ABSTRACT

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Investigation of Over-Permeable Superpave Surface Mixes in North Carolina.
(Under the direction of Dr. Ahktar Tayebali, Ph.D, PE)

Superpave was introduced in 1994, as a result of the Strategic Highway Research Program (SHRP) and was aimed at addressing the performance of asphalt concrete pavements. This SHRP study recommended new specifications for asphalt mixture designs which included performance graded binders, performance related aggregate properties, and new principles for volumetric mix designs. With these new mix design principles, it was evident that Superpave mix designs would contain more coarse aggregates and less natural sands. With the introduction of Superpave, many in the Asphalt industry feared that the recommended coarse structured mixes would be very difficult to compact and highly permeable compared to the Marshall mixes they were replacing. In North Carolina, we have seen instances where this theory was proven true and have witnessed coarse Superpave mixes that have been permeable. In general, the cause of this permeability may be due to increased voids caused by improper compaction that results from the coarse nature of the designs, which in turn creates interconnected voids that will transmit water through the pavement. A remedy to the permeability problem may be a new test that can determine in the laboratory if certain mixes are susceptible to permeability. This test is discussed in this study.

The data represented in this study shows the causes of permeability in a small group of pavements that were sampled and should not be misinterpreted to imply that this

small number of distressed pavements is indicative of all Superpave mixes. The selected mixes are a cross section of surface mixes used across the State of North Carolina.

The permeability test method discussed in this study is a very simple procedure and the data presented show that this test can be used for predicting field permeability of Superpave surface mixtures. The coarse graded mixes performed poorly in the lab permeability tests and the fine graded mixes proved to be very impermeable. This study also evaluated mixes using laboratory performance tests. The results show that the coarse graded mixes and fine graded surface mixes perform equally well under laboratory performance testing at a given void content that matches the in-situ properties.

Using these lab tests and the results of this study, a clearer understanding of the effects of water in asphalt pavements and how to prevent this water infiltration has been obtained and new specification changes have been recommended to reduce any future permeability problem with Superpave surface mixes.

INVESTIGATION OF OVER-PERMEABLE SUPERPAVE SURFACE MIXES IN NORTH CAROLINA

By

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BIOGRAPHY

Mr. Christopher Bacchi has been attending North Carolina State University since 1998 studying for his Masters of Science Degree in Civil Engineering focusing on Transportation Materials. Mr. Bacchi received his Bachelors degree in 1995 from West Virginia University in Morgantown, WV. He has been a registered Professional Engineer in the State of North Carolina since June 2001 and is currently employed with the North Carolina Department of Transportation as the State Asphalt Design Engineer in the Raleigh Materials and Tests lab. Mr. Bacchi has been with the Department since 1996.

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NOTATIONS

AASHTO	**American Association of State Highway and Transportation Officials**
AC	**Asphalt Cement**
APA	**Asphalt Pavement Analyzer**
ASTM	**American Society for Testing and Materials**
DOT	**Department of Transportation**
ESAL	**Equivalent Single Axle Load**
FDOT	**Florida Department of Transportation**
G_{mm}	**Maximum Theoretical Specific Gravity**
G_{sb}	**Mixture Bulk Specific Gravity**
HDS	**Heavy Duty Surface mix**
HMA	**Hot Mix Asphalt**
JMF	**Job Mix Formula**
NCAT	**National Center for Asphalt Technology**
NCDOT	**North Carolina Department of Transportation**
NMAS	**Nominal Maximum Aggregate Size**
QA/QC	**Quality Assurance/Quality Control**
RAP	**Reclaimed Asphalt Pavement**
RSCH	**Repeated Shear Constant Height**
SHRP	**Strategic Highway Research Project**
SST	**Superpave Shear Tester**
VFA	**Voids filled with Asphalt**
VMA	**Voids in the Mineral Aggregate**
VTM	**Voids total mix**

CHAPTER 1 – INTRODUCTION AND BACKGROUND

1.1 INTRODUCTION

Hot mix asphalt (HMA) covers 96% of all hard surface roadways in the United States and with its use increasing everyday, the quality of HMA is very important. One of the largest roadway systems in the nation is in North Carolina, where there are approximately 78,000 miles of state maintained roads. The North Carolina Department of Transportation (NCDOT) specifies HMA on all types of roads, from small two-lane rural state roads, to large multilane Interstates that carry up to 60 million equivalent single axle loads (ESAL's) during its 20 years design life. With much of the traveling public dependent on the asphalt pavements, the quality and durability of these roads is a very large concern of the NCDOT. HMA has been used for many years for new construction and for maintaining current roadways and over the years many methods and mixture types have been used. Throughout all of the changes though, a few things have remained constant. Asphalt mix is made up of aggregate and asphalt binder and when it is placed, it needs to be compacted to a specific density. This density is one of the simplest and most important specifications on any asphalt pavement. Past research *(4)* has shown that it is important to achieve in place air void contents above 3% in order to decrease the probability of premature rutting, and below 8% to prevent water and air infiltration which causes a potential for moisture damage, raveling and/or cracking. The recommended range for percent compaction for all mix types is generally in the area of 92% to 95%. Under the older Marshall specifications used in North Carolina *(1)*, the minimum percent compaction of a field core was 95% of the target specific gravity, which for a surface mix, such as a heavy duty surface course (HDS), was approximately 10.0% air voids. For these finer graded Marshall mixes, 10.0% air voids did not cause many permeable pavements due to the fact that finer mixes may have more air voids, but less interconnected air voids, like a dense graded coarse mixture. These interconnected voids are the pathways that allow water to seep through pavement layers and eventually deteriorate a pavement. Past research has shown that dense graded HMA mixtures become permeable to water at approximately 8% air voids *(3, 5)*. The structural skeleton of the new coarser Superpave mixes is more resistant to permanent deformations, but the orientation of the larger stone particles increase the mixes' sensitivity to void content and thus a greater chance of higher interconnected voids.

Prior research *(8)* has shown that if there are a larger number of interconnected voids on a surface mix it will cause water infiltration into the layer causing stripping under repeated traffic loads and ultimately affect the durability of the pavement. This water infiltration is a definite cause of premature pavement failure, including stripping, rutting and raveling.

In 1997, the NCDOT decided to make an effort to switch from the older Marshall design system to the new Superpave mix design system recommended by the Strategic Highway Research Program report A-410 *(2)*. This conversion was to be completed by 2000, with all new projects let utilizing Superpave. This transition has been a learning experience for both contractors and the Department alike, and this experience has included some poor performing mixes on some major roadways. One project where the NCDOT has seen problems is on Interstate 85 in Lexington, NC. This project was actually split into two separate projects and included approximately 17 miles of resurfacing over an existing concrete pavement. The first project was where the most trouble occurred. The pavement design called for 4 inches of I 19.0 mm D mix, and 2 inches of S 12.5-mm D mix. The intermediate mix called for PG 70-22 binder, and the surface course called for PG 76-22 binder. Both mixes were designed below the restricted zone.

A few months after completion of this project, NCDOT personnel began seeing water present on the pavement surface days after a rainfall and white stains appearing after this water dried. The NCDOT Materials and Tests Asphalt lab personnel visited the site and cut cores throughout a section of this roadway that was considered in poor condition. When these cores were removed, it was obvious that the binder layer was completely stripped and half of the layer was submerged in water. While some of this water was from the core drill, the condition of the cores led the DOT to conclude that the surface mix was allowing water to seep through and then infiltrate the very porous binder layer, and then attack the already deteriorating (alkali-silica reaction, ASR) concrete pavement. This was the cause of the white stains on the pavement. Prior research by Hearne and Wu *(6)* showed that this original concrete pavement had many problems with water seeping through the joints, apparently the original construction did not include edge drains. This original problem was a concern but it was at this point that the DOT started to feel that the coarse Superpave pavements might need some attention.

North Carolina's problem was not an isolated problem; the Florida Department of Transportation had a similar problem with their pavements and started an investigation, which in turn published a report *(3)* titled, "Investigation of Water Permeability of Coarse Graded Superpave Pavements." The results of this report led to a new device for testing permeability in the lab on field cores and gyratory compacted specimens. The device was similar to what is used in soils to test for permeability. This device was used for the North Carolina study. From this study *(3)*, Florida DOT wrote new specifications which include higher density requirements (>93.5% of theoretical maximum specific gravity, G_{mm}) for designs that pass below the restricted zone and also increased the minimum lift thickness of surface pavements to four times the nominal maximum size aggregate. The reasoning behind this was twofold: to be able to meet the higher compaction specifications a larger lift would be required, and a thicker asphalt layer would lessen the chances of interconnected voids.

1.2 OBJECTIVES AND RESEARCH APPROACH

Throughout the NCDOT's experience with Superpave, the problem of permeable asphalt has reoccurred occasionally and a solution is necessary. This study will discuss results from various projects across the state in order to obtain: (1) a better understanding of the causes of porous asphalt; (2) develop a specification for a maximum allowable coefficient of permeability for coarse designs, and; (3) recommend changes in the current design and production specifications to address the porous asphalt. A standard test method for permeability will also be recommended.

For this study, eight total mixes were investigated, six 12.5-mm surface mixes, and two 9.5-mm surface mixes were chosen for testing, six coarse graded mixes (below the restricted zone) and two fine graded mixes (above the restricted zone). The properties and gradations from the job mix formulas for each mix are provided in Table 1.1. Figures 1.1 and 1.2 show the 0.45 power chart for these designs. The mixes chosen were granite mixes, some containing RAP, and utilizing PG grades of binder with and without polymers. These designs are representative of what is being used across the state and were chosen based on the availability of data for each mix and their inherent problems with permeability. Chapter 3 will discuss the research approach more thoroughly.

Table 1.1 – Test Mix Poperties

	F1	***F2***	***C1***	***C2***	***C3***	***C4***	***C5***	***C6***
NMAS	12.5	12.5	12.5	12.5	12.5	12.5	9.5	9.5
AC%	4.60	5.50	5.00	4.70	4.80	4.80	5.50	5.60
Binder Grade	PG 64-22	PG 67-22	PG 70-22	PG 76-22	PG 76-22	PG 67-22	PG 64-22	PG 64-22
Gmm	2.445	2.492	2.464	2.500	2.593	2.513	2.456	2.513
Gmb, Ndes	2.347	2.390	2.365	2.400	2.489	2.412	2.358	2.413
VTM Ndes	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
VMA Ndes	14	15.4	14.4	15.3	15.3	14.8	16.5	15.3
VFA Ndes	72.0	75.0	71.0	75.0	74.0	69.0	75.5	73.3
Nini/Ndes/Nmax	8/100/160	8/100/160	8/100/160	9/125/205	9/125/205	8/100/160	7/75/115	8/100/160
% Binder from RAP	0.7	-	0.8	-	0.5	-	-	-
Blend Ratio	15/85	-	15/85	-	10/90	-	-	-
% AC in RAP	4.7	-	5	-	4.3	-	-	-
Gradations								
Seive, mm	***F1***	***F2***	***C1***	***C2***	***C3***	***C4***	***C5***	***C6***
25.0	100	100	100	100	100	100	100	100
19.0	100	100	100	100	100	100	100	100
12.5	93	95	94	95	94	93	100	100
9.5	87	88	88	86	87	81	98	97
4.75	59	69	54	49	48	50	63	67
2.36	43	52	33	29	32	33	43	41
1.18	35	36	25	20	25	23	29	28
0.600	23	26	18	13	18	17	21	20
0.300	15	17	11	9	11	11	15	14
0.150	8	10	7	6	8	7	9	10
0.075	4.2	5.8	4.0	4.3	4.5	4.3	5.6	6.8

FIGURE 1.1 - 12.5 mm GRADATIONS

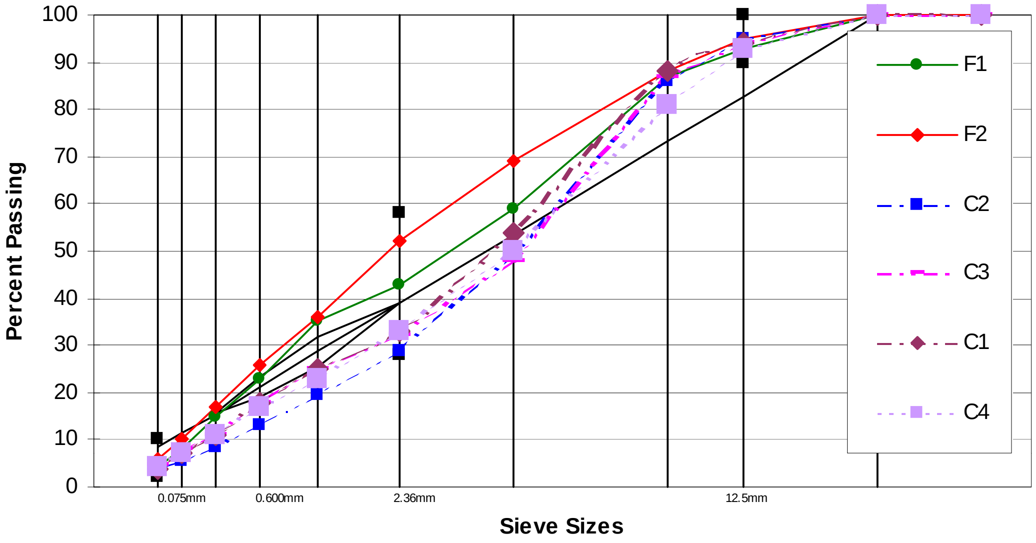
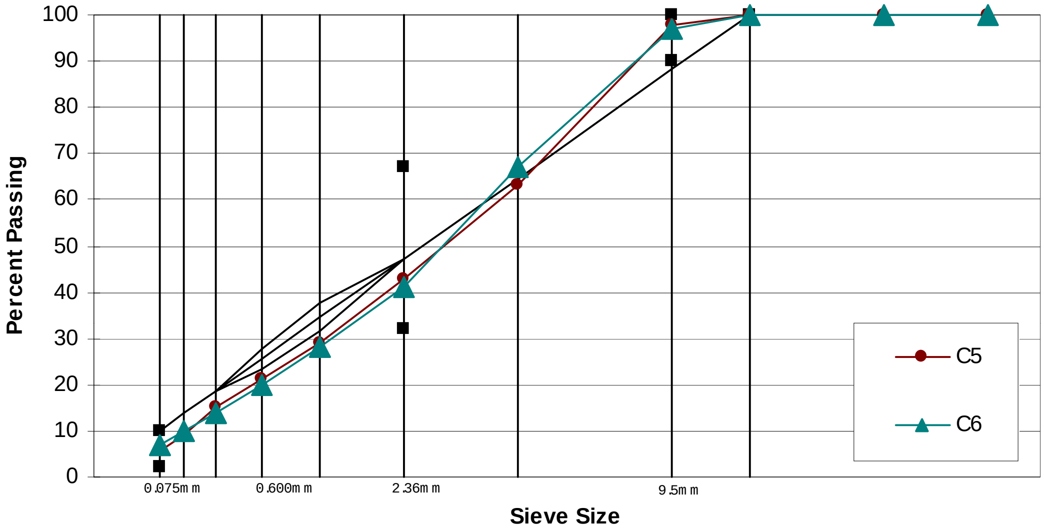


FIGURE 1.2 - 9.5-mm GRADATIONS



1.3 LITERATURE REVIEW

The possible causes of permeable asphalt would include insufficient in place compaction, improper lift thickness, mat segregation, and nominal size of the mix. This section discusses some of the previous research that has been done to study these causes and the effect they have on permeable asphalt.

1.3.1 PERMEABILITY STUDIES

Since the inception of the Superpave system, numerous researchers have investigated permeability in Superpave mixes. The Superpave mix design method addresses the reduction of three important components of pavement distress, permanent deformation, fatigue cracking and low temperature cracking. Permanent deformation, or rutting, can be caused by increased traffic loads due to truck traffic, truck tire inflation pressures exceeding the recommended limit, or by stop and go traffic applying increased loads to the pavements. The Superpave Level 2 mix analysis design method was proposed to test candidate mixtures with varying asphalt contents and volumetric properties to determine which mix will provide adequate performance in the field, i.e., resistance to rutting. Mix designs that fall into the specifications for voids, VMA, and VFA should be resistant to permanent deformation. The goal of the SHRP study was to identify and eliminate the causes of permanent deformation in asphalt pavements and the testing and mix design procedures recommended encourage coarse graded mixtures that have gradations that lie below the restricted zone. The coarse graded nature of Superpave mixes has raised the concerns of the Asphalt industry to the permeability problem and has spurned many research projects. Numerous researchers have shown that permeability, both lab and field, is related to pavement density *(3, 5, 8, 9, 10, 13).*

Certainly, this issue is not a new one. Zube, in 1962 *(8)* studied permeability in pavements in California. He noted that a large number of interconnected voids will allow the passage of air and water into the pavement and will adversely affect the durability and life of the pavement. Zube has indicated that dense graded HMA pavements become permeable to water at approximately 8% in place air voids. Brown, et al. *(7),* later confirmed this. He also noted that compaction played a very important role in the permeability of pavements as well as the time of the year a pavement was placed. Zube suggests that the time of the construction will affect the permeability characteristics. Pavements constructed in the spring can be expected to "seal up" due to the summer traffic thus reducing the permeability better than if the mix was placed in the fall. This is a valid point and shows why a fixed "paving season" is essential to quality pavements. Zube also discusses compaction procedures and types of rollers. Through his research, he shows that a breakdown roller (steel wheel), followed by a pneumatic roller, is the best way to achieve proper density. This point is applicable today in Superpave and is sometimes used in the North Carolina Department of Transportation Project Special Provisions as a requirement *(1).* This paper presents a field method for permeability testing that using a small amount of water soaking into the pavement surface over a certain time period. The water is confined in a small circular area using a "grease ring" or dam around the six-inch area. Other field test methods use this same principal, but with more modern equipment *(9).* Zube also contends that permeability across a mat of asphalt varies greatly due to the distribution of the paver causing segregation. Kennedy, et al. *(12)* suggests that segregation in asphalt mixes can be caused by the mixture gradation, asphalt content and, most likely, construction practices. Visual segregation on the roadway is usually not indicative of the improper mix design itself or the materials used, but is most likely caused by improper material handling or misuse of the paver. This would explain variability in field core permeability, and the consistency in lab specimen permeability that will be shown in this study.

Permeability research has recently become a hot topic and most of the recent research has helped us understand the causes and prevention of permeable asphalt pavements. Research by Cooley, et al. *(5, 9)* introduced a new generation field permeameter (Figure 1.3) that has a three-tiered column of water and is sealed to the pavement surface with silicone sealer. Water is pumped into the column and the time for the water to travel a specific distance is recorded. This method used the same principals used by Zube. Study of coarse graded pavements by Cooley, et al. *(5)* shows that there is a strong relationship between field permeability and in place air voids for coarse graded Superpave pavements. This issue will be researched further and verified by this study.

Figure 1.3 – Field Permeameter

Cooley also concluded that S9.5 mm and S12.5 mm surface mixes have similar permeability characteristics, and therefore the results can be combined for analysis. This conclusion will be verified and used in this research paper. Two critical values are presented by the Cooley, et al. research, 92.3% for percent density and 100x10^{-5} cm/s for permeability. These numbers represent the critical values at which permeability will be become too high in asphalt pavements.

Further research by Cooley, et al. *(9)* studied the correlation between the lab permeability device and the field device. This research shows that the field permeameter and lab permeameter are very similar in their results, except in cases where the permeability was very high (> 500 x 10^{-5} cm/s). An important conclusion from this research is that there is a relationship between in place density, lift thickness and permeability. As density increases, permeability decreases and as lift thickness increases, permeability decreases, which is expected. Cooley's research also discussed the relationship between nominal maximum

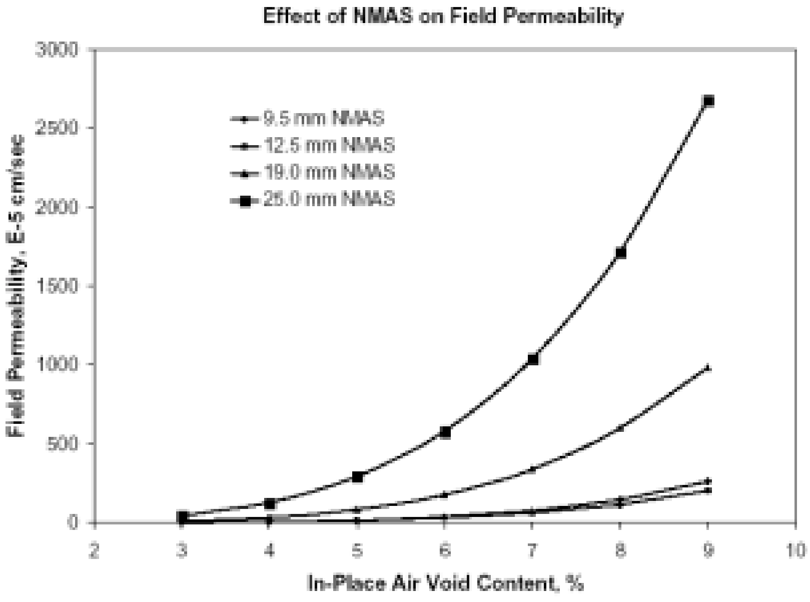


Figure 1.4 – Effect of NMAS on Permeability

aggregate size (NMAS) and permeability, as shown in Figure 1.4.

As expected, the coarser 25.0-mm and 19.0-mm mixes have a greater chance of permeability with higher voids. The 25.0-mm mixes showed that at a given void content, they will have almost 3 times higher permeability values than the 19.0-mm designs, and the 19.0-mm designs, at a given void content, will be significantly higher than the 9.5-mm and 12.5-mm mixes. This research also shows that the permeability characteristics of the 9.5-mm and 12.5-mm mixes are similar. These results also clearly show that the NMAS affects the permeability characteristics of a pavement.

An investigation of permeability by Choubane et al. with the Florida Department of Transportation (FDOT) *(3)* focuses on the permeability characteristics of coarse graded Superpave mixes using a lab device that is similar to the current Karol-Warner device used in this study. The FDOT investigated seven 19.0-mm mixes and nine 12.5-mm mixes from various projects across the State. They also checked each mix for stripping using the Tensile Strength ratio test (AASHTO T283) on field cores. This study led to the FDOT increasing their density requirements to 93.5% compaction for coarse graded designs as well as a limit on permeability for these designs. The limit on the coefficient of permeability, *k*, that was originally recommended by this study was 100 x 10^{-5} cm/s, but this number was later changed to 125 x 10^{-5} cm/s after FDOT switched to the prototype ASTM permeameter.

Further research in Florida by Mussleman et al. *(10)* has shown that lift thicknesses on the roadway can also affect density and permeability. The referenced report stated that existing lift thickness criteria for the current Marshall mix designs was not adequate for coarse graded Superpave designs and that these coarse graded mixes may require a higher level of density to reduce water permeability. Musselman et al. suggested minimum lift thicknesses for coarse graded Superpave mixes to be approximately 4 times the nominal maximum aggregate size. All of the mixes tested in this present study follow the North Carolina design specifications that call for a lift thickness that is only three times the nominal maximum aggregate size.

Research by Maupin *(13)* and the Virginia Department of Transportation studied field core permeability in an effort to determine the number of in-place mixes across Virginia that were susceptible to water infiltration. Using the specification limit set by FDOT for permeability (k = 125 x 10^{-5} cm/s), Maupin tried to determine the maximum voids needed for different nominal maximum aggregate size mixes to be impermeable. By comparing permeability results for field cores and lab specimens, Maupin tried to determine if lab specimens could be used to indicate field permeability. In four of the five mixes tested, highly permeable lab specimens were accurate predictors of permeable field mixes. Maupin also concluded that the segregation in the field produced large variability in the permeability results and was most likely caused by poor quality control.

1.3.2 RUTTING AND PERFORMANCE STUDIES

One of the objectives of this study is to differentiate between what makes an asphalt mix perform well and what makes an asphalt mix perform poorly. One way of making this determination is by doing performance tests on mix specimens. In this study the Asphalt Pavement Analyzer (APA) wheel tracking device and the Superpave Shear tester were used to determine if the performance characteristics of the test mixes correlate to the permeability results.

Research by Kandhal and Cooley *(11)* studied rutting potential for fine and coarse graded mixtures using the APA and the SST and found that there was no significant differences in rut potential for the two gradation types. If we assume this conclusion to be valid, then the mixes used for this research should be ranked based solely on their permeability performance, and not rutting results. More importantly, if a mix tends to rut poorly in the APA, it will most likely be caused by water infiltration into the mix resulting in stripping of the aggregates. Comparison of fine versus coarse graded mixes is an integral part of this study, as it will try to show that fine graded mixes are less susceptible to permeability than coarse mixes. A study performed by Zhang et al. *(14)* compared results from the APA and the SST and found a good correlation in their ability to evaluate permanent deformation in the asphalt mixes that were tested. This good correlation shows a similar behavior of HMA mixes under RSCH and APA loading conditions. The study also recommends a critical rut depth of 8.2 mm for APA specimens tested at a temperature corresponding to the high temperature of the PG grade used in the mix.

CHAPTER 2 – MATERIALS CHRACTERIZATION

Surface mixes with 9.5-mm and 12.5-mm nominal maximum sizes were chosen for evaluation due to the fact that this was where most of the permeability problems have been occurring, and the coarse nature of intermediate 19.0-mm and base 25.0-mm mixes would not contribute any useful data. This study was not designed per se from scratch, so each design used varies in material types and blends, with the only constant being that all the designs are 9.5-mm or 12.5-mm surface mixes.

The two 9.5-mm mixes and four of the 12.5-mm mixes chosen came from six projects in North Carolina and were designed by the Contractors' mix design technicians. The remaining two 12.5-mm mixes are from the National Center for Asphalt Technology Test Track facility in Auburn, AL, and were designed by the Materials and Tests Asphalt lab. The mixes are labeled as follows:

Table 2.1 – Nomenclature for Test Mixes

Design Number	***Mix Type***	***Binder Grade***
F1	RS 12.5C	PG 64-22
F2	S 12.5C	PG 67-22
C1	RS 12.5C	PG 70-22
C2	S 12.5D	PG 76-22
C3	RS 12.5D	PG 76-22
C4	S 12.5C	PG 67-22
C5	S 9.5B	PG 64-22
C6	S 9.5C	PG 64-22

The F prefix indicates the fine graded mixes (above restricted zone), and the C prefix indicates the coarse graded mixes (below restricted zone). Table 2.1 presents design information for all 8 of the designs utilized in this study. All of the original design information is included in Appendix A of this report.

2.1 INDIVIDUAL MIX CHARACTERIZATION

The sub-chapters below describe the materials used in each mix that was studied. Asphalt type, aggregate type and anti-strip agent used is described and some information about the field cores is included.

2.1.1 F1 – FINE GRADED RS12.5C

Mix F1 is a C level (8/100/160 gyration compaction effort), fine graded surface mix that was designed with 15% reclaimed asphalt pavement (RAP). The mix contains granite from Neverson Quarry located in Nash County in northeastern North Carolina. The aggregate sizes used for this design were #67 stone, # 78m stone, dry screenings, and natural sand. The natural sand is from the Daniels sand pit owned by the ST Wooten Corp. and is located in Wilson County. The RAP is from the Sims plant stockpile. Normally, in North Carolina, RAP stockpiles at each plant are mechanically crushed to a uniform size before introduction into the HMA mixture. AC content varies during construction, but QA/QC requirements suggest periodic tests on the AC content in the stockpile to account for variability. The percentages of each material, individual gradations, and the bulk and apparent specific gravities for the aggregates used in this design are shown in Table 2.2 below.

Table 2.2 – Material Gradation and Percentages, Mix F1

MATERIAL	#67	#78m	D. Scrgs	N. Sand	RAP	Blend
PERCENT (MD)	15.0	39.0	16.0	15.0	15.0	100.0
Sieves (mm) 50.0	100.0	100.0	100.0	100.0	100.0	100.0
37.5	100.0	100.0	100.0	100.0	100.0	100.0
25.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	95.0	100.0	100.0	100.0	100.0	99.3
12.5	53.0	100.0	100.0	100.0	98.0	92.7
9.5	33.0	90.0	100.0	99.0	95.0	85.2
4.75	7.0	30.0	95.0	96.0	84.0	55.0
2.36	3.0	8.0	77.0	91.0	71.0	40.2
1.18	2.0	3.4	56.0	78.0	60.0	31.3
0.600	1.8	3.0	39.0	47.0	39.0	20.6
0.300	1.4	1.9	24.0	32.0	26.0	13.5
0.150	1.0	1.0	14.0	6.0	14.0	5.8
0.075	0.7	0.7	7.8	2.8	5.8	2.9
Agg. Bulk Dry S.G.	2.630	2.610	2.560	2.601	2.620	2.605
Agg. Apparent S.G.	2.660	2.650	2.620	2.621	2.650	2.642

The overall absorption for this blend was 0.2%. The original design estimate for total AC was 4.6%, with 0.7% contributed from the reclaimed pavement. The asphalt binder used was a PG 70-22 grade from the Citgo asphalt terminal located in Wilmington, NC and contains 0.25% Arr-Mazz Adhere LOF 6500 anti-strip additive. The specific gravity for the asphalt cement was 1.039.

This mix was produced out of the ST Wooten Sims Plant in Sims, NC and placed on US 264 in Wilson County. This roadway is a 4-lane divided bypass with moderate truck traffic. Cores were cut from the roadway at random locations by the contractor during construction and delivered to the Materials and Tests lab in Raleigh for testing.

2.1.2 F2 – FINE GRADED 12.5C

Mix F2 is a fine graded S12.5C surface mix design that uses all crushed granite material from the Jamestown Quarry in Davidson County in central North Carolina. This mixture contains similar materials as F1, except this is a virgin mix with no RAP. The material sizes are #67's, #78m's, washed screenings and dry screenings. The percentages and gradations are shown in Table 2.3 below.

Table 2.3 - Material Gradation and Percentages, Mix F2

MATERIAL		#67	#78m	D. Scrgs	N. Sand	Blend
PERCENT (MD)		11.0	25.0	32.0	32.0	100.0
Sieves (mm)	50.0	100.0	100.0	100.0	100.0	100.0
	37.5	100.0	100.0	100.0	100.0	100.0
	25.0	100.0	100.0	100.0	100.0	100.0
	19.0	100.0	100.0	100.0	100.0	100.0
	12.5	58.0	99.0	100.0	100.0	95.1
	9.5	26.0	84.0	100.0	100.0	87.9
	4.75	5.0	22.0	98.0	99.0	69.1
	2.36	2.5	3.4	77.0	83.0	52.3
	1.18	1.5	2.2	51.0	60.0	36.2
	0.600	1.1	2.0	35.0	45.0	26.2
	0.300	1.0	1.0	21.0	31.0	17.0
	0.150	1.0	1.0	9.0	20.0	9.6
	0.075	0.8	0.6	2.6	13.0	5.2
Agg. Bulk Dry S.G.		2.690	2.673	2.684	2.655	2.673
Agg. Apparent S.G.		2.725	2.710	2.698	2.706	2.707

This particular mix contains PG 67-22 from Ergon terminal in Vicksburg, Mississippi and uses 0.75% Morton Morelife 2200 as an anti-strip. PG 67-22 is not an asphalt grade used in North Carolina, but this mix was placed on the National Center for Asphalt Technology Test Track in Auburn, Alabama, where PG 67-22 is a common grade. Generally, many PG 64-22 grade binders will grade out to PG 67-22 in laboratory performance tests, so this is actually a different grade AC in name only. The % optimum AC for this mix was 5.3% and the percent absorption was 0.5%. The specific gravity for this asphalt was 1.029.

The cores that were tested from this mix were cut from the North Carolina test section S10 from the Test Track at Auburn by the NCDOT forces and returned to the lab for testing. The surface layer thickness at the track was 3 inches. The field mix was produced at the test track plant and was placed by APAC- Couch, an Alabama contractor who was chosen to build the track.

2.1.3 C1 – COARSE GRADED RS12.5C

Test mix C1 is a coarse graded surface mix that utilizes 15 percent RAP, coarse and fine crushed granite, and a small percentage of natural sand. The sizes of materials in this design are #67's, #78m's, washed screenings, natural sand and as mentioned above, reclaimed asphalt pavement. The coarse aggregates and washed screenings are a granite material from the Garner Quarry in Wake County in central North Carolina and the natural sand is from Gelder's Raleigh sand pit. The RAP material is a reclaimed material that is stockpiled on the contractors' yard. Table 2.4 below shows the gradation and material percentages.

Table 2.4 - Material Gradation and Percentages, Mix C1

MATERIAL	#67	#78m	D. Scrgs	N. Sand	RAP	Blend
PERCENT (MD)	12.0	48.0	20.0	5.0	15.0	100.0
Sieves (mm) 50.0	100.0	100.0	100.0	100.0	100.0	100.0
37.5	100.0	100.0	100.0	100.0	100.0	100.0
25.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0	100.0	100.0	100.0
12.5	50.0	100.0	100.0	100.0	100.0	94.0
9.5	27.0	93.0	100.0	100.0	98.6	88.0
4.75	4.7	35.0	97.0	100.0	83.4	54.0
2.36	2.6	4.8	76.0	96.0	68.1	33.0
1.18	1.8	2.4	56.0	77.2	55.5	25.0
0.600	1.7	2.0	40.0	40.0	41.3	18.0
0.300	1.6	1.6	24.0	6.5	26.5	11.0
0.150	1.3	1.0	11.0	1.3	15.8	7.0
0.075	0.7	0.5	4.2	0.8	9.2	4.0
Agg. Bulk Dry S.G.	2.627	2.615	2.607	2.610	2.670	2.623
Agg. Apparent S.G.	2.664	2.665	2.657	2.656	2.670	2.664

Mix C1 contains 5.0% PG 70-22 from Citgo Wilmington. 0.8% of the AC is contributed from the RAP and there is 0.5% Arr-Maz Adhere LOF 6500 anti-strip. This design has 0.5% absorption. The AC specific gravity was 1.040.

This mix was produced in the Gelder and Associates plant in Garner and was placed on the NC 55 bypass (including y-lines) in Holly Springs. This roadway is still being built and is a 4 lane divided bypass around the town of Holly Springs. Cores were cut from the pavement at random locations by NCDOT personnel and delivered to NCDOT Materials and Tests lab.

2.1.4 C2 – COARSE GRADED S12.5D

Mix C2 is a coarse graded D level surface mix (9/125/205 design gyrations) that was produced by Mapco, Inc. out of Asheboro. This mix contains #78m's, #67, washed and dry screening. All of the materials are crushed granite and are from the central part of North Carolina. The 78m's, 67's and washed screenings are from Jamestown Quarry and the dry screenings are from the Asheboro Quarry. The percentages and gradations are shown below in Table 2.5. This mix was designed coarse graded and was one of the original Superpave designs used in the state of North Carolina for an Interstate, it was placed on I-85 in Randolph County. The D design level required a PG 76-22 polymer modified binder because of the large amount of truck traffic on the roadway. Arr-Maz Ad-Here LOF was used at a rate of 0.5% for anti-strip purposes. The design AC was set at 4.7% and the percent absorption for the mix was 0.4%. The AC specific gravity was 1.037. The cores used from this roadway were cut at random locations by NCDOT personnel and delivered to the Materials and Tests lab in Raleigh.

Table 2.5 – Material Gradation and Percentages, Mix C2

MATERIAL	#67	#78m	W. Scrgs	D. Scrgs	Blend
PERCENT (MD)	10.0	45.0	27.0	18.0	100.0
Sieves(mm) 50.0	100.0	100.0	100.0	100.0	100.0
37.5	100.0	100.0	100.0	100.0	100.0
25.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0	100.0	100.0
12.5	58.0	99.0	100.0	100.0	95.0
9.5	26.0	88.0	100.0	100.0	86.0
4.75	6.0	26.0	98.0	100.0	56.8
2.36	2.8	4.0	75.0	80.0	39.0
1.18	2.0	3.0	48.0	51.0	25.0
0.600	1.5	2.0	31.0	38.0	17.0
0.300	1.0	1.0	20.0	24.0	10.3
0.150	1.0	1.0	14.0	10.0	6.1
0.075	0.2	0.6	12.0	2.4	4.3
Agg. Bulk Dry S.G.	2.677	2.657	2.771	2.679	2.694
Agg. Apparent S.G.	2.709	2.688	2.788	2.699	2.719

2.1.5 C3 – COARSE GRADED RS12.5D

Mix C3 is another D level surface mix that utilizes reclaimed asphalt and PG 76-22 binder. The materials for this design are granite aggregate from the Bessemer City Quarry in Gaston County, south of Charlotte, NC. This design like most of the others uses a combination of #67 stone, #78m stone, washed screenings, natural sand, and reclaimed asphalt pavement from the Rea Construction stockpile at the Bessemer City plant. The gradation and percentages of each material is shown in Table 2.6. The optimum AC content for design was 4.8% and the percent absorption was 0.1%. The grade of AC was PG 76-22 polymer modified asphalt from Citgo Wilmington, and had 0.5% Morton Morelife 3300 as an anti-stripping agent. As with mix C2, the polymer modified asphalt was used due to traffic concerns on the roadway. The specific gravity of this AC was 1.034.

Table 2.6 – Material Gradation and Percentages, Mix C3

MATERIAL	#67	#78m	W. Scrgs	N. Sand	RAP	Blend
PERCENT (MD)	10.0	51.0	19.0	10.0	10.0	100.0
Sieves (mm) 50.0	100.0	100.0	100.0	100.0	100.0	100.0
37.5	100.0	100.0	100.0	100.0	100.0	100.0
25.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	98.0	100.0	100.0	100.0	100.0	99.8
12.5	51.0	99.0	100.0	100.0	97.0	94.3
9.5	25.0	92.0	100.0	100.0	87.0	88.0
4.75	6.0	24.0	97.0	99.0	66.0	54.0
2.36	3.0	5.0	77.0	92.0	54.0	33.0
1.18	2.0	5.0	55.0	71.0	46.0	25.0
0.600	1.0	4.0	43.0	36.0	38.0	18.0
0.300	1.0	3.0	32.0	9.0	20.0	11.0
0.150	1.0	3.0	16.0	3.0	18.0	7.0
0.075	1.0	1.8	6.2	2.0	12.0	4.0
Agg. Bulk Dry S.G.	2.865	2.865	2.860	2.570	2.570	2.805
Agg. Apparent S.G.	2.885	2.885	2.881	2.586	2.579	2.824

This mix was placed on I-85 in Gaston County. This section of I 85 is subject to large amounts of truck traffic, approximately 25%, and the use of PG 76-22 is warranted. The field cores were cut out of the roadway at random locations by NCDOT personnel and returned to the Materials and Tests lab in Raleigh.

2.1.6 C4 – COARSE GRADED S12.5C

Mix C4 is another coarse graded surface mix and, similar to mix F2, is from the Test Track at Auburn. This mix is a C level design with design gyrations of 8/100/160. The NCDOT has two sections at the test track using the same materials, one design is fine graded (F2) and the other is coarse graded (C4). Like the F2 mix, C4 uses granite from the Jamestown quarry. The sizes of materials used are #78m's, #67's, dry screenings and washed screenings. The percentages and gradations for each are shown below in Table 2.7.

Table 2.7 – Material Gradation and Percentages, Mix C4

MATERIAL		#67	#78m	W. Scrgs	D. Scrgs	Blend
PERCENT (MD)		15.0	47.0	18.0	19.0	100.0
Sieves (mm)	50.0	100.0	100.0	100.0	100.0	100.0
	37.5	100.0	100.0	100.0	100.0	100.0
	25.0	100.0	100.0	100.0	100.0	100.0
	19.0	100.0	100.0	100.0	100.0	99.0
	12.5	58.0	100.0	100.0	100.0	95.0
	9.5	26.0	84.0	100.0	100.0	86.0
	4.75	5.0	26.0	98.0	99.0	49.4
	2.36	3.0	3.0	77.0	83.0	39.0
	1.18	2.0	2.0	51.0	60.0	25.0
	0.600	1.1	2.0	35.0	45.0	17.0
	0.300	0.6	1.0	21.0	31.0	10.2
	0.150	0.5	1.0	9.0	20.0	6.0
	0.075	0.2	0.6	3.0	13.0	4.3
Agg. Bulk Dry S.G.		2.690	2.673	2.684	2.655	2.647
Agg. Apparent S.G.		2.725	2.710	2.698	2.706	2.682

This design uses PG 67-22 from the Ergon Vicksburg, Mississippi terminal and has 0.75% Morton Morelife 2200 as an anti-strip. The AC specific gravity was 1.029. The optimum AC content at design was 5.3% and the percent absorption was 0.5%. Field cores were taken from section S9 at the test track site at the same time cores for mix F2 were taken (section S10).

2.1.7 C5 – COARSE GRADED S9.5B

Mix C5 is the first of two 9.5-mm mixes used in this study. As mentioned in the literature review, 9.5-mm and 12.5-mm mixes have similar permeability characteristics and can be analyzed together if necessary. This particular 9.5-mm mix is a B mix and uses design gyrations of 7/75/115. The 9.5B mixes are usually used on lower volume roads and are rarely coarse graded, but this was an exception. The materials used for this design were a granite materials from the Holly Springs Quarry, in Holly Springs, and the Leon Gardner Quarry, in

Lillington. The Nello L. Teer Co. produced this mix out of their Holly Springs batch plant. Both quarries are located southwest of Raleigh in the central part of the state.

The sizes used include #78m's, washed screenings (Holly Springs) and dry screenings (Leon Gardner). The combination of two different quarry materials in a mix design is not uncommon and is sometimes necessitated by cost control, or availability. The gradations and percentages of each material are shown in Table 2.8 below. The optimum AC content for this design was 5.5% and the absorption was 0.1%. This asphalt binder used was a PG 64-22 from Citgo Wilmington with 0.5% Arr-Mazz Adhere LOF 6500 anti-strip. This AC had a specific gravity of 1.036.

This mix was placed on US 70 (Garner Road) in Garner, NC south of Raleigh. The contractor cut the field cores at random locations and delivered them to the Materials and Tests lab in Raleigh.

Table 2.8 – Material Gradations and Percentages, Mix C5

MATERIAL		#78m	D. Scrgs (LG)	W. Scrgs (HS)	Blend
PERCENT (MD)		53.0	45.0	2.0	100.0
Sieves (mm)	50.0	100.0	100.0	100.0	100.0
	37.5	100.0	100.0	100.0	100.0
	25.0	100.0	100.0	100.0	100.0
	19.0	100.0	100.0	100.0	100.0
	12.5	100.0	100.0	100.0	100.0
	9.5	96.0	100.0	100.0	98.0
	4.75	31.0	99.0	96.0	63.0
	2.36	7.0	84.0	68.0	43.0
	1.18	1.0	62.0	42.0	29.0
	0.600	1.0	45.0	26.0	21.0
	0.300	1.0	31.0	17.0	15.0
	0.150	1.0	19.0	10.0	9.0
	0.075	1.0	11.0	6.0	5.6
Agg. Bulk Dry S.G.		2.644	2.684	2.647	2.662
Agg. Apparent S.G.		2.735	2.719	2.692	2.727

2.1.8 C6 – COARSE GRADED S9.5C

Mix C6 is a coarse graded 9.5 C level design that was designed using 8/100/160 gyrations. This is a C level mix that was used on US 421 in Watauga County, in the northwestern part of North Carolina. This route is a four-lane divided highway with moderate traffic. Limestone coarse and fine aggregates from the Shouns Quarry were used. This is the only mix tested that uses limestone materials, all of the other mixes tested in this study use granite. The sizes used were a #8 P, which is a special size aggregate that is similar to a 78m, dry screenings and washed screenings. The percentages and gradations are shown in Table 2.9 below.

The optimum AC content was set at 5.6% and the absorption for this mix was 0.8%. The binder used was a PG 70-22 grade from the Tosco-Petro Bristol, TN terminal and used 0.5% Arr-Mazz Adhere LOF 6500 as an anti-strip. The specific gravity of the AC was 1.031. Cores were cut from this pavement by the contractor personnel and delivered to the Materials and Tests lab in Raleigh for further testing.

Table 2.9 – Material Gradation and Percentages, Mix C6

MATERIAL		#8 p	D. Scrgs	W. Scrgs	Blend
PERCENT (MD)		30.0	60.0	10.0	100.0
Sieves (mm)	50.0	100.0	100.0	100.0	100.0
	37.5	100.0	100.0	100.0	100.0
	25.0	100.0	100.0	100.0	100.0
	19.0	100.0	100.0	100.0	100.0
	12.5	99.0	100.0	100.0	100.0
	9.5	80.0	100.0	100.0	94.0
	4.75	15.0	92.0	75.0	67.0
	2.36	5.0	60.0	20.0	40.0
	1.18	2.0	40.0	4.0	26.0
	0.600	1.0	27.0	1.2	17.0
	0.300	1.0	18.0	0.6	12.0
	0.150	1.0	12.0	0.5	8.0
	0.075	1.0	8.9	0.4	6.4
Agg. Bulk Dry S.G.		2.684	2.680	2.683	2.682
Agg. Apparent S.G.		2.740	2.740	2.730	2.739

CHAPTER 3 - RESEARCH APPROACH AND METHODOLOGY

This research project is composed of three major phases. The first phase was to obtain field core specimens and evaluate these specimens for density and permeability using the lab permeameter. The second phase used lab specimens compacted to match the field density and were tested for permeability and rut susceptibility, both in wet and dry conditions, using the Asphalt Pavement Analyzer (APA). The third phase used lab compacted specimens tested in the Superpave Shear Tester (SST) to predict the effect of the poor compaction on field performance, similar to the APA, and to try to correlate the results of the two performance tests.

3.1 FIELD SAMPLE TESTING

In order to determine the degree of water permeability in the pavements, 150-mm cores were taken from each pavement at random locations and tested in the laboratory for density (specific gravity) and permeability.

Density was determined using AASHTO method T166, Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface-Dry Specimens. For the permeability testing, a modified version of the falling head permeability test developed by the Florida Department of Transportation, test FM 5-565 *(3)* was used. Following the proposed North Carolina DOT test specification (A-100), the core was saturated with a vacuum pump for 5 to 10 minutes, then tested with the permeability device. The permeability device used is manufactured by Karol-Warner (Figure 3.1) and consists primarily of a permeability cell, lined with a rubber membrane, a plastic graduated cylinder, and a small hand operated air pump. Using this lab permeameter, we were able to determine the coefficient of permeability, *k*.

Figure 3.1 – Karol-Warner Lab Permeameter

Three trials are run for each specimen, and the time it takes for the water in the cylinder to travel 40 cm is recorded. The three times are then averaged. The core diameter, and thickness are also measured at three locations and averaged.

For the Karol Warner device shown in Figure 3.1, water from a graduated cylinder flows through a saturated asphalt sample that is confined by a rubber membrane, and the time for this water to flow 40 cm is recorded. The coefficient of permeability, *k*, is determined based on Darcy's Law using the following equation:

$$k = \frac{a\,L}{A\,t}\,1\text{n}(h_1/h_2) \times t_c$$

Where: k = coefficient of permeability, cm/s;

a = inside cross-sectional area of the buret, cm^2 (constant)

L= average thickness of the test specimen, cm;

A= average 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 (constant);

h_2= final head across the test specimen, cm (constant);

t_c = temperature correction for viscosity of water;

3.2 LAB SAMPLE TESTING

Materials for each mix design were collected from the appropriate quarries, pits and terminals and lab specimens were prepared. These lab specimens were compacted using the Troxler Superpave Gyratory Compactor 4140 to average field density (voids) of each respective mix sampled. Using information from the job mix formulas, the specimens were mixed together at the corresponding blend percentages and AC content, as per the JMF. The materials used were described in Chapter 2 – Materials Characterization. These lab specimens were tested for voids, permeability, and then tested for rutting in the APA. The permeability and void analysis were described in the above section. The Asphalt Pavement Analyzer (APA), shown in Figure 3.2, is a loaded wheel rut tester that applies a constant pressure of 100 psi to 3 hoses that rest on the asphalt specimen. Steel wheels apply the pressure to the rubber hoses that are supplied with 100 psi of compressed air. These rubber hoses are used to simulate the effect of rubber tires on the roadway. The rut tester runs for 8000 cycles and has an automatic rut depth measurement utilizing transducers to track the change in depth. This information is fed into a computer and later printed out with a chart showing rut depth versus number of cycles. The rut tester also has the capability to submerge the specimens in water prior to and during testing.

For this study, it was decided that one set of specimens would be tested dry at 64°C as a control set. Wet specimens were saturated and conditioned as per AASHTO T283, Resistance of Compacted Asphalt Mixtures to Moisture Induced Damage. These specimens were then rut tested under water at a chamber temperature of 64°C and a water temperature of 60°. This was done to simulate the effect of water infiltration on the roadway.

Figure 3.2 – Asphalt Pavement Analyzer

3.3 LAB SAMPLE SST TESTING

Shear test specimens were compacted for only four of the 8 test mixes. The four mixes used were C2, C5, F1, and F2. Mixes C2 and C5 were chosen since they were the most permeable (field) coarse graded mixes. Since using fine graded mixes seems to be the easiest way to eliminate permeable pavements, it seems logical to compare the fine mixes F1 and F2 to the coarse mixes, C2 and C5, to see if there is a difference in performance between coarse and fine graded mixes. One 125 mm gyratory specimen was compacted to in-place voids for each of the listed mixes and then cut into 2 – 50 mm specimens for testing at North Carolina State University with the Cox and Sons Superpave Shear Tester. These mixes were subjected to the repeated shear at constant height (RSCH) test at 60°C using AASHTO TP-7 Procedure F. The SST is a closed loop feedback hydraulic system that consists of the following components: the testing apparatus, the test control unit and data acquisition system, the environmental chamber, and the hydraulic system. This test consists of applying a repeated haversine shear stress of 68kPa to a compacted HMA specimen while supplying necessary axial stress to maintain a constant height. The RSCH test is used to estimate the rut depth and is generally run to 5000 cycles or until the permanent shear strain reaches five percent. The predicted rut results from the SST will be compared to the APA rut results. From the prior research mentioned above *(14),* the RSCH and the APA test results correlate well when trying to predict permanent deformation.

3.4 SUMMARY OF RESEARCH APPROACH

The general research approach to evaluate the effects of water infiltration into Superpave surface mixes can be summarized as follows:

1) Evaluate cores retrieved from roadway for permeability and density.
2) Manufacture lab specimens to field specifications and test for density, permeability and resistance to rutting and compare lab specimens to field cores.
3) Compact lab specimens for testing in the Superpave Shear tester for simple shear at constant height.

The flow chart in Figure 3.3 outlines the research approach.

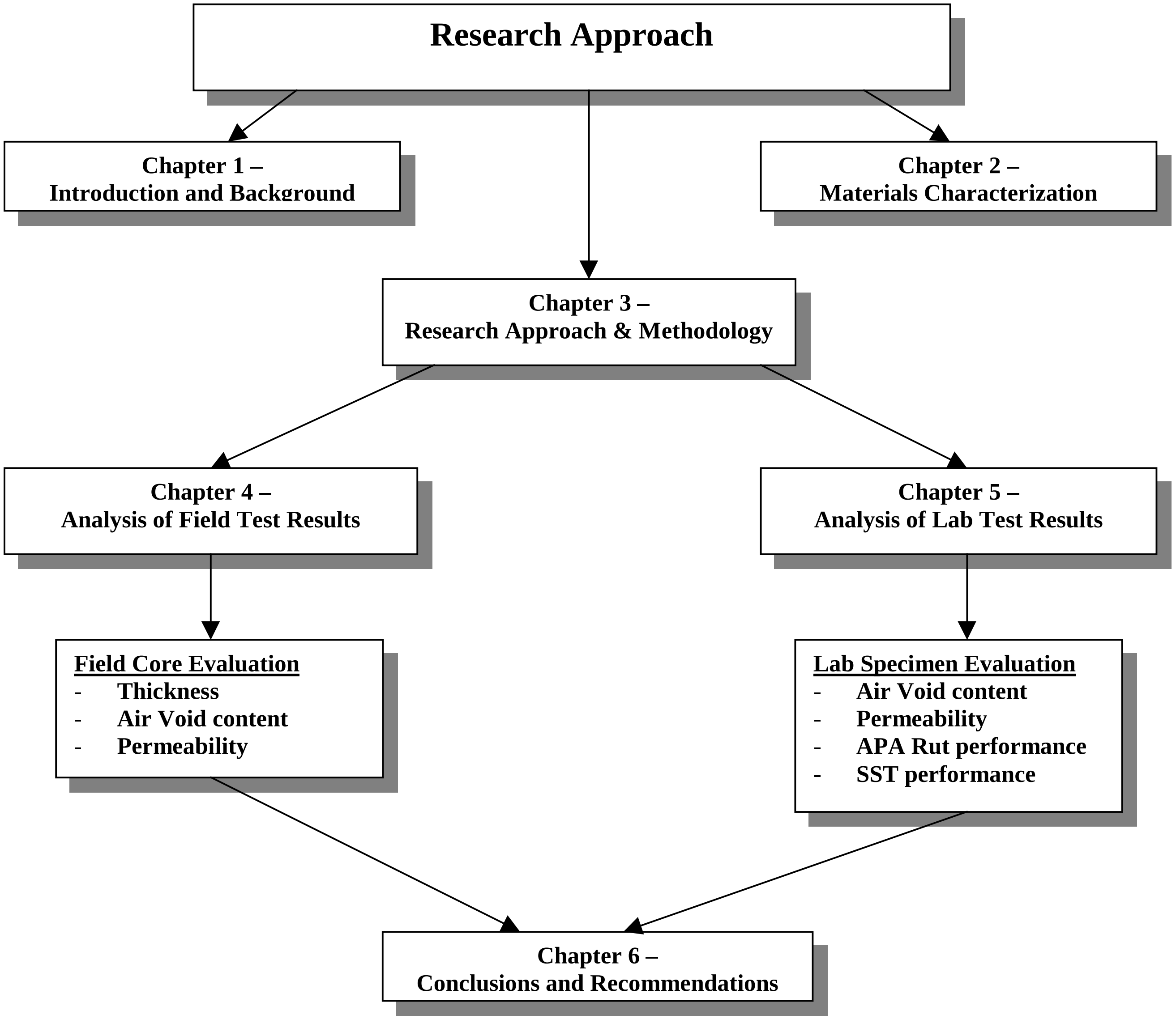


Figure 3.3 Summary of research approach and methodology

CHAPTER 4 – FIELD CORE SAMPLING AND TESTING

4.1 VOIDS AND LIFT THICKNESS

As indicated earlier, from each project, 150-mm (6-inch) core specimens were cut from the pavement layer at random locations and were brought back to the lab for further testing. A total of 89 cores from 8 locations were tested. Each core was removed from the pavement, brought to the lab, wet-saw cut at the interface of the surface layer and the intermediate layers, washed to remove any fines, then placed under a fan to dry. For this study, only the surface layers were tested. The sample thickness represented the lift thickness on the roadway and was measured with a digital caliper at three locations around the specimen and averaged. After drying thoroughly, the bulk specific gravity (G_{mb}) was determined using AASHTO T166. Then, using the maximum specific gravity (G_{mm}) from the original JMF, the voids were determined and recorded. The voids were calculated using the following formula:

$$\% \text{ Air Voids} = 100 \text{ x } [(G_{mm} - G_{mb}) / G_{mm}]$$

where;

G_{mm} = Maximum Specific Gravity from the original Job Mix Formula

G_{mb} = Bulk Specific Gravity determined by AASHTO T166

Tables 4.1 through 4.8 show the results for the field cores from each project. The number of cores taken at each site varied due to either the length of the project, or the personnel cutting the cores at the roadway. Generally, when the NCDOT personnel cut the cores, as many cores as possible were taken in case of damage during transport or storage. However, when contractor's personnel removed the cores, the amount varied.

Table 4.1 – Core Properties, Mix F1

Lab Number	*Mix Type*	*Percent Voids*	*In-Place % Compaction*	*Average Thickness, mm*
F1-1	RS12.5C	7.0	93.0	48.3
F1-2	RS12.5C	7.2	92.8	38.3
F1-3	RS12.5C	6.2	93.8	45.7
F1-4	RS12.5C	6.9	93.1	39.3
F1-5	RS12.5C	6.3	93.7	48.3
F1-6	RS12.5C	7.0	93.0	38.3
F1-7	RS12.5C	7.4	92.6	46.0
F1-8	RS12.5C	7.0	93.0	39.3
F1-9	RS12.5C	7.0	93.0	44.0
F1-10	RS12.5C	6.6	93.4	38.3

Table 4.2 – Core Properties, Mix F2

Lab Number	*Mix Type*	*Percent Voids*	*In-Place % Compaction*	*Average Thickness, mm*
F2-1	S12.5C	4.4	95.6	80.0
F2-2	S12.5C	4.6	95.4	80.0
F2-3	S12.5C	6.5	93.5	82.0
F2-4	S12.5C	5.5	94.6	85.3
F2-5	S12.5C	7.0	93.1	78.7
F2-6	S12.5C	6.6	93.4	81.7
F2-7	S12.5C	4.2	95.8	82.1
F2-8	S12.5C	4.0	96.0	81.3
F2-9	S12.5C	4.9	95.1	81.5
F2-10	S12.5C	5.6	94.4	82.0
F2-11	S12.5C	4.5	95.5	81.0
F2-12	S12.5C	5.0	95.0	82.3

Table 4.3 – Core Properties, Mix C1

Lab Number	*Mix Type*	*Percent Voids*	*In-Place % Compaction*	*Average Thickness, mm*
C1-1	RS12.5C	8.9	91.2	36.3
C1-2	RS12.5C	9.5	90.5	48.7
C1-3	RS12.5C	5.3	94.7	49.1
C1-4	RS12.5C	10.6	89.4	39.5
C1-5	RS12.5C	8.2	91.8	61.1
C1-6	RS12.5C	7.0	93.0	53.3
C1-7	RS12.5C	10.6	89.5	56.5
C1-8	RS12.5C	8.2	91.8	52.5

Table 4.4 – Core Properties, Mix C2

Lab Number	*Mix Type*	*Percent Voids*	*In-Place % Compaction*	*Average Thickness, mm*
C2-1	S12.5D	7.5	92.5	30.0
C2-2	S12.5D	9.0	91.0	36.7
C2-3	S12.5D	6.9	93.1	37.7
C2-4	S12.5D	5.4	94.6	37.7
C2-5	S12.5D	6.8	93.2	39.3
C2-6	S12.5D	5.7	94.3	39.7
C2-7	S12.5D	7.4	92.6	37.7
C2-8	S12.5D	8.9	91.1	39.0
C2-9	S12.5D	10.8	89.2	41.0
C2-10	S12.5D	13.4	86.6	38.3
C2-11	S12.5D	12.4	87.6	38.3
C2-12	S12.5D	12.6	87.5	40.7
C2-13	S12.5D	13.2	86.9	40.3
C2-14	S12.5D	12.4	87.6	39.7

Table 4.5 – Core Properties, Mix C3

Lab Number	*Mix Type*	*Percent Voids*	*In-Place % Compaction*	*Average Thickness, mm*
C3-1	RS12.5D	11.0	89.0	29.0
C3-2	RS12.5D	4.4	95.6	42.3
C3-3	RS12.5D	6.4	93.6	50.7
C3-4	RS12.5D	10.1	89.9	26.7
C3-5	RS12.5D	10.7	89.3	22.0
C3-6	RS12.5D	14.2	85.8	21.3
C3-7	RS12.5D	10.0	90.0	27.7
C3-8	RS12.5D	8.9	91.1	33.3
C3-9	RS12.5D	6.9	93.1	28.0
C3-10	RS12.5D	4.3	95.7	27.3
C3-11	RS12.5D	15.0	85.0	31.0
C3-12	RS12.5D	14.0	86.0	25.0
C3-13	RS12.5D	11.1	88.9	24.7
C3-14	RS12.5D	13.9	86.1	20.0
C3-15	RS12.5D	13.5	86.5	24.3

Table 4.6 – Core Properties, Mix C4

Lab Number	*Mix Type*	*Percent Voids*	*In-Place % Compaction*	*Average Thickness, mm*
C4-1	S12.5C	3.9	96.1	85.4
C4-2	S12.5C	4.9	95.1	86.7
C4-3	S12.5C	6.2	93.8	86.0
C4-4	S12.5C	5.0	95.1	85.0
C4-5	S12.5C	7.1	92.9	75.0
C4-6	S12.5C	7.0	93.0	75.7
C4-7	S12.5C	3.5	96.5	75.0
C4-8	S12.5C	2.7	97.3	76.0
C4-9	S12.5C	4.5	95.5	76.7
C4-10	S12.5C	4.9	95.1	76.7
C4-11	S12.5C	5.8	94.2	78.7
C4-12	S12.5C	5.7	94.3	75.7

Table 4.7 – Core Properties, Mix C5

Lab Number	*Mix Type*	*Percent Voids*	*In-Place % Compaction*	*Average Thickness, mm*
C5-1	S9.5B	7.3	92.7	26.7
C5-2	S9.5B	11.5	88.5	21.6
C5-3	S9.5B	10.2	89.8	33.9
C5-4	S9.5B	10.1	89.9	32.6
C5-5	S9.5B	9.2	90.8	34.0
C5-6	S9.5B	10.3	89.7	35.1

Table 4.8 – Core Properties, Mix C6

Lab Number	*Mix Type*	*Percent Voids*	*In-Place % Compaction*	*Average Thickness, mm*
C6-1	S9.5C	10.9	89.1	32.4
C6-2	S9.5C	6.3	93.8	37.1
C6-3	S9.5C	6.1	94.0	30.2
C6-4	S9.5C	7.8	92.2	51.2
C6-5	S9.5C	9.5	90.5	44.6
C6-6	S9.5C	8.6	91.4	36.6
C6-7	S9.5C	8.6	91.4	37.9
C6-8	S9.5C	12.4	87.6	35.0
C6-9	S9.5C	7.0	93.0	42.9
C6-10	S9.5C	10.0	90.0	36.1
C6-11	S9.5C	10.7	89.3	33.2
C6-12	S9.5C	9.1	90.9	40.7

Table 4.9 shows the average percent air voids, percent compaction, and specimen thickness for each test mix.

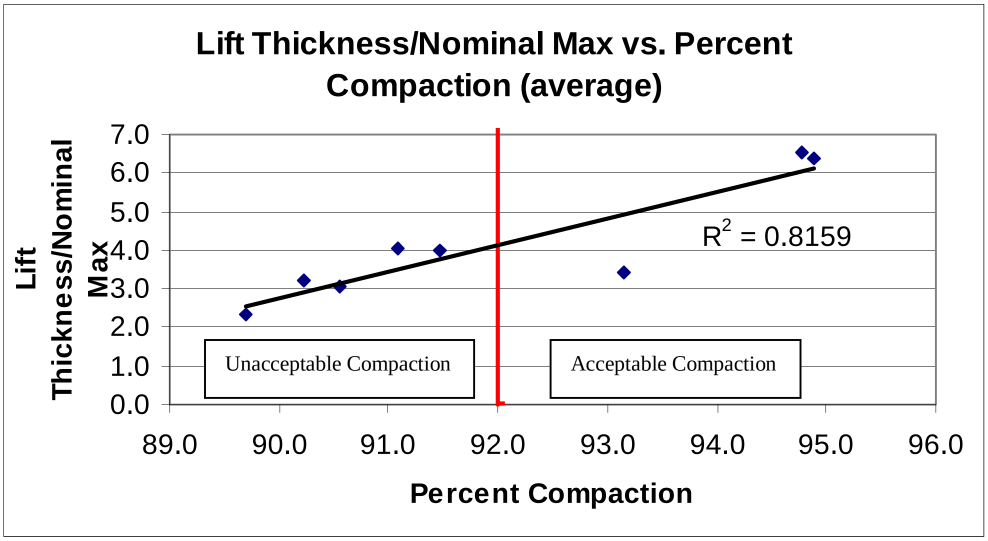
Table 4.9 – Average Core Properties, All Mixes

Test Mix	*Mix Type*	*Average Air Voids*	*Average Percent Compaction*	*Average Specimen Thickness, mm*	*Thickness/ NMAS**
F1	RS12.5C	6.86	93.14	42.60	3.41
F2	S12.5C	5.23	94.77	81.50	6.52
C1	RS12.5C	8.54	91.46	49.60	3.97
C2	S12.5D	9.45	90.55	38.30	3.06
C3	RS12.5D	10.30	89.70	28.90	2.31
C4	S12.5C	5.11	94.89	79.40	6.35
C5	S9.5B	9.77	90.23	30.70	3.23
C6	S9.5C	8.91	91.09	38.20	4.02

*NMAS - Nominal Maximum Aggregate Size

As can be seen by the averages in Table 4.9 above and in Figure 4.1 below, a relationship exists between percent compaction and lift thickness. Prior research *(10)* has indicated that thinner lifts may increase interconnected voids, which will likely increase permeability. The above data suggest that interconnected voids may increase with lower thicknesses, but how this effects permeability will be explored in Section 4.2. The above averages also show a difference in lift thicknesses between the Test Track mixes, F2 and C4, and the State project mixes, F1, C1, C2, C3, C5, and C6. Each lift for all of the above designs was specified to be placed at three times the nominal maximum aggregate size (NMAS), 37.5-mm (1 ½”) for 12.5-mm mixes, and 30-mm (1 ¼”) for 9.5-mm mixes. The measured difference in the cores may be due to the controlled environment that the Test Track was built in, where money and materials were not a limiting factor and there were ideal conditions for lay down and compaction. Figure 4.1 is a plot of lift thickness divided by nominal maximum aggregate size versus percent compaction. The lift thickness is divided by the NMAS so that the data will not be skewed by the two NMAS surface mixes being tested. This plot shows that there is a very strong correlation (R^2=0.82) between lift thickness and percent compaction.

Figure 4.1 – Lift Thickness/NMAS vs. Percent Compaction for Field Cores



Surprisingly, only three of the 8 mixes tested pass the 92% minimum percent compaction required by the NCDOT and two of these mixes were from the Test Track. Only one of the coarse test mixes, C4, passed the 92% specification, and none of the coarse NCDOT project mixes passed. It appears that coarse surface mixes placed less than four times the NMAS may be difficult to compact, thus causing high in place voids. Mix F1 falls below the trend line and may indicate that lift thicknesses three times the NMAS may be adequate for fine graded surface mixes. This verifies earlier research by FDOT *(10)*. From the above data, it is clear, and not unexpected, that in order to achieve proper voids in place there must be enough material thickness so that the mix will be able to compact into a denser condition and expulse excess air.

The field cores were next tested for permeability in the lab. The these test results are described in the next section.

4.2 PERMEABILITY TESTS

Following the test procedures mentioned in Section 3.1, laboratory permeability tests were performed on the surface layers for each of the 89 sample cores. These test results were analyzed for a normal distribution and it was discovered that the data was in fact log normally distributed. Figure 4.2 shows the log-based distribution of the permeability data. SAS version 8.2 software was used for the statistical analysis. Of the 89 specimens, 14 were considered outlying, and were eliminated from any further analysis. The outlying data points affected three of the 8 sample mixes, F2, C3, and C4. Table 4.10 presents the log-based averages for each test mix. The results for each of the 89 specimens are presented in

Appendix A. The average coefficient of permeability, *k*, is shown in 10^{-5} cm/s. The standard deviation shown is the log-based deviation for the permeability results.

Figure 4.2 – Normal Probability Plot, Log Permeability

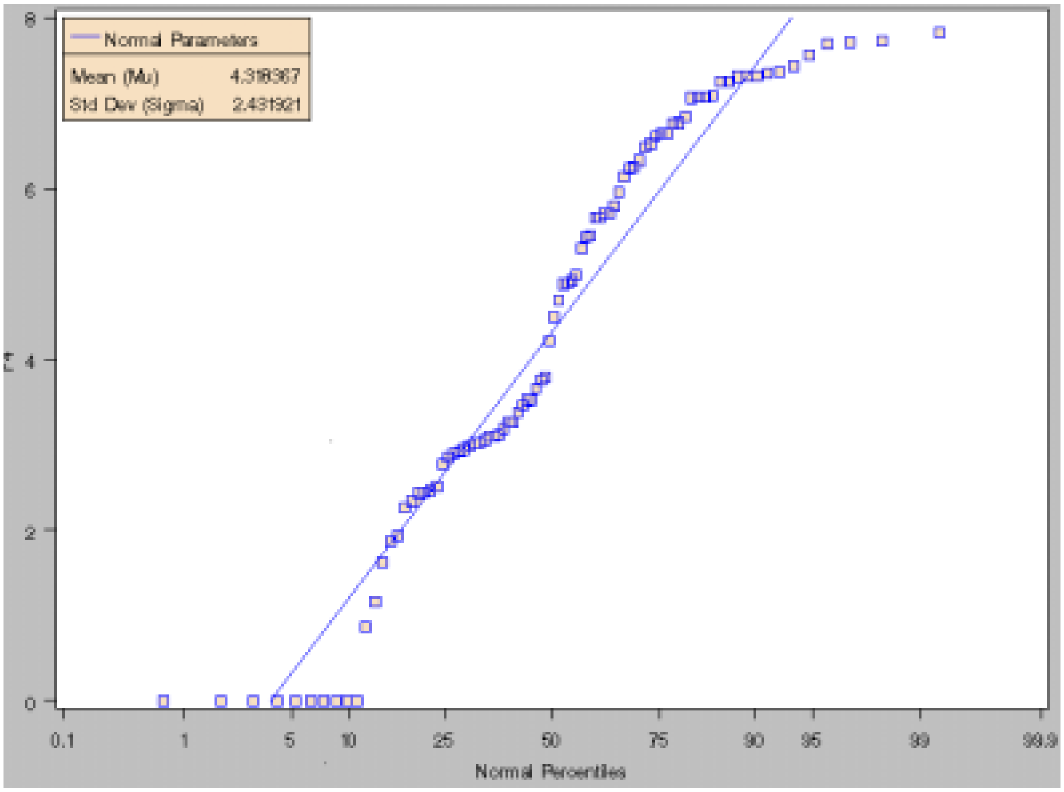


Table 4.10 – Permeability Results

Test Mix	*Mix Type*	*Average Voids*	*Average % Comp*	*Average k, 10^{-5} cm/s*	*Standard Deviation (ln based)*	*Variance (ln based)*	*Coefficient of Variation (ln based)*
F1	RS12.5C	6.86	93.14	18.0	0.48	0.233	51.2
F2	S12.5C	5.23	94.77	11.0	0.76	0.578	88.4
C1	RS12.5C	8.54	91.46	377.0	1.31	1.710	212.8
C2	S12.5D	9.45	90.55	432.0	1.39	1.944	244.7
C3	RS12.5D	10.30	89.70	326.0	2.08	4.341	870.4
C4	S12.5C	5.11	94.89	14.0	1.05	1.109	142.5
C5	S9.5B	9.77	90.23	557.0	0.84	0.701	100.8
C6	S9.5C	8.91	91.09	97.0	1.22	1.477	183.9

As expected, these results show that the test mixes with the higher voids have the greatest permeability. A plot of average voids versus average permeability, shown in Figure 4.3 below, shows good correlation, with a coefficient of determination (R^2) to be 0.87, between voids and permeability. This data also shows that the mixes with the highest permeability and voids have the greatest coefficient of variation. This could possibly mean that the lab permeability test is somewhat unreliable as permeability and voids increase in field specimens. This is also evident by the high coefficients of variation values. Although the coefficient of variation is high for the data presented, it is clear that the permeability of mixes increases by an order of magnitude with increase in air voids beyond 8%.

Mix C5 has the highest average permeability, 557.0 x 10^{-5} cm/s. Mix C3 has the highest air voids, 10.3%, as well as the highest standard deviation, 2.08 (ln based), and coefficient of variation, 870.4%. Mix F2 has the lowest permeability, 11.0 x 10^{-5} cm/s, and mix C4 has the lowest air voids, 5.11%.

Figure 4.4 shows the relation between lift thickness to NMAS versus permeability for the coarse graded mixes. Similar to previous research by the FDOT *(10),* there appears to be a good correlation between these two variables (R^2 = 0.79). This graph also shows that if the ratio of lift thickness/NMAS falls below four, the permeability increases dramatically for coarse graded surface mixes. In Table 4.9, which shows presents the ratio of lift thickness/NMAS for each mix, there are two data points that appear to be equal to about four for the ratio of lift thickness/NMAS. These points represent mixes C1 and C6. The data point for mix C6 is actually at 4.02 for lift thickness/NMAS, with a permeability of 97.0 x 10^{-5} cm/s and the data point for mix C1 is at 3.97 for lift thickness/NMAS, with a permeability of 377.0 x 10^{-5} cm/s. The lower permeability in mix C6 could be explained by the effect of the aggregates used in the mixes. Mix C6 is a S9.5C that uses only 30% of one aggregate that would be considered coarse, the #8p, while mix C1 is an RS12.5C that uses two coarse aggregates, #67's and #78m's, with a combined percentage of 60%. While the lift thickness to NMAS ratio and percent air voids for these mixes are similar, the aggregates used in the mixes seem to be effecting permeability. Perhaps the larger aggregates used in the RS12.5C mix are bridging and creating more interconnected voids in the pavement. This may indicate that the aggregate types used in these mixes have a larger effect on permeability than originally expected.

Figure 4.3 – Semi-log, Average Voids versus Average Permeability

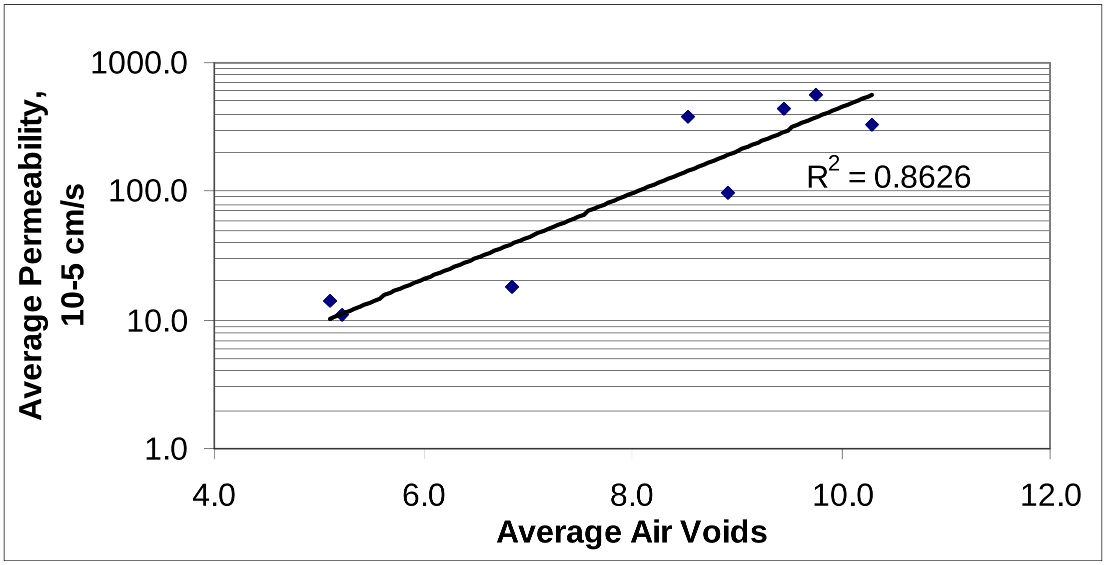


Figure 4.4 – Semi-log, Lift Thickness/NMAS vs. Permeability for the Coarse Mixes

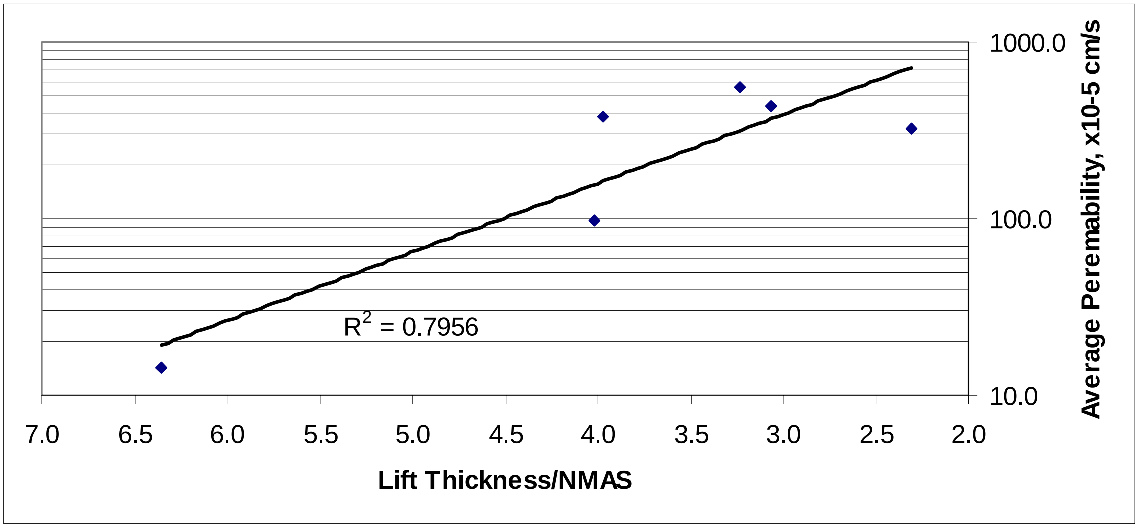


Figure 4.5 shows the results for the fine graded 12.5-mm mixes. Both mixes are impermeable, with *k* values of less than 45 x 10^{-5} cm/s. Neither mix shows any correlation between voids and permeability (F1: R^2 = 0.001; F2: R^2 = 0.32). These results are most likely due to the very low permeability values obtained from the testing. It is apparent that mixes above the restricted zone are much less likely to contain a large amount of interconnected voids, and therefore less likely to be porous.

Figure 4.5 – Semi-log, Voids vs. Permeability, Mixes F1 and F2

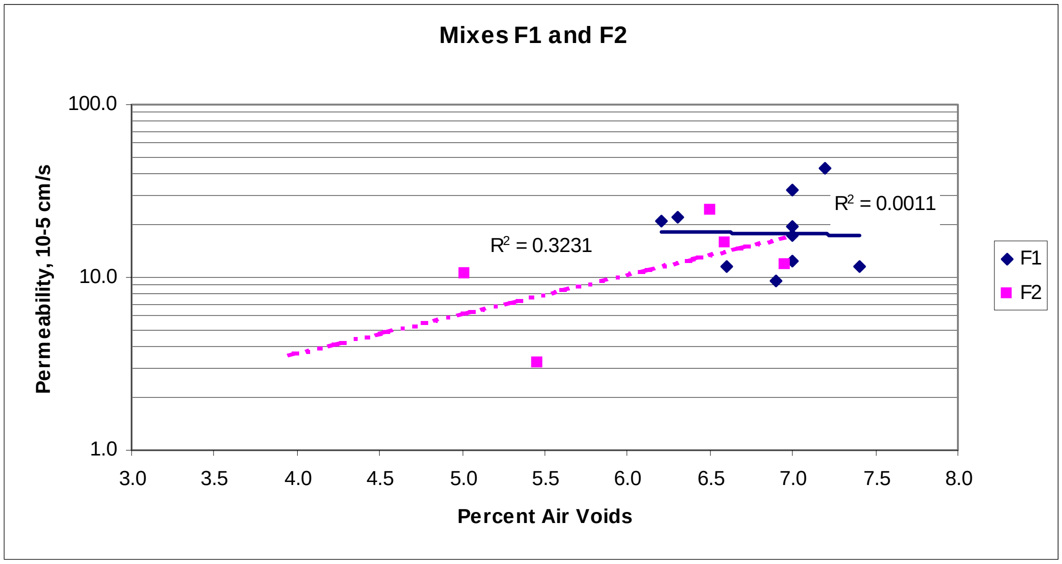
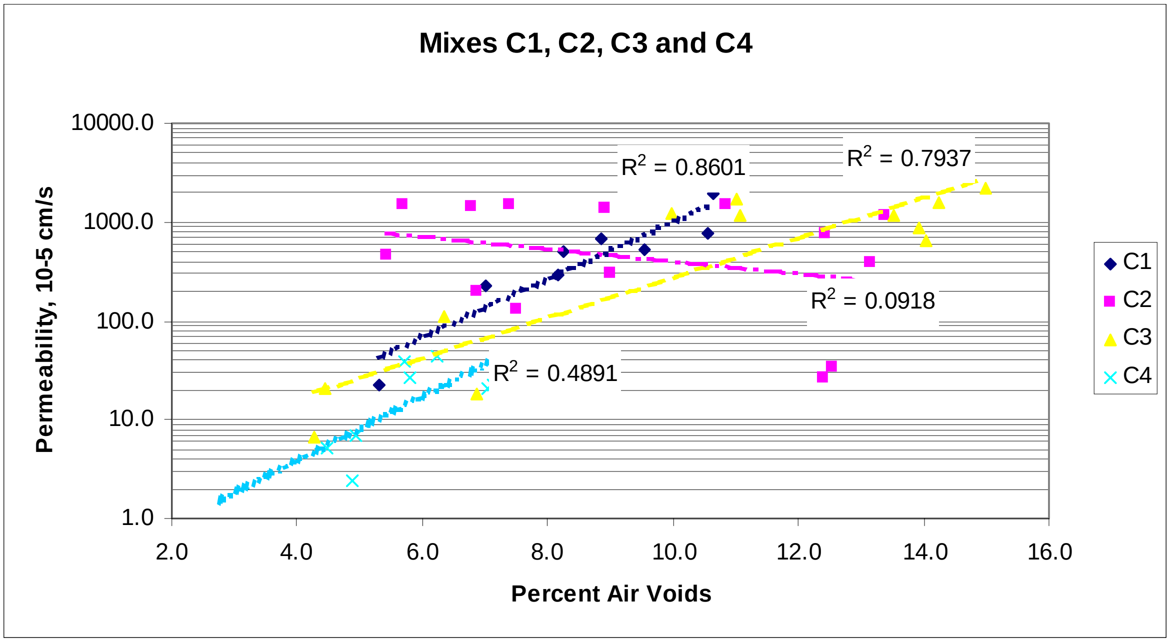


Figure 4.6 shows the individual results for the coarse graded mixes C1, C2, C3 and C4. Mixes C1, C3 and C4 show reasonable correlation between voids and permeability (R^2 = 0.86, 0.61, and 0.70 respectively). Except for mix C2, it appears that at or beyond 8% air voids, the permeability begins to rise very quickly, indicating the increase in interconnected voids, verifying prior research by Cooley et al. *(5)*, Brown, et al. *(7)*, and Zube *(8)*. For mix C2, there seems to be some scatter of the data points and a trend that is unlike what was seen for other mixes. There is an unusual grouping of points between 1400 and 1600 x 10^{-5} cm/s that are manipulating the trend line in a negative sloping direction. These data points are on consecutive specimens and may have resulted from faulty tests due to an air leak, or perhaps a tear in the rubber membrane that surrounds the sample, which would allow more water to pass through the sample.

Figure 4.6 – Semi-log, Voids vs. Permeability, Mixes C1, C2, C3 and C4



Looking at the individual mix permeability measurements, Mix C2 has a few data points that have low permeability and very high voids. This does not follow the trend expected and may indicate that this mix was produced with very poor gradation control causing the mix on the roadway to be extremely variable with some fine areas and some coarse areas, similar to findings by Maupin *(13)*. The void structure in this mix may fall into a category that Zube *(8)* refers to in the following statement, in which "Certain size dimensions of individual voids and the lack of interconnection between the voids could easily produce a pavement of relatively high void content and low permeability". This may have been caused by variability in gradation in mix C2.

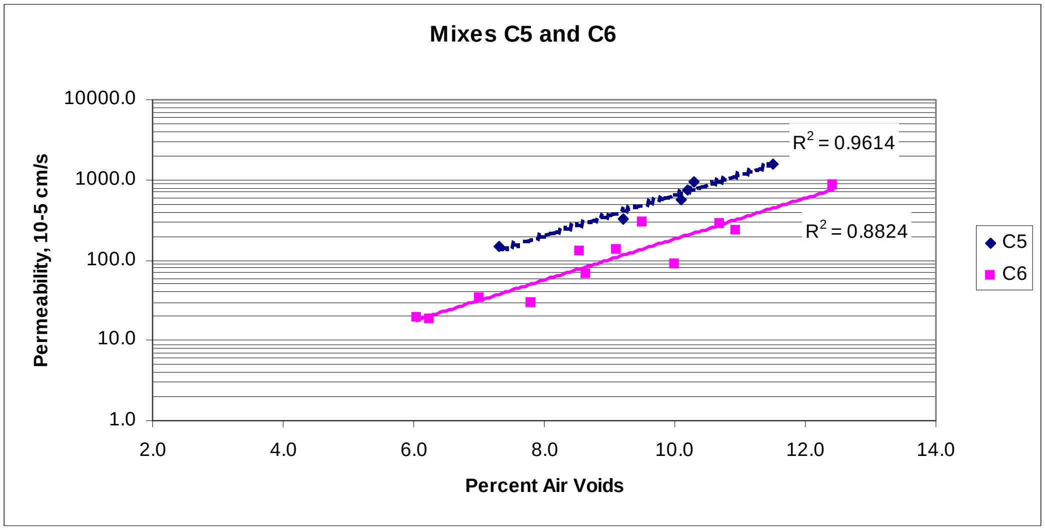
Mixes C3 and C4 show similar results for the coarse graded designs. Both of these trend lines show a reasonable relationship between voids and permeability and have coefficients of determination (R^2) of to 0.62 and 0.70, respectively.

There is one point worth noting about the C4 mix design. This is a coarse graded design that is being tested at the NCAT Test Track Facility in Auburn, AL and when this mix was placed, it had an average compaction of 93.4%. One of the main issues we face with the coarse graded mixes is inadequate density on the roadway. There is speculation that these mixes are too coarse and that compaction is too difficult to achieve the minimum 92%; however, this mix shows that with proper construction practices, it is possible to obtain above the minimum 92% compaction and be practically impermeable. The highest permeability value for mix C4 is approximately 45 x 10^{-5} cm/s, which is extremely low, comparable to impervious clay.

Figure 4.7 shows the results for the 9.5-mm mixes used in the study, C5 and C6. Both show very good correlation between air voids and permeability, with R^2 values of 0.96 for

C5 and 0.88 for C6. Again, once the void level rises above 8%, the permeability increases dramatically.

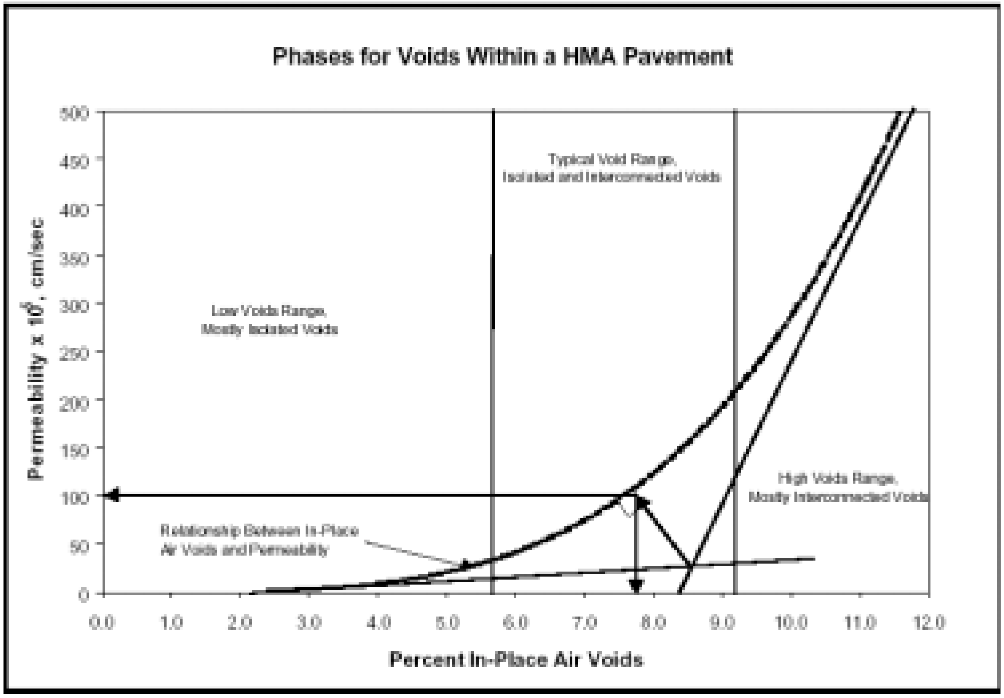
Figure 4.7 – Semi –log. Voids vs. Permeability, Mixes C5 and C6



Looking at the results in Figure 4.7, we see confirmation of the earlier research *(5)* that stated that the 12.5-mm and the 9.5-mm mixes have similar trends in terms of the relationship between voids and permeability. Since this relationship exists, we will use both 9.5 and 12.5-mm mixes to determine a critical value for permeability and voids in surface mixes. All of the figures mentioned above clearly show that a pavement's permeability is greatly influenced by the total air voids within the pavement. Based upon the relationship, for coarse graded 9.5 mm and 12.5 mm designs, permeability is very low at in-place air voids below 5 percent. From 5 to 7 percent air voids, the permeability begins to increase at a greater rate with changes in in-place voids. At voids above 8 percent, small changes in density result in large increases in permeability. This is very similar to the results presented by Choubane et al. *(3)*.

Using a procedure presented by Cooley, et al. *(5)*, a regression line is drawn for all of the results from the coarse mix designs. Figure 4.6 shows a typical figure when permeability is plotted versus voids. The regression line that is drawn shows that at low voids, pavements are virtually impermeable, and at high voids, there is a large percentage of interconnected voids, so permeability can be a problem.

Figure 4.8 – Typical Permeability vs. Voids Plot, *(5)*



The middle range shown in Figure 4.8 presents an area where the voids are isolated in the pavement and thus very few interconnected voids exist. By finding the point on the regression line where the ratio between the low void range and the high void range is 1, we can approximate a critical value for in-place air voids and permeability. Figure 4.9 shows the all of the data points for the coarse graded mixes used for this study. Figure 4.10 shows the procedure for finding the critical values for permeability and air voids.

Figure 4.9 – Plot of Coarse Graded Mixes

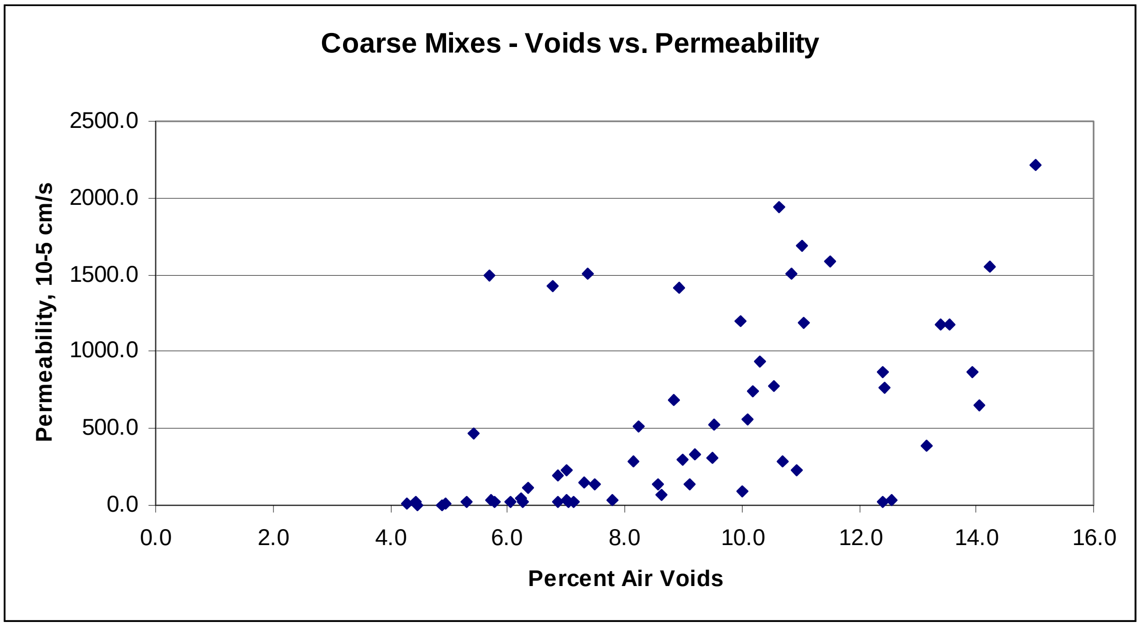
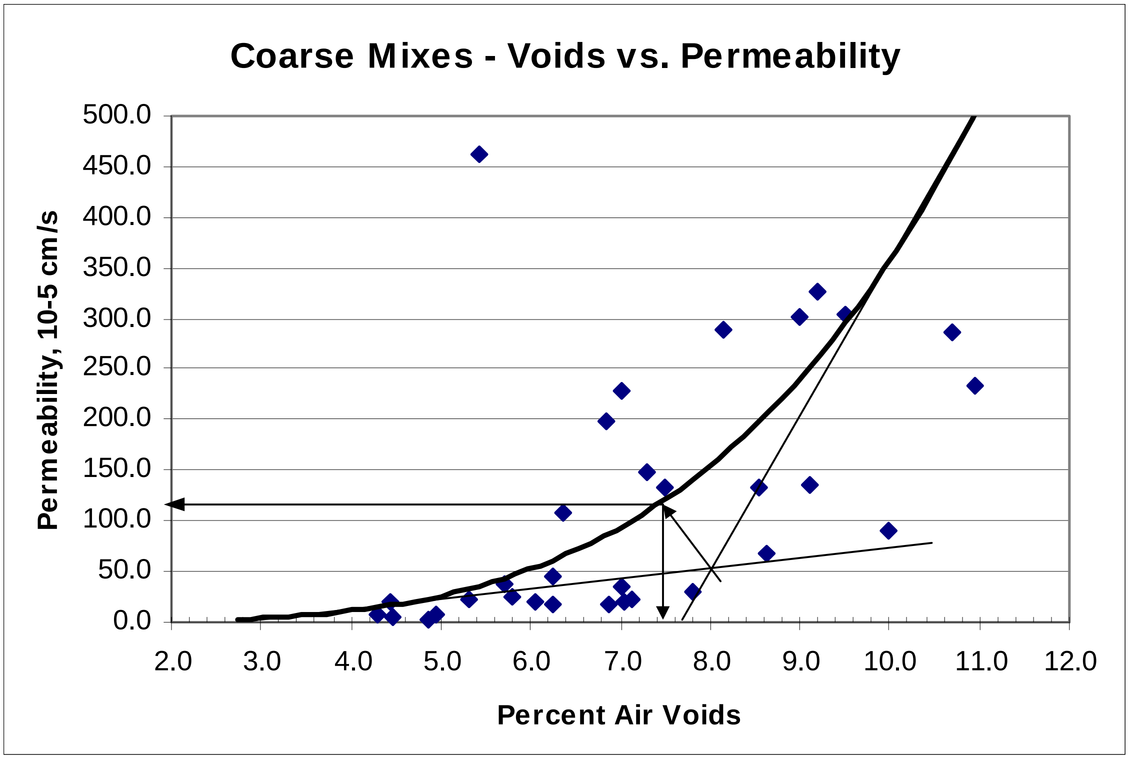


Figure 4.10 – Critical Value Determination for Voids and Permeability



Based on Figure 4.9, the bisecting line occurs at approximately 7.5% air voids and 125 x 10^{-5} cm/s permeability. Taking into account the amount of scatter in the data, this indicates that in-situ, 12.5-mm and 9.5-mm coarse graded mixes should have a minimum density between 92.0% and 93.0% of maximum theoretical density (G_{mm}), and a critical coefficient of permeability value between 100 x 10^{-5} cm/s and 150 x 10^{-5} cm/s. These results match closely to prior research on field permeability studies *(3, 5)*. Comparing this data to the current NCDOT density specification of 92%, the suggested range seems to fall above the current minimum. There is no current NCDOT specification addressing permeability.

It has been shown that at 8% air voids the coefficient of permeability, *k*, for HMA pavement increases dramatically, so in order to reduce the amount of porous sections in the roadway, we must strive for better compaction, meaning perhaps, >92.0% of theoretical maximum density. Also, if a lab permeability test is included in future specifications, the maximum limit allowable for the coefficient of permeability should be approximately 125 x 10^{-5} cm/s. For coarse graded mixes there also seems to be a relation between lift thickness and the amount of voids present on the roadway. Although the relation between permeability and the ratio of lift/NMAS was weak, it is apparent that as this ratio fell below four, the permeability increased dramatically (Figure 4.1). From these results, it is recommended that the lift thickness for coarse graded Superpave surface mixes be increased from three to four times the NMAS to aid in compaction of these mixes. The fine graded mixes appear to work well when placed at three times the NMAS.

CHAPTER 5 – LAB SPECIMENS SAMPLING AND TESTING

After reviewing the results from the field specimens, a more thorough analysis of these mixes was felt to be necessary. Of the eight mixes chosen for this study, four (two fine grade and two coarse graded) were chosen for further lab testing. Mixes C2 and C5 were chosen since they were the most permeable (field) coarse graded mixes. Since using fine graded mixes seems to be the easiest way to eliminate permeable pavements, it seems logical to compare the fine mixes F1 and F2 to the coarse mixes, C2 and C5, to see if there is a difference in performance between coarse and fine graded mixes.

For each mix, 12 specimens, 75 mm in height were compacted using the gyratory compactor and tested for percent voids and permeability, and then were subject to rut testing in the Asphalt Pavement Analyzer under dry and wet conditioning. Next, one specimen was gyrated to 125 mm and then sawed in half and trimmed to make two-50 mm specimens for testing in the Superpave Shear Tester (RSCH). All specimens were gyrated at or close to average in-place voids (target field air voids). The lab specimen results are presented below in Tables 5.1 through 5.4.

5.1 VOIDS AND PERMEABILITY

For each mix, 12 lab specimens were mixed and compacted using the raw materials specified in each mix design. After the mix was compacted and cooled, bulk specific gravities (G_{mb}) were measured using AASHTO T166. The maximum theoretical specific gravity (G_{mm}) was taken from the original job mix formula (JMF) and used to calculate voids. The results for voids and permeability for the lab specimens are shown below in Tables 5.1 through 5.4. The control for the lab specimens was very tight, as shown by the very low standard deviations of the air voids. Also, the lab specimen densities were very close to the in-place densities, or target densities. The lab specimens were tested for permeability in the same manner as the field specimens.

Table 5.1 – Lab Specimen Properties, Mix C2 (Target voids – 9.5%)

Sample Description	*Lab #*	*Percent Air Voids*	*Permeability, 10^{-5} cm/s*
C2W	1	9.5	128.0
C2W	2	9.6	1344.0
C2W	7	9.8	979.0
C2W	8	9.6	822.0
C2D	3	8.6	1198.0
C2D	4	9.9	1042.0
C2D	5	10.2	1220.0
C2D	6	10.0	775.0

For mix C2, only 8 specimens were used instead of the original 12 because four of the specimens fell apart and lack of materials and time constraints would not allow remaking these specimens, so they were eliminated.

Table 5.2 – Lab Specimen Properties, Mix C5, (Target voids – 9.8%)

Sample Description	*Lab #*	*Percent Air Voids*	*Permeability, 10^{-5} cm/s*
C5W	1	9.2	128.0
C5W	2	9.2	1210
C5W	3	9.2	157.0
C5W	4	9.2	147.0
C5W	5	8.9	130.0
C5W	6	8.9	200.0
C5D	1	10.3	405.0
C5D	2	10.4	318.0
C5D	3	10.7	265.0
C5D	4	10.6	293.0
C5D	5	10.6	280.0
C5D	6	10.7	314.0

Table 5.3 – Lab Specimen Properties, Mix F1, (Target voids – 6.9%)

Sample Description	*Lab #*	*Percent Air Voids*	*Permeability, 10^{-5} cm/s*
F1W	1	7.3	93.0
F1W	2	7.0	30.0
F1W	3	7.4	43.0
F1W	4	6.9	31.0
F1W	5	7.0	34.0
F1W	6	6.6	72.0
F1D	1	7.3	0.0
F1D	2	7.0	0.0
F1D	3	6.8	0.0
F1D	4	6.8	32.0
F1D	5	6.5	0.0
F1D	6	7.0	0.0

Table 5.4 – Lab Specimen Properties, Mix F2, (Target voids – 5.2%)

Sample Description	*Lab #*	*Percent Air Voids*	*Permeability, 10^{-5} cm/s*
F2W	1	5.0	0.0
F2W	2	5.2	0.0
F2W	3	4.9	0.0
F2W	4	4.9	0.0
F2W	5	5.1	0.0
F2W	6	5.3	0.0
F2D	1	5.2	0.0
F2D	2	5.3	0.0
F2D	3	5.2	0.0
F2D	4	5.2	0.0
F2D	5	5.0	0.0
F2D	6	5.0	0.0

The average voids and log transformed average permeability's are shown in Tables 5.5 and 5.6. All of the lab specimens were gyrated to 75 mm, so lift thickness remained constant for all mixes.

Table 5.5 – Average Lab Specimen Properties

Test Mix	*Mix Type*	*Average Gmb*	*Gmm*	*Average % Air Voids*	*Air Voids Std. Dev.*	*Target % Air Voids*
F1	RS 12.5C	2.298	2.464	6.73	0.27	6.9
F2	S12.5C	2.365	2.492	5.10	0.14	5.2
C2	S12.5D	2.259	2.500	9.64	0.48	9.5
C5	S9.5B	2.215	2.456	9.81	0.77	9.8

Table 5.6 – Average Permeability Results for Lab Specimens (ln based)

Test Mix	*Mix Type*	*Average k, 10^{-5} cm/s*	*Standard Deviation (ln based)*	*Variance (ln based)*	*Coefficient of Variation (ln based)*
F1	RS12.5C	43.0	0.456	0.208	48.076
F2	S12.5C	Impermeable	-	-	-
C2	S12.5D	1061.0	0.204	0.042	20.614
C5	S9.5B	212.0	0.427	0.182	44.722

It is evident from Table 5.6 that the fine graded lab specimens, similar to the field specimens, are impermeable according to the lab permeameter, while the coarse graded specimens follow the similar trend as the field specimens and are variable. For mix F1, five of the 12 specimens were practically impermeable to water and for mix F2, all 12 specimens failed to allow water to penetrate after 30 minutes of applied head. During the lab permeability test, if after 30 minutes there is no change in the water level, the specimen is considered impermeable. Similar to the field specimen analysis, the coarse graded and fine graded designs have different characteristics when evaluated for permeability in the lab.

Comparing the coefficients of variation for the field and lab specimens it is clear that the variability in the lab specimens is much less than the field cores. This would be expected since the lab is a controlled environment and repeatability in specimen preparation with the gyratory compactor is fairly standard. Unfortunately, the lab specimen permeability results were not accurate in predicting the field permeability, since mix C5 in the field had the highest permeability while mix C2 had the highest lab permeability. The results do show that the fine graded mixes, both lab and field, are less permeable than the coarse graded mixtures, as expected. In both instances, mix F2 had the lowest permeability, followed by F1. Although the lab permeability results were unable to predict the exact field performance for the mixes, those mixes that were impermeable in the field were impermeable in the lab and vice versa. If we use the recommended limit for permeability of 125 x 10^{-5} cm/s from Chapter 4 as a reference, the lab tests would eliminate the two coarse graded mixes, which also performed poorly when the field cores were tested, and accept the fine graded mixes. This would indicate that for predicting field permeability, it may be possible to use laboratory specimens compacted to in-place voids to determine if there will be a permeable pavement in the field. If the current minimum acceptable density, 92% of G_{mm} (or 8% voids) is considered the worst case, then lab specimens should be compacted to 8% voids, at a suitable height, and tested for permeability.

5.1.1 FINE GRADED MIXES F1 AND F2

Since Mix F2 had no permeability measurement, a comparison analysis of this mix is difficult, but we can look at the gradations for any clues as to why this mix was so impermeable to water. For mix F1, more coarse aggregates were used (fraction retained on the #8 sieve) compared to F2 and therefore the gradation for F1 was a bit coarser, with 57% (Table 1.1) retained on the 2.36-mm sieve (No. 8) compared to 48% retained on the 2.36-mm for F2. The maximum density line (MDL) separates coarse graded designs from fine graded designs and crosses the 2.36-mm sieve at 40%, meaning the F1 mix is borderline coarse. These results may make it possible to determine a specification range on the 2.36-mm sieve that will enable mixes to be impermeable. Analysis of the coarse graded designs may help narrow this range down. Table 5.7 shows the comparison between mixes F1 and F2. Permeability tests on field specimens for mix F1 resulted in 18.0 x 10^{-5} cm/s permeability versus the lab specimens permeability results of 41.0 x 10^{-5} cm/s. Field results for F2 were 11.0 x 10^{-5} cm/s, and the lab specimens were impermeable. These results are comparable and it appears that the lab permeameter is capable of predicting permeability, or lack thereof, in fine graded mix designs. If we accept the recommended limit from Chapter 4 of 125 x 10^{-5} cm/s, all of the fine graded specimens have acceptable permeability.

If it is assumed that the gradation in the lab matches the gradation in the field, and all other mix properties are similar, the differences in the lab and field permeability measurements must be due to the type of compaction used on the specimens. It is clear that lab compaction by the gyratory compactor differs from field compaction which is accomplished using heavy steel wheel and pneumatic rollers. This is the key difference when trying to predict field performance using laboratory specimens.

Table 5.7 – Comparison of Field vs. Lab specimens, Mixes F1 and F2

Test Mix	***F1***	***F2***
Mix Type	RS12.5C	S12.5C
Average % Air Voids, Field	6.85	5.23
Average % Air Voids, Lab	6.73	5.10
Average k, 10^{-5} cm/s, Field	18.0	11.0
Average k, 10^{-5} cm/s, Lab	43.0	impermeable

5.1.2 COARSE GRADED MIXES C2 AND C5

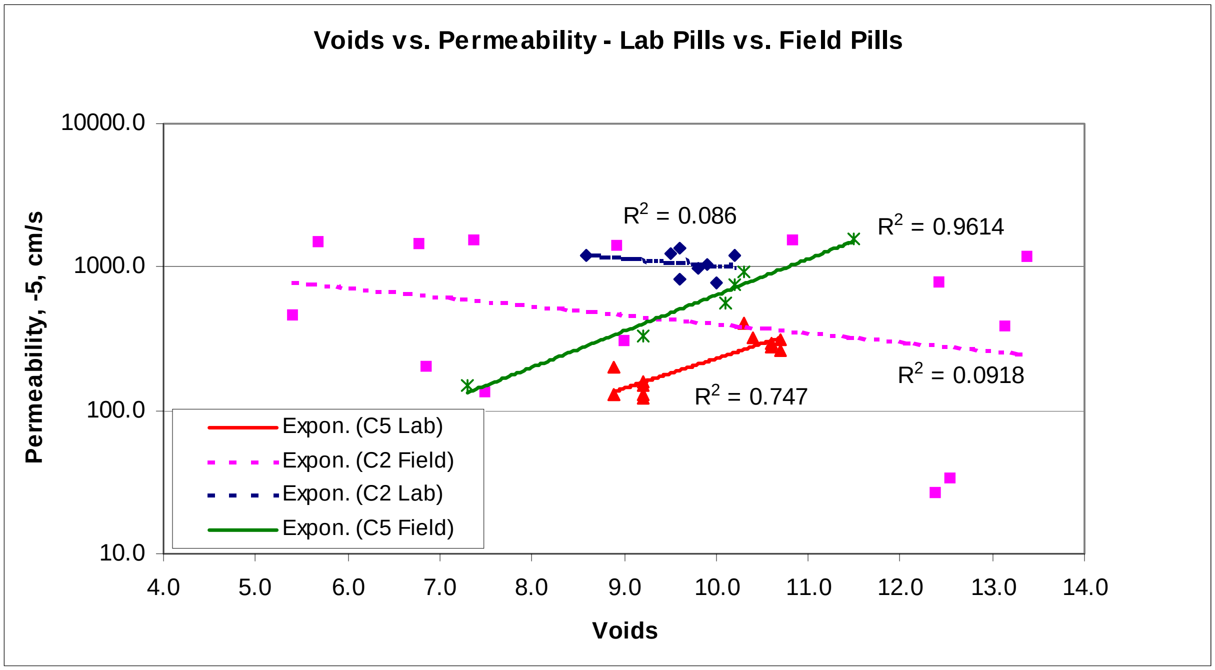
For analysis, this study has assumed that S12.5 mm and S9.5 mm NMAS surface mixes have similar characteristics in terms of permeability versus voids: however, mixes C2 and C5 show this may not be the case. Previous research *(9)* has stated that nominal maximum aggregate size (NMAS) affects the permeability characteristics of HMA mixes. This is evident with mixes C2 and C5, which are S12.5D and S9.5B surface mixes, respectively. Both mixes have a similar gradation even though they have two different NMAS. For example, the percent passing the 2.36-mm sieve is set at 39% for mix C2 and 43% for C5, respectively. The rest of the gradations are very close, except that C2 has a few percent more of 9.5-mm and 12.5-mm material, which is contributed by the #67 size aggregate being used in the 12.5-mm mix. We can see from the results in Table 5.8 that the voids for the lab specimens for both mixes are very close, but the permeability varies greatly. The differences between the field and lab results again point to the compaction type used mentioned above in Section 5.1.1. The cause of the high permeability in the C2, S12.5-mm lab specimens may be the extra 9.5-mm and 12.5-mm coarse material "bridging" within the specimens and generating larger interconnected voids. Mix C2 also has a high coefficient of variation, which again shows that the lab permeameter is very unreliable when mixes have very high permeability values. There does not appear to be a relation between the lab specimens and field specimens for the coarse graded mixes. However, the lab results do show that the coarse graded mixes do have a *k* value of greater than 125 x 10^{-5} cm/s, which should be considered the maximum.

A conclusion that can be drawn from the coarse mixes permeability results is that air voids are the main controlling factor affecting permeability, and that coarse graded designs seem to have higher air voids in the field. Figure 5.1 below shows the lab permeability test results from the field cores versus the lab specimens. Both coarse mixes have similar permeability characteristics in the lab and the field. For some reason, mix C2 shows a reverse trend to what we would expect and this trend is consistent between both lab and field specimens. This trend in the field specimens may be caused by the extreme variability in the specimens. As for the lab specimens, the small number of data points most likely causes the reverse trend.

Table 5.8 – Comparison of Field vs. Lab Specimens, Mixes C2 and C5

Test Mix	***C2***	***C5***
Mix Type	S12.5D	S9.5B
Average % Air Voids, Field	9.45	9.77
Average % Air Voids, Lab	9.64	9.81
Average k, 10^{-5} cm/s, Field	432.0	557.0
Average k, 10^{-5} cm/s, Lab	1061.0	212.0

Figure 5.1 – Field vs. Lab Specimens – Permeability



For mix C5 we see a good correlation between field and lab specimens, with R^2 values that are similar. The field specimens in both cases seem to have more interconnected voids that are causing greater permeability values. This may be due to the differences in compaction between the gyratory compactor and field compaction, which was shown in a study on internal structure by Tashman, et al. *(15)*. Tashman, et al. stated that air voids in different gyratory compacted specimens will be less uniformly distributed and that field compacted specimens will have a similar air void distribution throughout.

5.2 RUT TEST RESULTS – LAB SPECIMENS

Laboratory rut tests were performed using the APA device under wet and dry conditions. Each test mix was broken up into two sample sets, one to be run under wet conditions, and one to be run under dry conditions. The wet conditioned specimens were placed in a 60°C water bath for 24 hours, as per AASHTO T283, and then placed in the specimen molds and tested under water in the APA. The dry specimens were conditioned in the APA chamber at 64°C for 24 hours and tested under dry conditions. The results show a normal distribution, as shown in Figure 5.2, with only 2 points being considered outlying. These points were eliminated and the rest of the results were evaluated.

Figure 5.2 – Probability Plot for Lab Rut Tests

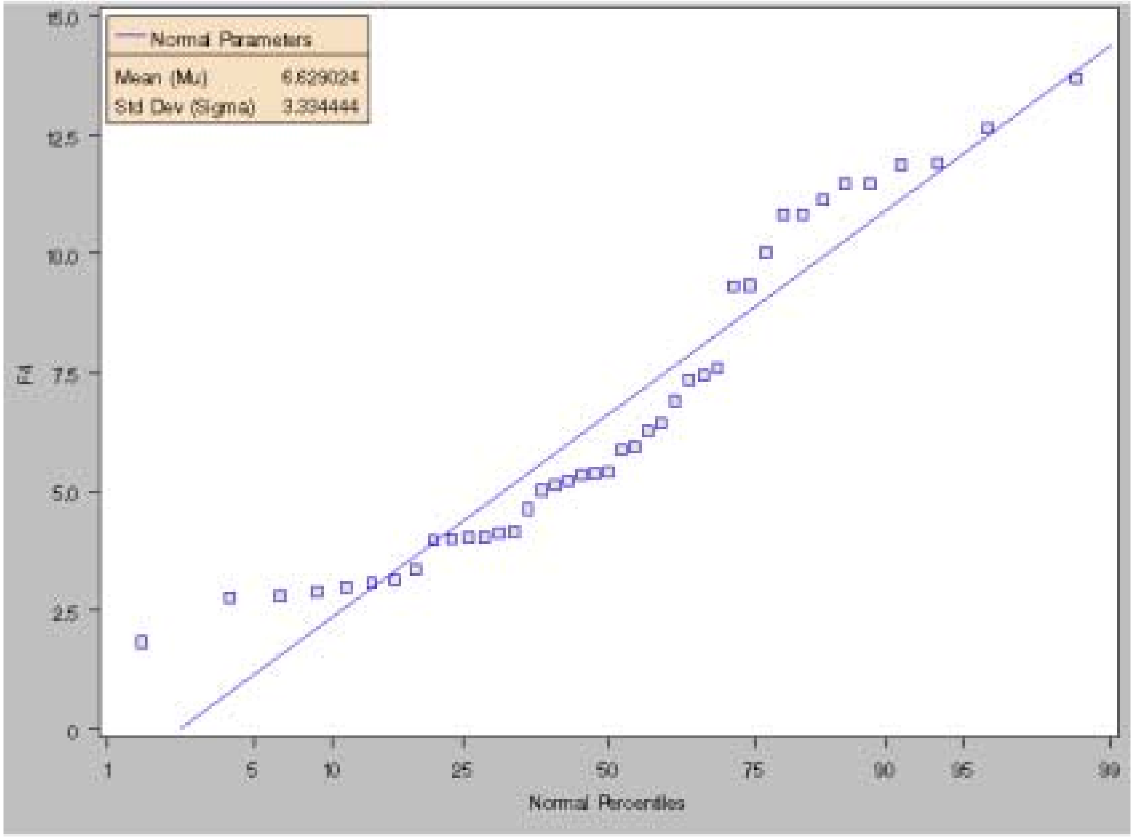


Table 5.9 provides the average rut depths for the wet and dry specimens for each test mix. Mix C5 performed the worst and mix C2 was the best performing mix under the wet conditions and mix F1 was the best performing mix in dry conditions. A surprising outcome of the results was that the wet specimens performed better than the dry specimens in all cases except for mix F1. Table 5.9 shows that, statistically speaking, there is no difference between the dry and wet specimens for mixes F1 and C2 and the water conditioning most effected mixes F2 and C5. The water-conditioned specimens may have performed better due to the pore water pressure that builds up in the sample voids. Since the specimens are confined, there is no way for the water to be released, and this pore pressure resists the wheel loads and reduces overall rutting of the specimens. Perhaps the coarse specimens with more air voids

had better resistance since the volume of water inside the specimens was greater than inside the fine specimens. On the roadway, there is an unlimited amount of area for water to flow, so pore water pressure will not affect the mix on the roadway. In the lab, there must be a way to try to eliminate this pore water pressure so that an accurate assessment of field permeability can be performed. Perhaps water conditioning the specimens and then running them under dry conditions could better simulate the in-place conditions.

There does not appear to be a correlation between the permeability and rut resistance for these mixes. Combining these results with the SST results may help in understanding the performance characteristics of these mixes.

Table 5.9 – Average Rut Depth Results, Lab Specimens, APA

Test Mix	*Mix Type*	*Average Rut Depth, Wet, mm*	*Standard Deviation, Wet*	*Average Rut Depth, Dry, mm*	*Standard Deviation, Dry*
F1	RS12.5C	4.5	0.934	3.9	1.541
F2	S12.5C	5.1	0.793	6.7	0.919
C2	S12.5D	3.6	0.498	4.8	1.320
C5	S9.5B	10.4	0.927	12.1	0.995

5.3 SST RESULTS

Specimens for the four test mixes were subjected to the RSCH test at a chamber temperature of 60° C. Two specimens were tested for each mix, for a total of 8 unconditioned specimens. Figure 5.3 below shows the average results of the RSCH test. Each mix was subjected to 10,000 cycles of applied load and a shear plastic strain of 0.5 % was considered failure. Mix C5 by far performed the worst, with failure occurring at only about 8000 cycles, and F2 was the second worst. F1 performed the best and C2 averaged about the same as F2. Figure 5.4 shows the results of both the APA and SST tests. There is a very good correlation between these two tests ($R^2 = 0.97$). This supports earlier research performed by Zhang, et al. *(14)* and Kandhal, et al. *(11)*, where a moderate correlation was found between the RSCH test and the APA test ($R^2 = 0.68$, and 0.53, respectively) for surface mixes.

Figure 5.3 – RSCH results

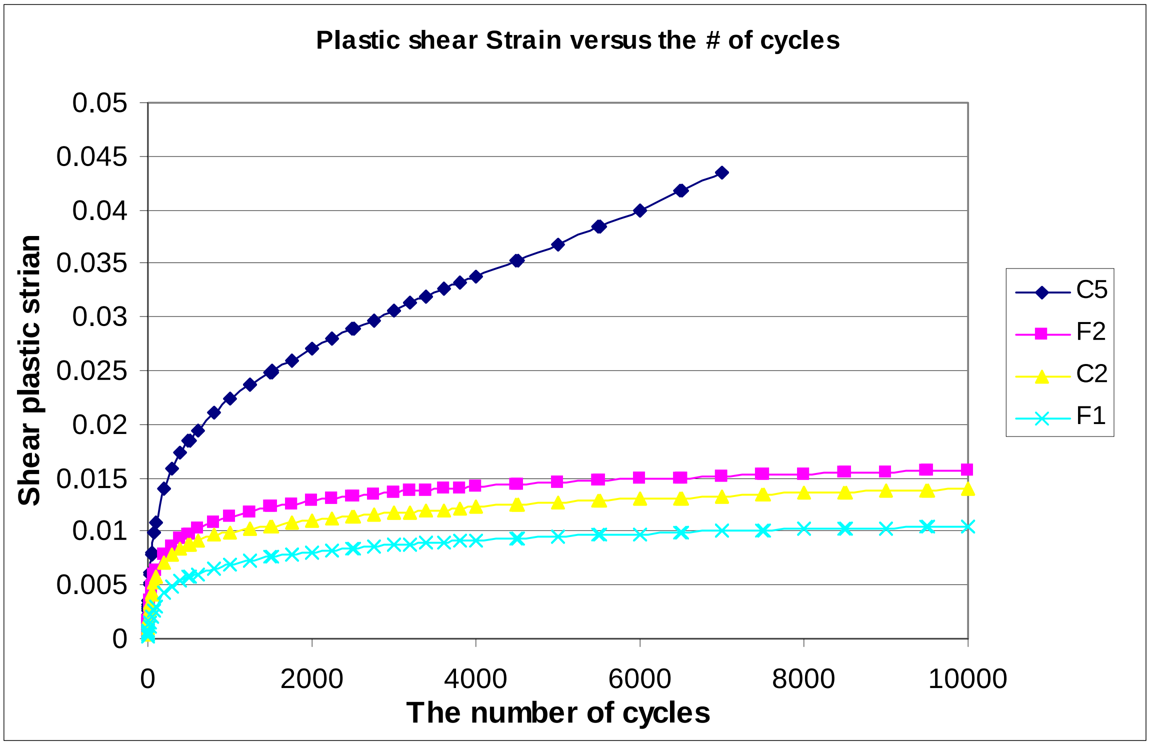
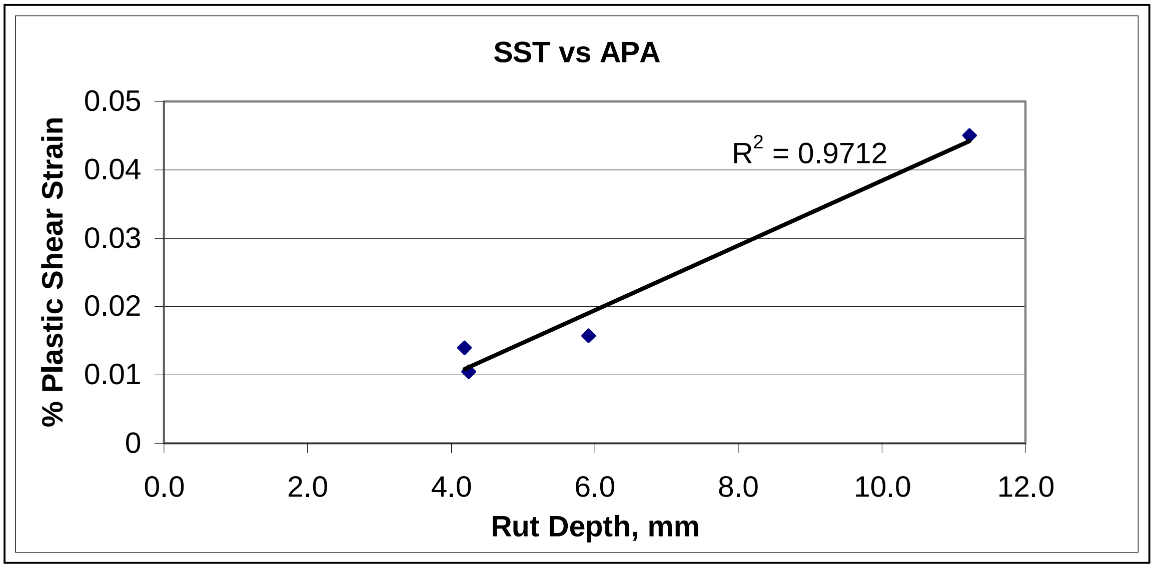


Figure 5.4 – APA versus SST Test Result Comparison



A comparison of each of the four test mixes shows that mix C5 (S9.5B) performed the worst in all aspects. If we use a performance scale from very good to poor, we see from Table 5.10 that C5 performed poor on all except one parameter, lab permeability. Mix F1 (RS12.5C) performed the best.

Table 5.10 – Evaluation of Test Mixes

Parameter	C2	C5	F1	F2
Voids (field)	Satisfactory	Poor	Good	V. Good
Voids (lab)	Satisfactory	Poor	Good	V. Good
Perm (field)	Satisfactory	Poor	Good	V. Good
Perm (lab)	Poor	Satisfactory	Good	V. Good
Rut Depth	V. Good	Poor	Good	Satisfactory
SST	Good	Poor	V. Good	Satisfactory

Rut and SST test specimens were compacted in the Gyratory Compactor

The air void evaluations in Table 5.10 are based on a maximum of 8% acceptable voids. This simple summary shows that according to laboratory testing to predict performance, fine graded surface mixes can perform just as well as the coarse graded surface mixes and are virtually impermeable. The effect of voids certainly has a bearing on these results, and a future study may be required with specimens blended at the same void levels; but, in the field, it is difficult to achieve the proper percent compaction on coarse graded mixes, so this study accurately reflects field mixes. Table 5.10 shows the relationship between voids and permeability, which is expected.

The Superpave system focuses on the use of more coarse aggregate in HMA mixtures because it is believed that the addition of this material will create mixes that are more resistant to permanent deformation, certainly more resistant than the fine graded mixtures that have been used in the past. But, if we assume the performance tests used in this study, the APA and SST, are valid in predicting roadway performance, then the above results prove that fine graded mixes can perform just as well as coarse graded mixes.

This chapter presents the results of the laboratory specimens in order to try and predict field performance of Superpave surface mixes. It can be observed from these results that the fine graded mixes are less permeable, but perform a bit worse in the performance tests. The coarse graded mixes are permeable and if they achieve the proper density, will perform better in the performance tests, as mix C2 shows. Table 5.10 presents a very simple analysis of the four mixes tested and shows that mix F1, a fine graded RS12.5C mix, performed the best, and mix C5, a coarse graded S9.5B mix performed the worst.

CHAPTER 6 - CONCLUSIONS AND RECOMMENDATIONS

The objectives of this study were to identify the causes of permeability variation in Superpave surface mixes, recommend a critical limit for permeability and density for surface mixes, and recommend any other changes in the current NCDOT specifications that may help reduce permeability in pavements.

It appears that the in place density, lift thickness, specimen type (field or lab) and NMAS all have an affect on the permeability characteristics of Superpave surface mixes. By comparing field and laboratory specimens it is clear that the different compaction types affect the void structure in the specimens, with field specimens that use rolling compactors containing more interconnected voids. It has also been shown that results from lab permeability test used in this study have large variability when testing specimens with high voids, as in the case with many field cores. This issue may indicate that field specimens should not be used to determine if a mixture is permeable, but rather lab specimens, compacted at a specific void content, should be used as will be discussed below.

The results of this research have shown that, at in place air voids above 8%, permeability tends to rise dramatically in the field. To try and decrease the chance of in place voids above 8%, the current density specifications may need to be changed. Using the procedure described in Chapter 4 for determining critical density and permeability, the following recommendations are made:

- An increase in field compaction from the current specification of 92.0% of G_{mm} for Superpave surface mix designs that have a coarse graded (BRZ)blend gradation;
- If a coarse design is requested, a lab permeability test should be performed on lab specimens compacted to between 6% and 8% air voids. These tests should be performed

using the lab permeameter and follow the NCDOT recommended permeability test Method A-100, and have a maximum coefficient of permeability of approximately 125.0 x 10^{-5} cm/s.

- A North Carolina Standard Test Method A – 100 is proposed for this permeability test. This test method is included in Appendix B.

Due to the large amount of scatter in the field core permeability data, the above conclusions are only tentative recommendations and need to be refined by further research.

For coarse graded mixes there also seems to be a relation between lift thickness and the amount of voids present on the roadway. Although the correlation between permeability and the ratio of lift/NMAS was weak ($R^2 = 0.53$), it is apparent that as this ratio fell below four, the permeability increased dramatically. From these results, it is recommended that:

- The lift thickness for coarse graded Superpave surface mixes be increased from three to four times the NMAS to aid in compaction of these mixes. The fine graded mixes appear to work well when placed at three times the NMAS, so this specification should remain intact for fine graded mixes.

It can be observed from the results that the fine graded mixes are less permeable, and perform just as well (in the performance tests) when compared to the coarse graded mixes. It was originally thought when Superpave began that coarse graded surface mixes would perform better than fine graded mixtures in terms of rutting, but apparently this is not the case, according to the SST and APA tests. Based on the results presented herein, following the current trend of designing fine graded mixes is a good decision and these fine mixes will work well in the field.

Future research utilizing a field permeameter device may be necessary to better evaluate the lab permeability tester. Also, the Asphalt Pavement Analyzer’s ability to test submerged samples should be evaluated further for prediction of field performance.

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APPENDIX A

Mix F1

REVISED 6-12-97

M&T FORM 601 (SP)

North Carolina Department Of Transportation
Materials and Tests Unit
Raleigh, North Carolina

REPORT ON SUPERPAVE MIX DESIGN
M&T Verification

MD# ____________

DATE SUBMITTED:				DATE APPROVED:		
PROJECT NO.:	misc			ASPHALT:	Citgo Wilmington	PG 70-22
COUNTY:	Johnston			ADDITIVE:	ARR-MAZ Ad-Here LOF 6500	(.25 %)
CONTRACTOR:	ST Wooten			Hanson	Neverson	# 67 Stone
PLANT:	Sims			Hanson	Neverson	78-M Stone
DESIGNED BY:	C. Croom			Hanson	Neverson	Dry Screenings
SPECIFICATION:	12.5 mm	Surface Mix		ST Wooten	Daniels	Natural Sand
GYRATIONS:	8/100/160	150 mm	144 °C			
TRAFFIC LEVEL:	C	Million ESALs		ST Wooten	Plant	Baghouse Fines
AC SPECIFIC GRAVITY:		1.039		ST Wooten	Stockpile	RAP

GRADATION OF MATERIALS USED

MATERIAL	# 67	78-M	D. Scrg.	Sand		Bag. Fines	RAP	BLEND	CONTROL
PERCENT	15.0	39.0	15.0	15.0		1.0	15.0		POINTS
50.0 mm	100.0	100.0	100.0	100.0		100.0	100.0	100.0	
37.5 mm	100.0	100.0	100.0	100.0		100.0	100.0	100.0	
25.0 mm	100.0	100.0	100.0	100.0		100.0	100.0	100.0	
19.0 mm	95.0	100.0	100.0	100.0		100.0	97.0	98.8	100.0
12.5 mm	53.0	100.0	100.0	100.0		100.0	93.0	91.9	90.0 - 100.0
9.50 mm	33.0	90.0	100.0	99.0		100.0	89.0	84.3	< 90.0
4.75 mm	7.0	30.0	95.0	96.0		100.0	75.0	53.7	
2.36 mm	3.0	8.0	77.0	81.0		100.0	62.0	37.6	28.0 - 58.0
1.18 mm	2.0	3.0	56.0	78.0		100.0	50.0	30.1	<25.6, >31.6
0.600 mm	2.0	3.0	39.0	47.0		100.0	37.0	20.9	<19.1, >23.1
0.300 mm	1.0	2.0	24.0	32.0		100.0	24.0	13.9	
0.150 mm	1.0	1.0	14.0	6.0		96.0	13.0	6.5	
0.075 mm	0.7	0.7	7.8	5.0		94.0	8.1	4.5	2.0 - 10.0
Elutriation Loss				5.0				0.8	
Agg. Bulk Dry S.G.	2.630	2.610	2.560	2.601		2.660	2.620	2.606	
Agg. Apparent S.G.	2.660	2.650	2.620	2.621		2.699	2.650	2.643	
						Agg. Effective S.G.:		2.616	

FORMULATIONS at N design

		[TRIAL]						
% Asphalt-Total Mix		3.5	4.0	4.5	5.0	5.5	% RAP / % Virgin:	15/85
Lab Bulk Specific Gravity		2.653	2.326	2.341	2.373	2.394	% AC in RAP:	4.7
Max. Specific Gravity (Rice)		2.771	2.467	2.449	2.432	2.415	% AC from RAP:	0.7
% Voids-Total Mix (VTM)		4.3	5.7	4.4	2.4	0.9	% AC Absorption:	0.2
% Solids-Total Mix		95.7	94.3	95.6	97.6	99.1	% ASH:	
% Effective AC Content		3.3	3.8	4.3	4.9	5.4	TSR % Retained :	
Dust to AC Ratio		1.33	1.16	1.02	0.92	0.83	Ignition Furn. Calibr.:	
% By Volume of Effective AC		8.5	8.6	9.8	11.1	12.3	% AC (Design):	
% Solids by Vol. of Agg. Only		87.2	85.7	85.8	86.5	86.8	Rice Specific Gravity:	
% Voids in Mineral Agg. (VMA)		1.8	14.3	14.2	13.5	13.2	Lab Specific Gravity:	
% Voids w/AC (VFA)		486.2	60.2	69.0	82.1	93.5	Percent Air Voids:	
% Gmm @ N initial	8	86.5	87.6	88.6	90.4	91.7	Percent VMA:	
% Gmm @ N design	100	95.8	94.4	95.6	97.7	99.2	Percent VFA:	
% Gmm @ N maximum	160	97.1	0.0	0.0	0.0	0.0	DUST/AC Ratio:	

COMMENTS:			% Gmm @ N initial	
	Sand Equivalent:	84.0	% Gmm @ N max.	
DESIGNED BY:	C. Agg. Angularity:	/	% AC TOTAL:	4.6
	F. Agg. Angularity:	50.0	% AC from RAP:	0.7
APPROVAL:	Flat & Elongated:	1.1	% AC ADDED:	3.9

Mix F2

REVISED 6-12-97

North Carolina Department Of Transportation

M&T FORM 601 (SP)

Materials and Tests Unit

Raleigh, North Carolina

REPORT ON SUPERPAVE MIX DESIGN

M&T Verification

MD# ____________

DATE SUBMITTED:				DATE APPROVED:		
PROJECT NO.:	Test Track			ASPHALT:	Ergon Mississippi	PG 67-22
COUNTY:				ADDITIVE:	Morton Inter. Morelife2200	(.75 %)
CONTRACTOR:	NCDOT			Martin Marietta	Jamestown	# 67 Stone
PLANT:	Auburn			Martin Marietta	Jamestown	78-M Stone
DESIGNED BY:	C Bacchi			Martin Marietta	Jamestown	Washed Scrgs.
SPECIFICATION:	12.5 mm	Surface Mix		Martin Marietta	Jamestown	Dry Screenings
GYRATIONS:	8/100/160	150 mm	297 °C			
TRAFFIC LEVEL:	C					
AC SPECIFIC GRAVITY:		1.035		Larco	Plant	Baghouse Fines

GRADATION OF MATERIALS USED

MATERIAL		# 67	78-M	W. Scrg.	D. Scrg.	Bag. Fines		BLEND	CONTROL
PERCENT		11.0	25.0	35.0	28.0	1.0			POINTS
50.0 mm		100.0	100.0	100.0	100.0	100.0		100.0	
37.5 mm		100.0	100.0	100.0	100.0	100.0		100.0	
25.0 mm		100.0	100.0	100.0	100.0	100.0		100.0	
19.0 mm		100.0	100.0	100.0	100.0	100.0		100.0	100.0
12.5 mm		58.0	99.0	100.0	100.0	100.0		95.1	90.0 - 100.0
9.50 mm		26.0	84.0	100.0	100.0	100.0		87.9	< 90.0
4.75 mm		5.1	27.0	98.0	99.0	100.0		70.3	
2.36 mm		2.5	3.4	77.0	83.0	100.0		52.3	28.0 - 58.0
1.18 mm		1.5	2.2	51.0	60.0	100.0		36.4	<25.6, >31.6
0.600 mm		1.1	2.0	35.0	44.6	100.0		26.4	<19.1, >23.1
0.300 mm		1.0	1.0	21.0	31.3	100.0		17.5	
0.150 mm		1.0	1.0	9.0	20.1	100.0		10.1	
0.075 mm		0.8	0.6	2.6	13.0	97.4		5.8	2.0 - 10.0
Elutriation Loss								0.0	
Agg. Bulk Dry S.G.		2.690	2.673	2.684	2.655	2.769		2.675	
Agg. Apparent S.G.		2.725	2.710	2.698	2.706	2.769		2.707	
						Agg. Effective S.G.:		2.709	

		[TRIAL]	FORMULATIONS at N design					
% Asphalt-Total Mix		3.5	5.0	5.5	6.0	6.5	% RAP / % Virgin:	0/100
Lab Bulk Specific Gravity		2.653	2.367	2.390	2.413	2.421	% AC in RAP:	
Max. Specific Gravity (Rice)		2.771	2.507	2.488	2.470	2.452	% AC from RAP:	0.0
% Voids-Total Mix (VTM)		4.3	5.6	3.9	2.3	1.2	% AC Absorption:	0.5
% Solids-Total Mix		95.7	94.4	96.1	97.7	98.8	% ASH:	
% Effective AC Content		3.0	4.5	5.0	5.5	6.0	TSR % Retained :	
Dust to AC Ratio		1.91	1.27	1.15	1.04	0.95	Ignition Furn. Calibr.:	
% By Volume of Effective AC		7.7	10.4	11.6	12.9	14.1	% AC (Design):	
% Solids by Vol. of Agg. Only		88.0	84.1	84.4	84.8	84.6	Rice Specific Gravity:	
% Voids in Mineral Agg. (VMA)		4.3	15.9	15.6	15.2	15.4	Lab Specific Gravity:	
% Voids w/AC (VFA)		181.0	65.0	74.7	84.8	91.9	Percent Air Voids:	
% Gmm @ N initial	**8**	86.5	87.6	88.8	90.3	91.7	Percent VMA:	
% Gmm @ N design	**100**	95.8	94.4	96.1	97.7	98.8	Percent VFA:	
% Gmm @ N maximum	**160**	97.1	0.0	0.0	0.0	0.0	DUST/AC Ratio:	

COMMENTS:			% Gmm @ N initial	
	Sand Equivalent:	76.0	% Gmm @ N max.	
DESIGNED BY:	C. Agg. Angularity:	/	**%** AC TOTAL:	
	F. Agg. Angularity:	48.0	**%** AC from RAP:	
APPROVAL:	Flat & Elongated:	1.7	**%** AC ADDED:	

Mix C1

REVISED 6-12-97

North Carolina Department Of Transportation

M&T FORM 601 (SP)

Materials and Tests Unit
Raleigh, North Carolina

REPORT ON SUPERPAVE MIX DESIGN
M&T Verification

MD# ____________

DATE SUBMITTED:		DATE APPROVED:		
PROJECT NO.:	misc	ASPHALT:	Citgo Wilmington	PG 70-22
COUNTY:	misc	ADDITIVE:	ARR-MAZ Ad-Here LOF 6500	(.5 %)
CONTRACTOR:	Gelder	Martin Marietta	Garner	# 67 Stone
PLANT:	Raleigh	Martin Marietta	Garner	78-M Stone
DESIGNED BY:	Chris Arnold	Martin Marietta	Garner	Washed Scrgs.
SPECIFICATION:	12.5 mm Surface Mix	Gelder	Raleigh	Natural Sand
GYRATIONS:	8/100/160 150 mm 154 °C			
MIX TYPE:	C	Gelder	Raleigh Plant	Baghouse Fines
AC SPECIFIC GRAVITY:	1.040	Gelder	Plant Stockpile	RAP Aggregate

GRADATION OF MATERIALS USED

MATERIAL	# 67	78-M	W. Scrg.	N. Sand		Bag. Fines	RAP	BLEND	CONTROL POINTS
PERCENT	12.0	48.0	18.5	5.0		1.5	15.0		
50.0 mm	100.0	100.0	100.0	100.0		100.0	100.0	100.0	
37.5 mm	100.0	100.0	100.0	100.0		100.0	100.0	100.0	
25.0 mm	100.0	100.0	100.0	100.0		100.0	100.0	100.0	
19.0 mm	100.0	100.0	100.0	100.0		100.0	100.0	100.0	100.0
12.5 mm	50.0	100.0	100.0	100.0		100.0	98.0	93.7	90.0 - 100.0
9.50 mm	27.0	100.0	100.0	100.0		100.0	91.0	89.9	< 90.0
4.75 mm	5.0	93.0	95.0	99.0		100.0	72.0	80.1	
2.36 mm	3.0	35.0	63.0	94.0		100.0	60.0	44.0	28.0 - 58.0
1.18 mm	2.0	5.0	44.0	73.0		100.0	49.0	23.3	<25.6, >31.6
0.600 mm	2.0	2.0	31.0	42.0		100.0	37.0	16.1	<19.1, >23.1
0.300 mm	2.0	2.0	20.0	13.0		100.0	25.0	10.8	
0.150 mm	1.0	1.0	9.0	5.0		99.0	15.0	6.3	
0.075 mm	1.0	1.0	2.5	3.4		97.0	9.0	4.0	2.0 - 10.0
Elutriation Loss				3.4				0.2	
Agg. Bulk Dry S.G.	2.627	2.615	2.607	2.610		2.689	2.670	2.624	
Agg. Apparent S.G.	2.664	2.665	2.657	2.656		2.689	2.670	2.664	
						Agg. Effective S.G.:		2.655	

FORMULATIONS at N design

		[TRIAL]						
% Asphalt-Total Mix		3.5	4.7	5.2	5.7	6.2	% RAP / % Virgin:	15/85
Lab Bulk Specific Gravity		2.653	2.339	2.384	2.389	2.402	% AC in RAP:	5.0
Max. Specific Gravity (Rice)		2.771	2.475	2.457	2.439	2.422	% AC from RAP:	0.8
% Voids-Total Mix (VTM)		4.3	5.5	3.0	2.1	0.8	% AC Absorption:	0.5
% Solids-Total Mix		95.7	94.5	97.0	97.9	99.2	% ASH:	
% Effective AC Content		3.0	4.3	4.8	5.3	5.8	TSR % Retained :	
Dust to AC Ratio		1.33	0.95	0.85	0.77	0.70	Ignition Furn. Calibr.:	
% By Volume of Effective AC		7.8	9.6	10.9	12.1	13.3	% AC (Design):	
% Solids by Vol. of Agg. Only		88.0	85.0	86.1	85.9	85.9	Rice Specific Gravity:	
% Voids in Mineral Agg. (VMA)		2.4	15.0	13.9	14.1	14.1	Lab Specific Gravity:	
% Voids w/AC (VFA)		319.8	63.5	78.6	85.4	94.1	Percent Air Voids:	
% Gmm @ N initial	**8**	86.5	86.5	87.1	87.6	88.2	Percent VMA:	
% Gmm @ N design	**100**	95.8	94.5	95.8	96.6	97.6	Percent VFA:	
% Gmm @ N maximum	**160**	97.1	0.0		0.0	0.0	DUST/AC Ratio:	

COMMENTS:			% Gmm @ N initial	
	Sand Equivalent:	81.0	% Gmm @ N max.	
DESIGNED BY:	C. Agg. Angularity:	/	% AC TOTAL:	5.0
	F. Agg. Angularity:	51.0	% AC from RAP:	0.8
APPROVAL:	Flat & Elongated:	1.6	% AC ADDED:	4.2

Mix C2

REVISED October 2002 M&T FORM 601 (SP)

NCDOT

Mix Design Spreadsheet

Raleigh, NC 27611

REPORT ON SUPERPAVE MIX DESIGN

MD# ___________

DATE SUBMITTED:				DATE APPROVED:			
PROJECT NO.:				BINDER*:	Citgo	Savannah AT02	**PG 76-22**
COUNTY:	Davidson			ADDITIVE:	ARR-MAZ	Ad-Here 6500 LOF	(.5 %)
CONTRACTOR:	Mapco Const			Martin Marietta	Jamestown CA46	# 67 Stone	
PLANT & NO.:	Mapco Asheboro AS85			Martin Marietta	Jamestown CA46	78-M Stone	
DESIGNED BY:	Brian Birdsall			Martin Marietta	Asheboro CA30	Dry Screenings	
SPECIFICATION:	S 12.5 D	Surface Mix		Martin Marietta	Jamestown CA46	Washed Scrgs.	
GYRATIONS:	9/125/205	150 mm	°C				
TRAFFIC LEVEL:	>30.0	Million ESALs					
BINDER SPECIFIC GRAVITY:		1.037				Baghouse Fines	
COMPACTOR TYPE:			Troxler				

GRADATION OF MATERIALS USED

MATERIAL	# 67 Stone	78-M Stone	Dry Screenings	Washed Scrgs			BgHsFines	Rap	BLEND	CONTROL POINTS
PERCENT (MD)	10.0	45.0	27.0	17.0			1.0		100.0	
PERCENT (JMF)	**10.0**	**45.0**	**27.0**	**18.0**					**100.0**	
Sieves(mm) 50.0	100.0	100.0	100.0	100.0			100.0		**100**	
37.5	100.0	100.0	100.0	100.0			100.0		**100**	
25.0	100.0	100.0	100.0	100.0			100.0		**100**	
19.0	100.0	100.0	100.0	100.0			100.0		**100**	100.0
12.5	58.0	99.0	100.0	100.0			100.0		**95**	90.0 - 100.0
9.5	26.0	88.0	100.0	100.0			100.0		**87**	< 90.0
4.75	6.0	26.0	98.0	100.0			100.0		**57**	
2.36	2.8	4.0	75.0	80.0			100.0		**37**	28.0 - 58.0
1.18	2.0	3.0	48.0	51.0			100.0		**24**	
0.600	2.0	2.0	31.0	38.0			99.0		**17**	
0.300	1.0	1.0	20.0	24.0			96.0		**11**	
0.150	1.0	1.0	14.0	10.0			90.0		**7**	
0.075	0.2	0.6	12.0	2.4			62.0		**4.6**	4.0 - 8.0
Ign.Furn. Corr.Factor										
Agg. Bulk Dry S.G.	2.677	2.657	2.771	2.679			2.700		**2.694**	
							Agg. Effective S.G.:		2.721	
Agg. Apparent S.G.	2.709	2.688	2.788	2.699			2.700		**2.719**	

Mix Properties at N design

		Opt. Pb				
% Asphalt Binder-Total Mix		[redacted]	5.1	5.6	6.1	6.6
Gmb @ Ndes **(or Nmax)**		[redacted]	2.403	2.424	2.428	2.433
Max. Specific Gravity(Gmm)		[redacted]	2.513	2.494	2.476	2.458
% Voids-Total Mix (VTM)		[redacted]	4.4	2.8	1.9	1.0
% Solids-Total Mix		[redacted]	95.6	97.2	98.1	99.0
% Effective Binder Content (Pbe)		[redacted]	4.7	5.2	5.7	6.2
Dust to P_{be} Ratio ($P_{0.075}$/Pbe)		[redacted]	0.98	0.88	0.81	0.74
By Volume of Effective Pb		[redacted]	10.9	12.2	13.3	14.5
% Solids by Vol. of Agg. Only		[redacted]	84.7	85.0	84.8	84.5
% Voids in Mineral Agg. (VMA)		[redacted]	15.4	15.1	15.4	15.6
% Voids Fillied w/Binder (VFA)		[redacted]	70.8	80.8	86.4	92.9
% Gmm @ Nini	**9**	[redacted]	85.6	86.7	87.2	88.1
% Gmm @ Ndes	**125**	[redacted]	95.6	97.2	98.1	99.0
% Gmm @ Nmax	**205**	[redacted]				

% RAP / % Virgin:	0/100
Pb in RAP:	
Pb from RAP:	
Pb Absorption:	0.4
% ASH:	
TSR % Retained :	
Ignition Furn. Calibr.:	
Pb (Design):	
Rice Specific Gravity:	
Lab Specific Gravity:	
Percent Air Voids:	
Percent VMA:	
Percent VFA:	
DUST/AC Ratio:	
% Gmm @ Nini	
% Gmm @ Nmax.	
Pb ADDED:	5.2
Pb from RAP:	
Pb TOTAL:	

COMMENTS:	Uncomp. Void Cont.*	
	Sand Equivalent:	77.0
DESIGNED BY:	C. Agg. Angularity	
	F. Agg. Angularity:	49.4
APPROVAL:	Flat & Elongated:	

Mix C3

REVISED 6-12-97

M&T FORM 601 (SP)

North Carolina Department Of Transportation

Materials and Tests Unit

Raleigh, North Carolina

REPORT ON SUPERPAVE MIX DESIGN

M&T Verification

MD# ____________

DATE SUBMITTED:		DATE APPROVED:			
PROJECT NO.:	misc	ASPHALT:	Citgo Wilmington		PG 76-22
COUNTY:	misc	ADDITIVE:	Morton Inter. Morelife 3300		(0.5 %)
CONTRACTOR:	Rea Construction				
PLANT:	Bessemer City	Martin Marietta	Bessemer City	# 67 Stone	
DESIGNED BY:	Mike Rector	Martin Marietta	Bessemer City	78-M Stone	
SPECIFICATION:	12.5 mm Surface Mix	Martin Marietta	Bessemer City	Washed Scrgs.	
GYRATIONS:	9/125/205 150 mm 157 °C	Rea	Pit 126	Sand	
TRAFFIC LEVEL:	D Million ESALs	Rea	Bessemer City Plant	Baghouse Fines	
AC SPECIFIC GRAVITY:	1.034	Rea	Bessemer City Plant	RAP Aggregate	

GRADATION OF MATERIALS USED

MATERIAL		# 67	78-M	W. Scrg.	Sand	Bag. Fines	RAP	BLEND	CONTROL
PERCENT		10.0	51.0	18.0	10.1	1.0	9.9		POINTS
50.0 mm		100.0	100.0	100.0	100.0	100.0	100.0	100.0	
37.5 mm		100.0	100.0	100.0	100.0	100.0	100.0	100.0	
25.0 mm		100.0	100.0	100.0	100.0	100.0	100.0	100.0	
19.0 mm		98.0	100.0	100.0	100.0	100.0	100.0	99.8	100.0
12.5 mm		51.0	99.0	100.0	100.0	100.0	97.0	94.3	90.0 - 100.0
9.50 mm		25.0	92.0	100.0	100.0	100.0	87.0	87.1	< 90.0
4.75 mm		6.0	24.0	100.0	96.0	100.0	66.0	48.1	
2.36 mm		3.0	5.0	88.0	91.0	100.0	54.0	34.2	28.0 - 58.0
1.18 mm		2.0	5.0	66.0	77.0	100.0	46.0	28.0	<25.6, >31.6
0.600 mm		1.0	4.0	51.0	46.0	100.0	38.0	20.7	<19.1, >23.1
0.300 mm		1.0	3.0	39.0	16.0	98.0	20.0	13.2	
0.150 mm		1.0	3.0	22.0	5.0	81.0	18.0	8.7	
0.075 mm		1.0	1.8	6.6	2.4	54.0	12.0	4.2	2.0 - 10.0
Elutriation Loss					2.4			0.2	
Agg. Bulk Dry S.G.		2.865	2.865	2.860	2.570	2.565	2.570	2.797	
Agg. Apparent S.G.		2.885	2.885	2.881	2.586	2.579	2.579	2.815	
						Agg. Effective S.G.:		2.807	

FORMULATIONS at N design

		[TRIAL]						
% Asphalt-Total Mix		3.5	4.5	5.0	5.5	6.0	% RAP / % Virgin:	9.9/90.1
Lab Bulk Specific Gravity		2.653	2.478	2.490	2.517	2.529	% AC in RAP:	5.1
Max. Specific Gravity (Rice)		2.771	2.606	2.585	2.565	2.545	% AC from RAP:	0.5
% Voids-Total Mix (VTM)		4.3	4.9	3.7	1.9	0.6	% AC Absorption:	0.1
% Solids-Total Mix		95.7	95.1	96.3	98.1	99.4	% ASH:	
% Effective AC Content		3.4	4.4	4.9	5.4	5.9	TSR % Retained :	
Dust to AC Ratio		1.24	0.95	0.86	0.78	0.71	Ignition Furn. Calibr.:	
% By Volume of Effective AC		8.7	10.5	11.7	13.1	14.4	% AC (Design):	
% Solids by Vol. of Agg. Only		87.1	84.6	84.6	85.1	85.0	Rice Specific Gravity:	
% Voids in Mineral Agg. (VMA)		8.5	15.4	15.4	14.9	15.0	Lab Specific Gravity:	
% Voids w/AC (VFA)		102.4	68.2	76.2	87.5	95.9	Percent Air Voids:	
% Gmm @ N initial	**9**	86.5	86.9	89.1	89.3	91.4	Percent VMA:	
% Gmm @ N design	**125**	95.8	94.5	96.8	97.5	99.5	Percent VFA:	
% Gmm @ N maximum	**205**	97.1	0.0	0.0	0.0	0.0	DUST/AC Ratio:	

COMMENTS:			% Gmm @ N initial	
	Sand Equivalent:	83.0	% Gmm @ N max.	
DESIGNED BY:	C. Agg. Angularity:	/	**% AC TOTAL:**	4.8
	F. Agg. Angularity:	53.0	**% AC from RAP:**	0.5
APPROVAL:	Flat & Elongated:	6.8	**% AC ADDED:**	4.3

Mix C4

REVISED 10-5-99

M&T FORM 601 (SP)

North Carolina Department Of Transportation

Materials and Tests Unit

Raleigh, NC

REPORT ON SUPERPAVE MIX DESIGN

for Auburn Test Track

MD# ____________

DATE SUBMITTED:				DATE APPROVED:			
PROJECT NO.:	test track			ASPHALT:	Ergon	Mississippi	PG 67-22
COUNTY:	Auburn			ADDITIVE:	Morton Inter.	Morelife 2200	(0.75 %)
CONTRACTOR:	NCDOT			Martin Marietta	Jamestown	# 67 Stone	
PLANT & NO.:	Auburn			Martin Marietta	Jamestown	78-M Stone	
DESIGNED BY:	Chris Bacchi			Martin Marietta	Jamestown	Washed Scrgs.	
SPECIFICATION:	S 12.5 C	Surface- coarse Mix		Martin Marietta	Jamestown	Dry Screenings	
GYRATIONS:	8/100/160	150 mm	155 °C				
TRAFFIC LEVEL:	10.0 to 30.0	Million ESALs					
AC SPECIFIC GRAVITY:		1.029		Larco	High point Plant	Baghouse Fines	

GRADATION OF MATERIALS USED

MATERIAL	67	78M	W Scrgs	D Scrgs		0	BgHsFines	Rap	BLEND	CONTROL
PERCENT (MD)	15.0	47.0	18.0	19.0		0.0	1.0	0.0	100.0	POINTS
PERCENT (JMF)						**0.0**	**0.0**	**0.0**	**0.0**	
Sieves(mm) 50.0	100.0	100.0	100.0	100.0			100.0		**100**	
37.5	100	100	100	100			100		**100**	
25.0	100	100	100	100			100		**100**	
19.0	99.7	100	100	100			100		**100**	100.0
12.5	58.2	99.3	100	100			100		**93**	90.0 - 100.0
9.5	26	84	100	100			100		**81**	< 90.0
4.75	5.1	26	98	99			100		**50**	
2.36	2.5	3.4	77	83			100		**33**	28.0 - 58.0
1.18	1.5	2.2	51	60			100		**23**	<25.6,>31.6
0.600	1.1	2	35	44.6			100		**17**	<19.1,>23.1
0.300	0.6	1	21	31.3			100		**11**	
0.150	0.5	1	9	20.1			100		**7**	
0.075	0.8	0.6	2.6	13.0			97.4		**4.3**	2.0 - 10.0
Agg. Bulk Dry S.G.	2.690	2.673	2.684	2.655			2.769		**2.675**	
Agg. Apparent S.G.	2.725	2.710	2.698	2.706			2.769		**2.710**	
							Agg. Effective S.G.:		2.709	

Mix Properties at N design

		Nmax				
% Asphalt Binder-Total Mix		**5.3**	4.0	4.5	5.0	5.5
Gmb @ Ndes **(or Nmax)**		**2.421**	2.340	2.369	2.417	2.448
Max. Specific Gravity(Gmm)		**2.496**	2.543	2.524	2.505	2.486
% Voids-Total Mix (VTM)		**3.0**	8.0	6.1	3.5	1.5
% Solids-Total Mix		**97.0**	92.0	93.9	96.5	98.5
% Effective AC Content (Pbe)		**4.8**	3.5	4.0	4.5	5.0
Dust to AC Ratio ($P_{0.075}$/Pbe)		**0.90**	1.23	1.08	0.96	0.86
% By Volume of Effective AC		**11.3**	8.0	9.2	10.6	11.9
% Solids by Vol. of Agg. Only		**85.7**	84.0	84.7	85.9	86.6
% Voids in Mineral Agg. (VMA)		**14.3**	16.0	15.4	14.2	13.5
% Voids Fillied w/AC (VFA)		**79.0**	50.0	59.7	74.6	88.1
% Gmm @ Nini	**8**	**89.2**	83.3	84.9	87.3	88.6
% Gmm @ Ndes	**100**	**96.5**	92.0	93.9	96.5	98.5
% Gmm @ Nmax	**160**	**97.0**				

% RAP / % Virgin:	
% AC in RAP:	0.0
% AC from RAP:	0.0
% AC Absorption:	0.5
% ASH:	
TSR % Retained :	
Ignition Furn. Calibr.:	
% AC (Design):	
Rice Specific Gravity:	
Lab Specific Gravity:	
Percent Air Voids:	
Percent VMA:	
Percent VFA:	
DUST/AC Ratio:	
% Gmm @ N initial	
% Gmm @ N max.	
% AC TOTAL:	
% AC from RAP:	
% AC ADDED:	

COMMENTS:	Sand Equivalent:	83.0
DESIGNED BY:	C. Agg. Angularity:	/
	F. Agg. Angularity:	49.0
APPROVAL:	Flat & Elongated:	2.1

Mix C5

REVISED 6-12-97

North Carolina Department Of Transportation

M&T FORM 601 (SP)

Materials and Tests Unit

Raleigh, North Carolina

REPORT ON SUPERPAVE MIX DESIGN

M&T Verification

MD# ____________

DATE SUBMITTED:		DATE APPROVED:		
PROJECT NO.:	misc	ASPHALT:	Citgo Wilmington	PG 64-22
COUNTY:	misc	ADDITIVE:	ARR-MAZ Ad-Here LOF 6500	(.5 %)
CONTRACTOR:	Nello Teer			
PLANT:	Holly Springs			
DESIGNED BY:	Andrew Johnson	Hanson	Holly Springs	78-M Stone
SPECIFICATION:	9.50 mm Surface Mix	Hanson	Holly Springs	Washed Scrgs.
GYRATIONS:	7/75/115 150 mm 145 °C	Hanson	Gardner	Dry Screenings
TRAFFIC LEVEL:	B			
AC SPECIFIC GRAVITY:	1.036			

GRADATION OF MATERIALS USED

MATERIAL			78-M	W. Scrg.	D. Scrg.			BLEND	CONTROL
PERCENT			53.0	45.0	2.0				POINTS
50.0 mm			100.0	100.0	100.0			100.0	
37.5 mm			100.0	100.0	100.0			100.0	
25.0 mm			100.0	100.0	100.0			100.0	
19.0 mm			100.0	100.0	100.0			100.0	
12.5 mm			100.0	100.0	100.0			100.0	100.0
9.50 mm			96.0	100.0	100.0			97.9	90.0 - 100.0
4.75 mm			31.0	91.0	99.0			59.4	< 90.0
2.36 mm			7.0	62.0	81.0			33.2	32.0 - 67.0
1.18 mm			1.0	38.0	56.0			18.8	
0.600 mm			1.0	24.0	41.0			12.2	
0.300 mm			1.0	14.0	29.0			7.4	
0.150 mm			1.0	8.0	19.0			4.5	
0.075 mm			1.0	6.2	12.4			3.6	2.0 - 10.0
Elutriation Loss								0.0	
Agg. Bulk Dry S.G.			2.644	2.684	2.647			2.662	
Agg. Apparent S.G.			2.735	2.719	2.692			2.727	
						Agg. Effective S.G.:		2.669	

		[TRIAL]	FORMULATIONS at N design					
% Asphalt-Total Mix		3.5	5.0	5.5	6.0	6.5	% RAP / % Virgin:	0/100
Lab Bulk Specific Gravity		2.653	2.325	2.359	2.382	2.392	% AC in RAP:	
Max. Specific Gravity (Rice)		2.771	2.474	2.456	2.438	2.421	% AC from RAP:	0.0
% Voids-Total Mix (VTM)		4.3	6.0	3.9	2.3	1.2	% AC Absorption:	0.1
% Solids-Total Mix		95.7	94.0	96.1	97.7	98.8	% ASH:	
% Effective AC Content		3.4	4.9	5.4	5.9	6.4	TSR % Retained :	
Dust to AC Ratio		1.05	0.73	0.66	0.60	0.56	Ignition Furn. Calibr.:	
% By Volume of Effective AC		8.7	11.0	12.3	13.6	14.8	% AC (Design):	
% Solids by Vol. of Agg. Only		87.0	83.0	83.7	84.1	84.0	Rice Specific Gravity:	
% Voids in Mineral Agg. (VMA)		3.8	17.0	16.3	15.9	16.0	Lab Specific Gravity:	
% Voids w/AC (VFA)		227.8	64.6	75.7	85.5	92.5	Percent Air Voids:	
% Gmm @ N initial	**7**	86.5	85.7	88.0	89.0	89.6	Percent VMA:	
% Gmm @ N design	**75**	95.8	94.9	97.2	98.5	99.9	Percent VFA:	
% Gmm @ N maximum	**115**	97.1	0.0	0.0	0.0	0.0	DUST/AC Ratio:	

COMMENTS:			% Gmm @ N initial	
	Sand Equivalent:	78.0	% Gmm @ N max.	
DESIGNED BY:	C. Agg. Angularity:	/	**% AC TOTAL:**	5.5
	F. Agg. Angularity:	46.1	**% AC from RAP:**	
APPROVAL:	Flat & Elongated:	1.3	**% AC ADDED:**	

Mix C6

REVISED 6-12-97

North Carolina Department Of Transportation

M&T FORM 601 (SP)

Materials and Tests Unit

Raleigh, North Carolina

REPORT ON SUPERPAVE MIX DESIGN

M&T Verification

MD# ____________

DATE SUBMITTED:			
PROJECT NO.:	misc		
COUNTY:	Watauga		
CONTRACTOR:	Maymead		
PLANT:	Boone		
DESIGNED BY:	Don Greer		
SPECIFICATION:	9.50 mm	Surface Mix	
GYRATIONS:	8/100/160	150 mm	152 °C
TRAFFIC LEVEL:	C	Million ESALs	
AC SPECIFIC GRAVITY:		1.031	

DATE APPROVED:		
ASPHALT:	Tosco Bristol	PG 70-22
ADDITIVE:	ARR-MAZ Ad-Here LOF 6500	(.5 %)
Maymead Limstn.	Mountain City	# 8 Stone
Maymead Limstn.	Mountain City	Screenings
Maymead Limstn.	Mountain City	Washed Scrgs.

GRADATION OF MATERIALS USED

MATERIAL	# 8	Scrg.	W. Scrg.			Bag. Fines		BLEND	CONTROL
PERCENT	30.0	58.5	10.0			1.5		100.0	POINTS
50.0 mm	100.0	100.0	100.0			100.0		100.0	
37.5 mm	100.0	100.0	100.0			100.0		100.0	
25.0 mm	100.0	100.0	100.0			100.0		100.0	
19.0 mm	100.0	100.0	100.0			100.0		100.0	
12.5 mm	99.0	100.0	100.0			100.0		99.7	100.0
9.50 mm	80.0	100.0	100.0			100.0		94.0	90.0 - 100.0
4.75 mm	15.0	92.0	75.0			100.0		67.3	< 90.0
2.36 mm	5.0	60.0	20.0			100.0		40.1	32.0 - 67.0
1.18 mm	2.0	40.0	4.0			100.0		25.9	
0.600 mm	1.0	27.0	1.2			100.0		17.7	
0.300 mm	1.0	18.0	0.6			100.0		12.4	
0.150 mm	1.0	12.0	0.5			85.0		8.6	
0.075 mm	1.0	8.9	0.4			75.0		6.7	2.0 - 10.0
Elutriation Loss								0.0	
Agg. Bulk Dry S.G.	2.684	2.680	2.683			2.705		2.682	
Agg. Apparent S.G.	2.740	2.740	2.730			2.728		2.739	
						Agg. Effective S.G.:		2.738	

FORMULATIONS at N design

		[TRIAL]				
% Asphalt-Total Mix		3.5	5.1	5.6	6.1	6.6
Lab Bulk Specific Gravity		2.653	2.396	2.403	2.424	2.436
Max. Specific Gravity (Rice)		2.771	2.525	2.506	2.487	2.469
% Voids-Total Mix (VTM)		4.3	5.1	4.1	2.5	1.3
% Solids-Total Mix		95.7	94.9	95.9	97.5	98.7
% Effective AC Content		2.7	4.3	4.9	5.4	5.9
Dust to AC Ratio		2.44	1.53	1.38	1.25	1.14
% By Volume of Effective AC		7.0	10.1	11.3	12.6	13.8
% Solids by Vol. of Agg. Only		88.7	84.8	84.6	84.9	84.8
% Voids in Mineral Agg. (VMA)		4.5	15.2	15.4	15.1	15.2
% Voids w/AC (VFA)		155.0	66.4	73.3	83.2	91.3
% Gmm @ N initial	8	86.5	85.2	85.6	86.8	88.2
% Gmm @ N design	100	95.8	94.5	95.1	96.7	98.6
% Gmm @ N maximum	160	97.1	0.0	0.0	0.0	0.0

% RAP / % Virgin:	0/100
% AC in RAP:	
% AC from RAP:	0.0
% AC Absorption:	0.8
% ASH:	
TSR % Retained :	
Ignition Furn. Calibr.:	
% AC (Design):	
Rice Specific Gravity:	
Lab Specific Gravity:	
Percent Air Voids:	
Percent VMA:	
Percent VFA:	
DUST/AC Ratio:	
% Gmm @ N initial	
% Gmm @ N max.	
% AC TOTAL:	5.6
% AC from RAP:	
% AC ADDED:	

COMMENTS:

DESIGNED BY:

APPROVAL:

Sand Equivalent:	94.8
C. Agg. Angularity:	/
F. Agg. Angularity:	47.3
Flat & Elongated:	1.5

Permeability Results

Sample No.	*Voids*	*% Comp*	*Mix Type*	*Average Time*	*Average Thickness*	*Area, cm3*	*h1*	*h2*	*tc*	*k, 10-5 cm/s*	*FAIL*	*Contractor, Mix Type*
F1-1	6.90	93.10	S12.5C	1140.00	3.93	162.86	92.96	52.95	1.02	9.60		ST Wooten, fine graded
F1-2	7.40	92.60	S12.5C	1085.00	4.60	162.86	93.63	53.62	1	11.45		ST Wooten, fine graded
F1-3	6.60	93.40	S12.5C	908.00	3.83	162.11	92.86	52.85	1	11.58		ST Wooten, fine graded
F1-4	7.00	93.00	S12.5C	876.00	3.83	161.36	92.86	52.85	1.02	12.30		ST Wooten, fine graded
F1-5	7.00	93.00	S12.5C	618.00	3.93	162.86	92.96	52.95	1	17.35		ST Wooten, fine graded
F1-6	7.00	93.00	S12.5C	604.00	4.40	162.86	93.43	53.42	1	19.73		ST Wooten, fine graded
F1-7	6.20	93.80	S12.5C	600.00	4.57	161.36	93.60	53.59	1.02	21.17		ST Wooten, fine graded
F1-8	6.30	93.70	S12.5C	600.00	4.83	161.36	93.86	53.85	1.02	22.32		ST Wooten, fine graded
F1-9	7.00	93.00	S12.5C	417.00	4.83	162.86	93.86	53.85	1.02	31.82		ST Wooten, fine graded
F1-10	7.20	92.80	S12.5C	250.67	3.83	162.11	92.86	52.85	1.02	42.79		ST Wooten, fine graded
F2-1	4.44	95.56	12.5C	Impermeable after 20 minutes								Test Track, Fine
F2-2	4.61	95.39	12.5C	Impermeable after 20 minutes								Test Track, Fine
F2-3	6.51	93.49	12.5C	783.33	8.20	179.87	97.23	57.22	1	24.34		Test Track, Fine
F2-4	5.45	94.55	12.5C	6300.00	8.53	180.66	97.56	57.55	1.02	3.18		Test Track, Fine
F2-5	6.95	93.05	12.5C	1560.00	7.87	179.08	96.90	56.89	1	11.83		Test Track, Fine
F2-6	6.59	93.41	12.5C	1200.00	8.17	179.08	97.20	57.19	1	15.90		Test Track, Fine
F2-7	4.20	95.80	12.5C	Impermeable after 20 minutes								Test Track, Fine
F2-8	3.96	96.04	12.5C	Impermeable after 20 minutes								Test Track, Fine
F2-9	4.93	95.07	12.5C	Impermeable after 20 minutes								Test Track, Fine
F2-10	5.58	94.42	12.5C	Impermeable after 20 minutes								Test Track, Fine
F2-11	4.53	95.47	12.5C	Impermeable after 20 minutes								Test Track, Fine
F2-12	5.01	94.99	12.5C	1800.00	8.23	180.66	97.26	57.25	0.98	10.37		Test Track, Fine
C1-1	5.32	94.68	S12.5C	540.00	4.91	179.08	93.94	53.93	1.02	22.66		Gelder, Coarse
C1-2	7.02	92.98	S12.5C	57.63	5.33	179.08	94.36	54.35	1.02	229.15	*	Gelder, Coarse
C1-3	8.16	91.84	S12.5C	51.85	6.11	179.08	95.14	55.13	1.02	288.82	*	Gelder, Coarse
C1-4	8.24	91.76	S12.5C	25.51	5.25	179.08	94.28	54.27	1.02	511.11	*	Gelder, Coarse
C1-5	9.54	90.46	S12.5C	22.43	4.87	179.08	93.90	53.89	0.98	520.49	*	Gelder, Coarse
C1-6	8.85	91.15	S12.5C	12.60	3.63	179.08	92.66	52.65	0.95	682.22	*	Gelder, Coarse
C1-7	10.55	89.45	S12.5C	18.08	5.65	179.08	94.68	54.67	1.02	771.41	*	Gelder, Coarse
C1-8	10.63	89.37	S12.5C	5.15	3.95	179.08	92.98	52.97	1.02	1942.20	*	Gelder, Coarse
C2-1	7.50	92.50	S12.5D	55.00	3.00	176.71	92.06	52.05	0.95	133.39	*	Mapco, coarse
C2-2	9.00	91.00	S12.5D	29.00	3.67	177.50	92.70	52.69	0.95	301.65	*	Mapco, coarse
C2-3	6.86	93.14	S12.5D	44.58	3.77	179.08	92.80	52.79	0.95	199.50	*	Mapco, coarse
C2-4	5.42	94.58	S12.5D	20.25	3.77	179.08	92.80	52.79	1.00	462.34	*	Mapco, coarse
C2-5	6.78	93.22	S12.5D	6.23	3.93	179.08	92.96	52.95	0.91	1424.61	*	Mapco, coarse
C2-6	5.70	94.30	S12.5D	6.43	3.97	179.08	93.00	52.99	0.98	1499.13	*	Mapco, coarse
C2-7	7.37	92.63	S12.5D	6.09	3.77	179.08	92.80	52.79	0.98	1507.42	*	Mapco, coarse
C2-8	8.93	91.07	S12.5D	6.72	3.90	179.08	92.93	52.92	0.98	1410.96	*	Mapco, coarse
C2-9	10.84	89.16	S12.5D	6.13	4.10	179.08	93.13	53.12	0.91	1506.40	*	Mapco, coarse
C2-10	13.39	86.61	S12.5D	7.71	3.83	179.08	92.86	52.85	0.95	1172.38	*	Mapco, coarse
C2-11	12.44	87.56	S12.5D	12.42	3.83	179.08	92.86	52.85	1.00	766.62	*	Mapco, coarse
C2-12	12.55	87.45	S12.5D	297.67	4.07	179.08	93.10	53.09	1.00	33.81		Mapco, coarse
C2-13	13.15	86.85	S12.5D	25.75	4.03	179.08	93.06	53.05	1.00	387.78	*	Mapco, coarse
C2-14	12.40	87.60	S12.5D	374.00	3.97	179.08	93.00	52.99	1.00	26.29		Mapco, coarse
C3-1	11.03	88.97	12.5D	4.24	2.90	179.08	91.93	51.92	0.98	1687.23	*	Rea, coarse
C3-2	4.44	95.56	12.5D	511.67	4.23	179.08	93.26	53.25	1	20.43		Rea, coarse
C3-3	6.36	93.64	12.5D	105.67	5.07	179.08	94.10	54.09	0.93	108.79		Rea, coarse
C3-4	10.07	89.93	12.5D	2.68	2.67	179.08	91.70	51.69	1		*	Rea, coarse
C3-5	10.72	89.28	12.5D	2.59	2.20	179.08	91.23	51.22	1.05		*	Rea, coarse
C3-6	14.23	85.77	12.5D	3.66	2.13	179.08	91.16	51.15	1.05	1556.69	*	Rea, coarse
C3-7	9.99	90.01	12.5D	5.83	2.77	179.08	91.80	51.79	1	1196.21	*	Rea, coarse
C3-8	8.91	91.09	12.5D	3.71	3.33	179.08	92.36	52.35	1		*	Rea, coarse
C3-9	6.86	93.14	12.5D	387.00	2.80	179.08	91.83	51.82	1	18.24		Rea, coarse
C3-10	4.28	95.72	12.5D	1080.00	2.73	179.08	91.76	51.75	1.02	6.51		Rea, coarse
C3-11	15.00	85.00	12.5D	3.45	3.10	179.08	92.13	52.12	0.98	2210.10	*	Rea, coarse
C3-12	14.04	85.96	12.5D	10.22	2.50	179.08	91.53	51.52	1.05	650.14	*	Rea, coarse
C3-13	11.07	88.93	12.5D	5.28	2.47	179.08	91.50	51.49	1	1182.74	*	Rea, coarse
C3-14	13.92	86.08	12.5D	6.14	2.00	179.08	91.03	51.02	1.05	871.978	*	Rea, coarse
C3-15	13.54	86.46	12.5D	5.52	2.43	179.08	91.46	51.45	1.05	1173.144	*	Rea, coarse
C4-1	3.90	96.10	12.5C	Impermeable after 20 minutes								Test Track, coarse
C4-2	4.91	95.09	12.5C	3600.00	8.67	180.66	97.70	57.69	0.98			Test Track, coarse
C4-3	6.24	93.76	12.5C	425.35	8.60	181.46	97.63	57.62	0.95	44.039		Test Track, coarse
C4-4	4.95	95.05	12.5C	2700.00	8.50	179.87	97.53	57.52	0.95	6.927		Test Track, coarse
C4-5	7.12	92.88	12.5C	753.00	7.50	179.08	96.53	56.52	0.95	22.315		Test Track, coarse
C4-6	7.04	92.96	12.5C	866.67	7.57	179.08	96.60	56.59	1	20.571		Test Track, coarse
C4-7	3.54	96.46	12.5C	Impermeable after 20 minutes								Test Track, coarse
C4-8	2.74	97.26	12.5C	Impermeable after 20 minutes								Test Track, coarse
C4-9	4.47	95.53	12.5C	3600.00	7.67	179.08	96.70	56.69	1.02	5.11		Test Track, coarse
C4-10	4.87	95.13	12.5C	7200.00	7.67	179.08	96.70	56.69	0.95	2.38		Test Track, coarse
C4-11	5.79	94.21	12.5C	686.00	7.87	179.87	96.90	56.89	0.98	26.25		Test Track, coarse
C4-12	5.71	94.29	12.5C	459.33	7.57	179.08	96.60	56.59	1	38.81		Test Track, coarse

Permeability Results, continued

Sample No.	***Voids***	***% Comp***	***Mix Type***	***Average Time***	***Average Thickness***	***Area, cm3***	***h1***	***h2***	***tc***	***k, 10-5 cm/s***	***FAIL***	***Contractor, Mix Type***
C5-1	7.30	92.70	9.5B	46.80	2.67	166.65	91.70	51.69	0.95	147.27	*	Nello Teer
C5-2	11.50	88.50	9.5B	3.63	2.16	167.42	91.19	51.18	0.98	1585.92	*	Nello Teer
C5-3	10.20	89.80	9.5B	12.17	3.39	167.42	92.42	52.41	1	744.17	*	Nello Teer
C5-4	10.10	89.90	9.5B	15.83	3.26	165.13	92.29	52.28	1	559.65	*	Nello Teer
C5-5	9.20	90.80	9.5B	27.97	3.40	169.72	92.43	52.42	1.02	326.96	*	Nello Teer
C5-6	10.30	89.70	9.5B	10.20	3.51	167.42	92.54	52.53	1.02	937.55	*	Nello Teer
C6-1	10.94	89.06	9.5C	37.63	3.24	162.11	92.27	52.26	0.98	233.68	*	Maymead
C6-2	6.25	93.75	9.5C	543.33	3.71	160.61	92.74	52.73	0.98	18.58		Maymead
C6-3	6.05	93.95	9.5C	423.00	3.02	160.61	92.05	52.04	0.95	19.02		Maymead
C6-4	7.80	92.20	9.5C	443.67	5.12	162.11	94.15	54.14	0.95	29.52		Maymead
C6-5	9.51	90.49	9.5C	39.00	4.46	162.86	93.49	53.48	0.98	303.05	*	Maymead
C6-6	8.64	91.36	9.5C	145.33	3.66	162.11	92.69	52.68	0.98	67.81		Maymead
C6-7	8.56	91.44	9.5C	73.00	3.79	162.86	92.82	52.81	0.93	131.92	*	Maymead
C6-8	12.42	87.58	9.5C	11.00	3.50	161.36	92.53	52.52	0.98	862.74	*	Maymead
C6-9	7.00	93.00	9.5C	335.33	4.29	162.11	93.32	53.31	0.98	34.17		Maymead
C6-10	10.00	90.00	9.5C	110.67	3.61	162.11	92.64	52.63	1	89.85		Maymead
C6-11	10.70	89.30	9.5C	33.00	3.32	8.64	92.35	52.34	1.02	285.98	*	Maymead
C6-12	9.10	90.90	9.5C	83.00	4.07	162.11	93.10	53.09	1.02	136.63	*	Maymead

APPENDIX B

NORTH CAROLINA TEST METHOD

For

MEASUREMENT OF WATER PERMEABILITY OF COMPACTED ASPHALT PAVING MIXTURES

Designation: North Carolina Test Method A-100

1. SCOPE

1.1 This test method covers the laboratory determination of the water conductivity of a compacted mixture sample. The measurement provides an indication of water permeability of that sample as compared to those of other asphalt specimens tested in the same manner.

1.2 The procedure uses field core specimens obtained from existing pavements.

1.3 The values stated in metric (SI) units are to be regarded as standard. Values given in parenthesis are for information and reference purposes only.

1.4 *This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety, and health practices and determine the applicability of regulatory limitations prior to use*

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2. APPLICABLE DOCUMENTS

2.1 AASHTO Standards:

M 231	Weights and Balances Used in the Testing of Highway Materials
T 166	Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface Dried Specimens

2.2 ASTM Standards:

D2041	Standard Method for Theoretical Maximum Gravity and Density of Bituminous Paving Mixtures

3. SUMMARY OF TEST METHOD

3.1 A falling head permeability test apparatus, as shown in Figure 1, is used to determine the rate of flow of water through the specimen. Water in a graduated cylinder is allowed to flow through a saturated asphalt sample and the interval of time taken to reach a known change in head is recorded. The coefficient of permeability of the asphalt sample is then determined based on Darcy’s law.

4. SIGNIFICANCE AND USE

4.1 This test method provides a means for determining water conductivity of water-saturated asphalt specimens. It applies to one-dimensional laminar flow of water. It is assumed that Darcy’s law is valid.

5. APPARATUS

5.1 *Permeameter* – See Figure 1. The device shall meet the following requirements:

a) A calibrated cylinder of 31.75 ± 0.5-mm (1.25 ± 0.02 in.) inner diameter graduated in millimeters capable of dispensing 500 ml of water.

b) A sealing tube using a flexible latex membrane 0.635 mm (0.025 in) thick and capable of confining asphalt concrete specimens up to 152.4 mm (6.0 in) in diameter and 80 mm (3.15 in) in height.

c) An upper cap assembly for supporting the graduated cylinder and expanding an o-ring against the sealing tube. The opening in the upper cap shall have the same diameter as the inner diameter of the calibrated cylinder mentioned previously in 5.1 a. The underside of the upper cap assembly should be tapered at an angle of 10 ± 1° (see Figure 1).

d) A lower pedestal plate for supporting the asphalt concrete specimen and expanding an o-ring against the sealing tube. The opening in the plate should have a minimum diameter of 18 mm (0.71 in). The topside of the lower cap should be tapered at an angle of 10 ± 1° (see Figure 1).

e) O-rings of sufficient diameter and thickness for maintaining a seal against the sealing tube.

f) A frame and clamp assembly for supplying a compressive force to the upper cap assembly and lower pedestal necessary to expand the o-rings.

g) An air pump capable of applying 103.42 kPa (15 psi) pressure and capable of applying vacuum to evacuate the air from the sealing tube / membrane cavity.

h) A pressure gauge with range 0 to 103.42 kPa (0 to 15 psi) with ± 2% accuracy.

i) Quick connects and pressure line for inflating and evacuating the sealing tube / membrane cavity.

j) An outlet pipe with a minimum inside diameter of 18 mm (0.71 in) with shutoff valve for draining water.

5.2 *Water*- A continuous supply of clean, non-aerated water, preferably supplied by flexible hose from water source of top graduated cylinder.

5.3 *Thermometer*- A mercury or thermocouple device capable of measuring the temperature of water to the nearest 0.1°C (0.2°F).

5.4 *Beaker*- A 600 ml beaker or equivalent container to be used while measuring the temperature of a water sample.

5.5 *Timer*- A stop watch or other timing device graduated in divisions of 0.1s or less and accurate to within 0.05% when tested over intervals of not less than 15 min.

5.6 *Measuring Device*- A device used to measure the dimensions of the specimen, capable of measuring to the nearest 0.5 mm or better.

5.7 *Saw*- Equipment for wet cutting the specimen to the desired thickness. Dry cut type saws are not to be used.

5.8 *Fan*- An electric fan for drying the wet asphalt specimen.

5.9 *Vacuum Pump*- Capable of evacuating air from the vacuum container to a residual pressure of 4.0 kPa (30 mm of Hg) or less.

5.10 *Vacuum Container*-Either a metal or plastic bowl with a diameter of approximately 180 to 260 mm (7 to 10.25 in.) and a bowl height of at least 160 mm (6.3 in.) shall be equipped with a transparent cover fitted with a rubber gasket and a connection for the vacuum line. Both the bowl and cover should be sufficiently stiff to withstand the applied vacuum pressure without visibly deforming.

6. PREPARATION OF TEST SPECIMENS

6.1 Field Specimens

6.1.1 Saw cut the field core or the laboratory compacted specimen to the desired test sample thickness. The thickness shall be as close to the actual or desired in-place thickness as possible.

6.1.2 Wash the test sample thoroughly with water to remove any loose, fine material resulting from saw cutting.

6.1.3 Determine the bulk specific gravity of the specimen, if necessary.

6.1.4 Measure and record, to the nearest 0.5 mm (0.02 in.) or better, the height and diameter of the sample at three different locations. The three height measurements shall not vary by more than 5 mm (0.2 in.). The diameter of the specimen shall not be less than 144 mm (5.67 in.). (Use M&T Form 651)

6.1.5 Place the specimen in the vacuum container supported above the bottom by a spacer. Fill the container with distilled water at room temperature so that the specimen has at least one-inch of water above the surface. Apply a vacuum of 13 – 67 kPa absolute pressure (10- 26 in. Hg partial pressure) for a short time (5-10 minutes). Remove the vacuum and leave the specimen submerged for a short time (5-10 minutes).

6.2 Laboratory Specimens

6.2.1 Laboratory specimens shall be weighed and compacted following AASHTO test procedure T312-3, except for the changes listed below.

6.2.2 The batch weights of the specimens shall be adjusted so that a compacted specimen will have a diameter of 150 mm and be 75 ± 5 mm in height, with a target void content of 7.5± 0.5% air voids. The number of gyrations shall be enough so that the specimen will not fall apart.

6.2.3 Determine the bulk specific gravity of the specimen using AASTO T166.

6.2.4 Measure and record, to the nearest 0.5 mm (0.02 in.) or better, the height and diameter of the sample at three different locations. The three height measurements shall not vary by more than 5 mm (0.2 in.). The diameter of the specimen shall not be less than 144 mm (5.67 in.). (Use M&T Form 651)

6.2.5 Place the specimen in the vacuum container supported above the bottom by a spacer. Fill the container with distilled water at room temperature so that the specimen has at least one-inch of water above the surface. Apply a vacuum of 13 – 67 kPa absolute pressure (10- 26 in. Hg partial pressure) for a short time (5-10 minutes). Remove the vacuum and leave the specimen submerged for a short time (5-10 minutes).

7. TEST PROCEDURE

7.1 Evacuate the air from the sealing tube / membrane cavity.

NOTE 1: Complete evacuation of the air is aided by pinching the membrane and slightly pulling it away from the hose barb fitting as the pump is stroked.

7.2 Place the specimen on top of the lower pedestal plate and center it.

7.3 Place the sealing tube over the specimen and lower pedestal plate making sure that the sealing tube is oriented so that the hose barb fitting will be located between the o-rings on the upper cap and lower pedestal.

7.4 Insert the upper cap assembly into the sealing tube and let it rest on top of the asphalt concrete specimen.

NOTE 2: Insertion of the upper cap assembly is aided if the graduated cylinder is already inserted into the upper cap assembly. The graduated cylinder can then be used as a handle.

7.5 Install the two cap assemblies onto the permeameter frame and evenly tighten each one, applying a moderate pressure to the upper cap assembly. This action seals the o-rings against the membrane and sealing tube.

7.6 Inflate the membrane to 68.9 ± 3.4 kPa (12 ± 0.5 psi). Maintain this pressure throughout the test.

7.7 Fill the graduated cylinder with water approximately halfway and rock the permeameter back, forth, and sideways enough to dislodge any trapped air from the upper cavity.

7.8 Fill the graduated cylinder to a level above the upper timing mark, see Figure 1. Start the timing device when the bottom of the meniscus of the water reaches the upper timing mark. Stop the device when the bottom of the meniscus reaches the lower timing mark. Record the time to the nearest second. Perform this test three times and check for saturation. While checking for saturation, do not allow the remaining water in the graduated cylinder to run out, as this will allow air to re-enter the specimen.

Saturation in defined as the repeatability of the time to run 500 ml of water through the specimen. A specimen will be considered saturated when the % difference between the first and third test is ≤ 4.0%. Therefore, a <u>minimum</u> of three tests will be required for each asphalt concrete specimen except as stated in Note 4. Saturation of the specimen may require many test runs prior to achieving the ≤ 4.0% requirement. One technique that aids in achieving saturation is to nearly fill the graduated cylinder with water and adjust the water inflow so that it equals the outflow. Allow the water to run in this manner for five or ten minutes and then begin the timed testing. If more than three test runs are required, which is typically the case, then the ≤ 4.0% requirement shall apply to the last three testing times measured.

NOTE 3: If after the third run, the test run time is greater than ten minutes, then the tester can use judgement and consider ending the test, using the lowest time recorded in the permeability evaluation.

NOTE 4: If the test time is approaching thirty minutes during the first test run without the water level reaching the lower timing mark, then the tester may mark the water level at thirty minutes and record this mark and time. Run the test one more time and record the mark and time. Use the mark and time that will result in the highest permeability value.

7.9 Obtain a sample of water in a beaker or other suitable container and determine the temperature to the nearest 0.1°C (0.2°F).

7.10 After the saturation has been achieved and the final time and mark recorded, then release the pressure from the container and evacuate the sealing tube / membrane cavity. Remove the clamp assemblies, upper cap, and specimen.

8. CALCULATIONS

8.1 The coefficient of permeability, k, is determined using the following equation:

$$k = \frac{a\,L}{A\,t}\ 1n(h_1/h_2)\ x\ t_c$$

Where: k = coefficient of permeability, cm/s;
a = inside cross-sectional area of the buret, cm^2
L= average thickness of the test specimen, cm;
A= average 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;
h_2= final head across the test specimen, cm;
t_c = temperature correction for viscosity of water; see Table 1.
A temperature of 20°C (68°F) is used as the standard.

8.2 h_1 and h_2 are the dimensions shown in Figure 1.

NOTE 5: It is beneficial to determine a set of constant dimensional values for a particular permeameter. The dimensions from the underside of the top cap assembly to the lower timing mark and from the underside of the top cap assembly to the upper timing mark are constants. Add the average specimen thickness to these two dimensions and h_1 and h_2 are determined. If the test is stopped at a mark other than the 0 ml lower mark, then add the difference to the h_2 value to arrive at the new h_2 value for this sample. It is helpful to create a spreadsheet that will calculate these values and permeability values automatically.

8.3 For each sample, the coefficient of permeability is computed based on the time and lower mark recorded in 7.8. The result is reported in whole units x 10^{-5} cm/s.

Table 1 – Temperature Correction Factors for Viscosity of Water

℃	℉	*Factor*
10	58	1.30
11	59	1.26
12	60	1.23
13	61	1.20
14	62	1.16
15	63	1.13
16	64	1.10
17	65	1.08
18	66	1.05
19	67	1.02
20	68	1.00
21	69	0.98
22	70	0.95
23	71	0.93
24	72	0.91
25	73	0.89
26	74	0.87
27	75	0.85
28	76	0.83
29	77	0.82
30	78	0.80
31	79	0.78
32	80	0.77
33	81	0.75
34	82	0.74
35	83	0.72