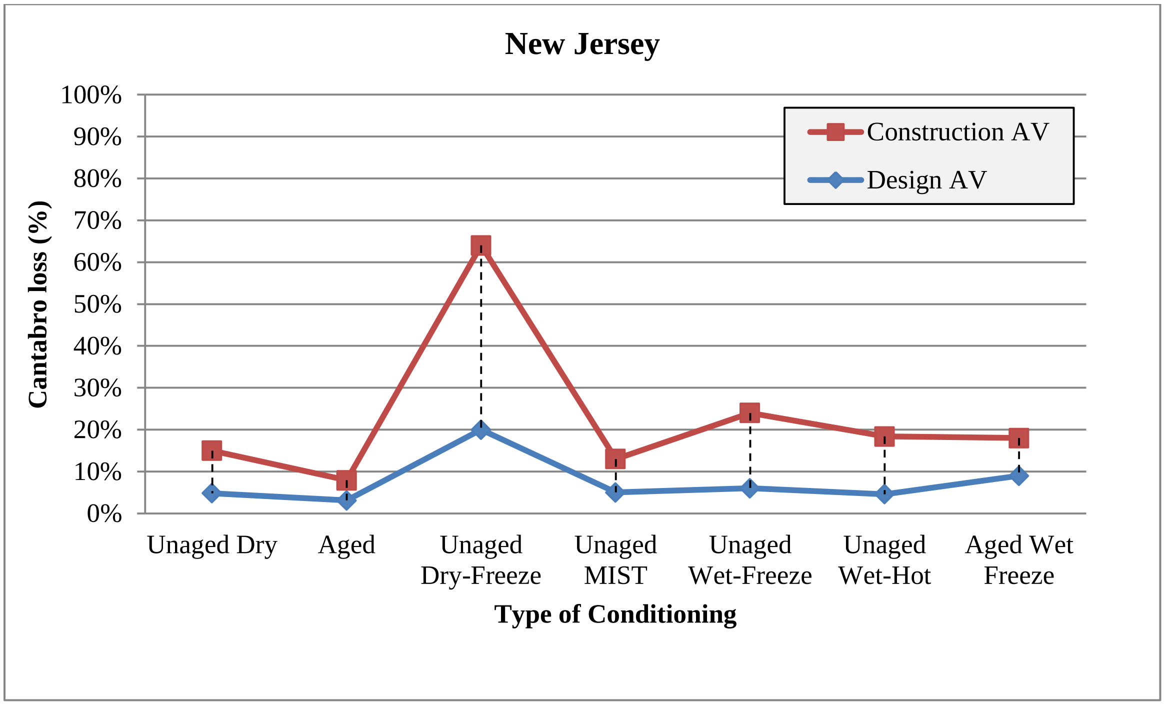


Figure 55. Effect of AV on Cantabro loss New Jersey material SGC specimens.

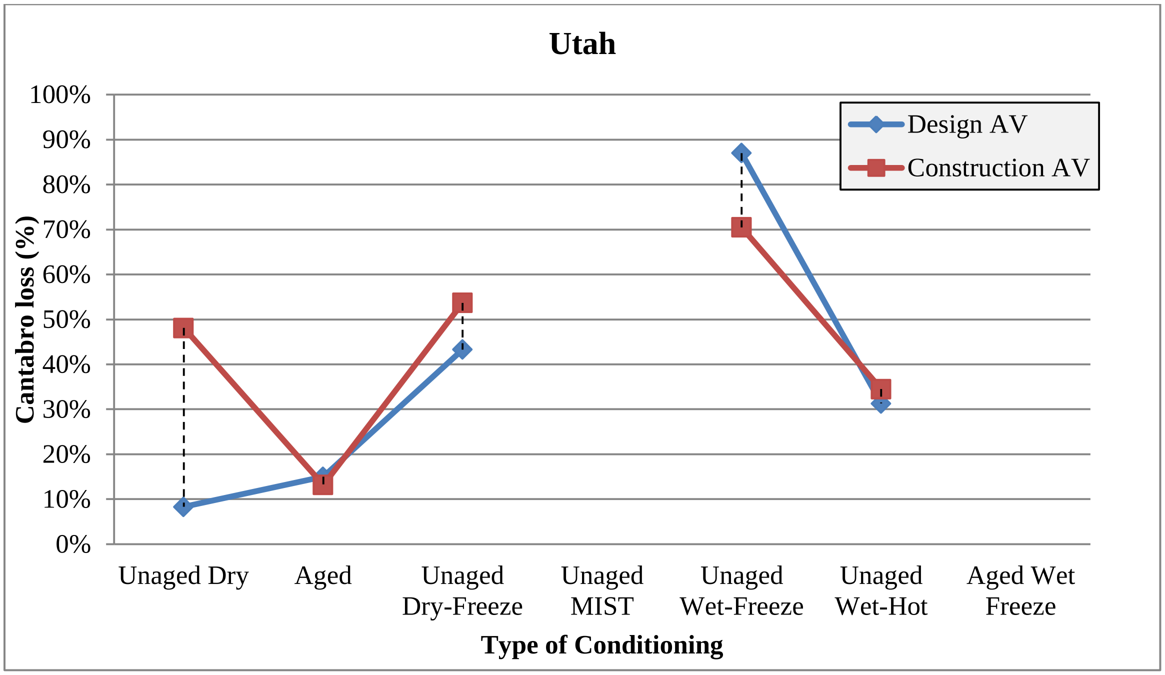


Figure 56. Effect of AV on Cantabro loss Utah material SGC specimens.

Based on the statistical analysis, the AV had a statistically significant effect on the Cantabro loss at α=0.05. The analysis also suggested that higher construction AV led to significantly higher Cantabro loss than design AV. For the Florida material, the Cantabro loss at construction AV was on average 30% higher than that obtained with the design AV specimens. The difference for the New Jersey and Texas materials was 45% and 21%, respectively. The Utah material was the only one in which some of the design AV measurements were higher than the construction AV; however, overall the difference was still 15% greater for construction AV higher as compared to design AV. Thus, these results indicate that even though increasing the AV could increase the permeability of the mixture, also presents a detriment for the functionality.

5.2.1.2 Slab Cores

Figure 57 and Figure 58 illustrate the difference between the Cantabro loss of design AV and construction AV specimens using the Florida material for the 1.5 in and 2.5 in cores, respectively. Figure 59 and Figure 60 present the same analysis for the New Jersey material, while Figure 61 and Figure 62 correspond to the Texas results.

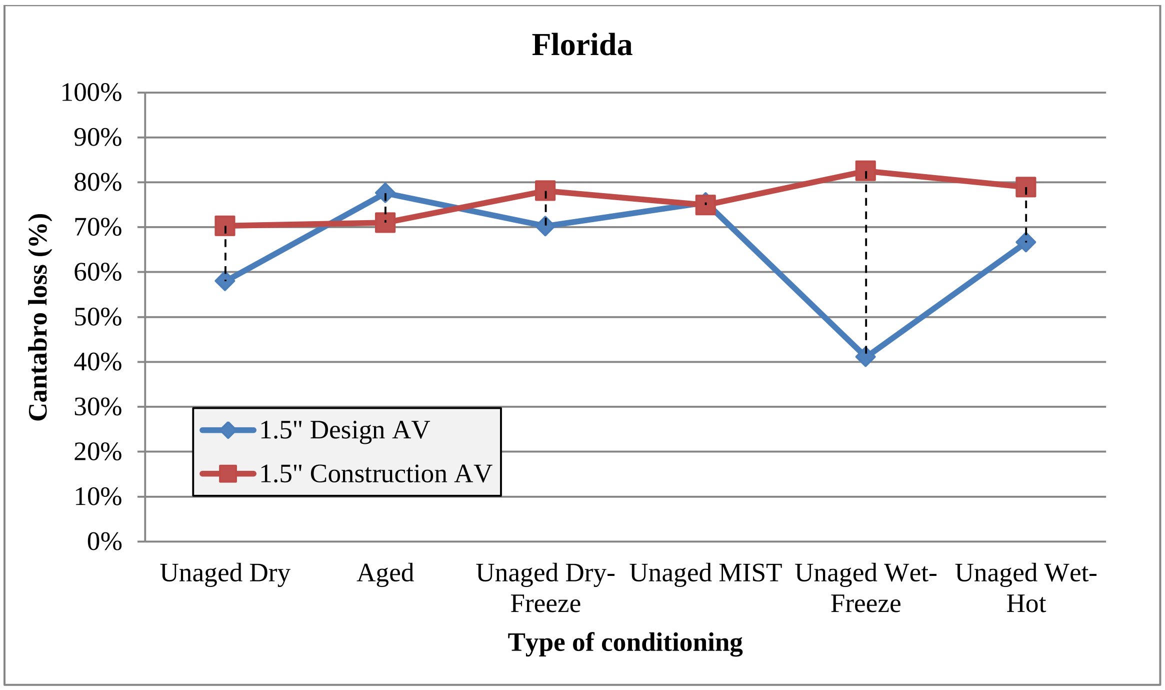


Figure 57. Effect of AV on Cantabro loss Florida material 1.5 in slab cores.

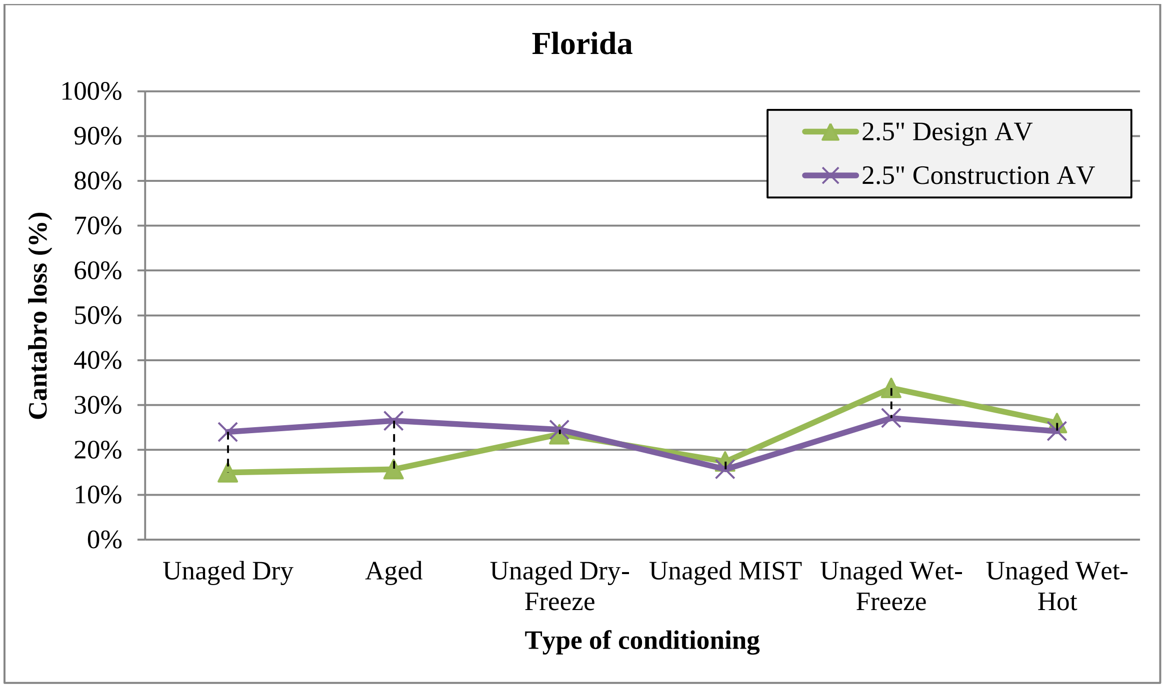


Figure 58. Effect of AV on Cantabro loss Florida material 2.5 in slab cores.

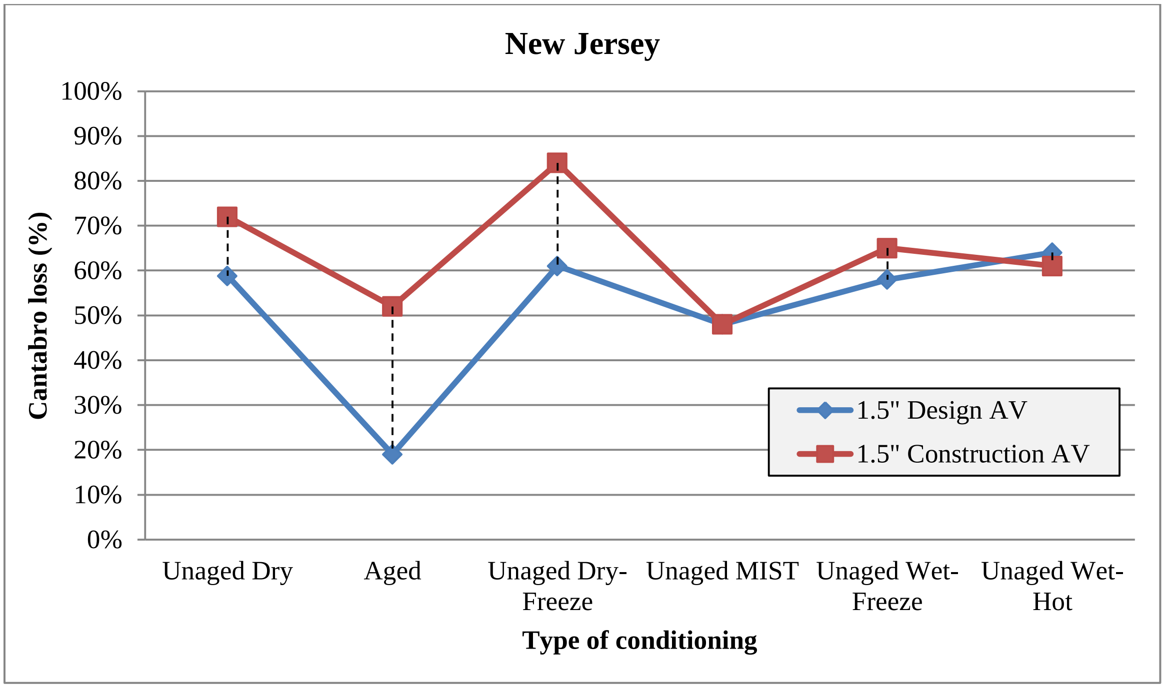


Figure 59. Effect of AV on Cantabro loss New Jersey material 1.5 in slab cores.

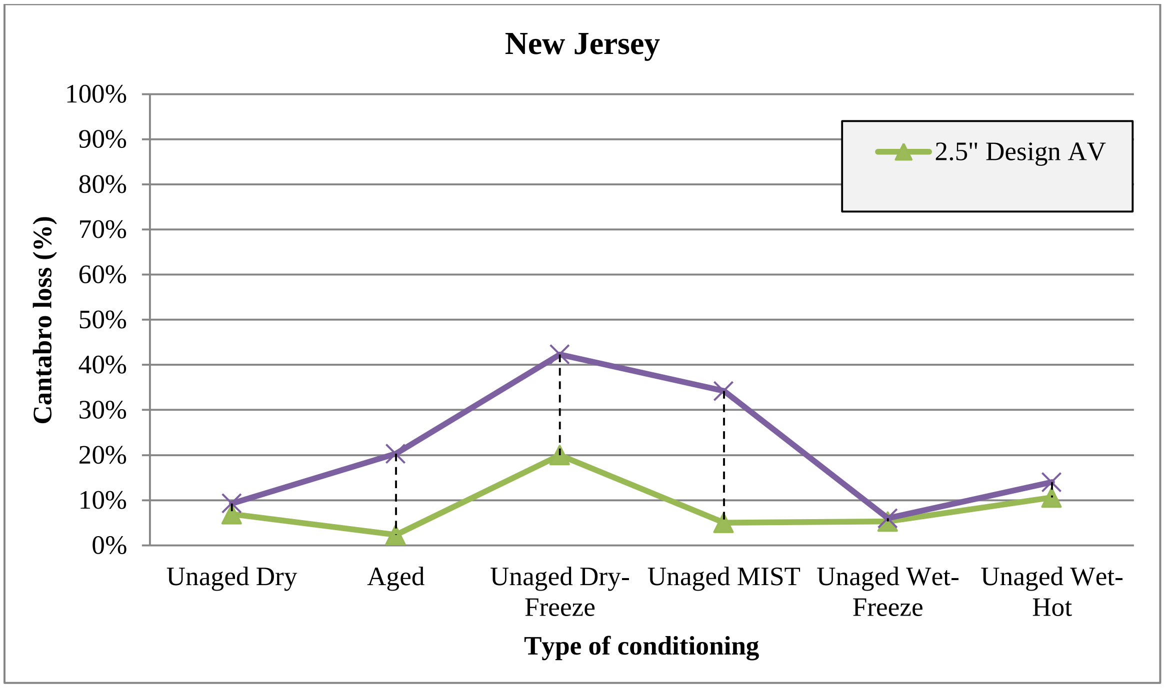


Figure 60. Effect of AV on Cantabro loss New Jersey material 2.5 in slab cores.

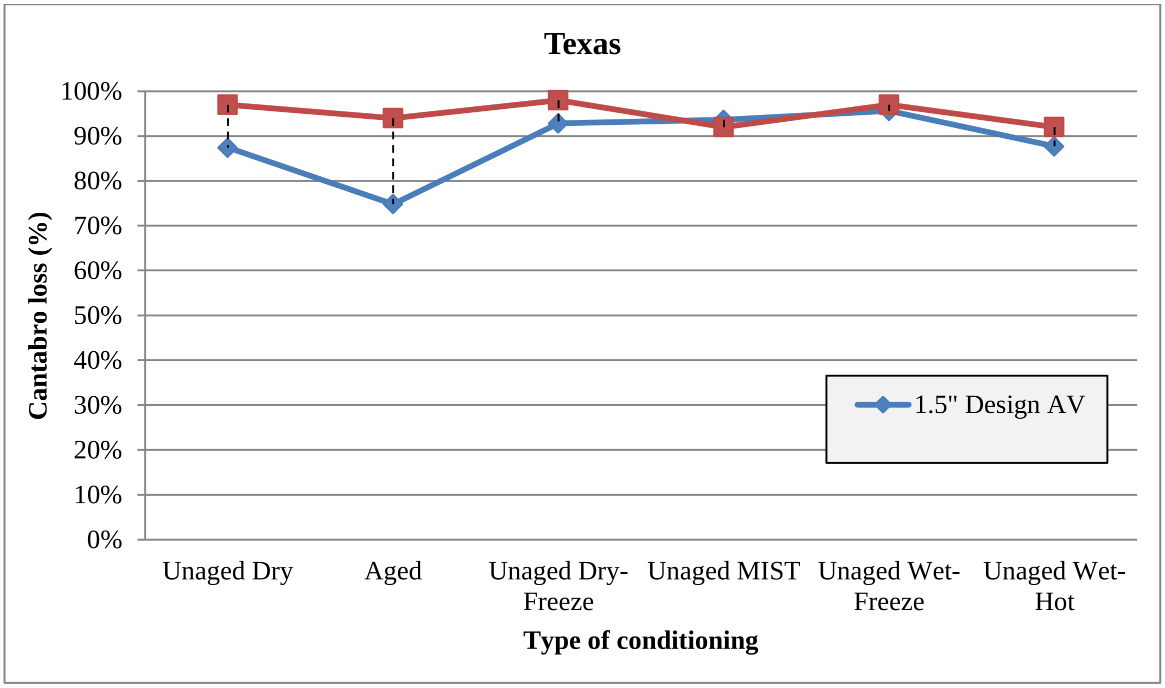


Figure 61. Effect of AV on Cantabro loss Texas material 1.5 in slab cores.

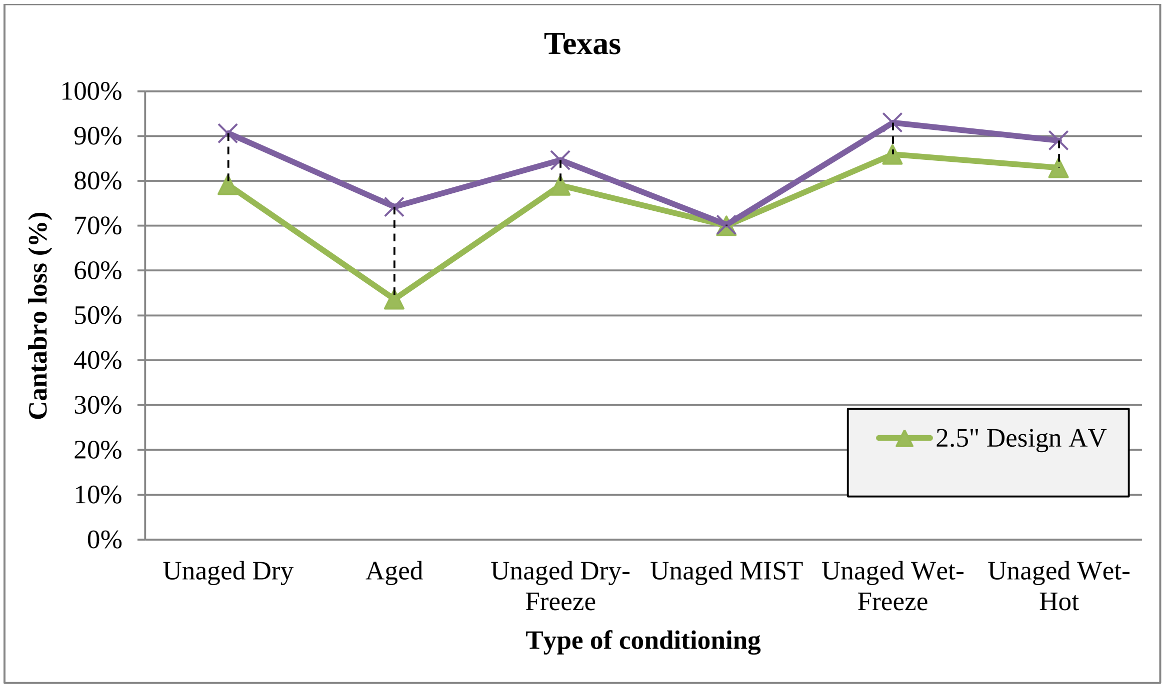


Figure 62. Effect of AV in Cantabro loss Texas material 2.5 in slab cores.

The complete statistical analysis performed with this data is presented in APPENDIX O. Based on that analysis, the lower design AV led to a significantly lower Cantabro loss than construction AV regardless of thickness or material type. The effect of AV on Cantabro loss is somewhat different depending on the conditioning protocol, however. While design AV led to a lower Cantabro loss than construction AV across all conditioning protocols, the difference between design and construction AV was statistically significant only for Unaged Dry-Freeze and Aged conditioning protocols. On average, the construction AV Cantabro loss was higher than that for the design AV by 13.9% and 6.6% for the 1.5 and 2.5 in Florida material, respectively; 19.2% and 48.1% for the same structures with the New Jersey material and 6.7% and 10.3% for the specimens compacted with the Texas materials.

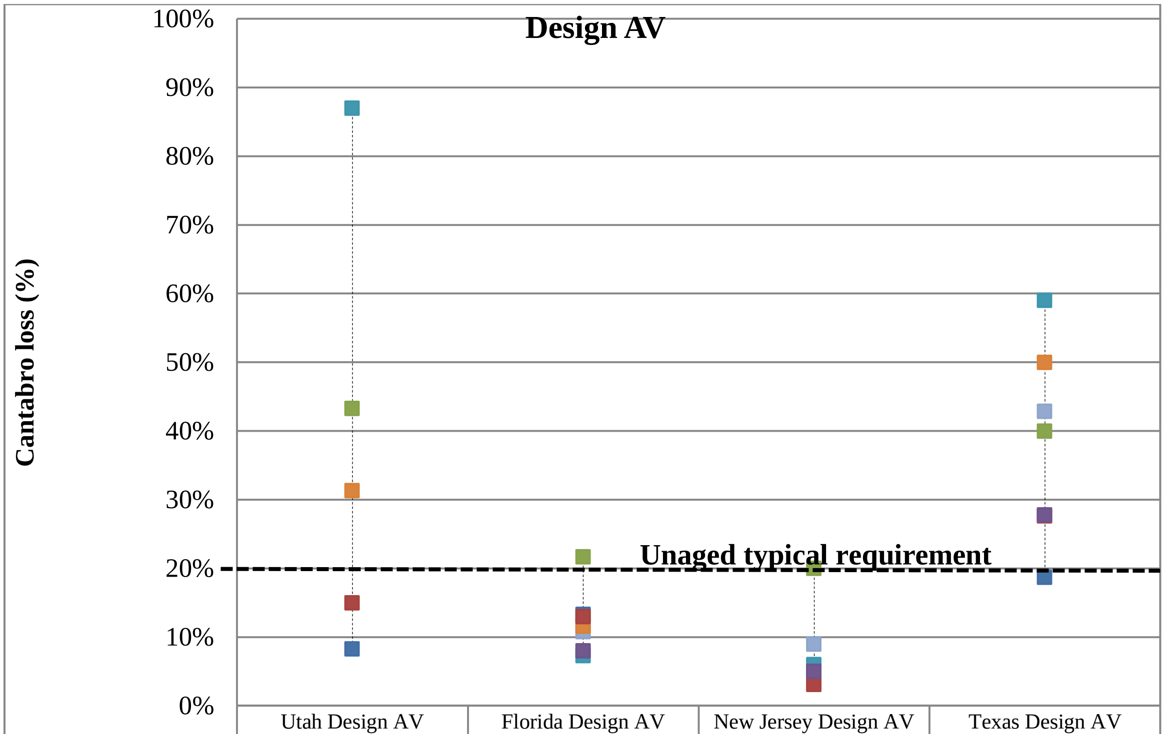
Thus regardless of the thickness or method of compaction, higher AV represented a detriment to the durability of PGA which is captured in the Cantabro loss test as higher Cantabro loss percentages. A mixture that is designed in the laboratory at a specific AV could result in Cantabro loss values between 6% and 50% higher when compacted in the field at higher AV. Hence, the control of AV is not only vital for functionality but also for durability. It is important to guarantee minimum AV to guarantee permeability but at the same time maximum AV to reduce the abrasion potential of the mixture.

5.2.2 Conditioning Protocol

Most of the current specifications regarding PGA included the Cantabro loss test with only two conditioning protocols: unaged and aged. Other conditioning protocols were evaluated in this study to assess the response of the mixture in its original state, after moisture damage with and without freezing and in an aged state. Seven different conditioning protocols for the SGC specimens were utilized; for the slab cores six conditioning protocols were used (the aged wet-freeze was introduced for the SGC specimens after testing was completed for the slabs).

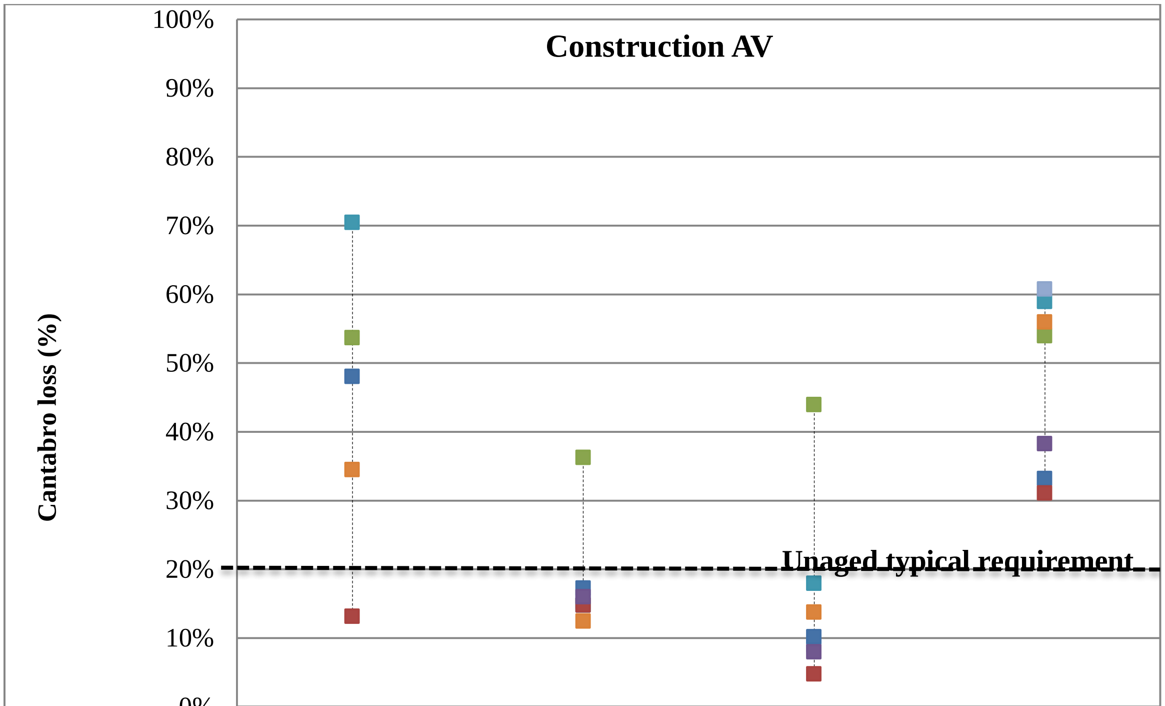
5.2.2.1 SGC Specimens

For the SGC specimens, materials from the four field projects were utilized. Figure 63 and Figure 64 present an illustration of the impact of each of the seven conditioning protocols for each of the four materials for design AV and construction AV, respectively. The Utah data contains only five conditioning protocols because the unaged MIST and aged wet-freeze protocols could not be supported by the mixture and the specimens were destroyed so the Cantabro loss test could not be performed.



	Utah Design AV	Florida Design AV	New Jersey Design AV	Texas Design AV
Unaged Wet-Freeze	87.0%	7.3%	6.0%	59.0%
Unaged Dry-Freeze	43.3%	21.7%	20.0%	40.0%
Aged Wet-Freeze		10.8%	9.0%	42.9%
Unaged Wet-Hot	31.3%	11.6%	4.6%	50.0%
Unaged Dry	8.3%	13.3%	4.8%	18.7%
Aged	15.0%	13.0%	3.1%	27.6%
Unaged MIST		8.0%	5.0%	27.8%

Figure 63. Effect of type of conditioning on Cantabro loss for Design AV SGC specimens.



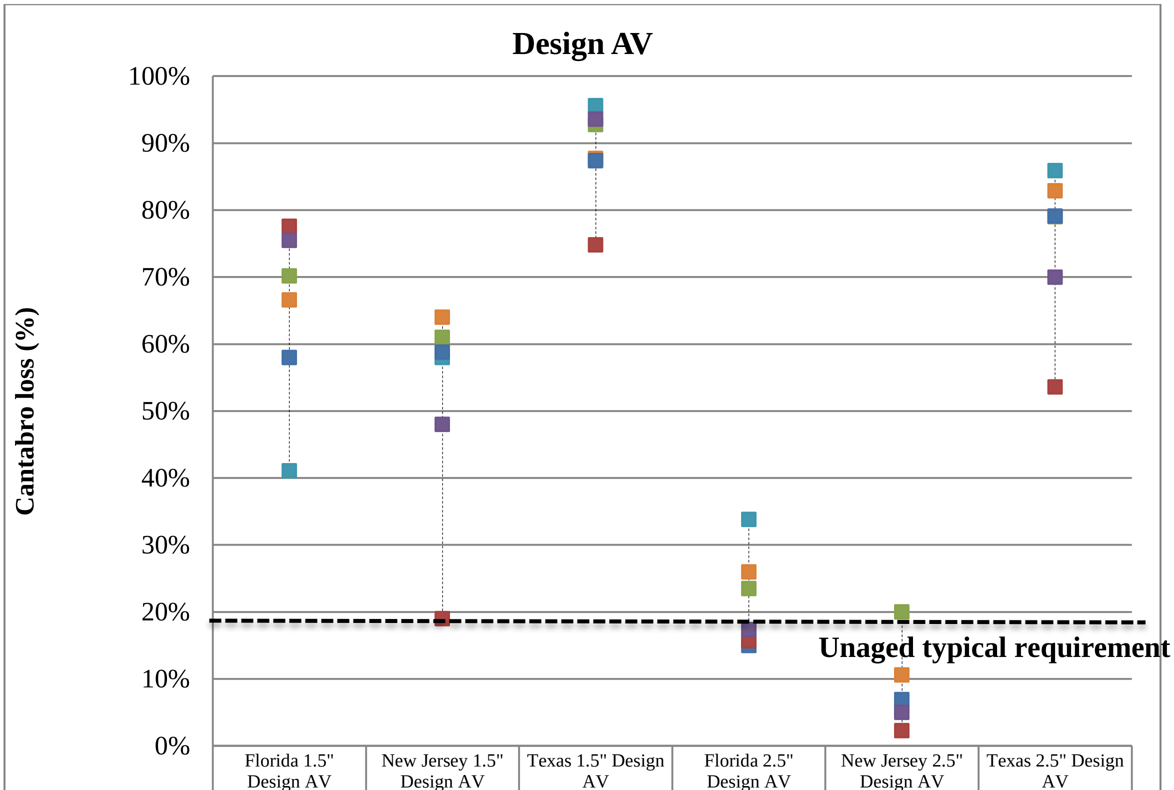
	Utah Constr AV	Florida Constr AV	New Jersey Constr AV	Texas Constr AV
Unaged Wet-Freeze	70.50%	14.80%	18%	59.00%
Unaged Dry-Freeze	53.7%	36.30%	44%	54.00%
Aged Wet-Freeze		15.30%	9%	60.80%
Unaged Wet-Hot	34.50%	12.50%	13.80%	56.00%
Unaged Dry	48.10%	17.30%	10.20%	33.20%
Aged	13.2%	14.80%	4.80%	31.13%
Unaged MIST		16.00%	8%	38.30%

Figure 64. Effect of type of conditioning on Cantabro loss for Construction AV SGC specimens.

A complete statistical analysis is presented in APPENDIX N. The type of conditioning protocol had a statistically significant effect on Cantabro loss at α=0.05. Unaged wet-freeze, unaged dry-freeze and aged wet-freeze led to higher Cantabro loss than unaged MIST, unaged dry or aged. This indicates that conditioning protocols which involve freezing were the most detrimental to the durability of the PGA mixtures. These results match what was encountered in the literature and can be related to the winter maintenance problems reported by several states and agencies. Hence, it is suggested to include in the specifications a conditioning protocol that includes freezing, especially for the locations in environmental zones classified as dry-freeze and wet-freeze. Another important thing is that in general the specifications require a maximum Cantabro loss of 20% for unaged specimens and 30% for aged specimens. However, in this study six of the eight combinations of materials and AV resulted in lower Cantabro loss values for the aged specimens compared with the unaged dry ones. Thus additional evaluation of the effect of aging on Cantabro loss is needed. In this case for instance, aging by itself enhanced the Cantabro loss properties; however, when aging was combined with a wet-freeze conditioning it resulted in an important decrease to the Cantabro loss value.

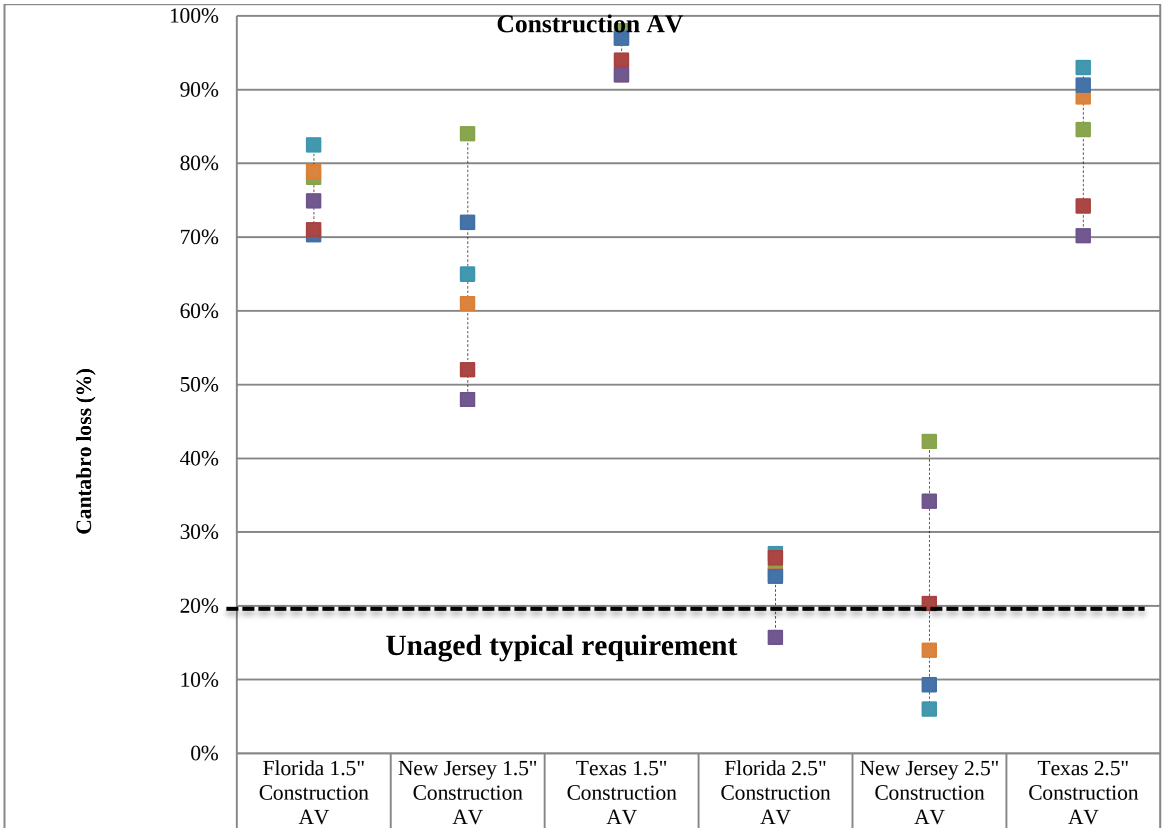
5.2.2.2 Slab Cores

The same analysis presented in the previous section was performed on the cores from the 1.5 in and 2.5 in slabs with the Florida, New Jersey and Texas materials. Figure 65 and Figure 66 illustrate the effect of conditioning protocol on these structures for design AV and construction AV, respectively.



	Florida 1.5" Design AV	New Jersey 1.5" Design AV	Texas 1.5" Design AV	Florida 2.5" Design AV	New Jersey 2.5" Design AV	Texas 2.5" Design AV
Unaged Wet-Freeze	41.1%	58.0%	95.6%	33.8%	5.3%	85.9%
Unaged Dry-Freeze	70.2%	61.0%	92.8%	23.5%	20.0%	79.0%
Unaged Wet-Hot	66.6%	64.0%	87.7%	26.0%	10.6%	82.9%
Unaged Dry	58.0%	58.8%	87.4%	15.0%	6.9%	79.1%
Aged	77.6%	19.0%	74.8%	15.7%	2.3%	53.6%
Unaged MIST	75.5%	48.0%	93.6%	17.4%	5.0%	70.0%

Figure 65. Effect of type of conditioning on Cantabro loss for Design AV slab cores.



	Florida 1.5" Construction AV	New Jersey 1.5" Construction AV	Texas 1.5" Construction AV	Florida 2.5" Construction AV	New Jersey 2.5" Construction AV	Texas 2.5" Construction AV
Unaged Wet-Freeze	82.5%	65.0%	97.0%	27.1%	6.0%	93.0%
Unaged Dry-Freeze	78.1%	84.0%	98.0%	24.5%	42.3%	84.6%
Unaged Wet-Hot	78.9%	61.0%	92.0%	24.2%	14.0%	89.0%
Unaged Dry	70.3%	72.0%	97.0%	24.0%	9.3%	90.6%
Aged	71.0%	52.0%	94.0%	26.5%	20.3%	74.2%
Unaged MIST	74.9%	48.0%	92.0%	15.7%	34.2%	70.2%

Figure 66. Effect of type of conditioning on Cantabro loss for Construction AV slab cores.

From the complete statistical analysis presented in APPENDIX O, the conditioning protocol had a significant effect on the Cantabro loss at α=0.05. In addition, the effect of the conditioning protocol on the Cantabro loss changed with the type of mixture. For Texas and Florida, unaged wet-freeze led to the highest Cantabro loss, but for New Jersey unaged dry-freeze led to the highest Cantabro loss. Hence, again the most detrimental conditioning protocols include freezing and that this effect was regardless of the thickness or AV of the mixtures. In terms of the aging analysis, similar behavior was encountered in that aged specimens did not always represent higher Cantabro loss values, but on the contrary were the same or sometimes less. In this case, five of the twelve structures exhibited lower values of Cantabro for aged specimens compared to the unaged dry ones.

According to both analyses, the type of conditioning protocol did have an effect on the Cantabro loss value of the PGA mixtures. Conditioning protocols that involve freezing were the most detrimental to the durability of the mixture and are worth considering for inclusion in specifications, especially for locations in dry-freeze and wet-freeze environmental zones. Additionally, it is suggested to reevaluate the aged threshold for the requirements and to contemplate the option of combining the aging procedure with some other procedure (i.e. freezing or moisture) to better assess the consequences of aging for PGA mixtures.

5.2.3 Type of Material

The Cantabro loss results were also utilized to identify if this test can be utilized to discriminate between mixtures and identify poor-performing mixtures, such as those without fibers, low binder content and/or low PG grade. Thus, the effect of the type of material on Cantabro loss for both SGC specimens and slab cores was analyzed.

5.2.3.1 SGC Specimens

For the SGC specimens, four different materials types were analyzed (Florida, Texas, New Jersey and Utah). Figure 67and Figure 68 illustrate the effect of the type of material on the Cantabro loss for design AV and construction AV, respectively.

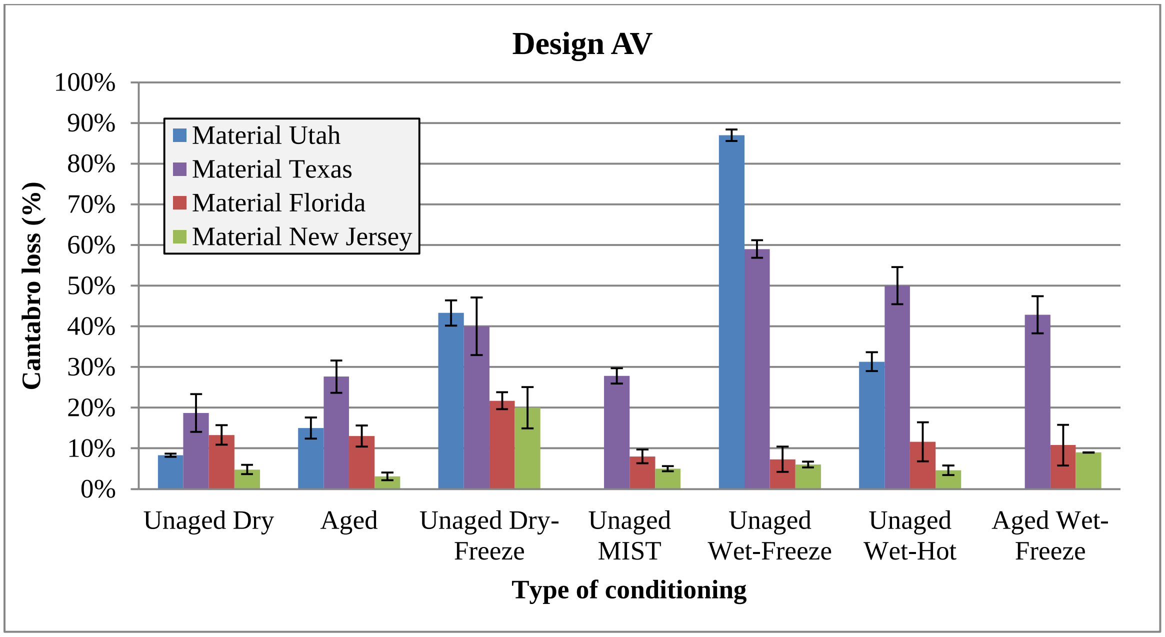


Figure 67. Effect of type of material on Cantabro loss SGC specimens Design AV.

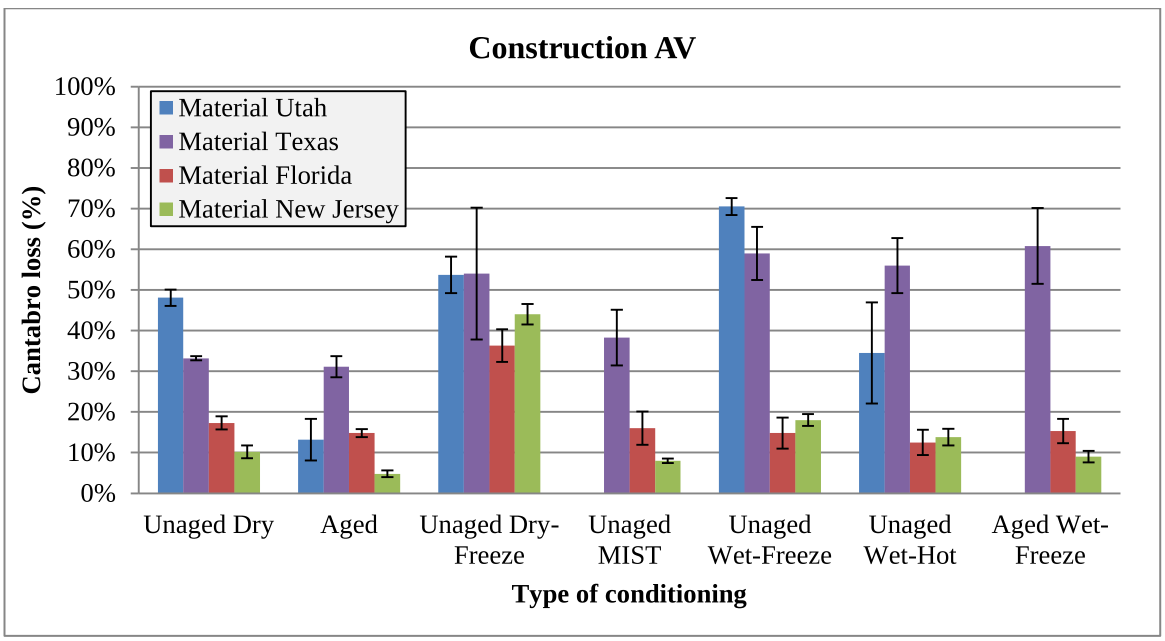


Figure 68. Effect of type of material on Cantabro loss SGC specimens Design AV.

From the statistical analysis described in APPENDIX N, Cantabro loss did capture a difference between the mixtures since the type of mixture had a statistically significant effect at $\alpha=0.05$. It was also determined that Florida and New Jersey materials led to a significantly lower Cantabro loss than Texas or Utah material. The ranking of the mixtures from best to worst was as follows:

1. New Jersey
2. Florida
3. Utah
4. Texas

However, the Utah mixture did not resist some of the conditioning protocols so the third and fourth place in the ranking is not definitive. It is important to underscore that the difference between ranks 1 and 2 as well as that between ranks 3 and 4 is not statistically significant, although there is a statistically significant difference between the top two and the bottom two based on Tukey's Honest Significant Difference (HSD) test. The difference between the New Jersey and Florida materials as compared to those from Utah could be explained due to the lower binder content, lower PG grade binder and lack of fiber. Hence, in this case the Cantabro test captures these differences in the material properties. However, the Texas mixture has a very similar mixture design to the Florida mixture, but its resistance to abrasion was significantly less. This could be due to lower quality of the aggregates or the binder.

5.2.3.2 Slab Cores

For the slab cores only three types of materials were utilized (Florida, New Jersey and Texas). Figure 69 and Figure 70 illustrates the type of material analysis for design and construction AV, respectively for the 1.5 in specimens while Figure 71 and Figure 72 show the 2.5 in specimens.

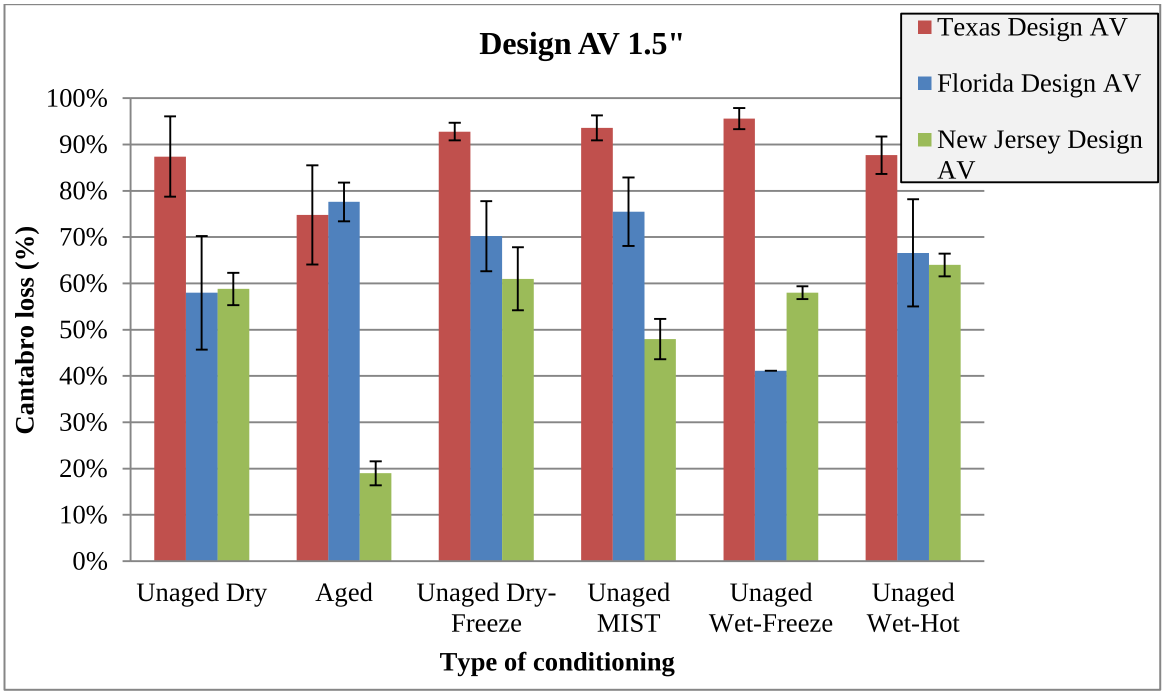


Figure 69. Effect of type of material on Cantabro loss slab cores 1.5 in specimens Design AV.

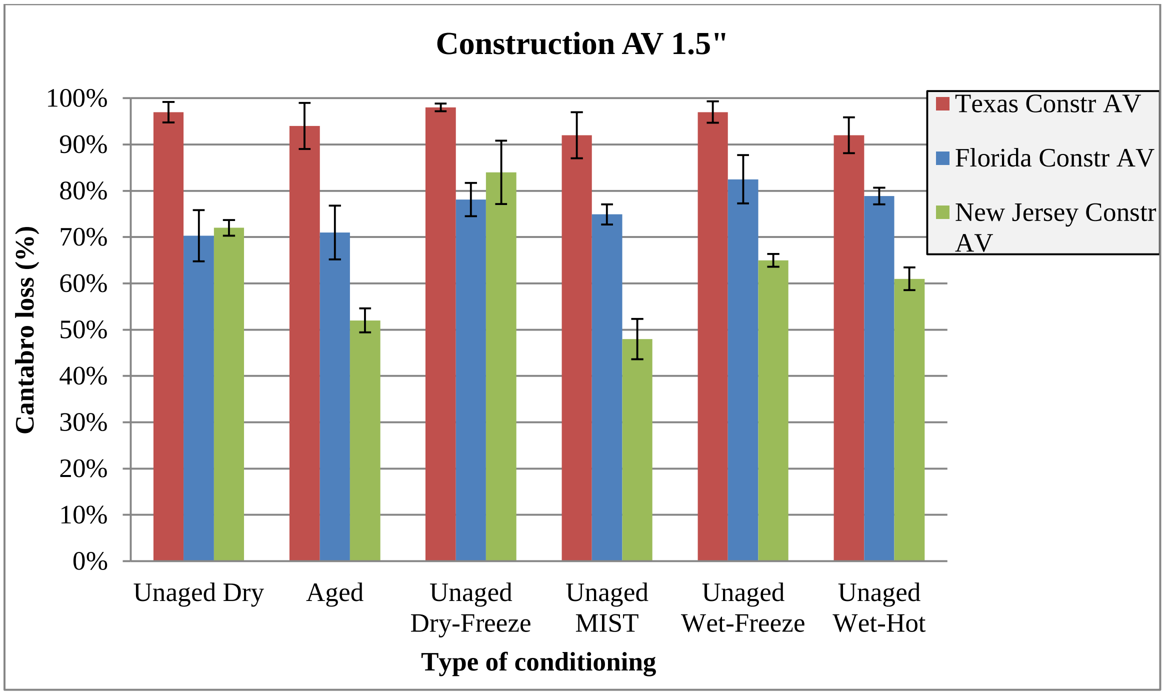


Figure 70. Effect of type of material on Cantabro loss slab cores 1.5 in specimens Construction AV.

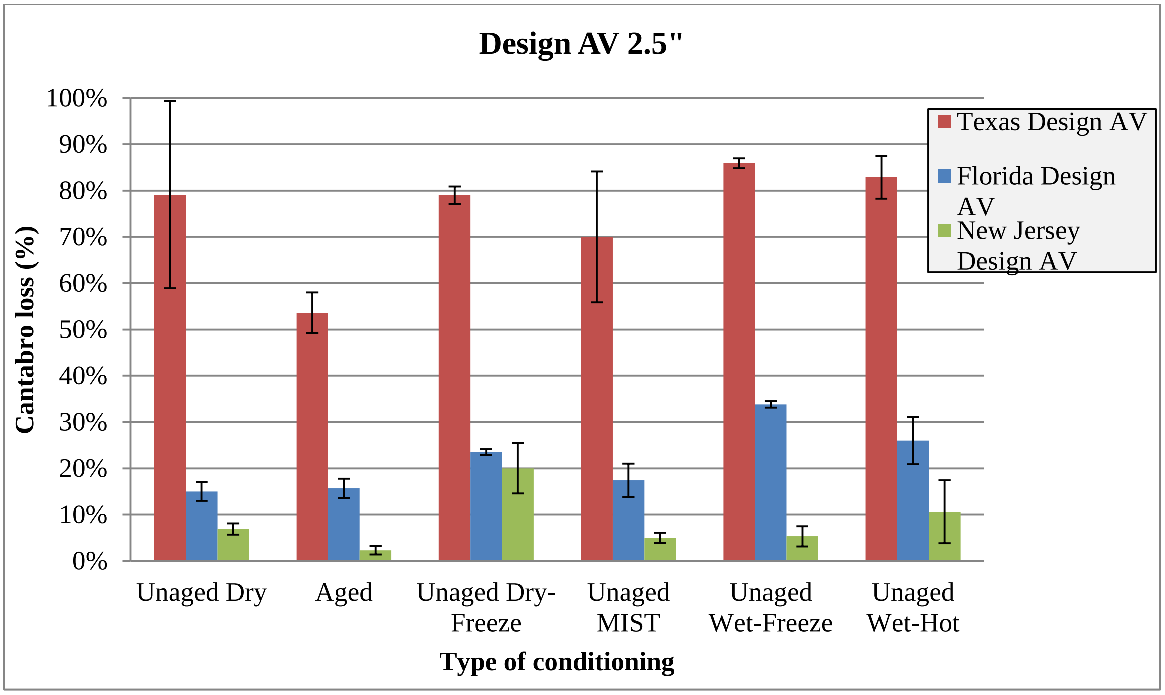


Figure 71. Effect of type of material on Cantabro loss slab cores 2.5 in specimens Design AV.

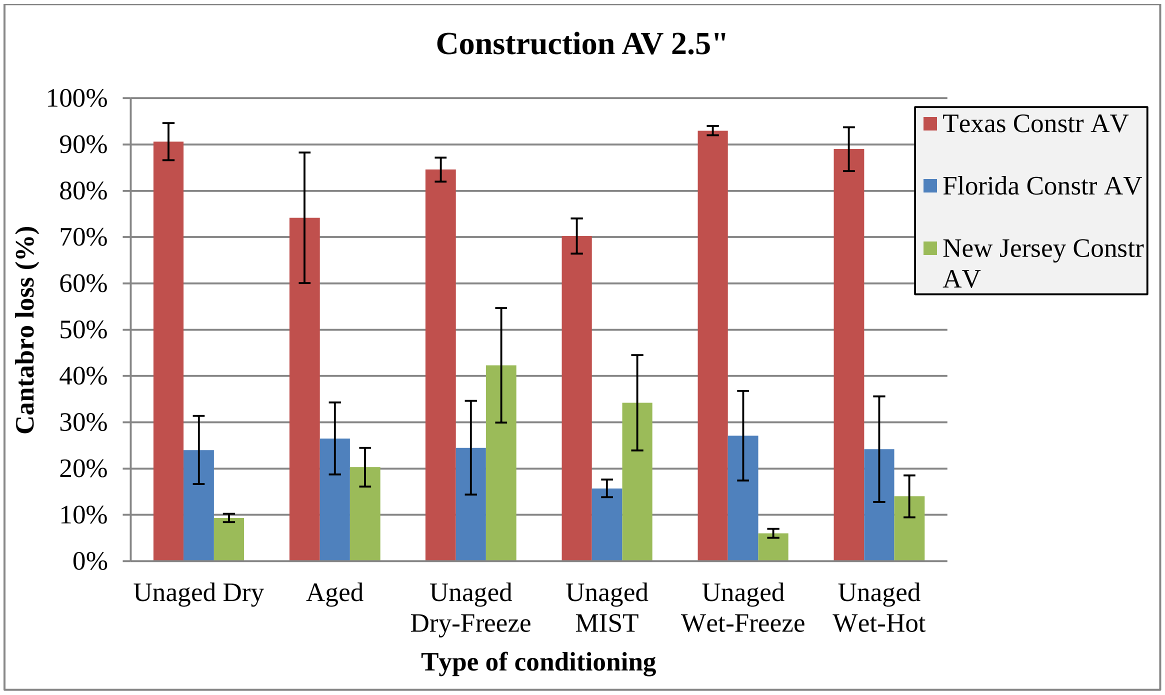


Figure 72. Effect of type of material on Cantabro loss slab cores 2.5 in specimens Construction AV.

According to this statistical analysis described in detail in APPENDIX O, the Cantabro loss test was also able to capture differences between the material types with a significant effect on Cantabro at α=0.05. For the slab cores, Texas led to the highest Cantabro loss regardless of the conditioning protocol or thickness. Between Florida and New Jersey, New Jersey led to lower Cantabro loss than Florida except after unaged dry-freeze conditioning. Hence, the ranking from best to worse was:

1. New Jersey
2. Florida
3. Texas

5.2.2 Thickness

Two different types of specimens were utilized in the durability assessment: SGC and slab cores. The SGC specimens had a height of 4.5 in, while the slab cores had heights of 2.5 in and 1.5 in. Even though the compaction method was different, the similar volumetric properties allowed for performing an analysis of the effect of thickness on Cantabro loss measurements. Current specifications require that the dimensions of the specimen of the Cantabro test must be 4.5 in; the effect of changing this thickness was analyzed. Figure 73, Figure 74, Figure 75, Figure 76, Figure 77 and Figure 78 illustrate the effect of the thickness on the Cantabro loss for both design and construction AV with the Florida, New Jersey and Texas materials, respectively.

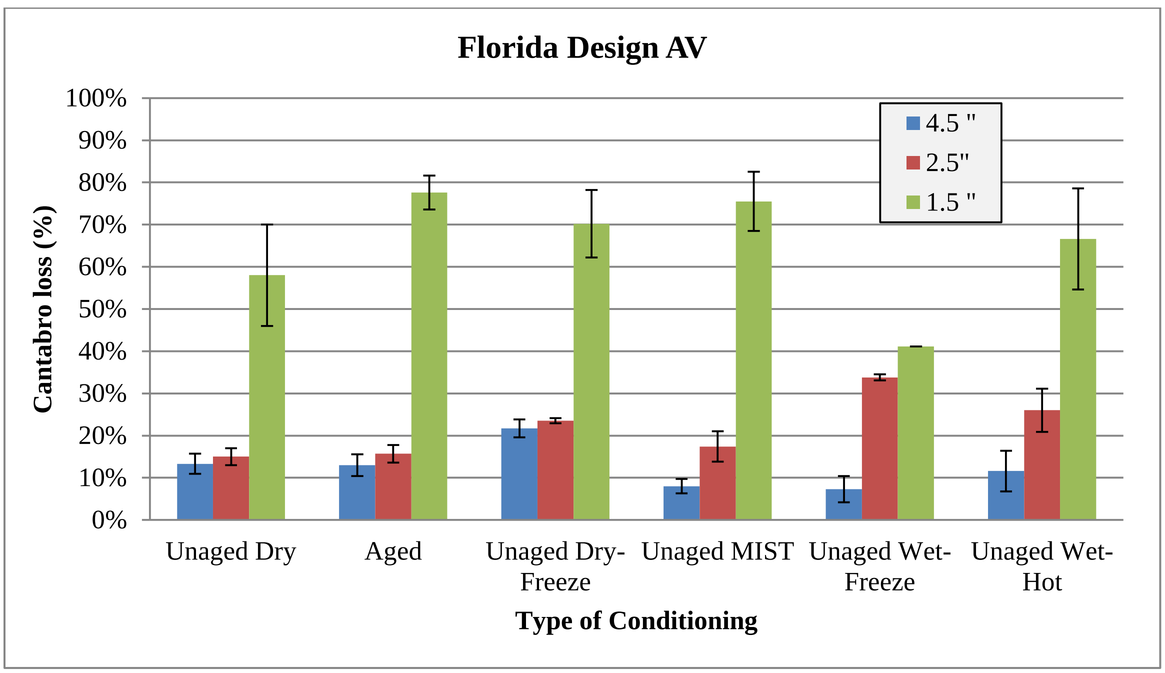


Figure 73. Effect of thickness on Cantabro loss for Florida material Design AV.

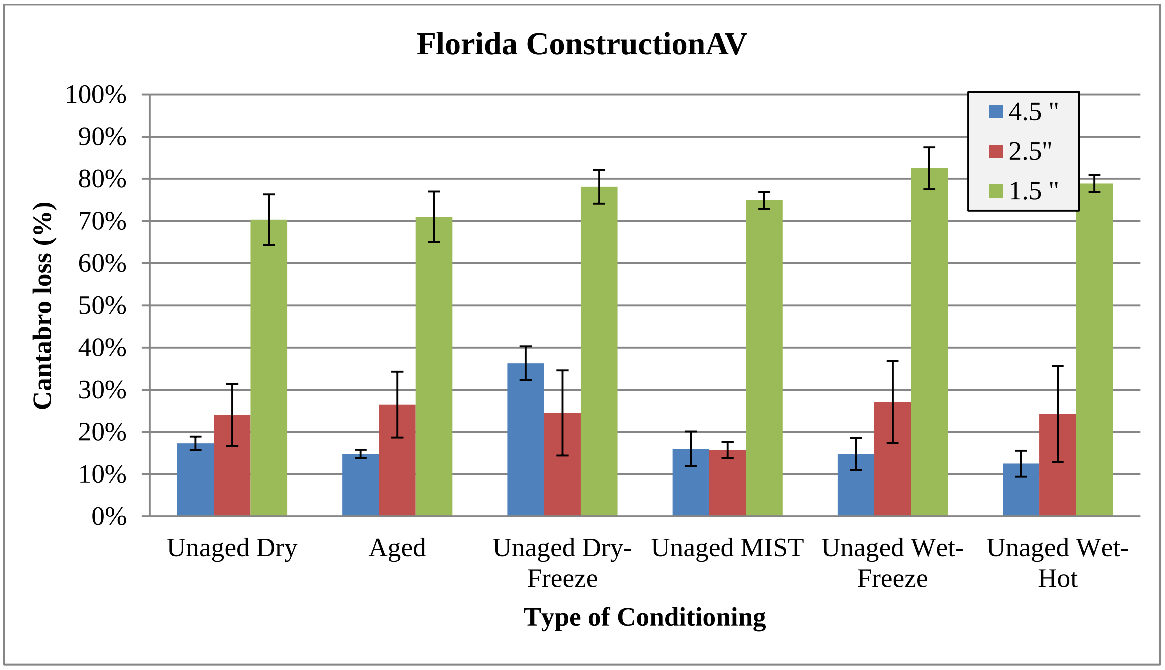


Figure 74. Effect of thickness on Cantabro loss for Florida material Construction AV.

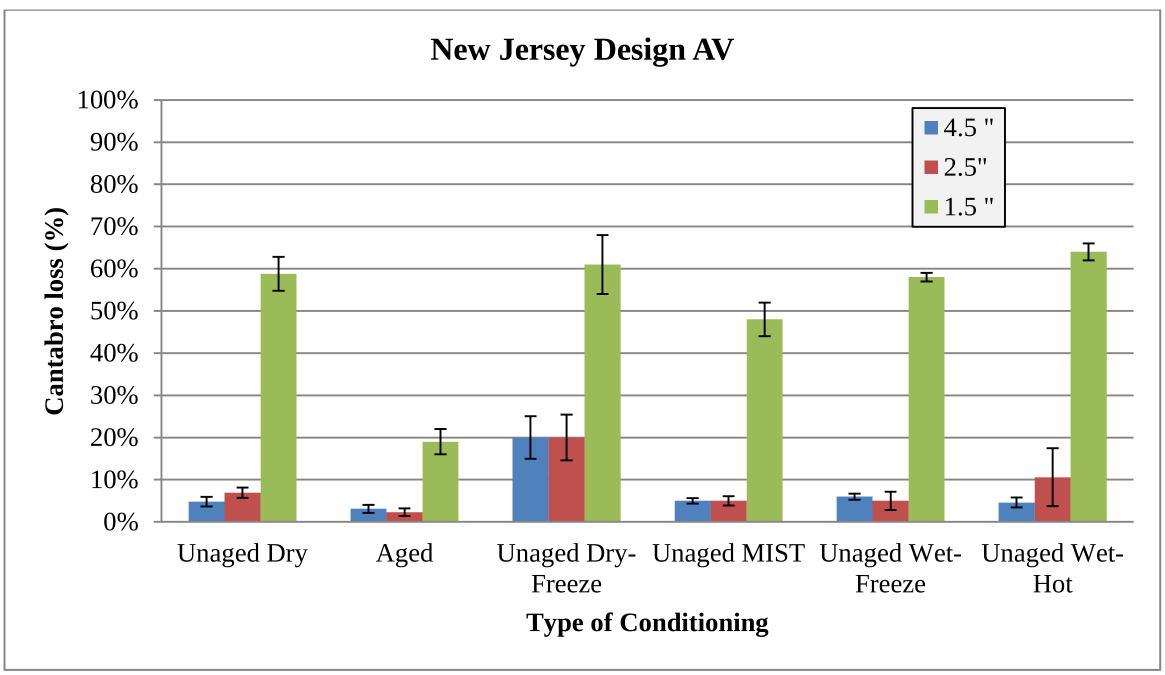


Figure 75. Effect of thickness on Cantabro loss for New Jersey material Design AV.

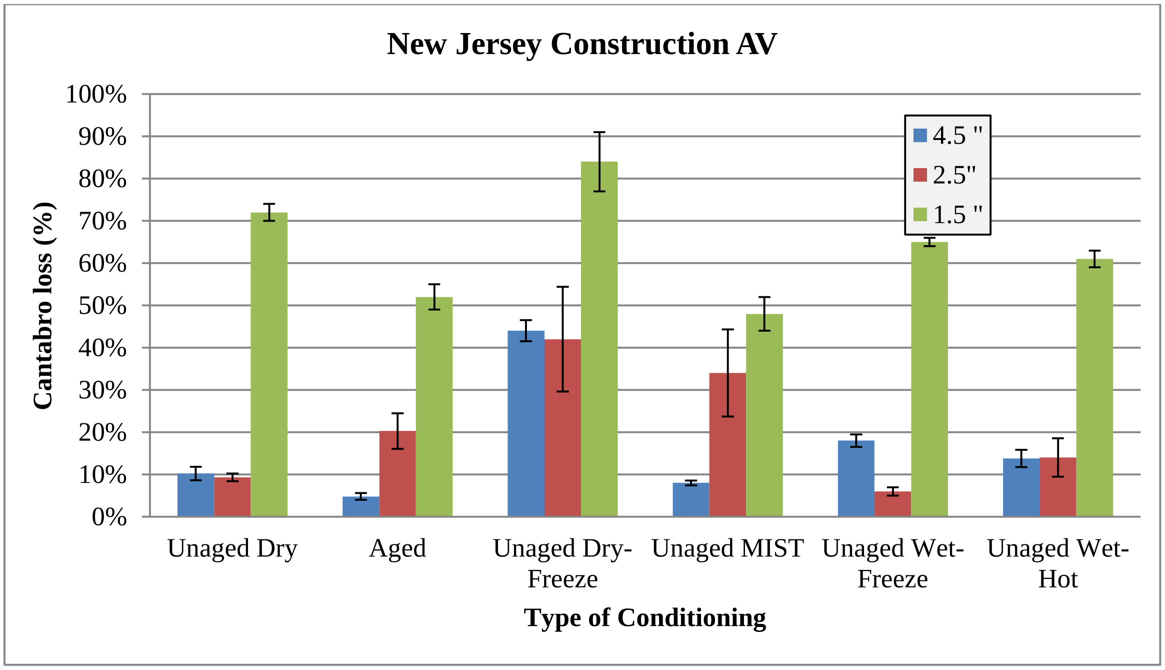


Figure 76. Effect of thickness on Cantabro loss for New Jersey material Construction AV.

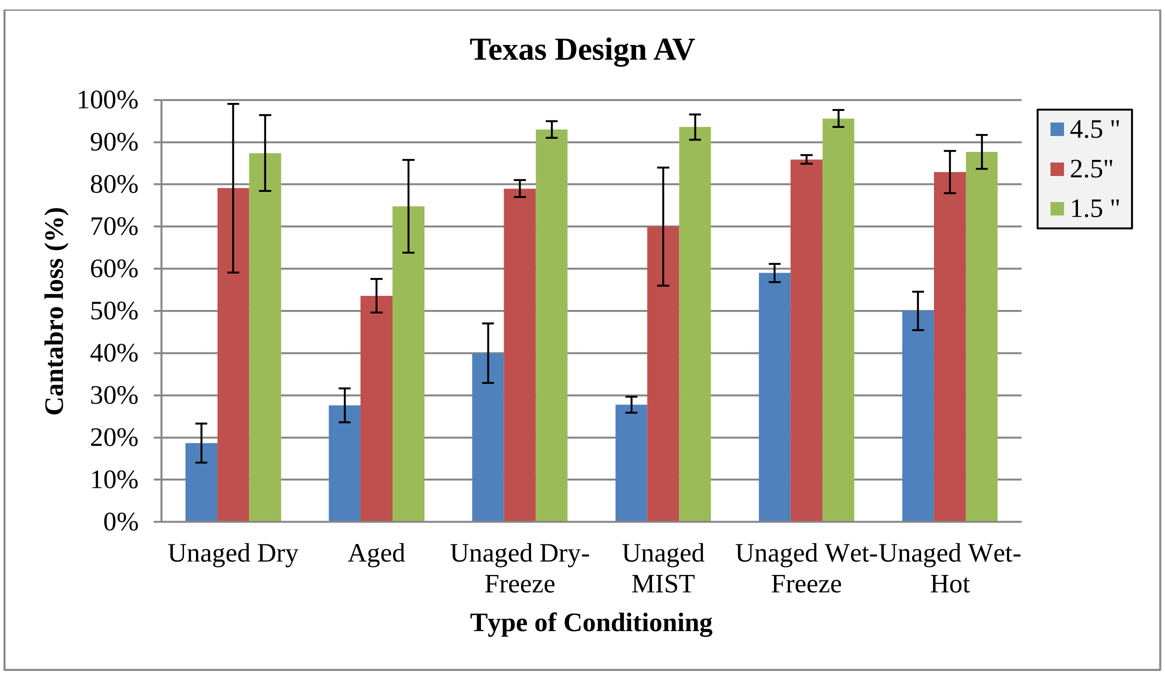


Figure 77. Effect of thickness on Cantabro loss for Texas material Design AV.

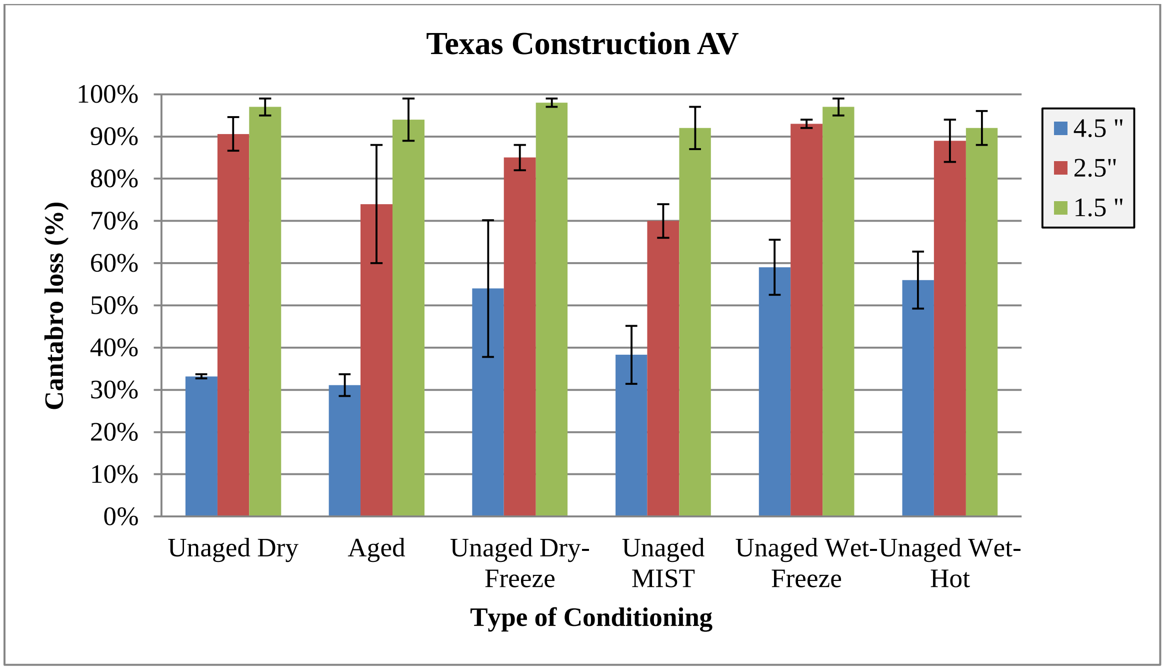


Figure 78. Effect of thickness on Cantabro loss for Texas material Construction AV.

Based on the statistical analysis, performed on this data and presented in APPENDIX P, in general the 1.5 in thickness led to the highest Cantabro loss, followed by the 2.5 in thickness and finally the 4.5 in thickness. However, the change between 1.5 in and 2.5 in was different than the change between 2.5 in and 4.5 in. On average, reducing the thickness from 2.5 in to 1.5 in increased the Cantabro loss by 52.2%; while decreasing the thickness from 4.5 in to 2.5 in increased the Cantabro loss by 32.8%. This suggests that the relationship between thickness and durability does not follow a linear relationship and further studies must be done to determine optimum thickness based on the durability, functionality, structural contribution of the pavement and cost.

5.3 Hamburg Wheel Tracking Test (HWTT)

The HWTT evaluates both rutting resistance and moisture susceptibility by subjecting a mixture sample to a repeated steel wheel load with a tracking device that moves back and forth on top of the sample. The samples used for these tests were SGC specimens with 2.5 in thickness. The output of the HWTT test is the rut depth of the specimens at the respective number of passes. The results for the four materials are presented in Figure 79.

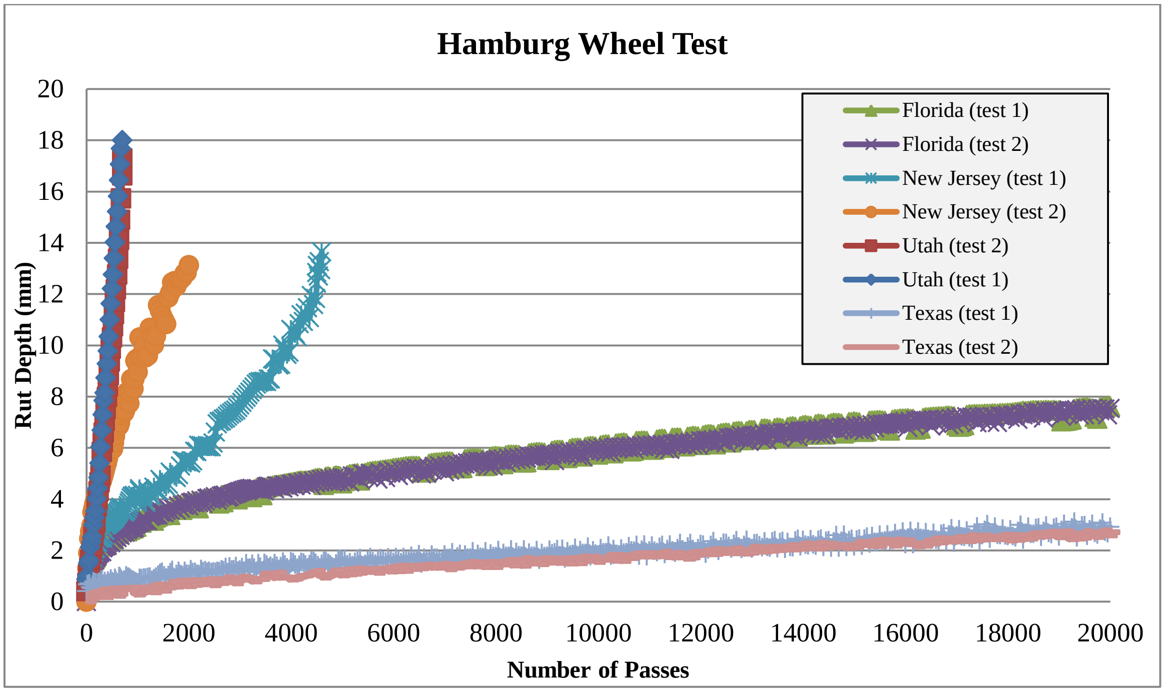


Figure 79. Rut Depth vs. Number of passes HWTT results.

In order to compare rutting and moisture susceptibility performance of the four type of materials, two methodologies were used: the AASHTO method and a TAMU method. The AASHTO method is the one described in AASHTO T324 (AASHTO, 2016b), while the TAMU method is a novel methodology proposed at Texas A&M University and detailed by Yin et al. (2014). Table 23 summarizes the main parameters of both methodologies utilized to analyze the rutting and moisture susceptibility performance of the four mixtures.

Table 23. HWTT results summary.

Material	Rutting Analysis		Moisture Susceptibility Analysis	
	Rut Depth 20,000 cycles (mm)	**Viscoplastic strain increment ($\Delta\varepsilon^{vp}_{LCsn}$)**	**Stripping Inflection Point**	**Load Cycles at which Stripping Number occurs (LC_{SN})**
New Jersey	Failed at 5000 cycles	2.07E-05	5510	1193
Florida	7.53	1.68E-06	>20000	>20000
Utah	Failed at 1000 cycles	Not enough data points for calculating		
Texas	2.81	2.30E-06	>20000	>20000

Only the Florida and Texas materials completed the HWTT test to 20000 load cycles. The Utah and New Jersey materials only reached 1000 and 5000 cycles, respectively, before they failed. It is suggested to verify whether this test is too harsh for this type of mixture taking into account the temperature of the test, the binder PG grade and the AV structure. Also, studies have shown that this test has a very high variability for PGA mixtures (Alvarez et al., 2010).

In terms of rutting, both the Florida and Texas mixtures fulfilled the requirement of 12.5 mm at 20,000 cycles proposed by NCAT (See Table 16). The Texas mixture only exhibited a 2.81 mm rut compared to 7.53 mm for the Florida mixture. The New Jersey and Utah materials failed before the test was completed. The TAMU analysis used the viscoplastic strain increment which is expected to be higher with mixtures more susceptible to rutting. For this parameter the Florida mixture exhibited the lower value, followed by the Texas mixture and then the New Jersey mixture. The Utah material failed so fast that there were not enough data points to calculate this parameter. The moisture susceptibility analysis suggested that the Florida and Texas mixture did not show stripping since the SIP and LC_{SN} were higher than the standard number of cycles of the test (i.e. 20,000 cycles). The New Jersey results for SIP and LC_{SN} were 5510 and 1193, respectively, suggesting that for this mixture there is a high stripping potential in the early life of the pavement. The Utah data points were not sufficient to calculate these two parameters either. After performing a statistical analysis which presented in APPENDIX Q, the ranking of the mixtures from best to worst (no statistically significant difference for the states in the same [ ]) was as follows:

- For Rut Depth at 20,000 cycles: [Florida (2)], [Texas (1)], [New Jersey (3),Utah (4)]
- For Viscoplastic strain increment: [Florida (1), Texas (2), New Jersey (3)]
- For Stripping Inflection Point: [Florida (1), Texas (2)], [New Jersey (3), Utah (4)]
- For Load Cycles at which Stripping Number occurs LC_{SN}: [Florida (1), Texas (2)], [New Jersey (3)]

There seems to be a relationship between the PG grading of the binder and the performance in this type of test. Florida and Texas materials which had a PG 76-22 binder exhibited the best performance, while Utah materials with a PG 58-22 binder exhibited the worst performance with New Jersey (PG 64-22 binder) in the middle. This could be due to the constant temperature of the test at 50ºC without taking into account the PG of the binder. Based on this, and also on the lack of literature referring to rutting in PGA, this test may not be applicable for capturing PGA performance.

5.4 Semi Circular Bending (SCB) Test

The SCB test is utilized to analyze the resistance of a mixture to cracking. The test consists of subjecting a semi-circular specimen to monotonic loading until fracture failure occurs under a constant rate of deformation in a three-point bending load as described in ASTM standard D8044 (ASTM, 2016). The output of the SCB test is a load vs. displacement curve for the material. In this case, four tests were run for each material (Florida, New Jersey, Texas and Utah) at the design AV of the mixture. The specimens utilized in this case were PMPC specimens. Figure 80 to Figure 83 illustrate the response of the four materials to the SCB test.

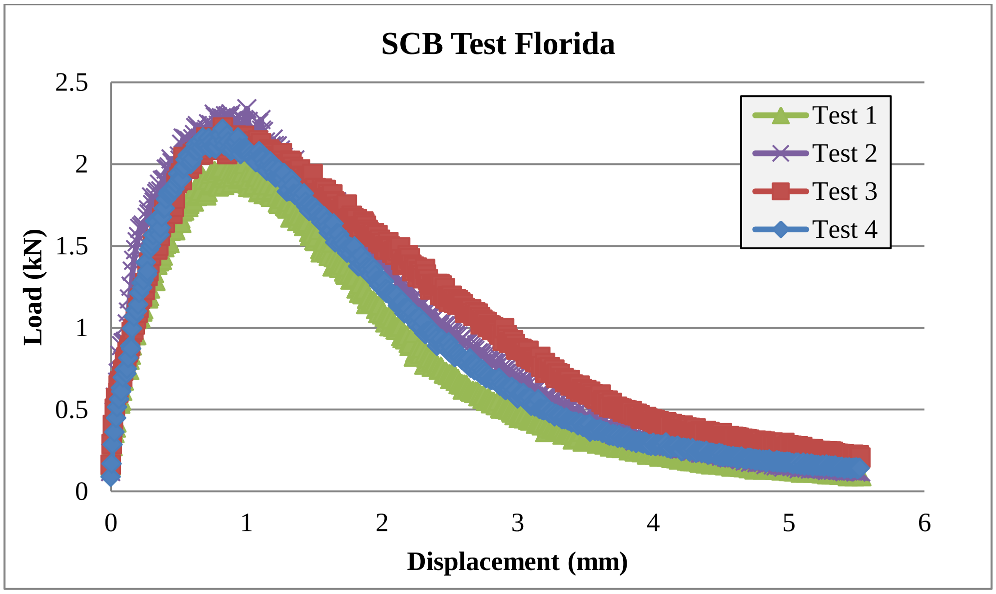


Figure 80. Load vs Displacement SCB test Florida material.

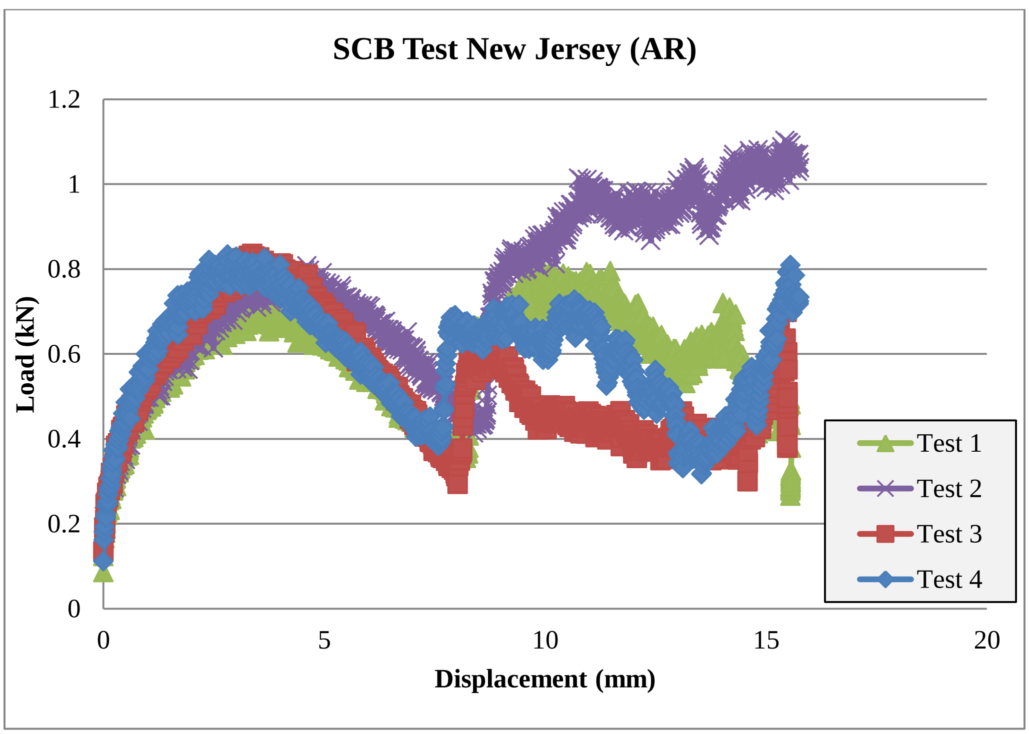


Figure 81. Load vs Displacement SCB test New Jersey material.

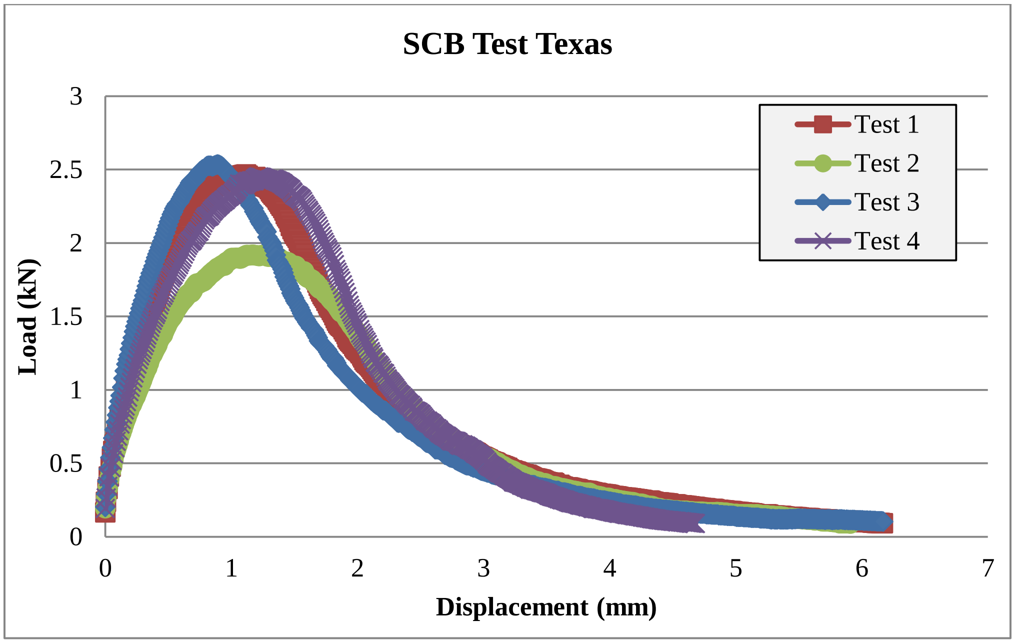


Figure 82. Load vs Displacement SCB test Texas material.

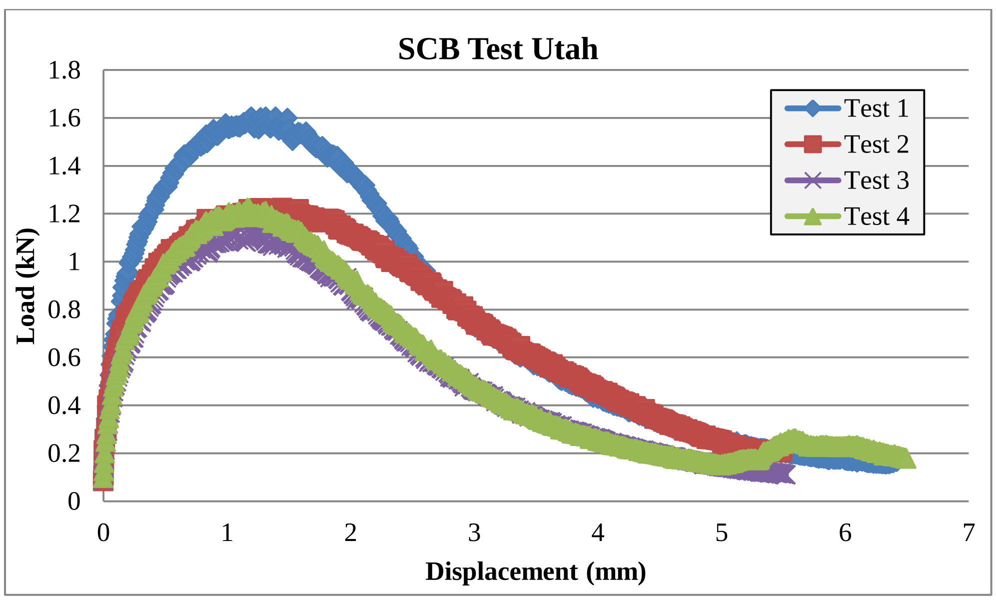


Figure 83. Load vs Displacement SCB test Utah material.

Figure 80, Figure 82 and Figure 83 present a normal viscoelastic response of an asphalt mixture subjected to the SCB test. However, Figure 81 does not have the same shape and this is because the presence of rubber in the binder. Rubber is an elastomer which classifies this material as hyperelastic, and its response to load is different than a purely viscoelastic material. Figure 81 has the initial shape of the viscoelastic material, but when it cracks the rubber starts to respond which is translated as a rise after displacements higher than 8 mm in Figure 30.

The typical analysis of the SCB is through the Flexibility Index (FI) described in the provisional standard AASHTO TP-124 (AASHTO, 2016a). The calculation of this parameter is presented in Equations 4, 5, 6 and 7. This index is an indicator of the elasticity of the material and its resistance to cracking based on the slope after the peak load, which means that is dependent on a perfect viscoelastic curve as shown in Figure 84. However, if the material is too brittle due to the use of Reclaimed Asphalt Pavement (RAP) and Recycled Asphalt Shingles (RAS) or too elastic as in the case of the New Jersey material due to the use of rubber, the shape varies and the parameter for calculating this index cannot be determined. Thus, it is crucial to determine whether this test and this criterion is applicable for PGA mixtures given that AR is widely used for this type of mixture.

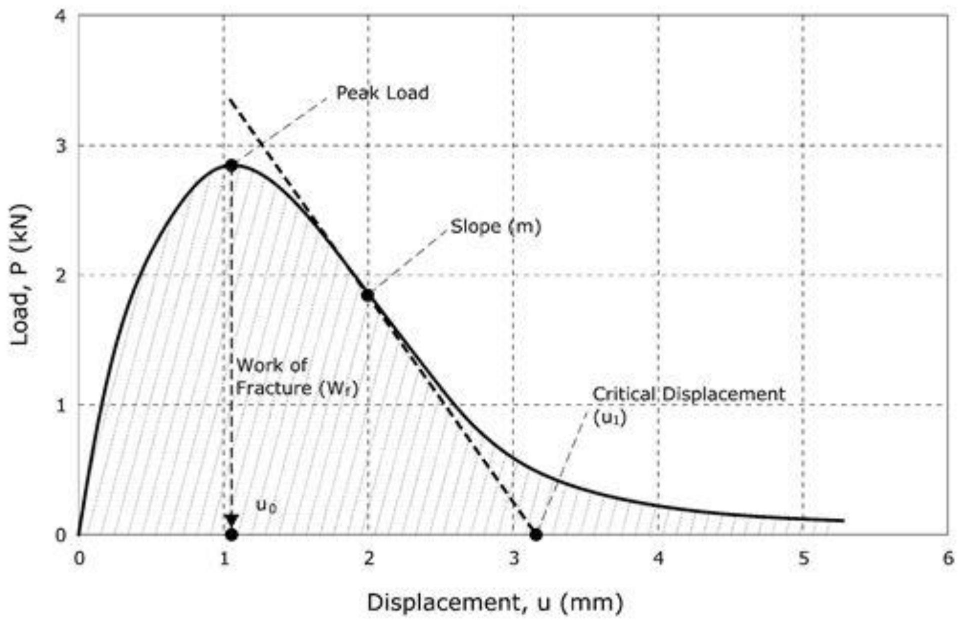


Figure 84. Example of Load vs Displacement Curve Parameters for SCB test (IDOT, 2016).

$$Area_{lig} = (r - a) * t \qquad \text{(Equation 4)}$$

$$G_f = \frac{W_f}{Area_{lig}} \qquad \text{(Equation 5)}$$

$$FI = \frac{G_f}{|m|} * 0.01 \qquad \text{(Equation 6)}$$

$$CI = \frac{G_f}{P} \qquad \text{(Equation 7)}$$

Where,

G_f = fracture energy (Joules/m^2)

W_f = work of fracture: *Area under the curve Load vs. Displacement* (Joules)

$Area_{lig}$ = ligament area (mm^2)

r = specimen radius (mm)

a = notch length (mm)

t = specimen thickness (mm)

m = post-peak slope (kN/mm)

P= peak load (kN)

FI= Flexibility Index (Joules/m^2)

CI=Cracking Resistance Index (Joules/kN*m^2)

As an alternative to the FI, the Cracking Index (CI) has been proposed by some researchers in Texas A&M to analyze the cracking behavior of the material (Kaseer et al., 2016). This parameter is calculated as shown in Equation 4 and depends only on the fracture energy and the peak load which means that it is only necessary to know the maximum load and the area under the curve regardless of the shape. Both parameters were calculated for the three types of mixtures, and the results are summarized in Table 24.

Table 24. Results for SCB test analysis.

Parameter	Florida	New Jersey	Utah	Texas
Flexibility Index (Joules/m2)	18.55	-	19.41	7.09
Cracking Index (Joules/kN*m2)	701.27	1710.67	843.20	534.55

According to the CI analysis, the New Jersey material was most flexible followed by Utah, Florida and then Texas. This parameter represented the behavior as expected because the New Jersey not only contained asphalt rubber but also a higher binder content of 8% which made it very flexible. Comparing the Florida and Utah mixtures which have similar binder contents (around 6%) without rubber, the Utah mixture exhibited higher CI values since its binder was more flexible (PG 58-22), while the Florida mixture had less flexibility since its binder was stiffer (PG 76-22). The Texas material had the lowest CI; however, this value was very different from that for the Florida material even though both have the same PG grade for the binder. This could represent misleading classification of the binder or in the binder properties which were also shown with a lack of durability of the Texas mixture according to the Cantabro test.

The FI index adequately represents the relationship between the Florida, Texas and Utah mixtures; however, this analysis could not be performed for the New Jersey material. The statistical analysis presented in APPENDIX R agrees that the effect of material type was statistically significant at α=0.05 for both FI and CI. The ranking of the mixtures from best to worst (no statistically significant difference for the states in the same [ ]) was as follows:

- For Flexibility Index (FI): [Utah (1), Florida (2)], [Texas (3)].
- For Cracking Index (CI): [New Jersey (1)], [Utah (2), Florida (3), Texas (4)].

In conclusion, the SCB test can be utilized to characterize cracking resistance of PGA mixtures; however, the analysis of the data is difficult for some PGA materials and some revision of the criteria proposed by NCAT, and presented in Table 26, is recommended.

5.5 Indirect Tensile (IDT) Strength/ Tensile Strength Ratio (TSR) Tests

The IDT strength test consists of loading a cylindrical specimen across its vertical diametral plane at a specified rate of deformation in order to identify the peak load and calculate the strength of the specimen. By performing this test on unconditioned and conditioned specimens according to AASHTO T283, the TSR can be calculated to represent the moisture susceptibility of the mixture. PMPC specimens were utilized. Figure 85 presents the results for the IDT and TSR tests.

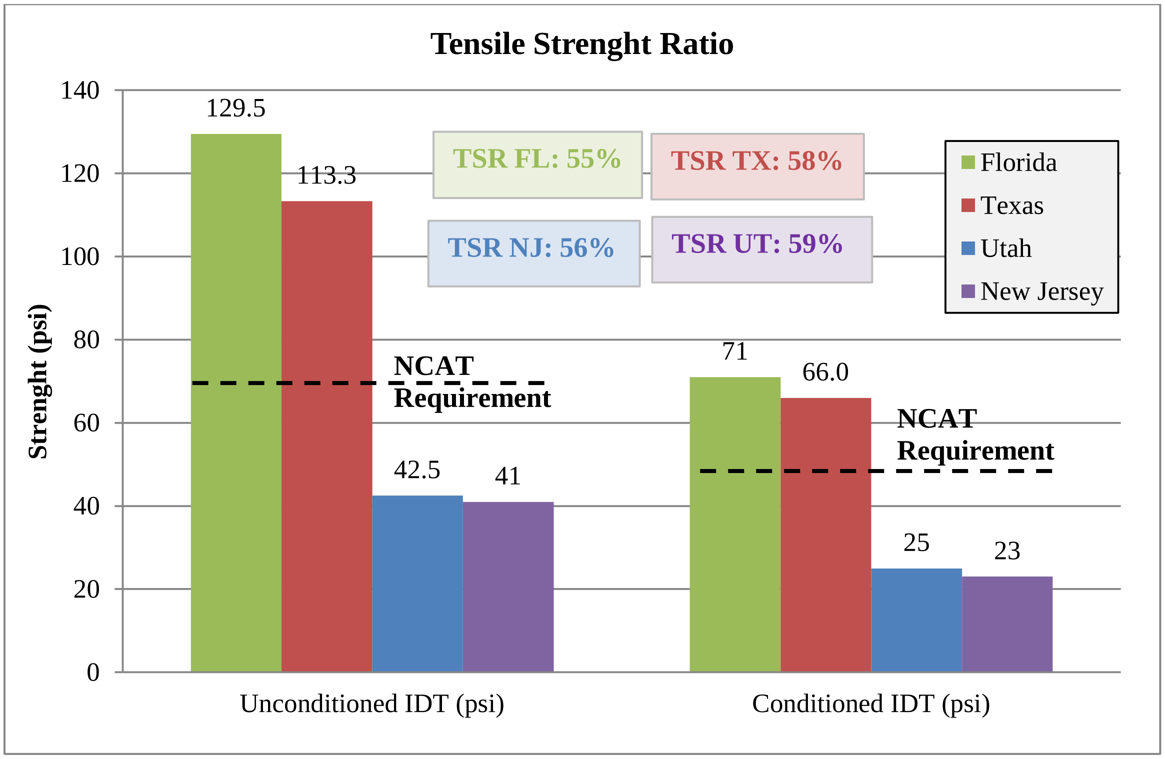


Figure 85. Indirect Tensile Strength and Tensile Strength Ratio results.

A complete statistical analysis was performed on the data as presented in APPENDIX S. The effect of material type was statistically significant at α=0.05 for unconditioned IDT and conditioned IDT while the effect was not statistically significant for TSR. The Florida mixture exhibited the best tensile strength both for conditioned and unconditioned specimens followed by the Texas material. Only these two mixtures fulfilled the requirements proposed by NCAT; however, none of the mixtures fulfilled the TSR requirement. The strengths of the New Jersey and Utah material were very similar, but both of them fulfilled neither the strength nor the TSR requirement. The ranking of the mixtures from best to worst was as follows:

- For Unconditioned IDT: [Florida (1), Texas (2)], [Utah (3), New Jersey (4)].
- For Conditioned IDT: [Florida (1), Texas (2)], [Utah (3), New Jersey (4)].

- For TSR: [Utah (1), Texas (2), New Jersey (3), Florida (4)].

Taking into account that PGA mixtures are not structural layers and that asphalt mixtures are generally not strong in tension, it is recommended to study whether a combination of the AASHTO T283 conditioning protocols with other tests (i.e. Cantabro loss) could be more adequate for this type of mixture.

5.6 Conclusions for Durability Analysis

The durability of four different types of PGA layers obtained from field sites in Florida, New Jersey, Texas and Utah was assessed via Cantabro loss, HWTT, SCB and IDT/TSR. Two different types of specimens were used the SGC and slab cores. The SGC specimens were 4.5 in height for the Cantabro loss and 2.5 in height for the HWTT, SCB and IDT/TSR and the slab cores that were only assessed via Cantabro had heights of 1.5 in and 2.5 in. The Cantabro loss assessment was performed in both design and construction AV while the HWTT, SCB and IDT/TSR were performed from PMPC specimens brought from the field. The following points summarize the main findings of the durability assessment:

- The AV has a statistically significant effect on the Cantabro loss at $\alpha=0.05$. The analysis also suggested that construction AV leads to significantly higher Cantabro loss than design AV despite of the thickness or method of. A mixture that is design in the lab at certain AV could result with Cantabro loss values between 6% and 50% higher when compacted in the field. Hence, it is demonstrated that the control of AV is not only vital for functionality issues but for durability issues too. It is important to guarantee minimum AV to guarantee permeability but at the same time maximum AV to reduce the abrasion potential of the mixture.
- According to both analyses it is evident that the type of conditioning does have an effect on the Cantabro loss value of the PGA. It was found that conditionings that involve freezing are the most detrimental to the durability of the mixture and are worth to be in consideration for including in the specifications, especially of places located in dry-freeze

and wet-freeze environmental zones. Additionally, it is suggested to reevaluate the aged threshold for the requirements and to contemplate the option of combining the aging procedure with some other procedure (i.e. freezing or moisture) to contemplate better the consequences of aging in PGA mixtures.

- In general the 1.5 in thickness leads to the highest Cantabro loss, followed by the 2.5 in thickness and finally the 4.5 in thickness leads to the lowest Cantabro loss. . In average reducing the thickness from 2.5 in to 1.5 in increases the Cantabro loss by 52.2%; while decreasing the thickness from 4.5 in to 2.5 in increases the Cantabro loss by 32.8%. This suggests that the relationship between thickness and durability does not follow a linear relationship and further studies must be done to determine optimum thickness based on the durability, functionality, structural contribution of the pavement and cost.
- The ranking of the mixtures (from best to worse) were calculated for the Cantabro, HWTT, SCB and IDT/TSR. The results are presented in Table 25. It seems that the Cantabro and SCB test are mostly susceptible to the binder type in which the addition of rubber did represent and improvement in this case. The HWTT and IDT/TSR seem to be more affected by the aggregates and the addition of fibers in the PGA mixtures.

Table 25. Ranking according Cantabro, HWTT, SCB and IDT/TSR tests.

Ranking (Best to Worse)	Test			
	Cantabro	**HWTT**	**SCB**	**IDT/TSR**
1	New Jersey	Florida	New Jersey	Florida
2	Florida	Texas	Utah	Texas
3	Utah	New Jersey	Florida	Utah
4	Texas	Utah	Texas	New Jersey

- It seems to be a relationship between the PG grading of the binder and the susceptibility for HWTT. This could be due to the constant temperature of the test at 50ºC without taking into account the PG of the binder. Based on this, and also on the lack of literature referring to rutting in PGA, this test may not be applicable for capturing PGA performance.

- The SCB test could be adequate to represent the elasticity of PGA however the analysis of this test is disregarding of some of PGA materials (e.g. asphalt rubber) and some revision of the criteria proposed by NCAT must be done.
- Taking into account that PGA mixtures are not structural layers and that asphalt mixtures are generally not strong in tension, it is recommended to study whether a combination of the AASHTO T283 conditioning protocols with other tests (i.e. Cantabro loss) could be more adequate for this type of mixture.

6. CONSTRUCTION AND MAINTENANCE CONSIDERATIONS

Construction and maintenance of PGA mixtures play a key role to guarantee adequate functionality and durability throughout the service life. This chapter describes general construction and maintenance recommendations in order to optimize PGA performance. The information was gathered from a review of state specifications and the results of the survey described previously.

6.1 Construction Considerations

PGA mixtures use several of the construction practices that are commonly utilized with DGHMA. However, the characteristics of this type of mixture require some modifications to the process. This section describes the general aspects in terms of production, transportation, placement, compaction, finishing and quality assurance in the construction of PGA structures. An important source of information for this section was a report authored by Cooley Jr. et al. (2009) that presents a comprehensive review of the construction and maintenance practices and considerations for PGA.

6.1.1 PGA Production

The production of an asphalt mixture is divided into two processes: 1) plant production and 2) mixture production. Plant production refers to the handling of all the materials used in a PGA mixture (i.e. aggregates, binder, stabilizing additives etc.) while mixture production refers to the blending and storage processes of the final product. Considerations for both processes are described in this section.

6.1.1.1 Aggregates

Aggregates used in the production of PGA mixtures must be stockpiled properly. Stockpiles should be built on clean, stable and sloped surfaces separated from each other (U.S. Army Corps of Engineers, 2000). Some states specify that the aggregates should be stockpiled for a period of time to allow drainage of the free moisture from the mixture. For instance, Arizona and Mississippi specify a maximum moisture content of the aggregates of 0.5% by total weight. This low moisture content is important to control the mixing temperature, regulate the asphalt content and to produce a more homogenous mixture (Alvarez et al., 2006). States such as Virginia require dry storage for the mineral filler with a waterproof cover over the stockpile at all times. It is recommended that the equipment used to stockpile the material place it in separate batches each no larger than a truckload. This helps prevent the material from running down the slopes and creating segregation as shown in Figure 86.

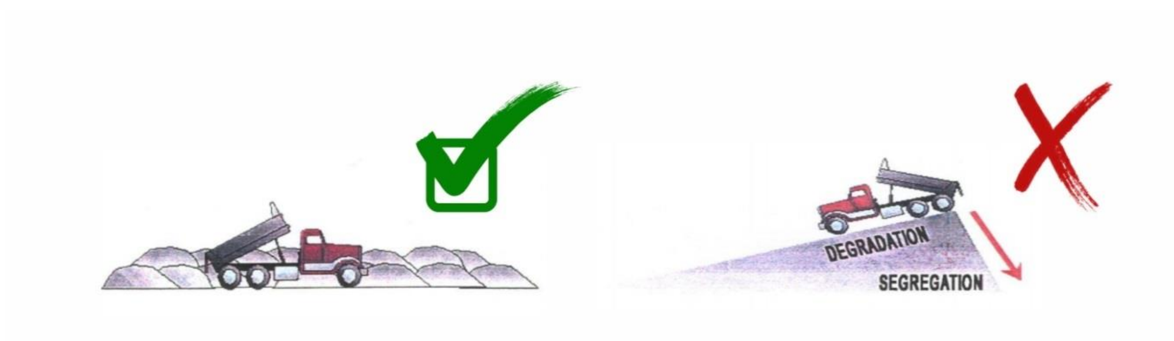


Figure 86. Stockpiling suggestion (modified after Kildow, 2014)

6.1.1.2 Asphalt Binder

The process of handling asphalt binder is similar to that used in the production of DGHMA. Additional storage tanks are required for modified binders (Cooley Jr. et al., 2009). There is a need to carefully follow the binder manufacturer's instructions in handling modified asphalt. Moreover, asphalt rubber is commonly used in PGA mixtures, and there are two types of processes for this type of modification: the dry method and the wet method. The wet method consists of mixing the crumb rubber into the hot asphalt binder and blending or digesting the rubber in the asphalt. The mixing requires special blending equipment as shown in Figure 87 and usually takes about 45 minutes to complete (Wright Asphalt Products Co., 2012). The dry method is not recommended for PGA mixtures.

Figure 87. Portable crumb-rubber blending plant.

6.1.1.3 Additives

The use of stabilizing fibers in PGA mixtures represents a major modification in the production of the mixture compared to standard DGHMA, although fibers are frequently used in the production of SMA mixtures. Stabilizing additives to reduce draindown in the mix include mineral fibers that are typically applied at a dosage rate of 0.4 % of the total weight of the mixture, and cellulose fibers that are typically added at a dosage rate of 0.3 %. Fibers are available in two forms: loose and pelletized. Pelletized fibers use a binding agent which may be a part of the asphalt binder of the mixture or a wax (Cooley Jr. et al., 2009).

In batch plants, loose fibers are usually introduced in plastic bags that are added into the weigh hopper above the pugmill in the dry mix cycle (Figure 88(a)). The bags usually melt with the mixing temperatures and can be added manually or automatically. Another method to add loose fibers is through a blower (Figure 88 (b)). The machine fluffs the fibers in large paddles to a certain density, and blows the required quantity to the weigh hopper or the pugmill at the appropriate time. This procedure can be also used in drum plants just by adding the fibers into the drums within one foot upstream of the asphalt binder line, so that they do not become evacuated to the baghouse (Kandhal, 2002).

Pelletized fibers are used in batch and drum plants (Kandhal, 2002). The pellets come in a bulk form and are placed into the pugmill or the drum with the help of a calibrated conveyor belt or through the reclaimed asphalt pavement (RAP) collar, where they can be mixed with aggregate before the binder is added (Figure 88 (c)). They contain a specific amount of binder that helps in the fiber’s blending process. Although this amount of binder is not significant, it is recommended that it be accounted for when computing the total binder content of the mixture (Kandhal, 2002). The pellets are introduced and mixed with the heated aggregates; the high temperature of the aggregates allows the binder contained inside the pellets to become fluid and facilitates the mixing of the aggregates with the fibers (Cooley Jr. et al., 2009).

(a) **(b)**

(c)

Figure 88. (a) Loose fibers, (b) Blowing machine for fibers (Wiggings, 2015) and (c) Collar for distributing fibers (Mcdaniel, 2015).

Independent of the selected method, all standards emphasize the importance of proper calibration to ensure that the correct quantity of fiber is added to the mixture. If the fiber content is not controlled, parts of the mixture could get dry and unworkable, and the surface could contain fat spots as a consequence.

6.1.1.4 Mixture Production

The first step in the production of PGA mixtures is to check the calibration of the plant, especially of the systems delivering any additives. The mixing temperature should be selected according to the asphalt binder grade according to NAPA publication EC-101, *Best Management Practices to Minimize Emissions During HMA Production* (National Asphalt Pavement Association, 2000). It is important to ensure that the mixing temperature allows sufficient time for the transportation, placing and compaction of the product (Cooley Jr. et al., 2009); but overheating the binder must be avoided to minimize the possibility of premature aging. Normally, common DGHMA mixing temperatures or a little higher value are used for PGA mixtures. South Carolina, for example, specifies temperatures between 325°F (163°C) and 350°F (177°C) and Virginia from 300°F (149°C) to 330°F (166°C). It is imperative that the mixing temperature is maintained through the production process or there is a risk of problems occurring. If the temperature is increased, draindown and premature distresses due to rapid oxidation will arise; while if it is decreased there is a higher potential of moisture damage, delamination and raveling.

The mixing time in the production of PGA mixtures should be greater than that of regular DGHMA both in the dry and wet cycles in a batch plant (Kandhal, 2002). This additional time guarantees a proper dispersion of the fibers in the dry cycle and a uniform coating of the aggregates in the wet cycle. According to Brown and Cooley Jr. (1999) the additional mixing time in batch plants should be from 5 to 15 seconds in each cycle. For drum plants, the asphalt binder injection line should be located to assure a better mixing of the pellets with the aggregates. Cooley Jr. et al. (2009) also suggest inspecting the mixture visually and, if the fibers are not being distributed properly, the time should be adjusted or the plant speed slowed in order to guarantee a proper mixing process.

According to general experience, PGA mixtures should not be stored for long periods of time because the high temperatures could lead to potential draindown problems (Kandhal, 2002). In fact, it has been suggested that PGA mixtures be stored for 2 hours or less to avoid negative effects (Cooley Jr. et al., 2009). California, for example, specifies a maximum of 2 hours for a PGA mixture being stored in silos (CalTrans, 2006), while a PGA study in Washington suggests a maximum of 4 hours (Anderson et al., 2013).

6.1.2 Transportation

The main concern for transporting PGA mixtures is maintaining the temperature until it arrives at the construction site. This due to PGA losing temperature faster than DGHMA (King Jr. et al., 2013). To control temperature loss, agencies have limited haul times, haul distances, haul practices or minimum arrival temperatures. Cooley Jr. et al. (2009) found that most agencies use the minimum arrival temperature requirement, which ranges from 225°F (107°C) to 300°F (149°C), although one agency has a maximum hauling time of 1 hour, and another specified a maximum hauling distance of 50 miles (80 km).

Haul trucks for PGA mixtures share the same characteristics as those for DGHMA. Trucks need to have clean and smooth beds, and a release agent should be applied to the bed to coat and prevent sticking. Any excess release agent should be drained. During transport, the material on the surface cools quicker and may form a hardened crust. In order to prevent crusting, the trucks should be covered with a tarp during transportation as a minimum requirement (Huber, 2000). A more aggressive way to prevent crusting is to use insulated truck beds or a “heated dump body” that heats the PGA mixture during hauling (Cooley Jr. et al., 2009).

Adequate transportation coordination and maintaining a balanced production that keeps pace with paving operations will help guarantee successful construction and prolonged service life. If there are too few trucks with PGA material, the paver will be forced to stop, which may produce bumps and cold spots. However, if there are too many trucks at the same time, some of the mixture will cool down, promoting the formation of cold lumps (Kandhal, 2002; National Asphalt Pavement Association, 1996).

6.1.3 Placement

6.1.3.1 Underlying Surface Preparation

The surface preparation of the underlying pavement is crucial to the performance of the PGA mixture. DOTs have reported using PGA layers on top of DGHMA, WMA and concrete pavements. As mentioned, PGA mixtures are placed above impervious layers on most roadways (Lefebvre, 1993). It is imperative to repair existing distressed areas and seal all existing cracks to minimize the infiltration of water. The underlying surface needs to be clean, dry and free of any debris or deleterious materials. A smooth profile of this layer is also required which may be achieved by either micromilling or milling to a depth ¾” deeper than the PGA thickness. However, it is important that the bottom of the PGA mixture be at the same level as the surface of the shoulder. Figure 89 presents a picture of a milled surface.

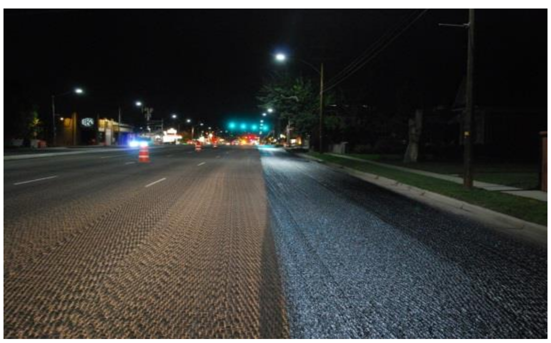


Figure 89. Milled surface.

A uniform tack coat is required to ensure a good bond between the existing surface and the PGA mixture as well as fill and seal the surface voids. PGA mixtures usually need higher tack coat rates compared to DGHMA since PGA mixtures are coarser than DGHMA and the point of contact of aggregate particles to the existing surface has less surface area which affects the bond between the layers (Tran, 2015). According to the FHWA (1990), the tack coat should be a slow-setting emulsion at a dosage rate of 0.05 to 0.10 gallons per square yard. However, others like Ruiz et al. (1990) suggests rates of 0.11 to 0.13 gallons per square yard. DOTs recently surveyed have reported using emulsified asphalt at rates between 0.05 and 0.15 gallons per square yard depending on the condition of the existing surface. Some states reported using hot asphalt binder PG 64-22 at rates of between 0.06 and 0.14 gallons per square yard.

6.1.3.2 Weather Requirements

The air, surface temperature and wind speed at the time of construction are very important to the success of the PGA mixture. PGA mixtures should not be placed in cold, rainy or excessively windy conditions , as this could lead to failures associated with the loss of bond between PGA mixtures and the underlying surface and raveling (CalTrans, 2006; Cooley Jr. et al., 2009). Most of the standards and specifications have either date or temperature requirements; Table 26 presents some typical values for those specifications. Wind speeds are not specified by agencies, but convection cooling can occur very rapidly on windy days when the temperature may seem adequate.

Table 26. Weather requirements for PGA placement.

State	Minimum Air Temperature (°F)	Minimum Surface Temperature (°F)	Dates
Arizona	65	85	-
Alabama	40	40	-
California	70	60	-
Georgia	55	-	-
Mississippi	55	-	-
Nebraska	65	65	-
Nevada	55	-	-
New Mexico	60	-	-
North Carolina	-	-	Do NOT place between October 31 and April 1 of the next year.
Oklahoma	-	60	-
Oregon	-	50	Place between March 15 and September 30.
South Carolina	60	-	Place between March 1 and October 31 inclusive.
Tennessee	55	-	Place between April 1 and November 1.
Texas	-	Infrared paver:50 Not Infrared Paver:70	-
Virginia	50	50	-
Wyoming	50		Place between June 1 and September 15.

6.1.3.3 Material placement

PGA mixtures may be transferred from the hauling trucks directly to the pavers. However, a remixing material transfer device is recommended in order to achieve a consistent temperature and allow continuous paver operation for a smoother surface (Kandhal, 2002). In the process of placing the mixture, conventional asphalt pavers are used. The paving speed is dictated by the ability to compact the mixture. Normally, no more than two or three passes of a steel-wheel roller are required. Paver stops and re-starts should be eliminated to the extent possible (Cooley Jr. et al., 2009). Raking and handwork should be avoided because PGA mixtures tend to be stiff and very sticky (Lefebvre, 1993).

6.1.4 Compaction

While compaction in DGHMA is a process conducted to make the mixture impermeable, in PGA mixtures this process is conducted for seating the aggregate and adhering the mixture to the tack coat (Cooley Jr. et al., 2009). For these layers, steel-wheeled rollers are used. Pneumatic-tired rollers are not recommended because the tires tend to pick up the aggregate and close the surface pores, thus reducing drainage capacity (Cooley Jr. et al., 2009; Huber, 2000).

Vibratory rollers should only work in the static mode. Steel-wheeled rollers should weigh between 8 and 12 tons as heavier rollers could lead to aggregate breaking and result in a loss of drainability and early failures while lighter equipment would not produce adequate seating of the aggregate (Huber, 2000). Finally, according to the specifications of many states between two and four passes are sufficient to compact the mixture (Nebraska DOT, n.d.; Oregon DOT, 2008; Wyoming DOT, 2010).

It is a best practice to wet the roller drums with a solution of soap and water to prevent adhesion of the PGA mixture to the roller. Nicholls (1997) recommended that the roller speed should not exceed 3 miles per hour (5 km/hr.). It is critical that the roller follow the paver closely, beginning as soon as possible after placement (Brown et al., 2002). Kandhal (2002) recommends a 50 ft. (15 m) maximum distance between the paver and the roller to guarantee that the material is still hot and workable. Some states specify minimum pavement temperatures for completing the compaction. Arizona, for example, requires a minimum of 200°F (93°C), while Tennessee and Virginia specify a minimum temperature of 185°F (85°C), and Oregon specifies 180°F (82°C).

6.1.5 Joints and Pavement Markings

Longitudinal joints are constructed by placing the mixture between 1.5 mm (1/16 in) and 3 mm (1/8 in) above the previously placed and compacted lane (Cooley Jr. et al., 2009; Kandhal, 2002), and they should always be located out of the wheel paths (Estakhri et al., 2008). To prevent excessive overlapping, it is essential that the edge of the screed or extension follows the joint precisely (Kandhal, 2002). Longitudinal joints should not be tack coated, because this would reduce the drainage capacity of the pavement (Kandhal, 2002).

Transverse joints are placed against PGA mixtures that have already been placed. The construction starts positioning the screed 1 foot (30 cm) behind the joint and laying the screed flat on the already laid PGA mat (Kandhal, 2002). Then, the joint must be cross rolled by a steel wheel roller (Brown and Cooley Jr., 1999). Due to the difficulty of constructing these joints, they should be avoided when possible.

Some problems have been reported with pavement markings that use conventional thermoplastic paints. This material can heat the asphalt binder on the PGA surface and cause localized draindown (Sholar et al., 2005). To overcome this, Lee et al. (1999) suggested the use of fully recessed thermoplastic markings, since they have shown adequate performance in terms of durability and reflectivity.

6.1.6 Quality Control, Quality Assurance and Acceptance.

Quality control (QC) and quality assurance (QA) tests are performed to ensure the control of the production and placement, and to verify the delivery of the specified material. Within the plant, samples are taken randomly to verify some mixture properties. For example, in New Mexico the specification requires one sample per 250 tons of aggregate for the first 2000 tons, and then every 500 tons, to verify gradation and fractured faces of the aggregates. Nebraska specifies a sample every 750 tons of mixture to evaluate G_{mb}, G_{mm}, binder content and the gradation of the mixture. In Florida, the mixture produced at the plant is accepted if it fulfills gradation, asphalt content and other volumetric requirements.

Most DOTS reported that there is more a verification of the mix design than a QC/QA evaluation, since it is done primarily at production. However, during the construction stage, sometimes cores are obtained and used to check gradation, binder content and specific volumetric properties. The main difference with DGHMA surfaces is that a density evaluation is not typically done. It is recommended that some quality assurance testing be done to check air void content and/or the permeability to ensure drainability and durability.

Some countries like Spain conduct density tests to verify the AV of the mixture (Ruiz et al., 1990). Spain and Japan both perform permeability tests, and Argentina uses the Cantabro Abrasion loss test in their quality control process (Bolzan et al., 2001; Ruiz et al., 1990). When construction is completed, smoothness testing is commonly performed as for DGHMA (Cooley Jr. et al., 2009).

Some recent research has studied the relationship between air void content and durability and functionality and found that more air voids imply more functionality but at the same time less durability (Hernandez-Saenz et al., 2017). Hence, it is recommended as apart of QA/QC to establish a minimum permeability requirement and a maximum permeability requirement to guarantee the correct functionality of the mixture while helping preventing durability issues. It is also suggested to verify this value during field construction. Having a portable permeameter could facilitate this task.

6.2 Maintenance Considerations

Several studies have been conducted on effective maintenance activities for PGA mixtures. However, there are limited standardized processes, and in the United States only a few agencies have specific requirements for these strategies. Results from the survey indicated that from all the states that reported current use of PGA, only California and Oregon include maintenance requirements as part of their practices. Appropriate maintenance of PGA mixtures is important to the longevity of the pavement. These maintenance processes are substantially different to those used for DGHMA. This section aims to describe typical maintenance strategies PGA mixtures, giving a special emphasis to winter maintenance activities.

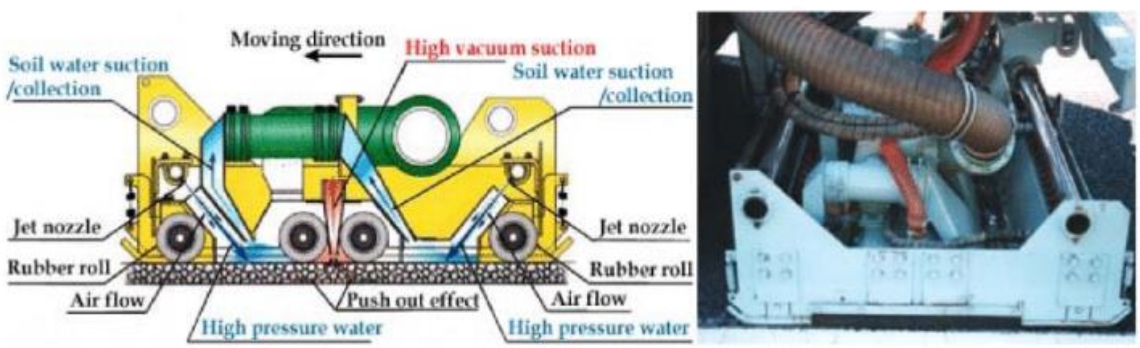
6.2.1 General Maintenance

Clogging is recognized as the main problem affecting the functionality of PGA mixtures. Hence, the general maintenance performed is to clean the PGA mixtures, to restore the permeability of the mixture. The cleaning is often difficult, and it is suggested to clean the structure while it is still permeable (Isenring et al., 1990).

Kandhal (2002) stated that there are three methods used to perform general clogging maintenance: 1) cleaning with a fire hose, 2) cleaning with a high-pressure cleaning device, and 3) cleaning with a specially manufactured vehicle. The cleaning vehicle is manufactured in Switzerland, and it washes the pavement with high pressure water (about 500 psi) and vacuums the water-dirt mixture during each pass (Hiershe and Freund, 1992) as is commonly done for hazardous materials spills on roadways. An example of this type of truck is shown in Figure 90(a). Another machine in Japan uses the same principle of washing and vacuuming (in this case with a pressure of 125 psi). The principal difference is that this machine uses special nozzles that cause water cavitation (Abe and Kishi, 2002). It has been reported that the high-pressure device is the most effective strategy to prevent clogging; a conclusion that was obtained based on permeability tests (Hiershe and Freund, 1992). Figure 90 (b) and (c) illustrates the novel machine and the designed high-pressure water ejection system with high vacuum suction systems.

(a)

(b)



(c)

Figure 90. (a) Cleaning truck (Pine Hall Brick, 2011); Truck-mounted function recovery machine (Abe & Kishi, 2002) and (c) High-pressure ejection and vacuum systems (Abe & Kishi, 2002).

None of the cleaning practices mentioned are very common or standardized in the US, and some of them are very costly. More states are starting to try these methods. For instance according to an additional email survey, Georgia and New Jersey reported vacuuming and sweeping PGA surfaces. Despite this, most of the states in the United States use PGA mixtures in roads with high volumes of traffic and high speeds which is assumed to clean the surface by the suction produced from rolling tires on wet surfaces.

6.2.2 Preventive Maintenance

The use of rejuvenators and fog seals are the most common preventive maintenance practices used worldwide to extend the service life of PGA mixtures. Rejuvenators and fog seals seal the pavement against air or water, slowing the oxidation of the mixture and thus preventing raveling, stripping and any other damage inside the pavement structure. Additionally, these products not only reduce but also reverse the aging of the surface (Brownridge, 2010). In general they are a low cost treatment that extends the life of PGA mixtures. The use of preventive maintenance is becoming more standardized and used recently due to the benefits to service life that it provides. Figure 91 shows the exact point of life when this type of maintenance must be performed in order to prevent several failures and more expensive maintenance costs.

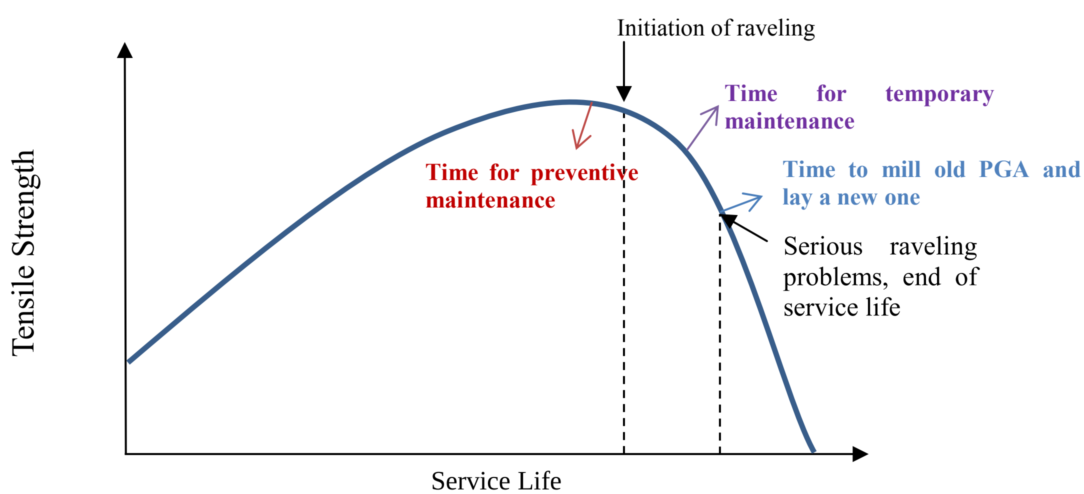


Figure 91. Maintenance techniques optimum timing. Modified after (J. Voskuilen, 2006).

In PGA, fog seals and rejuvenators have specifically demonstrated an improvement in raveling resistance (Y. Zhang, van de Ven, Molenaar, & Wu, 2012, 2016). The application rate needs to be carefully controlled so that the surface pores do not become plugged. The main difference between rejuvenator seals and fog seals is the chemical make-up. Rejuvenators are made from the lighter components of asphalt, and fog seals are diluted asphalt emulsions (Qureshi, Tran, Watson, & Jamil, 2013).

Fog seals are diluted bituminous emulsions sprayed at elevated temperatures. In general slow setting emulsion consisting on a 50:50 mixture of asphalt emulsion and water are used in a two-pass application at a dosage rate of 0.05 gallon per square yard (FHWA, 1990). Recently, a new additional step was introduced in the Netherlands where after the spraying truck there is a rubber scrapper and a steel brush that make the sprayed material flow into the pores of the PGA mixture. Additionally, the sprayed material is further blown with hot air to guarantee the full internal coating of the structure(Y. Zhang et al., 2016). It is recommended that fog seals be applied every 5-10 years. Recently, emulsions with polymer have been used at a rate of 0.2 gallons per square yard with some states reapplying this material every 3-4 years, while rejuvenators are usually applied at an application rate of 0.1 gallons per square yard. After the application of fog seals or rejuvenators, the frictional properties are reduced during the first month until the traffic wears the sealer off the surface. The macrotexture has been reported to not be affected by this treatment (Rogge (2002)).

New technology is also emerging in order to combat raveling, specifically the use of high frequency induction curing. The process basically consists of adding a steel wire mesh/Hot Dip Galvanized (G.I.) wire to a PGA mixture to make the road electrically conductive and suitable for induction heating. When microcracks are going to occur in the asphalt mastic (i.e. bitumen and fines), the wire mesh/GI is heated externally to cure the binder and prevent the formation of major cracks and thus raveling (Neogi et al., 2016).

6.2.3 Winter Maintenance

Due to the porosity, open structure and texture of PGA mixtures, this type of mixture has some disadvantages compared to conventional DGHMA, especially during winter. PGA mixtures tend to freeze more rapidly, clog from sand and debris, allow deicing chemicals to fall through the surface and retain snow and ice for longer periods (Fay and Akin, 2013). Moreover, because of the high air void content of the structure, heat conductivity is lower (about 40% to 70% of DGHMA), which causes PGA structures to have lower temperatures than regular DGHMA, causing ice and snow to accumulate faster, and produce slow thaw processes and more rapid refreezing processes (Yildirim et al., 2006). However, some of the main advantages reported for PGA mixtures during winter are that the probability of ice formation, also known as “black ice”, is lower on wet surfaces and in wheel paths, that the surface reduces glare and spray, and that friction is not only adequate but on occasion even better than DGHMA surfaces (Fay and Akin, 2013).

The most common winter maintenance procedures can be classified into two groups: 1) those performed before snow and icing are present, and 2) those performed after snow and ice form. The first group represents “preventive” maintenance, which is known as anti-icing treatments while the second ones are named de-icers. Even though the products used as anti-icers and de-icers are basically the same, the dosages and results are different.

6.2.3.1 Anti-icers

Anti-icing is performed to prevent the formation of ice and snow in the roads and to provide safer conditions, especially before storms. The materials used in these activities include salting, liquid chemicals and pellets.

Salting is currently the most used anti-icing procedure. It includes the use of calcium or magnesium chloride in many forms (i.e. rocks, pre-wetted, brine and solid) and their combination with other chemicals as described subsequently. In general, salting may only be used on dry surfaces before precipitation, at temperatures lower than 14°F (-10 °C) (Yildirim et al., 2006). The salt used for ice melting must have the following specific characteristics: it should not have a high grade of purity, unlike industrial and alimentary salts, and it should be in the form of crystals measuring less than one millimeter (Coldlay, 2011). In some European countries such as the Netherlands, pre-wetted salt is used on PGA mixtures so that the crystals adhere to the sides of the voids near the surface. This improves the efficacy of salt on these surfaces.

The application of liquid chemicals is used to prevent frost and black ice. These materials can be placed uniformly and fast. The most popular liquid chemicals used are magnesium chloride ($MgCl_2$)/salt brine, calcium magnesium acetate (CMA), calcium chloride ($CaCl_2$)/salt brine and potassium acetate (KAc) (Yildirim et al., 2006). Also, some glycol-based fluids and potassium acetate-based fluids have been utilized on airfield pavements (Dehdezi & Widyatmoko, 2015). A difficulty associated with these liquids, however, is that they drain quicker through PGA mixtures. Hence, more material is required, which results in higher costs and generates environmental concerns (Yildirim et al., 2006). This material is usually introduced in pellets during the mix production and activated under the action of traffic (Fay and Akin, 2013; Keneddy, 2015).

Recently**,** new technology has emerged providing products that are more environmentally friendly. For example, there is an ant-icer that lowers the freezing point of the water and avoids or delays the formation of ice crystals adhering to the pavement surface. It is introduced during the production of the asphalt mixture for the wearing layer as a filler from 3% to 5% of the aggregate weight.

Any “preventive” maintenance must be performed at the right time which requires information at the roadway location (Yildirim et al., 2006). In the Netherlands, roads have sensors that measure temperature, humidity, dew point and the current presence of salt (van der Zwan, 2011). This information is used as part of a system that combines these data with weather forecast in order to determine the exact time when salting is needed.

6.2.3.2 De-icers

De-icing is performed after snow and ice formation. Different salt products in different forms are used for de-icing. One main disadvantage of PGA mixtures is that they need more frequent application and greater quantities of deicing products as compared to DGHMA. Italy reported a 50% increase in the use of salt in de-icing of PGA mixtures (FHWA/AASHTO International Technology Scan, 2004). However, due to the open structure of PGA mixtures, common dry salting tends to be of little benefit because it penetrates the structure; thus, pre-wetted salts are normally used since they better adhere to the road surface and work faster and for longer periods of time. In this context, rock salts are presented as another alternative for these de-icing purposes (Yildirim et al., 2006).

To prevent the de-icing material from penetrating the structure, some states such as Oregon have suggested the study and use of organic de-icers with a higher viscosity and electrostatic charge technology that could be capable of improving the bonding of de-icers on the surface of the layer (Huber, 2000; Rogge, 2002). The principal winter maintenance salting products utilized in Europe and Japan and their typical dosages for de-icing products are useful when snow and ice are bonded to the surface and need to be removed. However, it is important to keep in mind that road safety conditions are not the best when de-icing products are used, because the surface tends to become slippery (Fay and Akin, 2013).

In order to improve surface friction conditions during winter, many states and countries have used sanding as a quick alternative; however, sand or any other abrasive product is not recommended since they can generate clogging, which will in turn produce a reduction of all the major benefits of PGA mixtures (Yildirim et al., 2006) .

6.2.3.3 Best Anti-icer and Dei-icer Practices

Authors like Padmos (2002) state that there is no definitive solution for winter maintenance of PGA mixtures since these layers can be treated differently depending on the location of the project and, especially, based on the experience of maintenance personnel. In general it can be concluded that salt-based solid products are the most successful practices in terms of PGA mixtures. Figure provides some general dosages of the most common products that are used for both anti-icing and de-icing processes.

Figure 92. Best Winter Maintenance Practices: Anti-icers and De-icers.

6.2.3.4 Alternative Methods

Besides the anti-icers and de-icers, there are some alternative treatments for winter maintenance. For instance, snow plowing is a procedure often used to remove snow for DGHMA. However, PGA surfaces are generally more susceptible to damage; and it has been reported that snow plowing can gouge and damage the pavement (Moore, Hicks, & Rogge, 2001). Fay and Akin (2013) summarize some recommendations obtained from different authors for reducing the damage caused by snowplows, and they concluded that the most effective strategy is either setting the plow blade higher (1 in above the pavement) or waiting until there is a minimum of 2 in of snow accumulated.

Pavement heating techniques are also alternative methods to anti-icing and de-icing products which have mainly been used on airfield pavements. An example of this method is the Heated Pavement System. The system includes a base layer consisting of copper cables, installed perpendicular to the runway surface, embedded in a 2-inch thick conductive material composed of a mixture of synthetic graphite and asphalt as shown in Figure 93 (Dehdezi & Widyatmoko, 2015; Superior Graphite, 2010). Electricity passing through the conductive layer generates enough heat to maintain the temperature of the pavement surface slightly above freezing, preventing ice from forming and melting any snow that may accumulate. This product is ideal for use with PGA surfaces since the melted snow will penetrate the open structure providing an efficient winter maintenance method. Also, the product is adjustable to meet varying degrees of snow, ice, wind velocity and temperature. However, the cost of these systems usually limits their application to particular pavement features such as intersections and curves.

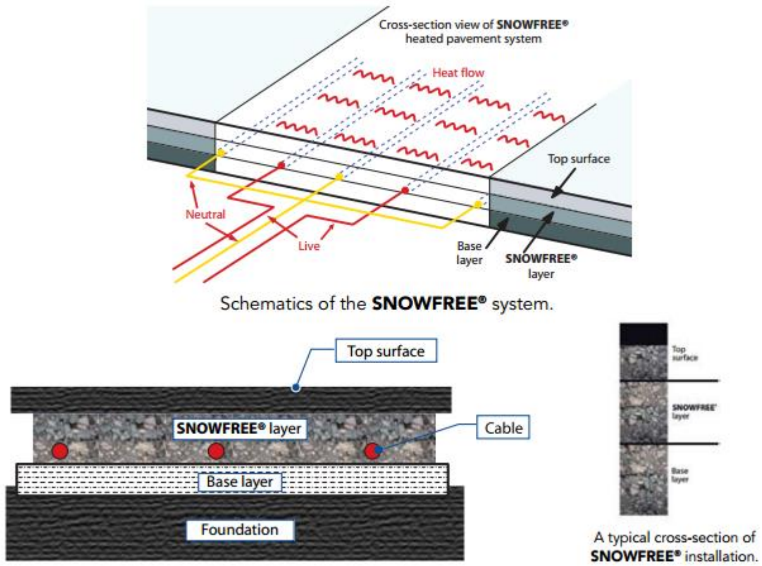


Figure 93. Heated Pavement System (Superior Graphite, 2010).

6.2.4 Corrective Maintenance

When PGA surface layers have damage such as potholes and delaminated areas, corrective maintenance is required. The main corrective maintenance activity is based on milling and inlaying new material in the damaged area (i.e., patching). This process is performed using either PGA or DGHMA mixtures. To decide which kind of material should be used, the FHWA advises considering the continuity of drainage by ensuring that the patched area is small and the flow of water around of the patch won't create excess surface runoff (FHWA, 1990). The DGHMA should be placed to provide a "diamond shape", as illustrated in Figure 94, in order to facilitate the flow of the water along the patch and reduce the wheel impact at the joint (CalTrans, 2006; Pucher et al., 2004). Alvarez et al. (2006) referenced a British specification in which a maximum DGHMA patching size of 1.64 ft by 1.64 ft (0.50 m by 0.50 m) is indicated.

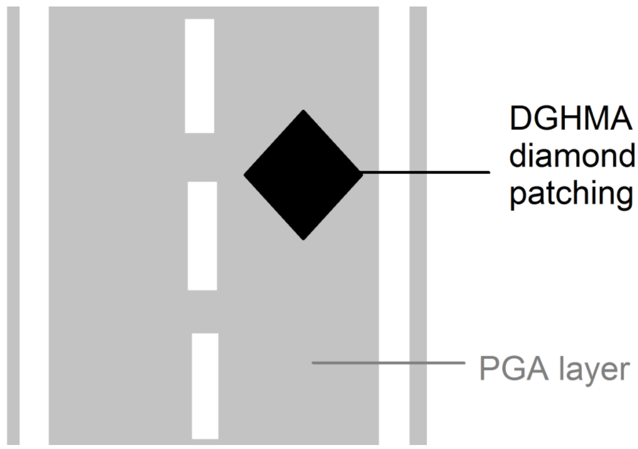


Figure 94. Schematic of diamond DGHMA patching.

The patching process using PGA mixtures has also been reported. Rogge (2002) stated that in Oregon, PGA patching is only used when the quantity of material is enough to justify this activity. If only a small quantity of material is needed, they prefer to use DGHMA to repair the area. According to Cooley Jr. et al. (2009) when the patching is done with PGA mixture, the tack coat should be applied only in the vertical faces of the existing pavement, and very lightly, in order to assure the flow of water through the patch.

Another common corrective maintenance practice is crack sealing. Transverse and longitudinal cracks may appear in PGA mixtures. Transverse cracks are usually sealed without major problems. However, longitudinal crack sealing may be problematic because the sealer could impede the transverse flow of the water within the structure (Cooley Jr. et al., 2009). Some potential solutions for longitudinal cracking described by Cooley Jr. et al. (2007) include milling off the PGA mixture in a narrow strip and placing an inlay with the material or, if required, rehabilitating the entire pavement.

6.2.5 Rehabilitation

Lefebvre (1993) indicates that minor and major rehabilitation are needed for pavements with PGA surfaces. Minor rehabilitation refers to small repairs associated with localized damage or distresses, while the rest of the pavement remains in a good state. This is also referred to as corrective maintenance practices. Major rehabilitation is when the entire layer is damaged and needs to be fully repaired. The principal techniques used in this case are the replacement of the entire layer, an overlay or refurbishment (Cooley Jr. et al., 2009). The replacement of the layer is usually performed by milling the entire layer and by placing a new surface, using either PGA or DGHMA materials. Refurbishment uses the same principle of completely removing the layer but the difference in this case is that this practice includes in-situ recycling. Recycling in the Netherlands is done by collecting the reclaimed PGA mixture and by combining it with new materials in a hot-mix production facility (Lefebvre, 1993). However, the results of this practice have not been as successful as expected.

The other option for major rehabilitation is overlaying the existing surface. Moore et al. (2001) reported overlaying flexible pavements with PGA layers with PGA or DGHMA materials; however Kandhal (2002) stated that a DGHMA overlay should not be used because water will be trapped in the PGA layer, causing premature deterioration of the pavement structure.

7. FINDINGS, CONCLUSIONS AND RECOMMENDATIONS

PGA mixtures are utilized as thin surface layers on top of conventional pavements. They are characterized by gap-graded aggregates, higher AV (between 15% and 20%), stiffer binders and higher asphalt contents compared to conventional DGHMA mixtures.

PGA mixtures present several advantages that can be classified into three groups: 1) safety improvements, 2) driving experience enhancements and 3) environmental benefits. In terms of safety, PGA mixtures are effective in reducing hydroplaning and splash and spray while enhancing road marking visibility and friction resistance. Driver experience enhancements include higher average speeds and traffic capacity, improvement in road smoothness and reduction in fuel consumption. Finally the environmental benefits include noise reduction and improved runoff quality. The safety benefits are the main reason these mixtures are used in the United States, while in Europe the environmental benefits are the most valued characteristic.

Some disadvantages are also related to the use of PGA mixtures including performance issues in terms of durability (resistance to distress and failure) and functionality (ability to maintain beneficial properties through time) of these layers. Increased costs and additional maintenance requirements also present some difficulties related to the use of this type of material.

The performance of PGA is usually evaluated in terms of durability and functionality. In terms of durability, the principal distress reported in PGA mixtures is raveling; but delamination, rutting due to studded tires and cracking have been also reported. Average service life is typically between 8 and 10 years. In terms of functionality, the main challenge is clogging by sediments deposited on the pavement surface, which reduces the permeability and the noise control capacity of the material significantly. In addition, aggregate crushing and draindown have also been observed to interfere with functionality.

Mix design methodologies of PGA mixtures start with adequate material selection. The information available in the literature stresses the importance of evaluating the angularity, abrasion potential, polish resistance, cleanliness and the morphological properties of the aggregate phase. In terms of asphalt binder, some recommendations include using a material that has a performance grade (PG) that is two grades stiffer than the normal binder used for DGHMA mixtures in the same environment. Stabilizing additives, principally mineral or cellulose fibers, are another important component of these mixtures since they prevent draindown effects. In order to obtain the Optimum Binder Content, four principal methodologies are utilized: 1) methods based on compacted specimens, 2) methods based on absorption of the predominant aggregate fraction, 3) methods based on visual determination and 4) a combination the three other methods. The first one is the most common method currently used in the United States.

PGA mixtures have a typical thickness values between 25 mm and 50 mm and usually its structural capacity is not considered as part of the design of the pavement.

There are important challenges associated with the current use of PGA mixtures. The development of more scientifically sound mix design methods and better quality assurance practices to assure durability and functionality, and the development of standardized maintenance procedures especially in winter conditions are probably the most relevant aspects toward increasing the utilization of this type of mixture.

The objective of this study was to overcome some of the current performance challenges of using PGA mixtures through a functionality evaluation and durability analysis. The idea was to ensure adequate pavement performance and longer service life and thus encourage utilization of this material nationwide. More specifically, this study evaluated the impact of parameters such as AV, thickness and type of mixture on both functionality and durability, identifying key tests for assessing these properties. Four different types of materials from Florida, New Jersey, Texas, and Utah collected from field projects in different environmental zones were utilized. Durability assessment of the mixture was performed via Cantabro loss, HWTT and SCB/IDT tests; functionality was evaluated with permeability tests using TXDOT, NCAT and FDOT permeameters and a noise absorption test using the impedance tube.

This chapter presents the main findings, conclusions and recommendations obtained from this study presented in four different sections: functionality, durability, functionality vs. durability and construction and maintenance.

7.1 Functionality Analysis

The functionality of PGA mixtures with three different pavement layer thicknesses (0.75 in, 1.5 in and 2.5 in) at two AV (design and construction) was evaluated via noise absorption and permeability using plant mixture collected from field projects in Florida, New Jersey and Texas. Permeability was evaluated with portable permeameters developed by TxDOT and NCAT, as well as with an FDOT laboratory apparatus. Noise absorption was evaluated using the impedance tube. The following points summarize the main findings related to functionality:

- The coefficient of permeability obtained with the TxDOT permeameter was on average 72% higher than that obtained with the NCAT permeameter, and 92% higher than that obtained with the FDOT apparatus. Additionally, the NCAT coefficient of permeability was 70% higher than the values obtained with the FDOT apparatus. The relationship between these three permeability methods was relatively consistent; however, permeability requirements for each type of permeameter need further development.
- Thickness of the PGA layer had a statistically significant effect on the coefficients of permeability. In general, the thicker the PGA layers, the higher the permeability. The change between 0.75 in and 1.5 in was less pronounced as compared to that between 1.5 in and 2.5 in. A life cycle cost analysis study is recommended to analyze the impact of changing the thickness of the layer.

- The AV had a statistically significant effect on the coefficient of permeability at $\alpha=0.05$. The higher the AV, the higher the permeability regardless of the type of permeameter or thickness.
- In general, Florida structures had the highest permeability, followed by those in Texas and then New Jersey, regardless of the type of permeameter or AV.
- Laboratory measurements provided higher values of permeability regardless of the type of permeameter or material as compared to field data. On average, laboratory measurements were 46% higher than field measurements.
- Permeability field measurements decreased significantly one year after construction, while the variability of the measurements increased. This could be an indication of densification of the mixture under the influence of traffic or the clogging of the interconnected AV structure.
- The average noise absorption coefficients varied for all structures between 0.45 and 0.65 and did not change significantly with the thickness of the PGA slabs or the AV. All structures analyzed had enhanced noise reduction characteristics as compared to DGHMA.
- Since both functionality and durability of PGA mixtures should be considered to achieve balanced performance, the coefficient of permeability is recommended as the primary indicator of functionality. Data that best captured the influences of thickness, AV and type of material was provided by the TXDOT apparatus, so permeability with TXDOT permeameter is utilized as the functionality parameter for the optimization analysis.

7.2 Durability Analysis

The durability of four different types of PGA layers obtained from field sites in Florida, New Jersey, Texas and Utah was assessed via Cantabro loss, HWTT, SCB and IDT/TSR. The Cantabro loss assessment was performed at both design and construction AV at three different thicknesses (4.5 in, 2.5 in and 1.5 in) while the HWTT, SCB and IDT/TSR were performed on PMPC specimens brought from the field at design AV and 2.5 in in height.

Several conditioning protocols for the Cantabro loss test were also proposed and evaluated. The following points summarize the main findings of the durability assessment:

- AV had a statistically significant effect on the Cantabro loss at $\alpha=0.05$. Construction AV led to significantly higher Cantabro loss than design AV regardless of thickness or conditioning protocol. A mixture that is designed at a specific AV could result in Cantabro loss values between 6% and 50% higher when compacted in the field. Hence, the control of AV is not only vital for functionality issues but also for durability issues. It is important to guarantee minimum AV to guarantee permeability but at the same time maximum AV to reduce the abrasion potential of the mixture.
- According to both analyses (i.e. SGC specimens and slab cores), it is evident that the type of conditioning does have an effect on the Cantabro loss value of the PGA mixture. Conditioning protocols that involve freezing are the most detrimental to the durability of the mixture and are worth considering for inclusion in specifications, especially for locations in dry-freeze and wet-freeze environmental zones. Additionally, it is suggested to reevaluate the aged threshold for the requirements and to contemplate the option of combining the aging procedure with some other procedure (i.e. freezing or moisture) to contemplate better the consequences of aging in PGA mixtures.
- In general PGA with 1.5 in thickness led to the highest Cantabro loss, followed by the 2.5 in thickness and then the 4.5 in thickness. On average, reducing the thickness from 2.5 in to 1.5 in increased the Cantabro loss by 52.2%; while decreasing the thickness from 4.5 in to 2.5 in increased the Cantabro loss by 32.8%. This suggests that the relationship between thickness and durability does not follow a linear relationship and further studies are suggested to determine optimum thickness based on the durability, functionality, structural contribution of the pavement and cost.
- The ranking of the mixtures (from best to worse) was calculated for the Cantabro, HWTT, SCB and IDT/TSR. The results are as follows:
    - Cantabro: New Jersey, Florida, Utah, Texas.
    - HWTT: Florida, Texas, New Jersey, Utah.

- SCB: New Jersey, Utah, Florida, Texas.
- IDT/TSR: Florida, Texas, Utah, New Jersey.

It seems that the Cantabro and SCB test are sensitive primarily to binder type, in which the addition of rubber did represent an improvement. The HWTT and IDT/TSR seem to be more affected by the aggregates and the addition of fibers in the PGA mixtures.

- There seems to be a relationship between the PG grading of the binder and the susceptibility to rutting in the HWTT. This could be due to the constant temperature of the test at 50ºC without taking into account the PG of the binder. Based on this, and also on the lack of literature referring to rutting in PGA, this test may not be applicable for capturing PGA performance.
- The SCB test is promising for characterizing cracking resistance of PGA mixtures; however, the analysis of the data is not possible for some PGA materials (e.g. asphalt rubber) and cracking has not been widely reported in the literature as a primary distress for PGA mixtures.
- Taking into account that PGA mixtures are not structural layers and that asphalt mixtures are generally not strong in tension, it is recommended to study whether a combination of the AASHTO T283 conditioning protocols with other tests (i.e. Cantabro loss) could be more adequate for this type of mixture.

7.3 Functionality vs. Durability Analysis

An analysis was performed in order to evaluate simultaneously the impact of AV on functionality and durability. This analysis included selecting one functionality measurement and one durability measurement for structures that have both of them (2.5 in and 1.5 in thickness) for the optimization and quantification of the impact of changing AV from design to construction for both parameters. From the functionality analysis, permeability was identified as the key parameter, and since the TXDOT permeameter was the one that better captured the effect of thickness and AV on permeability, the functionality optimization parameter selected was the permeability measurement using the TXDOT permeameter. For durability, the Cantabro test best represented the distresses in PGA mixtures better than the other tests evaluated. The conditioning protocol selected was the most standardized (unaged); hence, unaged Cantabro loss was utilized for representing durability in the optimization. However, since the Cantabro loss is a number that represents loss and not resistance to distresses; a parameter termed Cantabro Resistance and defined as (1-Cantabro loss) was utilized for the optimization to help illustrate durability resistance against functionality. Figure 95 and Figure 96 present the durability and functionality parameters selected versus the AV for the three types of materials analyzed in the study (Florida, New Jersey and Texas). In addition to the data for each type of material, there is a regression of the average of the data for both functionality and durability in which the intersection is highlighted.

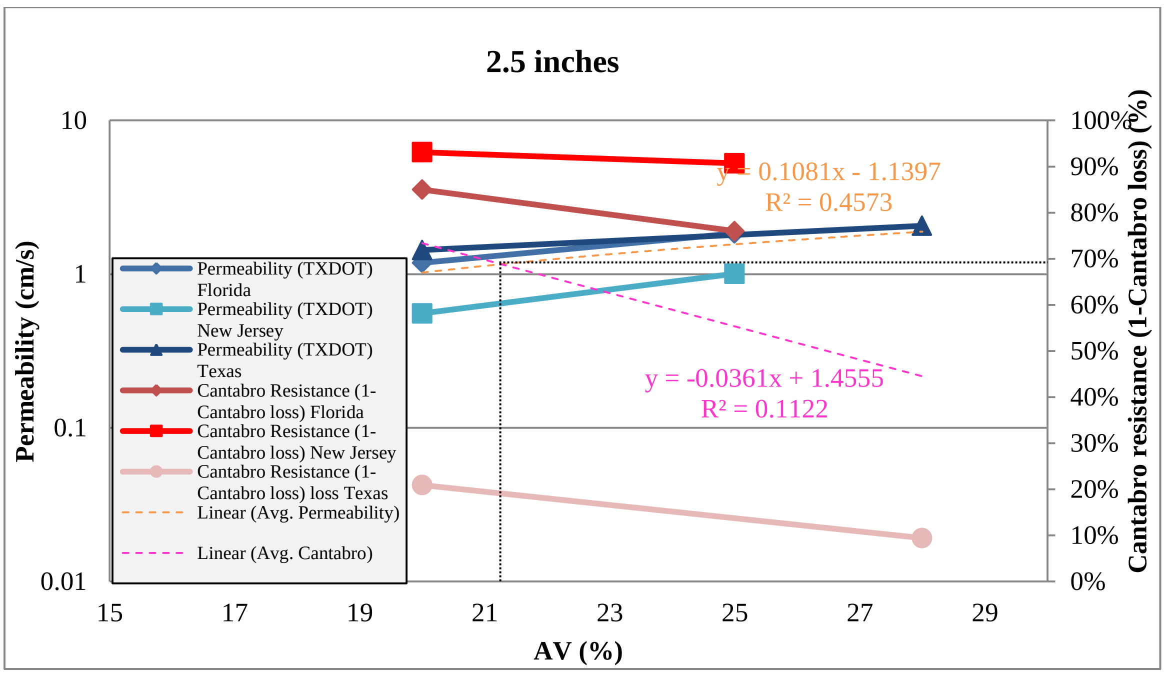


Figure 95. Durability vs Functionality in terms of AV for 2.5 in specimens.

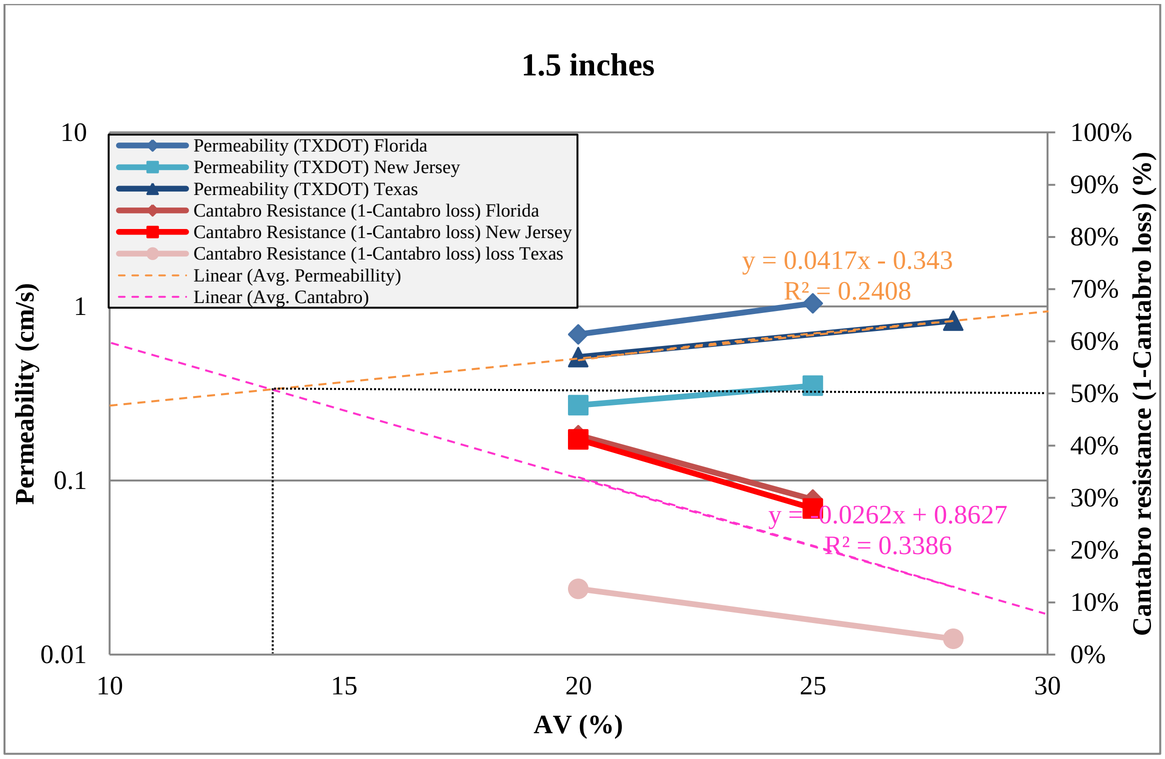


Figure 96. Durability vs Functionality in terms of AV for 2.5 in specimens.

Increasing the AV of PGA mixtures increases the permeability, which is beneficial for functionality, but at the same time the Cantabro resistance decreases, which translates to a greater potential for abrasion related distresses such as raveling and delamination and hence early failure. This behavior is exhibited regardless of thickness or type of material; however, the magnitude of the impact does change with these parameters. For instance, the intersection of the average durability and functionality regression lines for the 2.5 in specimens is around 21% while for the 1.5 in is about 14%. This is a first for this type of analysis and does not mean that that is the optimum AV for every PGA mixture, but it suggests that more strict requirements are needed to control AV in thinner structures and additionally that there must be a maximum and minimum requirement for AV that is around the optimum for each PGA mixture. It is suggested that future research focuses on developing models for optimizing this in terms not only of AV but also thickness, volumetric properties and costs.

Another finding is related to current Cantabro loss thresholds. The Cantabro loss test standard (ASTM, 2003) requires the dimension of the specimens to be 4.5 in height. However, in practice there are no PGA structures with this thickness, and there is no standardized test for testing cores. For the 4.5 in specimens, the maximum Cantabro loss for unaged structures is usually 20%. The optimum point (intersection) of the durability and functionality of Figure 95 and Figure 96 suggests that the unaged Cantabro loss threshold could be 30% maximum for the 2.5 in specimens and 50% maximum for the 1.5 in specimens. These values give a general idea of what represents an acceptable PGA mixture at these thicknesses.

7.4 Revision of Construction and Maintenance Practices

The construction and maintenance of PGA plays a key role in maintaining adequate functionality and enhancing durability of the mixture throughout the service life. Review of the literature and interviews were performed to formulate the following points toward adequate performance of PGA mixtures during their service life:

- Plants used for the production of DGHMA may be used for producing PGA; however, some modifications should be included, principally a system to deliver stabilizing additives which are commonly fibers.
- Mixing temperature should be controlled and maintained during production. If it is increased, draindown may occur; and if it is decreased, moisture damage, raveling and delamination may occur in the field.
- When placing the material in the field, it is important to assure an adequate air temperature, usually a minimum of 13ºC (55ºF), and a clean and functional underlying layer. During this process, tack coats are important to guarantee adequate bonding between the PGA mixture and the underlying layer. Commonly, a slow-setting emulsion at a dosage rate of 0.05 to 0.10 gallons per square yard is used for the tack coat.

- For PGA mixtures, steel-wheeled rollers must be used for compaction. Pneumatic-tired rollers are not recommended because the tires tend to pick up the aggregate and close the surface pores, thus reducing drainage capacity.
- It is recommended that some QA/QC testing be done to check AV and/or the permeability of the PGA mixture to ensure drainability and durability.
- General maintenance is suggested to be utilized to control clogging. The most common methods are: 1) fire hose cleaning, 2) high pressure cleaning or 3) the use of specialized vehicles that clean with pressurized water and vacuum.
- The use of rejuvenators and fog seals is the most common preventive maintenance practice to extend the service life of PGA mixtures. In general, slow setting emulsion consisting of a 50:50 mixture of asphalt emulsion and water are used in a two-pass application at a dosage of 0.05 gallon per square yard; emulsions with polymer are used at a rate of 0.2 gallons per square yard.
- Winter maintenance is completed before or after snow and freezing phenomena. Anti-icing products are used in order to prevent the formation of ice and snow, while de-icing products are used to remove snow and ice and address safety. Usually, the same products are used as both anti-icers and de-icers; however, the application rates are different. Salting is currently the most used anti-icing procedure. It includes the use of calcium or magnesium chloride in many forms (rocks, pre-wetted, brine or solid). For example, solid salt is used at a dosage of 5-20 grams per square meter as an anti-icer or 15-40 grams per square meter as a de-icer.
- The use of sand as a winter maintenance practice is not recommended since it can clog the AV structure. The most effective strategy for the use of snow plows is either setting the plow blade higher (1 in above the pavement) or waiting until there is a minimum of 2 in of snow accumulated.
- Corrective maintenance of PGA mixtures consists mainly of patching. This process can be performed using either PGA or DGHMA mixtures; however, it is important to

consider the continuity of drainage by ensuring that the patched area is small and the flow of water around the patch will not create excess surface runoff.

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APPENDIX A

PGA SURVEY DOT AGENCIES RESULTS

In addition to a review of past surveys and the general email contact of the DOTs, a more detailed survey was also developed and sent to some DOT representatives to have a more comprehensive understanding of the current use of PGA. The questionnaire was focused on the following topics: 1) use and site selection, 2) performance and structural contribution, 3) mix design, 4) construction, 5) preventive and corrective maintenance and 6) winter maintenance. In total 8 agencies responded to the survey, 7 of them are US DOTs from Georgia (GA), New Mexico (NM), Nevada (NV), California (CA), Maine (ME), New Jersey (NJ) and Washington (WA) and 1 was a representative from the Netherlands (NDS). The following sections summarize the results obtained.

Use and Site Selection

From the 8 agencies surveyed, 6 are currently using PGA and 2 of them (i.e. NJ and WA) discontinued their use mainly due to raveling which led to early failures in NJ and which was caused by studded tire wear in WA. For the other 6 agencies, the main reasons for use from more to less important are: 1) friction/skid resistance, 2) safety, 3) noise reduction and 4) surface layer (i.e. protection of the pavement underneath).

The locations used to place PGA were primarily interstates and highways; however, some other agencies like NV and NDS allow its use on all type of roads and facilities. In general, PGA is used on roads with high traffic volumes, moderate to heavy rainfall and high speed limits. PGA is not used on roads with speed limits less than 45mph or low volume roads. CA has some additional guidelines for when PGA is not be used, including: areas susceptible to snow and ice conditions, unsound pavement, areas with severe turning movements, muddy or sandy areas, areas prone to oil and fuel drippings and bridge decks.

Performance and Structural Contribution

In terms of performance, the typical service life of PGA is shown in the following figure, ranging from 5 to 15 years. NM, CA, NJ and ME did not report early raveling failure; while GA, NV and NDS reported they do encounter it mainly in transverse construction joints and due to poor compaction during placement.

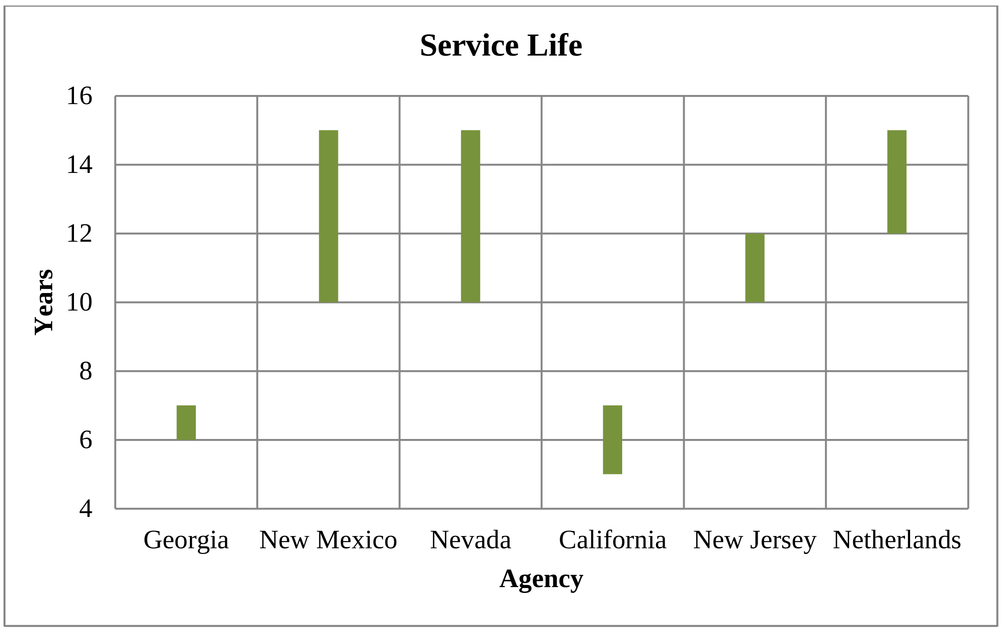


Figure A1.97 Service life of PGA according to survey

From the 8 agencies surveyed, only ME and NDS considered PGA as a structural layer and take it into account in pavement design. In ME, they assign a structural layer coefficient of 0.40; and in NDS they add the layer as an apparent 1 cm (0.4 in) contribution to the pavement design regardless of the mix or lift thickness. The thicknesses of the PGA for the agencies surveyed are presented in Figure A2. Three of the agencies (i.e. NV, NM and WA) have layers of less than 1 inch, while the rest of the agencies have PGA 1 inch or thicker.

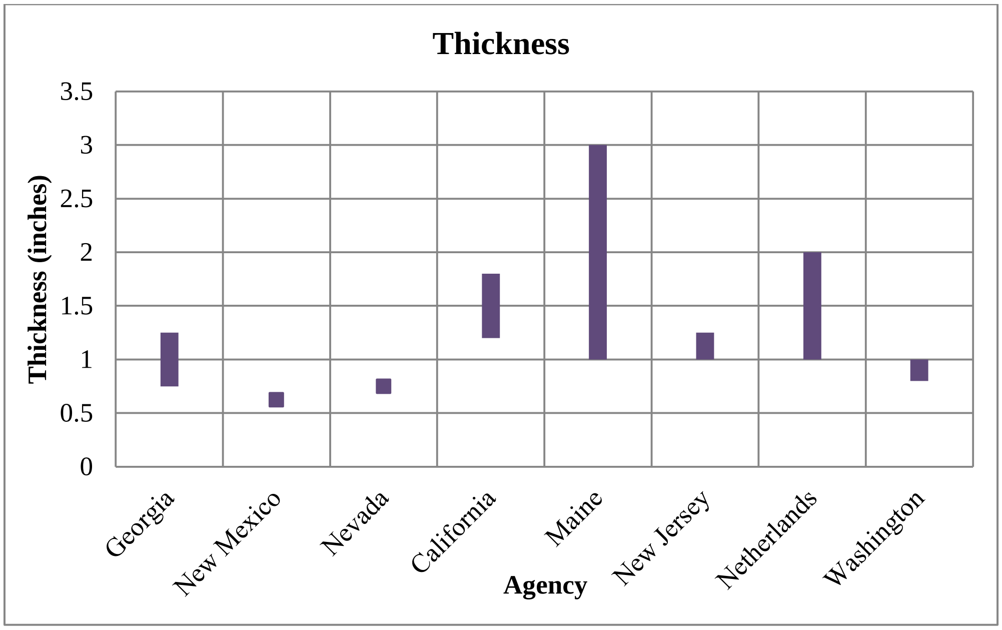


Figure A2. Thickness PGA according to survey

Mix Design

In terms of mix design, the agencies were surveyed in terms of air void requirements, optimum binder content procedures and criteria for controlling snow/ice control. GA has an air voids requirement of 18-22%, NM greater than 18%, ME 17.5-22.5%, NDS 20-24% and NV 8-12%. To find the optimum binder content of the mixture, 6 of the 7 agencies responded and two of them use a procedure based on compacted specimens (i.e. NM, ME), two of them use a procedure based on visual determination (i.e. NV, CA) and the last two (i.e. GA, NJ) use a combination of three methods which are visual determination, compacted specimens and aggregate absorption. These procedures are described in detail subsequently. In the NDS the binder contents are prescribed with ranges from 4.5-6% depending on aggregate gradation. None of the agencies surveyed have a criteria or test that controls snow/ice damage in their mix design criteria.

Construction

Agencies were also surveyed with respect to some general aspects related to the construction of PGA. Specifically they were asked about the type of surfaces on which they place PGA, the milling and tack coat procedures and the air void control requirements. All agencies place PGA on top of new asphalt layers (HMA or WMA in the case of NM); however agencies including NJ, CA, WA and NV also permit the use of PGA on old asphalt pavements that are sound. CA also underscored that sometimes PGA are also placed on concrete pavements.

Only 3 agencies (i.e. GA, NV and NJ) reported milling the surface before the placement. GA uses micro milling according to their specs, while NV mills 0.75 in deeper than the PGA and NJ uses milling on old asphalt layers if they do not place a new intermediate course. Tack coat is used in 6 of the 7 agencies that responded to this question. The only agency that does not use it is NM. From the agencies that use tack coats, the majority use emulsions; however, NJ also allows the use of PG 64-22 binder at a rate of 0.06-0.14 gal/SY. The emulsions utilized are usually Slow Setting (SS) emulsions, and the rates reported are between 0.06 and 0.15 gal/SY.

The only agency in the US that reported having some method of air void control during or after construction was ME. In ME, QA/QC testing is performed on mix samples using the gyratory compactor. Bulk specific gravity is determined using the vacuum seal method due to the high percentage of interconnected voids. In the NDS, the density is checked using a nuclear gage after construction.

Preventive and Corrective Maintenance

The survey also asked for information about preservation and maintenance procedures done to PGA to understand how they are trying to optimize the functionality and durability throughout the service life. In terms of preventive maintenance, two types of procedures were included: vacuum/declogging and the use of fog seals/rejuvenators. Only ME reported doing vacuum and sweeping procedures on their PGA; NM only sweeps but does not vacuum. NDS uses a European cleaning truck which uses a combination of pressurized spray water and vacuum, recycling the water. They clean both the lanes and the shoulders to maintain drainage to prevent frost damage. Regarding corrective maintenance, agencies were questioned on the use of patching. The only agency that does this procedure is NJ which utilizes DGHMA for this procedure.

Winter Maintenance

As mentioned in the previous sections, winter maintenance is one of the major problems that agencies using PGA are facing, and one of the main reasons for discontinuing its use. The survey captured the current maintenance practices of the agencies surveyed. The agencies were asked if they were using snow and ice control maintenance, and from the 7 agencies that answered this question, 6 stated that they are currently performing winter maintenance while CA was the only one who did not since they do not place PGA in snow areas. From the 6 agencies that perform winter maintenance, information was collected with respect to weather prediction systems and triggers for maintenance and the products and equipment used for snow/ice control. The results are presented in Table A1.

Table A1.Winter maintenance procedures according to survey.

State	Weather Prediction system/ triggers for maintenance		Anti icers/De icers		Snow Plow
	System	Trigger	Product name	Rate	
Georgia	National Weather Services	Apply products 48 hrs prior to onset of winter weather.	Brine	40 gal/lane-mile	Plows with carbide tipped blades
			Rock and Salt	2:1 ratio	
New Mexico			Rock salt mixed with sand or scoria		Snow plows with carbide tipped blades
Nevada	NDOT Road Weather Information System (RWIS)	Apply brine day before freezing pavement	Brine (NaCl)		Plows with carbide tips
			Sand mixed with rock salt		
Maine	Road Weather Information System. Also weather forecasts and patrol personnel		Salt brine (23% salt) as pre treatment		Plows with carbide blades
			Pre-wetted salt		
			Calcium or Magnesium Clhoride		
New Jersey		40 F or lower with precipitation forecast	Magnesium Chloride as pretreatment		
			Pre-wetted salt as treatment		
Netherlands			Mix of Salt and grit		
			Brine		

APPENDIX B

SPECIFICATIONS FOR AGGREGATE PROPERTIES

Properties	Arizona	Alabama	California	Wyoming	Florida	Mississippi	Nebraska	Nevada
Chart Area Bulk	2.35-2.85	>2.550						
Combined water absorption (%)	0-2.5%							≤4.0 %
Sand equivalent (%)	Min 55 %			Min 45%				
Fractured coarse aggregate particles (%)	Min. 85%(≥2 fract faces)& Min. 92% (≥1 fract face)		Min. 75%(≥2 fract faces)& Min. 90% (≥1 fract face)	Min. 90%(≥2 fract faces)& Min. 95% (≥1 fract face)		Min. 90%(≥2 fract faces)	Min. 90%(≥2 fract faces)& Min. 95% (≥1 fract face)	Min. 90%(≥2 fract faces)
Flat and Elongated Index (%)	Max 25% 3:1					Max 20% 3:1	Max 10% 5:1	
Carbonates (%)	Max 20%							
Abrasion (%)	Max 9% (100 rev) & Max 40% (500 rev)		Max 12% (100 rev) & Max 40% (500 rev)	Max 40%				
Soundness (%)				≤20 %				≤12 %

Properties	New Jersey	New Mexico	North Carolina	Oklahoma	Oregon	South Carolina	Tennessee	Texas
Combined water absorption (%)	≤2.0 %							
Sand equivalent (%)			Min 45%		Min 45%			
Fractured coarse aggregate particles (%)		Min. 75%(≥2 fract faces)	Min. 90%(≥2 fract faces)& Min. 95% (≥1 fract face)	Min. 95%(≥2 fract faces)& Min. 100% (≥1 fract face)	Min. 75%(≥2 fract faces) & Min. 90% (≥1 fract face)	Min. 90%(≥2 fract faces)	Min. 90%(≥2 fract faces)& 100% (≥1 fract face)	Min. 90%(≥2 fract faces)
Flat and Elongated Index (%)	Max 10% 5:1		Max 10% 5:1	Max 10% 5:1	Max 10% 5:1		Max 20% 3:1	Max 10% 5:1
Abrasion (%)	Max 50% (500 rev)			≤30 %	≤30 %	≤52 %	≤30 %	≤20 %
Soundness (%)		X	≤15 %		≤12 %	≤15 %	≤9 %	≤20 %
Binder	PG 64E-22	PG 70-28+	PG 76-22			PG 76-22		PG 76/ AR

Properties	Virginia	Louisiana	ASTM	NAPA/ NCAT	FHWA	China
Combined Bulk						>2.7
Combined water absorption (%)	≤2.0 %			≤2.0 %		≤2.0 %
Sand equivalent (%)		Min 60	Min 45%			
Fractured coarse aggregate particles (%)	Min. 90%(≥2 fract faces)& 100% (≥1 fract face)	Min. 90%(≥2 fract faces)	Min. 90%(≥2 fract faces)& Min. 95% (≥1 fract face)	Min. 90%(≥2 fract faces)& 100% (≥1 fract face)	Min. 75%(≥2 fract faces)& Min. 90% (≥1 fract face)	
Flat and Elongated Index (%)	Max 10% 5:1	Max 25% 3:1	Max 10% 5:1	Max 20% 3:1		
Carbonates (%)						
Abrasion (%)	Max 40%		≤30 %	≤30 %	Max 40%	≤20 %
Adhesion						≥5
Soundness (%)	≤15 %					≤12 %
Binder	PG 70-28					

APPENDIX C

SPECIFICATIONS FOR AGGREGATE GRADATION

Sieves		China (Jiangsu province)		Arizona		Alabama		California Type 1		California Type 2		California Type 3		Wyoming		Florida		Georgia Type 1		Georgia Type 2		Mississippi Type 1		Mississippi Type 2	
Sieve "	Sieve mm	LL	UL	LL	UL	LL	UL	LL	UL	LL	UL	LL	UL	LL	UL	LL	UL	LL	UL	LL	UL	LL	UL	LL	UL
1 1/2 "	37.5							100	100																
1"	25.4							99	100																
3/4"	19					100	100	85	96	100	100					100	100	100	100						
5/8"	16	100	100																						
0.53	13.2	90	100																						
1/2"	12.5					85	100	55	71	95	100	100	100	100	100	85	100	85	100	100	100	100	100	100	100
3/8"	9.51	60	80	100	100	55	61			78	89	90	100	97	100	55	75	55	75	85	100	80	89	90	100
No. 4	4.76	12	30	35	55	10	25	10	25	28	37	29	36	25	45	15	25	15	25	20	40	15	30	15	30
No. 8	2.38	10	22	0	15	5	10	6	16	7	18	7	18	10	25	5	10	5	10	5	10	10	20	10	20
No. 10	2																								
No. 16	1.18	6	18																						
No. 30	0.595	4	15							0	10	0	10												
No. 40	0.425																								
No. 50	0.3	3	12																						
No. 100	0.15	3	8																						
No. 200	0.075	2	6	0	3	2	4	1	6	0	3	0	3	2	7	2	4	2	4	2	4	2	5	2	5

APPENDIX D

FLORIDA IH 95 CONSTRUCTION REPORT

General Description

In coordination with the Florida DOT and Ranger Construction Industries, Inc. TTI was allowed to participate in construction monitoring and material collection of an OGFC field site located on IH 95 in Broward and Miami-Dade counties in Florida. The construction of this 30-mile long stretch of road commenced in December 2014 and, as of April 2015, is still ongoing. This portion of IH 95 carries an extremely high traffic volume and the number of lanes varies from 3 to 6 in each direction. Due to the high traffic volume, all construction activities were done during nighttime (Figure D1). One researcher from TTI visited the field site during the third week of January to monitor the plant production and paving operation. He also prepared PMLC specimens and obtained raw materials (aggregate, binder, fibers and loose plant mix) for further testing in the laboratory.

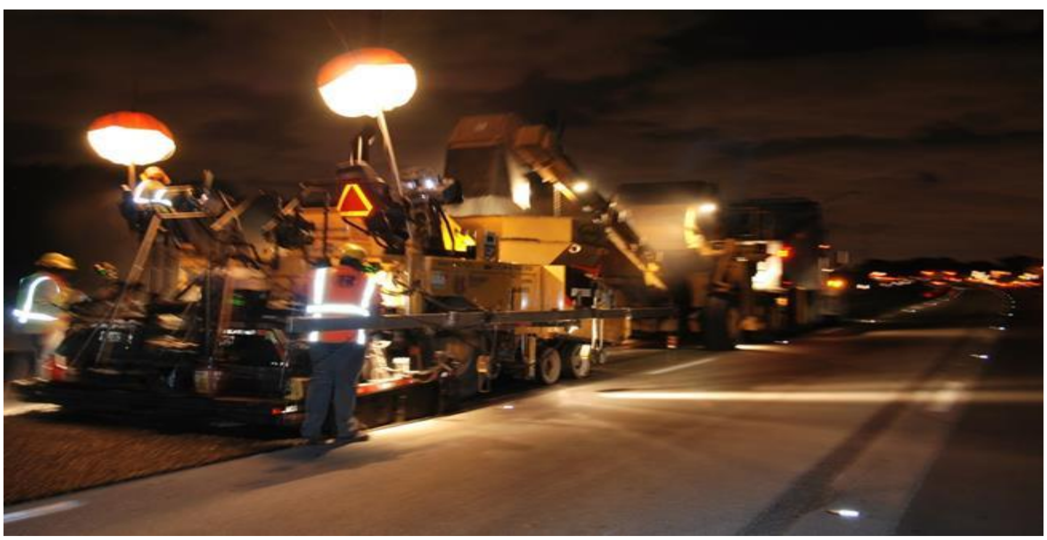


Figure D1. Paving of the OGFC surface course on IH 95 in Florida.

Mixture and Materials

Before placement of the OGFC surface course, the contractor placed 1.5 in. (38.1 mm) thick DGHMA mixture. Figure D2 presents the aggregate gradation and mix design parameters for the OGFC mixture, which was produced using limestone rock from the White Rock Quarries located near Miami-Hialeah. This limestone aggregate is relatively light and porous. The mix used two different stock piles form the same aggregate source. The binder employed to prepare the mixture was a PG 76-22 polymer modified with SBS supplied by South Florida Materials from their Riviera Beach terminal. An anti-stripping agent was mixed with the binder at the terminal. The OBC was 6.5 percent. Additionally, 0.4 percent mineral fiber was added to the mixture to prevent binder draindown. The mixture did not use any hydrated lime or recycled materials.

CORRECTED COPY STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION ***CORRECTED COPY***

ASPHALT MIX DESIGN

SUBMIT TO THE DIRECTOR, OFFICE OF MATERIALS, CENTRAL ASPHALT LABORATORY, 5007 NE 39TH AVE, GAINESVILLE, FL 32609

Contractor: Ranger Construction Industries, Inc. Address: 101 Sansbury's Way, West Palm Beach, FL 33411

Phone No.: (561) 793-9400 Fax No.: (561) 784-3493 E-mail: sgamble@rangerconstruction.com

Submitted By: Ranger Construction Industries, Inc. Type Mix: FC-5 Intended Use of Mix: Friction Course

Product Description	Product Code	Producer Name	Product Name	Plant/Pit Number	Terminal
1. S1A Stone	C41	White Rock Quarries	S1A Stone	87339	
2. S1B Stone	C51	White Rock Quarries	S1B Stone	87339	
3.					
4.					
5.					
6.					
7. PG Binder	916-76PMA		PG 76-22 (PMA)		

PERCENTAGE BY WEIGHT TOTAL AGGREGATE PASSING SIEVES

	Blend	55%	45%					JOB MIX	CONTROL	
	Number	1	2	3	4	5	6	FORMULA	POINTS	
SIEVE SIZES	3/4" 19.0mm	100	100					100	100	
	1/2" 12.5mm	74	100					86	85 - 100	
	3/8" 9.5mm	47	93					68	55 - 75	
	No. 4 4.75mm	9	45					25	15 - 25	
	No. 8 2.36mm	8	13					10	5 - 10	
	No. 16 1.18mm	7	9					8		
	No. 30 600μm	5	7					6		
	No. 50 300μm	4	4					4		
	No. 100 150μm	3	3					3		
	No. 200 75μm	2.0	3.0					2.5	2 - 4	
	G_{SB}	2.407	2.412					2.409		

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.

Total Binder Content 6.5 % FAA ____ % (Plant) Mixing Temperature 325 °F 163 °C

Spread Rate @ 1" 98 lbs/yd^2 $\%G_{mm}$ @ N_{des} ____ (Roadway) Compaction Temperature 325 °F 163 °C

VMA ____ % Ignition Oven Calibration Factor (+To Be Added)/(-To Be Subtracted) +0.08 Additives Antistrip 0.5 % Mineral Fiber 0.4 %

Figure D2. Mix design data sheet for the IH 95 field site in Florida.

Description of the Plant

Since the full length of this field site was approximately 30 miles, the contractor used three separate plants to produce the DGHMA mixture and two plants to produce the OGFC mixture. However, the majority of the OGFC was produced at a single plant located in Pompano Beach. During the third week of January 2015, when TTI's researcher was onsite, the OGFC mixture was being produced at the Pompano Beach plant and, at that time, the paving operation was located approximately 30 minutes away from the plant. Figure D3 shows an overview of the Pompano Beach asphalt plant. This double-barrel green counter-flow drum mix plant has capacity of producing 400 tons/hour; although the plant was operating at 200 tons/hour during the production of the OGFC mixture. Two out of five cold feed bins were used during production. The insulated silo has capacity of holding 200 tons of mixture. A drag slat conveyor carried the mix from the drum to the silo.

Figure D3. Pompano Beach drum plant where the OGFC mixture was produced.

Mixture Production

The OGFC mixture was produced at a rate of 200 to 250 tons/hour. Initially, the mixture was produced at a temperature slightly below 340°F (171°C), but the temperature was eventually raised to meet the temperature requirement for compaction on the highway. The ambient temperature during construction was in the mid-60s°F (around 18°C). After the adjustment at the plant, the temperature of the mixture discharged from the silo and loaded in the dump trucks was between 345 to 350°F (174 to 177°C). The typical silo storage time was 10 to 15 minutes. The dump trucks were equipped with a tarp to cover the mixture during transportation.

Construction

As previously mentioned, construction at this field site was done during nighttime with very limited road closure time in an effort to cause minimal disruption to traffic. The existing DGHMA surface course was laid about a month prior to placement of the OGFC mixture. The contractor applied a tack coat over the existing DGHMA mixture before paving. TTI's researcher noticed a non-uniform distribution of the tack coat during his visit as shown in Figure D4.

Figure D4. Tack coat application before placement of the OGFC mixture.

The dump trucks hauled the mixture to project site and placed it in the shuttle buggy. The shuttle buggy then transferred the mixture to the paver chute. The paving width and thickness of the portion of road being constructed when TTI's researcher was onsite were 12 ft (3.7 m) and 0.75 in. (19 mm), respectively. The temperature measured using and infrared gun right behind the paver ranged between 310 and 320°F (154 and 160°C).

Two vibratory steel-wheeled rollers immediately followed the paver. The rollers operated in static mode and made 2 or 3 passes. The research team did not notice any measure to monitor the degree of compaction achieved onsite.

Sample Collection

Plant mix was collected from the dump trucks at the plant by climbing on a scaffolding as shown in Figure D5. Each day samples were collected from the fifth or sixth dump truck leaving the asphalt plant, after the production temperatures were considered stable. The materials sampling scheme is presented in Table D1. The plant mixture collected from the dump trucks was immediately brought to the lab located within the plant facility for compaction (i.e., PMLC specimen preparation).

Figure D5. Plant mixture sampling from dump truck at the plant.

Table D1. Materials Sampling Scheme

Sample Type	Material	Point of Sampling
LMLC	Mineral Fiber	Plant
	Coarse Aggregate	Stockpile at Plant
	Binder (blended with Anti-Striping agent)	Terminal
PMLC	Plant Mixture	Dump truck at Plant
Field Cores	Field Cores	Random locations from travel lane

With the help of the paving contractor, nine six-inch diameter field cores were collected. These field cores could not be extracted immediately after paving of the portion of the road being constructed when TTI's researcher was onsite due to logistic issues; rather they were obtained several weeks afterwards. Some of the field cores, showing the full depth of the pavement structure, are presented in Figure D6. From this figure it is evident that the existing asphalt layers had varying thicknesses and even the thickness of the newly constructed OGFC surface course did not appear to be uniform. The measured thickness of the 9 field cores acquired after construction ranged from 0.6 to 1.3 in. (15.5 to 32.6 mm).

Figure D6. Field cores obtained after construction of the OGFC surface course on IH 95 in Florida.

Water Flow Value Tests

TTI's researcher evaluated the drainability of the freshly compacted OGFC surface course via WFV tests as part of the construction monitoring activities performed onsite. The WFV test followed the procedure outlined in Tex-246-F *Permeability or Water Flow of Hot Mix Asphalt*. When TTI's researcher was onsite, the OGFC mixture was being placed on one of the northbound travel lanes. At this location, IH 95 has six lanes in each direction. The test was conducted at three different stations on lane R4, where R6 and R1 are the outside and inside most lanes, respectively. At any given station, the WFV tests were conducted on the left wheel path, the right wheel path and between the wheel paths. Figures D7 and D8 show the test setup. There were a few instances where the WFV tests were repeated at same location (without moving the test cylinder) and one instance where the WFV test was repeated by moving the test cylinder just few in away from the initial location. Table D2 presents the resulting WFV, in seconds, required for the water level to drop certain height. Lower numbers indicate higher drainability.

Figure D7. WFV test on right wheel path of lane R4.

Figure D8. Inside view of the WFV test cylinder after application of the sealing putty.

Table D2. WFV Test Results (time in seconds)

Location	Left wheel path	Between wheel paths	Right wheel path
Station 98+100	12.5	20.9 24.3**	12.0
Station 100+00	33.9	34.8	22.2
Station 102+00	18.2	23.6	16.2
	18.1*	23.6*	16.1*

* 2nd run at same location, ** 2nd run at adjacent location

PMLC Specimen Preparation

The loose plant mixture collected from the dump trucks was quickly brought to the laboratory located within the premises of the plant and placed in the oven between 1 to 2 hours to stabilize to a compaction temperature of 325°F (163°C). This was selected based on the temperature of the OGFC surface course measured via infrared gun right behind the paver, which ranged between 310 and 325°F (154 and 163°C). Twelve 6.0 in. (150 mm) diameter by 2.5 in. (61 mm) tall PMLC specimens were compacted using the SGC equipment shown in Figure D9. The target AV of the compacted specimens was set to 20.0±2.0 percent and estimated with dimensional analysis.

Figure D9. SGC used to prepare the PMLC specimens.

Typically, the laboratory compaction of OGFC mixtures requires batching directly into the mold instead of batching in separate pans. This practice is followed because of the type of binders used in this type of mixture, which tend to stick in large quantities to the surface of the molds along with the fines of the mixture. The laboratory located within the premises of the plant did not have a scale large enough to measure the weight of the compaction mold along with the plant mixture. Therefore, TTI's researcher 'buttered' the pans prior to batching and scrapped the excess mastic stuck to the pan when pouring the batched loose mixture into the compaction mold. Most of the PMLC specimens required between 50 to 80 SGC gyrations to achieve the target AV.

APPENDIX E

SR-71 UTAH TEST SECTION CONSTRUCTION REPORT

General Description

The research team participated in construction monitoring and sample collection of a permeable friction course (PFC) construction job located on SR 71 in Salt Lake City, Utah. This particular section served as a test section in dry-cold climate region in the north-west part of the US. This construction job, approximately 4-mile long, is located just south of Salt Lake City downtown in Utah. The construction of friction course started in July 2015 and completed in August 2015. Staker Parsons Companies, a general construction contractor, produced and paved the mixtures.

SR 71 in this area as shown in Figure E1 is a city street with three lanes in each direction with intermittent median or turn lane. It carries moderate to high volume traffic with little truck traffic. Due to its proximity to Salt Lake City downtown, the entire paving was done during night time (Figure 1). One researcher from TTI visited the job site during the second week of August 2015. There, he monitored the plant production, paving operation, and field permeability testing. He also prepared some specimens and obtained samples for testing in the lab.

Figure E1. SR 71 in Salt Lake City.

Mixture and Materials

Before the placement of friction course, the contractor milled out the existing surface layer. FigureE2 presents the summary of friction course mixture design. Staker Parson Companies designed and produced this PFC mixture using rocks from their quarry located next to their Beck Street Hot Mix plant and rocks from their quarry located at Point Mountain, Utah. The Beck Street plant is located few miles north of Salt Lake City downtown. The mix used five different stock piles form the two aggregate sources as well as one percent hydrated lime. Aggregates from Beck Street quarry is classified as dolomite limestone. The PG 58-28 grade neat binder was obtained by Staker Parson Company's own asphalt terminal located in Ogden, Utah. The design asphalt content was 5.9 percent by weight of the mixture. Additionally, the mix used 1.0 percent hydrated lime as anti-stripping agent. The mix did not use any recycled asphalt pavement.

Material Source: Beck Street Hot Plant
Material Type: Open Graded Surface Course
Asphalt Source: Staker Parson PG 58-28 @ 5.9AC

Sieve	% Passing	Spec's
1/2"	100%	100
3/8"	90%	90-100
# 4	42%	35-45
# 8	17%	14-20
# 200	3.5%	2-4

Property	Result
Beck St. ½" Rock	6%
Beck St. ¼" Rock	13%
Beck St. Pep Fines	19%
Point Mtn. 3/8" Chips	42%
Point Mtn. Squeegee	19%
Lime	1%

Figure E2. Summary of Mixture design used on SR 71 Project in Utah.

Description of Plants

The Contractor's hotmix plant located at Beck Street produced the entire amount of mixture. Figure E3 show the overview of this plant. This fifteen year old Double-Barrel Green counter-flow drum mix plant has capacity of producing 650 tons/hour mixture; although the plant was operating at 300 tons of mix per hour during the production of friction course. The plant equipped with conventional bag-house emission control system re-introduced part of the fines back to the drum. Five out of eight cold feed bins were used during the production. Each of six insulated silo has capacity of holding 200 tons of mixture. A drag slat conveyor carried the mix from the drum to the silo. The binder was kept at 315F at binder storage tank.

Figure E3. Hot Mix Plant Located at Beck Street Used to Produce PFC Mixture.

Mixture Production

Friction course mixture was produced typically at 300 tons per hour. The production temperature measured at trucks in the plant was between 290 to 300F. Typical silo storage time was less than 30 minutes. The contractor was worried about drain-down as UDOT didn’t allow using mineral fiber for this mixture. Belly dump trucks used for hauling mixtures had tarp to cover the mixtures. The production temperature was slightly below 300F. During this time the ambient temperature was between upper 70s to lower 80s with light wind. Average hauling distance and time between the plant and job site were approximately 20 minutes and eight miles, respectively.

Construction

The paving of the entire job was done during the night time. Due to the site's proximity to the downtown the window of road closure was limited. This friction course mix was placed on top of a milled surface. The existing surface, also an OGFC, was milled off by one in thickness. Existing surface had moderate amount of longitudinal cracking and very few alligator cracking. Before the placement of mix, the contractor spread SS1 tack coat at 0.08 gal/sq. yd. rate. The quality of milling and tack coat application appeared uniform as shown in Figure E4. Belly dump trucks hauled the mixture to project site and they released the loose mixture on the road in front of shuttle buggy. The shuttle buggy then transferred the loose mix into the paver chute. The paving width and thickness were 11 feet and 1.5 in, respectively. The temperature measured using infra-red gun right behind the paver ranged between 260 to 275F. Two steel-wheeled seven feet wide roller compacted the mat in static mode. They ran in tandem (Figure E5) with three feet overlap at the center of the mat. Each roller had four passes. The rollers followed the paver closely. Later, a finish roller (front steel drum and two pneumatic rear wheels) made few passes to make the compacted mat surface smooth. The research team did not notice any measure to monitor the compaction.

Figure E4. Milled Surface and Tack Application before Paving on SR 71 in Utah.

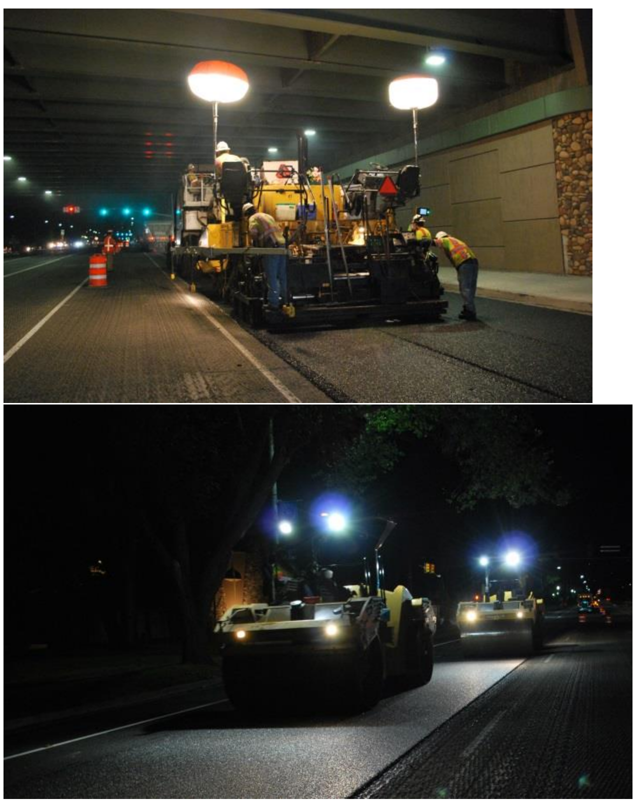


Figure E5. Paving of Friction Course on SR 71 in Utah.

Sample Collection

Plant mix was collected from the trucks at plants by climbing on scaffolding. Samples were collected from the fifth through seventh truck when the plant temperatures become stable. The materials sampling scheme is presented in Table E1. Mixture sample collected from the plant was immediately brought to the lab located within the plant facility for compaction. With the help of the paving contractor, the research team also collected nine six-inch diameter road cores from the job site.

Table E1. Materials Sampling Scheme.

Sample Type	Material	Point of Sampling
Lab-Mixed, Lab-Compacted	Hydrated Lime	Plant
	Coarse Aggregate	Stockpile at Plant
	Binder	Transport Truck at Plant
Plant-Mixed, Lab-Compacted	Loose Mix	Truck at Plant
Plant-Mixed, Field-Compacted	Road Cores	Shoulder Area

Field Specimen Compaction

12 six-inch (150 mm) diameter and 2.5 (61 mm) in tall specimens were compacted on-site using plant mix at the lab located within the plant premises. Loose plant mix collected at the plant was quickly brought to the lab and placed in the oven between 1 to 2 hours to achieve 265F compaction temperature. Note that the construction crew compacted friction course at around 265F on the road. The research team compacted these specimens using a Pine Brovold Superpave gyratory compactor (Figure E6) to 20±2 percent air voids. The air voids were measured using volumetric analyses. Batch weighed based on 20 percent air voids was placed in the mold and SGC compactor was set achieve desired specimen height. During the compaction, the team experienced issues with extracting the specimens from the mold. Due

to high air voids the specimens were falling apart during the extraction. In order prevent the specimens from breaking loose the compacted specimens were left in the mold and the mold was left under air circulation for cooling down. Even after extraction from the compaction mold, the specimens were placed in plastic molds until their core temperature drops down to room temperature. Obviously, it took much longer time to compact all 12 specimens. Most of the specimens required 60 to 80 gyrations to achieve the desired thickness (and or air voids).

Figure E6. Pine Brovold Superpave Gyratory Compactor used in the Plant Lab.

Field Permeability Test

During the construction monitoring trip the research team conducted field permeability testing on freshly compacted PFC surface. The test was conducted using both the Texas Permeability device following the test procedure outlined in Tex-246-F “Permeability or Water Flow of Hot Mix Asphalt” and field permeability device developed at NCAT. Tests were conducted two different locations: one on southbound bike lane and the other on northbound bike lane. Figure shows general location for permeability testing.

Figure E7. Field Permeability Testing on South Bound Bike Lane near Wilmington Avenue.

At any given station tests were conducted at 2 feet from the free edge, center of paving, and 2 feet from curb edge. Table E2 presents the Texas permeability testing results in seconds required for the water level to drop certain height. Lower number indicates higher flow rate or permeability. Table E3 shows the test results with NCAT device. During the testing it was observed that significant amount of amount of water ran off through the surface all way to curb gutter rather than draining through the friction course and then flow horizontally through the mixture.

Table E2. Field Permeability Test Results using Texas Device.

Location	2 ft. from free edge	Center of the mat	2 ft. from curb edge
	Time in Seconds		
Loc 1a (100 ft north of Wilmington Ave)	27.9		
Loc 1b (104 ft north of Wilmington Ave)	27.05		
Loc 1c (110 ft north of Wilmington Ave)		39.09	
Loc 1d (115 ft north of Wilmington Ave)		39.22	
Loc 1e (120 ft north of Wilmington Ave)			26.37
Loc 1f (125 ft north of Wilmington Ave)			28.01

TableE3. Field Permeability Test Results using NCAT Device.

Location	2 ft. from free edge	Center of the mat	2 ft. from curb edge
	Time in Seconds to drop water from 16 to 3		
Loc 1a (98 ft north of Wilmington Ave)	15.23		
Loc 1b (102 ft north of Wilmington Ave)	16.46		
Loc 1c (115 ft north of Wilmington Ave)		19.67	
Loc 1d (117 ft north of Wilmington Ave)		18.22	
Loc 1e (120 ft north of Wilmington Ave)			19.24
Loc 1f (125 ft north of Wilmington Ave)			26.06

At one area on northbound shoulder/bike lane, the research team measured the permeability using Texas device after each roller pass from the steel-wheeled roller. At three feet away from the retained edge time required to drop the water after 1st, 2nd, 3rd, and 4th pass was 11.83, 12.30, 12.47, and 15.46 seconds, respectively. These four testing could not be done exactly at same spot as there were some putty left on the surface from previous tests. So, there were done at 1 foot apart in longitudinal directions. At same general area after finish roller the researcher team conducted few more permeability testing and time was measured at 22.95 and 25.32 seconds at center of the mat and 27.02, and 25.25 at two feet from inside edge when tested with Texas device.

APPENDIX F

GARDEN STATE PARKWAY NEW JERSEY TEST SECTION CONSTRUCTION REPORT

General Description

The research team participated in construction monitoring and sample collection of an open graded friction course (OGFC) construction job located on south end of Garden State Parkway (GSPKWY) in Cape May County, New Jersey. This particular job served as a test section in wet-cold climate region in the north-east part of the US. This OGFC paving job was part of a larger project of improving Interchanges 9-11 of GSPKWY undertaken by New Jersey Turnpike Authority (NJTPA). The construction job, known as Garden State Parkway Interchange 10 Improvements, is located near the Township of Middle between Crest Haven and Shell Bay Avenue. The paving of friction course took place in May 2016. R.E. Pierson was the general contractor for this project. South State Inc. and Sea Shore Asphalt Company were paving contractor, and mixture supplier, respectively. STV-JMT Joint Venture acted as consultant for the NJTPA.

GSPKWAY in this area as shown in Figure F1 is a recently upgraded rural freeway with two lanes in each direction with wide shoulders on both sides. The project is approximately four-mile long in both directions. Southbound and northbound travel lanes are divided by wide median. Figure F2 presents existing and proposed pavement structure. It carries moderate to high volume traffic with little truck traffic. Paving was done during the day time. One researcher from TTI visited the job site during the third week of May 2016. There, he monitored the plant production, paving operation, and field permeability testing. He also prepared some specimens and obtained samples for testing in the lab.

Figure F1. GSPKWY in Cape May County, New Jersey.

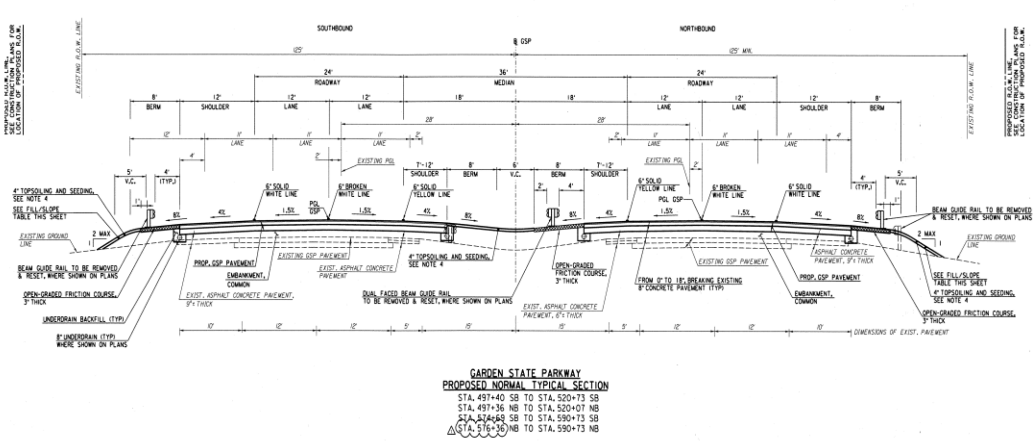


Figure F2. Proposed Typical Section.

Mixture and Materials

Figure F3 presents the mixture design of asphalt rubber open graded friction course designed by Western Technologies Inc., a company based in Phoenix, Arizona. Seashore Asphalt Inc., the mixture supplier, produced the mix at their plant located in Woodbine, NJ. The plant is located about 13 miles or twenty minutes from the job site.

ACFC Binder Design Summary — Western Technologies, Inc.

WT Product Code:	2144XL069 / H069-02	Date:	04-30-14
Client:	ECOPATH Contracting LLC	Mix Type:	AR-OGFC / NJDOT
Project Name:	New Jersey AR-OGFC	Source of Aggregate:	Martin Stone & Hanson Aggregates
Project No.:	Contract No. P300.162	Asphalt Source:	Valero / CRM
		Asphalt Grade:	PG 64-22/23.5% CRM (by Wt. of AC)
Project Loc.:	New Jersey	Type of Admix.:	None

Composite Aggregate Gradation			
Aggregate		Lab No.	Percentage w/ Admix
#9 Argillite		H069-02B	10.0
#8 Gneiss		H069-02C	90.0
Sieve (US/mm)	Composite Grading	AR-OGFC Specs	Production Tolerances
2" / 50	100		
1.5" / 37.5	100		
1" / 25	100		
3/4" / 19	100		
1/2" / 12.5	100	100	94-100
3/8" / 9.5	95	90-100	89.5-100
1/4" / 6.3	54		
#4 / 4.75	31	20-40	25.5-36.5
#8 / 2.36	9	5-10	4.5-13.5
#10 / 2.00	8		
#16 / 1.18	6		
#30 / .600	5		
#40 / .425	4		
#50 / .300	4		
#100 / .150	3		
#200 / .075	2.3	0-3.0	0.3-4.3

Recommended % Asphalt Rubber Binder (ARB): 8.0

Mix Designation: AR-OGFC / NJDOT
ARB Proportions: PG 64-22/23.5% CRM (by Wt. of AC)
ARB Source: Valero / CRM

Discharge Mixer Temperature: 325°F Maximum
Field Compaction Temperature: 275°F Minimum

Data at Recommended % Asphalt Rubber Binder (ARB)			Production
Property	Design	Specification	Tolerances
Percent of ARB:	8.0	8.0 Min	7.6-8.4
Bulk Specific Gravity :	1.971		
% by Volume of Asphalt Cement (Vb) :	15.8		
Bulk Specific Density (PCF):	123.0		
Theor. Max. Sp. Gr. (Gmm):	2.412		
Cantabro Test (% Loss):	0.9	30 Max	
Percent Air Voids (V_a):	18.3	15 Min	14 Min
VCA_{drc} :	38.5		
VCA_{mix}:	33.0	< VCA_{drc}	
G_{bca} :	2.677		
Dust to Eff. Asphalt Ratio:	0.32		
Effective Sp. Gr.:	2.733		

Aggregate Properties				
Property	Coarse	Fine	Combined	
Bulk (Dry) Sp. Gravity:	2.722	2.584	2.677	
"SSD" Sp. Gravity:	2.738	2.630	2.703	
Apparent Sp. Gravity:	2.766	2.708	2.748	
Water Absorption(%):	0.58	1.77	0.73	
			Binder Sp. Gravity:	1.027
G_{dca} - AASHTO T19 Dry Rodded Unit Weight (pcf):			102.8	
P_{ca} (%):			91	
Asphalt Absorbed into Dry Aggregate (%):			0.78	

(a)

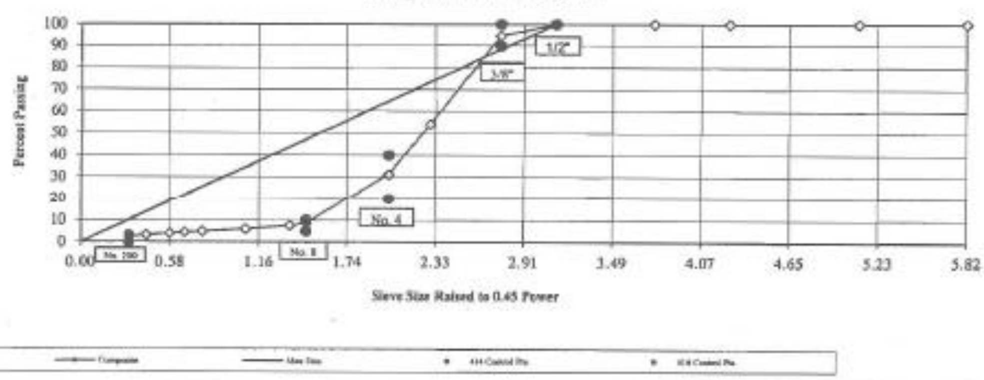
Aggregate Composite Grading - Stockpiles Western Technologies, Inc.

WT Project No:	2144XL069 / H069-02	Date:	04-30-14
Client:	ECOPATH Contracting LLC	Mix Type:	AR-OGFC / NJDOT
Project Name:	New Jersey AR-OGFC	Source of Aggregate:	Martin Stone & Hanson Aggregates
Project No.:	Contract No. P300.162	Asphalt Source:	Valero / CRM
	South State, Inc.	Asphalt Grade:	PG 64-22/23.5% CRM (by Wt. of AC)
Project Loc.:	New Jersey	Type of Admix.:	None

Lab No.		Aggregate Name	Source	Percentage	
H069-02B	Aggregate #1:	#9 Argillite	Hanson Aggregates	10.0	
H069-02C	Aggregate #2:	#8 Gneiss	Martin Stone Quarries	90.0	
	Aggregate #3:				
			Total:	100.0	
Test Method: AASHTO T27 & T11			Difference:	0.0	

H069-02B	H069-02C						Lab No.				
10.0	90.0						Percent				
Agg. #1	Agg. #2	Agg. #3					Sieve	Composite	Control	Production	
Percent Passing							(US/mm)	Grading	Points	Tolerances	
100.0	100.0						1/2" / 12.5	100	100	94-100	
99.8	94.1						3/8" / 9.5	95	90-100	89.5-100	
84.7	50.9						1/4" / 6.3	54			
70.6	26.9						#4 / 4.75	31	20-40	25.5-36.5	
17.4	7.6						#8 / 2.36	9	5-10	4.5-13.5	
14.3	6.9						#10 / 2.00	8			
9.6	5.6						#16 / 1.18	6			
7.2	4.6						#30 / .600	5			
6.5	4.2						#40 / .425	4			
6.0	3.7						#50 / .300	4			
5.1	3.0						#100 / .150	3			
1.5	2.4						#200 / .075	2.3	0-3.0	0.3-4.3	

Composite Power 45 Chart



(b)

Max Theor Gravity (Rice) Test & Aggregate Data

Western Technologies, Inc.

WT Product Code:	2144XL069 / H069-02	Date:	04-30-14
Client:	ECOPATH Contracting LLC	Mix Type:	AR-OGFC / NJDOT
Project Name:	New Jersey AR-OGFC	Source of Aggregate:	Martin Stone & Hanson Aggregates
Project No.:	Contract No. P300.162	Asphalt Source:	Valero / CRM
	South State, Inc.	Asphalt Grade:	PG 64-22/23.5% CRM (by Wt. of AC)
Project Loc.:	New Jersey	Type of Admix.:	None

Maximum Theoretical Gravity (Rice) Test		
Test Method: AASHTO T209		
Percent of binder in Sample:		4.0
Weight of Flask:	Flask 259	0.0
	Flask 564	0.0
	Flask 866	0.0
Weight of Sample and Flask:	Flask 259	1043.4
	Flask 564	1042.1
	Flask 866	1043.1
Wt. of Sample, Flask, Water, & Glass Plate:	Flask 259	4010.9
	Flask 564	4443.6
	Flask 866	3960.1
Weight of Glass Plate:	Flask 259	121.2
	Flask 564	121.2
	Flask 866	121.2
Weight of Sample in Air("Wmm"):	Flask 259	1043.4
	Flask 564	1042.1
	Flask 866	1043.1
Loss of binder from mixing:		-3.6
Wt. of Flask, and Water, (B):	Flask 259	3254.1
	Flask 564	3686.8
	Flask 866	3202.3
Wt. of Sample, Flask, & Water, (C):	Flask 259	3889.7
	Flask 564	4322.4
	Flask 866	3838.9
Volume of Voidless Mix ("Vvm"):	Flask 259	407.8
	Flask 564	406.5
	Flask 866	406.5
Maximum Sp. Gravity ("Gmm"):	Flask 259	2.559
	Flask 564	2.564
	Flask 866	2.566
Average Maximum Sp. Gravity ("Gmm"):		**2.563**
Average Maximum Density (PCF):		**159.7**
"Gmm" Range:		0.007

All Weights in Grams   0.0 = Area was tared

Maximum Theoretical Gravity (Rice) Test Design Calculations	
Asphalt Rubber Binder Specific Gravity @77°F / 77°F:	1.027
Effective Specific Gravity:	2.733
Asphalt Absorbed (%):	0.78

Coarse Specific Gravity	
Test Method: AASHTO T85	
Oven-Dry Weight(g):	2993.8
"SSD" Weight(g):	3011.2
Weight in Water(g):	1911.4
Bulk (Dry) Sp. Gravity:	2.722
"SSD" Sp. Gravity:	2.738
Apparent Sp. Gravity:	2.766
Water Absorption(%):	0.58

Fine Specific Gravity	
Test Method: AASHTO T84	
Oven-Dry Weight(g):	491.3
"SSD" Weight(g):	500.0
Weight of Flask & Water(g):	649.0
Weight of Flask, Water & Sample(g):	958.9
Bulk (Dry) Sp. Gravity:	2.584
"SSD" Sp. Gravity:	2.630
Apparent Sp. Gravity:	2.708
Water Absorption(%):	1.77

Combined Specific Gravity	
Combined Bulk (Dry):	2.677
Combined "SSD":	2.703
Combined Apparent:	2.748
Combined Water Absorption (%)	0.73

Composite Mineral Aggregate Properties		
Property	Value	
AASHTO T19 Dry Rodded Unit Weight (PCF):	102.8	

(c)

Bulk Density, Computed Binder Content, and Schellenberg Draindown Test — Western Technologies, Inc.

WT Product Code:	2144XL069 / H069-02	Date:	04-30-14
Client:	ECOPATH Contracting LLC	Mix Type:	AR-OGFC / NJDOT
Project Name:	New Jersey AR-OGFC	Source of Aggregate:	Martin Stone & Hanson Aggregates
Project No.:	Contract No. P300.162	Asphalt Source:	Valero / CRM
	South State, Inc.	Asphalt Grade:	PG 64-22/23.5% CRM (by Wt. of AC)
Project Loc.:	New Jersey	Type of Admix.:	None

Molding, AASHTO T312 (50 Gyrations)				Bulk Sp. Gr. Measured by AASHTO T331					Compaction / Mixing Temp: 335 F / 300 F
% Binder (Tot. Mix)	Spec. #	Bag Weight (g)	Sample Weight before Sealing (g)	Sealed Sample Weight in Water (g)	Sample Weight After Water Submersion (g)		Bulk Specific Gravity (g/cm^3)	Air Voids (%)	Maximum Theoretical Specific Gravity AASHTO T209
	1	95.5	4166.3	2047.0	4166.3		1.944	19.4	2.412
8.0	2	96.0	4175.3	2040.0	4175.3		1.983	17.8	
	3	96.0	4170.0	2039.0	4168.0		1.987	17.6	Va = 18.3 % (15 Minimum)
						Average:	1.971	18.3	Vb = 15.8 %
						Range:	0.039	1.8	

VCA_{drc} = 38.5

VCA_{mix} is required to be less than VCA_{drc}

VCA_{mix} = 33.0

Where:

V_a – Air voids in the mix

V_b – Percent by volume of asphalt cement

G_{sbca} = 2.677

G_{uwca} = 102.8 pcf

P_{ca} = 91 %

Where:

VCA_{drc} = The voids in the coarse aggregate fraction of the JMF aggregate skeleton.

VCA_{mix} = The voids in the coarse fraction of the mix. The coarse aggregate fraction of the aggregate is that portion of the JMF aggregate skeleton not passing the No. 4 sieve.

G_{sbca} = The bulk specific gravity of the coarse aggregate fraction as determined by AASHTO T85.

G_{uwca} = The unit weight of the coarse aggregate fraction as determined by AASHTO T19.

P_{ca} = The percent of the coarse aggregate fraction (plus No. 4) by weight of total mix.

(d)

Figure F3. (a-d) Mix Design New Jersey material.

The mix used aggregates from two different sources. Aggregates from Hanson, New Jersey quarry is classified as argillites and aggregates from Martin Marietta, New Jersey quarry is classified as gneiss. The PG 64-22 grade neat binder was obtained by from Valero Asphalt Refinery located at Paulsboro, NJ. The neat binder was blended with 23.5 percent (by weight of AC) ground crumb rubber. Arizona based company CRM/Ecopath blended the binder with crumb rubber at using their portable plant (Figure F4) placed inside the hot mix plant. The design asphalt rubber content was 8.0 percent by weight of the mixture. This mix did not use any recycled asphalt pavement or any other additive.

Figure F4. Portable Crumb-Rubber Blending Plant.

Description of Plants

The Contractor’s hot mix plant located at Woodbine, NJ produced the entire amount of mixture. Figure F5 show the overview of this plant. This Double-Barrel Green counter-flow drum mix plant has capacity of producing 400 tons/hour mixture; although the plant was operating at 250 tons of mix per hour during the production of friction course. The plant equipped with conventional bag-house emission control system re-introduced part of the fines back to the drum. Two out of six cold feed bins were used during the production. Each of the three insulated silo has capacity of holding 200 tons of mixture. A drag slat conveyor carried the mix from the drum to the silo. The binder was kept at 350 °F at binder storage tank.

Figure F5. Hot Mix Plant Located at Woodbine, New Jersey.

Mixture Production

Friction course mixture was produced typically at 250 tons per hour. The production temperature measured at trucks in the plant was between 305 and 315 °F. Typically silo storage time was less than 20 minutes. Belly dump trucks used for hauling mixtures had tarp to cover the mixtures. The production temperature was around 315 to 320 °F. During this time the ambient temperature was between upper 70s to lower 80s with light wind. Average hauling distance and time between the plant and job site were approximately 13 miles and 25 minutes, respectively.

As part of their quality control plan, the contractor compacted specimens using SGC device and measure their air voids. Using ignition oven they also measured the asphalt contents and aggregate gradations. Typical results are presented in Figure F6 and Figure F7.

South State, Inc. General Contractors

JOB	MIX	CONTRACTOR	SUPPLIER
Parkway 9.10.11	AROGFC	South State, Inc	Seashore Asphalt

5

Sample Wt	1698.5	**AC**	**8.17**
Wgt. Loss	144.7		
Sieve Temp Comp	0.21	Sample Wt After Ignition	1553.8
Cal. Factor	0.14	% Loss	8.52
Combined	0.35	% Bitumen	**8.17**

Sieve	Wt.	Ret%	Pass%	Specifications		Height	116.8
1/2" / 12.5	0	0.0	**100.0**	94	100		
3/8" / 9.5	80.1	5.2	**94.8**	89.5	100		
#4 / 4.75	1102.3	70.9	**29.1**	25.5	36.5		
#8 / 2.36	1402.3	90.2	**9.8**	4.5	13.5		
#16 / 1.18	1480.2	95.3	4.7				
#30 / .600	1488.7	95.8	4.2				
#50 / .300	1499.6	96.5	3.5				
#100 / .150	1503.3	96.7	3.3				
#200 / 0.75	1513.2	97.4	**2.6**	0.3	4.3		

6

Sample Wt	2963.4	**AC**	**8.35**
Wgt. Loss	257.9		
Sieve Temp Comp	0.21	Sample Wt After Ignition	2705.5
Cal. Factor	0.14	% Loss	8.70
Combined	0.35	% Bitumen	**8.35**

Sieve	Wt.	Ret%	Pass%	Specifications		Height	
1/2" / 12.5	0	0.0	**100.0**	94	100		
3/8" / 9.5	202	7.5	**92.5**	89.5	100		
#4 / 4.75	1889.9	69.9	**30.1**	25.5	36.5		
#8 / 2.36	2554.6	94.4	**5.6**	4.5	13.5		
#16 / 1.18	2624.3	97.0	3.0				
#30 / .600	2649	97.9	2.1				
#50 / .300	2665.4	98.5	1.5				
#100 / .150	2679.6	99.0	1.0				
#200 / 0.75	2640.2	97.6	**2.4**	0.3	4.3		

Figure F6. Typical QC Data showing Asphalt Content and Gradations.

South
State, Inc.
General Contractors

JOB	MIX	CONTRACTOR	SUPPLIER	DATE
Parkway 9.10.11	AROGFC	South State, Inc	Seashore Asphalt	5/19/2016

CoreLok	A	B	C	D	E	F	G	H	I	J	K	L
Sample	Bag Weight	Dry Sample Weight Before Sealing	Sealed Sample Weight In Water	Dry Sample Weight After Water	Ratio (B/A)	Bag Apparent Gravity From Table	Total Volume (A+D) - C	Volume of Bag (A/F)	Volume of Sample (G-H)	Bulk Specific Gravity (B/I)	Maximum Specific Gravity	Air Voids
1	50.6	4009.8	1977.2	4009.8	79.2	0.694	2083.2	72.9	2010.3	1.995	2.433	**18.0**
2	49.9	3975.1	1880.1	3975.1	79.7	0.694	2144.9	71.9	2073.0	1.918	2.440	**21.4**
3	51.3	3998.7	1891.4	3998.7	77.9	0.694	2158.6	73.9	2084.7	1.918	2.448	**21.6**
4	52	3997	1931.7	3997.5	76.9	0.694	2117.8	74.9	2042.9	1.957	2.458	**20.4**

Production Tolerance for Air voids = >14%

Draindown

A-mass of the empty wire basket
B-mass of the wire basket and sample
C-mass of the empty catch plate
D-mass of the catch plate plus drained material

Draindown percent = (D-C)/(B-A) x 100

Sample	1
A	173.6
B	1318.4
C	90
D	90
	0.00

Rices	**Samples**			
	1	2	3	4
Sample wt	1253.0	1497.7	1352.2	1297.4
Wt in H2O After Vac	737.9	883.8	799.8	769.5
Wt of Displaced H2O	515.1	613.9	552.4	527.9
Maximum Sp. Gr. (Gmm)	**2.433**	**2.440**	**2.448**	**2.458**
Rice Number Field	**151.8**	**152.2**	**152.7**	**153.4**

Figure F7. Typical QC Data Showing Air Voids, Draindown, and Rice Specific Gravity.

Construction

The paving of the entire job was done during the day time. This friction course mix was placed on top of a milled surface. Before the placement of the friction course, the contractor milled out part of the existing surface layer. The 'skin milling' reached a depth of 0.25 in to 0.50 in primarily to remove the surface irregularities. Existing surface had different distress levels depending on the location. The area where the researcher tested for permeability testing had very few to no surface distresses. Other locations had moderate amount of longitudinal cracking and very few alligator cracking on existing surface. Before the placement of mix, the contractor spread CRS-2 tack coat at 0.10 gal/yd^2 rate. The tack was spread at 150 °F. The quality of milling and tack coat application appeared uniform. The researcher noticed considerable 'picking' of tack coat by the construction equipment (Figure F8).

Figure F8. Picking of Tack by the Construction Equipment.

Belly dump trucks hauled the mixture to project site and they released the loose mixture into the chute of shuttle buggy or material transfer device. The shuttle buggy then transferred the loose mix into the paver chute. The paving width and thickness were 18 feet and 2.0 in, respectively. The temperature measured using infra-red gun right behind the paver ranged between 295 and 305 °F. Two steel-wheeled seven feet wide roller compacted the mat in static mode. The breakdown roller made four passes (Figure F9). The finish roller, operated on static mode, made four to eight passes as directed the technicians measuring in-situ density using a nuclear density gauge. The technician was directing the roller operator to achieve density between 14 to 23%. He mentioned that most of his in-situ density measurements were between 18-21.5 percent for this project. A picture of the final product is presented in Figure F10.

Figure F9. Paving of Friction Course on Garden State Parkway in New Jersey.

Figure F10. Close-up View of Freshly compacted Surface.

Sample Collection

Plant mix was collected from the trucks at plants by climbing on scaffolding. Samples were collected from the fifth through seventh truck when the plant temperatures become stable. The materials sampling scheme is presented in Table F1.

Table F1. Materials Sampling Scheme.

Sample Type	Material	Point of Sampling
Lab-Mixed, Lab-Compacted		Plant
	Coarse Aggregate	Stockpile at Plant
	Blended Binder	Storage Tank at Plant
Plant-Mixed, Lab-Compacted	Loose Mix	Truck at Plant
Plant-Mixed, Field-Compacted	Road Cores	

Figure F11 show the collection of plant mix and binder at hot mix plant. Mixture sample collected from the plant was immediately brought to the lab located within the plant facility for compaction. As of writing this field report NJTPA has not collected the cores. The research team is working with NJTPA to obtain the cores from shoulder area.

Figure F11. Collection of Plant Mix and Binder and Plant.

Field Specimen Compaction

12 six-inch (150 mm) diameter and 2.5 (61 mm) in tall specimens were compacted on-site using plant mix at the lab located within the plant premises. Loose plant mix collected at the plant was quickly brought to the lab and placed in the oven between 1 to 2 hours to achieve

265F compaction temperature. Note that the construction crew compacted friction course at around 295 °F on the road. The research team compacted these specimens using a Pine Superpave gyratory compactor (Figure F12) to 20±2 percent air voids. The air voids were measured using volumetric analyses. Batch weighed based on 20 percent air voids was placed in the mold and SGC compactor was set achieve desired specimen height. During the compaction, the team experienced issues with extracting the specimens from the mold. Due to high air voids the specimens were falling apart during the extraction. In order prevent the specimens from breaking loose the compacted specimens were compacted at around 225 F and left just above the mold under air circulation for cooling down before removing it. Even after extraction from the compaction mold, the specimens were placed in plastic molds until their core temperature drops down to room temperature.

Figure F12. Pine Superpave Gyratory Compactor used in the Plant Lab.

Field Permeability Test

During the construction monitoring trip the research team conducted field permeability testing on freshly compacted OGFC surface. The test was conducted using both the Texas Permeability device following the test procedure outlined in Tex-246-F “Permeability or Water Flow of Hot Mix Asphalt” and field permeability device developed at NCAT. Tests were conducted two different days at two different locations: one on northbound inside lane near Mile Marker 8.1 and the other on northbound inside lane near Mile Marker 9.0. At any given station, tests were conducted at left wheel path, between the wheelpaths, and right wheelpath. At any given location, testing was done at three stations apart by approximately 100 feet. Right wheelpath measurement was located 3 ft away from the free edge. Table F2 and Table F3 present the permeability testing results in seconds required for the water level to drop certain height for two locations. Lower number indicates higher flow rate or permeability. While using NCAT device time was measured in seconds to drop the water head from 17 cm level to 10 cm. During the testing it was observed that little amount of amount of water ran off through the surface up to 10-12 feet before completely disappear from the surface.

Table F2. Field Permeability Results for Location 1.

Location 1, NBIS lane at Mile Marker 8.1	**3 ft. from free edge (RWP)**		**Center of the lane (BWP)**		**9 ft. from left edge (LWP)**	
	Time in Seconds					
	Texas	**NCAT (17 cm to 10 cm)**	**Texas**	**NCAT (17 cm to 10 cm)**	**Texas**	**NCAT (17 cm to 10 cm)**
Station 1A	21.73	15.86	18.57	15.86	16.81	12.64
Station 1B	27.56	16.81	20.61	14.25	20.33	14.18
Station 1C	31.94	15.83	24.12	11.69	21.70	14.37

Table F3. Field Permeability Results for Location 2.

Location 2, NBIS lane at Mile Marker 9.0	3 ft. from free edge (RWP)		Center of the lane (BWP)		9 ft. from left edge (LWP)	
	Time in Seconds					
	Texas	**NCAT (17 cm to 10 cm)**	**Texas**	**NCAT (17 cm to 10 cm)**	**Texas**	**NCAT (17 cm to 10 cm)**
Station 2A	26.70	14.91	24.31	14.83	29.76	20.08
Station 2B	18.52	11.99	24.06	16.27	40.81	27.40
Station 2C	Center of left shoulder, 5 feet away from edge				19.55	15.07

APPENDIX G

TEXAS A&M RIVERSIDE CONSTRUCTION REPORT

General Description

Texas A&M Transportation Institute (TTI) constructed an asphalt test pad in the Fall of 2016 under a research project sponsored by Florida Department of Transportation (FDOT). This pad was intended to investigate some of the highway safety elements used in Florida. The FDOT sponsored project has the objective of developing a test method for evaluating the impact resistance of delineators when mounted on concrete and asphalt surfaces. As a requirement from FDOT, the top layer of the test pad was paved with their typical OGFC mixture. The research team for the FHWA sponsored study participated in this construction project to monitor and evaluate the OGFC layer. This surface layer was paved on October 27, 2016. BPI, a general contractor was responsible for the entire constriction and Knife River, Inc. produced and supplied the mixture. Figure G1 shows the overview of test pad.

Figure G1. Texas A&M Riverside Test Pad.

The 200 feet long and 60 feet wide test pad was constructed by removing part of the old concrete apron of an air force base built during 1950s. The Riverside campus is located in Bryan, Texas. The construction contractor, BPI excavated approximately 16 in below the existing apron surface. The contractor then lime stabilized 12 in of the existing soil. The contractor then placed subbase (TxDOT Type D mix) on top of stabilized base. Next the contractor paved one 4-inch and one 2-inch lifts of Type D Asphalt. This brought the facility back to even grade with the existing apron facility. Finally, a ¾” layer of OGFC was placed as a surface layer. The edges were tapered to help vehicles transition when crossing the test facility. Figure G2 and Figure G3 show the aerial view of Riverside campus and the test pad on an existing apron, respectively. Figure G4 gives detailed cross-sectional view of the facility recently built.

Figure G2. Aerial View of the Texas A&M Riverside Campus.

Figure G3. Test Pad Constructed on Existing Apron.

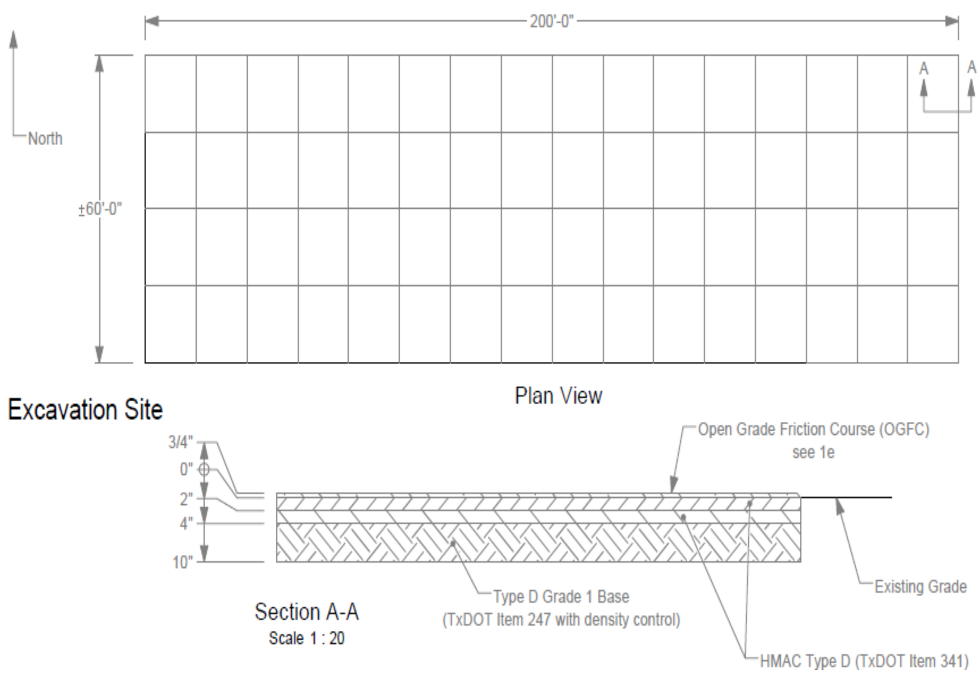


Figure G4. Plan and Cross Sectional View of the Test Pad.

Mixture and Materials

Figure G5 presents the mixture design of OGFC used in this project. Originally designed with TxDOT specifications, the mixture was modified to conform to FDOT requirements. Knife River, Inc. produced the mix at their plant located in Bryan, Texas which was only two miles away from the job site. The mix used aggregates from two stockpiles of one aggregate source. This sandstone aggregate came from Capitol Aggregate’s Brownlee quarry located in central Texas. The PG 76-22 grade SBS modified binder was obtained by from Valero Asphalt Refinery located at Houston, Texas. The design asphalt content was 6.1 percent by weight of the mixture. This mix also used 0.30 percent fiber. Note that this mixture did not use any recycled asphalt pavement or any other additive.

HMACP MIXTURE DESIGN : COMBINED GRADATION

File Version: 10/12/15 10:00:39

SAMPLE ID:		SAMPLE DATE:	
LOT NUMBER:		LETTING DATE:	
SAMPLE STATUS:		CONTROLLING CSJ:	
COUNTY:		SPEC YEAR:	2014
SAMPLED BY:		SPEC ITEM:	
SAMPLE LOCATION:		SPECIAL PROVISION:	
MATERIAL CODE:		MIX TYPE:	342-PFC-C
MATERIAL NAME:			
PRODUCER:			
AREA ENGINEER:		PROJECT MANAGER:	

WMA Additive in Design?	No
Target Discharge Temp., °F:	
WMA TECHNOLOGY:	
WMA RATE:	UNITS:

COURSE\LIFT:	Surface	STATION:		DIST. FROM CL:		CONTRACTOR DESIGN # :	KRPFC01-J76 (mod.)

Maximum Allowable, %	
Frac RAP:	10.0
Unfrac RAP	0.0
RAS:	5.0
RB Ratio:	15.0

Recycled Binder, %	
Bin No.8 :	0.0
Bin No.9 :	0.0
Bin No.10 :	0.0

Use this value in the QC/QA template>> ***Total*** ***0.000***

Ratio of Recycled to Total Binder, %
(based on binder percent (%) entered below in this worksheet)
0.0

Aggregate	AGGREGATE BIN FRACTIONS Bin No.1	Bin No.2	Bin No.3	Bin No.4	Bin No.5	Bin No.6	Bin No.7	"RECYCLED MATERIALS" Bin No.8	Bin No.9	Bin No.10	
Source:	Sandstone	Sandstone									Material Type
Pit:	Brownlee	Brownlee									Material Source
Number:	1402704	1402704									RAS Type
Producer:	Capitol Aggregates	Capitol Aggregates									RAP/RAS Producer
Sample ID:	C ROCK	GRADE 4									Sample ID
Recycled Material?:								Recycled Asphalt Binder (%)			
Asphalt%:											
Hydrated Lime?:								% of Tot. Mix	% of Tot. Mix	% of Tot. Mix	Total Bin
Individual Bin (%):	20.0 Percent	80.0 Percent	Percent	Percent	Percent	Percent	Percent	0.0 % of Aggreg	0.0 % of Aggreg	0.0 % of Aggreg	100.0%
	48.1										

Sieve Size:	Bin 1 Cum.% Passing	Bin 1 *Wtd* Cum. %	Bin 2 Cum.% Passing	Bin 2 *Wtd* Cum. %	Bin 3 Cum.% Passing	Bin 3 *Wtd* Cum. %	Bin 4 Cum.% Passing	Bin 4 *Wtd* Cum. %	Bin 5 Cum.% Passing	Bin 5 *Wtd* Cum. %	Bin 6 Cum.% Passing	Bin 6 *Wtd* Cum. %	Bin 7 Cum.% Passing	Bin 7 *Wtd* Cum. %	Bin 8 Cum.% Passing	Bin 8 *Wtd* Cum. %	Bin 9 Cum.% Passing	Bin 9 *Wtd* Cum. %	Bin 10 Cum.% Passing	Bin 10 *Wtd* Cum. %	Combined Gradation Cum. % Passing	Lower	Upper	Within Spec's
3/4"	100.0	20.0	100.0	80.0																	100.0	100.0	100.0	Yes
1/2"	49.5	9.9	99.2	79.4																	89.3	85.0	100.0	Yes
3/8"	19.4	3.9	77.0	61.6																	65.5	55.0	75.0	Yes
No. 4	13.7	2.7	14.4	11.5																	14.3	15.0	25.0	No
No. 8	11.1	2.2	3.5	2.8																	5.0	5.0	10.0	Yes
No. 200	5.3	1.1	1.6	1.3																	2.3	2.0	4.0	Yes

Lower & Upper Specification Limits

(Bold Italic) Not within specifications ***(Bold Italic)*** Not within specificaitons- Restricted Zone *(Italic)* Not cumulative

Lift Thickness, in:		Binder Substitution?		Binder Originally Specified:	PG 76-22
Substitute Binder:					
Asphalt Source:		Binder Percent, (%):	6.1	Asphalt Spec. Grav.:	1.031
Membrane Target Application Rate, gal/yd2					
Antistripping Agent:		Percent, (%):		Fiber Content, %:	0.30
Dry Rodded Unit Weight of Coarse Agg. (pcf)					

Remarks:

Notes:

Figure G5. OGFC Mixture Design used for Riverside Test Pad.

Mixture Production

Friction course mixture was produced at 150 tons per hour at Knife River's drum plant. Note that this entire test pad required only 60 tons of OGFC mixture. The temperature of mixture measured at trucks at construction site was between 290 to 295°F. End dump trucks used for hauling mixtures had tarp to cover the mixtures. The production temperature was around 315 to 320°F. During this time the ambient temperature was between upper lower 80s with light wind. Average hauling distance and time between the plant and job site were approximately 2 miles and 10 minutes, respectively. Due to the small size of the production, the contractor did not conduct any quality control testing at plant.

Construction

The paving of OGFC layer for the entire test pad was completed in couple of hours in the morning of October 27, 2016. This friction course mix was placed on top of a Type D surface mix paved on previous day. So, naturally existing surface didn't have any distress or surface defects. Before the placement of mix, the contractor spread SS-1h tack coat at 0.04 gal/yd^2 rate. The tack was spread at 176 °F temperature. The quality of tack coat application appeared non-uniform as shown in Figure G6.

Figure G6. Tack Coat application prior to paving of OGFC layer.

Belly dump trucks hauled the mixture to project site and they released the loose mixture into the chute of the paver directly (Figure G7). The paving width and thickness were 15 feet and 0.75 in, respectively. The temperature measured using infra-red gun right behind the paver ranged from 275 to 285 °F. Due to the thin layer the loose mix was losing temperature quickly. Two steel-wheeled seven feet wide roller compacted the mat in static mode (Figure G8). Two rollers compacted the loose mix in tandem and they followed the paver very closely. Temperature measured during the rolling varied significantly.

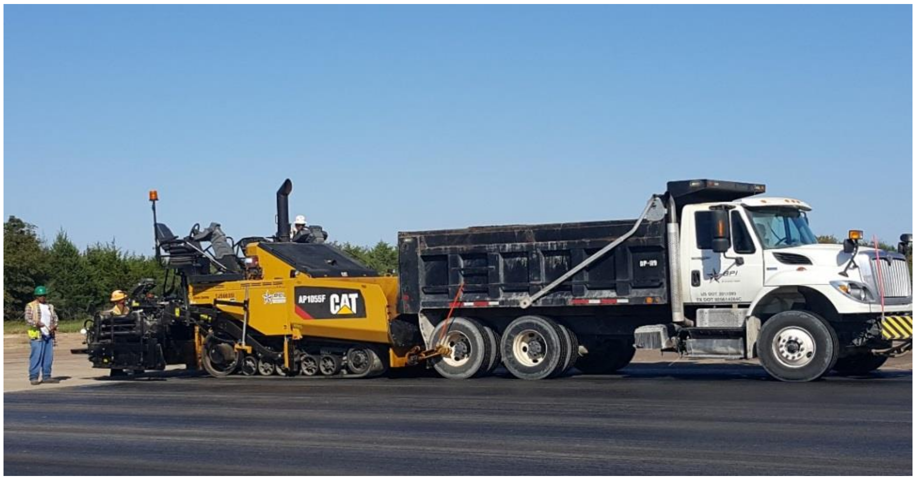


Figure G7. Paving at Riverside.

Figure G8. Compaction with two steel-wheeled roller in tandem.

A technician from the paving contractor measured the in-site density after the final pass using a nuclear density gauge. Although the density measurement for this thin layer is not very reliable, the contractor wanted to have some idea about the compaction effort. Later, the research team also measured the permeability value at the same locations using Texas permeability device.

During the paving of OGFC layer, the research team equipped the larger roller with TTI's construction monitoring device (CMS). This GPS based device records the location of roller, effective compaction effort, and the mat temperature at real-time. Figure G9 shows some results from the CMS system.

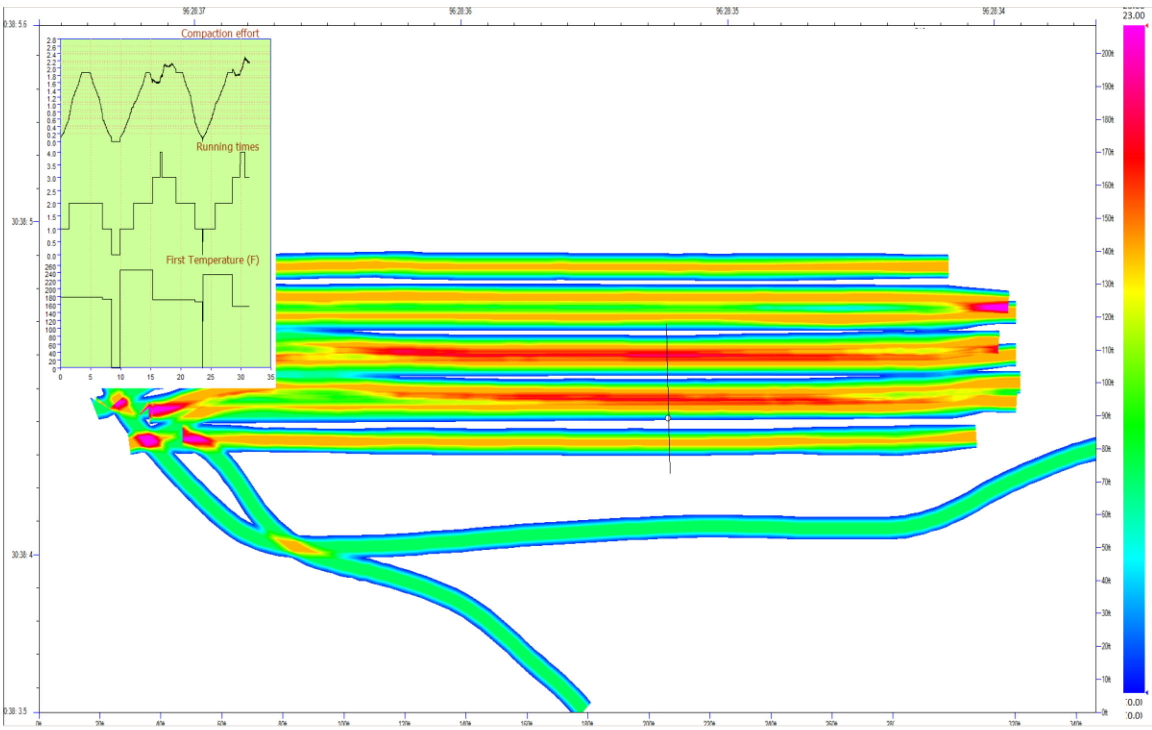


Figure G9. Compaction Effort with Roller coverage measured with TTI's CMS.

The research team measured the stiffness of the structure using a Falling Weight Deflectometer (FWD) prior to placement of OGFC layer and after the placement of OGFC layer (Figure G10). Besides, the FWD, the research team also used TTI's ground penetrating radar to scan the entire pad close interval.

Figure G10. FWD Measurement at Test Pad.

Sample Collection

Plant mix was collected from the truck at the construction site. Samples were collected from the fifth through seventh truck when the plant temperatures become stable. The materials sampling scheme is presented in Table G1. Mixture sample collected from the site was immediately brought to the TTI lab located 12 miles away from the test pad. Field cores were obtained couple of weeks after the completion of paving. Figure G11 shows some core pictures.

Table G1. Materials Sampling Scheme.

Sample Type	Material	Point of Sampling
Lab-Mixed, Lab-Compacted	Fiber	Plant
	Coarse Aggregate	Stockpile at Plant
	Blended Binder	Storage Tank at Plant
Plant-Mixed, Lab-Compacted	Loose Mix	Truck at Plant
Plant-Mixed, Field-Compacted	Road Cores	Close to edge of the test pad

Figure G11. Cores obtained from the Test Pad.

Field Permeability Test

During the construction monitoring, the research team conducted field permeability testing on freshly compacted OGFC surface. The test was conducted using both the Texas Permeability device following the test procedure outlined in Tex-246-F "Permeability or Water Flow of Hot Mix Asphalt" and field permeability device developed at NCAT. Tests were conducted on all four lanes. Table G2 present the permeability testing results in seconds required for the water level to drop certain height for Texas device. Lower number indicates higher flow rate or permeability. This table also contains the results when measured with NCAT device.

Table G2. Measurement of Permeability using Texas NCAT Devices.

	Measurement Location					
Property/ Parameter	**Lane 1 Center**	**Lane 2 Center**	**Lane 3 Center**	**Lane 4 Center**	**Lane 3 East End**	**Lane 4 East End**
Texas Permeability Value (in Seconds) after first roller pass	8.5	7.66	8.67	-	-	-
Texas Permeability Value (in Seconds) after second roller pass	10.45	13.24	11.66	-	-	-
Texas Permeability Value (in Seconds) after third or final roller pass	12.59	-	-	-	10.45	8.9, and 9.67
Permeability Value (cm/s) measured using NCAT Device	0.039405	0.029497	0.039713	0.038558	-	-

TableG3 shows the comparison of in-site density (and hence air voids) measurements using nuclear density gauge and corresponding permeability value when tested with Texas device at the same locations after final roller pass.

Table G3. Permeability and Air voids.

Location	**Texas Permeability Value (in Seconds) after third or final roller pass**	**In-situ Density**	**In-situ Airvoids**
Lane 1 Center	12.20 Sec	122.6 pcf	18.1 %
Lane 2 Center	13.89 Sec	122.9 pcf	17.8 %
Lane 3 Center	11.17 Sec	125.0 pcf	16.5 %
Lane 4 Center	11.79 Sec	122.8 pcf	17.9 %

APPENDIX H

COMPARISON OF SLAB COMPACTORS LINEAR KNEADING VS. ASPHALT ROLLER

A linear kneading compactor (LKC) has been used in this study for preparing slabs. In the middle of testing a malfunction in the LKC appeared, which due to the age of the equipment was not able to be resolved. Texas A&M Transportation Institute had recently acquired a new asphalt roller compactor (ARC), which molds slabs through a roller foot with multiple linear rotating compactors simulating the compaction of a steel wheel roller

The ARC was tested to verify that the slabs obtained with the new equipment would be equivalent to the ones manufactured by the LKC before its malfunction. Cantabro loss, AV, and permeability were used to assess the differences in the final product of both compactors. Plant mix from the New Jersey field project was used to produce slabs with a target air void content of 20% and thickness of 38.1 and 63.5 mm (i.e., 1.5 and 2.5 in). The results are summarized in Table H127H1.

Table H127. Results of comparison between LKC and ARC data.

Thickness (in)	2.5			1.5		
Property	**LKC**	**ARC**	**Difference**	**LKC**	**ARC**	**Difference**
Cantabro loss unaged (%)	10.3	7.3	41.2%	60	63	-5.1%
Standard Deviation	0.03	0.01		0.05	0.02	
Air void Content (%)	25.73	23.17	11.0%	28.01	25.79	8.6%
Standard Deviation	2.14	1.00		3.06	2.02	
Permeability FDOT (cm/s)	0.10	0.11	-2.8%	0.08	0.05	58.6%
Standard Deviation	0.033	0.017		0.016	0.0045	
Permeability TxDOT (cm/s)	0.55	0.56	-1.6%	0.27	0.29	-5.9%
Standard Deviation	0.0090	0.0063		0.0062	0.0096	
Permeability NCAT (cm/s)	0.09	0.10	-6.7%	0.05	0.07	-29.5%
Standard Deviation	0.0013	0.0040		0.0017	0.0020	

Analyzing the AV, it is evident the ARC is more accurate in achieving the target as compared with the LKC. The average AV for all the slabs was 24.5% using the ARC versus 26.9% using LKC. It is also important to underscore than the ARC had a lower standard deviation for the AV, which represents a more uniform compaction and hence AV distribution. There were not considerable differences in the permeability utilizing the FDOT, TxDOT and NCAT methods for the 2.5 inch slabs, and the standard deviation for all permeability tests was less for the slabs compacted with the ARC. Finally, the Cantabro loss results obtained for specimens obtained from the two compactors are comparable, and again, the standard deviation for the ARC-compacted slabs was less than the LKC-compacted slabs. From this comparison, it is safe to assume that producing slabs with the ARC compactor is not detrimental to the objectives of this study, as less variable data will be obtained with the use of the ARC compactor. The specifications of this compactor are presented in Figure H1.

SMARTER ROLLER COMPACTOR

The **Asphalt Roller Compactor (ARC)** is an electromechanical system that does not require a compressed air source or hydraulics. Compaction is performed through a roller foot with multiple linear rotating compactors simulating the compaction of a steel-wheel roller. Slabs are compacted to the selected density, loading pressure or thickness. The slabs can then be cored or cut to obtain cylinders or beams for wheel-tracking systems, fatigue cracking, indirect tensile, static and dynamic creep tests, stiffness tests and more.

The ARC can be programmed to target a specific load or specimen thickness. If a specifi c thickness is requested, the system will automatically select the correct load and produce a perfectly uniform asphalt specimen with representative particle orientation and air void content. Three transducers inside the roller compactor control the roller, table movement, and vertical load. The ARC is designed to produce various slab sizes to fi t your specifi c testing needs.

FEATURES:

- User-friendly software for easy test setup and immediate test execution
- Color touch screen display for easy control and operation
- Windows® driven software platform for data management, analysis and graphically displayed test results
- Fully integrated data acquisition system
- Sturdy steel frame with mold table and vertical load displacement system
- Internet connection allows remote diagnostic checks, trouble-shooting and software updates
- Unlimited storage with 2 USB ports, and SD card slot.

ARC

ASPHALT ROLLER COMPACTOR

SPECIFICATIONS

Vertical force	Up to 40 kN
Power supply	230V 50/60Hz, 550W
Dimensions	87"(d) x 41"(w) x 74"(h) *95" (h) w/ guard open
Weight	3,300 lbs.
Min mold size	12" x 12" x 2" thick
Max mold size	87"(d) x 41"(w) x 74"(h)
*Other mold sizes available	

ARC for compacting repeatable asphalt samples

InstroTek®, Inc., Raleigh, NC *phone:* 919.875.8371 *e-mail:* sales@InstroTek.com *visit us at:* www.InstroTek.com
Other Locations: Las Vegas, NV *phone:* 702.270.3885 • Concord, CA *phone:* 925.363.9770 • Grand Rapids. MI *phone:* 616.726.5850

Figure H1. Specifications of ARC.

APPENDIX I

STATISTICAL ANALYSIS PERMEABILITY SLABS

Results of Fitting ANOVA to the Permeability Slabs Data with 162 Measurements

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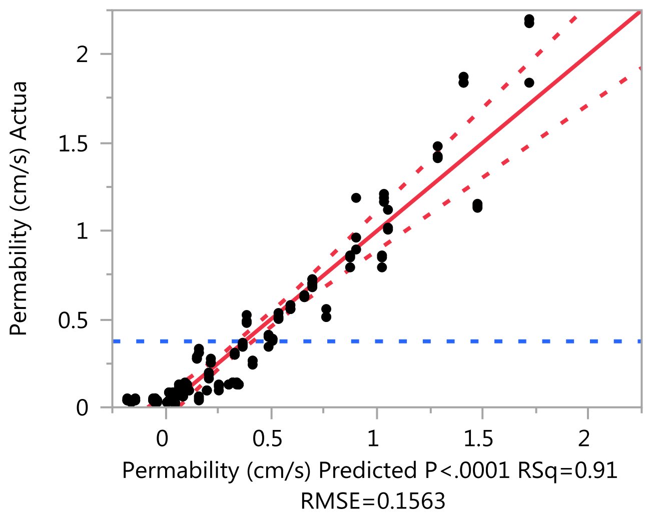
Response Permeability (cm/s)
Actual by Predicted Plot



Summary of Fit

RSquare	0.906564
RSquare Adj	0.889389
Root Mean Square Error	0.156276
Mean of Response	0.373787
Observations (or Sum Wgts)	162

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	25	32.226406	1.28906	52.7819
Error	136	3.321432	0.02442	**Prob > F**
C. Total	161	35.547839		<.0001*

Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Thickness (in)	2	2	2.136271	43.7361	<.0001*

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Air Voids	1	1	0.975107	39.9269	<.0001*
Type of Permeameter	2	2	16.949621	347.0112	<.0001*
Material	2	2	3.486638	71.3823	<.0001*
Type of Permeameter*Thickness (in)	4	4	1.560776	15.9770	<.0001*
Type of Permeameter*Air Voids	2	2	0.770926	15.7832	<.0001*
Type of Permeameter*Material	4	4	2.399375	24.5613	<.0001*
Thickness (in)*Air Voids	2	2	0.070459	1.4425	0.2399
Thickness (in)*Material	4	4	3.718967	38.0694	<.0001*
Air Voids*Material	2	2	0.158266	3.2402	0.0422*

Effect Details

Thickness (in)

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
0.75	0.30927007	0.02126652	0.309270
1.5	0.27697810	0.02126652	0.276978
2.5	0.53511329	0.02126652	0.535113

Air Voids

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Construction	0.45137050	0.01736404	0.451371
Design	0.29620380	0.01736404	0.296204

Type of Permeameter

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
FDOT	0.06812367	0.02126652	0.068124
NCAT	0.23188459	0.02126652	0.231885
TXDOT	0.82135321	0.02126652	0.821353

Material

Least Squares Means Table

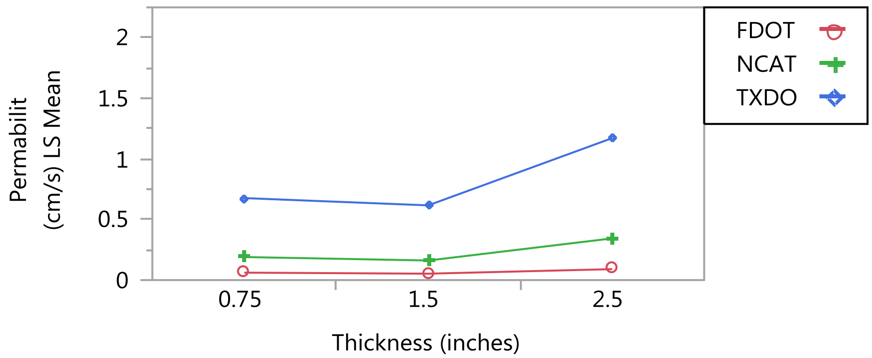
Level	Least Sq Mean	Std Error	Mean
Florida	0.52717179	0.02126652	0.527172
New Jersey	0.17610497	0.02126652	0.176105
Texas	0.41808470	0.02126652	0.418085

Type of Permeameter*Thickness (in)

Least Squares Means Table

Level	Least Sq Mean	Std Error
FDOT,0.75	0.0612714	0.03683469
FDOT,1.5	0.0528903	0.03683469
FDOT,2.5	0.0902092	0.03683469
NCAT,0.75	0.1901359	0.03683469
NCAT,1.5	0.1615869	0.03683469
NCAT,2.5	0.3439309	0.03683469
TXDOT,0.75	0.6764029	0.03683469
TXDOT,1.5	0.6164570	0.03683469
TXDOT,2.5	1.1711997	0.03683469

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

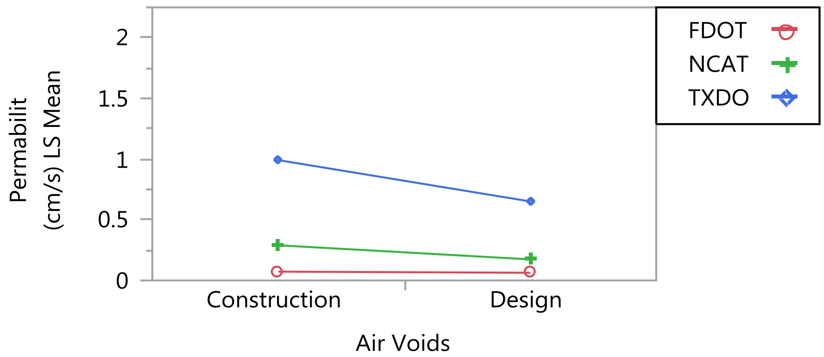
Level					Least Sq Mean
TXDOT,2.5	A				1.1711997
TXDOT,0.75		B			0.6764029
TXDOT,1.5		B			0.6164570
NCAT,2.5			C		0.3439309
NCAT,0.75			C	D	0.1901359
NCAT,1.5				D	0.1615869
FDOT,2.5				D	0.0902092
FDOT,0.75				D	0.0612714
FDOT,1.5				D	0.0528903

Levels not connected by same letter are significantly different.

Type of Permeameter*Air Voids
Least Squares Means Table

Level	Least Sq Mean	Std Error
FDOT,Construction	0.07270848	0.03007540
FDOT,Design	0.06353886	0.03007540
NCAT,Construction	0.28991711	0.03007540
NCAT,Design	0.17385206	0.03007540
TXDOT,Construction	0.99148592	0.03007540
TXDOT,Design	0.65122049	0.03007540

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

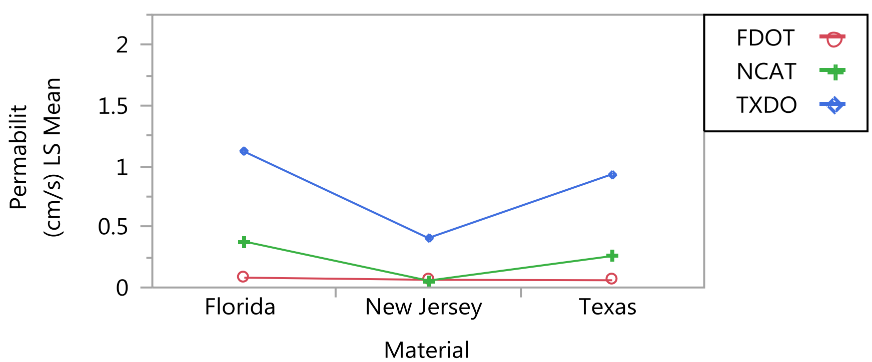
Level					Least Sq Mean
TXDOT,Construction	A				0.99148592
TXDOT,Design		B			0.65122049
NCAT,Construction			C		0.28991711
NCAT,Design			C	D	0.17385206
FDOT,Construction				D	0.07270848
FDOT,Design				D	0.06353886

Levels not connected by same letter are significantly different.

Type of Permeameter*Material
Least Squares Means Table

Level	Least Sq Mean	Std Error
FDOT,Florida	0.0811018	0.03683469
FDOT,New Jersey	0.0635543	0.03683469
FDOT,Texas	0.0597149	0.03683469
NCAT,Florida	0.3793525	0.03683469
NCAT,New Jersey	0.0566984	0.03683469
NCAT,Texas	0.2596029	0.03683469
TXDOT,Florida	1.1210611	0.03683469
TXDOT,New Jersey	0.4080622	0.03683469
TXDOT,Texas	0.9349363	0.03683469

LS Means Plot



LSMeans Differences Tukey HSD
α=0.050

Level					Least Sq Mean
TXDOT,Florida	A				1.1210611
TXDOT,Texas		B			0.9349363
TXDOT,New Jersey			C		0.4080622
NCAT,Florida			C		0.3793525
NCAT,Texas			C		0.2596029
FDOT,Florida				D	0.0811018
FDOT,New Jersey				D	0.0635543
FDOT,Texas				D	0.0597149
NCAT,New Jersey				D	0.0566984

Levels not connected by same letter are significantly different.

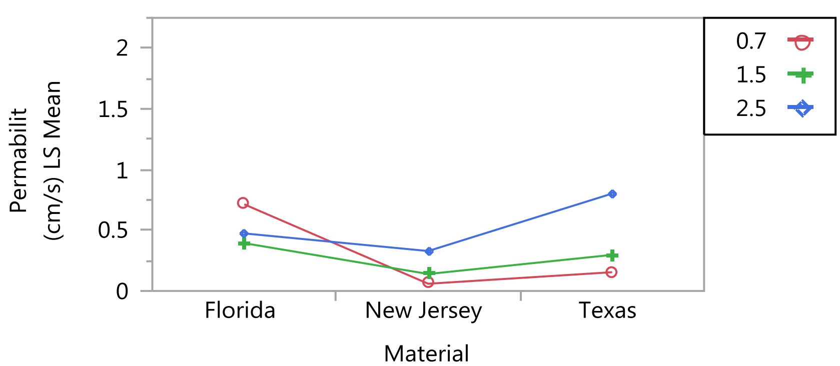
Thickness (in)*Air Voids
Least Squares Means Table

Level	Least Sq Mean	Std Error
0.75,Construction	0.37740669	0.03007540
0.75,Design	0.24113346	0.03007540
1.5,Construction	0.33508829	0.03007540
1.5,Design	0.21886790	0.03007540
2.5,Construction	0.64161653	0.03007540
2.5,Design	0.42861005	0.03007540

Thickness (in)*Material
Least Squares Means Table

Level	Least Sq Mean	Std Error
0.75,Florida	0.71368797	0.03683469
0.75,New Jersey	0.05951831	0.03683469
0.75,Texas	0.15460394	0.03683469
1.5,Florida	0.39374726	0.03683469
1.5,New Jersey	0.13981969	0.03683469
1.5,Texas	0.29736733	0.03683469
2.5,Florida	0.47408014	0.03683469
2.5,New Jersey	0.32897690	0.03683469
2.5,Texas	0.80228283	0.03683469

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level						Least Sq Mean
2.5,Texas	A					0.80228283
0.75,Florida	A					0.71368797
2.5,Florida		B				0.47408014
1.5,Florida		B	C			0.39374726
2.5,New Jersey		B	C			0.32897690
1.5,Texas			C	D		0.29736733
0.75,Texas				D	E	0.15460394
1.5,New Jersey				D	E	0.13981969
0.75,New Jersey					E	0.05951831

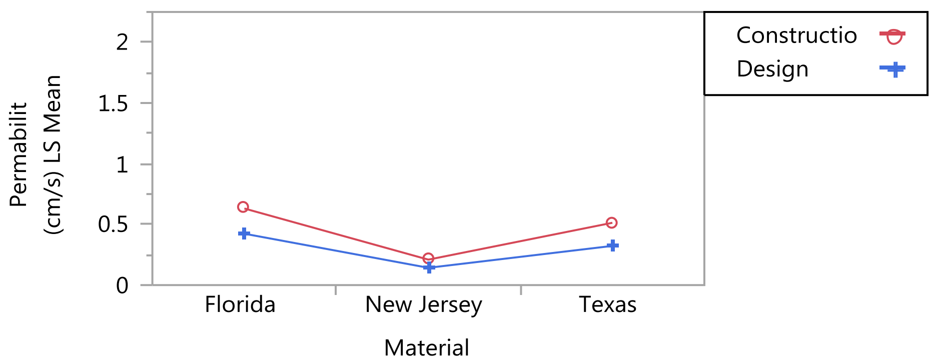
Levels not connected by same letter are significantly different.

Air Voids*Material

Least Squares Means Table

Level	Least Sq Mean	Std Error
Construction,Florida	0.63099360	0.03007540
Construction,New Jersey	0.20976183	0.03007540
Construction,Texas	0.51335607	0.03007540
Design,Florida	0.42334998	0.03007540
Design,New Jersey	0.14244810	0.03007540
Design,Texas	0.32281333	0.03007540

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level						Least Sq Mean
Construction,Florida	A					0.63099360
Construction,Texas	A	B				0.51335607
Design,Florida		B	C			0.42334998
Design,Texas			C	D		0.32281333
Construction,New Jersey				D	E	0.20976183
Design,New Jersey					E	0.14244810

Levels not connected by same letter are significantly different.

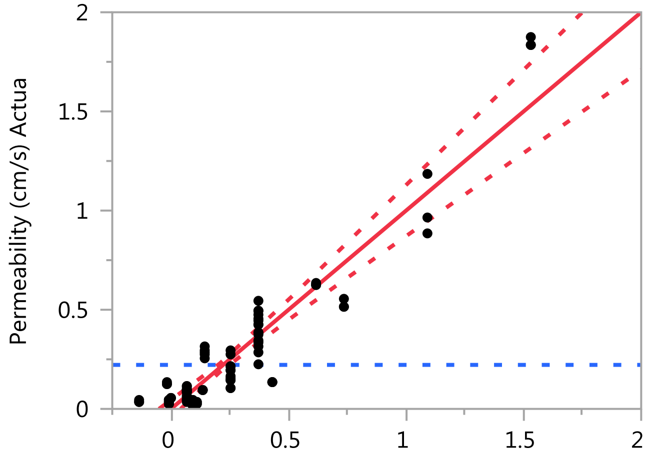
APPENDIX J

STATISTICAL ANALYSIS PERMEABILITY LABORATORY VS FIELD

Results of Fitting ANOVA to the Permeability vs Field Data with 116 Measurements

Response Permeability (cm/s)

Actual by Predicted Plot



Summary of Fit

RSquare	0.896982
RSquare Adj	0.883852
Root Mean Square Error	0.115735
Mean of Response	0.221836
Observations (or Sum Wgts)	116

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	13	11.895880	0.915068	68.3166
Error	102	1.366241	0.013395	**Prob > F**
C. Total	115	13.262121		<.0001*

Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Type of Permeameter	2	2	5.7066190	213.0207	<.0001*
Material	2	2	0.9916032	37.0153	<.0001*
Type of measurement	1	1	2.1540666	160.8170	<.0001*
Type of Permeameter*Material	4	4	0.2138447	3.9913	0.0048*
Type of Permeameter*Type of measurement	2	2	2.5145174	93.8637	<.0001*
Material*Type of measurement	2	2	1.2623164	47.1206	<.0001*

Effect Details

Type of Permeameter

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
FDOT	0.08139424	0.02251712	0.043460
NCAT	0.15270396	0.03025235	0.130443
TXDOT	0.69225420	0.02225545	0.476057

Material

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Florida	0.47520681	0.02818810	0.238488
New Jersey	0.29468209	0.02183997	0.244342
Texas	0.15646350	0.02449698	0.146541

Type of measurement

Least Squares Means Table

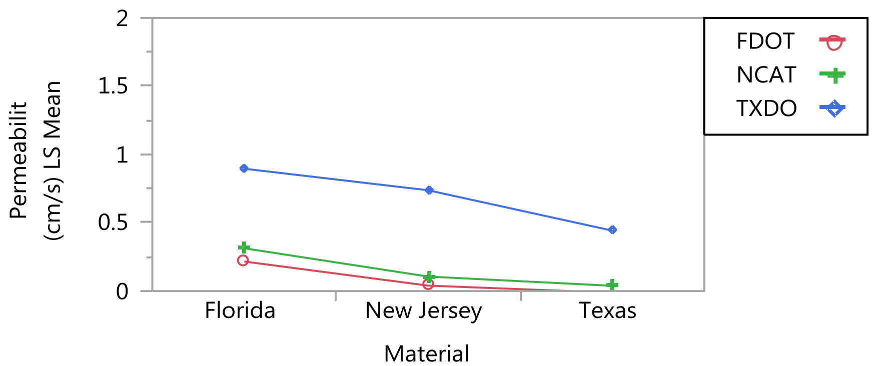
Level	Least Sq Mean	Std Error	Mean
Field	0.12605083	0.01828803	0.140023
Laboratory	0.49151743	0.02227315	0.491517

Type of Permeameter*Material

Least Squares Means Table

Level	Least Sq Mean	Std Error
FDOT,Florida	0.2163655	0.03211767
FDOT,New Jersey	0.0397956	0.03989029
FDOT,Texas	-0.0119784	0.03993410
NCAT,Florida	0.3132804	0.07950034
NCAT,New Jersey	0.1061792	0.03401825
NCAT,Texas	0.0386523	0.04401633
TXDOT,Florida	0.8959746	0.03507920
TXDOT,New Jersey	0.7380714	0.03308908
TXDOT,Texas	0.4427166	0.04164814

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level **Least Sq Mean**

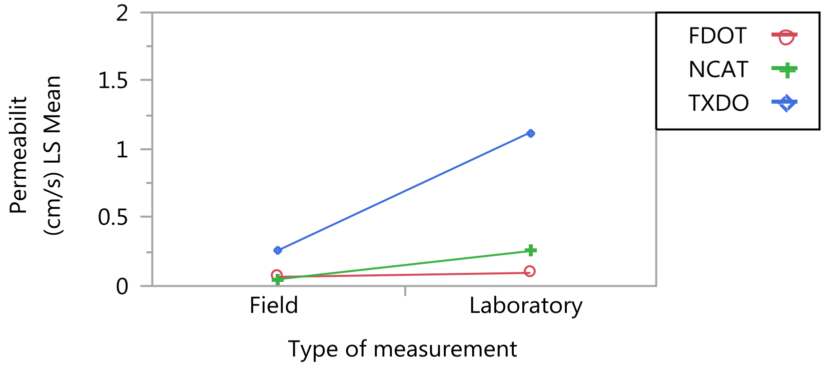
Level							Least Sq Mean
TXDOT,Florida	A						0.8959746
TXDOT,New Jersey		B					0.7380714
TXDOT,Texas			C				0.4427166
NCAT,Florida			C	D	E		0.3132804
FDOT,Florida				D			0.2163655
NCAT,New Jersey				D	E	F	0.1061792
FDOT,New Jersey					E	F	0.0397956
NCAT,Texas					E	F	0.0386523
FDOT,Texas						F	-0.0119784

Levels not connected by same letter are significantly different.

Type of Permeameter*Type of measurement
Least Squares Means Table

Level	Least Sq Mean	Std Error
FDOT,Field	0.0668608	0.02323367
FDOT,Laboratory	0.0959277	0.03857823
NCAT,Field	0.0497906	0.04661050
NCAT,Laboratory	0.2556173	0.03857823
TXDOT,Field	0.2615010	0.02220225
TXDOT,Laboratory	1.1230074	0.03857823

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level				Least Sq Mean
TXDOT,Laboratory	A			1.1230074
TXDOT,Field		B		0.2615010
NCAT,Laboratory		B		0.2556173
FDOT,Laboratory			C	0.0959277
FDOT,Field			C	0.0668608
NCAT,Field			C	0.0497906

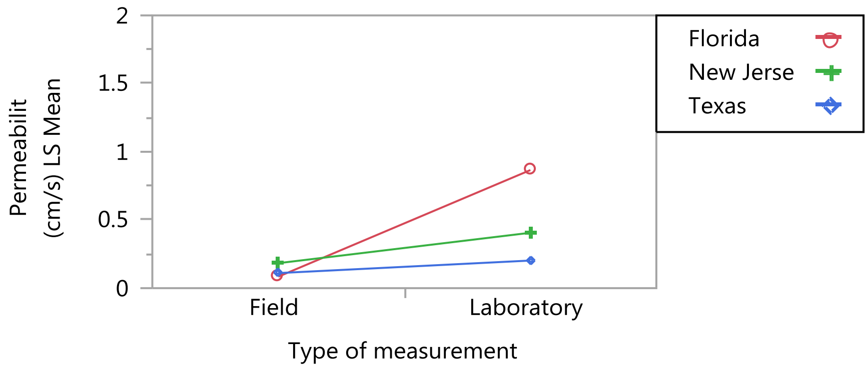
Levels not connected by same letter are significantly different.

Material*Type of measurement
Least Squares Means Table

Level	Least Sq Mean	Std Error
Florida,Field	0.08446798	0.04110957
Florida,Laboratory	0.86594564	0.03857823

Level	Least Sq Mean	Std Error
New Jersey,Field	0.18320407	0.02048556
New Jersey,Laboratory	0.40616010	0.03857823
Texas,Field	0.11048045	0.03020146
Texas,Laboratory	0.20244656	0.03857823

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level				Least Sq Mean
Florida,Laboratory	A			0.86594564
New Jersey,Laboratory		B		0.40616010
Texas,Laboratory			C	0.20244656
New Jersey,Field			C	0.18320407
Texas,Field			C	0.11048045
Florida,Field			C	0.08446798

Levels not connected by same letter are significantly different.

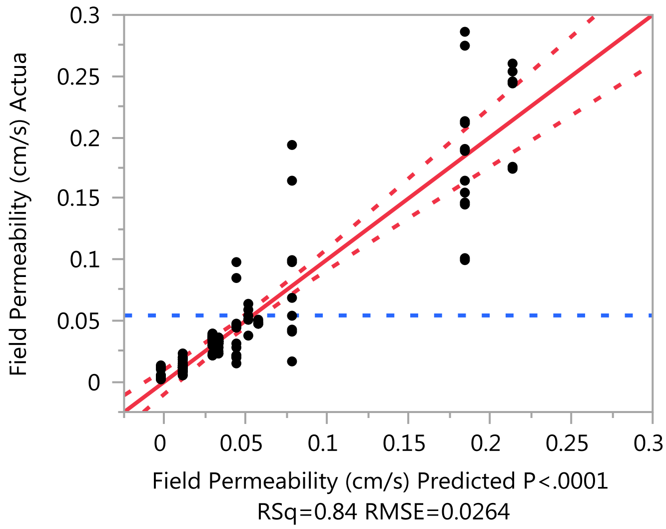
APPENDIX K

STATISTICAL ANALYSIS PERMEABILITY VS AGING STATE

Results of fitting ANOVA to the Permeability vs time data with 135 measurements

Response Field Permeability (cm/s)

Actual by Predicted Plot



Summary of Fit

RSquare	0.841914
RSquare Adj	0.830532
Root Mean Square Error	0.026403
Mean of Response	0.05404
Observations (or Sum Wgts)	135

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	9	0.46409352	0.051566	73.9675
Error	125	0.08714291	0.000697	**Prob > F**
C. Total	134	0.55123642		<.0001*

Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Time of Measurement	1	1	0.07573323	108.6337	<.0001*
Type of Permeameter	2	2	0.23923514	171.5825	<.0001*
Material	1	1	0.00133593	1.9163	0.1687
Type of Permeameter*Time of Measurement	2	2	0.08434681	60.4946	<.0001*
Type of Permeameter*Material	2	2	0.00449941	3.2270	0.0430*

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Time of Measurement*Material	1	1	0.01979998	28.4016	<.0001*

Effect Details

Time of Measurement

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Construction	0.08801770	0.00499108	0.090392
One year after construction	0.02963489	0.00315387	0.029806

Type of Permeameter

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
FDOT	0.02965696	0.00400802	0.029846
NCAT	0.01614916	0.00756406	0.019516
TXDOT	0.13067277	0.00432492	0.121807

Material

Least Squares Means Table

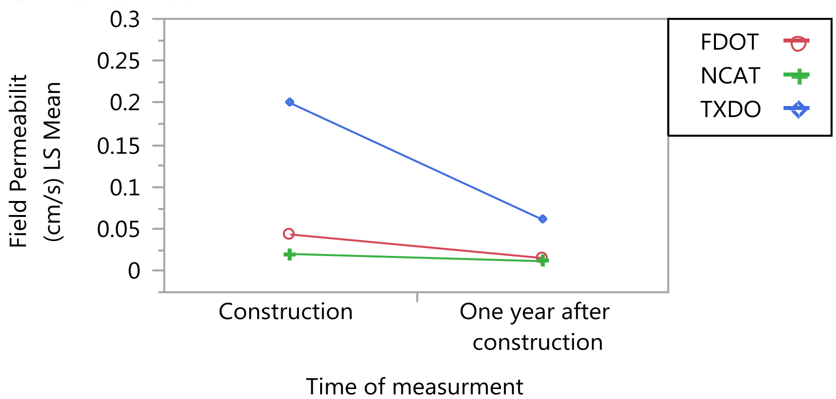
Level	Least Sq Mean	Std Error	Mean
Florida	0.05472604	0.00467226	0.057448
Utah	0.06292655	0.00385824	0.048928

Type of Permeameter*Time of Measurement

Least Squares Means Table

Level	Least Sq Mean	Std Error
FDOT,Construction	0.04367219	0.00574173
FDOT,One year after construction	0.01564172	0.00515614
NCAT,Construction	0.02045040	0.01347805
NCAT,One year after construction	0.01184791	0.00550072
TXDOT,Construction	0.19993051	0.00648738
TXDOT,One year after construction	0.06141503	0.00580723

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level		Least Sq Mean
TXDOT,Construction	A	0.19993051

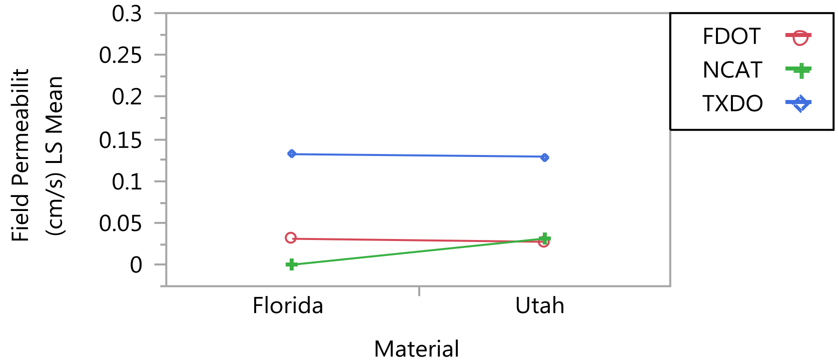
Level		Least Sq Mean
TXDOT,One year after construction	B	0.06141503
FDOT,Construction	B	0.04367219
NCAT,Construction	B C	0.02045040
FDOT,One year after construction	C	0.01564172
NCAT,One year after construction	C	0.01184791

Levels not connected by same letter are significantly different.

Type of Permeameter*Material
Least Squares Means Table

Level	Least Sq Mean	Std Error
FDOT,Florida	0.03145563	0.00370302
FDOT,Utah	0.02785828	0.00710366
NCAT,Florida	0.00061882	0.01235355
NCAT,Utah	0.03167950	0.00631985
TXDOT,Florida	0.13210366	0.00580723
TXDOT,Utah	0.12924188	0.00648738

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level		Least Sq Mean
TXDOT,Florida	A	0.13210366
TXDOT,Utah	A	0.12924188
NCAT,Utah	B	0.03167950
FDOT,Florida	B	0.03145563
FDOT,Utah	B	0.02785828
NCAT,Florida	B	0.00061882

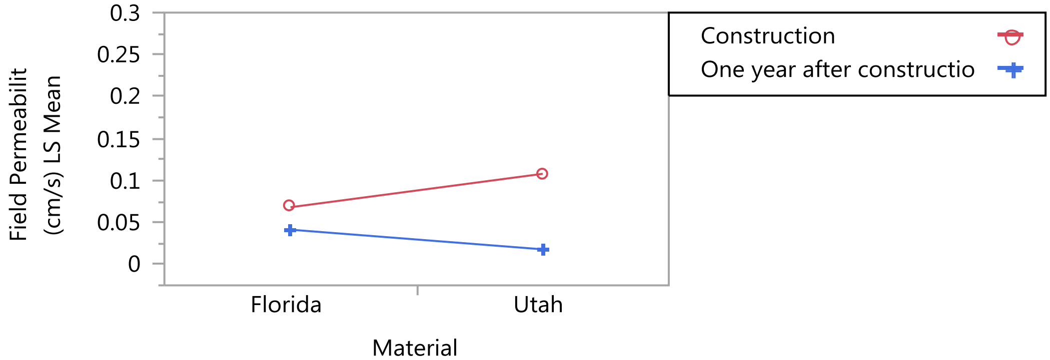
Levels not connected by same letter are significantly different.

Time of Measurement*Material
Least Squares Means Table

Level	Least Sq Mean	Std Error
Construction,Florida	0.06810606	0.00780469
Construction,Utah	0.10792934	0.00622336
One year after construction,Florida	0.04134601	0.00442275

Level	Least Sq Mean	Std Error
One year after construction,Utah	0.01792377	0.00456221

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level		Least Sq Mean
Construction,Utah	A	0.10792934
Construction,Florida	B	0.06810606
One year after construction,Florida	C	0.04134601
One year after construction,Utah	D	0.01792377

Levels not connected by same letter are significantly different.

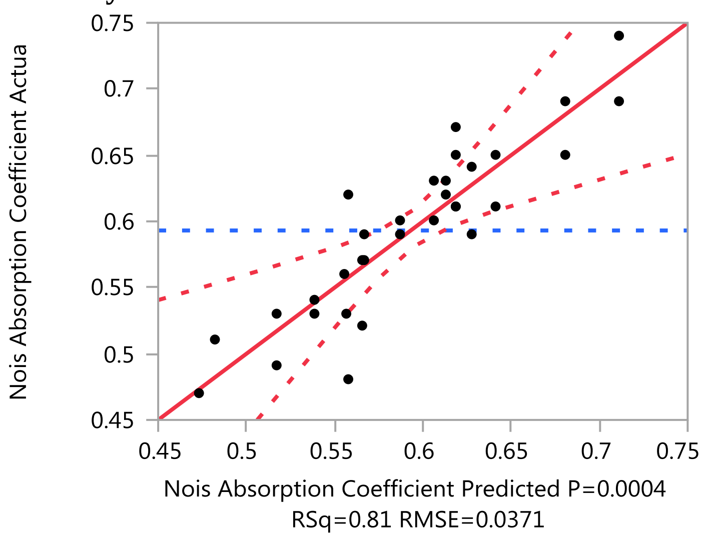
APPENDIX L

STATISTICAL ANALYSIS OF NOISE ABSORPTION DATA

Results of Fitting ANOVA to the Noise Absorption Data with 32 Measurements

Response Noise Absorption Coefficient

Actual by Predicted Plot



Summary of Fit

RSquare	0.810528
RSquare Adj	0.673687
Root Mean Square Error	0.037146
Mean of Response	0.593125
Observations (or Sum Wgts)	32

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	13	0.10625008	0.008173	5.9231
Error	18	0.02483742	0.001380	**Prob > F**
C. Total	31	0.13108750		0.0004*

Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Thickness (in)	2	2	0.01365128	4.9466	0.0194*
Air Voids	1	1	0.00117552	0.8519	0.3682
Material	2	2	0.01651438	5.9841	0.0102*
Thickness (in)*Air Voids	2	2	0.03285424	11.9049	0.0005*
Thickness (in)*Material	4	4	0.02868561	5.1972	0.0058*
Air Voids*Material	2	2	0.02082091	7.5446	0.0042*

Effect Details

Thickness (in)

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
0.75	0.55416667	0.01384366	0.572500
1.5	0.60916667	0.01072325	0.609167
2.5	0.59083333	0.01072325	0.590833

Air Voids

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Construction	0.59093434	0.00959924	0.605000
Design	0.57851010	0.00959924	0.581250

Material

Least Squares Means Table

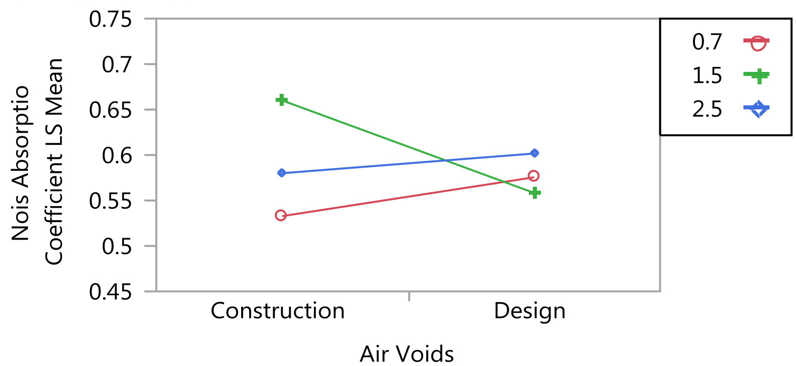
Level	Least Sq Mean	Std Error	Mean
Florida	0.61166667	0.01072325	0.611667
New Jersey	0.55500000	0.01238214	0.562000
Texas	0.58750000	0.01238214	0.602000

Thickness (in)*Air Voids

Least Squares Means Table

Level	Least Sq Mean	Std Error
0.75,Construction	0.53280303	0.01921863
0.75,Design	0.57553030	0.01921863
1.5,Construction	0.66000000	0.01516496
1.5,Design	0.55833333	0.01516496
2.5,Construction	0.58000000	0.01516496
2.5,Design	0.60166667	0.01516496

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level			Least Sq Mean
1.5,Construction	A		0.66000000
2.5,Design	A	B	0.60166667
2.5,Construction		B	0.58000000
0.75,Design		B	0.57553030

Level		Least Sq Mean
1.5,Design	B	0.55833333
0.75,Construction	B	0.53280303

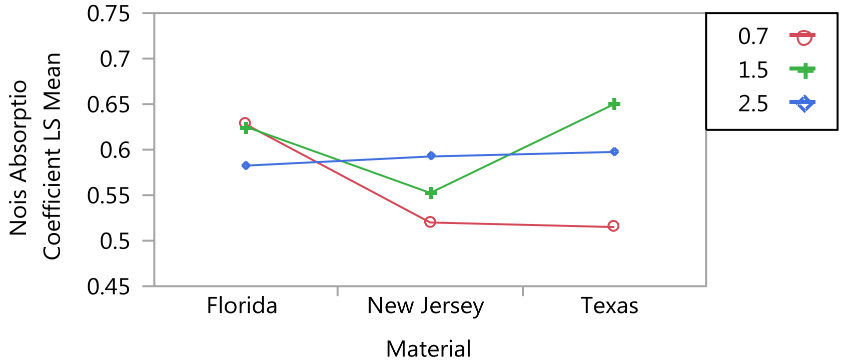
Levels not connected by same letter are significantly different.

Thickness (in)*Material

Least Squares Means Table

Level	Least Sq Mean	Std Error
0.75,Florida	0.62750000	0.01857321
0.75,New Jersey	0.52000000	0.02626649
0.75,Texas	0.51500000	0.02626649
1.5,Florida	0.62500000	0.01857321
1.5,New Jersey	0.55250000	0.01857321
1.5,Texas	0.65000000	0.01857321
2.5,Florida	0.58250000	0.01857321
2.5,New Jersey	0.59250000	0.01857321
2.5,Texas	0.59750000	0.01857321

LS Means Plot



LSMeans Differences Tukey HSD

α=
0.050

Level			Least Sq Mean
1.5,Texas	A		0.65000000
0.75,Florida	A	B	0.62750000
1.5,Florida	A	B	0.62500000
2.5,Texas	A	B	0.59750000
2.5,New Jersey	A	B	0.59250000
2.5,Florida	A	B	0.58250000
1.5,New Jersey		B	0.55250000
0.75,New Jersey		B	0.52000000
0.75,Texas		B	0.51500000

Levels not connected by same letter are significantly different.

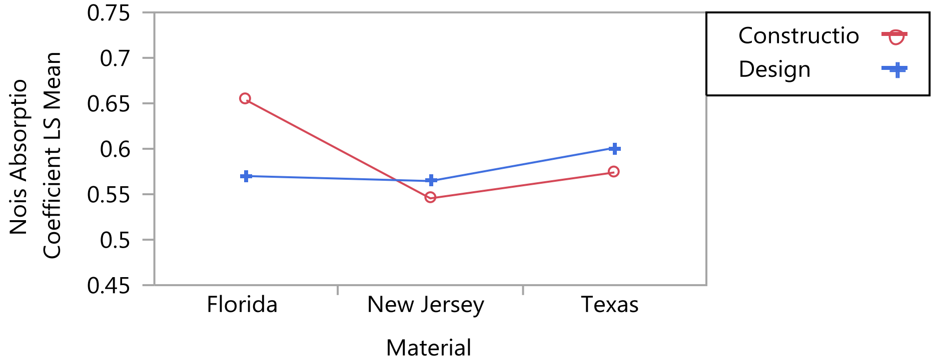
Air Voids*Material

Least Squares Means Table

Level	Least Sq Mean	Std Error
Construction,Florida	0.65333333	0.01516496
Construction,New Jersey	0.54548485	0.01718966

Level	Least Sq Mean	Std Error
Construction,Texas	0.57398485	0.01718966
Design,Florida	0.57000000	0.01516496
Design,New Jersey	0.56451515	0.01718966
Design,Texas	0.60101515	0.01718966

LS Means Plot



LSMeans Differences Tukey HSD

α=
0.050

Level			Least Sq Mean
Construction,Florida	A		0.65333333
Design,Texas	A	B	0.60101515
Construction,Texas		B	0.57398485
Design,Florida		B	0.57000000
Design,New Jersey		B	0.56451515
Construction,New Jersey		B	0.54548485

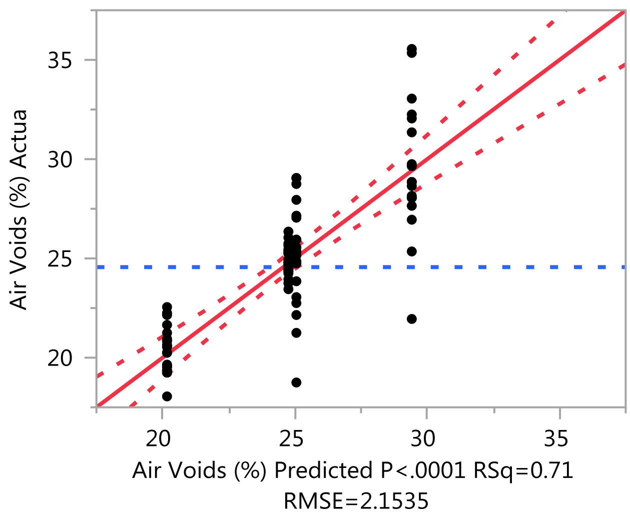
Levels not connected by same letter are significantly different.

APPENDIX M

STATISTICAL ANALYSIS TYPE OF COMPACTION VS AIR VOIDS

Results of Fitting ANOVA to the Type of Compaction Data

Table 1a:
Response Air Voids (%) Material=Florida
Actual by Predicted Plot



Summary of Fit

RSquare	0.705447
RSquare Adj	0.694401
Root Mean Square Error	2.153542
Mean of Response	24.56095
Observations (or Sum Wgts)	84

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	3	888.5816	296.194	63.8660
Error	80	371.0193	4.638	**Prob > F**
C. Total	83	1259.6009		<.0001*

Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Air Voids Measurement	1	1	413.45300	89.1496	<.0001*
Type of Compaction	1	1	463.77650	100.0005	<.0001*
Air Voids Measurement*Type of Compaction	1	1	0.18484	0.0399	0.8423

Effect Details

Air Voids Measurement

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Construction	27.141667	0.33574230	26.8093
Design	22.658542	0.33574230	22.3126

Type of Compaction

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
SGC	22.526042	0.31083697	22.5260
Slabs	27.274167	0.35892362	27.2742

Air Voids Measurement*Type of Compaction

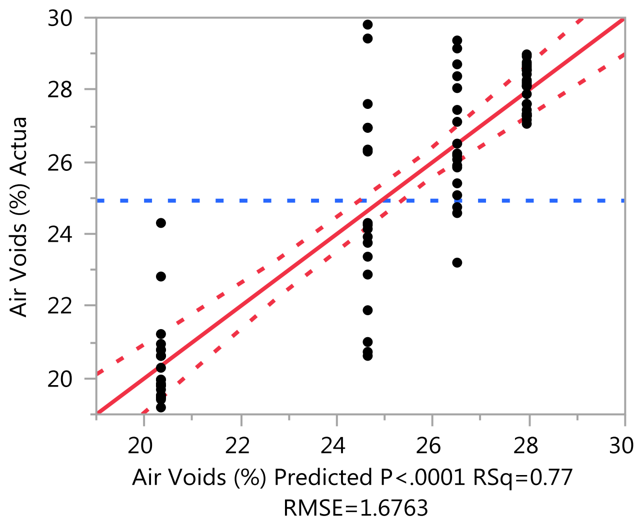
Least Squares Means Table

Level	Least Sq Mean	Std Error
Construction,SGC	24.815000	0.43958986
Construction,Slabs	29.468333	0.50759464
Design,SGC	20.237083	0.43958986
Design,Slabs	25.080000	0.50759464

Table 1b:

Response Air Voids (%) Material=Texas

Actual by Predicted Plot



Summary of Fit

RSquare	0.768865
RSquare Adj	0.760087
Root Mean Square Error	1.676328
Mean of Response	24.92566

Observations (or Sum Wgts)	83

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	3	738.46269	246.154	87.5970
Error	79	221.99595	2.810	**Prob > F**
C. Total	82	960.45864		<.0001*

Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Air Voids Measurement	1	1	457.54501	162.8230	<.0001*
Type of Compaction	1	1	40.67989	14.4764	0.0003*
Air Voids Measurement*Type of Compaction	1	1	168.40829	59.9302	<.0001*

Effect Details

Air Voids Measurement

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Construction	27.252556	0.25909376	27.3716
Design	22.510783	0.26638599	22.2963

Type of Compaction

Least Squares Means Table

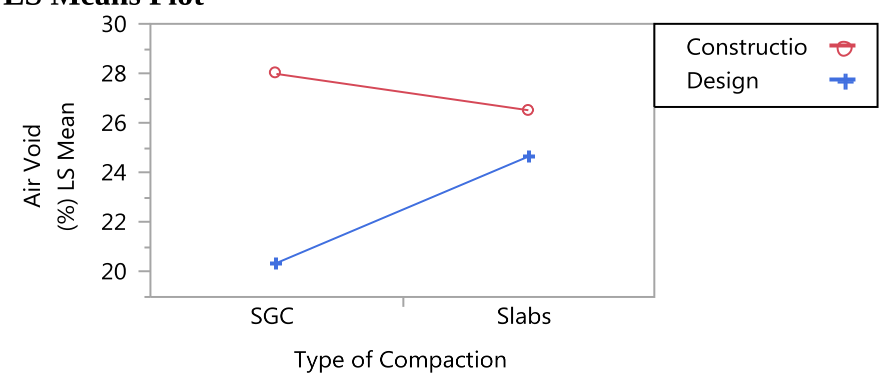
Level	Least Sq Mean	Std Error	Mean
SGC	24.174727	0.24501720	24.4179
Slabs	25.588611	0.27938799	25.5886

Air Voids Measurement*Type of Compaction

Least Squares Means Table

Level	Least Sq Mean	Std Error
Construction,SGC	27.984000	0.33526559
Construction,Slabs	26.521111	0.39511428
Design,SGC	20.365455	0.35739432
Design,Slabs	24.656111	0.39511428

LS Means Plot



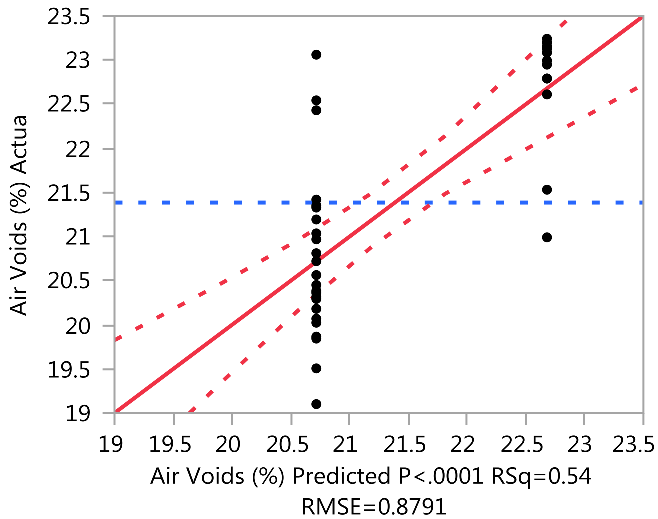
LSMeans Differences Tukey HSD

α=0.050

Level		Least Sq Mean
Construction,SGC	A	27.984000
Construction,Slabs	B	26.521111
Design,Slabs	C	24.656111
Design,SGC	D	20.365455

Levels not connected by same letter are significantly different.

Table 1c:
Response Air Voids (%) Material=New Jersey
Actual by Predicted Plot



Summary of Fit

RSquare	0.539192
RSquare Adj	0.525639
Root Mean Square Error	0.879108
Mean of Response	21.38556
Observations (or Sum Wgts)	36

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	1	30.745868	30.7459	39.7835
Error	34	26.276221	0.7728	**Prob > F**
C. Total	35	57.022089		<.0001*

Parameter Estimates

Term	Estimate	Std Error	t Ratio	Prob>|t|
Intercept	21.712292	0.155406	139.71	<.0001*
Type of Compaction[SGC]	-0.980208	0.155406	-6.31	<.0001*

Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Type of Compaction	1	1	30.745868	39.7835	<.0001*

Effect Details

Type of Compaction

Least Squares Means Table

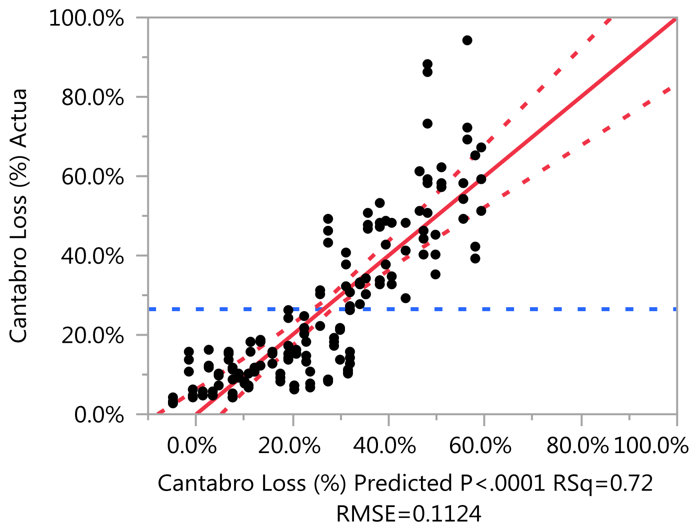
Level	Least Sq Mean	Std Error	Mean
SGC	20.732083	0.17944707	20.7321
Slabs	22.692500	0.25377648	22.6925

APPENDIX N

STATISTICAL ANALYSIS CANTABRO TEST SGC SPECIMENS

Results of Fitting ANOVA to the Cantabro SGC Data with 156 Measurements

Response Cantabro Loss (%)
Actual by Predicted Plot



Summary of Fit

RSquare	0.718802
RSquare Adj	0.699409
Root Mean Square Error	0.112431
Mean of Response	0.264815
Observations (or Sum Wgts)	156

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	10	4.6852650	0.468526	37.0652
Error	145	1.8328897	0.012641	**Prob > F**
C. Total	155	6.5181547		<.0001*

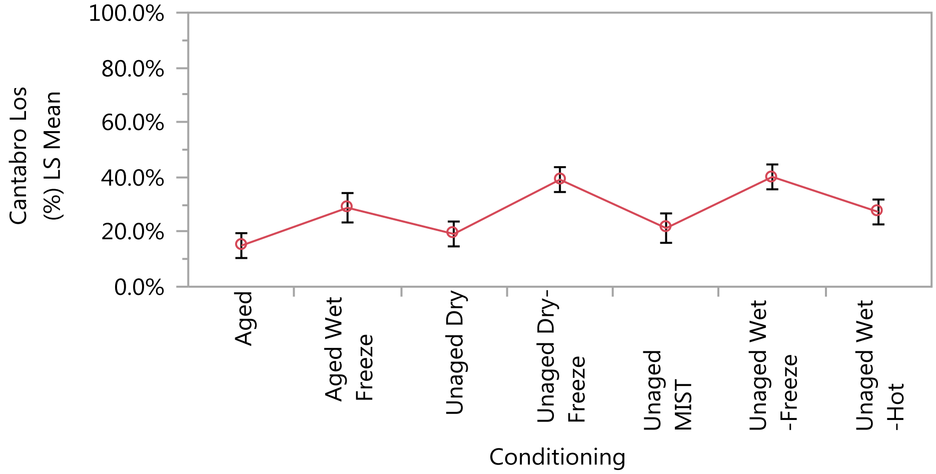
Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Conditioning	6	6	1.3090315	17.2596	<.0001*
Air Voids	1	1	0.2716840	21.4929	<.0001*
Material	3	3	2.9857108	78.7332	<.0001*

Effect Details
Conditioning
Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Aged	0.15002917	0.02294978	0.150029
Aged Wet Freeze	0.28813000	0.02715455	0.246778
Unaged Dry	0.19247917	0.02294978	0.192479
Unaged Dry-Freeze	0.39087500	0.02294978	0.390875
Unaged MIST	0.21379667	0.02715455	0.172444
Unaged Wet-Freeze	0.40079167	0.02294978	0.400792
Unaged Wet-Hot	0.27270833	0.02294978	0.272708

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level					Least Sq Mean
Unaged Wet-Freeze	A				0.40079167
Unaged Dry-Freeze	A	B			0.39087500
Aged Wet Freeze		B	C		0.28813000
Unaged Wet-Hot			C		0.27270833
Unaged MIST			C	D	0.21379667
Unaged Dry			C	D	0.19247917
Aged				D	0.15002917

Levels not connected by same letter are significantly different.

Air Voids
Least Squares Means Table

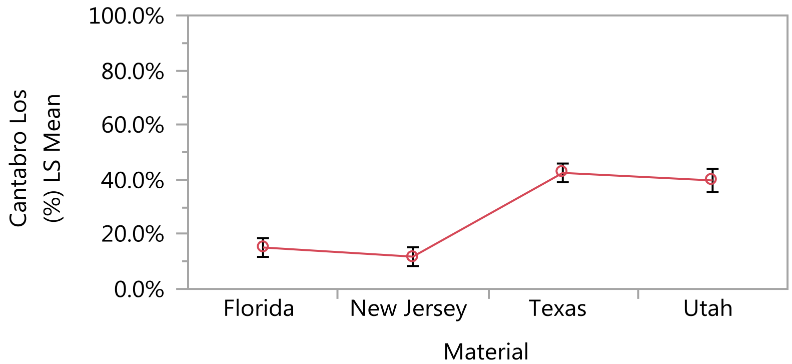
Level	Least Sq Mean	Std Error	Mean
Construction	0.31441919	0.01289588	0.306547
Design	0.23095509	0.01289588	0.223083

Material
Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean

Level	Least Sq Mean	Std Error	Mean
Florida	0.15154762	0.01734840	0.151548
New Jersey	0.11812381	0.01734840	0.118124
Texas	0.42433333	0.01734840	0.424333
Utah	0.39674381	0.02148215	0.405433

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level			Least Sq Mean
Texas	A		0.42433333
Utah	A		0.39674381
Florida		B	0.15154762
New Jersey		B	0.11812381

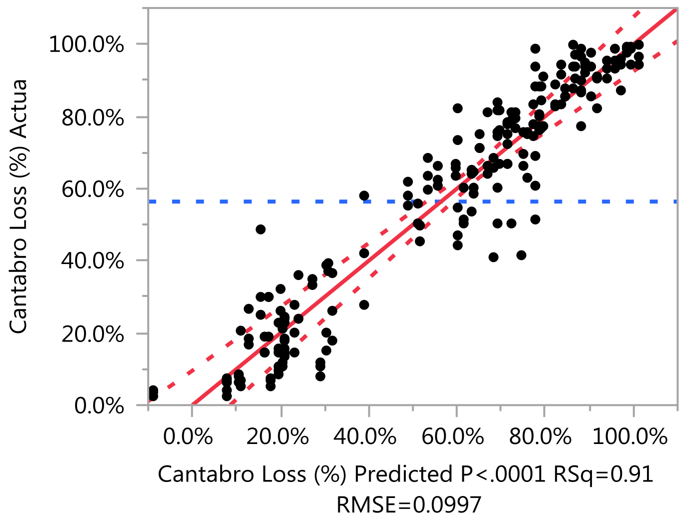
Levels not connected by same letter are significantly different.

APPENDIX O

STATISTICAL ANALYSIS CANTABRO TEST SLAB CORES

Results of Fitting ANOVA to the Cantabro Slab Cores Data with 208 Measurements

Response Cantabro Loss (%)
Actual by Predicted Plot



Summary of Fit

RSquare	0.914606
RSquare Adj	0.897823
Root Mean Square Error	0.099662
Mean of Response	0.563844
Observations (or Sum Wgts)	208

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	34	18.404129	0.541298	54.4971
Error	173	1.718340	0.009933	**Prob > F**
C. Total	207	20.122469		<.0001*

Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Air Voids	1	1	0.3986032	40.1308	<.0001*
Conditioning	5	5	0.5156028	10.3820	<.0001*
Material	2	2	9.5743298	481.9648	<.0001*
Thickness (in)	1	1	6.0173286	605.8159	<.0001*
Conditioning*Thickness (in)	5	5	0.0328257	0.6610	0.6535
Conditioning*Air Voids	5	5	0.1228872	2.4744	0.0341*
Conditioning*Material	10	10	0.3343093	3.3658	0.0005*
Thickness (in)*Air Voids	1	1	0.0007140	0.0719	0.7889
Thickness (in)*Material	2	2	1.2636986	63.6137	<.0001*
Air Voids*Material	2	2	0.0400240	2.0148	0.1365

Effect Details

Air Voids

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Construction	0.60782257	0.00981390	0.610631
Design	0.51985585	0.00986676	0.517057

Conditioning

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Aged	0.47182562	0.01688415	0.468114
Unaged Dry	0.57024722	0.01661041	0.570247
Unaged Dry-Freeze	0.63028409	0.01688359	0.625457
Unaged MIST	0.53697222	0.01661041	0.536972
Unaged Wet-Freeze	0.59858967	0.01854558	0.602697
Unaged Wet-Hot	0.57511645	0.01688398	0.584600

Material

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Florida	0.47597420	0.01263103	0.470148
New Jersey	0.35958649	0.01184231	0.353394
Texas	0.85595694	0.01174533	0.855957

Thickness (in)

Least Squares Means Table

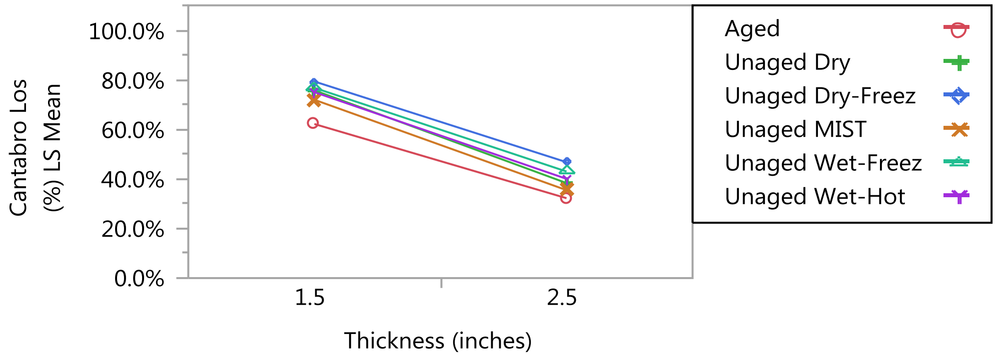
Level	Least Sq Mean	Std Error	Mean
1.5	0.73471727	0.00991564	0.734628
2.5	0.39296115	0.00976375	0.396312

Conditioning*Thickness (in)

Least Squares Means Table

Level	Least Sq Mean	Std Error
Aged,1.5	0.62142902	0.02425875
Aged,2.5	0.32222222	0.02349066
Unaged Dry,1.5	0.75728333	0.02349066
Unaged Dry,2.5	0.38321111	0.02349066
Unaged Dry-Freeze,1.5	0.79301262	0.02425720
Unaged Dry-Freeze,2.5	0.46755556	0.02349066
Unaged MIST,1.5	0.71922222	0.02349066
Unaged MIST,2.5	0.35472222	0.02349066
Unaged Wet-Freeze,1.5	0.76735642	0.02653325
Unaged Wet-Freeze,2.5	0.42982291	0.02519622
Unaged Wet-Hot,1.5	0.75000000	0.02349066
Unaged Wet-Hot,2.5	0.40023290	0.02425827

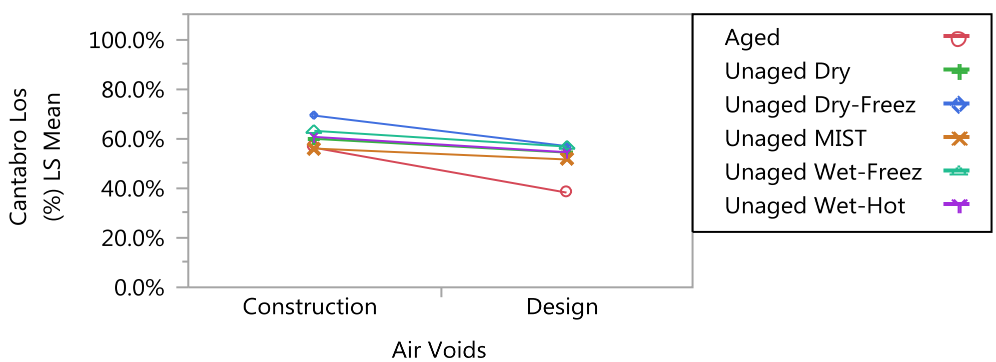
LS Means Plot



Conditioning*Air Voids
Least Squares Means Table

Level	Least Sq Mean	Std Error
Aged,Construction	0.56211111	0.02349066
Aged,Design	0.38154013	0.02425875
Unaged Dry,Construction	0.59799444	0.02349066
Unaged Dry,Design	0.54250000	0.02349066
Unaged Dry-Freeze,Construction	0.69167929	0.02425720
Unaged Dry-Freeze,Design	0.56888889	0.02349066
Unaged MIST,Construction	0.55900000	0.02349066
Unaged MIST,Design	0.51494444	0.02349066
Unaged Wet-Freeze,Construction	0.63025102	0.02519625
Unaged Wet-Freeze,Design	0.56692831	0.02653300
Unaged Wet-Hot,Construction	0.60589956	0.02425827
Unaged Wet-Hot,Design	0.54433333	0.02349066

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level				Least Sq Mean
Unaged Dry-Freeze,Construction	A			0.69167929
Unaged Wet-Freeze,Construction	A	B		0.63025102
Unaged Wet-Hot,Construction	A	B	C	0.60589956
Unaged Dry,Construction	A	B	C	0.59799444
Unaged Dry-Freeze,Design		B	C	0.56888889

Level		Least Sq Mean
Unaged Wet-Freeze,Design	B C	0.56692831
Aged,Construction	B C	0.56211111
Unaged MIST,Construction	B C	0.55900000
Unaged Wet-Hot,Design	B C	0.54433333
Unaged Dry,Design	B C	0.54250000
Unaged MIST,Design	C	0.51494444
Aged,Design	D	0.38154013

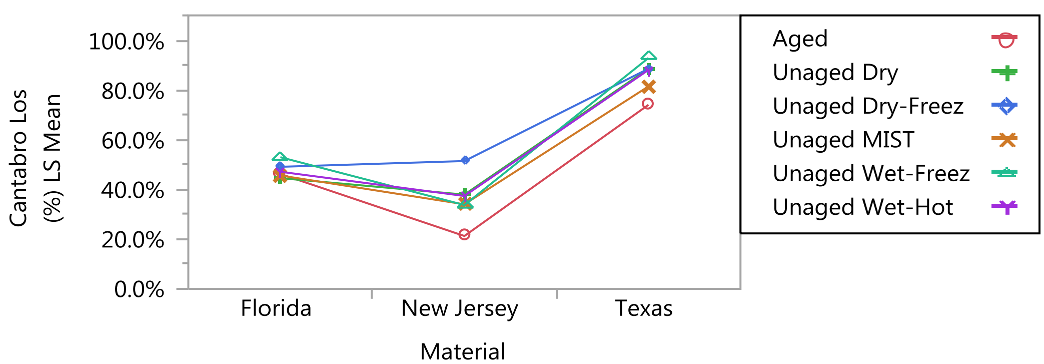
Levels not connected by same letter are significantly different.

Conditioning*Material

Least Squares Means Table

Level	Least Sq Mean	Std Error
Aged,Florida	0.46231020	0.03017013
Aged,New Jersey	0.21275000	0.02877007
Aged,Texas	0.74041667	0.02877007
Unaged Dry,Florida	0.44558333	0.02877007
Unaged Dry,New Jersey	0.37925000	0.02877007
Unaged Dry,Texas	0.88590833	0.02877007
Unaged Dry-Freeze,Florida	0.49100000	0.02877007
Unaged Dry-Freeze,New Jersey	0.51426893	0.03016733
Unaged Dry-Freeze,Texas	0.88558333	0.02877007
Unaged MIST,Florida	0.45616667	0.02877007
Unaged MIST,New Jersey	0.34025000	0.02877007
Unaged MIST,Texas	0.81450000	0.02877007
Unaged Wet-Freeze,Florida	0.52968566	0.03794749
Unaged Wet-Freeze,New Jersey	0.33733333	0.02877007
Unaged Wet-Freeze,Texas	0.92875000	0.02877007
Unaged Wet-Hot,Florida	0.47109935	0.03016927
Unaged Wet-Hot,New Jersey	0.37366667	0.02877007
Unaged Wet-Hot,Texas	0.88058333	0.02877007

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level			Least Sq Mean
Unaged Wet-Freeze,Texas	A		0.92875000
Unaged Dry,Texas	A		0.88590833
Unaged Dry-Freeze,Texas	A		0.88558333
Unaged Wet-Hot,Texas	A	B	0.88058333
Unaged MIST,Texas	A	B	0.81450000

Level				Least Sq Mean
Aged,Texas	B			0.74041667
Unaged Wet-Freeze,Florida	C			0.52968566
Unaged Dry-Freeze,New Jersey	C			0.51426893
Unaged Dry-Freeze,Florida	C			0.49100000
Unaged Wet-Hot,Florida	C	D		0.47109935
Aged,Florida	C	D		0.46231020
Unaged MIST,Florida	C	D		0.45616667
Unaged Dry,Florida	C	D		0.44558333
Unaged Dry,New Jersey	C	D		0.37925000
Unaged Wet-Hot,New Jersey	C	D		0.37366667
Unaged MIST,New Jersey		D	E	0.34025000
Unaged Wet-Freeze,New Jersey		D	E	0.33733333
Aged,New Jersey			E	0.21275000

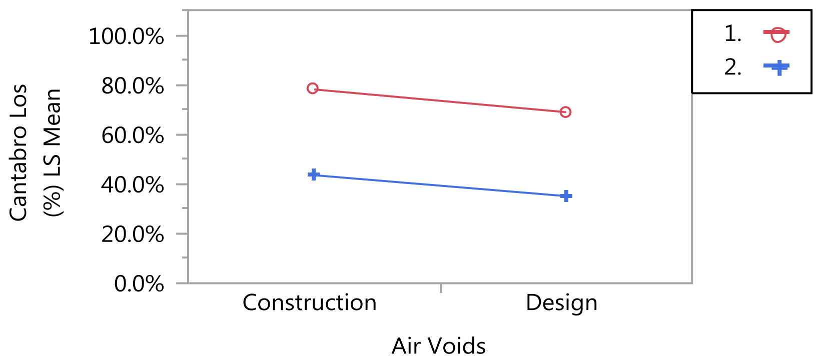
Levels not connected by same letter are significantly different.

Thickness (in)*Air Voids

Least Squares Means Table

Level	Least Sq Mean	Std Error
1.5,Construction	0.78055689	0.01386203
1.5,Design	0.68887765	0.01411305
2.5,Construction	0.43508825	0.01386997
2.5,Design	0.35083405	0.01371661

LS Means Plot

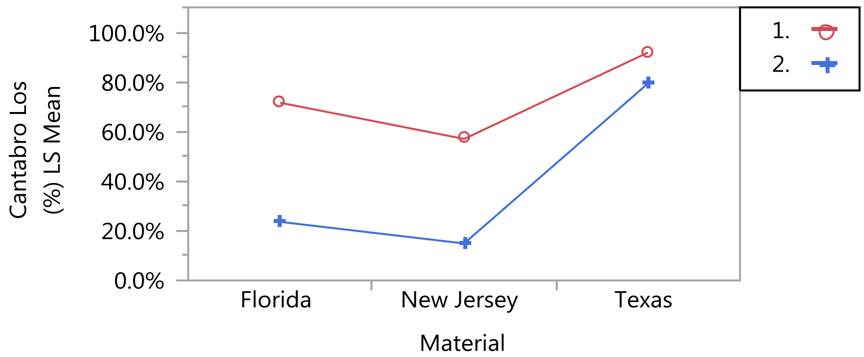


Thickness (in)*Material

Least Squares Means Table

Level	Least Sq Mean	Std Error
1.5,Florida	0.71586494	0.01801029
1.5,New Jersey	0.57011742	0.01688359
1.5,Texas	0.91816944	0.01661041
2.5,Florida	0.23608346	0.01749760
2.5,New Jersey	0.14905556	0.01661041
2.5,Texas	0.79374444	0.01661041

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level							Least Sq Mean
1.5,Texas	A						0.91816944
2.5,Texas		B					0.79374444
1.5,Florida			C				0.71586494
1.5,New Jersey				D			0.57011742
2.5,Florida					E		0.23608346
2.5,New Jersey						F	0.14905556

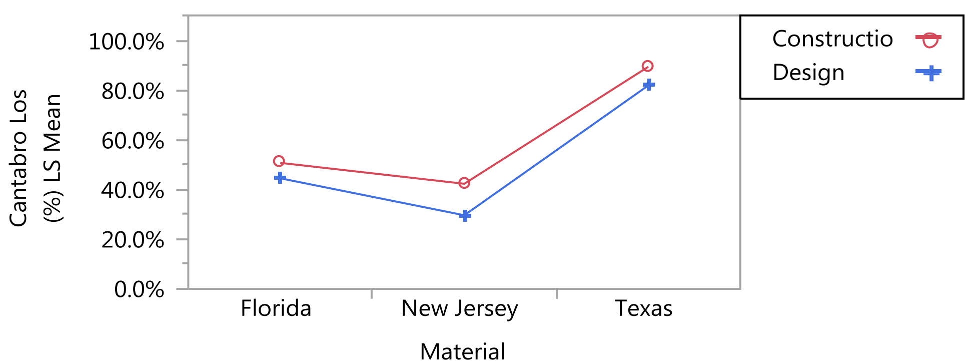
Levels not connected by same letter are significantly different.

Air Voids*Material

Least Squares Means Table

Level	Least Sq Mean	Std Error
Construction,Florida	0.50701974	0.01749768
Construction,New Jersey	0.42289520	0.01688359
Construction,Texas	0.89355278	0.01661041
Design,Florida	0.44492866	0.01801014
Design,New Jersey	0.29627778	0.01661041
Design,Texas	0.81836111	0.01661041

LS Means Plot

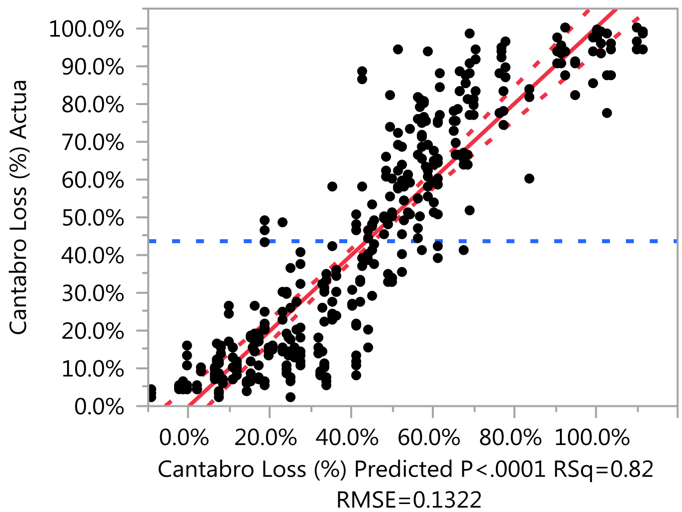


APPENDIX P

STATISTICAL ANALYSIS CANTABRO TEST THICKNESS DATA

Results of Fitting ANOVA to the Cantabro Loss Combined Data (thickness analysis)

Response Cantabro Loss (%)
Actual by Predicted Plot



Summary of Fit

RSquare	0.822859
RSquare Adj	0.816803
Root Mean Square Error	0.132165
Mean of Response	0.435689
Observations (or Sum Wgts)	364

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	12	28.480467	2.37337	135.8728
Error	351	6.131129	0.01747	**Prob > F**
C. Total	363	34.611596		<.0001*

Effect Tests

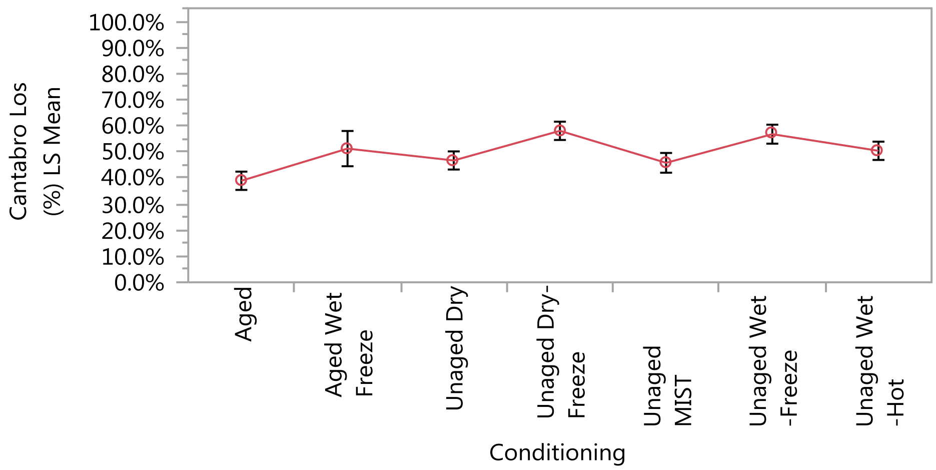
Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Conditioning	6	6	1.510376	14.4112	<.0001*
Thickness (in)	2	2	13.725070	392.8721	<.0001*
Air Voids	1	1	0.698441	39.9849	<.0001*
Material	3	3	12.197628	232.7667	<.0001*

Effect Details
Conditioning
Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean

Level	Least Sq Mean	Std Error	Mean
Aged	0.38909988	0.01782081	0.338724
Aged Wet Freeze	0.51228815	0.03436105	0.246778
Unaged Dry	0.46686347	0.01767567	0.419140
Unaged Dry-Freeze	0.58041857	0.01782051	0.530034
Unaged MIST	0.45780784	0.01925230	0.415463
Unaged Wet-Freeze	0.56753391	0.01846246	0.514593
Unaged Wet-Hot	0.50390525	0.01782069	0.457729

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level					Least Sq Mean
Unaged Dry-Freeze	A				0.58041857
Unaged Wet-Freeze	A	B			0.56753391
Aged Wet Freeze	A	B	C		0.51228815
Unaged Wet-Hot		B	C		0.50390525
Unaged Dry			C		0.46686347
Unaged MIST			C	D	0.45780784
Aged				D	0.38909988

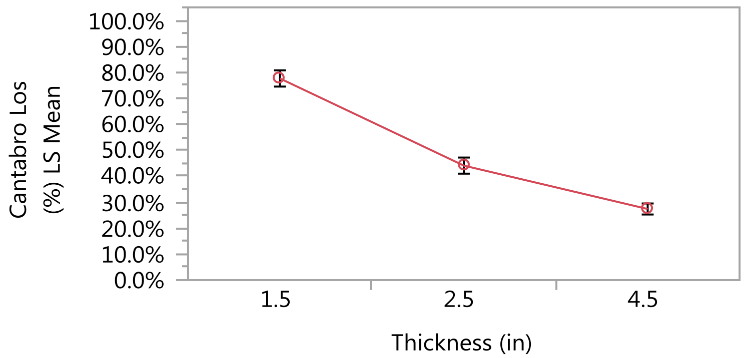
Levels not connected by same letter are significantly different.

Thickness (in)

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
1.5	0.77613042	0.01585860	0.734628
2.5	0.44072568	0.01575514	0.396312
4.5	0.27367979	0.01079319	0.264815

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level				Least Sq Mean
1.5	A			0.77613042
2.5		B		0.44072568
4.5			C	0.27367979

Levels not connected by same letter are significantly different.

Air Voids

Least Squares Means Table

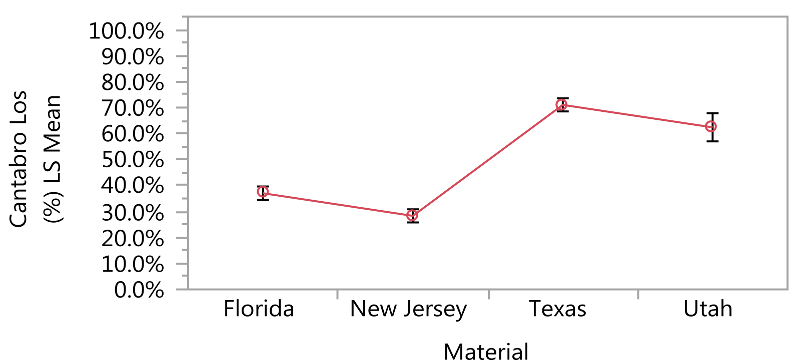
Level	Least Sq Mean	Std Error	Mean
Construction	0.54065592	0.01194528	0.480309
Design	0.45303467	0.01194908	0.391068

Material

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Florida	0.37015548	0.01322361	0.345090
New Jersey	0.28303696	0.01285331	0.265949
Texas	0.71030884	0.01279719	0.696938
Utah	0.62387992	0.02749054	0.405433

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level					Least Sq Mean
Texas	A				0.71030884
Utah		B			0.62387992
Florida			C		0.37015548
New Jersey				D	0.28303696

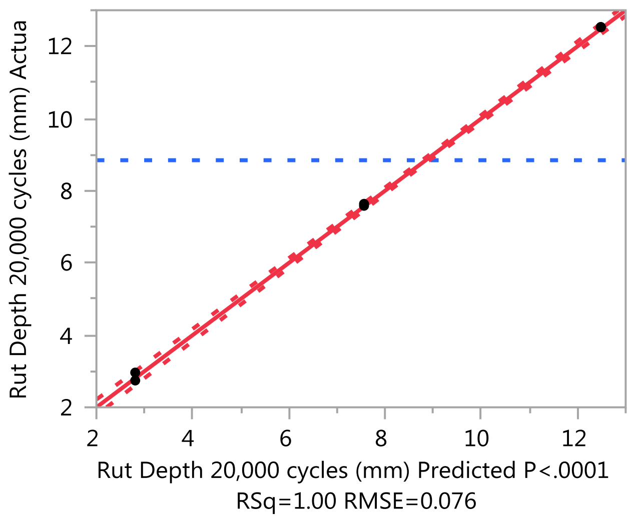
Levels not connected by same letter are significantly different.

APPENDIX Q

STATISTICAL ANALYSIS HWTT DATA

Results of Fitting ANOVA to the HAMBURG Test Data

(3a) Rut Depth 20,000 cycles
Fit Group
Response Rut Depth 20,000 cycles (mm)
Actual by Predicted Plot



Summary of Fit

RSquare	0.999822
RSquare Adj	0.999688
Root Mean Square Error	0.076027
Mean of Response	8.844875
Observations (or Sum Wgts)	8

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	3	129.53235	43.1775	7469.986
Error	4	0.02312	0.0058	**Prob > F**
C. Total	7	129.55547		<.0001*

Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Material	3	3	129.53235	7469.986	<.0001*

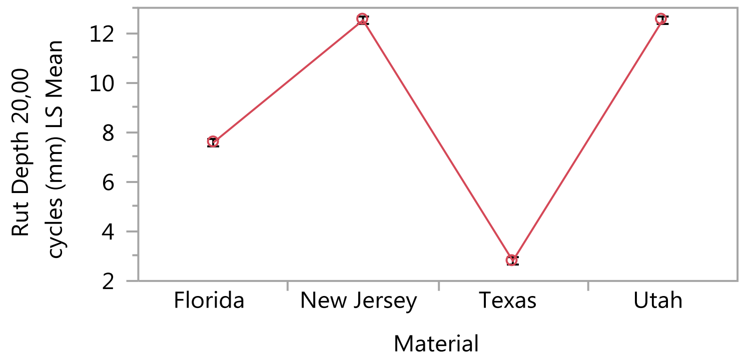
Effect Details

Material

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Florida	7.569500	0.05375930	7.5695
New Jersey	12.500000	0.05375930	12.5000
Texas	2.810000	0.05375930	2.8100
Utah	12.500000	0.05375930	12.5000

LS Means Plot



LSMeans Differences Tukey HSD

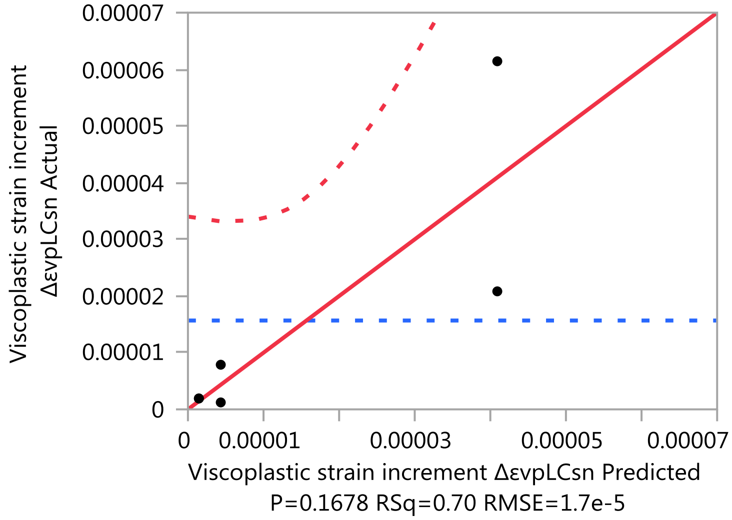
α=
0.050

Level				Least Sq Mean
New Jersey	A			12.500000
Utah	A			12.500000
Florida		B		7.569500
Texas			C	2.810000

Levels not connected by same letter are significantly different.

(3b) Viscoplastic strain increment $\Delta\varepsilon^{vp}_{LCsn}$

Response Viscoplastic strain increment ΔεvpLCsn
Actual by Predicted Plot



Summary of Fit

RSquare	0.695812
RSquare Adj	0.49302
Root Mean Square Error	1.674e-5
Mean of Response	1.568e-5
Observations (or Sum Wgts)	6

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	2	1.92357e-9	9.618e-10	3.4312
Error	3	8.4093e-10	2.803e-10	**Prob > F**
C. Total	5	2.76449e-9		0.1678

Effect Tests

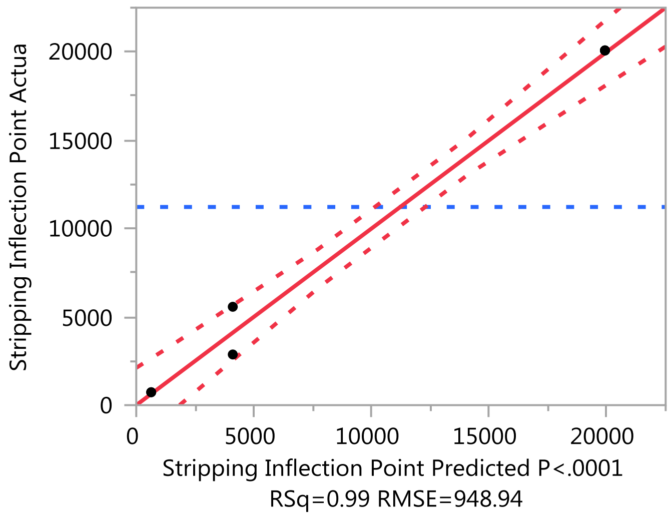
Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Material	2	2	1.92357e-9	3.4312	0.1678

Effect Details
Material
Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Florida	0.00000169	0.00001184	1.685e-6
New Jersey	0.00004095	0.00001184	0.000041
Texas	0.00000440	0.00001184	4.395e-6

(3c) Stripping Inflection Point

Response Stripping Inflection Point
Actual by Predicted Plot



Summary of Fit

RSquare	0.994306
RSquare Adj	0.990036
Root Mean Square Error	948.9373
Mean of Response	11218
Observations (or Sum Wgts)	8

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	3	628987488	209662496	232.8336
Error	4	3601928	900482	**Prob > F**
C. Total	7	632589416		<.0001*

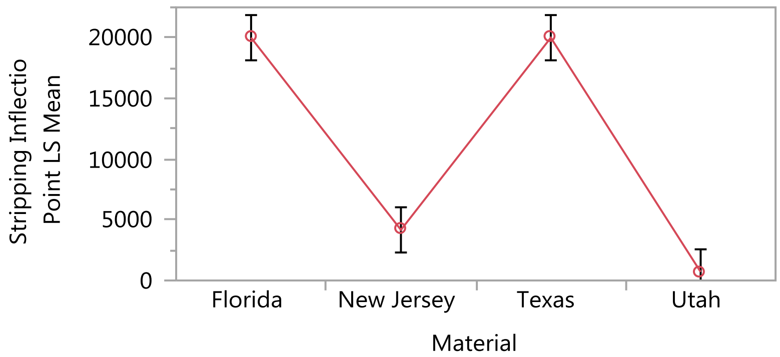
Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Material	3	3	628987488	232.8336	<.0001*

Effect Details
Material
Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Florida	20000.000	671.00000	20000.0
New Jersey	4168.000	671.00000	4168.0
Texas	20000.000	671.00000	20000.0
Utah	704.000	671.00000	704.0

LS Means Plot



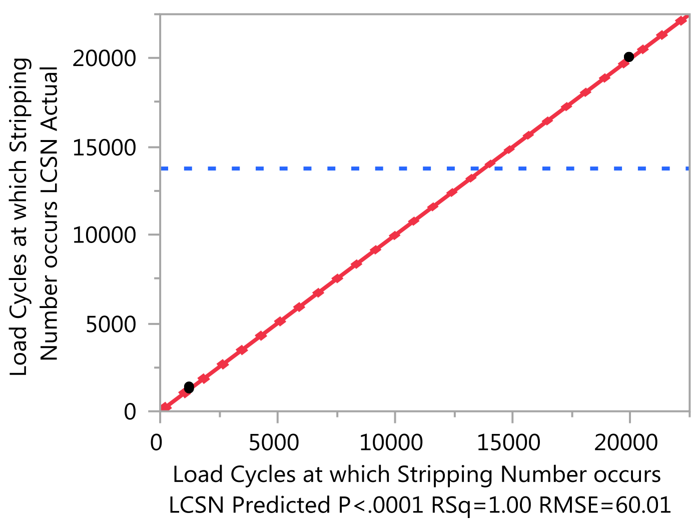
LSMeans Differences Tukey HSD

α=
0.050

Level			Least Sq Mean
Florida	A		20000.000
Texas	A		20000.000
New Jersey		B	4168.000
Utah		B	704.000

Levels not connected by same letter are significantly different.

(3d) Load Cycles at which Stripping Number occurs LC_{SN}
Response Load Cycles at which Stripping Number occurs LCSN
Actual by Predicted Plot



Summary of Fit

RSquare	0.999977
RSquare Adj	0.999962

Root Mean Square Error	60.0125
Mean of Response	13755.5
Observations (or Sum Wgts)	6

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	2	467925363	233962682	64962.57
Error	3	10805	3601.5	**Prob > F**
C. Total	5	467936168		<.0001*

Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Material	2	2	467925363	64962.57	<.0001*

Effect Details

Material

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Florida	20000.000	42.435245	20000.0
New Jersey	1266.500	42.435245	1266.5
Texas	20000.000	42.435245	20000.0

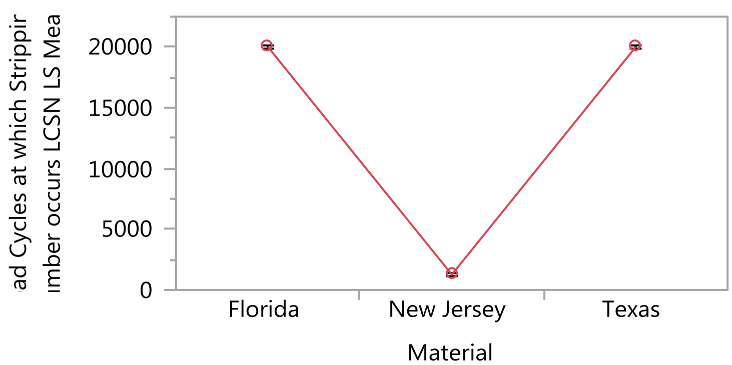
LSMeans Differences Tukey HSD

α=0.050

Level			Least Sq Mean
Florida	A		20000.000
Texas	A		20000.000
New Jersey		B	1266.500

Levels not connected by same letter are significantly different.

LS Means Plot

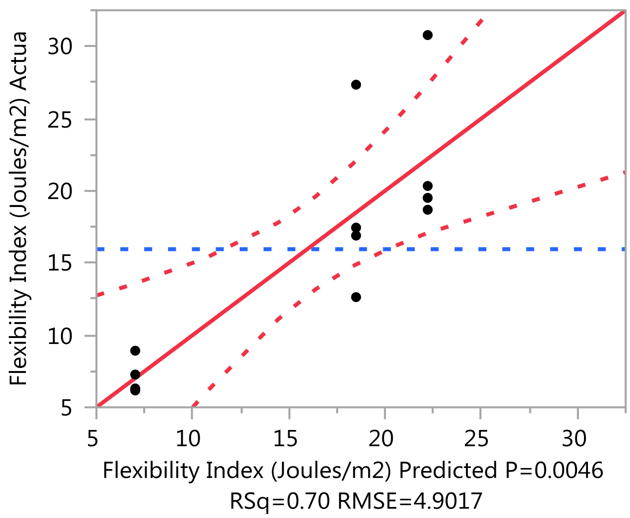


APPENDIX R

STATISTICAL ANALYSIS SCB DATA

Results of Fitting ANOVA to the SCB Test Data

(2a) Flexibility Index
Fit Group
Response Flexibility Index (Joules/m2)
Actual by Predicted Plot



Summary of Fit

RSquare	0.697419
RSquare Adj	0.630179
Root Mean Square Error	4.901718
Mean of Response	15.95167
Observations (or Sum Wgts)	12

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	2	498.41522	249.208	10.3721
Error	9	216.24155	24.027	**Prob > F**
C. Total	11	714.65677		0.0046*

Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Material	2	2	498.41522	10.3721	0.0046*

Effect Details
Material
Least Squares Means Table

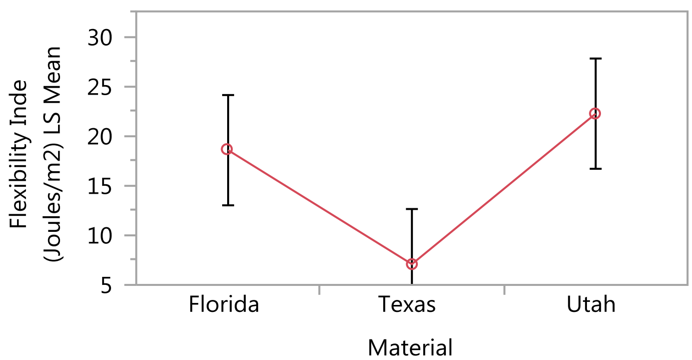
Level	Least Sq Mean	Std Error	Mean
Florida	18.547500	2.4508590	18.5475
Texas	7.087500	2.4508590	7.0875
Utah	22.220000	2.4508590	22.2200

LSMeans Differences Tukey HSD
α=0.050

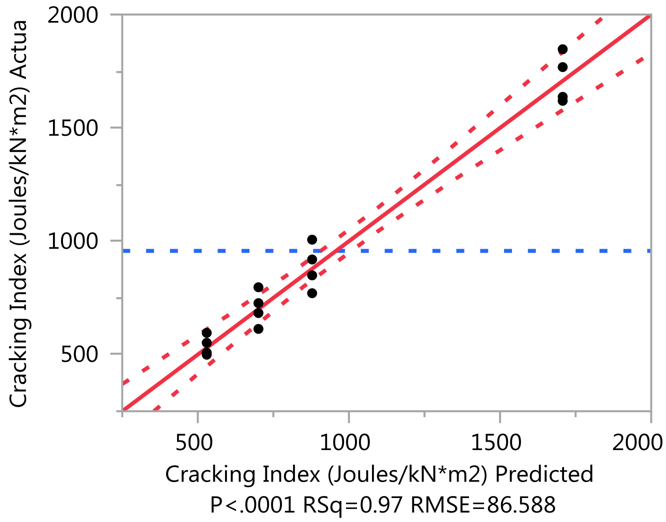
Level			Least Sq Mean
Utah	A		22.220000
Florida	A		18.547500
Texas		B	7.087500

Levels not connected by same letter are significantly different.

LS Means Plot



(2b) Cracking Index
Fit Group
Response Cracking Index (Joules/kN*m2)
Actual by Predicted Plot



Summary of Fit

RSquare	0.973219
RSquare Adj	0.966524
Root Mean Square Error	86.58769
Mean of Response	957.3381
Observations (or Sum Wgts)	16

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	3	3269504.9	1089835	145.3612
Error	12	89969.1	7497	**Prob > F**
C. Total	15	3359474.0		<.0001*

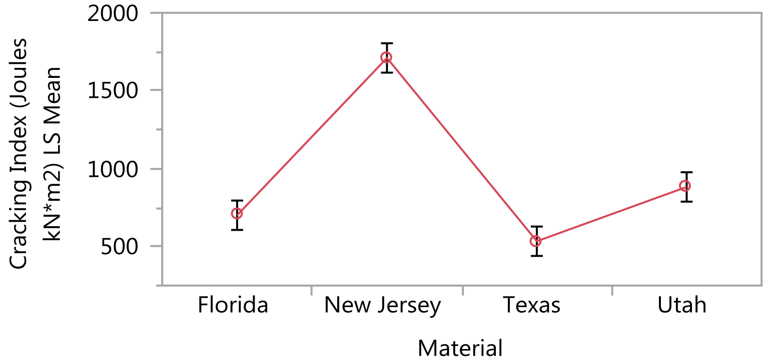
Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Material	3	3	3269504.9	145.3612	<.0001*

Effect Details
Material
Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Florida	701.2700	43.293846	701.27
New Jersey	1710.6700	43.293846	1710.67
Texas	534.5500	43.293846	534.55
Utah	882.8625	43.293846	882.86

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level			Least Sq Mean
New Jersey	A		1710.6700
Utah	B		882.8625
Florida	B	C	701.2700
Texas		C	534.5500

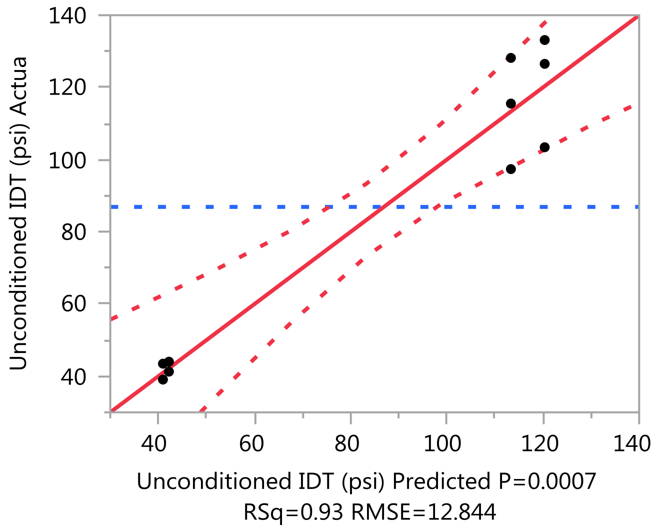
Levels not connected by same letter are significantly different.

APPENDIX S

STATISTICAL ANALYSIS IDT/TSR DATA

Results of Fitting ANOVA to the TSR AASHTO T283 Test Data

(1a) Unconditioned IDT test data
Least Squares Fit
Response Unconditioned IDT (psi)
Actual by Predicted Plot



Summary of Fit

RSquare	0.932494
RSquare Adj	0.898741
Root Mean Square Error	12.84415
Mean of Response	86.9
Observations (or Sum Wgts)	10

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	3	13673.067	4557.69	27.6270
Error	6	989.833	164.97	**Prob > F**
C. Total	9	14662.900		0.0007*

Effect Tests

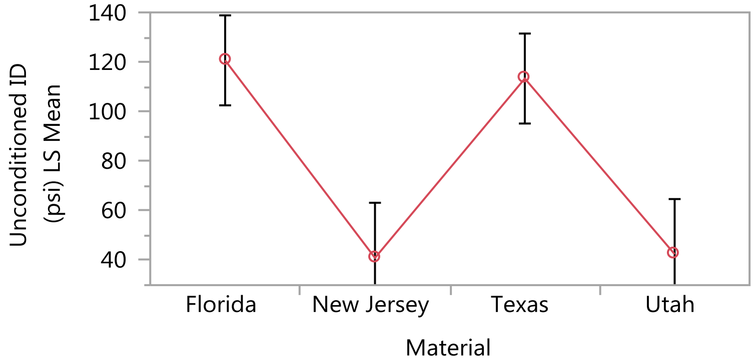
Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Material	3	3	13673.067	27.6270	0.0007*

Effect Details
Material
Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean

Level	Least Sq Mean	Std Error	Mean
Florida	120.66667	7.4155742	120.667
New Jersey	41.00000	9.0821865	41.000
Texas	113.33333	7.4155742	113.333
Utah	42.50000	9.0821865	42.500

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

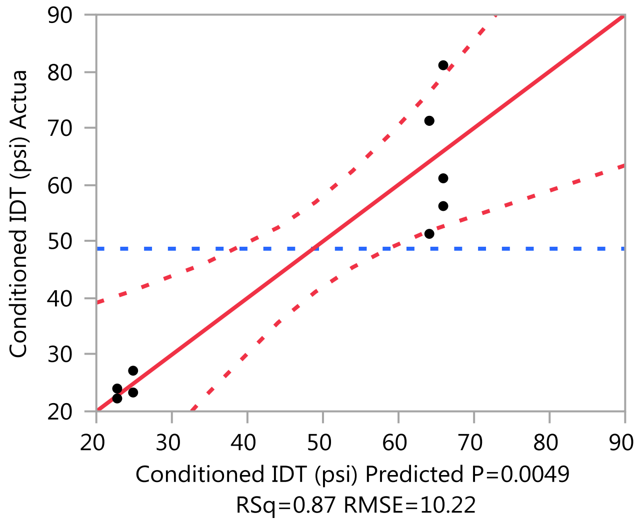
Level			Least Sq Mean
Florida	A		120.66667
Texas	A		113.33333
Utah		B	42.50000
New Jersey		B	41.00000

Levels not connected by same letter are significantly different.

(1b) Conditioned IDT test data

Least Squares Fit

Response Conditioned IDT (psi)
Actual by Predicted Plot



Summary of Fit

RSquare	0.866726
RSquare Adj	0.800089
Root Mean Square Error	10.21981
Mean of Response	48.7
Observations (or Sum Wgts)	10

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	3	4075.4333	1358.48	13.0067
Error	6	626.6667	104.44	**Prob > F**
C. Total	9	4702.1000		0.0049*

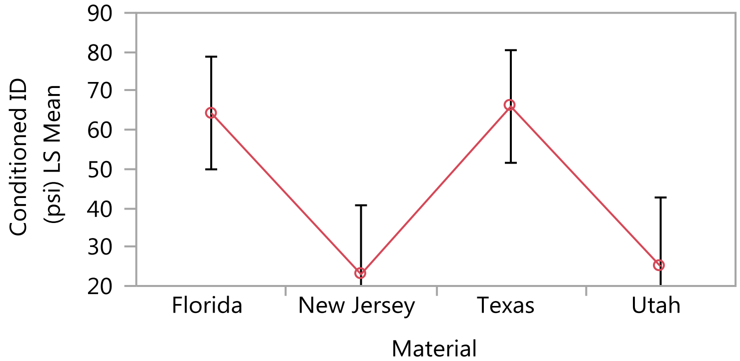
Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Material	3	3	4075.4333	13.0067	0.0049*

Effect Details
Material
Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Florida	64.333333	5.9004080	64.3333
New Jersey	23.000000	7.2264945	23.0000
Texas	66.000000	5.9004080	66.0000
Utah	25.000000	7.2264945	25.0000

LS Means Plot



LSMeans Differences Tukey HSD

α=0.050

Level			Least Sq Mean
Texas	A		66.000000
Florida	A		64.333333
Utah		B	25.000000
New Jersey		B	23.000000

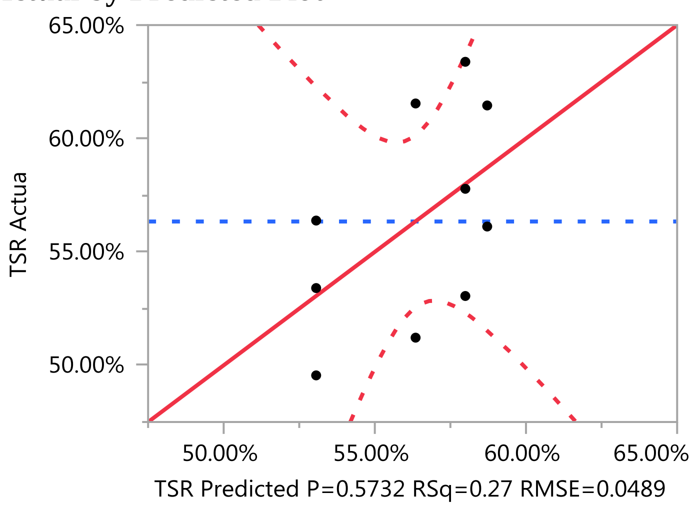
Levels not connected by same letter are significantly different.

(1c) TSR tests data

Least Squares Fit

Response TSR

Actual by Predicted Plot



Summary of Fit

RSquare	0.26601
RSquare Adj	-0.10098
Root Mean Square Error	0.048914

Mean of Response	0.5634
Observations (or Sum Wgts)	10

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	3	0.00520273	0.001734	0.7248
Error	6	0.01435567	0.002393	**Prob > F**
C. Total	9	0.01955840		0.5732

Effect Tests

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Material	3	3	0.00520273	0.7248	0.5732

Effect Details

Material

Least Squares Means Table

Level	Least Sq Mean	Std Error	Mean
Florida	0.53066667	0.02824070	0.530667
New Jersey	0.56350000	0.03458765	0.563500
Texas	0.58000000	0.02824070	0.580000
Utah	0.58750000	0.03458765	0.587500