DETERMINATION OF OPTIMUM GRADATION FOR RESISTANCE

TO PERMEABILITY,

RUTTING AND FATIGUE CRACKING

by

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16. Abstract

Permeability has become a major concern in recent years with the implementation of Superpave mixtures. It is important to have pavements that possess characteristics of low permeability which would minimize the effects due to moisture damage and increase the service life of pavements. The permeability of mixtures mainly depends on percent air voids, nominal maximum size of aggregate and the type of gradation. At any given air void content or nominal size, the permeability depends on the size and continuity of air voids. The nature of gradation influences the size and continuity of voids. So the aggregate gradation was optimized for lower permeability without compromising the fatigue and rutting performance of mixtures. The application of Bailey method of gradation analysis showed that the permeability is greatly influenced by the aggregates of #4, #8 and # 16 sizes. Guidelines were proposed for the mix designers to arrive at aggregate blends with low or high permeability. The guidelines recommend a low proportion of #4 size aggregates and high proportion of #8 and # 16 size aggregates for low permeable mixtures. The higher proportions of 1/2" and 3/8" aggregates can be used to ensure the discontinuity of smaller voids. These guidelines were validated by permeability tests on a separate set of newly developed gradations.

The gradation and the binder content of the field cores were used to compact a reference mixture using the Superpave Gyratory compactor in the laboratory. This reference mixture, the "unmodified gradation", is used for permeability tests and performance analysis. Four new mixtures were designed for both 12.5mm and 9.5mm mixtures in such a way that two mixtures had lower permeability and other two mixtures had higher permeability than the coefficient of the unmodified gradation. Performance evaluation tests were conducted on field cores, the unmodified gradations, low permeable and high permeable mixtures before and after moisture damage. The performance evaluation tests included the Frequency Sweep Test at Constant Height, the Repeated Shear Test at Constant Height and the Asphalt Pavement Analyzer Test. The performance analysis indicated that the permeability of the mixtures directly influences their performance in terms of fatigue life, rutting life and moisture susceptibility. The low permeable mixtures have higher fatigue life and rutting life than the high permeable mixtures. The low permeable mixtures have lower percent of reduction in service life due to moisture damage than the high permeable mixtures.

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CHAPTER 1

INTRODUCTION

1.1 Background and Problem Statement

From 1988 to 1993, the Strategic Highway Research Program (SHRP) spent approximately $50 million on research to develop new methods for specifying and designing Hot Mix Asphalt. The result of this large research effort was a new mix design system called "Superpave". Superpave included requirements for aggregates, asphalt binders, and the compacted mixture. Checks and balances were included within the Superpave mix design system to help ensure that the resulting pavements would be both rut resistant and durable. One of the basic and important requirements of asphalt concrete pavements is durability. Durability is critical to the long-term performance of asphalt pavements as it reflects the ability of the mixture to resist weathering from air, water, and solar radiation, as well as abrasion from traffic action. A recent review of the performance of Superpave designed mixes conducted by National Center of Asphalt Technology (NCAT) showed that they provide good resistance to rutting (1). However, the review also indicated that there was a potential durability problem with some Superpave mixtures.

Permeability of asphalt pavements has become a significant concern and an important issue in recent years. Several states have expressed concerns that the Superpave designed pavements are more permeable than pavements previously designed with the Marshall Mix Design. If the mix is too permeable, premature stripping occurs, shortening the life expectancy of the pavement. Permeability is a problem because the relatively coarser nature of newer asphalt mixtures produces a greater number of interconnected voids, allowing air and water to penetrate a pavement. Air increases the likelihood of oxidation of the asphalt binder which can lead to pavement cracking. When water enters the pavement structure, a variety of problems can result, including rutting and stripping, as well as base and subgrade problems.

The amount of voids plays an important role in the durability of asphalt concrete pavements and, in particular, in influencing the resistance to the action of air and water. High voids make the pavement structure more permeable to air and water. High permeability to water encourages stripping of the asphalt from the aggregate particles, and endangering the subgrade layer and the base course as well. Low voids cause rutting and shoving of asphalt mixtures. Low asphalt content, on the other hand, causes pavements to ravel under the action of traffic. Voids content in an asphalt concrete pavement can have a contrasting effect on its properties and performance. Therefore, the voids must be carefully chosen so that none of the important characteristics are sacrificed. The voids are directly related to the density of a mixture: thus, density must be closely controlled to ensure that the initial in-place voids for dense-graded mixtures should be no higher than eight percent and never fall below three percent during the life of the pavement (2).

McLaughlin and Goetz surmised that permeability actually gives a better measure of a pavement's durability than does density (3). Permeability provides an indication of how

HMA will transmit water through the pavement, whereas density is just an indirect measure of in-place air voids. As long as the voids are below eight percent, permeability should not be a problem, but the permeability increases quickly as the void level exceeds eight percent. It is important to realize that the permeability does not depend solely on the total void content but also the nature of the voids in the mix. Mixtures of identical voids content have significantly different permeability coefficients. Size and continuity of air voids should be considered along with the total void content. Large size air pockets are associated with coarse graded mixes, and the larger the air pockets, the greater the possibility to obtain continuity between them. Once the continuity is established, water can easily flow through these connected voids, and eventually causes serious damage to the asphalt pavement layer and the layers underneath it. The permeability of a mixture depends on the aggregate gradation and the compaction level.

With the new Superpave mix design system in use today, more and more coarse graded mix designs are being used. Superpave mixtures are said to have a different void structure than the conventional dense graded mixtures. It is believed that the air voids within the Superpave mixtures are larger in size than the conventional dense graded mixtures if both are compacted to the same air void content. Since the voids are larger, there are more interconnected voids in the pavement layers, causing higher permeability. These permeable pavements allow water to pass through them and cause premature failures. Thus it is important that gradation be developed for surface course mixtures which are coarser in nature with fewer interconnected voids so that their performance is not affected by the moisture damage.

1.2 Objectives

The primary objectives of this study were to

1. Select several 12.5mm and 9.5mm mixtures with gradations on the coarser side and review their void structure and permeability characteristics.
2. Modify the gradations in (1) above, using the Bailey Method to arrive at the desired void structure.
3. Evaluate the gradations developed in (2) above in terms of permeability and recommend a gradation band that will have different permeability levels.
4. Evaluate the effect of permeability on the performance characteristics of mixtures, e.g. rutting and fatigue characteristics.
5. Evaluate the effect of moisture damage on performance characteristics of mixtures such as fatigue life and rutting.

1.3 Research Plan and Methodology

The research plan had the following four main tasks:

1. The field sites were selected in consultation with NCDOT from which test samples were collected.
2. The void structure and permeability characteristics of the field cores were evaluated. Modifications in aggregate gradation were made to alter the void structure and permeability of the field mixtures to minimize the moisture damage as well as help in maintaining the high quality performance of the pavements. A band of gradation for both 12.5mm and 9.5mm mixtures was proposed for arriving at different permeability levels.

3. The performance characteristics were evaluated that included parameters from Repeated Shear at Constant Height (RSCH) tests and Frequency Sweep at Constant Height (FSCH) tests for both unconditioned and conditioned specimens.

Task 1: Field Site Selection and Test Material Procurement

The location and the total number of test sites were selected after consultation with NCDOT. Ideally, these sites were the pavements that contain SUPERPAVE™ volumetrically designed mixtures being used in either new or overlay constructions. These test sites typically had coarser gradations of 12.5mm and 9.5mm nominal maximum size of aggregate (NMSA) with high permeability. The mixture information including gradations and mix designs were obtained. The field cores from these sites as well as the raw materials used in these field sections were procured for further analysis in the laboratory.

Task 2.1: Modification of Gradations

The concepts of Bailey Method of Gradation Evaluation were used to modify the gradations of 12.5 and 9.5mm field mixtures to have a coarser aggregate structure but with fewer interconnected voids or low permeability. Voids in an asphalt mixture, which are fundamental in mixture design, are greatly influenced by changes in the volume percentage of coarse aggregate in the mixture. Permeability of a mixture depends not only on the VMA but also on the size and interconnectivity of the voids. Changing the gradation of the coarse aggregate changes the size of the voids in the coarse aggregate, in turn affecting the resulting VMA in the mixture. The aggregate ratios play an important role in the modification of the nature of gradations. Based on the concepts provided by the Bailey's method, the aggregate gradations were modified for different levels of permeability.

Task 2.2: Evaluation of Permeability Characteristics

Permeability can be defined as the property of a material which permits the passage of fluids through the pores. The permeability characteristics of pavements are based on the Darcy's empirical law that was established for fine grained soils. The seepage of water through pavements is characterized by the parameter "Darcy Coefficient of Permeability (k)". Two general approaches are used to measure permeability of a material using Darcy's law: a constant head test and a falling head test. The falling head test is generally applied for studying the permeability characteristics of pavement cores. The falling head test involves determining the amount of head loss through a given sample over a given time. For the falling head test, the coefficient of permeability is calculated as follows:

$$k = (a\,L / At) * \ln(h_1/h_2)$$

where

k = coefficient of permeability

a = area of stand pipe

L = length of sample

A = cross-sectional area of sample

t = time over which head is allowed to fall

h_1 = water head at beginning of test

h_2 = water head at end of test

Literature suggests that for asphalt mixtures, the falling head test apparatus is better suited than the constant head test apparatus (4). The constant head apparatus does not allow the low-pressure differentials necessary to measure water flow in semi-porous mixtures. However, for open-graded friction courses, a constant head permeability test may be more appropriate.

Therefore, the falling head permeability test was chosen for this study. The test procedure followed to the “Florida Method of Measurement of Water Permeability of Compacted Asphalt Paving Mixtures” currently used by NCDOT (5).

Task 3: Performance Evaluation

The performance testing and performance prediction models are important in designing and managing pavements. The mixtures with modified gradations using the Bailey’s method were evaluated for its performance characteristics using the APA and the SST.

Task 3.1: Asphalt Pavement Analyzer

The rutting susceptibility of the mixtures was assessed by placing samples under repetitive loads of a wheel-tracking device, known as Asphalt Pavement Analyzer (APA). APA is the new generation of the Georgia Loaded Wheel Tester (GLWT). The APA has additional features that include a water storage tank and is capable of testing both gyratory and beam specimens. The APA basically consists of three parallel steel wheels, rolling on a pressurized rubber tube, which applies loading to beam or cylindrical specimens in a linear track. The test specimens, loading tubes and wheels are all contained in a thermostatically controlled environmental chamber. The depth of rutting in the test specimens was measured after the application of 8000 loading cycles.

Task 3.2: Simple Shear Tester

The SST is a closed-loop system that consists of four major components such as the testing apparatus, the test control unit and data acquisition system, the environmental control chamber, and the hydraulic system. In this study, repeated shear test at constant height and frequency sweep test at constant height were used to analyze the performance of HMA mixtures. A full description of the test procedures can be found in AASHTO TP7. The rutting and fatigue analyses were conducted using the test results.

The frequency sweep test at constant height was used to analyze the permanent deformation and fatigue cracking. A repeated shearing load was applied to the specimen to achieve a controlled shearing strain of 0.05 percent. The specimen was tested at each of the following loading frequencies: 10, 5, 2, 1, 0.5, 0.2, 0.1, 0.05, 0.02 and 0.01 Hz. The dynamic shear modulus G^* and phase angle ϕ were determined by this test.

The repeated shear test at constant height was performed to identify an asphalt mixture that is prone to tertiary rutting. Tertiary rutting occurs at low air void contents and is the result of bulk mixture instability. In this test, repeated synchronized shear and axial load were applied to the specimen. The test specimens were subjected to load cycles of 5000 cycles or until the permanent strain reached five- percent. One load cycle consists of 0.1-

second load followed by 0.6-second rest period. The permanent shear strains were measured in this test.

Task 3.3: Analysis of the Service Life of the Pavement Structure

The resulting parameters of FSCH and RSCH tests are the material responses that can be used to predict the pavement's performance under service for distresses such as fatigue cracking and rutting. Fatigue and rutting analysis were performed using surrogate models developed by SHRP 003-A project. Fatigue analysis of SHRP model considers material properties as well as pavement structural layer thickness whereas rutting analysis considers only the material properties. Such a rutting and fatigue analysis of representative pavement sections was performed. A brief summary of the procedure for rutting and fatigue analysis is presented in the following sections.

<u>Task 3.3.1: Fatigue Analysis</u>

Fatigue distress is a function of and dependent on both mixture properties as well as the pavement structural layer thickness. The fatigue analysis procedure requires an estimate of the flexural stiffness modulus (S_o) of the asphalt-aggregate mix at the desired temperature. In this investigation, it is assumed that the effective temperature for fatigue cracking is 20^o C. This estimated flexure stiffness was used in the multilayered elastic analysis to determine the critical strain to which asphalt concrete mixture will be subjected under a standard traffic loading. The critical strain was then used to compute fatigue life of the mixture.

The flexural properties of mixtures were estimated from the RSCH tests using the following equations:

$$S_o = 8.560(Go)^{0.013}$$

$$S_o'' = 81.125\,(Go'')^{0.725}$$

where,

S_o = flexural stiffness in psi

G_o = dynamic shear stiffness at 10Hz in psi

S_o'' = flexural loss stiffness in psi, and

G_o'' = dynamic shear loss stiffness at 10 Hz in psi

The critical tensile strain (ε_o) under the asphalt concrete layer was evaluated. The following equation is used to evaluate the laboratory fatigue resistance of the asphalt mix.

$$Nsupply = 273800 * e^{(0.077.VFA)} * (e_o)^{-3.624} * (S_o'')^{-2.720}$$

where,

Nsupply = the number of E18 load repetitions to laboratory fatigue cracking,

ε_o = critical tensile strain,

S_o'' = the flexural loss stiffness in psi, and

VFA = the voids filled with asphalt in percent.

Task 3.3.2: Rutting Analysis

The rut depth is calculated using the following relationship:

$$Rut\ Depth\ (in.) = 11\ x\ (g_p)$$

where,

γ_p= the maximum permanent shear strain in the RSCH test.

The conversion of the number of RSCH cycles to ESALS is done as follows:

Log (Cycles) = -4.36 + 1.24 log (ESALs)

The rut depths were estimated from the shear strains of RSCH tests using the above equations.

CHAPTER 2

LITERATURE REVIEW

In this chapter, the earlier research studies about the permeability characteristics of the asphalt pavements are discussed. The theory of permeability and the various factors influencing permeability are discussed. The Bailey method of gradation analysis is also described in this chapter.

2.1 Permeability

Permeability can be defined as the ability of a porous medium to transmit fluid. Based on Darcy's studies, the fundamental theory of permeability for soils was established. He showed that the rate of water flow was proportional to the hydraulic gradient and area of a sample. The hydraulic gradient is a very important concept when evaluating permeability. It can be defined as the head loss per unit length. The head loss increases linearly with the velocity of water transmitted through a medium as long as the flow of water is laminar. Once the flow of water becomes turbulent, the relationship between head loss and velocity is nonlinear. Thus in a turbulent water flow condition, Darcy's law is invalid (6). Two general approaches are used to measure permeability of a material using Darcy's law: a constant head test and a falling head test.

2.1.1 Constant Head and Falling Head Tests

The constant head test with soils testing setup similar to that described in ASTM D 5084 was used, as shown in Figure 2.1. The 152-cm-diameter specimen was enclosed in a rubber membrane with porous stones at the top and bottom. The specimen was then placed in a cell, and water was used to apply a confining pressure. Both the inlet pressure and outlet pressure could be controlled on the water as it flowed through the length of the specimen. It was desirable to have low differential pressure so as not to get turbulent flow (4).

Figure 2.1 Constant Head Permeameter

The coefficient of permeability was calculated according to the following formula:

$$k = \left(\frac{QL}{Ath} \right)$$

where

k = permeability, cm/s

Q = quantity of flow, cm3

L = length of specimen, cm

A = cross-sectional area of specimen, cm2

t = interval of time over which flow Q occurs, s

h = difference in hydraulic head across the specimen, cm.

The falling head test involves determining the amount of head loss through a given sample over a given time. This type of test is more suitable for less permeable materials (4).

For the falling head test, the coefficient of permeability is calculated as follows:

$$k = (a\,L / At) * \ln(h_1/h_2)$$

where

- k = coefficient of permeability
- a = area of stand pipe
- L = length of sample
- A = cross-sectional area of sample
- t = time over which head is allowed to fall
- h_1 = water head at beginning of test
- h_2 = water head at end of test

2.2 Permeability in Pavements

Within the hot mix asphalt community, it is generally accepted that the proper compaction of HMA is vital for a stable and durable pavement. For dense-graded mixtures, numerous studies have shown that initial in-place air void contents should not be below 3 percent or above approximately 8 percent. Low air voids have been shown to lead to rutting and shoving, while high air voids are believed to allow water and air to penetrate into the pavement resulting in an increased potential for moisture damage, raveling, and/or cracking.

In the past, it has been thought that for most conventionally designed dense-graded HMA (Hveem and Marshall), increases in in-place air void contents have meant increases in permeability. Zube showed that dense-graded HMA pavements become highly permeable to water at approximately 8 percent air voids (7). Figure 2.2 shows that as long as the voids are below 8 percent, permeability should not be a problem, but the permeability increases quickly as the void level exceeds 8 percent. Brown, Collins and Brownfield, in a study of segregated mixes in Georgia, also showed that HMA mixtures are impermeable to water as long as the air void content was below approximately 8 percent (8). Figure 2.2 shows that permeability increases rapidly at voids content above 8 percent.

However, due to problems associated with some coarse-graded Superpave designed mixes in Florida (gradation passing below maximum density line and restricted zone), the size and interconnectivity of the air voids have been shown to greatly influence pavement permeability(9). The problems encountered with coarse-graded Superpave mixes in Florida and elsewhere have put a high emphasis on the permeability testing of HMA pavements. This is likely due to permeability giving a better indication of a pavement's durability than density alone.

2.3 Factors affecting Permeability Characteristics of Pavements

Permeability in hot mix asphalt pavements is not a new problem. However, since the adoption of the Superpave mix design system the problem has gotten a lot of publicity. Numerous studies have been conducted to identify and investigate the factors affecting permeability characteristics of pavements. The findings of earlier research are compiled here. A number of mixture and construction factors seem to significantly affect the permeability characteristics of pavements. The major factors that affect the permeability characteristics are as follows:

- Air voids
- Nominal Maximum Size of the Aggregate (NMAS)
- Gradation of Aggregates (above or below the Maximum Density Line)
- Lift Thickness
- Roller type
- Time of Construction

Air voids:

The prominent factor that affects the permeability of a pavement is the air voids. Numerous studies have reported that the permeability increases with increasing air voids. A pronounced increase in the permeability is observed when the air voids level of the pavement increases above 8 percent. Choubane et al found that there was no significant change in permeability when the amount of air voids falls below seven percent (9). The researchers recommended an air void content of 6 percent or less for an impervious coarse-graded Superpave pavement. Cooley et al found a strong relationship between the permeability and in-place air voids (10). Studies by Mallick et al confirmed the influence of air voids on the permeability, as shown in Figure 2.2 (11).

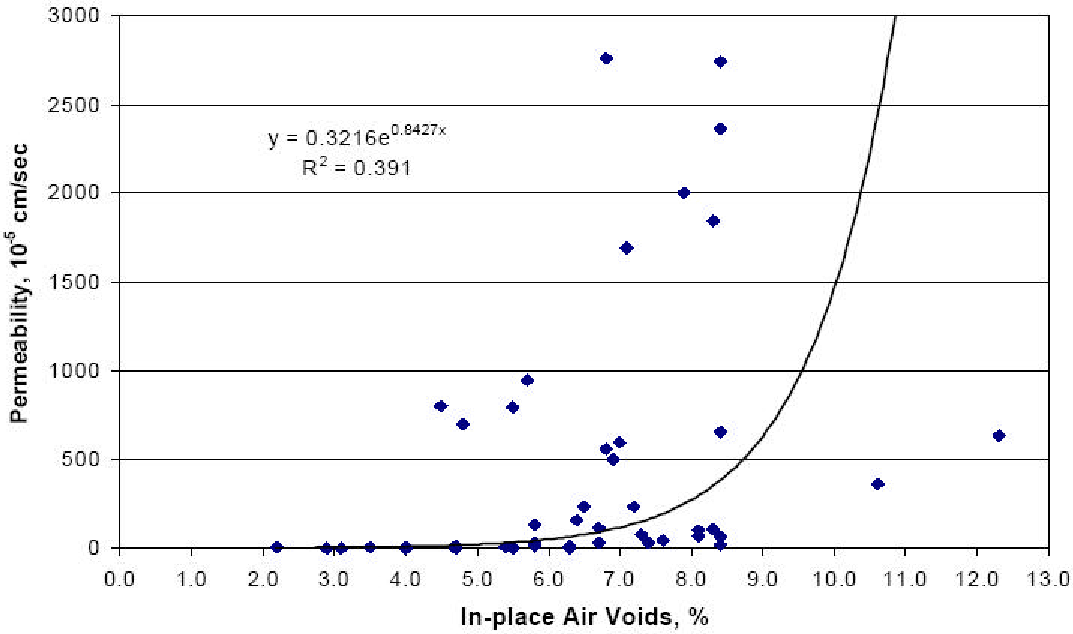

Figure 2.2 Permeability vs Air Voids

Nominal Maximum Size of Aggregate (NMAS):

The permeability of pavements has a direct relationship with the nominal sizes of the aggregates. As the NMAS increases, the size of air voids within a pavement also likely increases, especially in coarse graded Superpave mixes. As the size of voids increases, the potential for interconnected air voids also increases. The in-place permeability of pavements is directly related to the amount of interconnected voids. Therefore, as the NMAS increases the air void level at which a pavement becomes excessively permeable would be expected to decrease.

Permeability Studies by Mallick et al (11) show the change of permeability characteristics with an increase in NMAS, as shown in Figure 2.3. The figure clearly shows that permeability increases with increasing NMAS and so do the in-place air voids.

The authors provide an instance of permeability values for different nominal sizes at an in-place air void level of 6 percent, as shown in Table 2.1.

Table 2.1 Typical Permeability Values for Different Nominal Maximum Sizes

NMAS mm	Permeability, cm/sec
9.5	6×10^{-5}
12.5	40×10^{-5}
19.0	140×10^{-5}
25.0	1200×10^{-5}

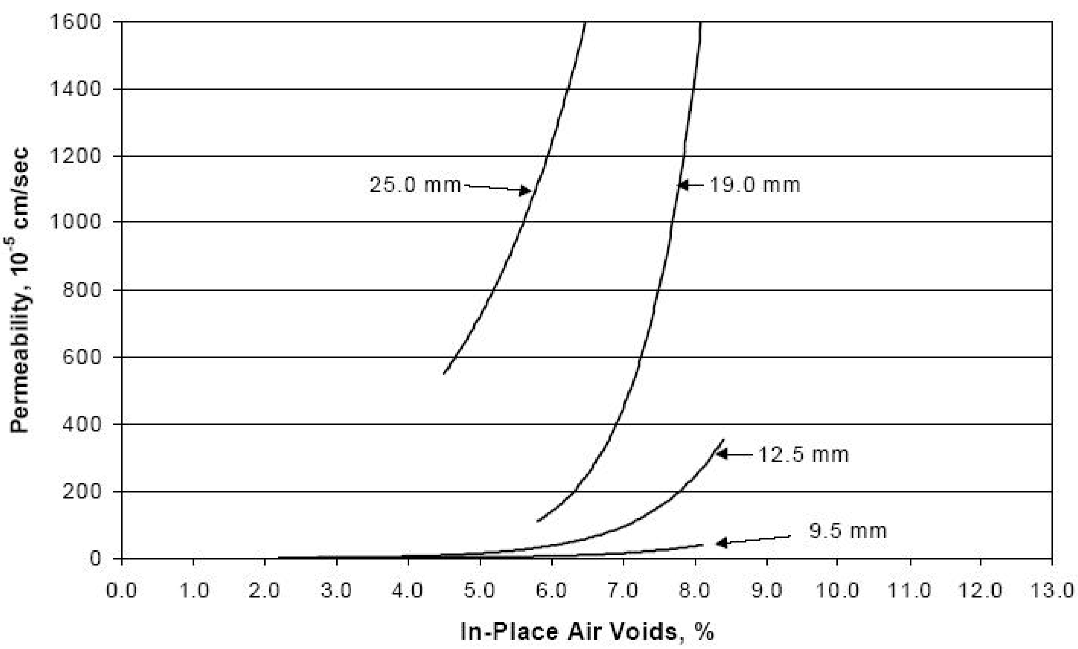

Figure 2.3 Influence of Nominal Maximum Size of Aggregates on Permeability

Another study by Cooley et al confirms the effect of NMAS on permeability (12). The authors found that at a given air void content, the 19.0 mm NMAS mixes showed significantly higher permeability values than the 9.5 and 12.5 mm NMAS mixes. Also, the 25.0 mm NMAS mixes had about three times higher permeability value than the 19.0 mm NMAS mixes at the same air void content. The results are illustrated in Figure 2.4. So the nominal maximum aggregate size of the mix affects the permeability characteristics of a pavement. Mixes having larger nominal maximum aggregate sizes have a potential for high permeability.

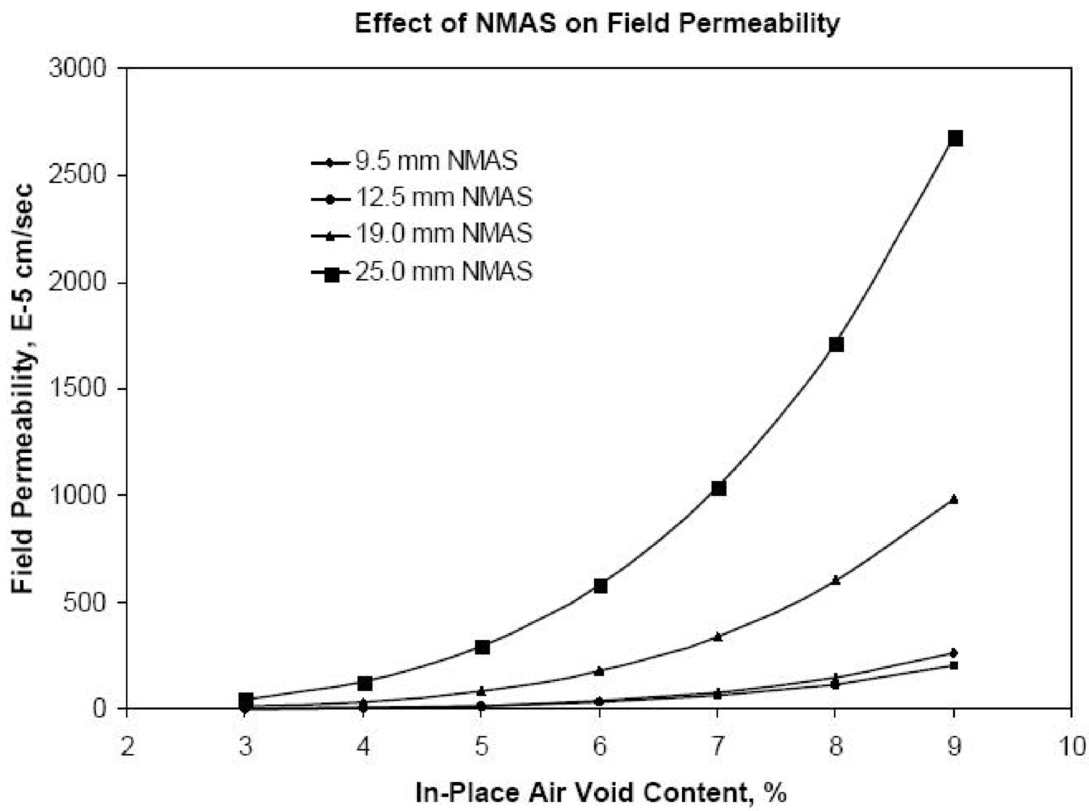

Figure 2.4 Effect of Nominal Maximum Size on Field Permeability

Gradation:

Another factor that affects permeability characteristics is a mixture's gradation shape. Gradations that pass below the maximum density line (MDL) tend to become excessively permeable at lower in-place air void contents than mixes having gradations that pass on the fine side of the MDL. Similar to NMAS, gradation shape likely affects the size of the air voids within a compacted pavement. Coarser gradations contain a higher percentage of coarse aggregate which results in larger individual air voids and, thus, a higher potential for interconnected air voids.

One of the earlier papers that dealt with the influence of gradation on permeability characteristics is "Influence of Aggregate Type and Gradations on Voids of asphalt Concrete Pavements" by Waddah Abdullah et al (13). The researchers used three aggregate types and five gradations in their study. The three gradations were crushed limestone, crushed basalt and crushed granite. As shown in Figure 2.5, the five gradations were as follows:

- ASTM upper limit
- ASTM lower limit
- ASTM middle limit
- Gradation within the ASTM, designated as "A"
- Gradation obtained by Lees' rational method, designated as "rational"

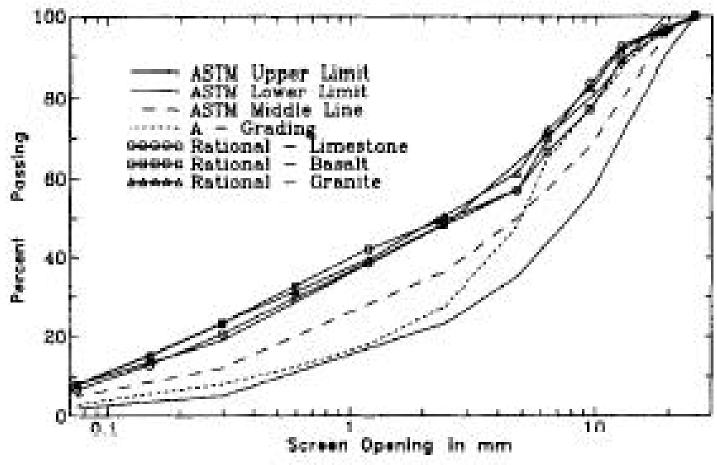

Figure 2.5 Gradations used in Studies by Waddah Abdullah et al

Water permeability studies showed that the coefficient of permeability of a specific asphalt concrete mix yields more or less a straight line relationship with air voids in the mix when plotted as "log10 k (permeability coefficient) vs. air content" in that mix. The test results showed that asphalt concrete mixtures prepared with the ASTM lower limit gradation and the A-gradation yielded the highest permeability values for the three types of aggregates. Asphalt concrete mixtures prepared by Lees rational gradation yielded the lowest values of permeability for the types of aggregates. The results showed that the permeability values were influenced by the size of the voids and voids content itself.

The water permeability index I_{wk} was calculated for the combinations of the aggregate types and gradations, as shown in Table 2.2. The index I_{wk} is defined as follows:

$I_{wk} = d\log_{10} K/dA_v$

The rate of increase of the coefficient of permeability with increasing voids in the mix is not the same for asphalt mixes made with different aggregate gradations. The coarser the mix, the higher the rate of increase and vice versa. This behavior is attributed to the fact that asphalt mixes, based on coarse gradations, have large size void sizes. The more the air voids of this kind, the more the connectability of these voids, thus giving rise to large diameter conduits for water to flow through them.

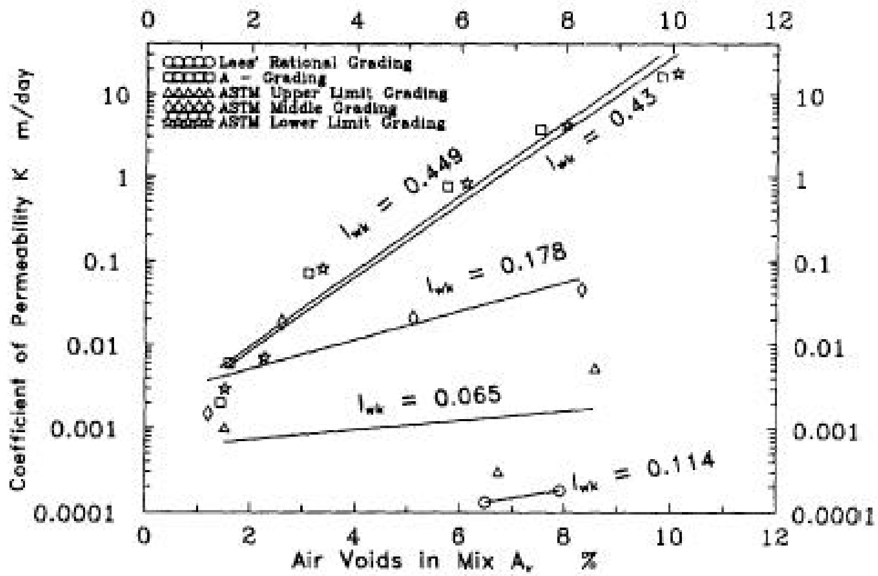

Figure 2.6 Effect of Air Voids on Permeability for Different Gradations

Table 2.2 Permeability Index for Different Gradations

(Studies by Waddah Abdullah et al)

Type of Aggregate	Permeability index for Different Gradations.				
	Rational	A	Upper	Middle	Lower
Limestone	0.017	0.301	0.10	0.143	0.301
Granite	0.114	0.43	0.065	0.178	0.449
Basalt	0.04	0.37	0.10	0.174	0.484

Lift Thickness:

A construction issue that could also affect permeability is the lift thickness at which a pavement is placed. As the lift thickness increases, the potential for permeability is likely to decrease. There are two reasons for lift thickness to make a difference. First, thicker lifts are generally easier to compact in the field because a thicker lift retains heat better and allows more room for aggregate particles to orientate properly; hence, an increase in pavement density. Secondly, permeability is the result of interconnected voids. Within dense-graded hot mix asphalt, all air voids are not interconnected. As lift thickness increases, the chance for voids being interconnected with a sufficient length to allow water to flow decreases. For this reason, thinner pavements may have more potential for permeability.

As with soils, HMA permeability is directly related to the amount of interconnected voids within the pavement. The interconnected voids are the conduits through which water flows. However, unlike soils all voids within HMA pavements are not interconnected. Take for instance a pavement that is 50 mm thick. A single interconnected air void may exist that allows water to flow through the pavement. If the same pavement is 75 mm thick, it is not necessarily true that the same interconnected air void will exist throughout the entire pavement since all air voids are not interconnected.

A study by Mallick et al shows that the thicker pavements are less permeable (11). Figure 2.7 illustrates that the permeability of asphalt pavements is related the thickness of samples. This figure present the results of the laboratory experiment conducted on lab compacted loose mix from three of the five projects. The projects include the coarse graded 9.5 mm, 12.5 mm, and 19.0 mm NMAS mixtures. This figure shows that as the thickness increases (or thickness to NMAS ratio increases), the permeability also decreases. At a thickness to NMAS (t/NMAS) ratio of 2.5, all three mixes exhibit the largest permeability value. Likewise, at the t/NMAS ratio of 4, all mixes had the lowest permeability value. The 12.5 mm NMAS mix did show an anomaly at the 3.5:1 t/NMAS ratio in that the permeability was higher than at the 3 t/NMAS ratio. However, the permeability did again decrease at the t/NMAS ratio of 4. Even with the anomaly, the data clearly shows that the thicker pavements should be less permeable.

Another study by Cooley et al on the issues pertaining to the permeability characteristics of coarse graded mixtures proved that the field permeability is a function of lift thickness and density (12). A multiple linear regression was performed to relate permeability to density, thickness and nominal size. The relationships for field permeability and laboratory permeability with R^2 values of 0.66 and 0.51, respectively, are shown below:

Ln(Field Permeability) = -1.787 + 0.592(Air Voids) + 0.196(NMAS) - 0.23(t/NMAS)

Ln(Lab Permeability) = -5.335 + 4.61*Ln(Air Voids) + 0.138(NMAS) - .024(Thickness)

Figures 2.7, 2.8 and 2.9 illustrate the results of multiple linear regression for field as well as laboratory data.

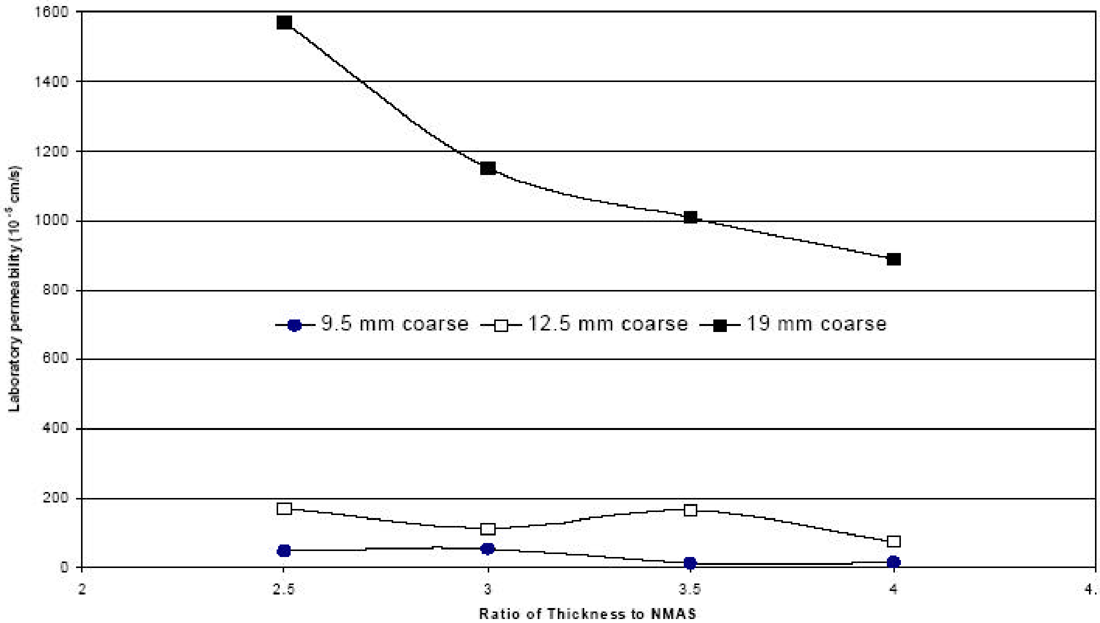

Figure 2.7 Lab Permeability vs Ratio of Lift Thickness to NMAS

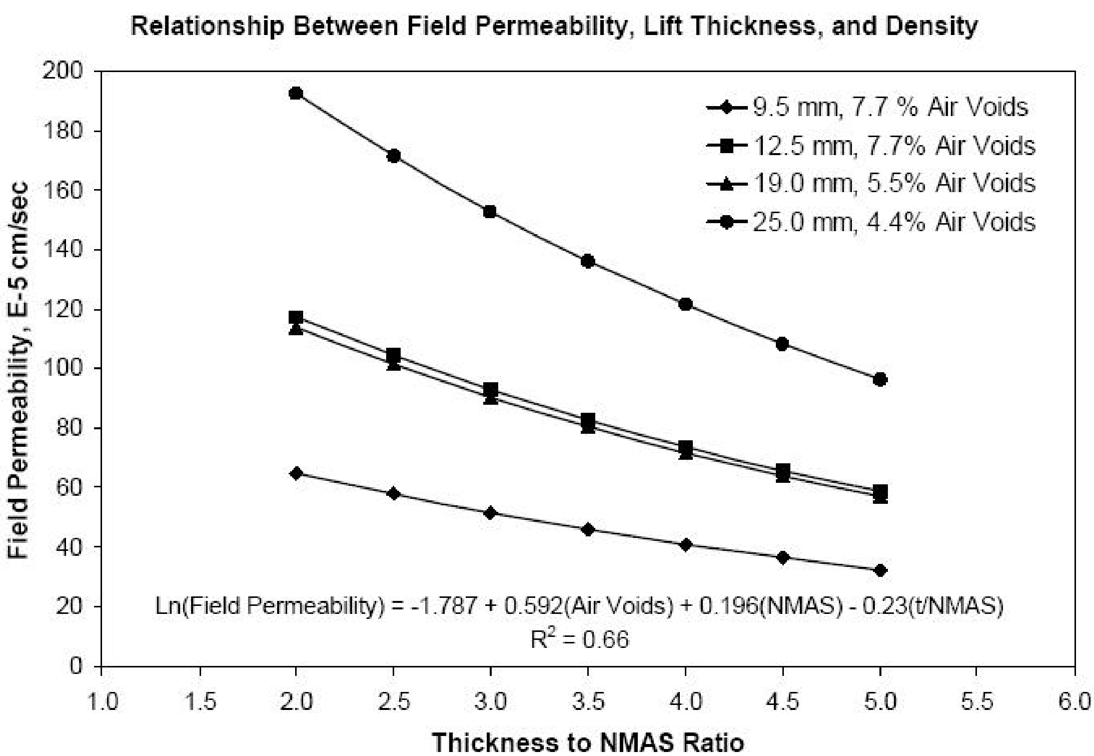

Figure 2.8 Relationship between Field Permeability Lift Thickness and Density

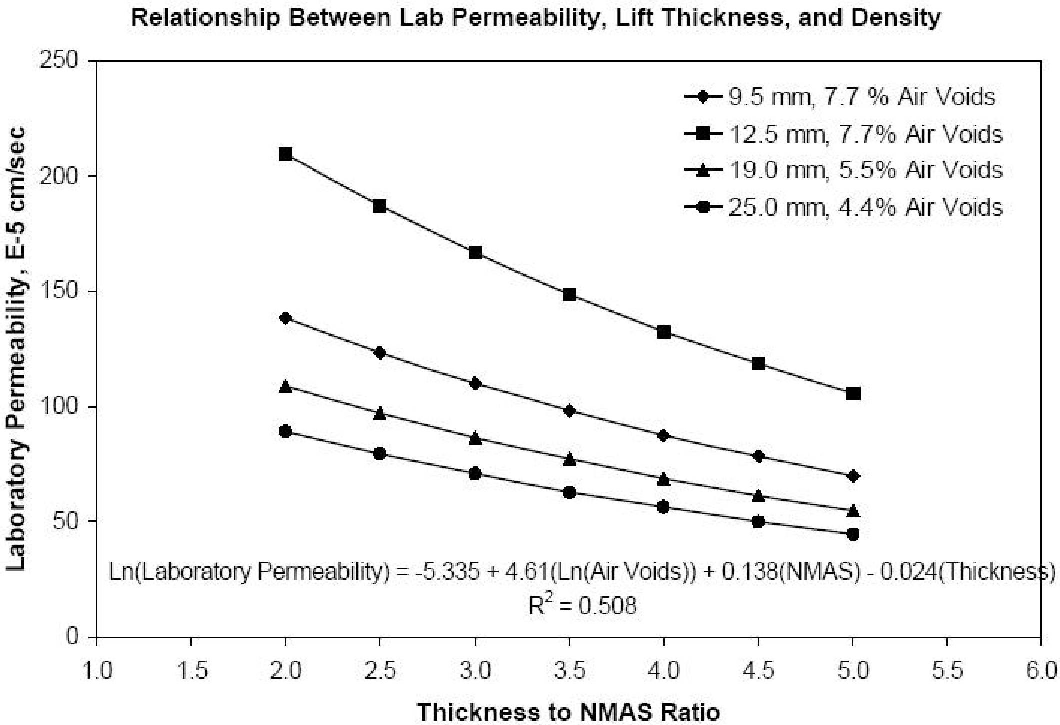

Figure 2.9 Relationship between Lab Permeability Lift Thickness and Density

A study by Arkansas State Highway and Transportation department shows that the permeability coefficient k decreases with increasing lift thickness as shown in Figure 2.10 (13). They developed a regression equation that related permeability coefficient with in-place air voids and thickness. The regression equation (R^2 = 0.748) is shown below:

$$k = (1.38 \times 10^{-7})\,(3.92\%^{AV})\,(0.61^{\text{Lift Thickness}})$$

The study recommended that the minimum lift thickness should be greater than 2 inches or 4 times the maximum nominal aggregate size.

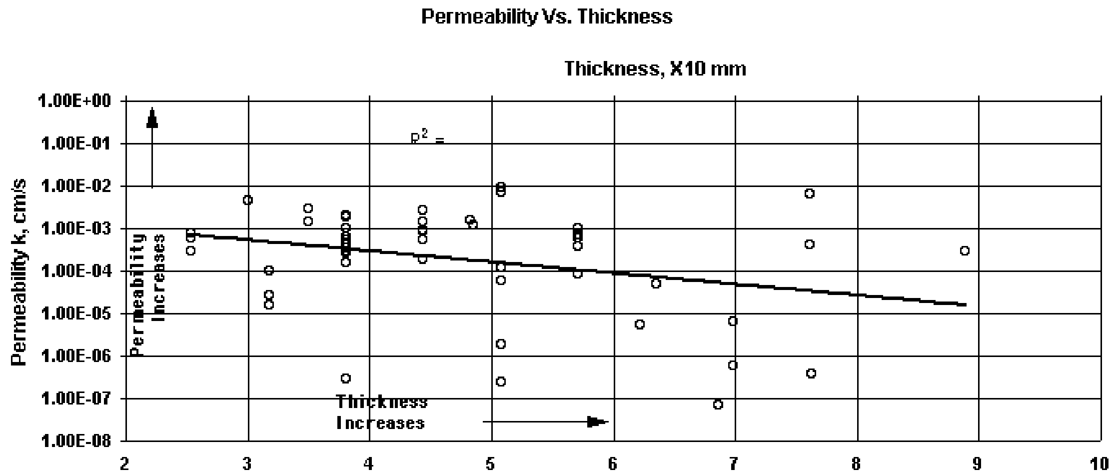

Figure 2.10 Influence of Thickness of Asphalt Layer on Permeability

<u>Roller Type:</u>

Another construction issue that may influence permeability is roller type. Work by Cechetini has suggested that the type of roller and rolling pattern during construction can affect pavement permeability (14). According to Cechetini, vibratory rollers reduce the potential for permeability. Also, there was a suggestion in the past that the use of pneumatic tire rollers may decrease the potential for permeable pavements. Pneumatic rollers tend to knead the pavement during compaction which may reduce the potential for interconnected voids.

<u>Time of Construction:</u>

Zube found that the time of the construction will affect the permeability characteristics (7). 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. Pavements constructed in the fall may not "seal up" due to cooler weather and lead to permeability problems for an extended time period. This is a valid point and shows why a fixed “paving season” is essential to quality pavements.

2.4 Critical Permeability Values

Few research studies have drawn up certain guidelines for categorizing pavement sections according to the permeability values of their representative cores. The University of Arkansas as part of the AHTD’s Transportation Research Project No. 82, "Asphalt Mix Permeability" categorizes as shown in Table 2.3. The above classification is categorized only on the permeability coefficients, irrespective of the NMAS and density.

Table 2.3 Critical Permeability Values as Provided by AHTD Studies

Permeability Category	Permeability Rates
High Permeability	10^{1}-10^{-4}
Low Permeability	10^{-4}-10^{-6}
Practically Impervious	10^{-6}-10^{-9}

In a research study at NCAT, Cooley et al developed critical values of permeability taking the pavement density and nominal size into account (15). Table 2.4 furnishes the critical values of permeability and pavement density for various nominal sizes such as 9.5mm, 12.5mm, 19.0mm and 25.0 mm. The term “critical” used in that study inferred the point at which a pavement becomes excessively permeable. For the larger NMAS mixes, some permeability may be acceptable as long as the upper layers are impermeable.

Table 2.4 Critical Permeability Values for Various Nominal Maximum Sizes

NMAS, mm	Permeability, cm/sec	Density
9.5	100×10^{-5}	92.3 %
12.5	100×10^{-5}	92.3%
19.0	120×10^{-5}	94.5%
25.0	150×10^{-5}	95.6%

2.5 Permeability and Shear Strength

Kentucky Transportation Center investigated a pavement section which had permeability related problems (16). The field permeability tests showed that in the areas and locations where the mat failed to meet density requirements exhibited very high permeability in those locations. When the direct shear tests were performed on these cores, the results showed a direct relationship between maximum shear strength and density. The authors concluded that nearly all of the laboratory shear tests had maximum shear strengths less than the stresses calculated from the layered elastic analysis. There is a high probability that rutting would have occurred due to excessive amount of consolidation in the wheel paths. So an inverse relationship seems to exist between permeability and shear strengths of the mixtures.

2.6 Correlation between Lab and Field Permeability values

It would be interesting to study the correlation between the lab and field permeability values. This is important as the mechanism of percolation of water is different in laboratory and field measurement. In the laboratory test, Darcy’s law of one-dimensional flow is applicable. Measuring the in-situ permeability of an in-place pavement is different as water flow is two dimensional. Other factors that would affect the measurement are the degree of saturation, boundary conditions of the flow etc. In a study on Kansas pavements, the field permeability values are always much higher than the laboratory permeability values. Cooley et al (15) found that there was a nearly one-to-one correlation for permeability values less than 500 x 10^{-5} cm/sec. But as the permeability increases, field permeability values are expected to be higher than the lab permeability values, as shown in Figure 2.11. However at high air voids (which leads to a high probability of large interconnected air voids), laboratory permeability results were higher.

In a similar study by Mallick et al, there was no significant difference between the lab and field permeability values for 9.5mm and 12.5mm values (11). However, for the 19 mm coarse and 25 mm coarse mixes, the differences were very significant, all of the differences were positive (which indicates field permeability is higher), and the differences tend to increase with an increase in VTM. It is believed that permeability was strongly influenced by the aggregate structure and flowpaths in the mixes.

2.7 Factors Influencing Lab Measurement of Permeability

Maupin found that there are few factors that influence the measurement of permeability in laboratory (4). The factors are listed as follows:

- Constant Head or Falling Head Permeameter
- Use of sealant
- Confining pressure
- Faces of Specimens (sawed/unsawed)

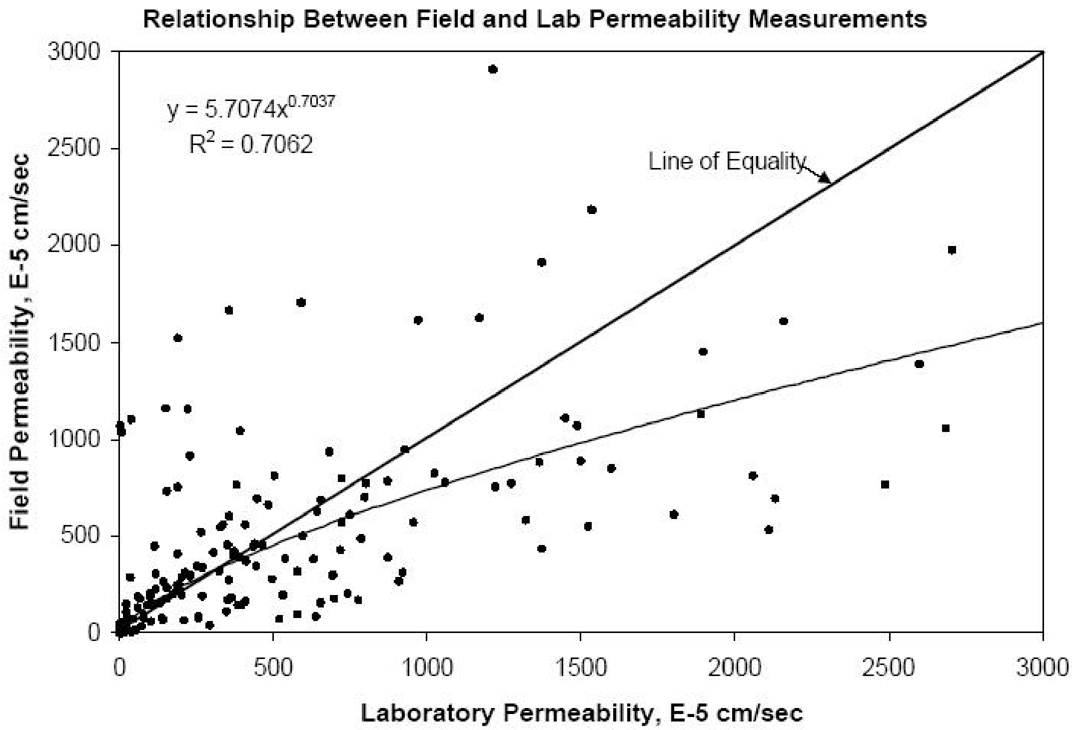

Figure 2.11 Relationships between Field and Lab Permeability Measurements

2.8 Analysis of Gradations: A Background Study

The consensus properties of aggregates are discussed (17) in this section. In HMA design, aggregate type and gradation are considered routinely. The Bailey's method for optimizing gradation is discussed in the next chapter. Mix designers learn by experience the combination of aggregates that will provide adequate voids in the mineral aggregate. Adequate rules or laws that govern the effect of gradation on aggregate packing are not available to mix designers. In a mix design with a given compactive effort, three aggregate properties control the packing characteristics (VMA):

- Gradation
- Surface texture
- Shape

Gradation:

Changing the gradation (particle size distribution) of a mixture will influence the amount of space in the aggregate skeleton. The effect of gradation is separated from shape and surface texture effects if all sized particles have the same shape and texture.

Surface Texture:

The way in which aggregate particles pack together for any given gradation is influenced by the surface texture of the particles. Rougher textures generate more friction between aggregate particles and resist compaction. Therefore, under a standard compactive effort (say, a design number of gyrations), the mixture will not compact as much and VMA will be higher. Typically crushed faces have more texture than non-crushed faces. In the case of gravel aggregate, the more of the particle surface that has a crushed face, the more surface texture will be available. Usually the more crushed a particle is, the more surface texture it will have, but not always. Some aggregates fracture with very smooth faces, so crushing may not always increase texture.

Shape:

For any given gradation, the shape of the particles influences the density to which aggregate particles will pack. Cube-shaped particles will not pack as tightly as flat potato chip particles. In the gyratory compactor, as under traffic, the flat particles lay down flat, one on top of the other. Therefore, there is not much space between them and the VMA is low.

CHAPTER 3

OPTIMIZATION OF AGGREGATE GRADATIONS AND PERMEABILITY TESTS OF MIXTURES

The permeability of the asphalt mixtures depends not only on the total void content but also on the size and continuity of the voids. The aggregate gradation plays an important role in determining the size and continuity of the voids. To control the permeability of the mixtures, the aggregate gradations can be modified. This chapter deals with the optimization of the aggregate gradations for a required level of permeability.

3.1 Bailey Method of Gradation Analysis

Changing the aggregate gradation of a mixture alters the particle size distribution which in turn influences the amount of space in the aggregate skeleton. The Bailey Method of Gradation Analysis can be used for optimizing aggregate gradations. The Bailey method primarily deals with the estimation/measurement of aggregate interlock for required rut resistance using a regression relationship between VMA and packing coefficients. The methodology of the Bailey Method of Gradation Analysis takes into consideration the packing characteristics of individual aggregates and provides quantified criteria that can be used to adjust the packing characteristics of a blend of materials (17). The Bailey Method involves the following approach:

- Evaluates packing of coarse and fine aggregates individually
- Contains a definition for coarse and fine aggregate
- Evaluates the ratio of different size particles
- Evaluates the individual aggregates and the combined blend by volume

The end result is an aggregate blend that is packed together in a systematic manner to form an aggregate skeleton. This method also provides the user with a method to closely evaluate and adjust an aggregate blend to

- Achieve or maintain volumetric properties
- Alter mix compactibility
- Alter mix-handling characteristics

The Bailey Method of Gradation Analysis presents the foundation for a comprehensive gradation evaluation procedure. It outlines a method to combine aggregates that provides aggregate interlock as the backbone for the aggregate skeleton. Aggregate ratios, which are based on particle packing principles, are used to analyze the particle packing of the overall aggregate structure. This method postulates the use of coarse aggregate as the primary component in an asphalt mixture for developing the aggregate structure and the effect of aggregate gradation on VMA.

Four sieves are defined to quantify the shape of the gradation curve. The sieves are represented in Figure 3.1.

- The primary control sieve is selected as the split between coarse aggregate and fine aggregate.
- The half sieve is selected as an intermediate sieve in the coarse aggregate
- The fine aggregate break sieve is selected as the split between the coarser and finer part of the fine aggregate

➢ The fine aggregate break sieve is selected as an intermediate sieve in the finest part of the aggregate gradation

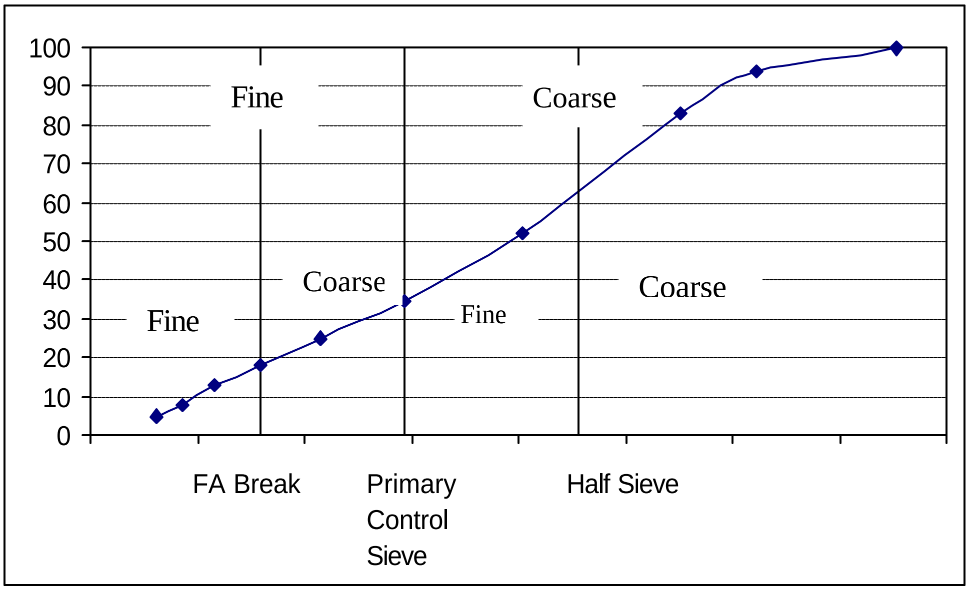

Figure 3.1 Division Points in Coarse and Fine Aggregate Fractions

The primary control sieve is determined as follows to find the closest sized sieve:

$$PCS = NMPS \ x \ 0.22$$

where,

PCS = primary control sieve for the overall blend

NMPS – nominal maximum particle size for the overall blend

The value 0.22 is the factor that gives the average size opening between the coarse particles, considering the different shapes of aggregates. Therefore, the average size of the coarse aggregate voids in a 9.5-mm nominal maximum size mix is smaller than the voids in a 19-mm nominal maximum size mix. Thus, coarse aggregate void size is a function of particle size and shape. The coarse portion of any blend is defined as that portion retained on the primary control sieve. The coarse aggregate can be further broken down into what is considered to be the coarse portion of the coarse aggregate and the fine portion of the coarse aggregate using a ‘half’ sieve, which is determined as follows:

$$\textit{Half sieve} = \textit{NMPS} \times 0.5$$

The half sieve represents a division in the coarse aggregate structure where changes could alter the packing characteristics of the coarse aggregate fraction of the mix.

The Fine Aggregate Initial Break (FAIB) sieve and the Fine Aggregate Secondary Break (FASB) sieve are defined as follows:

$$FAIB = PCS \times 0.22$$

$$FASB = FAIB \times 0.22$$

Three ratios define the shape of the gradation curve. One ratio defines the shape of the coarse aggregate portion of the gradation. The second ratio defines the shape of the coarse portion of the fine aggregate, and the third ratio defines the shape of the fine portion of the fine aggregate. All three ratios influence VMA in the combined gradation.

The CA ratio is used to represent the packing characteristics of the coarse aggregate fraction of the combined blend and is defined as follows:

$$\textit{CA Ratio} = (\%\textit{half sieve} - \%\ PCS) / (100 - \%\ \textit{half sieve})$$

where,

% half sieve = percent passing the half sieve

% PCS = percent passing the primary control sieve

The top half of this ratio is the fine portion of the coarse aggregate, referred as interceptors because they will push apart the larger rock sizes. The bottom half of the ratio is the coarse aggregate, referred as pluggers because adding a rock of this size will fill space and reduce VMA. The CA ratio normally falls between 0.4 and 0.8. Mixtures with a low CA ratio can be prone to segregation since there is an unbalance in the coarse aggregate fraction of the mix. As this ratio approaches 1.00, the mixture may be hard to compact, especially in the field, and tend to move more under rolling. As the CA ratio exceeds 1.00, the fine portion of the coarse aggregate dominates the formation of coarse aggregate skeleton. At this point, the coarse portion of the coarse aggregate begins to act as pluggers and close the voids in the coarse aggregate skeleton since they are completely spread apart from each other.

The FA_C ratio of the fine aggregate is used to estimate the packing characteristics of the coarse portion of the fine aggregate:

$$FA_C = \%\ FAIB\ /\ \%\ PCS$$

For most dense graded mixtures, the FA_C ratio should be approximately 0.25 to 0.50. As the ratio increases, the fine aggregate fraction of the overall blend packs together tighter.

The FA_F ratio of the fine aggregate is used to estimate the packing characteristics of the fine portion of the fine aggregate:

$$FA_F = \%\ FASB\ /\ \%\ FAIB$$

The FA_F ratio should be approximately 0.25 to 0.50 to prevent overfilling the voids created by the larger particles. It influences the voids that will remain in the overall fine portion of the blend since it represents the particles that fill the smallest voids created.

The FA_C ratio has the greatest influence on VMA in the combined blend. As the FA_C ratio decreases, VMA increases. Also, the CA ratio and the FA_F ratio influence the amount of VMA of the mixture. The VMA increases with a decrease in the FA_F ratio and an increase in CA ratio. These ratios can be used to estimate the VMA of a mixture by the following relationship:

$$VMA = -24.6 + 20.1(CA)^2 - 3.8\,CA - 191.1FA_C^{\,2} + 181.0\,FA_C + 87.3\,FA_F^{\,2} - 36.6FA_F$$

Multiple regression was performed to create the above model with the data collected from 25 mixtures. The R-square of the model is 0.92. The model indicates that the coarse portion of the fine aggregate is of the highest importance in the development of the aggregate structure. Although the data set used in the generation of this model is not comprehensive in the independent changing of all the aggregate ratios, the resulting model is appropriate for the prediction of VMA with the combination of the given aggregates. Table 3.1 provides the control sieves for 12.5mm and 9.5mm mixtures.

Table 3.1 Control Sieves

Nominal Size, mm	Primary Control Sieve	Half Sieve	Initial Break	Secondary Break
12.5	2.36	4.75	0.6	0.15
9.5	2.36	4.75	0.6	0.15

3.2 Trial Gradations

The concepts of Bailey method was applied for optimizing aggregate gradations for a required level of permeability. The gradations of field cores of 12.5mm and 9.5mm mixtures were modified. Trial gradations were designed with various combinations of the aggregate coefficients such as CA ratio, FA_C, FA_F. The permeability tests were conducted on the mixtures of these trial gradations to select two mixtures with higher permeability and two mixtures with lower permeability.

The gradation curves of the field cores of both the mixtures were below the restricted zone. The gradations that are below the restricted zone are highly susceptible to permeability problems. When these gradations pass above the restricted zone, the proportion of fine aggregate particles would be higher than the coarse aggregate particles, and thereby nature of the mix becomes fine. The effect of permeability is insignificant when the gradation of the mixture passes above the restricted zone. So it was decided to modify the gradation of the mixtures without shifting the gradation to the region above the restricted zone. All the modified mixtures would have gradations passing only below the restricted zone.

Ten trial gradations were selected with different combinations of aggregate ratios. The cylindrical specimens were compacted using the Superpave Gyratory Compactor at five percent asphalt content. The Rice specific gravity values (G_{mm}) of the mixtures were measured. All the mixtures were compacted at 8.5% air voids. Permeability tests were performed on all these mixtures. The permeability of the mixtures increases drastically as the voids level exceeds the eight percent level. The permeability of a mixture is usually low as long as the voids are below eight percent. In this study, all permeability tests and performance evaluation tests were performed at 8.5% air voids as the permeability of the specimen would be critical at this air void content.

The trial gradations of 12.5mm mixtures and 9.5mm mixtures are furnished in Tables 3.2 and 3.3, respectively. The aggregate ratios (CA, FA_C, FA_F) for all the gradations are provided in Tables 3.4 and 3.5 for 12.5mm and 9.5mm mixtures, respectively.

3.3 Permeability Apparatus and Test

A new permeability apparatus was procured and calibrated. Tests for calibration were done in accordance with "North Carolina Test Method A-100". A brief description of the permeability test is described below. The objective of the permeability test is to study the water conductivity of a compacted mixture sample. A falling head permeability apparatus, as shown in Figures 3.2 and 3.3, was used to determine the rate of flow of water through the specimen.

The asphalt concrete specimen, chosen for the study, was confined using a flexible latex membrane. The dimensions of the specimen were 6 inches (152.4mm) in diameter and 3.15 inches (80mm) in height. The faces of the test sample were sawed. The sample was washed thoroughly with water to remove any loose and fine material resulting from saw cutting. The bulk specific gravity of the specimen was determined. The average height of the sample was measured at three different points was determined.

Table 3.2 Trial Gradations for 12.5mm Mixtures

Sieve Size	Unmodified	Grad1	Grad2	Grad3	Grad4	Grad 5	Grad 6	Grad 7	Grad 8	Grad 9	Grad 10
19	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
12.5	93.6	91.0	91.0	92.0	89.0	96.0	93.6	93.6	90.0	90.0	87.0
9.5	83.5	83.5	85.0	81.0	81.0	86.0	83.5	83.5	85.0	80.0	79.0
4.75	50.8	54.8	58.0	56.4	66.0	59.0	62.3	57.6	68.0	58.0	64.0
2.36	32.2	36.0	42.0	29.0	46.0	33.0	32.2	36.0	48.0	28.0	44.0
1.18	22.5	22.5	22.0	22.0	22.0	25.0	18.0	22.5	20.0	20.0	23.0
0.6	16.8	12.9	14.7	15.4	15.0	13.0	12.9	17.7	12.0	14.0	16.0
0.3	11.9	8.0	10.0	11.0	9.0	9.0	8.0	10.0	10.0	8.0	10.5
0.15	7.1	3.9	5.9	7.7	6.0	6.5	6.4	7.1	6.0	6.0	7.5
0.075	4.1	2.5	3.5	3.5	4.0	4.5	4.1	4.1	4.1	3.9	4.0
Pan	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Table 3.3 Trial Gradations for 9.5mm Mixtures

Sieve Size	Unmodified	Grad1	Grad2	Grad3	Grad4	Grad 5	Grad 6	Grad 7	Grad 8	Grad 9	Grad 10
19	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
12.5	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0
9.5	91.0	91.0	92.0	94.0	94.0	92.0	93.0	91.0	91.0	95.0	92.0
4.75	56.0	66.0	60.0	50.0	70.0	55.0	72.0	49.0	50.0	51.0	52.0
2.36	37.0	40.0	45.0	30.0	50.0	30.0	40.0	38.0	35.0	29.0	33.0
1.18	27.0	26.0	29.0	21.0	25.0	19.0	25.0	29.0	25.0	19.0	22.0
0.6	19.0	14.0	20.0	15.0	15.0	12.0	20.0	22.0	20.0	12.0	15.0
0.3	14.0	10.0	12.0	11.0	11.0	9.0	15.0	17.0	15.0	9.0	8.0
0.15	9.0	7.0	8.0	8.0	8.0	7.0	10.0	11.0	10.0	7.0	6.0
0.075	5.3	4.0	5.0	5.0	5.0	4.0	5.0	5.0	5.0	4.0	3.0
Pan	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Table 3.4 Summary of Aggregate Ratios for 12.5mm Mixtures

Gradation	CA	FA_C	FA_F
Unmodified	0.38	0.52	0.42
Grad 1	0.42	0.36	0.30
Grad 2	0.38	0.35	0.40
Grad 3	0.63	0.53	0.50
Grad 4	0.59	0.33	0.40
Grad 5	0.63	0.39	0.50
Grad 6	0.80	0.40	0.50
Grad 7	0.51	0.49	0.40
Grad 8	0.63	0.25	0.50
Grad 9	0.71	0.50	0.43
Grad 10	0.56	0.36	0.47

Table 3.5 Summary of Aggregate Ratios for 9.5mm Mixtures

Gradation	CA	FA_C	FA_F
Unmodified	0.43	0.51	0.47
Grad 1	0.76	0.35	0.50
Grad 2	0.38	0.44	0.40
Grad 3	0.40	0.50	0.53
Grad 4	0.67	0.30	0.53
Grad 5	0.56	0.40	0.58
Grad 6	1.14	0.50	0.50
Grad 7	0.22	0.58	0.50
Grad 8	0.30	0.57	0.50
Grad 9	0.45	0.41	0.58
Grad 10	0.40	0.45	0.40

The test sample was placed in the permeability apparatus. The container was filled with water so that the specimen has at least one inch of water above the surface. The air was evacuated from the membrane cavity. The membrane was inflated to 12.5 psi and this pressure was maintained throughout the test. Water was filled to a level above the graduated, upper timing mark. The timing device was started when the bottom of the meniscus of the water reached the upper timing mark. The timing device was stopped when the bottom of the meniscus of the water reached the lower timing mark. The time was recorded to the nearest second. This test was performed three times and checked for saturation. Saturation is defined as the repeatability of the time to run 500ml of water through the specimen. A specimen is considered saturated when the percent difference between the first and the third test is ≤ 4%.

The coefficient of permeability is determined using the following equation:

$$k = (a\,L / At) * \ln(h_1/h_2)\, t_c$$

where

- k = coefficient of permeability, cm/s
- a = cross sectional area of stand pipe
- L = average thickness of the sample
- A = cross-sectional area of the sample
- t = elapsed time between h_1 and h_2
- h_1 = initial water head across the test specimen
- h_2 = final water head across the test specimen
- t_c = temperature correction factor for viscosity of water.

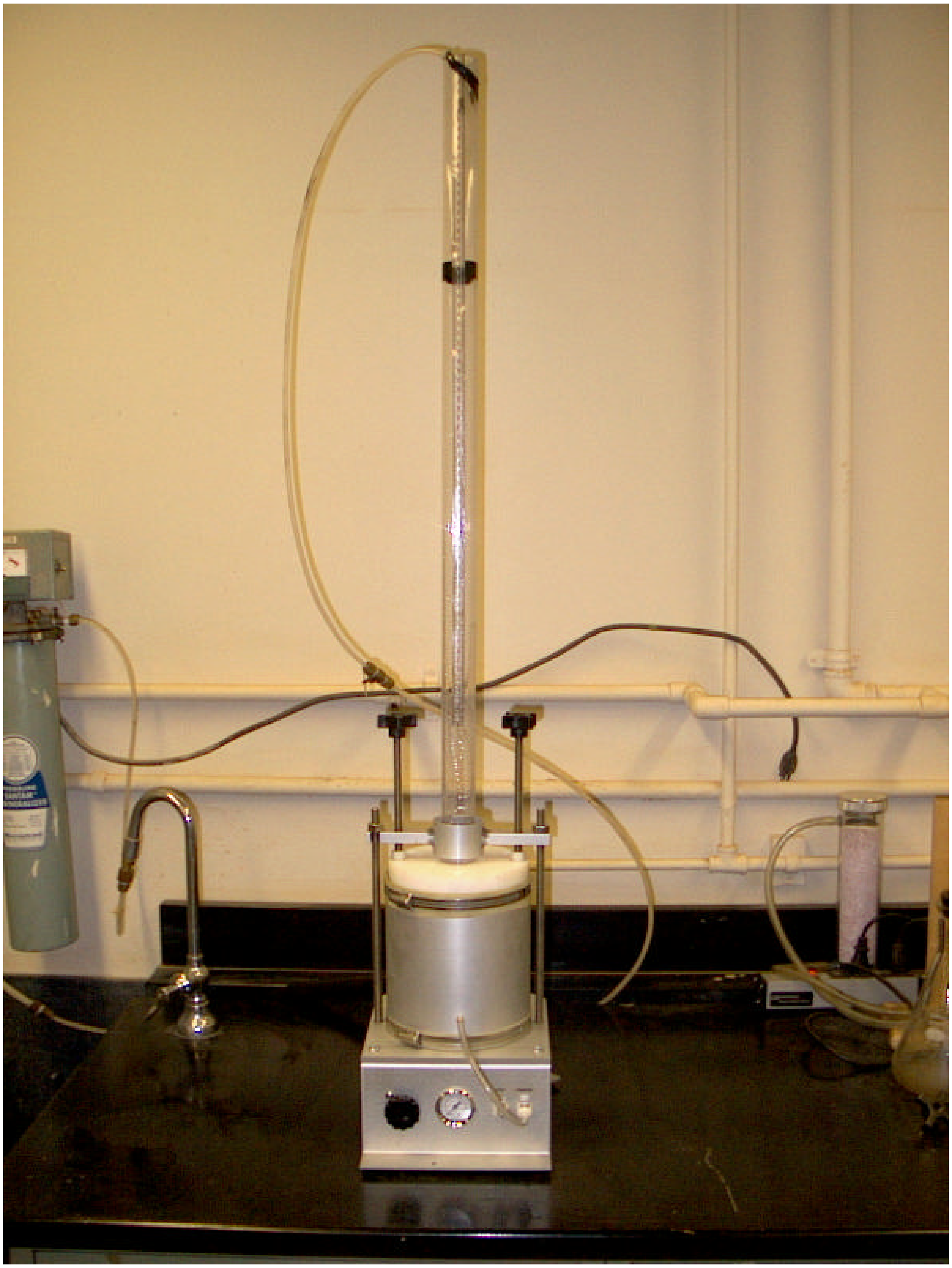

Figure 3.2 Permeability Apparatus

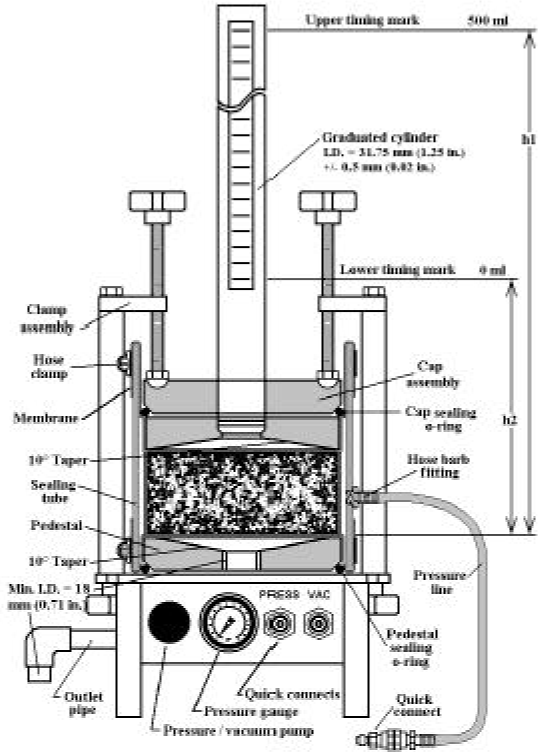

Figure 3.3 Permeability Apparatus

The permeability tests were conducted on the specimens of these gradations and field cores. The parameters that were used in the equation such as head levels, cross section of burette and specimens are given in Table 3.6. The permeability coefficients (k) for 12.5mm and 9.5mm mixtures are furnished in Tables 3.7 and 3.8, respectively. The tables also provide the time taken for 75mm thick specimens for the water to percolate down from its initial head to the final head. As the height of the specimens might not be exactly same, the measured time is normalized to 75mm. The comparison of permeability coefficients is shown in Figures 3.4 and 3.5 for 12.5mm mixtures and 9.5mm mixtures, respectively.

Table 3.6 Parameters used in Permeability Tests

Area of Burette (a)	784.26 mm2
Area of Specimen (A)	70685.83 mm2
Initial Head	51 cm
Final Head	1 cm
Height of Specimen (L)	Variable
Time	Measured

Table 3.7 Permeability Coefficients of 12.5mm mixtures

Gradations	Permeability (k), cm/sec	Time for 75mm Specimen	
		Minutes	Seconds
Field Cores	3.24E-04	7	19
Unmodified	5.78E-05	40	58
Grad 1	1.33E-04	17	48
Grad 2	5.81E-05	40	46
Grad 3	1.22E-04	19	25
Grad 4	2.80E-05	84	35
Grad 5	5.42E-05	43	42
Grad 6	4.24E-05	55	51
Grad 7	5.42E-05	43	42
Grad 8	3.61E-05	65	36
Grad 9	1.57E-04	15	5
Grad 10	3.70E-05	64	0

Table 3.8 Permeability Coefficients of 9.5mm mixtures

Gradations	Permeability (k), cm/sec	Time for 75mm Specimen	
		Minutes	Seconds
Field Cores	3.27E-04	7	15
Unmodified	4.11E-04	5	46
Grad 1	2.46E-04	9	38
Grad 2	2.84E-04	8	20
Grad 3	5.73E-04	4	8
Grad 4	1.59E-04	14	54
Grad 5	2.02E-04	11	43
Grad 6	1.35E-04	17	33
Grad 7	1.74E-04	13	37
Grad 8	1.78E-04	13	18
Grad 9	8.36E-04	2	50
Grad 10	3.47E-04	6	50

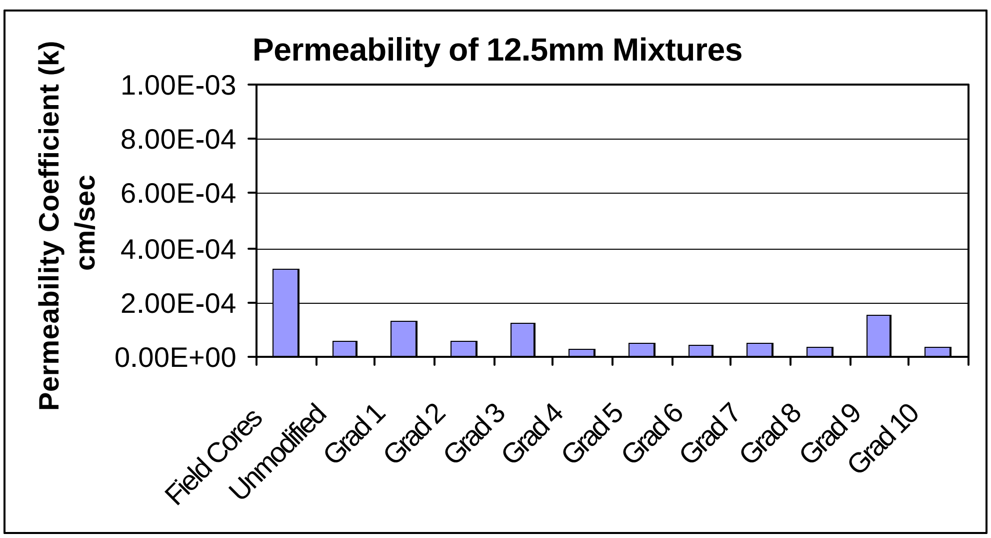

Figure 3.4 Comparison of Permeability Coefficients (12.5mm Mixtures)

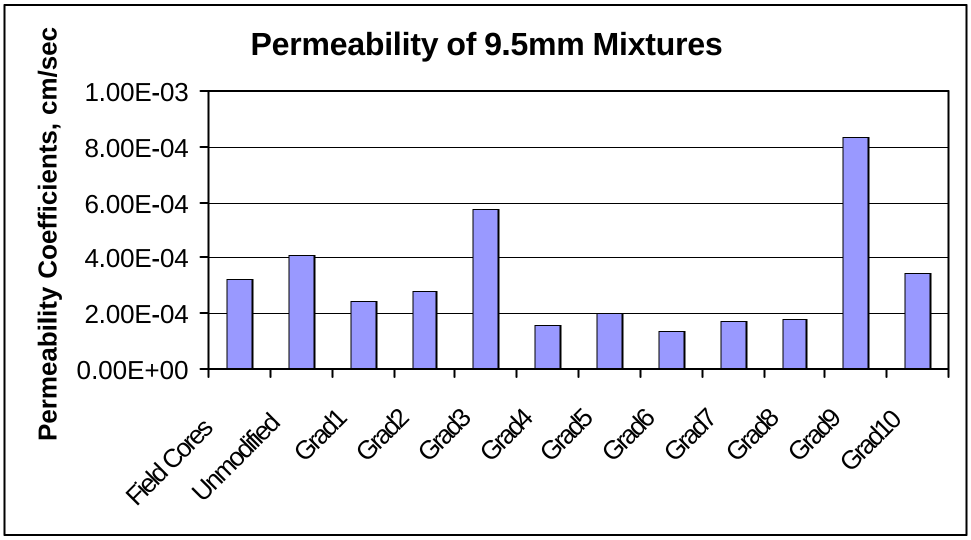

Figure 3.5 Comparison of Permeability Coefficients (9.5mm Mixtures)

Four mixtures for each nominal size were selected from the spectrum based on the permeability values. The permeability coefficient of "Unmodified" mixture was used as a reference value. Two mixtures with higher permeability and two mixtures with lower permeability were selected for 12.5mm as well as 9.5mm mixtures. The selected mixtures were designated as L1, L2, H1 and H2 where L and H represent low and high permeability, respectively. Table 3.9 shows the selected mixtures for both the 12.5 and 9.5mm mixtures.

Table 3.9 Mixtures Selected for Performance Evaluation

Mixture	12.5mm Mixture	9.5mm Mixture
L1	Grad 4	Grad 4
L2	Grad 10	Grad 6
H1	Grad 3	Grad 3
H2	Grad 9	Grad 9

3.4 Guidelines for Selecting Aggregate Gradations

In this section, the guidelines for selecting aggregate gradations for better control of permeability are discussed. These guidelines are framed based on the concepts of Bailey method and interpretations from the permeability tests of trial gradations. If these guidelines are followed, the mix designer would have better chance of formulating gradations with low permeability.

It should be borne in mind that size and continuity of voids influences the permeability characteristics of asphalt concrete mixtures. The Bailey method defines the role of half sieve which divides coarse and fine fractions of coarse aggregate. For 12.5mm and 9.5mm mixtures, half sieve is #4 sieve (4.75mm). The primary control sieve (PCS) defines the break between fine and coarse aggregates in an aggregate blend. For 12.5mm and 9.5mm mixtures, PCS is # 8 sieve (2.36mm). The coarse aggregate part of aggregate blend should be modified for arriving at gradations with low permeability. The fine aggregate part of aggregate blend would not allow sufficient space for modification, as all the gradations pass below the restricted zone.

The Bailey method defines “CA ratio” as the ratio of the fine fraction of coarse aggregate to the coarse fraction of coarse aggregate.

$$\text{CA ratio} = (\%\ \text{half sieve} - \%\ \text{PCS}) / (100 - \%\text{half sieve})$$

$$\text{CA ratio} = (\%\ 4.75 - \%\ 2.36) / (100 - \%\ 4.75)$$

for 12.5mm and 9.5mm mixtures

The top half of this ratio is the fine portion of the coarse aggregate, referred to as interceptors because they will push apart the larger rock sizes. The bottom half of this ratio is the coarse portion of the coarse aggregate, referred to as pluggers because adding a rock of this size will fill space and reduce VMA. These concepts of "interceptors" and "pluggers" are used in developing the guidelines.

The permeability characteristics of aggregate blend mainly hinges on the amount of #4 (4.75) sizes and #8 (2.36) sizes. The amount of #4 and #8 fractions dictate the permeability characteristics and other fractions would fall in place relative to the amount of these fractions. The Bailey method of gradation analysis is not directly employed in these guidelines. The Bailey method considers the overall packing characteristics of the aggregates. The method provides a relationship between VMA and blend ratios. For this study, only sieve sizes greater than #16 are considered as all the gradations should pass below the restricted zone. The concepts of this method were taken into account and modified for the requirements of this study. The CA ratio for mixtures as recommended by Bailey method falls within the range of 0.4 to 0.8. The value of 0.5 can be considered as "break" between low and high permeable mixtures. At the same time, the control points and restricted zone for sieve sizes as recommended by the Superpave mixture design system should also be taken into account.

3.4.1 Guidelines for 12.5mm Mixtures

The guidelines are discussed for gradations with a nominal size of 12.5mm mixtures. The control points and boundaries of restricted zone for #8 and #16 fractions are reproduced in Table 3.10. Two types of percentages are used in this discussion – Percent Passing (PP) and Percent Fraction (PF). Percent Fraction is the percent of a particular fraction of aggregate in the total blend of aggregates e.g., 20% PF of #4 means that the total blend of aggregates contains 20% of 4.75 mm size aggregates. The guidelines are provided as bands of gradations for #4, #8 and #16 sieve sizes in Figure 3.6. Separate bands are provided for both low and high permeability. The gradation curves should pass through the bands for achieving either low or high permeability.

Table 3.10 Control Points for 12.5mm Mixtures

Sieve, mm	Control Points		Restricted Zone Boundary	
2.36 (#8)	28.0	58.0	39.1	39.1
1.18 (#16)	-	-	25.6	31.6

1. <u>Number 4 Sieve Size</u>: For low permeable mixtures, the percent passing of #4 would fall in the range from 60% to 65%. For high permeable mixtures, the percent passing of #4 would fall between in the range of 45% to 53%.
2. <u>Number 8 Sieve Size</u>: For low permeable mixtures, the percent passing of #8 would fall in the range from 35% to 40%. For high permeable mixtures, the percent passing of #8 would fall between in the range of 30% to 35%.
3. <u>Number 16 Sieve Size</u>: For any type of mixture, the percent passing of # 16 would fall in the range of 20% to 25%. The minimum boundary of restriction zone for #16 is 25.6 %. The percent fraction of # 16 for low permeable mixtures would be

more than 10% PF, whereas the PF of # 16 for high permeable mixtures would be less than 10% PF.

4. <u>1/2" and 3/8" Sieve Size</u>: Lower fractions of # 4 size and higher fractions of # 8 and # 16 sizes would decrease the void size. Smaller void size alone would not provide low permeable mixtures, as the continuity of voids can still exist. The continuity of small size voids would not decrease permeability until this continuity is broken. It is recommended to use relatively higher fractions of 12.5mm and 9.5mm fractions in the coarser part of coarse aggregates. By increasing these sizes of aggregates, the percent fraction of #4 reduces. A presence of larger particle size would plug the continuity of voids efficiently.

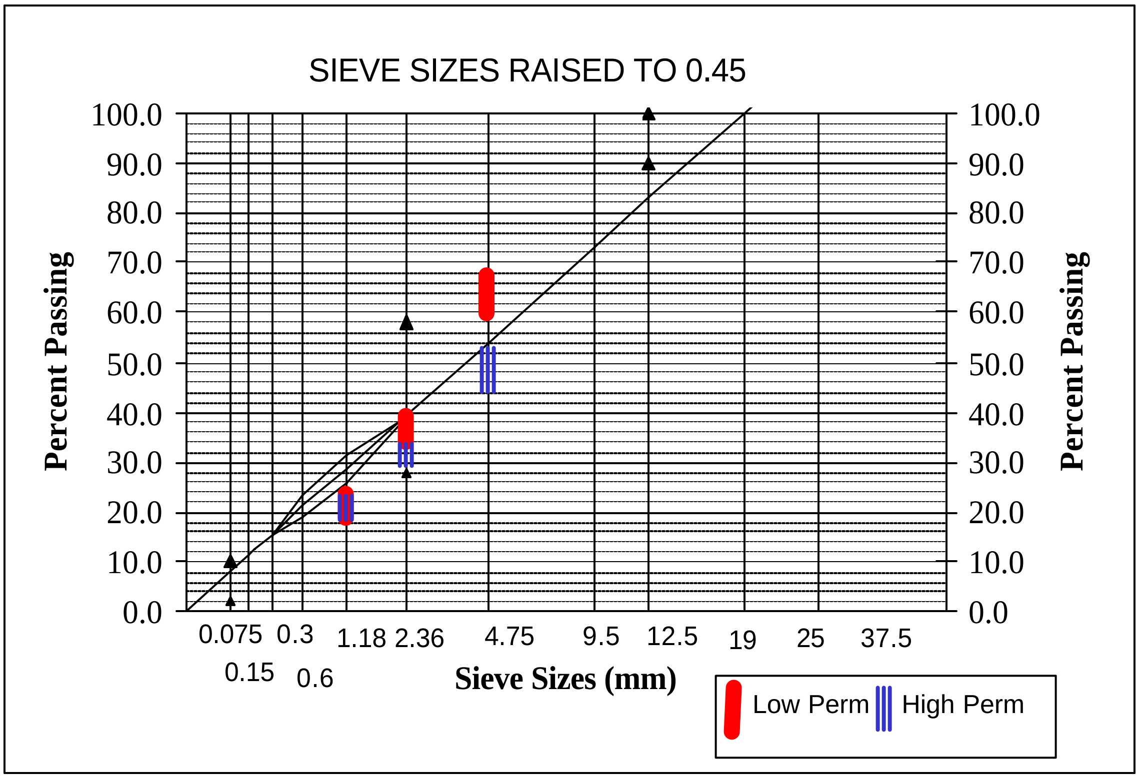

Figure 3.6 Recommended Gradation Bands for 12.5mm Mixtures

3.4.2 Guidelines for 9.5mm Mixtures

The guidelines are discussed for gradations with a nominal size of 9.5mm mixtures. The control points and boundaries of restricted zone for #8 and #16 fractions are reproduced in Table 3.11. The recommended band gradations are shown in Figure 3.7

Table 3.11 Control Points for 9.5mm Mixtures

Sieve, mm	Control Points		Restricted Zone Boundary	
2.36 (#8)	32.0	67.0	47.2	47.2
1.18 (#16)	-	-	31.6	37.6

1. Number 4 Sieve Size: For low permeable mixtures, the percent passing of #4 would fall in the range from 60% to 67%. For high permeable mixtures, the percent passing of #4 would fall between in the range of 45% to 50%. If we assume that the PP of 3/8 " is 90%, the PF of #4 for low permeable mixtures would range from 23% to 30% and the PF of #4 for high permeable mixtures would range from 40% to 45%.
2. Number 8 and 16 Sieve Sizes: As the control points restrict the percent passing of # 8 to 32%, the percent passing of # 8 can be extended to a maximum value of 32.0%. The restricted zone restricts that the percent passing of # 16 should be kept below 31.6%, and therefore, a number below 30% can be selected for the percent passing of # 16 size aggregates.

   For high permeable mixtures, the PF of # 8 ranges from 10 % to 18%, only up to a PP of 32%. The PF of #16 can range from 5% to 10% but should not exceed 10%.

For low permeable mixtures, the combined PF of #8 and # 16 fractions should be a minimum of 35%. The fractions of # 8 and # 16 can be split in the proportion of 3:2 to 2:1.

3. <u>1/2" and 3/8" Sieve Size</u>: It is recommended to use relatively higher fractions of 12.5mm and 9.5mm fractions in the coarser part of coarse aggregates. By increasing these sizes of aggregates, the percent fraction of #4 reduces. A presence of larger particle size would plug the continuity of voids efficiently.

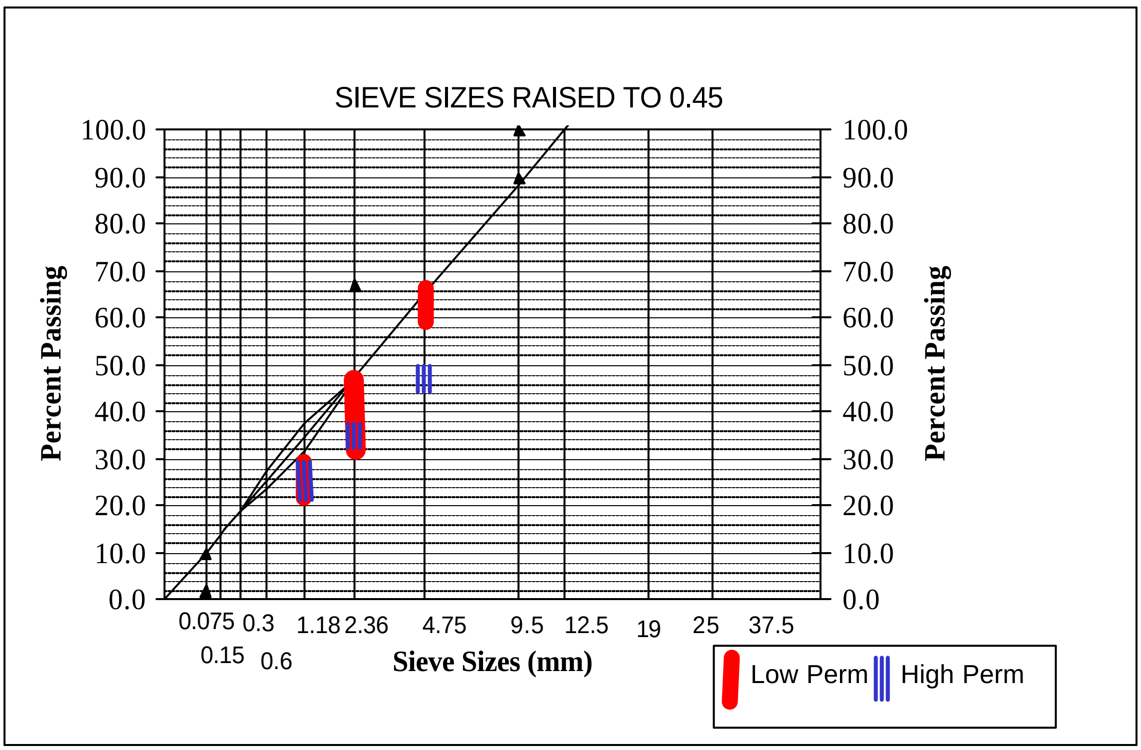

Figure 3.7 Recommended Gradation Bands for 9.5mm Mixtures

3.5 Validation of Guidelines

The proposed guidelines for selecting gradations were validated with new set of gradations. A total of 10 gradations, five each for 12.5mm and 9.5mm mixtures, were selected. The gradations were selected in such a way that two gradations were from low permeability zone, two from high permeability zone and one in-between the high and low permeable zones. The mixtures were designated like 12.5VL1, 9.5VH2, 12.5VM where 12.5/9.5 is the nominal size, V is for validation and L/H/M is for low/high/intermediate zone and 1or 2 is the number assigned to the gradation. Tables 3.12 and 3.13 provide the aggregate gradations for 12.5mm and 9.5mm mixtures, respectively. Permeability tests were conducted on the specimens of these gradations at 8.5% air voids. The results of permeability tests are provided in Table 3.14. The permeability coefficients are compared in Figures 3.8 and 3.9 for 12.5mm and 9.5mm mixtures, respectively.

The permeability test results validate the guidelines developed for selecting aggregate gradations. The results clearly identified the high permeable mixtures viz. 12.5H1, 12.5H2, 9.5H1 and 9.5 H2. These guidelines can be used while designing the aggregate blends. As packing characteristics of different fractions in the spectrum of an aggregate blend is very complex in nature, following the guidelines alone can not completely ensure the desired level of permeability. There might be a few discrepancies but a trial and error procedure by following these guidelines would certainly provide aggregate blends with the desired level of permeability.

Table 3.12 Gradations of 12.5mm Mixtures for Validation

Sieve Size	12.5VL1	12.5VL2	12.5VM	12.5VH1	12.5VH2
19	100	100	100	100	100
12.5	89	90	92	91	92
9.5	80	81	85	83	84
4.75	62	60	55	51	47
2.36	37	35	35	31	31
1.18	25	24	22	22	20
0.6	18	16	15	15	13
0.3	12	11	11	11	10
0.15	7	8	8	8	8
0.075	4	4	3.5	5	6
Pan	0	0	0	0	0

Table 3.13 Gradations of 9.5mm Mixtures for Validation

Sieve Size	9.5VL1	9.5VL2	9.5VM	9.5VH1	9.5VH2
19	100	100	100	100	100
12.5	98	98	98	98	98
9.5	92	91	95	94	96
4.75	66	65	57	51	44
2.36	41	46	40	37	32
1.18	28	30	28	26	22
0.6	20	22	21	20	17
0.3	15	17	14	15	11
0.15	9	12	8	10	7
0.075	4	6	4	5	4
Pan	0	0	0	0	0

Table 3.14 Permeability Test Results

Mixture	Permeability Coefficient (k), cm/sec	Time for 75mm Specimen	
		Minutes	Seconds
12.5H1	6.67E-04	3	33
12.5H2	2.42E-04	9	48
12.5M	9.40E-05	25	12
12.5L1	7.56E-05	31	20
12.5L2	8.66E-05	27	20
9.5H1	5.30E-04	4	28
9.5H2	1.33E-03	1	47
9.5M	2.26E-04	10	30
9.5L1	1.70E-04	13	57
9.5L2	1.50E-04	15	50

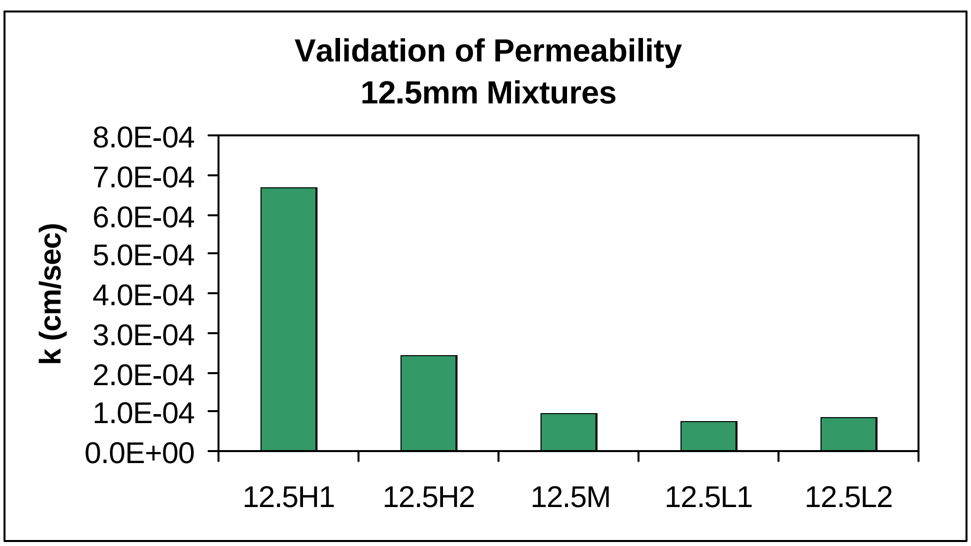

Figure 3.8 Comparison of Permeability Coefficients for 12.5mm Mixtures

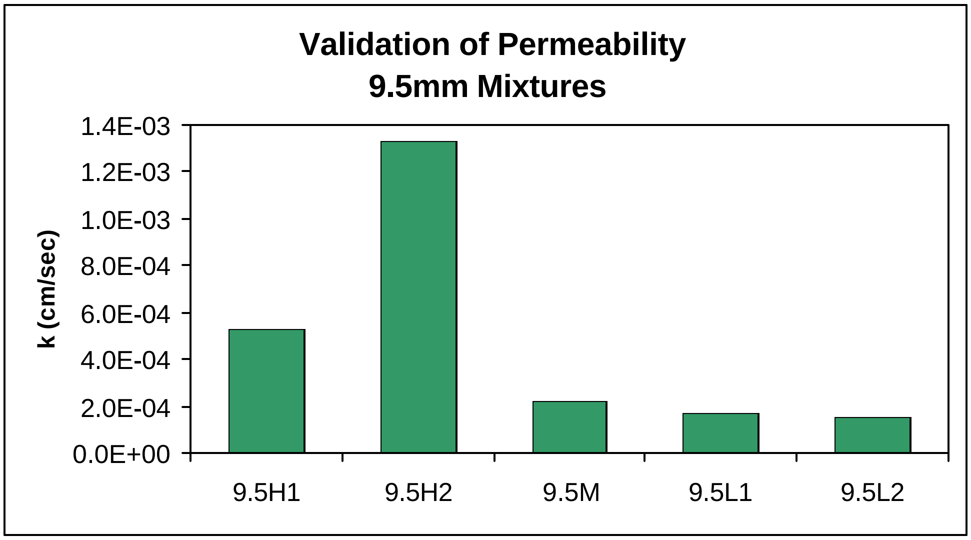

Figure 3.9 Comparison of Permeability Coefficients for 9.5mm Mixtures

CHAPTER 4

ASPHALT MIXTURE DESIGN

In this chapter, the results of Superpave volumetric mixture design for 12.5mm and 9.5mm mixtures are presented. Four new mixtures for each nominal size were designed and the volumetric properties are briefly described in this chapter.

4.1 Field Specimens

The field cores of 12.5mm and 9.5mm mixtures were obtained from the pavement sections of Davison county and Forsyth county, respectively. The pavement section of I-85 in Davidson County was a surface mix with 12.5mm nominal maximum size of aggregates with a gradation passing below the restricted zone. The source of aggregates was from Martin Marietta, Jamestown, NC. The effective specific gravity of the aggregates was 2.708. The grade of asphalt binder used was PG 64-22 with a specific gravity of 1.03. The field cores of 9.5mm mixtures were obtained from Forsyth County, NC. The pavement section was a surface course mix with 9.5mm nominal size of aggregates with Type RS 9.5C. This mixture contained Reclaimed Asphalt Pavement (RAP) material of 15% and an anti-strip additive (Arr-Maz products) of 0.5%. The asphalt binder grade was PG 70-22. Table 4.1 provides the information of the mixture properties.

Table 4.1 Mixture Properties of Field Cores

Properties	Mixture Type	
	12.5mm	9.5mm
% Asphalt Binder	4.9	4.9
G_{mb} @ Ndes	2.419	2.509
Max. Sp.Gr (G_{mm})	2.507	2.614
% Voids – Total Mix (VTM)	3.5	4.0
% Solids – Total Mix	96.5	96.0
% Voids in Mineral Agg. (VMA)	14.4	15.70
% Voids filled with AC (VFA)	75.63	73.60
% Asphalt from RAP Binder	-	0.7
Gyrations	8/100/160	8/100/160

4.2 Design of Asphalt Concrete Mixtures

4.2.1 Unmodified Gradation

The aggregate gradation of the field cores of the mixtures was not changed. The SGC compacted mixtures, which used the same job mix formula of the field cores, are designated as “Unmodified” throughout this study. In the laboratory, the mixtures were designed using the asphalt binder of PG 64-22 and the gyration levels of 8/100/160. The unmodified gradations of the two mixtures are given in Table 4.2. Figures 4.1 and 4.2 give the gradation of the field specimens of 12.5mm as well as 9.5mm nominal sizes, respectively. As the 9.5mm mixtures contained RAP, an analysis was conducted to determine the gradation of the RAP material.

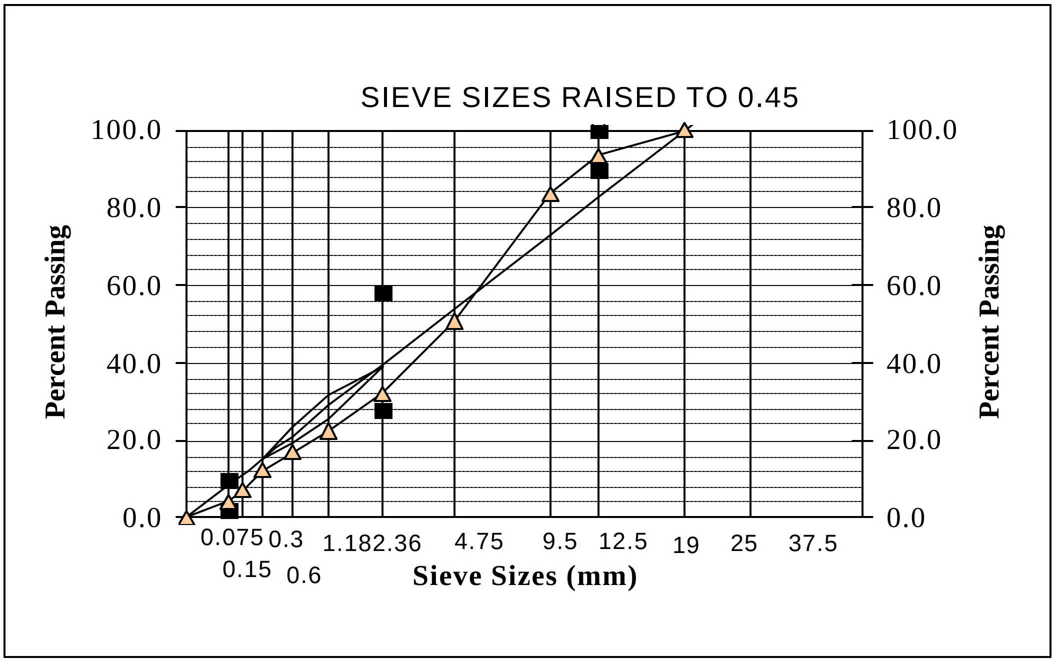

Figure 4.1 Gradation Curve of 12.5mm Mixture

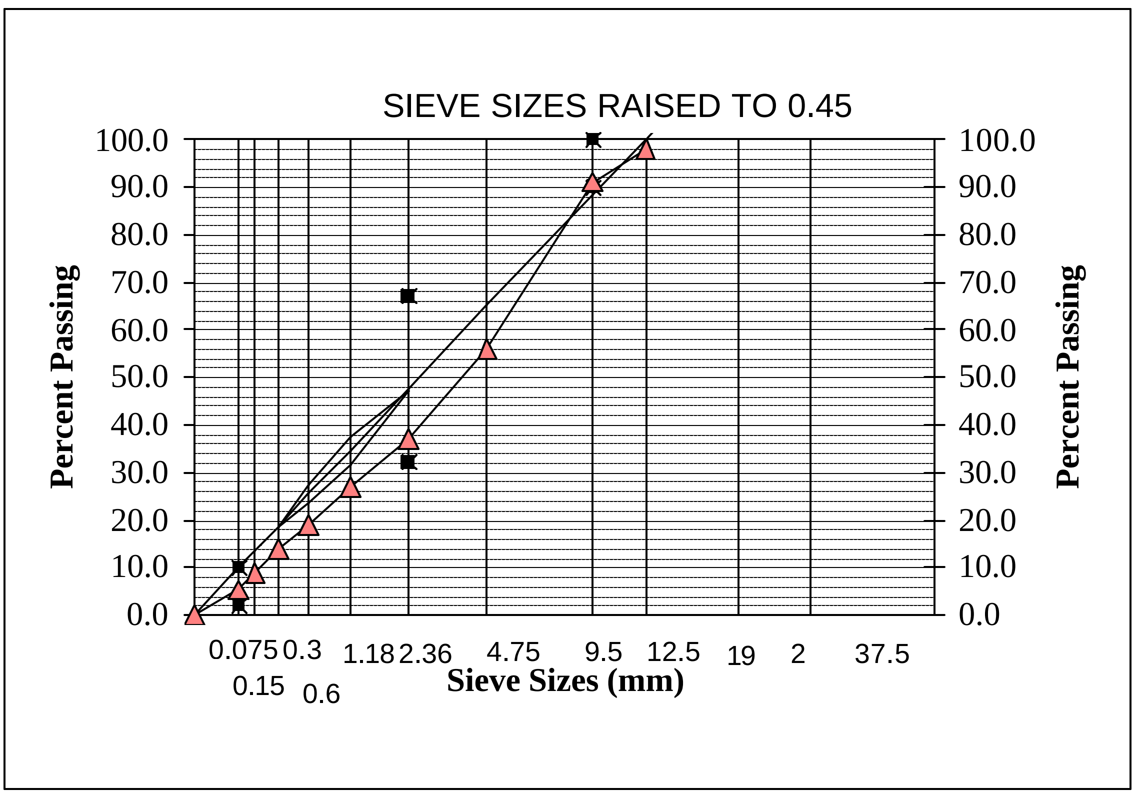

Figure 4.2 Gradation Curve of 9.5mm Mixture

Table 4.2 Unmodified Gradations

Sieve Size	Percent Passing	
	12.5mm	9.5mm
19	100.0	100
12.5	93.6	98
9.5	83.5	91
4.75	50.8	56
2.36	32.2	37
1.18	22.5	27
0.6	16.8	19
0.3	11.9	14
0.15	7.1	9
0.075	4.1	5.3
Pan	0.0	0

<u>RAP Analysis:</u>

The RAP material was analyzed to determine its gradation in accordance with AASHTO TP 53 "Standard Test Method for Determining the Asphalt Content of the HMA using the Ignition Method." In this test procedure, a sample of RAP or asphalt concrete was heated to approximately 538^{o}C and the asphalt cement binder was removed from the sample by ignition and burning. The sample was heated until the sample reached a constant final mass and the asphalt content of the sample was determined from the difference in the initial and final sample masses. In addition to the binder content of the asphalt mixture sample, the ignition method produced a clean aggregate sample that was used to determine aggregate gradation of the RAP material. Figure 4.3 shows an ignition oven used in this study while Figure 4.4 shows the clean aggregate byproduct of the ignition method. The remaining aggregate after burning was subjected to sieve analysis. AASHTO T27-88 "Sieve Analysis of Fine and Coarse Aggregates" and AASTHO T11-

90 “Material Finer Than 75µm (No. 200) Sieve in Mineral Aggregates by Washing” were used to determine the gradation of the aggregate recovered from the RAP samples by ignition. The average of three RAP samples was determined from this analysis. Table 4.3 shows the combined gradation and RAP for each sieve size.

Table 4.3 Combined Gradation and RAP

Sieve Size	Percent Passing	
	Combined	RAP
19	100	100
12.5	98	96.0
9.5	91	87.4
4.75	56	55.5
2.36	37	36.8
1.18	27	26.4
0.6	19	19.9
0.3	14	13.6
0.15	9	8.3
0.075	5.3	4.0
Pan	0	0.0

Figure 4.3 Ignition Oven

Figure 4.4 Aggregate Leftover from Ignition Method

Volumetric Properties:

The mixtures with unmodified gradations for both the 12.5mm and 9.5mm nominal sizes were mixed and compacted. Tables 4.4 and 4.5 provide the volumetric data for 12.5mm mixtures and 9.5mm mixtures, respectively. The PG 64-22 grade asphalt binder was used in the design of both 12.5mm and 9.5mm mixtures. For 9.5mm mixtures, the RAP material contributed 0.7% of asphalt binder.

As mentioned earlier, the unmodified mixtures were mixed and compacted using the information such as the gradation, binder content and temperatures provided in the job-mix formula of the field cores. The mixtures were not designed in the laboratory. The unmodified mixture was used as a reference mixture for comparison with low and high permeable mixtures. The volumetric data provided in Tables 4.4and 4.5 show the air voids of the mixture were low for both 12.5mm and 9.5mm mixtures.

Table 4.4 Volumetric Properties of 12.5mm Mixture with Unmodified Gradation

Mix Parameter	Value
% Asphalt Binder	4.9
$\%G_{mb}$ @ Ndes	96.77
Max. Sp.Gr (G_{mm})	2.507
% Voids – Total Mix (VTM)	3.23
% Voids in Mineral Agg. (VMA)	14.29
% Voids filled with AC (VFA)	72.01
$\%G_{mm}$ @ N_{ini} 8	87.34
$\%G_{mm}$ @ N_{max} 160	97.92

Table 4.5 Volumetric Properties of 9.5mm Mixture with Unmodified Gradation

Mix Parameter	Value
% Asphalt Binder (Total)	4.9
% Asphalt Binder (RAP)	0.7
% Asphalt Binder (Actual)	4.2
$\%G_{mb}$ @ Ndes	97.58
Max. Sp.Gr (G_{mm})	2.614
% Voids – Total Mix (VTM)	2.42
% Voids in Mineral Agg. (VMA)	14.48
% Voids filled with AC (VFA)	72.38
$\%G_{mm}$ @ N_{ini} 8	88.62
$\%G_{mm}$ @ N_{max} 160	98.71

4.2.2 Design of 12.5mm Mixtures

The aggregate gradations L1, L2, H1 and H2 were selected from the trial gradations based on the permeability test results. The gradations were selected in such a way that two mixtures (L1 and L2) have low permeability and two mixtures (H1 and H2) have high permeability. The percent passing of these four gradations on each sieve is summarized in Table 4.6. Figures 4.5 and 4.6 show the gradation curves of low permeable mixtures L1 and L2 and high permeable mixtures H1 and H2, respectively.

Table 4.6 Percent Passing on Each Sieve (12.5mm Mixtures)

Sieve Size	Unmodified	H1	H2	L1	L2
19(3/4)	100.0	100	100	100.0	100.0
12.5(1/2)	93.6	92.0	90.0	89.0	87.0
9.5(3/8)	83.5	81.0	80.0	81.0	79.0
4.75(4)	50.8	56.4	58.0	66.0	64.0
2.36(8)	32.2	29.0	28.0	46.0	44.0
1.18(16)	22.5	22.0	20.0	22.0	23.0
0.6(30)	16.8	15.4	14.0	15.0	16.0
0.3(50)	11.9	11.0	8.0	9.0	10.5
0.15(100)	7.1	7.7	6.0	6.0	7.5
0.075 (200)	4.1	3.5	3.9	4.0	4.0

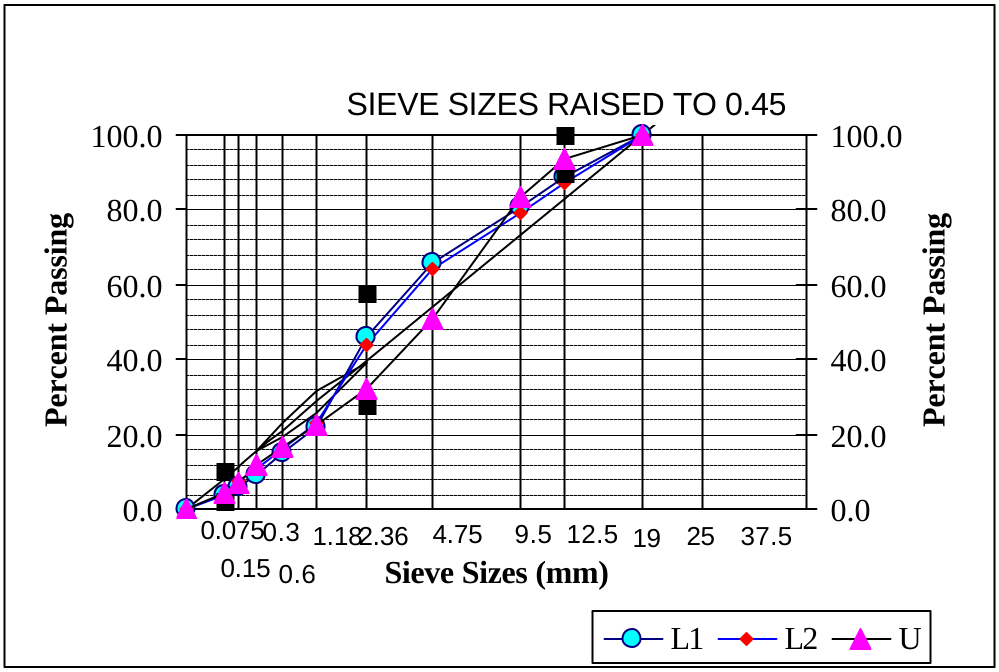

Figure 4.5 Gradation Curves for Low Permeable Mixtures (12.5mm Mixtures)

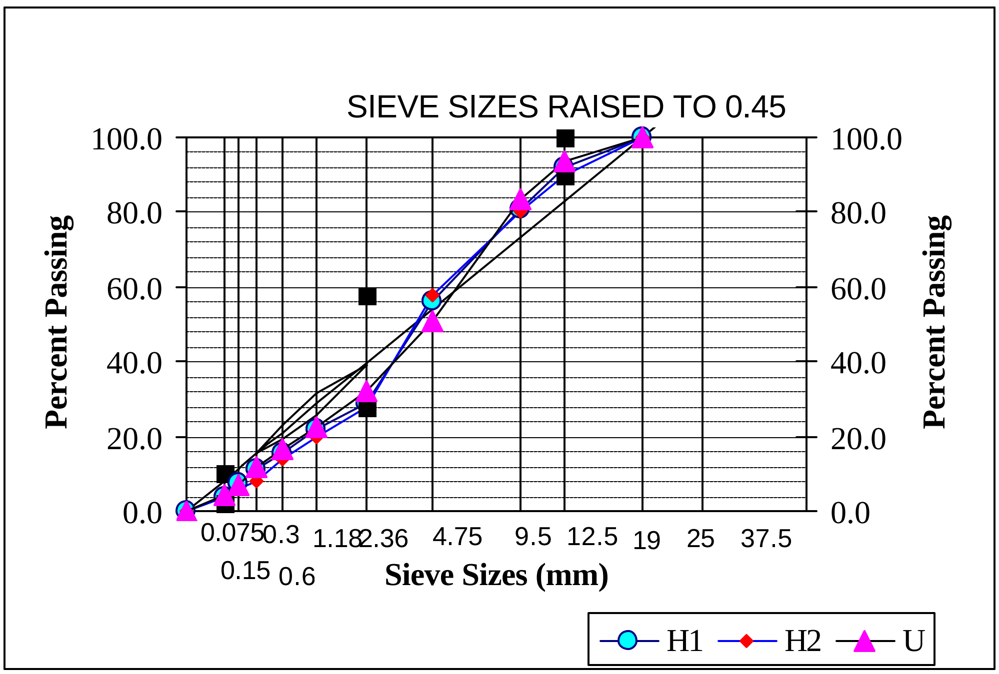

Figure 4.6 Gradation Curves for High Permeable Mixtures (12.5mm Mixtures)

The mixtures of selected gradations were designed in accordance with the Superpave design criteria. The compaction criteria used for the design of mixtures are shown in Table 4.7. The summary of the volumetric properties and densification information for the mixtures is shown in Table 4.8. As is evident from Table 4.8, all the mixtures met the Superpave specifications. The air voids of all the mixtures are within the acceptable range of 3.0 to 5.0%. The VMA values were greater than the minimum criterion of 14.0. The mixtures satisfied the requirements of VFA, percent G_{mm} values at initial and final number of gyrations.

Table 4.7 Compaction Criteria

ESAL Range (millions)	0.3 to 10
Temperature	149^{o}C
Binder	PG 64 -22
Gyrations	8/100/160

Table 4.8 Summary of Mix Design for Selected Gradations (12.5mm Mixtures)

Gradation	Percent Asphalt	Gmm @ Ndes	Percent Air	VMA	VFA	%Gmm @ Nini	%Gmm @ Nmax
H1	4.9	96.2	3.8	15	74.7	86.5	97.5
H2	5.2	96.0	4.0	15.9	74.8	83.8	97.3
L1	5.7	95.8	4.2	16.7	75.8	87.5	97.0
L2	5.2	96.0	4.0	15.6	74.3	88.2	97.4
Criteria			*4.0*	*14.0 Min*	*65-76*	*89.0 Max*	*98.0 Max*

4.2.3 Design of 9.5mm Mixtures

Based on the permeability test results, four aggregate gradations of 9.5mm mixtures L1, L2, H1 and H2 were selected from the trial gradations. The gradations were selected in such a way that two mixtures (L1 and L2) have low permeability and two mixtures (H1 and H2) have high permeability. The percent passing of these four gradations on each sieve is summarized in Table 4.9. Figures 4.7 and 4.8 show the gradation curves of low permeable mixtures and high permeable mixtures, respectively.

Table 4.9 Percent Passing on Each Sieve (9.5mm Mixtures)

Sieve Size	L1	L2	H1	H2
19(3/4)	100	100	100	100
12.5(1/2)	98	98	98	98
9.5(3/8)	94	93	94	95
4.75(4)	70	72	50	51
2.36(8)	50	40	30	29
1.18(16)	25	25	21	19
0.6(30)	15	20	15	12
0.3(50)	11	15	11	9
0.15(100)	8	10	8	7
0.075 (200)	5	5	5	4

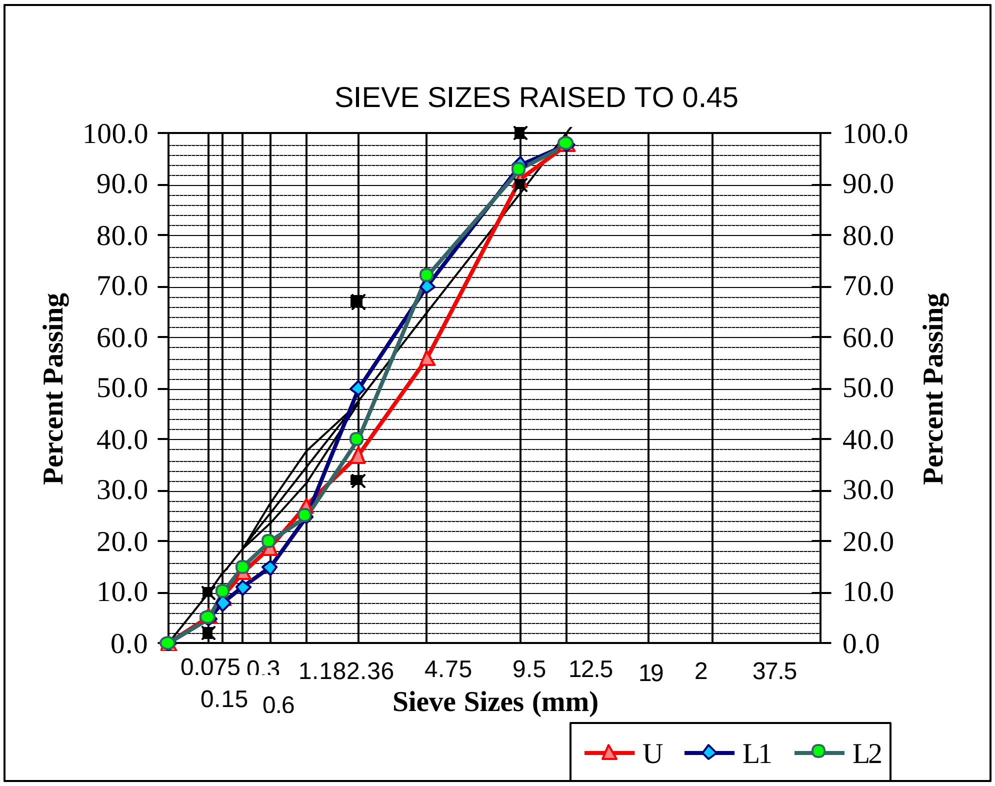

Figure 4.7 Gradation Curves for Low Permeable Mixtures (9.5mm Mixtures)

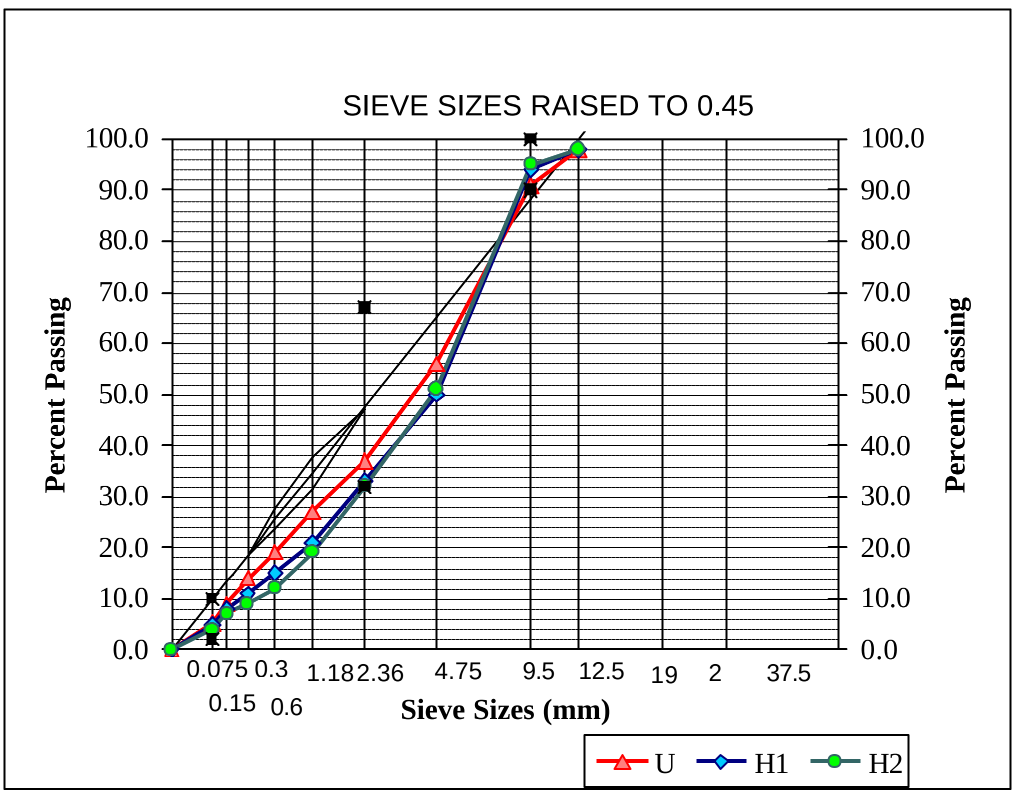

Figure 4.8 Gradation Curves for High Permeable Mixtures (9.5mm Mixtures)

Mix designs for the 9.5mm mixtures were conducted for the selected gradations to meet the Superpave design criteria. The summary of the volumetric properties and densification information for the mixtures is shown in Table 4.10. The mixtures had the air void content of 4% which was within the acceptable range of 3 to 5%. All the mixtures met the Superpave requirements of volumetric properties such as VMA, VFA, and Percent G_{mm} at initial as well as at final number of gyrations.

Table 4.10 Summary of Mix Design for Selected Gradations (9.5mm Mixtures)

Gradation	Percent Asphalt	%Gmm @ Ndes	Percent Air	VMA	VFA	%Gmm @ Nini	%Gmm @ Nmax
L1	5.8	96.0	4.0	17.6	77.2	86.8	97.3
L2	4.9	96.0	4.0	16.2	75.2	86.7	96.7
H1	4.9	96.0	4.0	15.3	73.8	86.3	97.5
H2	5.2	96.0	4.0	16.7	75.8	85.5	97.3

The mixtures designed in the laboratory were evaluated for their performance using the Shear tests and APA rut tests. The results of performance evaluation tests are discussed in detail in the next chapter.

CHAPTER 5

PERFORMANCE EVALUATION OF MIXTURES

In this chapter, the mixtures were characterized for their overall mixture stiffness and shear strain. The effect of permeability was studied on different mixtures before and after moisture damage. The characteristics of mixtures were evaluated using the Simple Shear Tester and the Asphalt Pavement Analyzer.

5.1 Performance Evaluation using the Simple Shear Tester

Shear tests were performed to measure the stiffness and phase angles of mixtures by dynamic Frequency Sweep at Constant Height (FSCH) Test using the Superpave Simple Shear Tester (SST). The SST was also utilized to measure shear strength of each mixture, or what more accurately may be termed as the mixture resistance to plastic shear strain, using the Repeated Shear at Constant Height (RSCH) Test. Both test procedures were conducted in accordance with AASHTO TP7-94 “Test Method for Determining the Permanent Deformation and Fatigue Cracking Characteristics of Hot Mix Asphalt (HMA Using the Simple Shear Test (SST) Device” (18). The tests were conducted on unconditioned as well as conditioned specimens of 12.5mm and 9.5mm mixtures. The results were used to ascertain the damage induced by water on the mixtures of different permeability levels.

<u>Specimen Preparation:</u>

The specimens prepared for shear tests were 150mm (6-in.) in diameter. The specimens were sawed to a thickness of 50 mm (2-in.). The specific gravities of the specimens were measured. The specimens were then glued between the loading platens using ‘DEVCON’ 5-minute plastic putty and were allowed to cure for several hours before testing. All the specimens were compacted using the Superpave Gyratory Compactor at 8.5% air voids. The reason for choosing a high level of air voids is that the effect of permeability on specimens would be more pronounced at 8.5% air voids than at 7% or 4% air voids. The shears were conducted for specimens in unconditioned and conditioned state.

The conditioning of specimens was performed in accordance with AASHTO T-283 procedure. The specimens were partially saturated with water by applying vacuum to around 65-70% degree of saturation. The specimens were immersed for conditioning in a water bath for 24 hours at a temperature of 60^{o}C. Then the specimens were cooled at the room temperature for drying. This procedure was performed to induce moisture damage in the specimens.

Selection of Test Temperature:

In the abridged fatigue analysis (SHRP A-003A) procedure, the pavement temperature is assumed to be 20^{o}C through out the year. The resistance of a mix to fatigue cracking is calculated based on the mix properties evaluated using FSCH at 20^{o}C. The seven-day average high pavement temperature at 50-mm depth from pavement surface at 98% reliability was estimated using SHRPBIND version 2.0 software for our immediate area is 58.9^{o}C.

5.2 Frequency Sweep Test at Constant Height

The Frequency Sweep Test at Constant Height (FSCH test) for asphalt concrete mixtures is performed to measure linear visco-elastic properties of asphalt concrete for rutting and fatigue cracking analysis. The FSCH tests used in this study were conducted in accordance with Procedure E of AASHTO TP7-94 "Standard Test Method for Determining the Permanent Deformation and Fatigue Cracking Characteristics of Hot Mix Asphalts (HMA) Using the Simple Shear Test (SST) Device". This test used a dynamic type of loading and is a strain controlled test with the maximum shear strain limited to ± 0.005 percent (maximum peak to peak of 0.0001 mm/mm). This test is conducted at a constant height requiring the vertical actuator to be controlled by the vertical LVDT. The specimen is preconditioned by applying a sinusoidal horizontal shear strain with amplitude of approximately 0.0001 mm/mm at a frequency of 10 Hz for 100 cycles. After preconditioning the specimen, a series of 10 tests are conducted in descending order of frequency. The following order of frequencies is used: 10, 5, 2, 1, 0.5, 0.2, 0.1, 0.05, 0.02 and 0.01 Hz. A specific number of cycles between 4 and 50 are applied. During the test, axial and shear loads and deformations are measured and recorded.

The FSCH test gives the mixture's stiffness as measured by the complex modulus (G*) over the range of frequencies tested. Asphalt concrete mixtures are thermal visco-elestic materials, meaning that they exhibit predominantly viscous or elastic stress-strain responses depending on loading rate and test temperature. For a given temperature, the complex modulus for a mixture will decrease with a decrease in loading rate or frequency.

5.3 Analysis of FSCH Test Results

Three replicate specimens were used to conduct the FSCH test. Since the FSCH test is a nondestructive test, the replicates could be reused for the destructive RSCH test. This procedure ensured that the average volumetric properties of the test specimens would be identical to those determined during the design process. All mixtures were tested using the FSCH Test at a temperature of 20^{o}C. Although AASHTO requires that the FSCH Test be conducted at 4, 20 and 40^{o}C for a complete SUPERPAVE mixture analysis, the SHRP A-003A performance prediction model used in this research simply requires the mixture's stiffness at 20^{o}C.

5.3.1 FSCH Test Results for 12.5mm Mixtures

The measured dynamic modulus |G*| and phase angle (δ) with respect to loading frequency from the FSCH test for 12.5mm mixtures are shown in Tables 5.1 to 5.3. Figures 5.1 and 5.2 show the results of mixture stiffness for all the mixtures, both unconditioned and conditioned specimens. The results show that as loading frequency increases, the stiffness of the mixtures increases. Figures 5.3 and 5.4 show the change of phase angles for unconditioned as well as conditioned specimens of all the mixtures. This behavior was anticipated due to the widely accepted theory of an asphaltic material's visco-elastic response under loading. The visco-elastic theory is also manifested in the measured phase angle of the mixtures shown in Figures 5.3 and 5.4. As the load frequency increases, the phase angle (time differential between applied load and measured strain response) generally decreases as the elastic component (G') of the mixture's stiffness becomes more predominant in the load response of the material.

Table 5.1 Frequency Sweep at Constant Height Test Results (Field Cores and Unmodified)

Freq. (Hz.)	Average G* (Pa)/Phase Angle(Deg)			
	FC-UC	FC-C	UM-UC	UM-C
0.01	1.51E+08/33.36	6.03E+07/39.67	1.03E+08/42.95	4.57E+07/36.26
0.02	1.89E+08/33.81	7.63E+07/39.91	1.39E+08/43.78	6.07E+07/37.14
0.05	2.56E+08/33.16	1.04E+08/39.24	2.05E+08/43.03	8.84E+07/38.45
0.1	3.20E+08/33.51	1.32E+08/39.20	2.75E+08/42.88	1.17E+08/40.15
0.2	4.02E+08/31.83	1.67E+08/38.74	3.69E+08/41.69	1.56E+08/40.83
0.5	5.42E+08/29.74	2.27E+08/37.34	5.45E+08/39.02	2.27E+08/41.61
1	6.80E+08/27.64	2.87E+08/35.96	7.32E+08/35.01	3.02E+08/40.87
2	8.52E+08/23.35	3.64E+08/33.56	9.83E+08/31.87	4.01E+08/38.72
5	1.15E+09/23.15	4.96E+08/31.83	1.45E+09/25.78	5.83E+08/37.11
10	1.44E+09/17.52	6.21E+08/29.82	1.95E+09/20.90	7.75E+08/34.84

Table 5.2 FSCH Test Results (Low Permeable Mixtures)

Freq. (Hz.)	Average G* (Pa)/Phase Angle(Deg)			
	L1-UC	L1-C	L2-UC	L2-C
0.01	7.18E+07/42.40	2.71E+07/43.06	7.51E+07/42.74	4.83E+07/42.14
0.02	9.67E+07/42.96	3.72E+07/45.01	1.07E+08/41.75	6.75E+07/43.93
0.05	1.43E+08/42.65	5.66E+07/46.11	1.69E+08/42.50	1.05E+08/44.30
0.1	1.93E+08/42.36	7.76E+07/48.27	2.40E+08/42.73	1.47E+08/44.38
0.2	2.60E+08/41.19	1.07E+08/49.01	3.40E+08/42.40	2.05E+08/44.77
0.5	3.85E+08/38.37	1.62E+08/47.52	5.41E+08/41.29	3.20E+08/42.39
1	5.18E+08/34.97	2.22E+08/48.53	7.67E+08/39.47	4.47E+08/39.57
2	6.97E+08/29.46	3.05E+08/44.12	1.09E+09/31.93	6.25E+08/38.09
5	1.03E+09/26.54	4.63E+08/42.16	1.73E+09/28.91	9.73E+08/34.00
10	1.39E+09/22.91	6.35E+08/43.06	2.45E+09/21.60	1.36E+09/28.88

Table 5.3 FSCH Test Results (High Permeable Mixtures)

Freq. (Hz.)	Average G* (Pa)/Phase Angle(Deg)			
	H1-UC	**H1-C**	**H2-UC**	**H2-C**
0.01	1.34E+08/44.73	4.26E+07/44.59	6.82E+07/41.74	1.11E+08/42.60
0.02	1.81E+08/45.25	5.99E+07/45.18	9.86E+07/42.22	1.47E+08/42.67
0.05	2.71E+08/44.95	9.41E+07/46.59	1.60E+08/41.54	2.12E+08/43.73
0.1	3.67E+08/44.84	1.32E+08/47.90	2.32E+08/41.95	2.80E+08/44.89
0.2	4.98E+08/43.80	1.86E+08/47.48	3.35E+08/40.21	3.70E+08/45.49
0.5	7.44E+08/41.29	2.93E+08/47.23	5.45E+08/38.36	5.34E+08/44.16
1	1.01E+09/36.68	4.12E+08/46.03	7.88E+08/35.87	7.06E+08/43.28
2	1.37E+09/31.28	5.79E+08/41.55	1.14E+09/29.92	9.32E+08/41.80
5	2.04E+09/27.08	9.10E+08/40.39	1.85E+09/28.82	1.35E+09/37.68
10	2.77E+09/22.63	1.28E+09/36.42	2.68E+09/23.80	1.78E+09/34.21

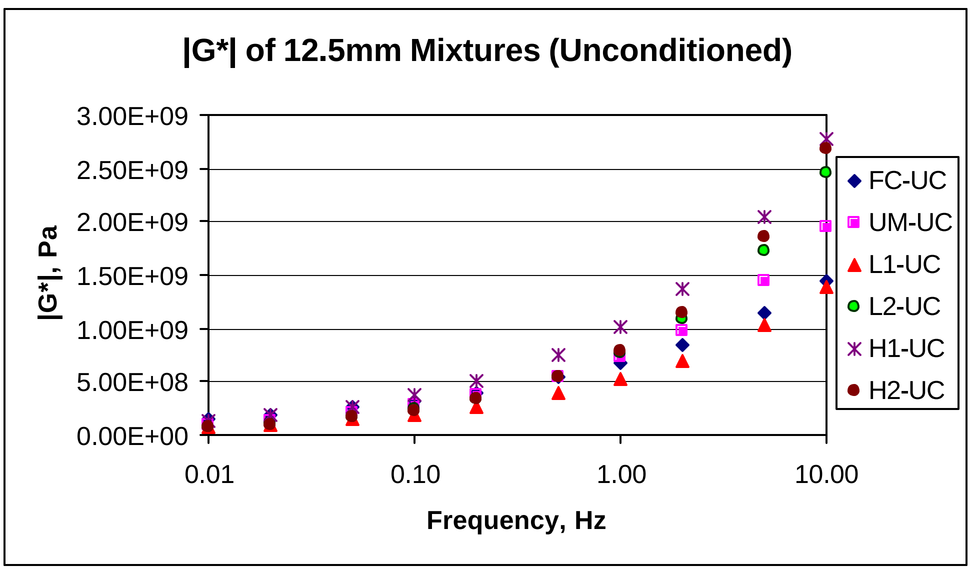

Figure 5.1 Dynamic Modulus vs Frequency (Unconditioned Specimens)

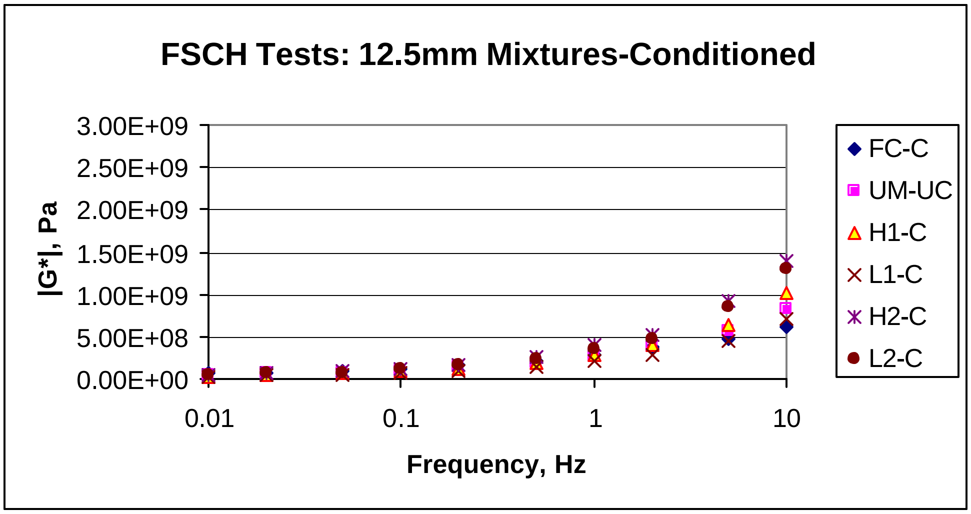

Figure 5.2 Dynamic Modulus vs Frequency (Conditioned Specimens)

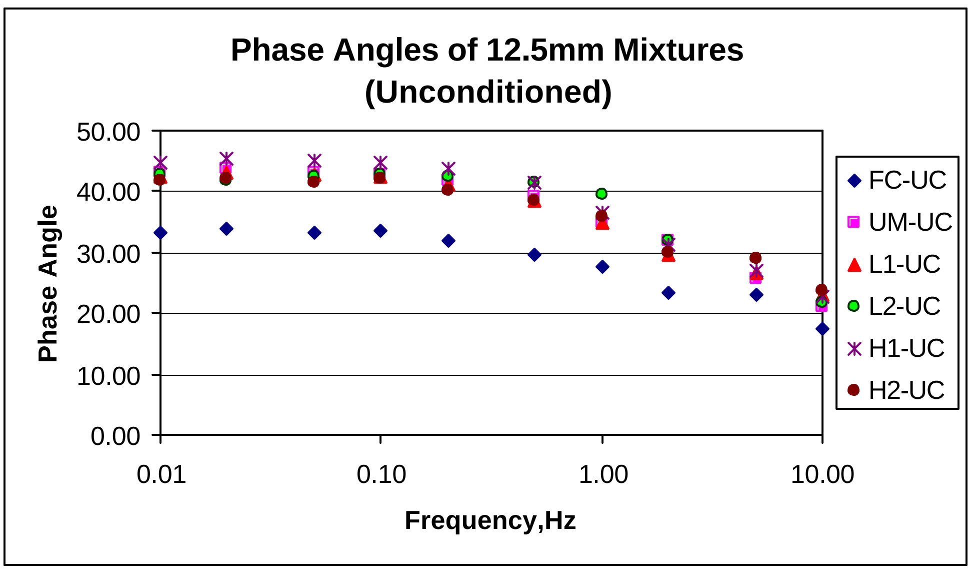

Figure 5.3 Phase Angle vs Frequency (Unconditioned Specimens)

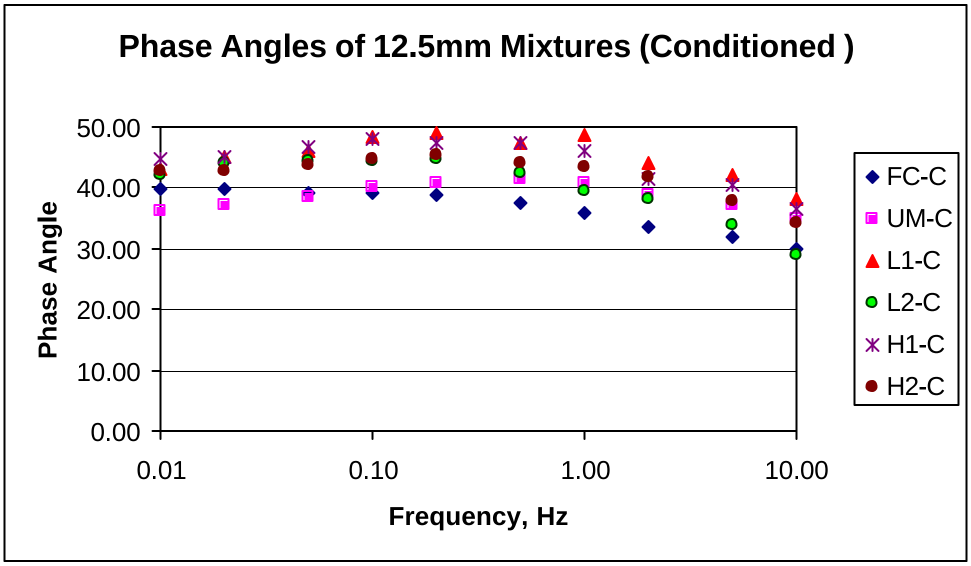

Figure 5.4 Phase Angle vs Frequency (Conditioned Specimens)

For the unconditioned mixtures, the results indicate that three mixtures H1, H2 and L2 have higher stiffness than the other mixtures. The mixture L1 and field specimens have lower stiffness than the other mixtures. The field cores have lower stiffness than the SGC specimens. The variability between the stiffness values of H1 and H2 is smaller than compared to the variability between that of L1 and L2. The stiffness of L1 is at least 45% of the stiffness of L2. The difference in the stiffness values between the two mixtures of low permeability indicates that the stiffness of the mixture does not depend on the permeability of the specimen.

When the specimen is subjected to moisture damage in accordance with AASHTO T-283, the stiffness of the mixture reduces. The stiffness values of undamaged specimens are higher than the stiffness values of moisture damaged specimens. After moisture damage, the same set of mixtures H1, H2 and L2 have higher stiffness than the other mixtures.

The results also show that there is no clear differentiation between the phase angles of low and high permeability. But the average values of phase angles of high permeable mixtures are little higher than the phase angles of low permeable mixtures. This trend is found in both unconditioned and conditioned specimens. Figure 5.5 and Table 5.4 summarize the dynamic modulus of both unconditioned and conditioned specimens. The percent reduction of stiffness due to moisture induced damage is almost around 50%, ranging from 33% to 60%. The presence of moisture causes the loss of bonding between the aggregate surface and the asphalt binder and thereby reduces the stiffness of the mixture.

Table 5.4 Comparison of |G*| @ 10Hz of 12.5mm Mixtures

Mixture	|G*| @ 10Hz, Pa		% Reduction
	Unconditioned	Conditioned	
FC	1.44E+09	6.21E+08	56.9
UM	1.95E+09	7.75E+08	60.3
L1	1.39E+09	6.35E+08	54.3
L2	2.45E+09	1.36E+09	44.5
H1	2.77E+09	1.28E+09	53.8
H2	2.68E+09	1.78E+09	33.6

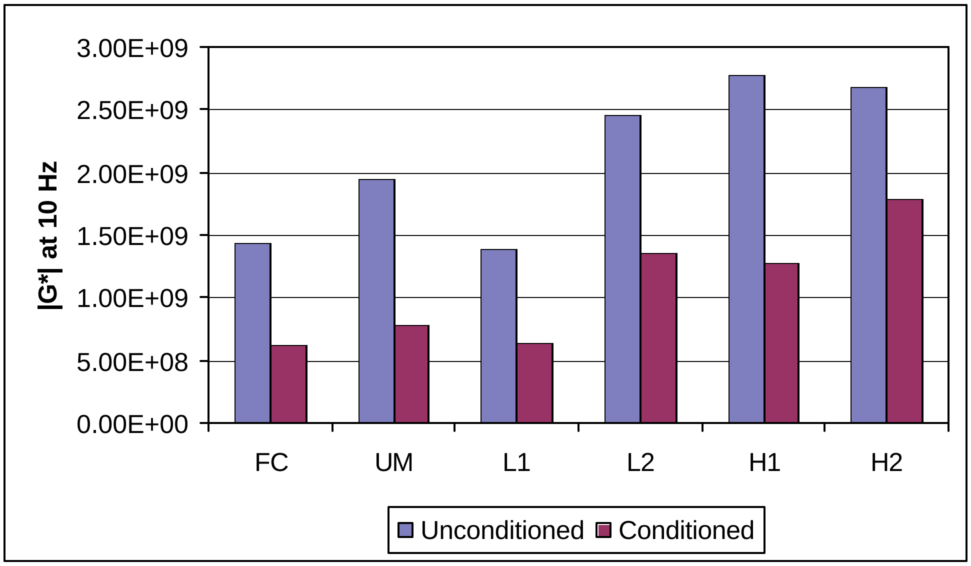

Figure 5.5 Comparison of Dynamic Modulus at 10 Hz of 12.5mm Mixtures

The fatigue life of a mixture depends on the stiffness of mixture at 10Hz, the corresponding phase angle and the VFA content. In the SHRP model, the fatigue life of the mixture is estimated by the combination of these three parameters, (G*| at 10Hz, δ at 10 Hz and VFA. Though the individual values of stiffness and phase angles give unclear trend with respect to permeability, the estimated fatigue life using these parameters in the SHRP fatigue model provides a clear trend as discussed in the next chapter.

Statistical Variability:

The average (μ) stiffness values for all the mixtures are summarized in Table 5.5. The standard deviation (σ) of the stiffness values were calculated. The percentage of σ/μ is a measure of scatter of data from its average. The results of statistical variability are summarized in Table 5.5. The results show that the variability in data ranges from 1.6%

to 16.1% for unconditioned specimens and from 3.1% to 21.8% for conditioned specimens.

Table 5.5 Statistical Analysis of |G*| at 10 Hz for 12.5mm Mixtures

Mixture	Unconditioned			Conditioned		
	μ	σ	σ/μ*100	μ	σ	σ/μ*100
FC	1.44E+09	1.55E+08	10.8	6.21E+08	1.91E+07	3.1
UM	1.95E+09	1.25E+08	6.4	7.75E+08	7.49E+07	9.7
L1	1.39E+09	2.23E+08	16.1	6.35E+08	7.71E+07	12.1
L2	2.45E+09	4.58E+07	1.9	1.36E+09	2.72E+08	20.0
H1	2.77E+09	4.36E+07	1.6	1.28E+09	2.79E+08	21.8
H2	2.68E+09	4.09E+08	15.2	1.78E+09	2.69E+08	15.1

Analysis of Variance:

Analysis of Variance (ANOVA) tests were performed on the dynamic modulus values of 12.5mm mixtures. The tests were performed at 95% confidence level. It was hypothesized that there was no significant difference among the stiffness values of the six mixtures. The rejection of hypothesis indicates that there exists a statistically significant difference among the stiffness values of the mixtures. Tables 5.6 and 5.7 furnish the results of ANOVA tests for unconditioned and conditioned specimens. The F_m statistic (measured F) and the limiting value of F at certain degrees of freedom is shown in the tables. The results from the ANOVA table indicate that the F_m statistics are higher than their limiting values. The F_m values were 18.85 and 16.50 for unconditioned and conditioned specimens whereas the limiting F value for 95% confidence level is 3.11.

This indicates that there exists significant difference among the dynamic modulus values at 10 Hz of the 12.5mm mixtures.

Table 5.6 ANOVA Results for 12.5mm Mixtures (Unconditioned)

Parameter	Sum of Squares	df	Mean Sum of Squares	F_m	$F_{5,12}$
Between treatments	5.61E+18	5	1.12E+18	18.85	3.11
Error	7.14E+17	12	5.95E+16		
Total	6.13E+18	17	3.6E+17		

Table 5.7 ANOVA Results for 12.5mm Mixtures (Conditioned)

Parameter	Sum of Squares	df	Mean Sum of Squares	F_m	$F_{5,12}$
Between treatments	3.33E+18	5	6.66E+17	16.50	3.11
Error	4.84E+17	12	4.04E+16		
Total	3.80E+18	17	2.24E+17		

5.3.2 FSCH Test Results for 9.5mm Mixtures

The measured dynamic modulus $|G^*|$ and phase angle (δ) with respect to loading frequency from the FSCH test for 9.5mm mixtures are shown in Table 5.8 to 5.10. Figures 5.6 and 5.7 show the results of stiffness values for all 9.5mm mixtures, both unconditioned and conditioned specimens. It can be observed that as loading frequency increases, the stiffness of the mixtures increases. Figures 5.8 and 5.9 show the change of phase angles with varying frequencies for unconditioned as well as conditioned specimens of all the mixtures.

Table 5.8 Frequency Sweep at Constant Height Test Results (Field Cores and Unmodified)

Freq. (Hz.)	Average G* (Pa)/Phase Angle(Deg)			
	FC-UC	FC-C	UM-UC	UM-C
0.01	1.17E+08/42.86	7.44E+07/44.59	2.12E+08/38.47	1.21E+08/39.86
0.02	1.55E+08/42.88	9.50E+07/43.95	2.75E+08/38.83	1.57E+08/38.20
0.05	2.13E+08/41.73	1.32E+08/42.10	3.75E+08/37.60	2.13E+08/37.77
0.1	3.15E+08/40.73	2.83E+08/41.96	5.34E+08/37.02	2.99E+08/37.22
0.2	4.42E+08/39.50	3.84E+08/41.75	7.08E+08/35.05	3.95E+08/36.66
0.5	7.26E+08/37.16	5.81E+08/34.43	1.02E+09/31.61	5.84E+08/35.05
1	1.24E+09/33.51	7.73E+08/30.86	1.37E+09/27.33	7.92E+08/30.74
2	1.25E+09/25.15	1.10E+09/26.50	1.62E+09/22.22	1.03E+09/25.57
5	2.37E+09/22.20	1.41E+09/24.89	2.18E+09/19.15	1.44E+09/23.87
10	3.10E+09/19.23	2.23E+09/23.48	2.50E+09/15.34	1.84E+09/19.20

Table 5.9 FSCH Test Results (Low Permeable Mixtures)

Freq. (Hz.)	Average G* (Pa)/Phase Angle(Deg)			
	L1-UC	L1-C	L2-UC	L2-C
0.01	1.06E+08/37.14	1.47E+08/35.88	2.55E+08/33.67	2.13E+08/39.16
0.02	1.39E+08/37.36	1.82E+08/35.61	3.06E+08/33.40	2.22E+08/38.90
0.05	1.92E+08/36.76	2.36E+08/34.71	3.80E+08/31.71	2.79E+08/35.57
0.1	2.75E+08/36.27	3.20E+08/34.99	5.06E+08/32.08	4.44E+08/35.84
0.2	3.70E+08/35.70	4.14E+08/34.42	6.42E+08/30.31	5.86E+08/36.43
0.5	5.56E+08/34.32	5.83E+08/33.21	9.00E+08/28.92	8.91E+08/34.25
1	7.60E+08/31.82	7.53E+08/30.80	1.17E+09/25.85	1.27E+09/28.89
2	9.21E+08/26.48	9.42E+08/27.76	1.29E+09/22.17	1.35E+09/20.95
5	1.32E+09/24.09	1.22E+09/24.15	1.83E+09/18.05	1.66E+09/18.22
10	1.62E+09/18.73	1.47E+09/20.63	2.16E+09/14.40	1.90E+09/16.52

Table 5.10 FSCH Test Results (High Permeable Mixtures)

Freq. (Hz.)	Average G* (Pa)/Phase Angle(Deg)			
	H1-UC	H1-C	H2-UC	H2-C
0.01	1.22E+08/36.91	1.45E+08/38.03	1.24E+08/38.08	9.38E+07/40.70
0.02	1.65E+08/36.62	1.81E+08/38.36	1.59E+08/37.73	1.19E+08/40.71
0.05	3.04E+08/36.31	2.36E+08/37.03	2.14E+08/36.97	1.56E+08/40.19
0.1	3.75E+08/36.78	3.26E+08/37.46	3.08E+08/36.08	2.19E+08/40.15
0.2	4.96E+08/35.80	4.31E+08/36.88	4.07E+08/35.23	2.89E+08/39.40
0.5	6.80E+08/32.62	6.37E+08/34.83	6.26E+08/33.57	4.22E+08/37.35
1	9.34E+08/28.43	8.60E+08/32.01	8.42E+08/30.88	5.67E+08/35.17
2	1.11E+09/24.42	1.02E+09/28.50	9.97E+08/26.81	7.27E+08/32.63
5	1.83E+09/22.85	1.51E+09/26.98	1.38E+09/23.89	9.92E+08/31.03
10	2.47E+09/19.23	1.87E+09/24.68	1.74E+09/21.11	1.21E+09/28.78

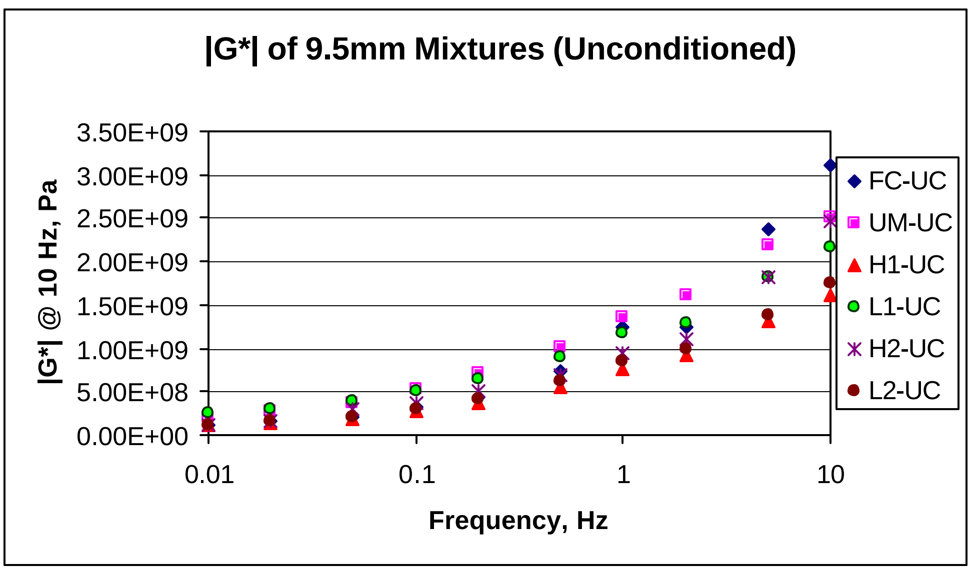

Figure 5.6 Dynamic Modulus vs Frequency (Unconditioned Specimens)

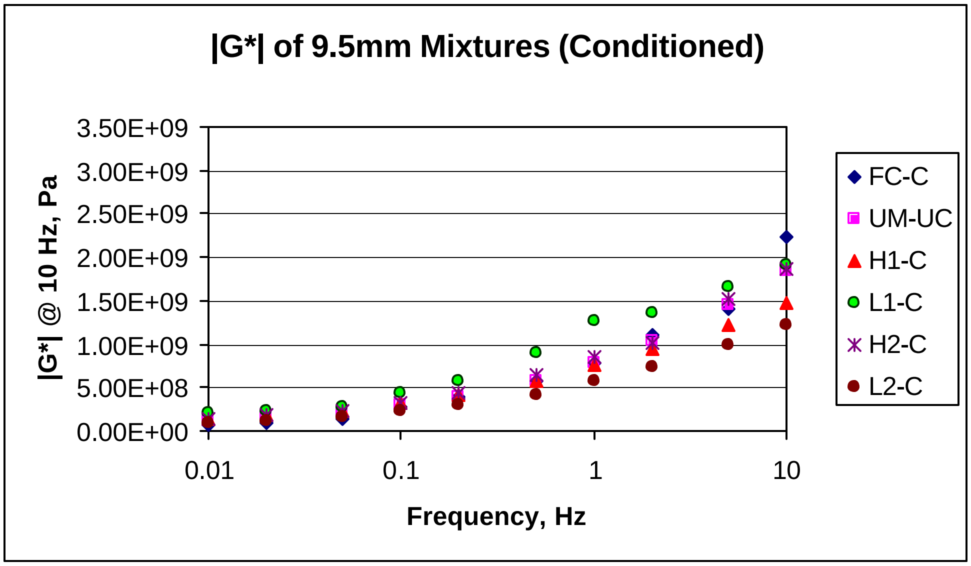

Figure 5.7 Dynamic Modulus vs Frequency (Conditioned Specimens)

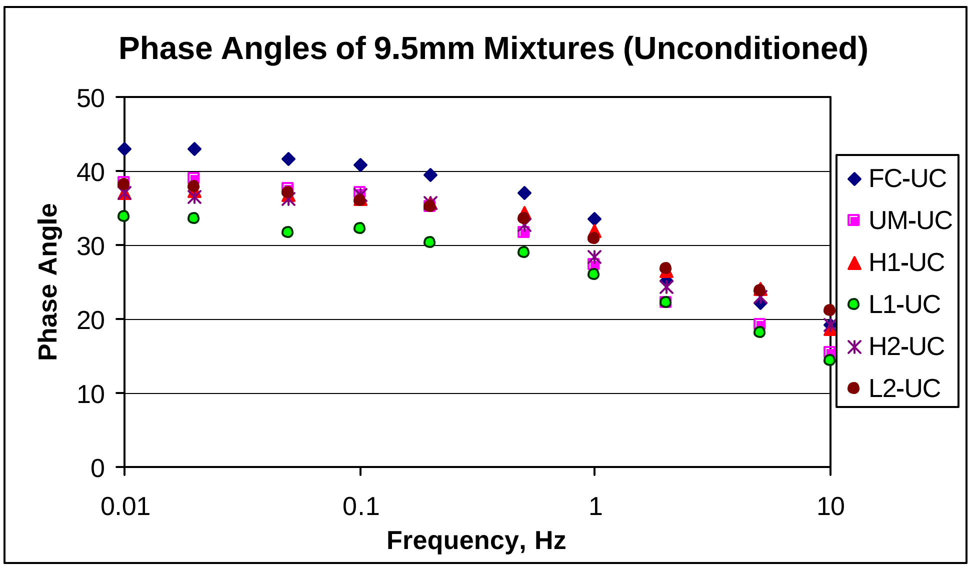

Figure 5.8 Phase Angle vs Frequency (Unconditioned Specimens)

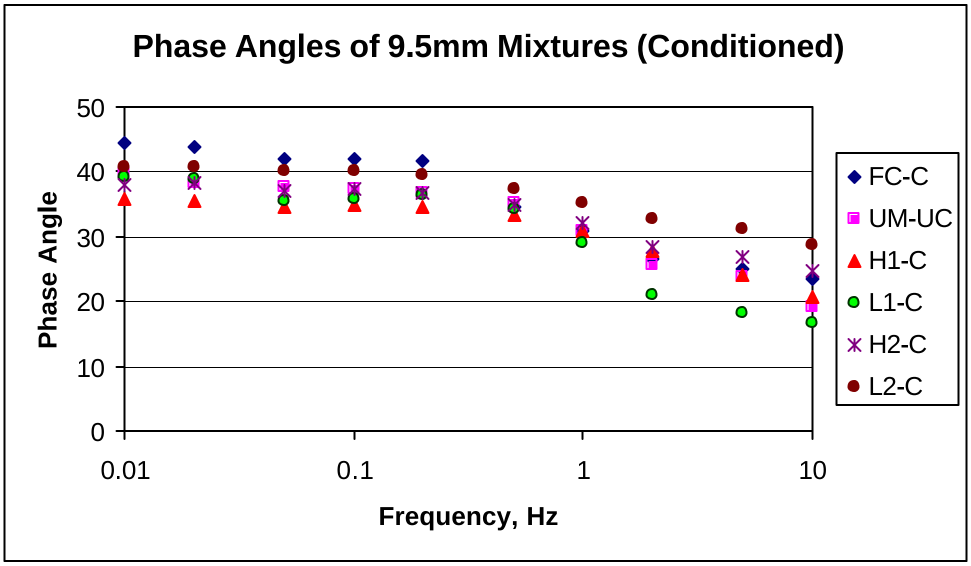

Figure 5.9 Phase Angle vs Frequency (Conditioned Specimens)

Figure 5.10 and Table 5.11 summarize the dynamic modulus of both unconditioned and conditioned specimens. The results show that the field cores have higher stiffness than the other mixtures. The mixtures L1 and H2 have lower stiffness than the mixtures L2 and H1. The mixtures H1 and UM have almost same stiffness values for unconditioned specimens. This trend is found in both unconditioned and conditioned specimens. Even though there is no clear trend observed in the phase angles of the mixtures but the phase angles of high permeable mixtures are little higher than the phase angles of low permeable mixtures. A reduction in the stiffness of mixtures is observed after the conditioning of mixtures. The conditioned specimens have higher phase angles than the unconditioned specimens. The mixtures with low permeability have lower reduction in stiffness around 10% compared to other mixtures. The other mixtures have reduced stiffness of around 25% due to moisture damage. The water induced during the conditioning process causes the damage in the form of stripping of aggregate surfaces and asphalt binder and hence the reduction in stiffness.

Table 5.11 Comparison of |G*| @ 10Hz of 9.5mm Mixtures

Mixture	|G*| @ 10Hz, Pa		% Reduction
	Unconditioned	Conditioned	
FC	3.10E+09	2.23E+09	28.1
UM	2.50E+09	1.84E+09	26.4
L1	1.62E+09	1.47E+09	9.3
L2	2.16E+09	1.90E+09	12.0
H1	2.47E+09	1.87E+09	24.3
H2	1.74E+09	1.21E+09	30.5

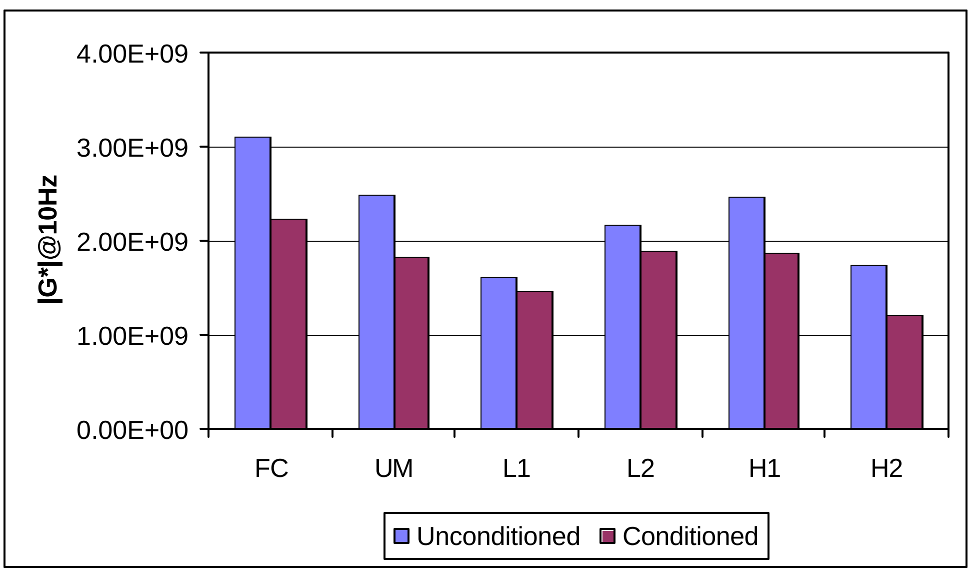

Figure 5.10 Comparison of Dynamic Modulus at 10 Hz of 9.5mm Mixtures

Statistical Variability:

The average (μ) stiffness values for all the 9.5mm mixtures are summarized in Table 5.12. The standard deviation (σ) of the stiffness values were also calculated. The percentage of σ/μ is a measure for determining the variability in the data. The results of statistical variability are summarized in Table 5.12. The results show that the variability in data ranges from around 10% to 20% for unconditioned specimens and from 5.5% to 18.1% for conditioned specimens.

Analysis of Variance:

Analysis of Variance (ANOVA) tests were also performed on the dynamic modulus values of 9.5mm mixtures. The tests were performed at the same 95% confidence level. It was hypothesized that there was no significant difference among the stiffness values of all the mixtures. The rejection of hypothesis indicates that there exists a statistically significant difference among the stiffness values of the mixtures. Tables 5.13 and 5.14 furnish the results of ANOVA tests for unconditioned and conditioned specimens. The F_m statistics and the limiting value of F at certain degrees of freedom are shown in the tables.

The ANOVA results indicate that the F_m statistics are higher than their limiting values. The F_m values were 17.16 and 5.47 for unconditioned and conditioned specimens whereas the limiting F value for 95% confidence level is 3.11. Thus there exists a significant difference among the dynamic modulus values at 10 Hz of the 9.5mm mixtures.

Table 5.12 Statistical Analysis of |G*| at 10 Hz for 12.5mm Mixtures

Mixture	Unconditioned			Conditioned		
	μ	σ	σ/μ*100	μ	σ	σ/μ*100
FC	3.10E+09	3.06E+08	9.9	2.23E+09	2.63E+08	11.8
UM	2.50E+09	4.94E+08	19.7	1.84E+09	1.18E+08	6.4
L1	1.62E+09	3.16E+08	19.5	1.47E+09	8.08E+07	5.5
L2	2.16E+09	3.82E+08	17.7	1.90E+09	3.08E+08	16.2
H1	2.47E+09	4.10E+08	14.2	1.87E+09	3.39E+08	18.1
H2	1.74E+09	2.80E+08	16.1	1.21E+09	1.18E+08	9.8

Table 5.13 ANOVA Results for 9.5mm Mixtures (Unconditioned)

Parameter	Sum of Squares	df	Mean Sum of Squares	F_m	$F_{5,12}$
Between treatments	5.46E+18	5	1.09E+18	17.16	3.11
Error	7.64E+17	12	6.37E+16		
Total	7.12E+18	17	4.19E+17		

Table 5.14 ANOVA Results for 9.5mm Mixtures (Conditioned)

Parameter	Sum of Squares	df	Mean Sum of Squares	F_m	$F_{5,12}$
Between treatments	1.92E+18	5	3.84E+17	5.47	3.11
Error	8.43E+17	12	7.02E+16		
Total	2.55E+18	17	1.50E+17		

5.4 Repeated Shear Test at Constant Height

This test was performed to measure the accumulating plastic shear strain of the mixtures and there by estimating their rutting potential. The visco-elastic properties of an asphalt mixture at high temperatures are related to its permanent deformation characteristics. The accumulation of plastic shear strain in a mixture under repeated loading can give some indication about the mixture's resistance to permanent deformation. The repeated shear testing at constant height was selected to evaluate the accumulated shear strain and permanent deformation characteristics of the mixtures.

The RSCH test is a stress-controlled test with the feedback to the vertical load actuator from the magnitude of the shear load. The test is conducted at constant height, requiring the vertical actuator to be controlled by the vertical LVDT. The horizontal actuator under control by the shear load cell applies haversine loads. It preconditions the specimen by applying a haversine load corresponding to a 7-kPa shear stress for 100 cycles. The 0.7-second load cycle consists of a 0.1-second shear load followed by 0.6-second rest period. After preconditioning the specimen, it applies a 68 ± 5 kPa haversine shear pulse for 5,000 cycles or until 5% shear strain is reached. This corresponds to a frequency of approximately 1.43 Hz. During the test, axial and shear loads and deformations are measured and recorded. This test was conducted according to AASHTO TP-7 Procedure F (15). RSCH tests were performed on specimens of four mixtures and three compaction methods. The tests were conducted at their respective seven-day average high pavement temperature at 50-mm depth from the pavement surface.

5.5 Analysis of RSCH Test Results

Three replicate specimens were used to conduct the RSCH test. Since the FSCH test is a nondestructive test, the replicates were reused for the destructive RSCH test. All mixtures were tested using the RSCH test at a temperature of 58.4°C. The RSCH tests were conducted either for 5000 loading cycles or until the maximum shear strain of 5% was reached, whichever was earlier. The permanent shear strain accumulated at the end of 5000 cycles was measured. This shear strain was used for predicting the in-service rut depth using the SHRP Rutting model, as discussed in the next chapter. The results of RSCH tests for both 12.5mm and 9.5mm mixtures are discussed in the following sections.

5.5.1 RSCH Test Results for 12.5mm Mixtures

Table 5.15 furnishes the results of RSCH test results for 12.5mm mixtures. The shear strains for both unconditioned specimens and conditioned specimens are provided. In Figures 5.11 and 5.12, the change of plastic shear strain with number of loading cycles is plotted for unconditioned and conditioned specimens, respectively. Figure 5.13 shows a bar chart comparing the shear strains of all the mixtures.

All the mixtures passed the RSCH test criterion of 5000 cycles. This indicates that the designed mixtures have good resistance for rutting and are expected have rut depths below the allowable rut depth of 0.5 inch. The results show that the shear strains of mixtures of high permeability of unconditioned specimens are higher than the other mixtures. The mixtures of low permeability have lower shear strains than the field cores and the mixture of unmodified gradation. The results of conditioned specimens show that mixtures of high permeability have higher shear strains. The field cores and the mixture of unmodified gradation have lower shear strains. The mixture L1 has much higher shear strain in conditioned state than in unconditioned state. The shear strains of unconditioned specimens are higher than the shear strains of the conditioned specimens. The increase in shear strain indicates the loss in shear strength of the mixtures due to the damage induced during the conditioning. The increase in shear strain due to conditioning is higher in the mixtures of low permeability than in the mixtures of high permeability. This observation was drawn out from the results of the RSCH tests. The low permeable mixtures have higher moisture damage than high permeable mixtures owing to higher surface area available for stripping. However, low permeable pavement sections may not experience more rutting damage than the high permeable pavement sections.

The moisture damage in specimens is induced in the laboratory in accordance with the AASHTO T-283 procedure. The specimens are saturated to a level of 70% and then conditioned for 24 hours at 60^{o}C. The low permeable mixtures would generally take longer time than the high permeable mixtures to reach any level of saturation. Irrespective of the permeability levels, the specimens are forced to saturate to the same level for both low and high permeable mixtures. In the field, the pavement sections take their natural course of time to saturate to a given level due to the different levels of permeability. The low permeable pavement sections would take longer time than the high permeable pavement sections to reach the same level of saturation that would activate the moisture related damage.

Table 5.15 RSCH Test Results of 12.5mm Mixtures

Mixture	Unconditioned		Conditioned	
	Shear Strain	Cycles	Shear Strain	Cycles
Field Cores	0.0207	5000	0.0308	5000
Unmodified	0.0220	5000	0.0270	5000
Low Perm – 1	0.0132	5000	0.0368	5000
Low Perm – 2	0.0154	5000	0.0292	5000
High Perm – 1	0.0310	5000	0.0339	5000
High Perm - 2	0.0320	5000	0.0381	5000

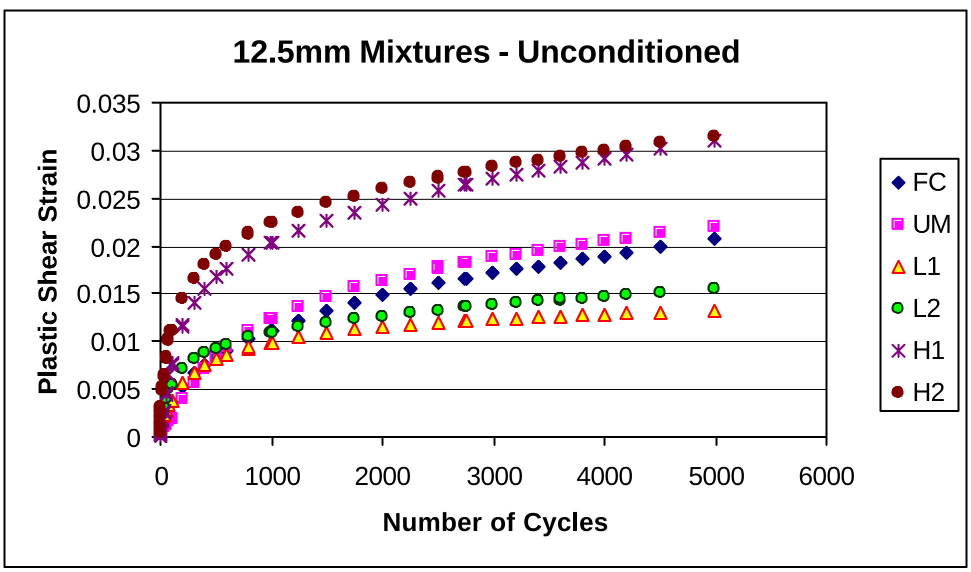

Figure 5.11 RSCH Test Results for 12.5mm Mixtures (Unconditioned)

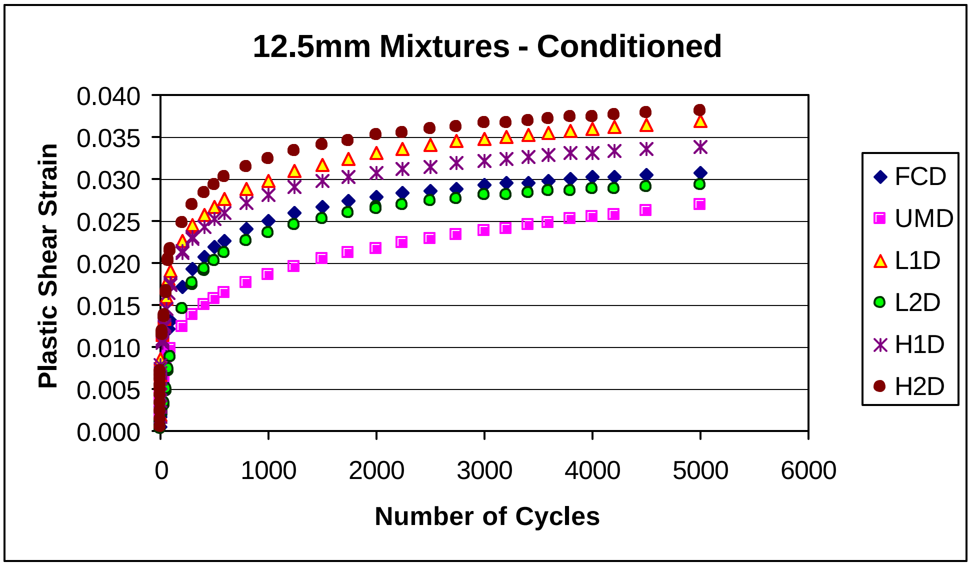

Figure 5.12 RSCH Test Results for 12.5mm Mixtures (Conditioned)

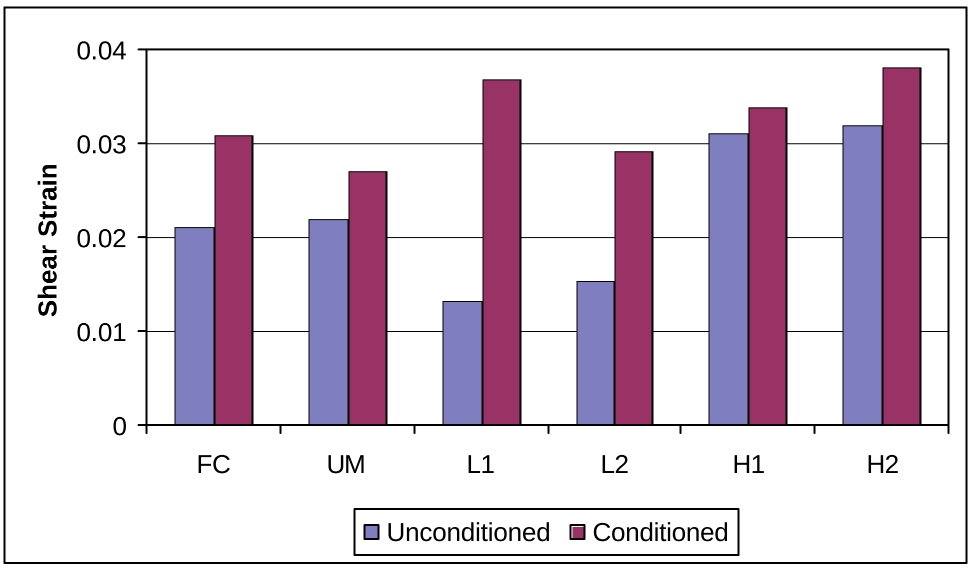

Figure 5.13 Comparison of Shear Strains of 12.5mm Mixtures

Statistical Variability:

The average (μ) shear strains for all the 12.5mm mixtures are summarized in Table 5.16. The standard deviation (σ) of the stiffness values were also calculated. The percentage of σ/μ is a measure of scatter of data from its average. The results of statistical variability are summarized in Table 5.16. The results show that the variability in data is around 10% for unconditioned specimens as well as conditioned specimens.

Analysis of Variance:

Analysis of Variance (ANOVA) tests were also performed on the shear strains of 12.5mm mixtures. The tests were performed at the same 95% confidence level. It was hypothesized that there was no significant difference among the shear strains of the mixtures. The rejection of hypothesis indicates that there exists a statistically significant difference among the stiffness values of the mixtures. Tables 5.17 and 5.18 furnish the results of ANOVA tests for unconditioned and conditioned specimens. The F_m statistics and the limiting value of F at certain degrees of freedom are shown in the tables. The ANOVA results indicate that the F_m statistics are higher than their limiting values. The F_m values were 18.88 and 6.16 for unconditioned and conditioned specimens whereas the limiting F value for 95% confidence level is 3.11. Thus there exists a significant difference among the shear strains of the 12.5mm mixtures. The F_m value of conditioned specimens is lower than that of unconditioned specimens. The above observation indicates that the magnitude of difference in shear strains among the mixtures in unconditioned state decreases compared to the shear strains of the same specimens measured after conditioning. The decrease in F_m is due to the fact that the effect of moisture damage on shear strains of low permeable mixtures is higher than that of high permeable mixtures.

Table 5.16 Statistical Analysis of Shear Strain for 12.5mm Mixtures

Mixture	Unconditioned			Conditioned		
	μ	σ	μ/σ*100	μ	σ	μ/σ*100
FC	0.021	2.43E-03	11.6	0.0308	1.93E-03	6.3
UM	0.022	1.74E-03	7.9	0.0270	1.47E-03	5.5
L1	0.0132	1.01E-03	7.7	0.0368	3.16E-03	8.5
L2	0.0154	1.14E-03	7.4	0.0292	6.81E-04	2.3
H1	0.031	2.32E-03	7.6	0.0339	3.61E-04	1.1
H2	0.032	3.84E-03	12.0	0.038	3.61E-03	9.6

Table 5.17 ANOVA Results for 12.5mm Mixtures (Unconditioned)

Parameter	Sum of Squares	df	Mean Sum of Squares	F_m	$F_{5,12}$
Between treatments	8.87E-04	5	1.77E-04	18.88	3.11
Error	1.13E-04	12	9.40E-06		
Total	9.50E-04	17	5.59E-05		

Table 5.18 ANOVA Results for 12.5mm Mixtures (Conditioned)

Parameter	Sum of Squares	df	Mean Sum of Squares	F_m	$F_{5,12}$
Between treatments	2.78E-04	5	5.55E-04	6.16	3.11
Error	1.08E-04	12	9.01E-06		
Total	3.37E-04	17	1.98E-05		

5.5.2 RSCH Test Results for 9.5mm Mixtures

Table 5.19 provides the shear strains of 9.5mm mixtures in unconditioned as well as conditioned states. In Figures 5.14 and 5.15, the change of plastic shear strain with number of loading cycles is plotted for unconditioned and conditioned specimens, respectively. A comparison of shear strains is shown in Figure 5.16.

The results of 9.5mm mixture indicate that the shear strains of mixtures with high permeability of unconditioned specimens are higher than the other mixtures. The mixtures of low permeability have lower shear strains than the field cores and the mixture of unmodified gradation. The results of conditioned specimens show that mixtures of high permeability have higher shear strains than other mixtures. The mixtures L1 and L2 have lower shear strains than other mixtures in conditioned state too. The field cores and the mixture of unmodified gradation have shear strains in between low permeable and high permeable mixtures.

The shear strains of conditioned specimens are higher than the shear strains of the unconditioned specimens. The increase in shear strain indicates the loss in shear strength of the mixtures due to the damage induced during the conditioning. The increase in shear strain due to conditioning is higher in the mixtures of low permeability than in the mixtures of high permeability. In spite of a greater increase in shear strain as observed in low permeable mixtures due to conditioning, the shear strains of mixtures L1 and L2 are lower than the mixtures H1 and H2.

Table 5.19 RSCH Test Results of 9.5mm Mixtures

Mixture	Unconditioned		Conditioned	
	Shear Strain	Cycles	Shear Strain	Cycles
Field Cores	0.0200	5000	0.0366	5000
Unmodified	0.0211	5000	0.0277	5000
Low Perm 1	0.0164	5000	0.0225	5000
Low Perm 2	0.0126	5000	0.0211	5000
High Perm 1	0.0256	5000	0.0332	5000
High Perm 2	0.0241	5000	0.0308	5000

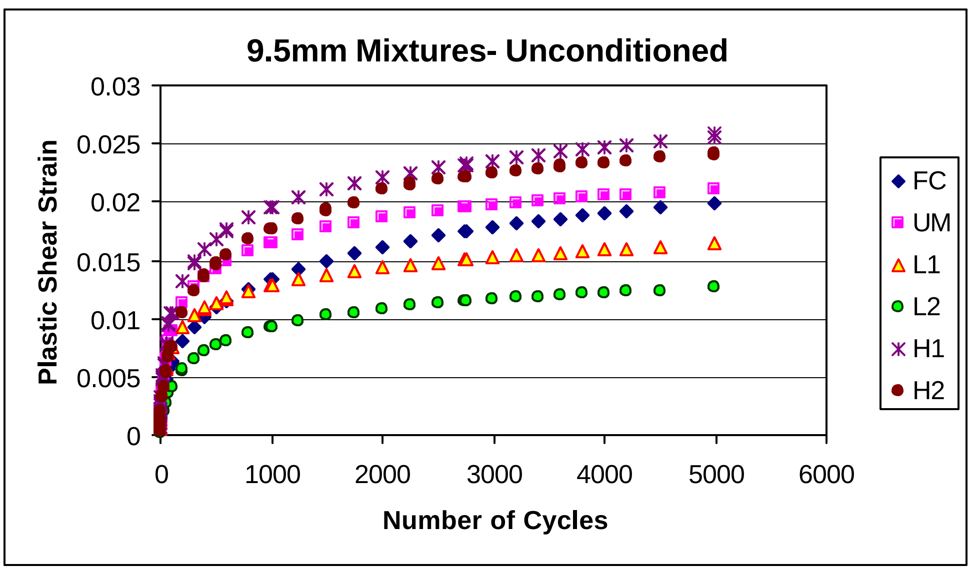

Figure 5.14 RSCH Test Results for 9.5mm Mixtures (Unconditioned)

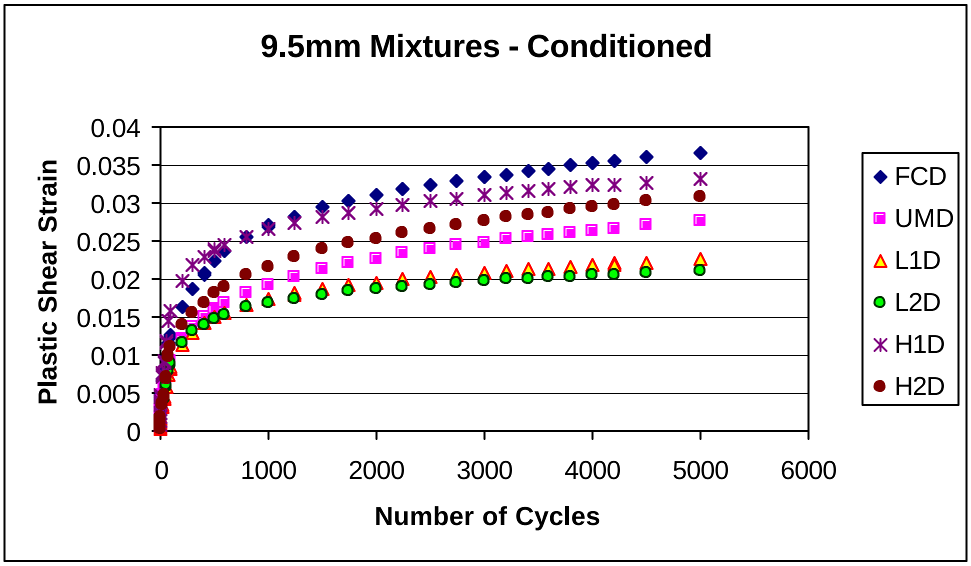

Figure 5.15 RSCH Test Results for 9.5mm Mixtures (Conditioned)

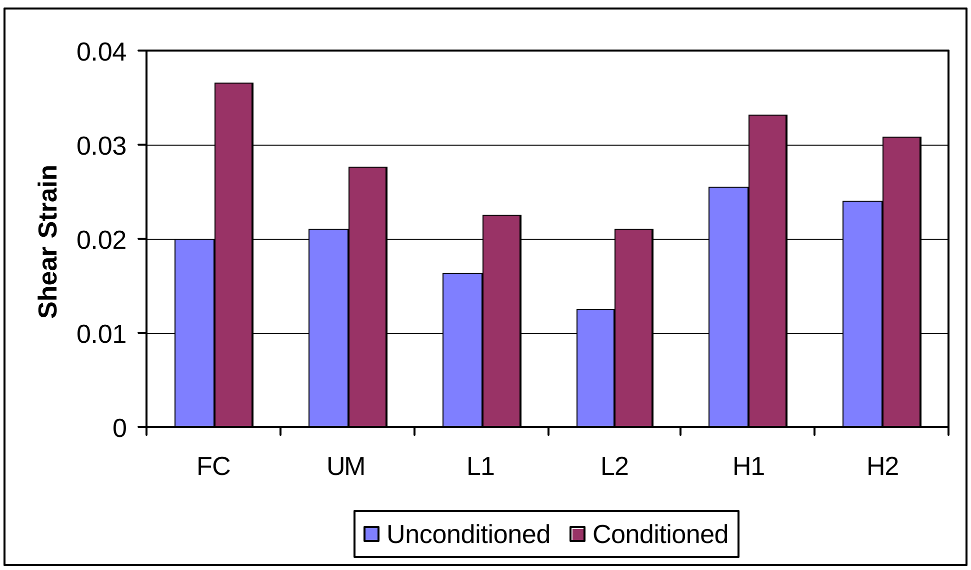

Figure 5.16 Comparison of Shear Strains of 9.5mm Mixtures

Statistical Variability:

The average (μ) shear strains for all the 12.5mm mixtures are summarized in Table 5.20. The standard deviation (σ) of the stiffness values were also calculated. The percentage of σ/μ gives is a measure to explain the scatter of data. The results of statistical variability are summarized in Table 5.20. The results show that the variability in data ranges from 3.7% to 14% for unconditioned specimens and from 4.2% to 19.5% for conditioned specimens.

Analysis of Variance:

Analysis of Variance (ANOVA) tests were also performed on the shear strains of 9.5 mm mixtures. The tests were performed at the same 95% confidence level. It was hypothesized that there was no significant difference among the shear strains of the mixtures. The rejection of hypothesis indicates that there exists a statistically significant difference among the stiffness values of the mixtures. Tables 5.21 and 5.22 furnish the results of ANOVA tests for unconditioned and conditioned specimens. The F_m statistics and the limiting value of F at certain degrees of freedom are shown in the tables. The ANOVA results indicate that the F_m statistics are higher than their limiting values. The F_m values were 15.0 and 7.2 for unconditioned and conditioned specimens whereas the limiting F value for 95% confidence level is 3.11. Thus there exists a significant difference among the shear strains of the 12.5mm mixtures. As observed in the 12.5mm mixtures, the difference in F_m between unconditioned and conditioned mixtures can be attributed to the fact that the percent increase in shear strains of high permeable mixtures due to moisture damage is lower than the percent increase in shear strains of low permeable mixtures.

Table 5.20 Statistical Analysis of Shear Strain for 9.5mm Mixtures

Mixture	Unconditioned			Conditioned		
	μ	σ	μ/σ*100	μ	σ	μ/σ*100
FC	0.0200	1.53E-03	7.8	0.0366	7.11E-03	19.5
UM	0.0211	1.60E-03	7.6	0.0277	3.05E-03	11.0
L1	0.0164	6.08E-04	3.7	0.0225	9.50E-04	4.2
L2	0.0126	1.70E-03	13.5	0.0211	8.72E-04	4.2
H1	0.0256	3.61E-03	14.1	0.0332	4.19E-03	12.6
H2	0.0241	1.97E-03	8.2	0.0308	3.75E-03	12.2

Table 5.21 ANOVA Results for 9.5mm Mixtures (Unconditioned)

Parameter	Sum of Squares	df	Mean Sum of Squares	F_m	$F_{5,12}$
Between treatments	3.51E-04	5	7.02E-04	15.00	3.11
Error	5.62E-04	12	4.68E-06		
Total	4.01E-04	17	2.36E-05		

Table 5.22 ANOVA Results for 9.5mm Mixtures (Conditioned)

Parameter	Sum of Squares	df	Mean Sum of Squares	F_m	$F_{5,12}$
Between treatments	5.59E-04	5	1.12E-04	7.20	3.11
Error	1.86E-04	12	1.55E-05		
Total	7.45E-04	17	4.39E-05		

5.6 APA Rut Tests

The APA rut tests were conducted on specimens of both 12.5mm and 9.5mm mixtures to evaluate the rutting susceptibility of the mixtures. The Asphalt Pavement Analyzer (APA) is a modification of the Georgia Loaded Wheel Tester (GLWT) and was first manufactured in 1996 by Pavement Technology, Inc. The APA is a multifunctional Loaded Wheel Tester used for evaluating permanent deformation, fatigue cracking and moisture susceptibility. The APA features controllable wheel load and contact pressure that are representative of actual field conditions. Each sample is subjected to repetitive linear wheel tracking actions with controlled pressure on cylindrical or beam specimens for rut testing. The theory behind a loaded wheel tester is to apply an appropriate cyclical loading to asphalt concrete specimens to best simulate actual traffic. This is accomplished by air pressurized hoses lying across samples with a loaded wheel coming in contact with the hose and applying a predetermined load to the hose and thus the specimens. The wheel rolls back and forth up to 8,000 times or cycles. The rut depth is then measured at the end of 8000 loading cycles.

In the APA testing, only rutting potential of the mixtures was investigated. The rutting tests were conducted on the field cores and unconditioned specimens of all the mixtures. All the specimens were tested at 8.5% air voids. The test temperature was 64^{o}C. The results of the APA test on unconditioned specimens are given in Table 5.23. The rut depths are compared in Figure 5.17. There are few issues in the evaluation of conditioned specimens using the APA rut test. As a simulative test, the mechanism of rutting in APA rut test is different from the measurement of shear strain in RSCH test. Only two of the six mixtures were tested for rutting in the conditioned state. The results of APA test of mixtures after conditioning are provided in Table 5.24.

Table 5.23 APA Rut Depths of Unconditioned Specimens

Mixture	APA Rut Depth, mm	
	12.5mm Mixture	9.5mm Mixture
Field Cores	4.78	4.11
Unmodified	4.98	4.27
Low Perm 1	4.47	3.90
Low Perm 2	4.55	3.22
High Perm 1	5.75	5.26
High Perm 2	5.41	4.48

Table 5.24 APA Rut Depth of Conditioned Specimens

Mixture	12.5mm Mixture	9.5mm Mixture
Low Perm 2	4.62	2.13
High Perm 2	5.03	3.64

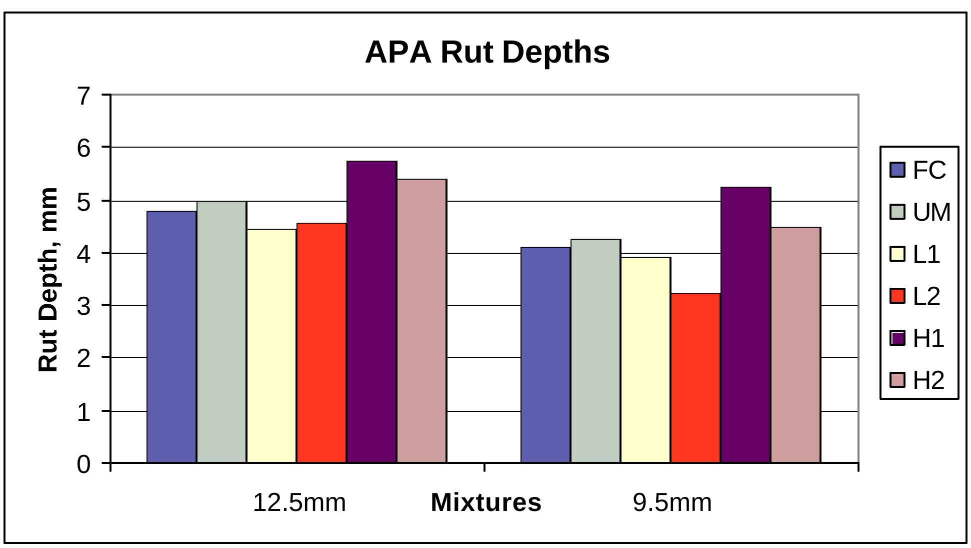

Figure 5.17 Comparison of APA Rut Depths of Unconditioned Mixtures

As indicated by the results in Table 5.23, the trends of APA rut tests reasonably agree with shear strain trends of RSCH tests. The low permeable mixtures have lower rut depths than high permeable mixtures and unmodified mixtures. The high permeable mixtures have higher rut depths than all other mixtures. The same trend was observed in the results of RSCH tests. For conditioned mixtures, the APA rut depths of low permeable mixtures (12.5 L2 and 9.5 L2) were a little lower than the rut depths of high permeable mixtures (12.5 H2 and 9.5 H2).

However, the rut depths of conditioned specimens are lower than the rut depths of unconditioned specimens. The decrease in rut depth of mixtures due to conditioning is contrary to the expected trends. The conditioning procedure of the specimens might have contributed to this unexpected trend. The APA test was conducted on conditioned specimens in dry state. The ASTM procedure recommends slightly a different method for determining moisture susceptibility of mixtures by the APA test using freeze-thaw cycles. In this study, dry specimens after AASHTO T-283 conditioning were used for the APA test, as the same procedure was adopted for the RSCH test. But the AASHTO T-283 procedure was not successful in measuring the effect of moisture damage on APA rut depths.

The stiffness, phase angles and shear strains of mixtures were used for performance prediction of mixtures in the estimation of fatigue and rutting life. The performance of low and high permeable mixtures and the effects of moisture damage are discussed in the Chapter 6.

CHAPTER 6

PERFORMANCE ANALYSIS OF MIXTURES

The FSCH tests and RSCH tests were conducted to characterize the material properties of 12.5mm and 9.5mm mixtures. The stiffness, phase angle and shear strain of the mixtures are the material responses that were used to predict the pavement's performance under service for distresses such as fatigue cracking and rutting. Fatigue and Rutting analysis are performed using surrogate models developed by SHRP 003-A project. Fatigue analysis of SHRP model considers material properties as well as pavement structural layer thickness whereas rutting analysis considers only the material properties.

6.1 SUPERPAVE Fatigue Model Analysis

The abridged fatigue analysis system from SHRP A-003A predicts the resistance of mix to fatigue distress for a pavement structure under a given traffic load. The resistance of a mix to fatigue cracking depends on the material properties such as initial flexural loss stiffness and voids filled with asphalt (VFA) and the pavement structural property, horizontal tensile strain at the bottom of the asphalt concrete layer. The abridged procedure requires an estimate of the flexural stiffness modulus of the asphalt aggregate mix at 20^{o}C (19). The flexural stiffness can be estimated from the shear stiffness of the mixture as measured from the FSCH tests at 10 Hz at 20^{o}C. This estimate is used in the multilayer elastic analysis to determine the critical level of strain to which the mix is subjected under the standard traffic load.

Multi-layer elastic analysis is used to determine the design strain, the maximum principal tensile strain at the bottom of the asphalt concrete layer, under the standard AASHTO axle load of 18 kips. For this purpose, a pavement structure was assumed to conduct this analysis. The pavement structure and loading are given in Figure 6.1. The assumed pavement structure consists of an asphalt concrete layer, an aggregate base course, a subbase resting on the subgrade. The asphalt concrete layer is 4 inches thick and the two layers beneath have a thickness of 8 inches each. The Poisson ratios and modulus of layers are shown in Figure 6.1. A standard 18-kip single axle load with dual tires inflated to 100 psi was used. The horizontal tensile strains at the bottom of AC layer are estimated at outer edge, center, inner edge, and center of dual tires using *EVERSTRESS* software for multilayer elastic analysis of pavement sections. The critical tensile strain is used as the design strain in this analysis.

The flexural properties of the mixtures are estimated using the following regression equations.

$$S_o = 8.56 * (G_o)^{0.913} \qquad R^2 = 0.712$$

$$S_o'' = 81.125 * (G_o'')^{0.725} \qquad R^2 = 0.512$$

where

S_o = initial flexural stiffness at 50th loading cycle is psi

G_o = shear stiffness at 10 Hz in psi

S_o'' = initial flexural loss stiffness at 50th loading cycle is psi

G_o'' = shear loss stiffness at 10 Hz in psi

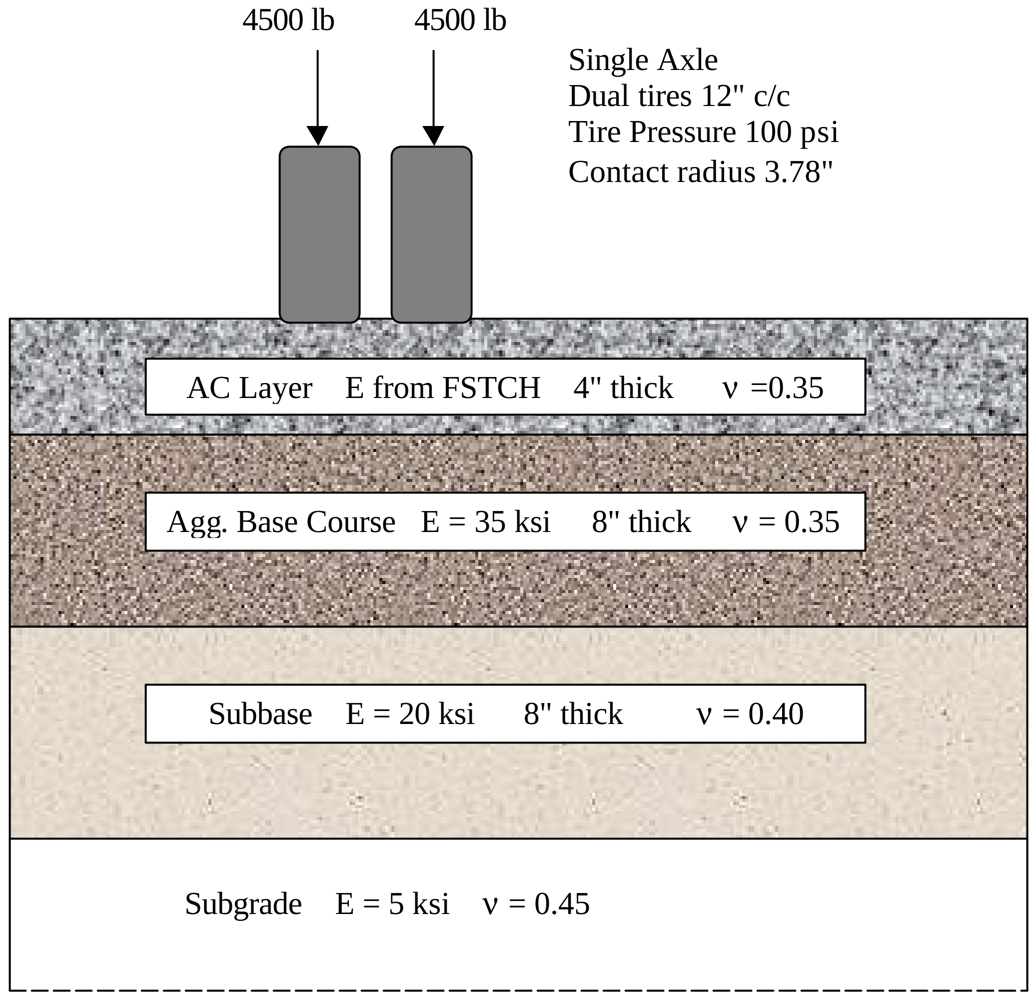

Figure 6.1 Typical Pavement Structure and Loading

The fatigue resistance of a mix is then estimated from the following strain-dependent surrogate model.

$$N_{supply} = 2.738E5 * e^{\,0.077VFB} * e_0^{-3.624} * S_0''^{\,-2.72}$$

where

Nsupply = estimated fatigue life of the given pavement section in ESALs

VFB = voids filled with asphalt

ε_0 = critical tensile strain at the bottom of AC layer

The coefficient of determination for the surrogate model for fatigue analysis is 0.79 with a coefficient of variation of 90 percent. The estimation of fatigue life for 12.5mm mixtures and 9.5mm mixtures are discussed in the following sections.

6.1.1 Fatigue Analysis of 12.5mm Mixtures

The fatigue life of the 12.5mm mixtures were estimated using the abridged fatigue analysis system for both unconditioned and conditioned specimens. First, the flexural stiffness modulus values of the mixtures were estimated using the shear stiffness and phase angles at 10 Hz measured in the FSCH tests. The flexural and shear modulus values of 12.5mm mixtures are summarized in Table 6.1.

Table 6.1 Summary of Estimated Material Properties for 12.5mm Mixtures

Mix	|G*| pa	Phase Angle	|G*| psi	Go"	So	So"
FC-UC	1.44E+09	17.5	2.09E+05	6.29E+04	6.16E+05	2.44E+05
FC-C	6.21E+08	29.8	9.01E+04	4.48E+04	2.86E+05	1.91E+05
UM-UC	1.95E+09	20.9	2.83E+05	1.01E+05	8.12E+05	3.45E+05
UM-C	7.75E+08	34.8	1.12E+05	6.42E+04	3.50E+05	2.48E+05
L1-UC	1.39E+09	22.9	2.02E+05	7.83E+04	5.96E+05	2.87E+05
L1-C	6.35E+08	38.1	9.21E+04	5.68E+04	2.92E+05	2.27E+05
L2-UC	2.45E+09	21.6	3.55E+05	1.31E+05	1.00E+06	4.15E+05
L2-C	1.36E+09	28.9	1.97E+05	9.52E+04	5.85E+05	3.30E+05
H1-UC	2.77E+09	22.6	4.02E+05	1.54E+05	1.12E+06	4.69E+05
H1-C	1.28E+09	36.4	1.86E+05	1.10E+05	5.53E+05	3.67E+05
H2-UC	2.68E+09	23.8	3.89E+05	1.57E+05	1.09E+06	4.74E+05
H2-C	1.78E+09	34.2	2.58E+05	1.45E+05	7.47E+05	4.48E+05

The fatigue resistance of the mixtures, as expected in service life of the pavement, is estimated in terms of number of ESALs (Nsupply) that corresponds to 50% reduction in the mixture stiffness. The Nsupply values were estimated by considering the flexural loss modulus, voids filled with asphalt (VFA) and critical tensile strain at the bottom of asphalt concrete layer, as shown in Table 6.2. The results for 12.5mm mixtures are summarized in Table 6.3 and Figure 6.2. The percent reduction in fatigue life of different mixtures due to conditioning is given in Table 6.3.

Table 6.2 Fatigue Life Analysis of 12.5mm Mixtures

Mix	So"	VFA	Strain	Nsupply
FC-UC	2.44E+05	71.92	2.37E-04	2.11E+06
FC-C	1.91E+05	71.92	3.29E-04	1.25E+06
UM-UC	3.45E+05	71.92	2.06E-04	1.37E+06
UM-C	2.48E+05	71.92	3.05E-04	8.15E+05
L1-UC	2.87E+05	75.8	2.41E-04	1.74E+06
L1-C	2.27E+05	75.8	3.27E-04	1.09E+06
L2-UC	4.15E+05	74.35	1.85E-04	1.48E+06
L2-C	3.30E+05	74.35	2.43E-04	1.02E+06
H1-UC	4.69E+05	74.67	1.74E-04	1.37E+06
H1-C	3.67E+05	74.67	2.50E-04	7.15E+05
H2-UC	4.74E+05	74.8	1.76E-04	1.28E+06
H2-C	4.48E+05	74.8	2.16E-04	7.13E+05

Table 6.3 Summary of Fatigue Life of 12.5mm Mixtures (Nsupply)

Mix	Unconditioned (UC)	Conditioned (C)	Percent Difference = (UC- C)/UC *100
Field Cores	2.11E+06	1.25E+06	40.5
Unmodified	1.37E+06	8.15E+05	40.3
Low Perm – 1	1.74E+06	1.09E+06	37.4
Low Perm – 2	1.48E+06	1.02E+06	31.1
High Perm – 1	1.37E+06	7.15E+05	48.0
High Perm - 2	1.28E+06	7.13E+05	44.4

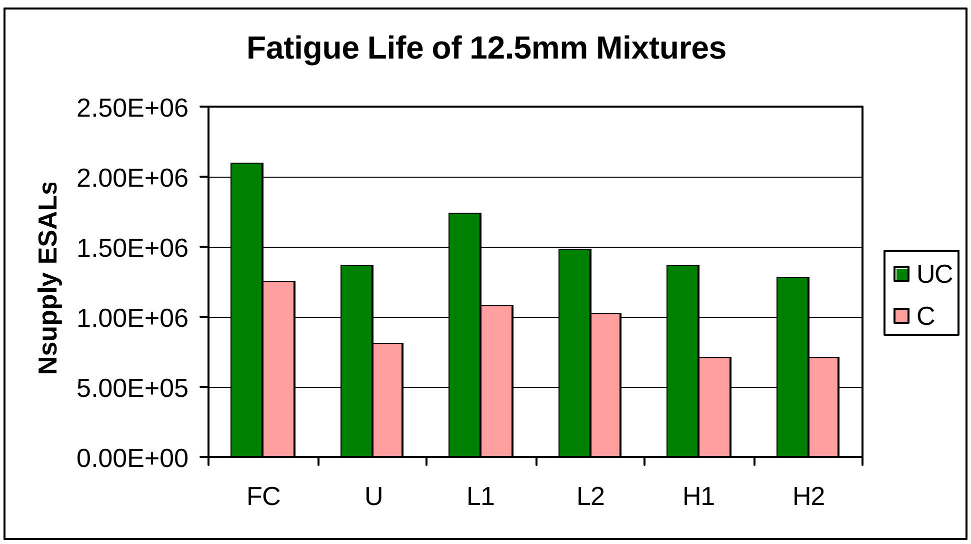

Figure 6.2 Fatigue Life of 12.5mm Mixtures

The results clearly indicate that the mixtures of low permeability have higher fatigue life than the mixtures of higher permeability in both states of conditioning. For the unconditioned specimens, the field cores have the highest fatigue life. The SGC compacted specimen that had the same gradation as that of the field cores have lower fatigue life than the field cores. The same trend is observed even after conditioning.

These specimens experience the same amount of reduction in fatigue life due to the damage caused during the conditioning process. The low permeable mixtures have higher fatigue life than the high permeable mixtures and the mixture of unmodified gradation. The mixture L1 has better fatigue life than L2 for the low permeable mixtures whereas the mixture H1 is better than H2 for the high permeable mixtures. This trend is observed in both states of conditioning.

The fatigue life values of low permeable mixtures, L1 and L2, reduce by 37.4 % and 31.1 % due to moisture damage, respectively. The high permeable mixtures, H1 and H2 have a reduction of 48.0% and 44.4% in their fatigue life due to moisture damage, respectively. For field cores and the unmodified mixtures, the fatigue life reduces by almost 40.5% and 40.3%, respectively. The Nsupply values and the percent reduction due to conditioning indicate that the mixtures with low permeability have higher fatigue life and lower damage compared to the mixtures of high permeability. The loss in tensile strength could be lower in low permeable mixtures than in the high permeable mixtures.

Analysis of Variance:

The ANOVA tests were conducted for the results of fatigue life of 12.5mm mixtures. The values of fatigue life given in Table 6.2 were calculated by assuming the mean dynamic modulus and phase angle at 10Hz of each mixture. The values used in Table 6.1 and 6.2 are the mean averages of data points of three replicates used in FSCH tests. The individual data points, $|G^*|$ and δ at 10 Hz, were used to compute the individual fatigue life values of mixtures for the ANOVA test. The test was conducted to check whether at least one significant difference exists among the fatigue life values of mixtures. The ANOVA tests were conducted separately for unconditioned and conditioned mixtures. The results of the ANOVA tests are provided in Tables 6.4 and 6.5 for unconditioned and conditioned mixtures, respectively.

Table 6.4 ANOVA Results for Fatigue Life of 12.5mm Mixtures (Unconditioned)

Parameter	Sum of Squares	df	Mean Sum of Squares	F*	$F_{5,12}$
Between treatments	1.51E+12	5	3.01E+11	9.24	3.11
Error	3.91E+11	12	3.26E+10		
Total	1.90E+12	17	1.12E+11		

Table 6.5 ANOVA Results for Fatigue Life of 12.5mm Mixtures (Conditioned)

Parameter	Sum of Squares	df	Mean Sum of Squares	F*	$F_{5,12}$
Between treatments	6.29E+11	5	1.28E+11	18.37	3.11
Error	8.35E+10	12	6.96E+09		
Total	7.23E+11	17	4.25E+10		

The results show that the computed F* statistic is greater than the limiting F value at 95% confidence level for both type of specimens. This indicates that at least one significant difference exists among the fatigue life values of mixtures. This difference is more pronounced among the conditioned specimens than the unconditioned specimens as the F* value of conditioned specimens (18.37) is higher than the F* value of unconditioned specimens (9.24)

6.1.2 Fatigue Analysis of 9.5mm Mixtures

The fatigue life of the 9.5mm mixtures were estimated using the abridged fatigue analysis system for both unconditioned and conditioned specimens. The estimations of flexural and shear modulus values of 9.5mm mixtures are summarized in Table 6.6.

Table 6.6 Summary of Estimated Material Properties for 9.5mm Mixtures

Mix	|G*| pa	Phase	|G*| psi	Go"	So	So"
FC-UC	3.10E+09	19.23	4.50E+05	1.48E+05	1.24E+06	4.55E+05
FC-C	2.23E+09	23.48	3.23E+05	1.29E+05	9.18E+05	4.11E+05
UM-UC	2.50E+09	15.34	3.63E+05	9.59E+04	1.02E+06	3.32E+05
UM-C	1.84E+09	19.20	2.67E+05	8.78E+04	7.70E+05	3.11E+05
L1-UC	1.62E+09	18.73	2.35E+05	7.54E+04	6.86E+05	2.79E+05
L1-C	1.47E+09	20.63	2.13E+05	7.51E+04	6.28E+05	2.78E+05
L2-UC	2.16E+09	14.40	3.13E+05	7.79E+04	8.92E+05	2.85E+05
L2-C	1.90E+09	16.52	2.76E+05	7.84E+04	7.93E+05	2.87E+05
H1-UC	2.47E+09	19.23	3.58E+05	1.18E+05	1.01E+06	3.86E+05
H1-C	1.87E+09	24.68	2.71E+05	1.13E+05	7.82E+05	3.74E+05
H2-UC	1.74E+09	21.11	2.52E+05	9.09E+04	7.32E+05	3.19E+05
H2-C	1.21E+09	28.78	1.75E+05	8.45E+04	5.25E+05	3.03E+05

The fatigue life of mixtures was estimated using the SHRP fatigue relationship discussed earlier in this chapter. The Nsupply values as estimated by considering the flexural loss modulus, voids filled with asphalt (VFA) and critical tensile strain at the bottom of asphalt concrete layer are shown in Table 6.7. The results for 9.5mm mixtures are summarized in Table 6.8 and Figure 6.3. The percent reduction in fatigue life due to conditioning of different mixtures is given in Table 6.8.

Table 6.7 Fatigue Life Analysis of 9.5mm Mixtures

Mix	So"	VFA	Strain	Nsupply
FC-UC	4.55E+05	72.38	1.64E-04	1.54E+06
FC-C	4.11E+05	72.38	1.94E-04	1.11E+06
UM-UC	3.32E+05	72.38	1.83E-04	2.44E+06
UM-C	3.11E+05	72.38	2.12E-04	1.69E+06
L1-UC	2.79E+05	77.2	2.25E-04	2.67E+06
L1-C	2.78E+05	77.2	2.35E-04	2.30E+06
L2-UC	2.85E+05	75.25	1.97E-04	3.53E+06
L2-C	2.87E+05	75.25	2.09E-04	2.79E+06
H1-UC	3.86E+05	73.8	1.84E-04	1.78E+06
H1-C	3.74E+05	73.8	2.11E-04	1.17E+06
H2-UC	3.19E+05	75.83	2.18E-04	1.88E+06
H2-C	3.03E+05	75.83	2.56E-04	1.21E+06

It is evident from the results of 9.5mm mixtures that the mixtures of low permeability have higher fatigue life than the mixtures of higher permeability in both states of conditioning. The mixture L2 has the highest fatigue life in unconditioned and conditioned state. Another low permeable mixture, L1 has the second highest fatigue life among all the mixtures. The high permeable mixtures have lower fatigue life than the low permeable mixtures and the unmodified mixture. This trend is observed in both states of conditioning.

Table 6.8 Summary of Fatigue Life of 9.5mm Mixtures (Nsupply)

Mix	Unconditioned (UC)	Conditioned (C)	Percent Difference = (UC- C)/UC *100
Field Cores	1.54E+06	1.11E+06	27.9
Unmodified	2.44E+06	1.69E+06	30.6
Low Perm – 1	2.67E+06	2.30E+06	13.8
Low Perm – 2	3.53E+06	2.79E+06	20.9
High Perm – 1	1.78E+06	1.17E+06	34.0
High Perm - 2	1.88E+06	1.21E+06	35.7

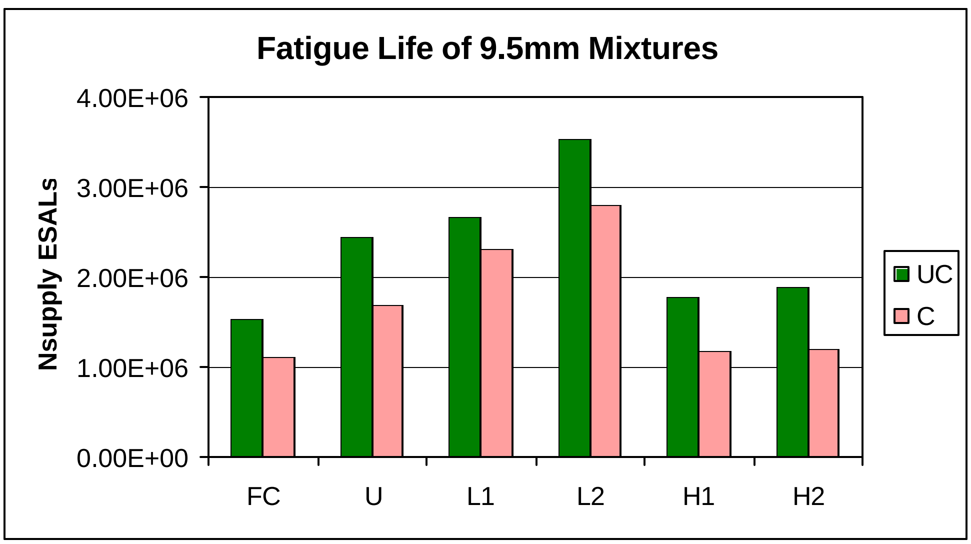

Figure 6.3 Fatigue Life of 9.5mm Mixtures

Contrary to the trend of 12.5mm mixtures, the field cores of 9.5mm mixtures have the lowest fatigue life. The specimens of unmodified mixture have higher fatigue life than the field cores. But the field cores as well as unmodified mixture have the same amount of reduction in fatigue life due to the induced moisture damage during the conditioning process.

The percent reduction in fatigue life for low permeable mixture is less than 20%. The high permeable mixtures have a reduction in fatigue life by around 35% due to moisture damage. The field cores and unmodified mixture have reduction in their fatigue life by around 28-30% which lies in between the values of low and high permeable mixtures. The N_{supply} values and the percent reduction due to conditioning indicate that the mixtures with low permeability have higher fatigue life and lower damage when compared to the mixtures of high permeability. The loss in tensile strength could be lower in low permeable mixtures than in the high permeable mixtures.

Analysis of Variance:

The ANOVA tests were performed for the fatigue life values of 9.5 mm mixtures. The fatigue life values estimated from the individual dynamic modulus and phase angles of replicates of mixtures. The results are summarized in Tables 6.9 and 6.10. As the calculated F* statistic value is higher than the limiting value of F for both states of specimen conditioning of 9.5mm mixtures, there exists at least one significant difference among the fatigue life values of mixtures at 95% confidence level. The F* values are 6.60 and 22.46 for unconditioned and conditioned specimens, respectively. This observation indicates that the difference is more pronounced in conditioned specimens than the unconditioned specimens.

Table 6.9 ANOVA Results for Fatigue Life of 9.5mm Mixtures (Unconditioned)

Parameter	Sum of Squares	df	Mean Sum of Squares	F*	$F_{5,12}$
Between treatments	8.71E+12	5	1.74E+12	6.80	3.11
Error	3.07E+12	12	2.56E+11		
Total	1.18E+13	17	6.93E+11		

Table 6.10 ANOVA Results for Fatigue Life of 9.5mm Mixtures (Conditioned)

Parameter	Sum of Squares	df	Mean Sum of Squares	F*	$F_{5,12}$
Between treatments	7.55E+12	5	1.51E+12	22.46	3.11
Error	8.07E+11	12	6.72E+10		
Total	8.36E+12	17	4.92E+11		

6.2 SUPERPAVE Rutting Model Analysis

The permanent deformation system of SHRP A-003A estimates rut depth from the maximum permanent shear strain obtained from RSCH test using the following relation.

*Rut depth (in.) = 11 * Maximum permanent shear strain*

If rutting in millimeters is desired, the coefficient of the above equation is about 275. The above relationship is obtained for a tire pressure of 100psi and asphalt layer thickness of 15inch. Studies performed for the similar pavement structure at 200psi and 500psi suggest that this relationship is independent of the tire pressure. But the same is not true in the case of pavement thickness. The coefficient is expected to decrease with a decrease in asphalt layer thickness(19).

The conversion of the number of RSCH test cycles to ESALs is determined by the following equation:

$$\log(\text{cycles}) = -4.36 + 1.24 \log(\text{ESALs})$$

where,

cycles = number of cycles obtained from the RSCH test

ESALs = equivalent 18-kip single axle load

According to the above relationship, 5000 cycles of the RSCH test correspond to 3.156 million ESALs. Tables 6.11 and 6.12 give the estimated rut depth of 12.5mm mixtures and 9.5mm mixtures, respectively. As the shear strains are to be multiplied by the factor 11 for estimating the rut depth, the same trend of the shear strains would be observed here.

The results show that the rut depths of all the mixtures increase due to damage caused by conditioning. This increase is observed in both 12.5mm and 9.5mm mixtures. All the mixtures have rut depths less than the maximum allowable rut depth of 0.5 inches. The mixtures with low permeability have lower rut depths than the mixtures with higher permeability. The unconditioned specimens of low permeable mixtures have rut depths less than 0.2 inches whereas the conditioned specimens have rut depths higher than 0.3 inches for 12.5mm mixtures and 0.25 for 9.5mm mixtures. The increase in rut depth due to moisture damage for low permeable mixtures is higher than the increase in rut depth for high permeable mixtures.

Table 6.11 Rut Depths of 12.5mm Mixtures

Mixture	Unconditioned	Conditioned
Field Cores	0.231	0.339
Unmodified	0.242	0.297
Low Perm – 1	0.145	0.408
Low Perm – 2	0.169	0.321
High Perm – 1	0.341	0.373
High Perm - 2	0.352	0.418

Table 6.12 Rut Depths of 9.5mm Mixtures

Mixture	Rut Depth, inches	
	Unconditioned	Conditioned
Field Cores	0.22	0.40
Unmodified	0.23	0.30
Low Perm 1	0.18	0.25
Low Perm 2	0.14	0.23
High Perm 1	0.28	0.37
High Perm 2	0.27	0.34

Thus the performance analysis of mixtures shows that the fatigue life and rutting life is higher for low permeable mixtures than the fatigue life and rutting life high permeable mixtures. In general, the effect of permeability is more predominant in fatigue life than rutting. The water percolation owing to the high permeability of pavements causes moisture damage in the mixtures. The moisture damage causes failure in the aggregate-asphalt interface that result in loss of cohesion and adhesion. The reduction in tensile strength and stiffness of the mixtures due to stripping, reduces the fatigue life of mixtures. The rutting susceptibility of the mixtures also increases due to asphalt-aggregate debonding. By comparing the reduction in fatigue life and increase in rutting, it can be judged that the effect of moisture damage or permeability is more predominant in fatigue life than in rutting.

CHAPTER 7

DISCUSSION OF RESULTS AND CONCLUSIONS

7.1 Discussion of Results

The permeability of pavements has become a major concern in recent years with the implementation of Superpave mixtures. High permeable pavements allow the percolation of water which in turn causes loss of adhesion between asphalt binder and aggregate surface and subsequent crack initiation for fatigue damage. The permeability of mixtures depends on air void content, nominal maximum size, gradation of aggregates and other factors. The size and continuity of air voids in the aggregate structure controls the permeability of a mixture. The aggregate gradations can be fine-tuned to have a desired level of permeability. So it was decided to optimize the gradations that pass below the restricted zone for lower permeability without comprising the performance of mixtures in terms of fatigue life and rutting.

The concepts of Bailey method of gradation analysis was used for optimizing aggregate blends. This method defines different “sieves” and “aggregate ratios” to quantify the shape of aggregate gradation. Aggregate ratios, which are based on particle packing principles, are used to analyze the particle packing of the overall aggregate structure. The method recommended the use of #4 or 4.75mm sieve (half sieve) and #8 or 2.36 sieve (primary control sieve) for 12.5mm and 9.5mm mixtures. Applying the concepts of Bailey method, trial gradations were developed with different values of CA ratios. Other ratios such as the FA_C ratio and the FA_F ratio of the fine aggregate is not given importance as this method postulates the use of coarse aggregate as the primary component in an asphalt mixture for developing the aggregate structure. Moreover the trial gradations were selected in such a way that all gradations would pass below the restricted zone so that proportion of fine aggregates in the blend would be less than 30%. This leaves fewer options for varying the fine aggregate fractions in the overall blend of aggregates.

The permeability tests were conducted with mixtures of these trial gradations at 8.5 % air voids content. Two gradations were chosen by the NCDOT which had potential permeability problems. One gradation had a nominal size of 12.5mm and another gradation had a nominal size of 9.5mm gradation. The permeability coefficient of this "unmodified" gradation was considered as the reference gradation for comparing permeability level of trial gradations. Four gradations were chosen for each nominal size such that two mixtures had higher permeability and other two mixtures had lower permeability than the permeability coefficient of reference or unmodified gradation. Lessons learned from the permeability tests of trial gradations were applied to propose guidelines for developing aggregate blends for desired level of permeability. Separate guidelines were developed for 12.5mm and 9.5mm mixtures. These guidelines would help the designers to arrive at aggregate blends with low or high permeability. The #4 (4.75) size of aggregates forms the fulcrum of permeability characteristics of both 12.5mm and 9.5mm mixtures. The proportion of #4 fraction influences relative proportion of other fractions in the aggregate blend. High permeable mixtures have higher proportion of #4 fraction (around 40%) and lower proportion of #8 and # 16 whereas low permeable mixtures have lower proportion of #4 fraction (around 25%) and higher proportion of #8 and #16 fractions. In a gradation that passes below the restricted zone, the amount of #4 size of aggregates influences the amount of #8 size needed in a coarse gradation. A lower amount of #4 size would increase the amount of #8 size aggregates. The high proportion of #8 and # 16 reduces the size of voids but does not ensure the discontinuity of voids. The continuity of smaller size voids would not lock up the water flow in the specimens. It is recommended to have relatively higher fractions of 12.5mm and 9.5mm sizes for low permeable mixtures in order to plug the continuity of voids. The new gradations were developed with these guidelines for both the 12.5mm and 9.5mm mixtures. The permeability coefficients of these mixtures were used to validate the guidelines proposed in this study.

The mixtures of selected gradations were designed in the laboratory using the Superpave mixture design system. These mixtures along with the field cores were used for performance evaluation tests. A total of 12 mixtures with 6 each for 12.5mm and 9.5mm mixtures were tested using Shear tests and APA tests. Two sets of these 12 mixtures were tested with one set of specimens unconditioned and another set conditioned in accordance with AASHTO T-283 procedure. Performance evaluation tests included FSCH tests, RSCH tests and APA tests.

The dynamic modulus and phase angles of the mixtures were measured from the FSCH tests. These parameters are used in the SHRP model to estimate the fatigue life of the mixtures. The $|G^*|$ values at 10Hz of 12.5mm mixtures show that all unconditioned mixtures are stiffer than the conditioned mixtures. The stiffness of the mixtures decreases due to the moisture damage induced during the conditioning process. The percentage reduction in the stiffness due to conditioning is around 50 percent. The |G*| values at 10Hz of 9.5mm mixtures also show that unconditioned mixtures are stiffer than the conditioned mixtures. The percentage reduction in stiffness for low permeable mixture is lower than other mixtures. For low permeable mixtures, the percent reduction is about 10% and for other mixtures, the percentage increases to about 25%. The stiffness values do not show any trend among the mixtures in both 12.5mm and 9.5mm mixtures. The stiffness values combined with phase angles influence the fatigue characteristics of the mixtures. For 12.5mm mixtures, the phase angles of unconditioned mixtures are lower than the phase angles of conditioned mixtures. The phase angle at 10Hz ranges from 17.5 to 23.8 for 12.5mm mixtures and from 15.24 to 21.11 for 9.5mm mixtures in unconditioned state. When the specimens are subjected to conditioning, the phase angles increase to a range of 28.9 to 43.1 for 12.5mm mixtures and 16.5 to 28.8 for 9.5mm mixtures. There is an increase in phase angle values for all mixtures when these specimens are subjected to moisture damage. The phase angles of the mixtures do not show any trend among the mixtures but considering the averages, the phase angle seems to be higher for the high permeable mixtures than the low permeable mixtures. The average percent increase in phase angle due to conditioning is about 60% for 12.5mm mixtures and about 23% for 9.5mm mixtures.

The fatigue life of mixtures was estimated using the SHRP model for fatigue analysis and assumed pavement structure. The results of the SHRP fatigue analysis show that the low permeable mixtures have higher fatigue life than the high permeable mixtures. This trend is observed in both 12.5mm mixtures and 9.5mm mixtures. The field cores and unmodified mixtures rank between the low and the high permeable mixtures. The percent reduction in fatigue life is lower for low permeable mixtures than all of the other mixtures. For 12.5mm mixtures, the average reduction in fatigue life due to conditioning is about 30% for the low permeable mixtures, 44% for the high permeable mixtures and 37% for the unmodified mixtures. For 9.5mm mixtures, the average reduction in fatigue life is about 20% for the low permeable mixtures, 30% for the high permeable mixtures and 26% for the unmodified mixtures. This indicates that permeability has a direct influence on the fatigue performance of the mixtures.

The RSCH tests were conducted to evaluate the rutting potential of mixtures. The results of RSCH tests show that all the mixtures passed the allowable criteria of 5000 cycles with none of the mixtures crossing the 5% shear strain limit. For unconditioned specimens, the low permeable mixtures of both the nominal sizes have lower shear strains than the other mixtures, whereas the high permeable mixtures have higher shear strains. The shear strains increase when the specimens are subjected to moisture damage. For conditioned specimens, the same trend is observed as the low permeable mixtures have lower shear strains and the high permeable mixtures have higher shear strains as compared with all other mixtures. The rut depths of mixtures were estimated using the SHRP rutting model.

7.2 Conclusions

Based on the analysis and discussion of the test data, the following conclusions can be drawn:

1. The permeability of the mixtures directly influences their performance in terms of fatigue life, rutting life and moisture susceptibility. The low permeable mixtures are more desirable than the high permeable mixtures as these mixtures have higher fatigue life and rutting life.
2. The percent reduction in service life due to moisture damage is lower in low permeability mixtures than in the high permeable mixtures.
3. The guidelines for developing aggregate blends for a desired level of permeability are proposed in this study. These guidelines would help the designers to select appropriate gradations for Superpave mixture design.
4. The #4 size of aggregate fraction plays a pivotal role in determining the permeability characteristics of both 12.5mm and 9.5mm mixtures. A higher fraction of #4 size in an aggregate blend increases the permeability whereas a lower fraction of #4 size decreases the permeability. A maximum of 25% for #4 size aggregates is recommended for low permeability.
5. Higher fractions of #8, #16, 3/8" and 1/2" sizes in an aggregate blend are desired for low permeable mixtures. The fractions of #8 and #16 would decrease the size of air voids but might not efficiently plug the continuity of voids. The inclusion of higher fractions of 3/8" and 1/2" along with #8 and #16 would ensure the discontinuity of voids and thereby decreasing the permeability of the mixtures.
6. The proportion of #4 size in an aggregate blend influences the amount of #8 and #16 fractions to be included in the gradations that pass below the restricted zone.

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