Report No. K-TRAN: KSU-00-2
FINAL REPORT

A STUDY OF FACTORS AFFECTING THE PERMEABILITY OF SUPERPAVE MIXES IN KANSAS

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NOVEMBER 2004

K-TRAN

A COOPERATIVE TRANSPORTATION RESEARCH PROGRAM BETWEEN:
KANSAS DEPARTMENT OF TRANSPORTATION
KANSAS STATE UNIVERSITY
THE UNIVERSITY OF KANSAS

<table>
<tr><td>1 Report No.
K-TRAN: KSU-00-02</td><td>2 Government Accession No.</td><td>3 Recipient Catalog No.</td></tr>
<tr><td colspan="2" rowspan="2">4 Title and Subtitle
A STUDY OF FACTORS AFFECTING THE PERMEABILITY OF SUPERPAVE MIXTURES IN KANSAS</td><td>5 Report Date
November 2004</td></tr>
<tr><td>6 Performing Organization Code</td></tr>
<tr><td colspan="2">7 Author(s)
Aneel K. Gogula, Mustaque Hossain, Ph.D., P.E., and Stefan A. Romanoschi, Ph.D., P.E.</td><td>8 Performing Organization Report No.</td></tr>
<tr><td colspan="2" rowspan="2">9 Performing Organization Name and Address
Kansas State University
Department of Civil Engineering
Manhattan, Kansas 66506-2905</td><td>10 Work Unit No. (TRAIS)</td></tr>
<tr><td>11 Contract or Grant No.
C1247</td></tr>
<tr><td colspan="2" rowspan="2">12 Sponsoring Agency Name and Address
Kansas Department of Transportation
Bureau of Materials and Research
700 SW Harrison Street
Topeka, Kansas 66603-3754</td><td>13 Type of Report and Period Covered
Final Report
September 2000 – July 2004</td></tr>
<tr><td>14 Sponsoring Agency Code
RE-1247-01</td></tr>
<tr><td colspan="3">15 Supplementary Notes
For more information write to address in block 9.</td></tr>
<tr><td colspan="3">16 Abstract
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The results show that for any given nominal maximum size Superpave mixture, the fine-graded mixture is generally less permeable than the coarse-graded mixtures. Percent material passing 600-micron (No. 30) sieve, asphalt film thickness and air voids significantly influence the permeability of 12.5 mm NMAS mixtures. For 19 mm mixtures, significant variables are percent air voids in the compacted mixture sample, percent material passing 600-micron (No. 30) sieve and the number of gyrations required to reach the target air void. Superpave mixtures with lower permeability values performed, irrespective of gradation, very well under the Hamburg wheel rut tester indicating that less permeable mixtures are less susceptible to stripping and rutting. Optimum limits for the significant variables found in this study were determined so that the permeability of Superpave mixtures could be minimized. No significant relationship between the laboratory-measured and the field permeability values was found.</td></tr>
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<tr><td colspan="2">17 Key Words
Air Voids, AC, Asphalt Concrete, Pavement, Permeability and Superpave</td><td colspan="2">18 Distribution Statement
No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161</td></tr>
<tr><td>19 Security Classification (of this report)
Unclassified</td><td>20 Security Classification (of this page)
Unclassified</td><td>21 No. of pages
116</td><td>22 Price</td></tr>
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Final Report

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A Report on Research Sponsored By

THE KANSAS DEPARTMENT OF TRANSPORTATION
TOPEKA, KANSAS

KANSAS STATE UNIVERSITY
MANHATTAN, KANSAS

November 2004

PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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ABSTRACT

Permeability influences the performance of Superpave pavements. Percolation of water through interconnected voids of an asphalt pavement causes stripping of the asphalt-bound layer(s) as well as deterioration of the foundation layers. Laboratory (falling head) permeability tests were conducted on different Superpave mixtures with 19 mm and 12.5 mm nominal maximum aggregate sizes (NMAS), and coarse and fine gradations, to identify factors that affect the permeability of these mixtures in Kansas. Hamburg wheel tests were performed to study rutting and stripping potential of these mixtures. Field permeability tests were also conducted on different projects with 19 mm and 12.5 mm NMAS Superpave mixtures in order to study the correlation between laboratory-measured and field permeability values.

The results show that for any given nominal maximum size Superpave mixture, the fine-graded mixture is generally less permeable than the coarse-graded mixtures. Percent material passing 600-micron (No. 30) sieve, asphalt film thickness and air voids significantly influence the permeability of 12.5 mm NMAS mixtures. For 19 mm mixtures, significant variables are percent air voids in the compacted mixture sample, percent material passing 600-micron (No. 30) sieve and the number of gyrations required to reach the target air void. Superpave mixtures with lower permeability values performed, irrespective of gradation, very well under the Hamburg wheel rut tester indicating that less permeable mixtures are less susceptible to stripping and rutting. Optimum limits for the significant variables found in this study were determined so that the permeability of Superpave mixtures could be minimized. No significant relationship between the laboratory-measured and the field permeability values was found.

ACKNOWLEDGEMENTS

The financial support for this study was provided by the Kansas Department of Transportation (KDOT) under the Kansas Transportation and New Developments (K-TRAN) program. The authors wish to thank Mr. Glenn Fager, the project monitor from KDOT, for his invaluable help and advice throughout the project. Special thanks to Mr. Bryce Barkus and Ms. Jennifer Jacka who helped out with the lab testing. The help of KDOT area personnel and contractors is also acknowledged.

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CHAPTER 1

INTRODUCTION

1.1 Introduction

Asphalt concrete roads constitute more than 90% of the paved road network in the United States [Superpave System, 1995]. Asphalt pavements are typically designed for 20 years. However, frequent failures are noticed in such pavements. The failures could be attributed to several causes, such as, improper mix design, increased traffic volume, tire pressure and axle loading and deficiency in specifications. Three major distress types observed on asphalt pavements are: rutting, fatigue cracking and low temperature cracking. These distresses occur due to high temperatures combined with traffic loading, repeated load applications, aging, moisture damage and thermal stresses due to daily/seasonal temperature cycle. Development of a new system for specifying asphalt materials began in 1987 by the Strategic Highway Research Program (SHRP). The primary objective of the research was to improve the performance and durability of asphalt pavements in the United States. The final product of the SHRP asphalt research is a new system called Superpave, which is the short for **Su**perior **Per**forming Asphalt **Pave**ments. Superpave represents an improved system for specifying asphalt binders and mineral aggregates, developing an asphalt mixture design and analyzing and establishing pavement performance prediction [Superpave, 1995]. Superpave incorporates performance-based asphalt material characterization as a function of project environmental conditions to improve performance by controlling major distresses. The system is a performance-based specification system, in which the tests and analyses have direct relationships with the field performance. The activities in Superpave mix design include selection of materials based on the specifications, volumetric mix design and performance tests and prediction. Superpave mixture design and analysis is performed at one of three increasingly rigorous levels, with each level providing more information about mixture performance. Superpave volumetric design (originally termed Superpave level 1) is an improved material selection and volumetric mix design process and is applicable to projects with design traffic (ESAL's) up to 1,000,000. Superpave abbreviated mix analysis (original level 2 mix design) procedures use the volumetric mix design as a starting point and include a battery of Superpave Shear Test (SST) and Indirect Tensile Tests (IDT) to arrive at a series of performance predictions. This level is applicable to traffic level in between 1,000,000 and 10,000,000. Superpave full mix analysis (original level 3 mixture design) includes a more comprehensive array of SST IDT tests and results to achieve a more reliable level of performance prediction for projects with traffic level greater than 10,000,000 [Hossain, 2001]. Tests for moisture-induced damage or stripping potential are conducted at each level.

1.2 Problem Statement

Hot Mix Asphalt (HMA) is used to provide smooth, stable and durable pavements. It can be anticipated that the life of a permeable pavement would be shorter than that of an impermeable pavement. This is due to the fact that the asphalt mix will degrade and deteriorate through water and air infiltration and that would cause subsequent raveling, stripping and hardening of the binder due to oxidation. To maximize the performance of HMA pavements they need to be constructed with adequate field density and they should be relatively impermeable to moisture. Inadequate surface and/or subsurface drainage provides moisture or water vapor, which is the necessary ingredient for inducing stripping (moisture-induced damage). If excessive moisture or water is present in the pavement system the HMA pavement can strip prematurely. A number of states across the United States have reported problems with unacceptable permeability (also known as coefficient of permeability) associated with the use of coarse-graded Superpave mixtures [Hicks, 1991]. Figure 1.1 shows the percentage of asphalt pavements experiencing moisture distresses. It appears that the problem is predominant all over the country.

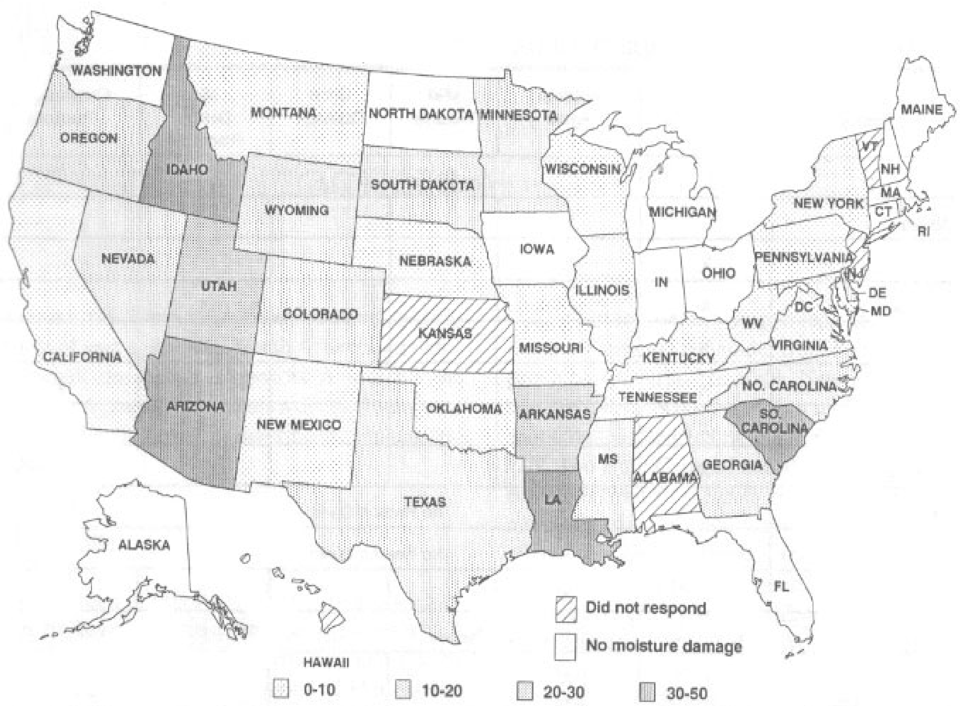


Figure 1.1 Estimated Percentage of Pavements Experiencing Moisture-Related Distress [Hicks, 1991]

There has been a continuing discussion regarding the in-place air voids and layer thickness needed to ensure an impermeable pavement. Some states have increased field density requirements and/or lift thickness requirements for coarse graded Superpave mixes. Currently, the Superpave mix design does not have any required criteria for acceptable permeability limits.

The Kansas Department of Transportation (KDOT) has implemented Superpave mix design on most of its road projects. A number of projects have been built and many are being planned. The implementation of this mixture design system represents a new era of providing pavements to the users in Kansas. It has been observed in Kansas that mixes with coarse gradation and higher nominal maximum size aggregates cause the water to percolate down to the subgrade after rain, sometimes making it unsuitable for supporting the paving train. Hence a study is needed to cope with the permeability problems associated with the coarser Superpave mixtures in Kansas.

1.3 Research Objective

The objective of this project was to study various factors that affect the permeability of Superpave pavements and also to establish acceptable permeability limits for the Superpave mixtures in Kansas. Permeability evaluation of various coarse (with gradation passing below the maximum density line and restricted zone) and fine graded (with gradation passing above the maximum density line and above the restricted zone) Superpave mixes was conducted on Superpave gyratory compactor-compacted specimens. Twelve different mixtures with nominal maximum aggregate sizes (NMAS) of 19 mm and 12.5 mm were used in the study. Figure 1.2 illustrates various NMAS used in Superpave mixture design. Statistical analysis software, SAS, was used to identify different factors and also to develop a regression equation that would predict the permeability of the mix.

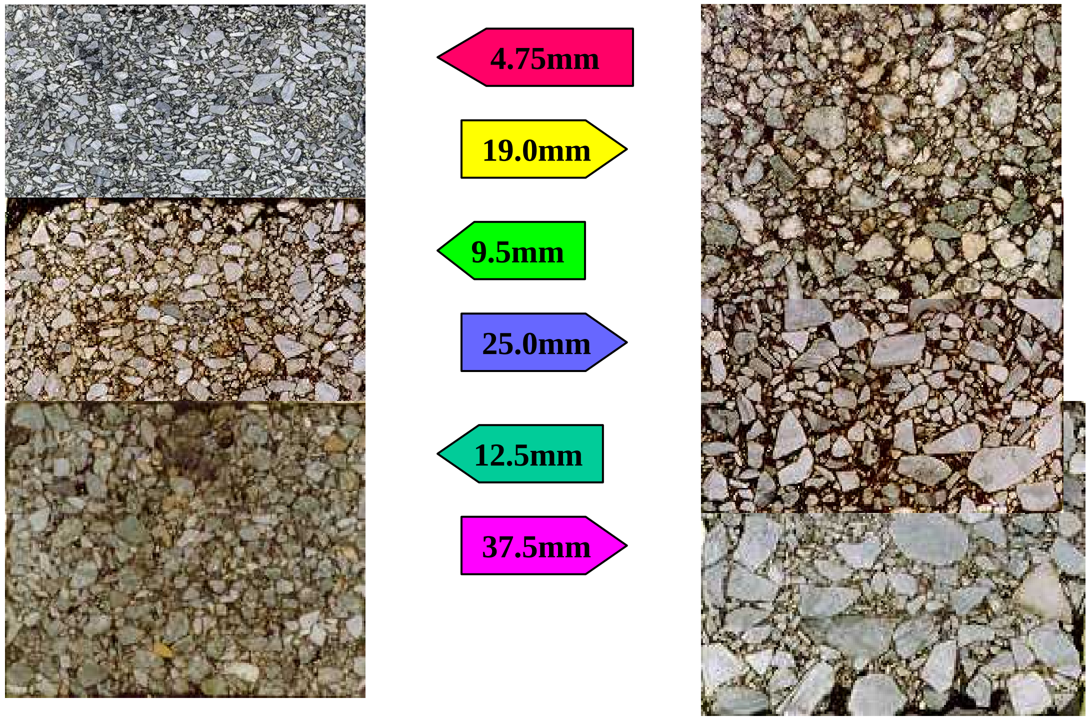


Figure 1.2 Different Aggregate Sizes Used in Superpave Mix Design

1.4 Outline

This report is divided into five chapters. Chapter 1 is an introduction, which deals with the problem statement, objectives and organization of the report. Chapter 2 is a review of literature, and it describes the background and Superpave terminology used. Chapter 3 discusses the test procedure used and the data collection. The analysis of the laboratory test results and discussion is presented in Chapter 4. Chapter 5 discusses the field permeability testing and results. Finally, Chapter 6 presents the conclusions and recommendations based on this study.

CHAPTER 2

LITERATURE REVIEW

The asphalt mixtures are designed to construct a road surface that is smooth and will retain smoothness for the designed period without any premature failure under the expected traffic at the lowest possible cost, including maintenance. For achieving high quality asphalt pavements proper placement and compaction are necessary. There are various factors related to the mix design and materials that may lead to failures, such as, rutting, fatigue cracking, low temperature cracking, and stripping. The type of liquid asphalt used, crude source, refining processes and the type of asphalt mix are some of the factors related to materials that affect the failure of an asphalt pavement. Traditional mix design methods, such as, Hveem and Marshall methods, are empirical methods, which were not developed to address the current in-field performance problems [Roberts, 1996]. In addition, increased traffic load, axle-loads and tire pressures are the most common factors affecting pavement performance these days. There are other distresses such as raveling, reduced skid resistance and bleeding, but rutting, fatigue cracking, low temperature cracking and moisture damage are considered to be more important with regard to the performance of the asphalt pavement.

Fatigue cracking is caused by repeated applications of loading. Low temperature thermal cracking is caused by the development of thermal stresses that exceed the tensile strength of asphalt concrete at low temperatures. Rutting or permanent deformation is caused by progressive shear movement of materials under repeated loads at high temperature. Moisture damage or stripping of asphalt is caused by the lack of adhesion bond between the aggregate and asphalt due to presence of moisture. Stripping may contribute to rutting and fatigue cracking, which are the worst cases of pavement disintegration [Roberts, 1996].

Hot Mix Asphalt (HMA) should be able to withstand the traffic loading and environmental factors such as, temperature and moisture, in order to overcome common pavement distresses. HMA mainly consists of visco-elastic asphalt binder and aggregates compacted to form a matrix. The aggregate skeleton is used to carry and withstand the traffic loading applied to the aggregate asphalt mixture, whereas the asphalt binder serves as an adhesive holding the aggregate particles together [McGennis, 1995; Bolling, 1999]. Asphalt binder is addressed as a visco-elastic material because at higher temperatures the binder is largely viscous and less elastic, and at lower temperatures it behaves as an elastic solid material [McGennis, 1995].

2.1 Superpave Mixture Design

Superpave is an acronym for Superior Performing Asphalt Pavements and was introduced as a part of the Strategic Highway Research Program (SHRP) in 1993 [Roberts, 1996]. Since the 1940's the Hveem (ASTM D1560) and the Marshall (ASTM D1559) methods of mix designs have been used as they served the existing conditions. However these methods did not address the basic properties of compacted asphalt mixtures related to pavement design and field performance, since both were empirical methods [Little, 1990;Roberts, 1996; Bolling, 1999]. An empirical test does not measure a fundamental engineering property. If the Marshall stability is considered, for example, it is not fundamentally related to rutting, and thus, cannot be used to properly predict the performance of HMA under loading. It is known to have a marginal relationship with rutting [Bolling, 1999]. In 1987, SHRP was initiated with an allotted budget of $150 million for a 5-year period. One of the major objectives of this program was to come up with an improved mix design procedure that can be applied to various traffic volumes, axle loads and climatic conditions. The objective of the SHRP asphalt research program was to improve the pavement performance through a research program that would provide increased understanding of the chemical and physical properties of the asphalt cement and asphalt concrete. The results of the research would then be used to develop specifications, tests, etc., needed to achieve the performance and control of asphalt mixtures. It was intended that the final product would be performance-based specifications for asphalt, with or without modification, and the development of an Asphalt-Aggregate Mixture Analysis System (AAMAS) [Huber, 1993]. In 1993, a new mix design method called "Superpave" was introduced as a product of the SHRP research. The main features included a new grading system for the asphalt binders, Performance Graded (PG) grading system, aggregate specifications, new mix design procedure, and mixture testing and analysis procedures [Huber, 1993; McGennis, 1995; Superpave, 1995; Roberts, 1996]. The mix design represents an improved system for the design of pavement mixtures that are affected by traffic loading, environmental factors and structural section of the pavement in the field. The mix design selects the most suitable asphalt binder, aggregates and modifiers, if necessary. The procedure is applicable to virgin and recycled, dense graded HMA, with or without modification for use in overlays and new construction. The Superpave system mainly addresses minimization and control of three distresses namely, rutting, fatigue cracking and low temperature cracking. Moisture sensitivity and aging are also considered in material selection and mix design [Huber, 1993; Cominsky, 1994; McGennis, 1995]. Superpave is widely accepted and currently used in most states. Many research projects are being performed to refine the specifications, including test procedures and performance prediction models [Bolling, 1999].

In the PG grading system, the binders are specified based on the climate and the chosen level of reliablity. The requirements for the physical properties of the asphalt binders are the same, whereas the temperature at which the binder is supposed to achieve the properties changes depending on the climate [McGennis, 1995]. For example, the aged stiffness ($G^*\sin\delta$) of the asphalt binder should be below 5000 kPa to control fatigue cracking [Roberts, 1996], but if the binder is expected to serve at high temperatures, then the requirement should be attained at that high temperature. The PG binders are specified in the form PG X-Y. The first number 'X' is called the high temperature grade and it represents the temperature at which the particular binder should possess adequate physical properties. This temperature would be the maximum pavement temperature expected for the considered project. The second number 'Y', represents the lowest temperature at which this binder is expected to serve and the temperature at which the binder possess sufficient flexibility to prevent cracking. For example, PG 70-28 can be used with good performance characteristics for climate where maximum temperature of the pavement would be 70°C and the minimum temperature would be -28°C.

Aggregates play a significant role in overcoming pavement distress. They contribute to the stability of the mix. The stability is obtained by the shape and texture of the aggregate [Bolling, 1999; McGennis, 1995]. The Superpave system specifies aggregate properties used in pavement construction to account for different traffic levels. These aggregate properties are known as consensus properties and source properties. Consensus properties include Coarse Aggregate Angularity (CAA), Fine Aggregate Angularity (FAA), flat and elongated particles, and clay content. The CAA and FAA values are specified to obtain a high degree of internal friction and high shear strength to resist rutting. If the asphalt mixture has a certain percentage of crushed faces for the large size aggregates and if the mix can be properly compacted, the stability of the mix would increase. If smooth, round and poorly crushed aggregates are present in the mix, the stability of the mix would decrease and the pavement may undergo permanent deformation. The usage of flat, elongated particles is limited to avoid the breaking of aggregates during handling, construction and later by traffic. By placing limitations on the amount of clay in aggregates, the bond between the aggregates and the asphalt binder would be ensured. The source properties are toughness, soundness and deleterious materials [Superpave, 1995]. These properties are used to control the quality of the aggregates.

The Superpave mix design introduced a new compaction method that replicated field conditions better and the field validated conditioning procedures, such as short-term aging and long-term aging [Cominsky, 1994; McGennis, 1995]. In the mix design procedure, a Superpave gyratory compactor (SGC) is used to carry out the compaction of the Superpave mixture samples in the laboratory. SGC was found to be effective in simulating the real world compaction and ensures that the properties of the samples compacted in the laboratory are similar to the mix placed in the field [Cominsky, 1994]. The samples compacted using the gyratory compactor are cylindrical in shape with a diameter of 150 mm. The design gyrations for the mixes depends upon the project traffic. The analysis of the compacted samples is done in terms of percent of theoretical maximum specific gravity at three levels of compaction. These levels are initial number of gyrations ($N_{initial}$), design number of gyrations (N_{design}) and maximum number of gyrations ($N_{maximum}$) [D'Angelo, 1995]. Recent studies by Brown and Buchanan [1999], have recommended changes in selection of the number of gyrations. The PG binder specification requires asphalt binder aging to simulate short-term aging (aging during mixing, transportation and compaction) and long-term aging (aging during the first 5-10 years of service). The short-term aging is done in the rolling thin film oven test (RTFO) (AASHTO T240), while the long-term aging is achieved by additional aging of the binder using a Pressure Aging Vessel (PAV) (AASHTO PP1) [Cominsky, 1994].

Before compaction each loose mixture of asphalt and aggregate is placed in a tray and kept in an oven at a compaction temperature for two hours. This process simulates the mixing and placement of asphalt mixture in the field and the absorption of asphalt by the aggregates [Cominsky, 1994; Harrigan, 1994; McGennis, 1995]. The aggregate structure is selected based on the stockpile proportions and gradations that provide rut-resistance and enough space for asphalt to coat aggregates. Superpave introduced control points and a restricted zone, to establish the gradations. The optimum asphalt binder content is determined according to the estimation of performance and the volumetric requirements for air Voids in Total Mix (VTM), Voids in Mineral Aggregate (VMA) and Voids Filled with Asphalt (VFA), based on traffic and nominal maximum aggregate size in the mixture [Cominsky, 1993; Cominsky, 1994; D'Angleo, 1995]. The trial mixes are also subjected to a moisture sensitivity test using the AASHTO T283 tests or SHRP M-006 Method of test [Cominsky, 1994; Harrigan, 1994].

The other proposed components of the Superpave mixture design are the performance-based tests and performance prediction models for asphalt mixtures [McGennis, 1995]. To evaluate the performance of the design mix, various tests are carried out in the laboratory. The tests are performed using a shear test device and indirect tensile test device. The tests performed using a Superpave shear test device are frequency sweep test, simple shear test, uniaxial strain test, volumetric test, repeated shear at constant stress ratio and repeated shear at constant height. The tests performed using indirect tensile test device are indirect tensile creep test and indirect tensile strength test. The data obtained from these tests is used to come up with detailed predictions of the actual pavement performance with respect to Equivalent Single Axle Loads (ESALs) or time to attain a certain level of the rutting, fatigue cracking and low temperature transverse cracking [McGennis, 1995]. Research in this field is active and continuous changes of specification parameters, test methods and prediction models are being made.

2.2 Factors Affecting Rutting of Hot Mix Asphalt

Permanent deformation or rutting of a pavement is caused by repeated applications of traffic load at high temperatures and it usually appears as longitudinal depressions in the wheel paths accompanied by small projections to the sides [Aschenbrener, 1995; Roberts, 1996; Izzo, 1999]. Rutting is caused when the asphalt mixture becomes weak in shear strength to resist the repeated heavy loads. The pavements that undergo rutting cause serious problems due to accumulation of water in the channelized depressions formed, that may cause hydroplaning, accumulation of ice, and stripping of the HMA [Lai, 1989; Aschenbrener, 1995; Izzo, 1999]. As the pavement experiences increased stress and high temperatures, significant permanent deformation may take place [McGennis, 1995]. Figure 2.1 schematically shows the formation of channelized depressions along the wheel paths on the surface of the pavement. Figure 2.2 shows the rutting in HMA pavements with water accumulated in the channelized depressions.

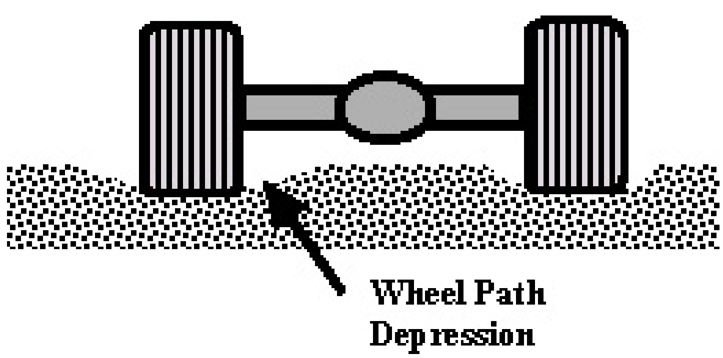


Figure 2.1 Formation of Channelized Depressions along the Wheel Path

Rutting is a complex phenomenon in which aggregate, asphalt and the aggregate-asphalt mixtures play an important role. As rutting is caused by the accumulation of very small plastic deformations due to repeated wheel loads, the plastic flow can be reduced by using a stiffer asphalt. When contribution of the aggregates is considered, aggregates with rough surface texture and angular shape, graded in such a manner that there is better contact between particles, would help prevent rutting [McGennis, 1995].

Figure 2.2 Rutting of HMA Pavements

The amount of air voids in the total mix (VTM) is another important factor that contributes to the rutting phenomenon on HMA pavements. If the air voids in the asphalt pavement is less than three percent, the pavement is prone to severe rutting. This is because the asphalt binder will reduce contact between the aggregates by acting as a lubricant. The target air voids during construction is about seven percent. This is done to ensure that the mix will attain air voids of approximately four percent after further densification of the pavement due to application of the traffic load [Brown, 1989; Roberts, 1996; Izzo, 1999]. Presence of moisture is considered to cause permanent deformation in some mixtures, since it could affect the bonding between the asphalt and the aggregate thus reducing resistance to the shear stresses [McGennis, 1995].

2.3 Factors Affecting Moisture Susceptibility of Hot Mix Asphalt

Stripping or moisture induced damage is another major concern when asphalt pavement-related distresses are considered. This pavement distress occurs when there is weakening of the adhesion or bond between the asphalt cement and aggregate surface in an asphalt pavement due to presence of moisture [Fromm, 1974; Hicks, 1991; Kandhal, 1992; Roberts, 1996]. When stripping starts at the surface and progresses downwards, it results in ravelling. HMA pavements derive their strength from the strong aggregate interlock and good adhesion between the aggregate and the asphalt cement. This adhesion is possible only when a firm bonding of asphalt and aggregates exists. Stripping of asphalt pavements occurs in five different mechanisms. These mechanisms are: detachment, displacement, spontaneous emulsification, pore pressure and hydraulic scouring [Roberts, 1996]. Figure 2.3 schematically shows stripping in asphalt pavements.

Understanding the basics of aggregate-asphalt adhesion and compatibility of various asphalt aggregate pairs and their sensitivity to water, can help prevent the problem of stripping. It is important to note that the mechanism of stripping involves penetration of water through the asphalt film at one or two points and subsequent displacement of asphalt from the aggregates [Fromm, 1974].

Various studies have showed that aggregate properties play a major role of stripping and adsorption when compared to the asphalt binder [Fromm, 1974; Kandhal, 1992]. The aggregates that are hydrophilic (attracted to water) are detrimental to the asphalt mix. Adsorption can be defined as adhesion in an extremely thin layer of molecules to the surface of solid bodies or liquids with which they are in contact. The difference in the adsorption behavior of different aggregates when combined with one single asphalt binder was far more than the adsorption behavior of the one aggregate combined with different asphalt binders [Al-Joaib, 1993]. The binders with high stiffness or viscosity were found to resist displacement better by water than the binders with low stiffness or viscosity [Roberts, 1996].

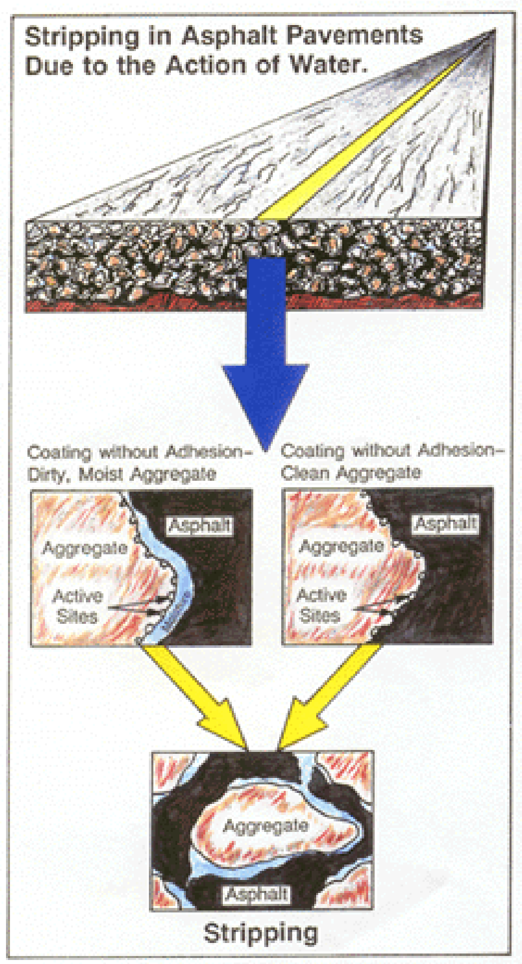


Figure 2.3 Stripping in Asphalt Pavements (http://www.quartzite.com/march97hdline.htm)

A number of field studies have shown that a number of variables influence the permeability of a pavement during its construction and subsequent service life [Zube, 1962]:

1. Segregation of mix during placing;
2. Temperature of mix during breakdown rolling;
3. Temperature of mix during pneumatic rolling;
4. Weight of the breakdown roller;
5. Tire or contact pressure of the pneumatic roller;
6. Ambient temperature during placing of mix;
7. Void content of the compacted mix; and
8. Amount of traffic before winter rains.

2.4 Superpave Aggregate Gradation

Aggregates constitute more than 90 percent of the asphalt mixture by weight. Gradation of the aggregate is considered important as it plays a significant role in providing stability to the asphalt mixture. In the Superpave mix design process, several requirements were introduced for the aggregate gradation. These included the Superpave gradation control limits, restricted zone, and the maximum density line plotted on a 0.45 gradation chart. This was done to ensure that the percentage of particles of maximum size present in the mixture is not too large or too small and to accommodate sufficient voids in the mineral aggregates [El-Basoyouny, 1999]. The restricted zone is an area surrounding the maximum density line from the 2.36 mm sieve to the 0.3 mm sieve. The combined aggregate gradations should avoid the restricted zone. The control points, along with the restricted zone, are used to control the shape of the gradation curve. Specifications require that all gradations should pass through the control limits and at the same time avoid the maximum density line and the restricted zone [Anderson, 1997]. This would provide a good aggregate structure that would enhance resistance of the mixture to rutting and also achieve sufficient void space for mixture durability. The restricted zone is used for two purposes. Gradations passing through the restricted zone have been observed to have problems meeting some compacted mixture properties, specifically the percentage of voids in the mineral aggregate (%VMA). Figures 2.4 and 2.5 show the 0.45 power gradation chart consisting of control points, restricted zone and the maximum density line, for nominal maximum aggregate sizes of 19 mm and 12.5 mm, respectively. The basic purpose of the restricted zone is to discourage the excessive use of fine natural sand in an aggregate blend [Anderson, 1997;Brown, 1999]. This serves to prevent gradations having a "hump" around the 1.18 and 0.6 mm sieves. In effect, the zone restricts the use of a high percentage of rounded sands. This is advantageous since excessive rounded aggregates are generally associated with poor shear resistance - a major cause of rutting in asphalt mixtures [Aschenbrener, 1994; Superpave, 1995]. Recent studies have shown that the gradations passing above the restricted zone (fine gradation) showed better performance than the gradation passing below the restricted zone (coarse gradation) [Adu-Osei, 1999]. It has also been found recently that mixes with gradations passing through the restricted zone are performing the same or even better than the mixes with gradations that are not passing through the restricted zone [Johnson, 1997]. Thus, KDOT has recently discontinued the use of the restricted zone in Superpave mixture gradations.

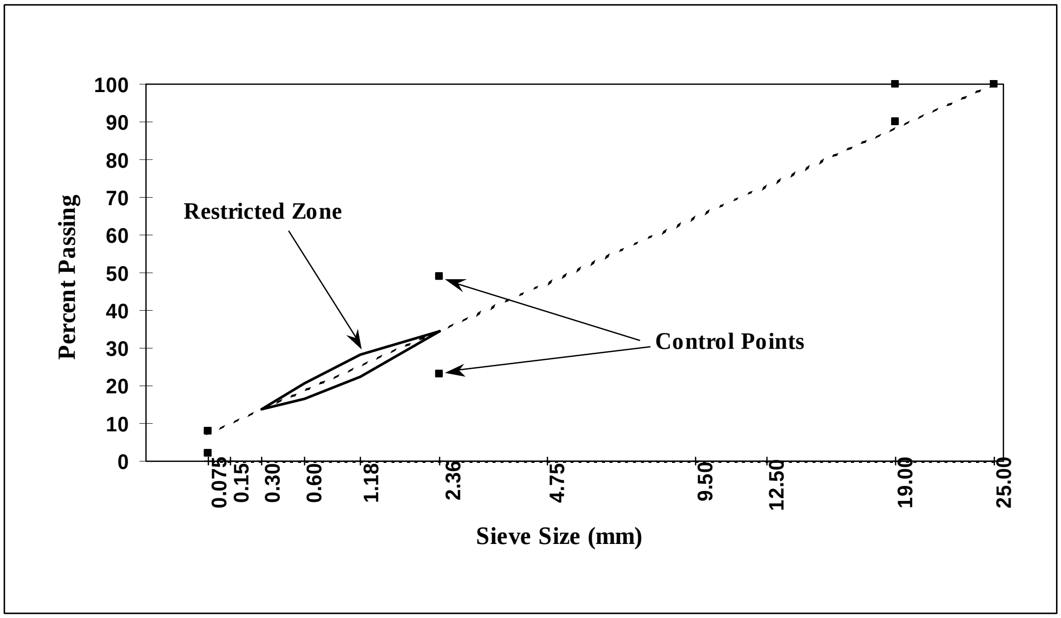


Figure 2.4 Gradation Chart for NMAS 19 mm Superpave Mix Design

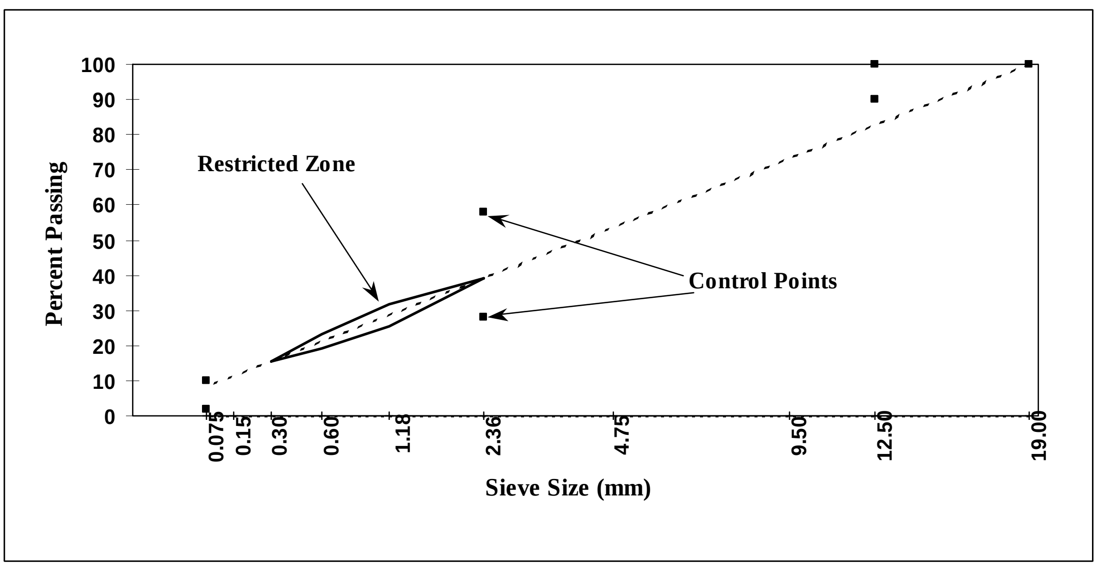


Figure 2.5 Gradation Chart for NMAS 12.5 mm Superpave Mix Design

2.5 Hydraulic Conductivity (Permeability)

Hydraulic conductivity or permeability is an important property of pavement materials. It can be expected that a pavement which is dense graded will prevent the percolation of water through it, while a pavement which is open graded, such as an open graded asphalt treated base, will have the maximum drainability so that the water will not stay in the pavement structure. Hydraulic conductivity and coefficient of permeability are the terms that generally mean the same, and are usually used interchangeably [Huang, 1999]. In this discussion Coefficient of permeability or hydraulic conductivity will be referred to by the term "permeability." Hydraulic conductivity is defined by Darcy's Law, which states that the fluid discharge velocity is directly proportional to the hydraulic gradient [Das, 1994] or it can also be defined as the volume of a fluid of unit viscosity passing through in unit time, a unit cross section of the porous medium, under the influence of a unit pressure gradient [McLaughlin, 1955]. Darcy's law depends on the flow condition and is only valid when the fluid travels at a very low speed in the porous media and no turbulence occurs. Usually, this criterion is not checked when applying Darcy's law to characterize flow in drainable layers of pavement materials.

2.5.1 Darcy's Law

In 1856, Henry Darcy investigated the flow of water in vertical homogeneous sand filters in connection with the fountains of the city of Dijon, France. From his study, he concluded that the rate of flow (volume of water per unit time, Q) is:

a. proportional to the cross-sectional area , A;

b. proportional to the water head loss, $h_1 - h_2$; and

c. inversely proportional to the length, L.

When combined together, these conclusions give the famous Darcy's Law or equation, which can be represented as:

$$Q = KA\frac{(h_1 - h_2)}{L} \qquad (2.1)$$

or

$$v = Ki \qquad (2.2)$$

Where, K is the proportional factor called hydraulic conductivity (or coefficient of permeability),

v = Q/A, is the discharge velocity; and

i = ∂h/∂L is the hydraulic gradient.

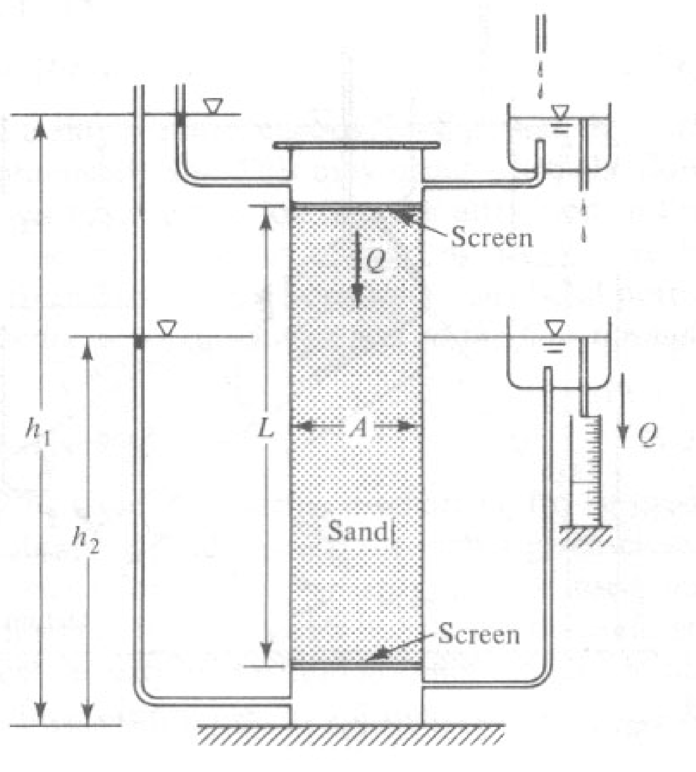


Figure 2.6 Darcy's Experiment

Figure 2.6 shows the experimental setup that Darcy used for measuring permeability. Later researchers have found that the coefficient of permeability depends on the following factors [Das, 1997]:

1. Viscosity of the fluid, i.e., permeant,
2. Percent of air voids present,
3. Degree of Saturation (permeability increases with increase in degree of saturation),
4. Size of the particles, through which the permeant flows, and
5. Temperature of the permeant.

2.5.2 Validity of Darcy's Law

Darcy's Law is valid for laminar flow through the void spaces of a pavement material. Darcy's Law does not take into account the variations in interstitial pressure associated with the inertia of the pore liquid as it moves around the grains or along the convoluted pathways between the grains of the material. If at some point the trajectory of the pore fluid has a radius of curvature, r, the fluid inertia sets up an additional pressure gradient $\rho \nu^2/r$, where ρ is the mass density of the fluid and ν is the pore velocity, which provides the centripetal acceleration associated with the curved trajectory [Huang, 1999]. Reynolds number, R, is used as a criterion for investigating if the flow is laminar or turbulent. In the range of Reynolds number between one and ten the viscous forces are more predominant compared to the inertial forces. The Reynolds number is a dimensionless quantity and can be defined as the ratio of the inertial and viscous forces acting on a fluid. In equation form it can be defined as:

$$R = \frac{vD\rho}{\mu} \qquad (2.3)$$

Where, ν = discharge (superficial) velocity, cm/s;

D = average diameter of the soil particle, cm;

ρ = mass density of the fluid, g/cm^3 ; and

μ = coefficient of viscosity, g/cm.s

If the value of R << 1 then the flow is known as a creeping flow. For laminar flow conditions the value of R is less than one [Das, 1997]. Research has also shown that for the validity of Darcy's Law, the Reynolds number should not exceed some value between one and ten (Figure 2.7).

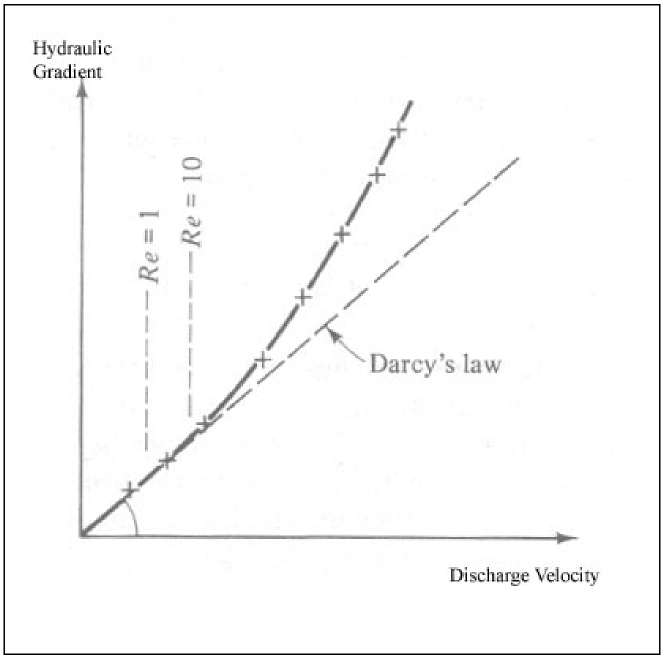


Figure 2.7 Schematic Curve Relating i to v [Bear, 1979]

2.5.3 Discharge Velocity

Discharge velocity is defined as the quantity of water flowing in unit time through a unit gross cross-sectional area of soil at right angles to the direction of flow [Das, 1994]. From Equation (2.2), the discharge or superficial velocity, "v" is based on the gross cross-sectional area of the soil. However, the actual velocity of water is the seepage velocity (v_s), which is greater than “v”. Equation (2.4) gives a relationship between the seepage velocity v_s and the discharge velocity, v.

$$v_s = \frac{v}{n} \tag{2.4}$$

In the above equation, n is the porosity of the material. When water flows through coarse sands, gravels and boulders, turbulent flow of water can be expected. The Reynolds number is then usually greater than the 1-10 range specified. There are two main equations to approximate the relationship of hydraulic gradient and flow velocity [Huang, 1999]:

Binomial Form: $i = av + bv^2$ (2.5)

Potential Form: $i = Cv^m$ (2.6)

In the above equations v is the discharge velocity, a, b and C are experimental constants, i is the hydraulic gradient and m describes the state of the flow. Though neither of the above two forms can be applied with unified material parameters, the second form, Equation (2.6), seems to be accepted more in the literature [Huang, 1999; Tan, 1997], with a validity zone being attached to a given value of power.

2.6 Previous Research on Hot Mix Asphalt (HMA) Permeability

Excessive infiltration of water into the pavement can damage both surface and subsurface layers. Water tightness or permeability of HMA is an important factor in design and construction of HMA mixes. Studies have been conducted on the problem of permeability of HMA mixes across the United States. The findings of different studies can be broadly classified into three major categories:

- Studies on moisture damage of HMA pavements
- Studies regarding development of an appropriate permeability measuring procedure for laboratory and field samples
- Studies on the effects of different material properties on permeability of the HMA mix

The following sections summarize the research in each of the above areas.

2.6.1 Studies on Moisture Damage of HMA Pavements

Stripping has been identified as the most significant problem of moisture damage in HMA pavements. Stripping is a complex process. The most common factors contributing to stripping in asphalt pavements has been identified as the inadequate surface drainage capacity of the pavements [Kandhal, 1989;Kandhal, 1992; Kiggundu, 1998]. Other factors that affect stripping in HMA pavements include inadequate compaction of the HMA, excessive coating of dust on the aggregates, inadequate drying of the aggregates and entrapment of sub base water in HMA overlays of concrete pavements. It has also been seen that in conventional pavement materials, the moisture content is usually between 0.34 % and 0.35 %. If the moisture content is greater than these values, there is a possibility of stripping [Kandhal, 1992].

2.6.2 Studies on Development of an Appropriate Permeability Measuring Device

Several studies have been conducted to develop an appropriate permeability-measuring device, in the field as well as in the laboratory, for HMA. The permeability measuring devices are based on the principle of falling-head permeability, which involves the measurement of only vertical permeability thorough the HMA material in the laboratory [Choubane, 1998; Cooley, 1999; Mallick, 2001]. But it is not possible to measure only vertical permeability in the field, as there is a lateral movement of water as well. Past studies have reported measurement of vertical permeability in the field and on cores obtained from the field [Westerman, 1998; Tan, 1999; Maupin, 2000]. Tan et al., [1999] developed a falling head permeameter, which was used for expedient measurement of field permeability. The measured pseudo three-dimensional permeability of the pavement material in that study had been converted to the constant permeability in all directions by performing a finite element analysis of the pavement section. It has also been shown that there could be considerable anisotropic behavior of permeability of the pavement material. The change of head over time had been assumed to be of a cubic polynomial type. It is expected that due to the complicated void structure of asphalt pavement materials, the actual variation of head over time is a complex phenomena rather than a simple cubic polynomial variation and needs to be verified through a detailed analysis of the flow mechanics [Tan, 1999].

2.6.3 Studies of the Effects of Material and Sample Properties on Permeability

Several studies have been conducted to study the effects of HMA materials on the permeability values. A study by Maupin [2000] examined the effect of sawing of HMA cylindrical samples on permeability. It was shown that the sawing process actually reduced the permeability by causing the smearing of asphalt and sealing off the voids. The effects of air voids and lift thickness have also been investigated in different studies. It has been found that a lift thickness equal to four times the nominal maximum aggregate size has to be provided to reduce the permeability of HMA mixes [Westerman, 1998]. Studies on the effect of aggregate size on permeability, has shown that an increase in the percentage of volume of solids to the total volume of the mix results in an increase in wear resistance and dynamic stability of the pavement.

CHAPTER 3

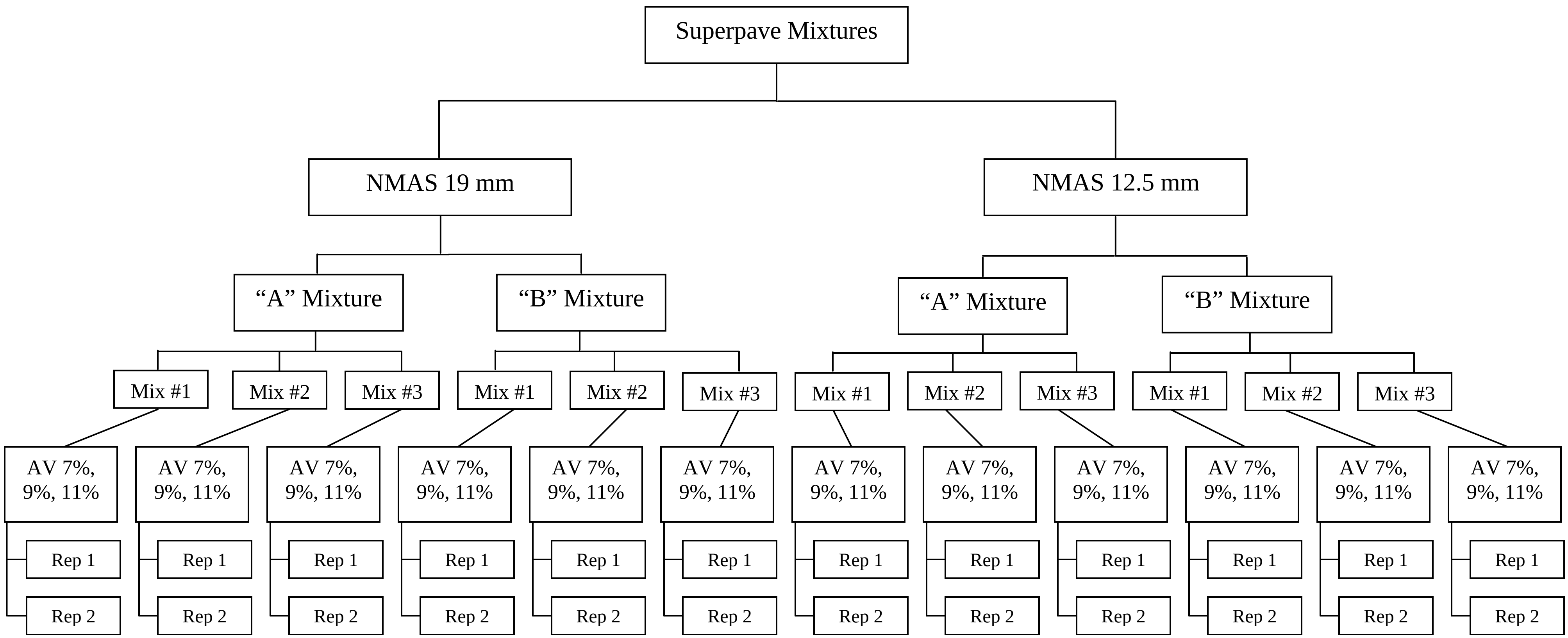
TESTING AND DATA COLLECTION

3.1 Introduction

As described in Chapter 1, percolation of water and air through the pavement can cause stripping and oxidation of the binder, rutting of the surface layer, and reduction of the pavement support from the underlying layers. Increased implementation of Superpave technology around the United States has created concern for this permeability issue. In most cases, a Superpave mixture is more open-graded when compared to the mixtures designed by the Marshall and Hveem methods. This is especially true for so-called "coarse" mixtures, where the combined gradation of the aggregate blend in a Superpave mixture passes below the restricted zone. This problem has been noticed particularly for the 19.0 mm nominal maximum size Superpave mixture in Kansas. Similar observations have been made in Arkansas and Florida [Choubane, 1998; Westerman, 1998]. The main objective of this research was to study and to determine how permeability of different Superpave mixtures in Kansas are affected by different mixture design parameters, so that recommendations to minimize permeability of Superpave mixtures could be developed.

3.2 Experimental Design

The statistical experiment design was a Completely Randomized Design (CRD). Both fine and coarse Superpave mixtures (SM) with 19 mm and 12.5 mm nominal maximum aggregate sizes (NMAS) were used in this study. SM designation in Kansas refers to the Superpave mixtures with virgin aggregates. Also in Kansas, fine mixtures, i.e. those Superpave mixtures with the combined gradation passing above the restricted zone are designated as "A" mixtures, while those passing below the restricted zone (coarse) as "B" mixtures. Each mixture type was sampled from three different projects, and replicate test specimens were prepared using the Superpave gyratory compactor at three different air void levels (7%, 9% and 11%). Figure 3.1 shows the experimental design used in the test program. These air void levels were selected to simulate different compaction levels achieved in Superpave pavement construction. The experiment involved a total of 12 mixtures (2 mixture sizes x 2 mixture types (fine/coarse) x 3 projects). The PG binder grade varied from PG 58-22 to PG 70-28. The asphalt content of the mixtures varied from 4.9% to 6.4%. All mixtures were used as binder courses in the asphalt concrete layer. Table 3.1 summarizes the characteristics of the mixes obtained from different Superpave projects. The mixture properties presented in Table 3.1 are the results obtained from the quality control tests during construction. These mixture properties satisfied the Superpave and current KDOT criteria. Figures 3.2 and 3.3 show the aggregate gradation charts for the 19 mm and 12.5 mm mixtures used in this study. It appears that some mixture gradations for the finer mixtures pass through the restricted zone. Also some mixture gradations appear to have a “hump" around the 1.18 and 0.6-millimeter sieves. A "humped" gradation is generally associated with a disproportionally high percentage of fine, rounded sand in the mixture. In effect, the restricted zone prepared by the Superpave research program restricted the use of a high percentage of rounded sands [Kandhal, 2001].



AV = Target Air Voids
Rep = Replicate

Figure 3.1 Experimental Design

Table 3.1 Properties of the Superpave Mixes used in the Study

Mixture/Aggregate Blend Property	Description	Design ESALs (millions)	N_{design}	PG Binder Grade	Binder Content (%)	Air Voids (%) at Ndes	VMA (%)	VFA (%)	Dust-Binder Ratio	%Gmm at Nini	% Gmm at Nmax
SM 19A(I)	KDOT Research Special	3	75	PG 58-22	6.2	4.1	13.7	59.8	0.9	87.6	97.2
SM 19A(II)	Venture US 169- 4A	2.9	75	PG 64-22	4.6	4.2	13.3	68.5	0.7	90.4	96.6
SM 19A(III)	Venture K US169-1A	2.9	75	PG 64-22	4.96	3.1	13.8	77.7	0.9	91.5	97.7
SM 19B(I)	Ritchie K-42	1	86	PG 58-28	5.5	3.8	14.4	73.9	0.85	89	97.3
SM 19B(II)	Shilling US 75 6C	0.2	68	PG 70-28	5	3.9	13.6	75.8	0.9	86.1	97.8
SM 19B(III)	Shilling US 75 9C	0.2	68	PG 58-28	5.1	2.6	13	74.4	0.9	86.7	97.9
SM 12.5A(I)	Henningsen K-56	1.1	86	PG 64-22	6.2	4.5	14.5	68.9	1.33	96	96.9
SM 12.5A(II)	Shilling K-4	0.9	76	PG 58-28	6.4	4.3	14.7	70.9	1.12	90.2	97
SM 12.5A(III)	Venture K-140	0.9	75	PG 64-22	4.9	4.7	14.6	68.2	1.21	89.2	96.0
SM 12.5B(I)	APAC Shears US-50	3.5	100	PG 70-22	4.85	6.6	15.2	56.3	1	84	94.5
SM 12.5B(II)	KAPA Junction City	1	76	PG 70-28	6.25	6.4	15.3	58.2	1.14	85.3	94.8
SM 12.5B(III)	Shillling K-54	3.3	96	PG 64-22	5.9	4.4	13.3	67.3	0.9	85.8	95.65

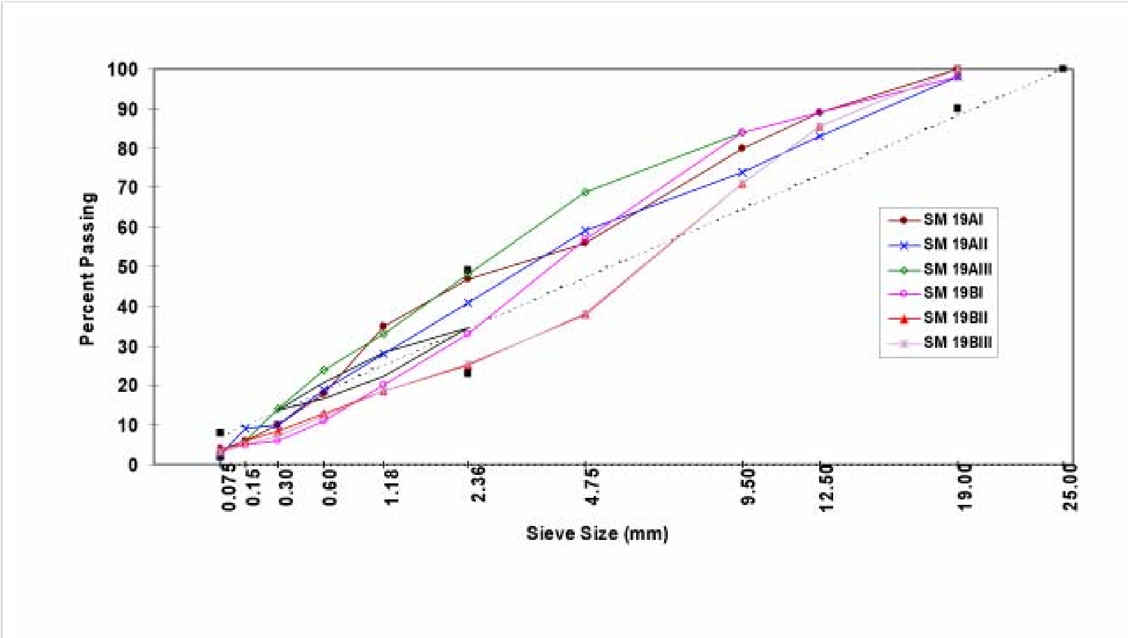


Figure 3.2 Aggregate Gradation Chart (SM 19 A & B Mixes)

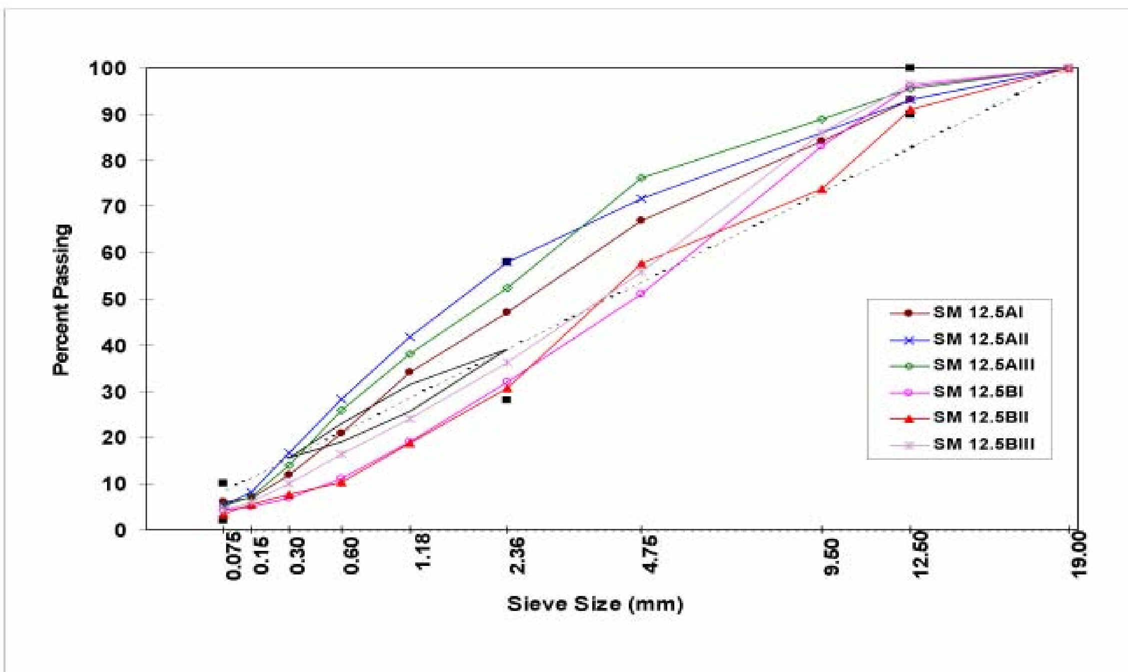


Figure 3.3 Aggregate Gradation Chart (SM 12.5 A & B Mixes)

Recently KDOT has been allowing contractors to have their gradation pass through the restricted zone on some of the finer sieves, provided all required volumetric, compactibility parameters (% Gmm at Nini and % Gmm at Nmax) and the dust-to-binder ratio criteria are fulfilled.

3.3 Laboratory Permeability Testing

The following sections describe the preparation of test specimens, the laboratory test equipment used to perform the permeability and rutting susceptibility tests on different mixes used in this study.

3.3.1 Permeability Test Specimen Preparation

Test specimens for permeability tests were compacted using a Pine Superpave gyratory compactor. The Superpave Gyratory Compactor (SGC) was developed so that HMA could be realistically compacted in the laboratory to densities achieved under actual pavement climate and loading conditions [Superpave, 1995]. Specimens of 150 mm diameter and accommodating aggregate up to 50 mm maximum (37.5 mm nominal) size can be produced using the compactor. The Superpave gyratory compactor consists of the following main components:

- Reaction frame, rotating base and motor
- Loading system, loading ram and pressure gauge
- Height measuring and recording system
- Mold and base plate

A loading mechanism presses against the reaction frame and applies a load to the loading ram to produce a 600 kPa compaction pressure on the specimen. A pressure gauge measures the ram loading to maintain constant pressure during compaction. The mix is placed in a mold, which has a diameter of 150 mm. The SGC base rotates at a constant 30 revolutions per minute during compaction with the mold positioned at a compaction angle of 1.25 degrees. Figure 3.4 shows a schematic of a Superpave gyratory compactor.

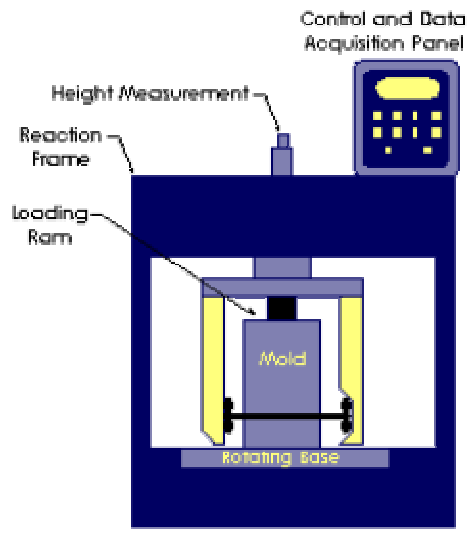


Figure 3.4 Superpave Gyratory Compactor

The mixes tested in the laboratory were obtained directly from the HMA plants. The mixes were reheated to compaction temperature in an oven before compaction in the Superpave gyratory compactor. As mentioned earlier, three air void levels (7%, 9% and 11%) were targeted. In order to achieve the different target air voids, the "Compact to specified height" method of compaction was used. In this mode of operation the Superpave gyratory compactor will apply, the pre set consolidation pressure and gyrate the specimen until the preset specimen height is reached. The weight of the mixture used to compact the sample was varied, to achieve the target air voids. The number of gyrations required to reach the given specimen height was then tabulated. In some cases it was observed that the preset height has been reached prior to the compactor stopping. This is because the compactor rounds off the specimen height on the compactor display to the nearest 0.1 mm and stops on the first gyration after the specimen height is actually reached. For each mix, two replicate specimens were made at the same target air void content and their results were averaged. Theoretical maximum specific gravity (G_{mm}) of the loose mixtures and bulk specific gravity (G_{mb}) of the compacted specimens were also determined. KDOT standard test methods KT-39 (AASHTO T209) and KT-15 (AASHTO T166) Procedure III were used to determine G_{mm} and G_{mb} , respectively [Hossain, 2001]. The air voids in the compacted specimen were calculated using the formula (3.1):

$$\%\,AirVoids = \frac{100 \times (G_{mm} - G_{mb})}{G_{mm}} \tag{3.1}$$

The actual air void of the gyratory compacted samples was found to vary from 6.18% to 11.61%. The compacted samples were saturated using vacuum saturation before permeability testing. The target saturation level was 100%, however, a value greater than 90 percent was considered satisfactory since it is not always possible to achieve 100% saturation.

It is understandable that permeability of a porous medium like the Superpave mixture depends, to a large degree, upon the interconnectivity of the air voids. It was assumed in this study that the asphalt film thickness (in microns) of the mixture may play a role in reducing interconnected voids, and was considered a factor during statistical analysis. The asphalt film thickness was calculated using the formula (3.2):

$$FilmThickness(average) = \frac{V_{asp}}{SA \times W} \tag{3.2}$$

Where:

V_{asp} = effective volume of asphalt cement, liters
SA = Surface area of the aggregate, m^2 per kg of the aggregate
W = weight of aggregate, kg

3.3.2 Rut Testing Specimen Preparation

A linear kneading compactor shown in Figure 3.5 was used to produce samples for use in the Hamburg wheel tester. The compactor used in this study has been manufactured by PMW, Inc. Two slab samples of 320 × 260 mm and 40 mm or 80 mm height can be produced. The samples were compacted to a known height; hence, the target air void of the compacted sample could be achieved easily. The mold is filled with a pre-determined mass of material from the knowledge of the theoretical maximum specific gravity of the mix. The sample was then compacted within ± 1% of the targeted air voids. A series of 12 mm (0.5 in) wide steel plates were placed on the loose mix in the mold (Figure 3.6). The downward motion of the roller applied a force to the top of each plate while the mold moved back and forth on a sliding table. A linear compression wave was produced in the mix by the bottom edges of the plates as the roller pushed down on each plate. This kneading action allowed the mixture to be compacted without fracturing the aggregates and was probably very similar to a steel wheel roller [Stevenson, 1994]. The compaction time was less than 10 minutes.

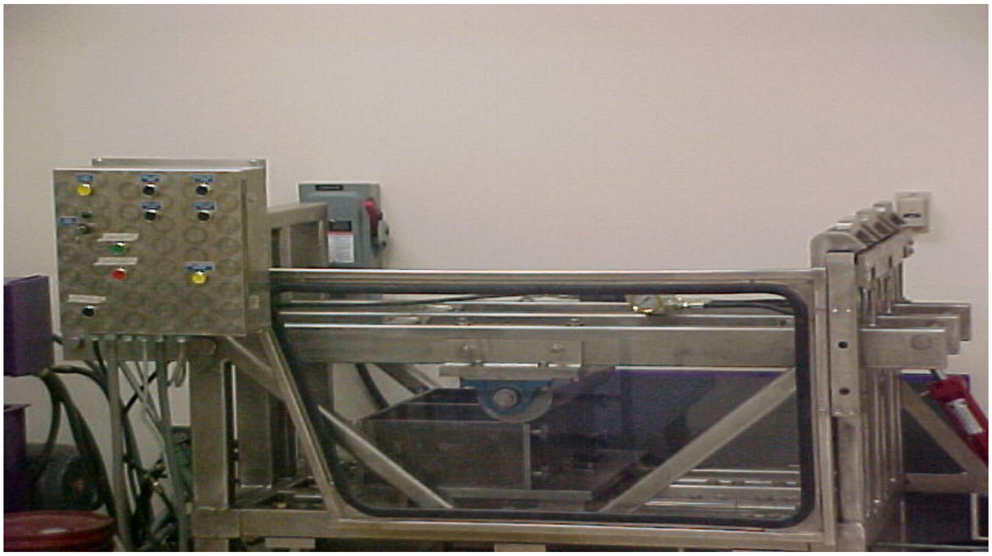


Figure 3.5 Linear Kneading Compactor

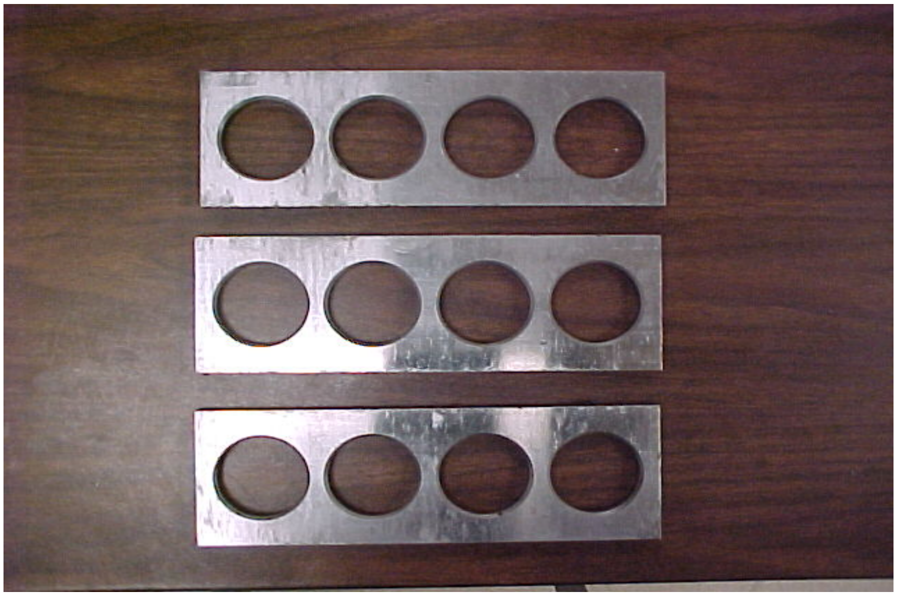


Figure 3.6 Steel Plates used In the Linear Kneading Compactor

3.3.3 Permeability Testing Device

Currently in the United States, there is no standardized method to measure the water permeability of the Hot Mix Asphalt specimens. Different agencies and investigators have been developing concepts and procedures for testing HMA [Huang, 1999; Hall, 2000; Lynn, 1997; Mallick, 2001; Maupin, 2000]. A discussion on the history and development of HMA permeability testing has been given by Lynn [Lynn, 1997]. This summary was prepared as part of an effort by the American Society for Testing and Materials (ASTM) subcommittee D04.23 to develop a “standard” HMA permeability test. Falling head permeability test was performed using an apparatus, which is currently under development by the ASTM Subcommittee D04.23. Figure 3.7 shows a schematic of the apparatus used in this study. This apparatus has been recommended by the ASTM Subcommittee D04.23.

The device currently used to measure the falling head permeability is also known as the Carol-Warner Flexible Permeameter. The apparatus consists of a metal cylinder, with a nominal

150 mm inside diameter, to accommodate the HMA specimens obtained from the gyratory compactor. A flexible latex membrane is placed on the inside diameter of the metal cylinder, where a confining pressure can be applied to the circumferential surface of the HMA specimen. This is done to prevent seepage of water through the sides of the sample and to make sure that the water passes through the top and bottom faces of the cylindrical specimen. The cylinder of the testing device has removable hard plastic plates and can be sealed when in use. The top plate has a hole with a graduated standpipe, in millimeters. The calibrated graduated cylinder has a diameter of 31.75 ± 0.5 mm and is capable of dispensing about 500 ml of water. Water level was observed in this standpipe while conducting the falling head permeability tests. HMA sample that had been compacted in the Superpave gryratory compactor to the required air void level and cooled were saturated to about 90 –100 % level. The samples were then coated with a thin layer of petroleum jelly around the circumferential surface prior to placement in the permeameter for testing. This was done to prevent the flow of water along the surface of the specimen. The specimen was then placed on the bottom plate of the permeameter and the metal cylinder containing the membrane was placed over the specimen. The top plate was then placed over the specimen in the metal cylinder and clamps were used to compress and seal the top and bottom plates. The attached graduated cylinder was then filled with water and the permeameter was tilted and gently tapped to remove air bubbles that may have been trapped above the HMA sample. A confining pressure of 96.5 ± 7 KPa was applied to the membrane surrounding the cylindrical specimen. The valve on the bottom of the permeameter was then opened to allow flow of water through the sample vertically. The graduated cylinder was filled with water up to the 50 cm mark, and the time taken for the water to flow down to the 0 cm mark was recorded.

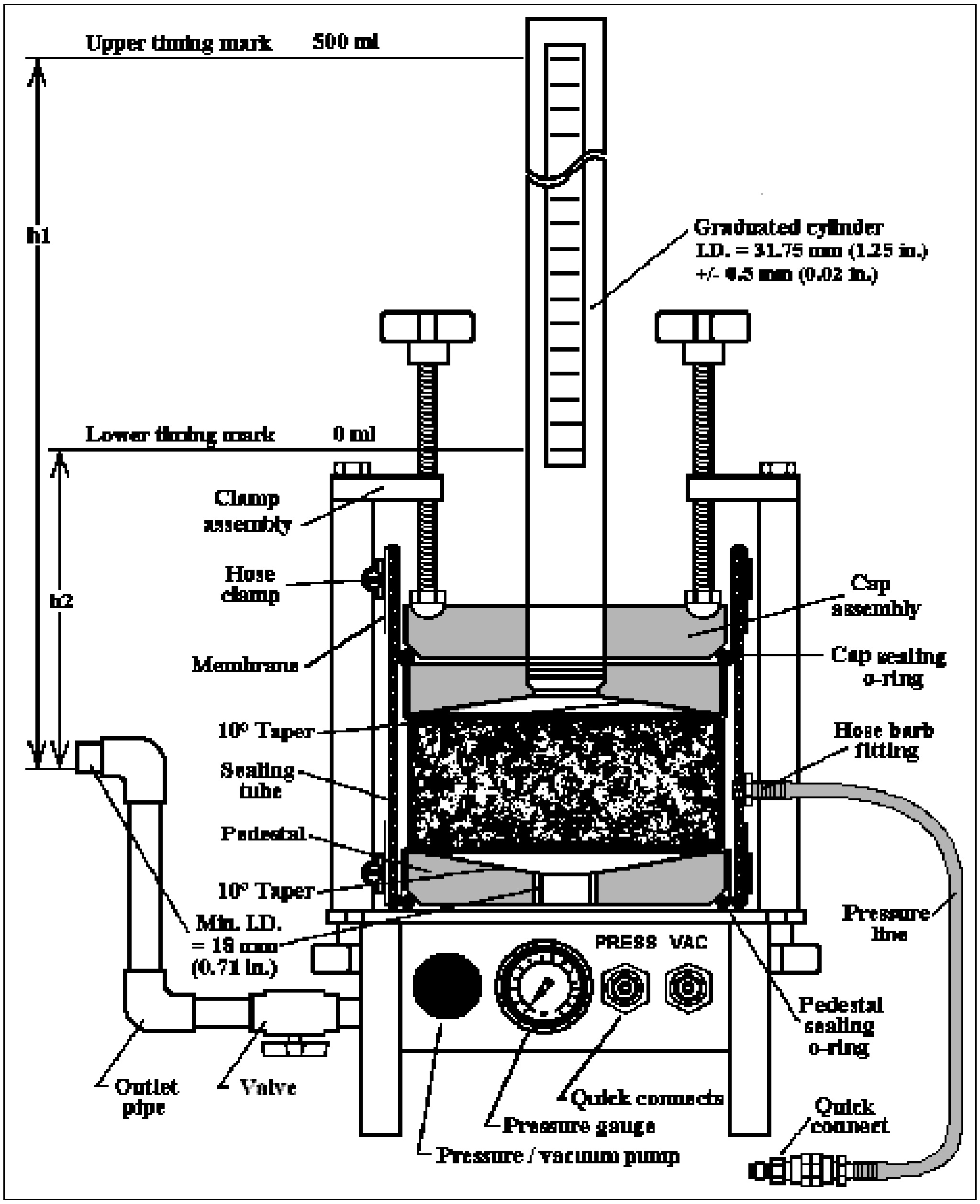


Figure 3.7 Water Permeability Testing Apparatus (Not to Scale)

Some samples were found to take more than 60 minutes for the water head to fall 50 cm, so the upper and lower water levels were noted for a 30 minute timed test. The graduated cylinder was refilled and the test was conducted again. The permeability was then calculated using the formula based on Darcy's Law, shown below [Lynn, 1997; ASTM, 1998; Huang, 1999; Hall, 2000; Maupin, 2000; Mallick, 2001]:

$$K = \frac{al}{At}\ln\left(\frac{h_1}{h_2}\right) \tag{3.3}$$

Where,

K = Coefficient of permeability in cm/sec;

a = inside cross sectional area of inlet standpipe in cm^2;

l = thickness of the HMA specimen, cm;

A = cross-sectional area of the HMA specimen, cm^2;

t = time taken for water to flow from h_1 to h_2, seconds;

h_1 = initial head of water, cm; and

h_2 = final head of water, cm.

The hydraulic conductivity (permeability) was then corrected to a temperature of 20°C (68°F), K_{20}, by multiplying the calculated K value with the ratio of viscosity of water at the test temperature to the temperature of water at 20°C (68°F), R_T as follows [ASTM, 1998]:

$$K_{20} = R_T K \tag{3.4}$$

Three permeability tests were performed on one sample and the results were averaged. Figure 3.8 shows the permeability-testing device that was used for conducting the falling head permeability tests in this study.

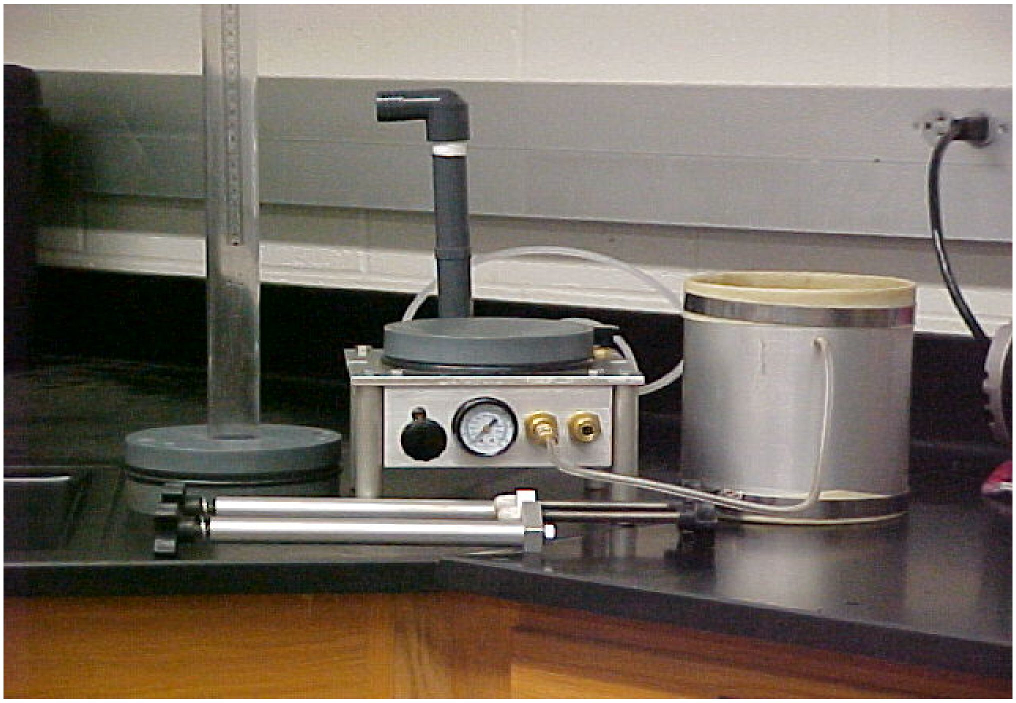


Figure 3.8 Laboratory Permeability Testing Device

3.3.4 Hamburg Wheel Tester

As discussed earlier, most common problems associated with HMA pavements are rutting and stripping. Both of these problems tend to occur during the early stages of a pavement life and trigger early undesirable maintenance actions. A number of tests that simulate the passage of traffic loads with laboratory-scale wheels moving repeatedly over an asphalt mix sample fabricated in the laboratory are currently in use. Many highway agencies have been using Loaded Wheel Testers (LWT's) for accelerated evaluation of the rutting and stripping potential of designed mixes [Lai, 1989; Aschenberner, 1995; Buchanan, 1997]. The absence of a mechanical test for the Superpave volumetric mixture has made this type of test very attractive for evaluating potentially undesirable mixtures. The Hamburg wheel tester is one such device that can be used to predict the rutting and stripping potential of asphalt mixes. The Hamburg wheel-tracking device used in this study has been manufactured by PMW, Inc. based out of Salina, Kansas and is capable of testing a pair of samples simultaneously. Wes Track Forensic

Team conducted a study on the performance of coarse graded mixes at Wes Track sections [Wes Track, 1998]. As part of the study the Hamburg Wheel Tester was used to study the performance of coarse graded HMA. In this study it was found that the correlation between the performance of the HMA mixes in the field and in the laboratory, using the Hamburg Wheel Tester, was high. It was also found that the Hamburg test has also been shown to identify mixes that tend to strip [Brown, 2001]. Figure 3.9 shows the Hamburg wheel tester at Kansas State University.

Figure 3.9 Hamburg Wheel Tester

The sample tested is usually 260 mm wide, 320 mm long and 40 mm deep. The slab sample has a mass of 7.6 kg and is compacted to 7 ± 1 % air voids. The samples are submerged under water at 45°C, although the temperature can be varied from 25°C to 70°C. The wheel of the tester is made of steel and is 4.7cm wide (Figure 3.10). The wheel applies a load of 705N and makes 52 passes per minute. Each sample is loaded for 20,000 passes or until 20mm deformation occurs. Figure 3.11 shows the HMA samples in the Hamburg Wheel tester, prior to testing. The maximum velocity of the wheel reached is 340 mm/sec, which is at the center of the sample. Around 6 to 6 ½ hours are required for a test for maximum of 20,000 passes. Rut depth or deformation is measured at 11 different points along the length of each sample with a Linear Variable Differential Transformer (LVDT). The various results that can be interpreted from the Hamburg Wheel Tester are the number of passes to 20 mm rut depth, creep slope, stripping slope, and the stripping inflection point as depicted in Figure 3.12 [Aschenbrener, 1995].

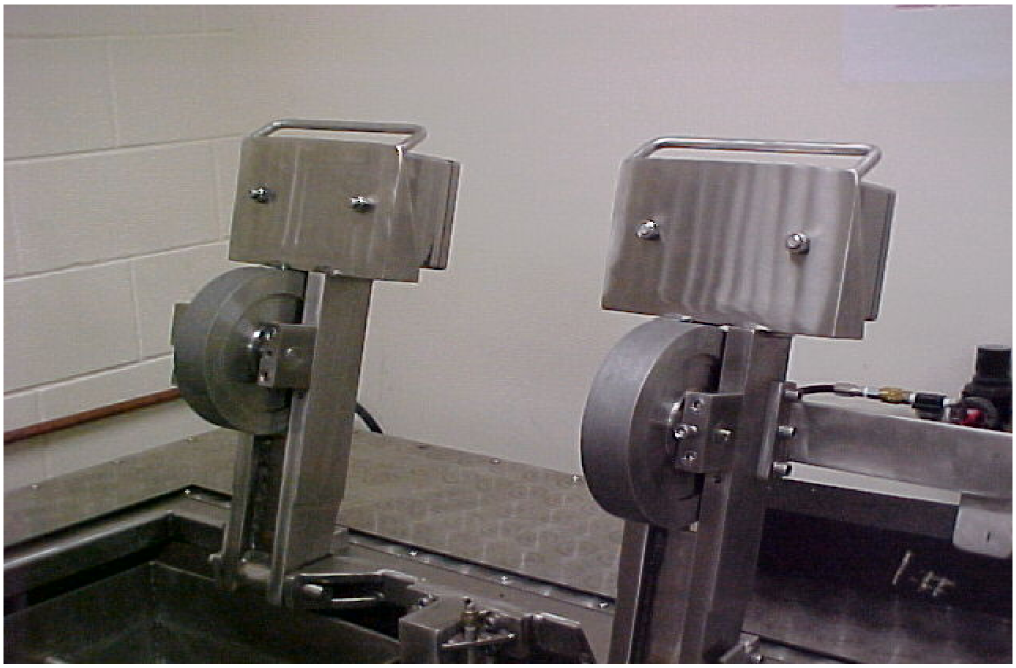


Figure 3.10 Steel Wheels of the Hamburg Wheel Tester

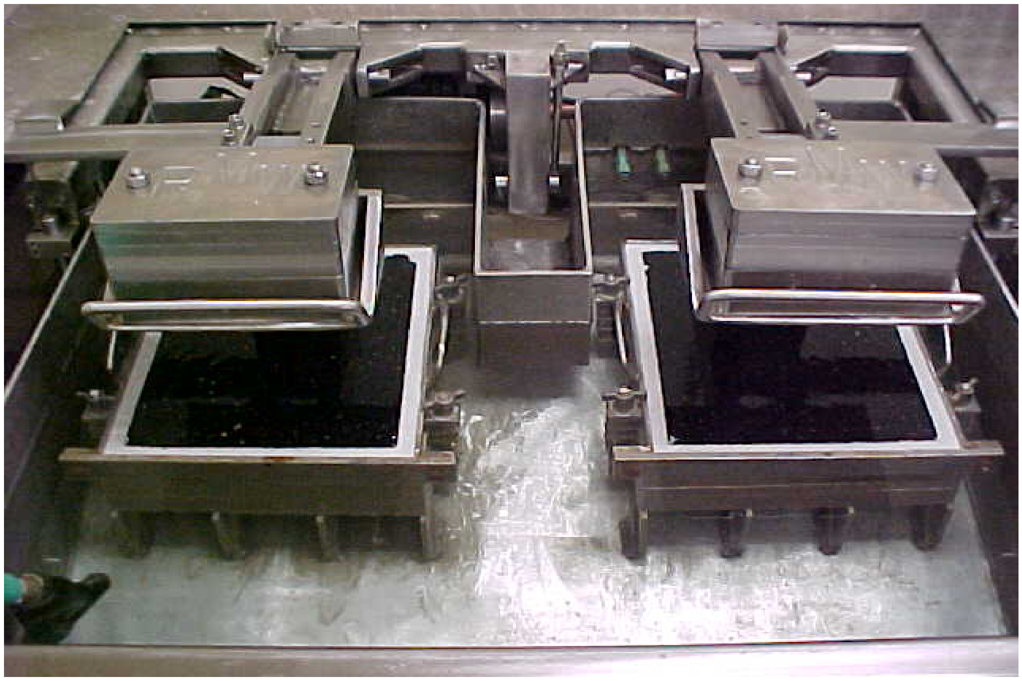


Figure 3.11 Loaded Samples in the Hamburg Wheel Tester

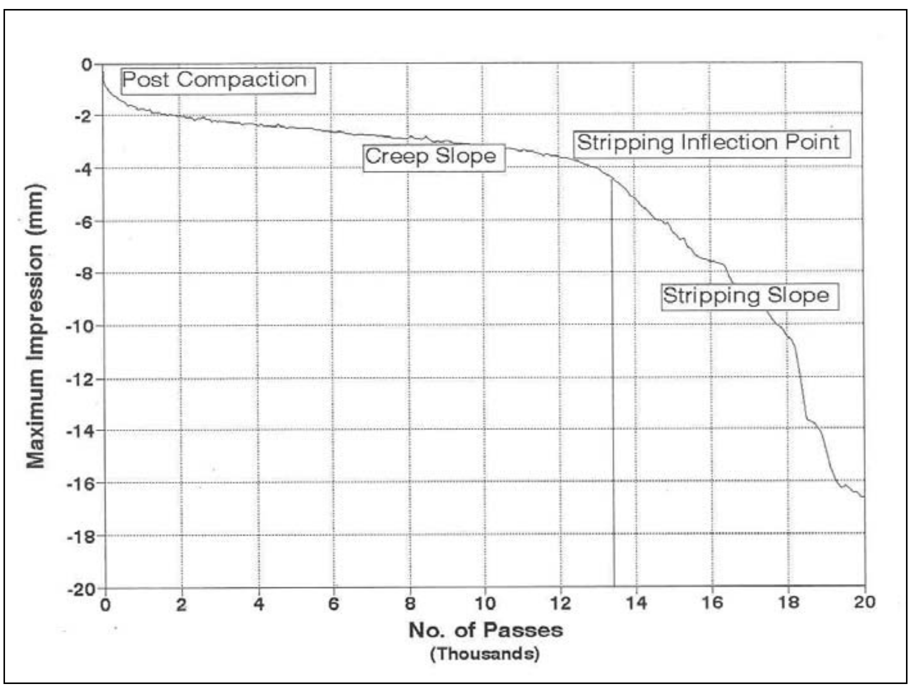


Figure 3.12 Interpretation of Results from the Hamburg Wheel Tester [Buchanan, 1995]

The creep slope relates to rutting from plastic flow and is the inverse of the rate of deformation in the linear region of the deformation curve, after post compaction effects have ended and before the onset of stripping. The stripping slope is the inverse of the rate of deformation in the linear region of the deformation curve, after stripping begins and until the end of the test. It is the number of passes required to create one mm impression from stripping, and is related to the severity of moisture damage. The stripping inflection point is the number of passes at the intersection of the creep slope and the stripping slope and is related to the resistance of the HMA to moisture damage. An acceptable mix is specified by the City of Hamburg to have less than 4 mm rut depth, after 20,000 passes, at 50°C test temperature [Aschenbrener, 1995].

<u>3.3.5 Air Permeability Tests</u>

Air permeability tests were conducted at the Bituminous Research Laboratory of the Kansas Department of Transportation following ASTM D3637-84 test method. The test method measures the rate at which air can be forced (pressure system) or drawn (vacuum system) at low pressure through bituminous mixtures. In this study, a vacuum system was used. In this method, the air permeability, *K* is derived from Darcy's law on the flow of fluids through a porous medium as follows:

$$K = : Q L / (A (P_1 - P_2)) = : V L / (TA (P_1 - P_2)) \qquad (3.5)$$

Where: *K* is the permeability, cm^2

: = viscosity of air = 1.885×10^{-7} gm-sec./cm^2

Q = volume rate of air flow, ml/sec

V = volume of air flow per test, ml

T = average time per test, sec.

L = height of sample, inches

A = area of sample, cm^2

$P_1 - P_2$ = pressure difference, inches of water.

3.3.6 Other Tests

The bulk specific gravity of the permeability test specimens (G_{mb}) was conducted using the KDOT test method, KT-15 (AASHTO T166). The theoretical maximum specific gravity of the HMA mixture was also conducted using KDOT test method, KT-39 (AASHTO T209) [Hossain, 2001]. The bulk specific gravity and the theoretical maximum specific gravity were used to calculate the actual percent air voids present in the gyratory compacted specimens at each of the target air void levels.

CHAPTER 4

DATA ANALYSIS AND DISCUSSION OF RESULTS

4.1 Introduction

As discussed earlier, permeability tests were performed on different Superpave mixtures, using a Carol-Warner Flexible Permeameter. Permeability was calculated based on the Darcy's equation since the flow of water through the cylindrical specimens prepared in the Superpave gyratory compactor is in one direction or the flow is a one-dimensional flow. The actual mix designs obtained from the contractor were used to get the design number of ESALs and the mixture properties. Laboratory tests were conducted to find the maximum specific gravity of the mix, the bulk specific gravity of the gyratory-compacted specimens, and finally the actual percentage of air voids present in the sample. An optimization study was done to find out the optimum values of different factors that affect permeability, based on a minimum value of permeability. The following sections present the results of the permeability tests conducted and a discussion of the results obtained.

4.2 Results of Water Permeability Testing

Permeability testing was conducted on the Superpave mixes with Nominal Maximum Aggregate Sizes (NMAS) of 19 mm and 12.5 mm. Twelve different Superpave mixes were chosen for this study, out of which six mixes had a NMAS of 19 mm and six other had a NMAS of 12.5 mm. In each size category, three mixes were of "A" type (with aggregate gradation passing above the maximum density line and above the restricted zone) and three mixes were of "B" type (with aggregate gradation passing below the maximum density line and restricted zone). Table 4.1 tabulates the permeability test results for all 36 samples (2 mixture sizes x 2 mixture types (fine or coarse) x 3 projects x 3 air voids). The permeability values were obviously higher for samples with higher air voids. In most cases, the finer (or “A”) mixes had lower permeability although some coarser (or “B”) mixes, such as, SM-19B on US-75, had very low permeability. When the results were ranked by permeability for the 7% air void samples, the SM-19A mixture designed by the KDOT research section had the lowest permeability of 1 x 10^{-6} cm/sec and the SM-12.5B mixture by APAC Shears on US-50 had the highest permeability of 217 x 10^{-6} cm/sec (Figure 4.1). However, another SM-12.5B mixture designed by the Kansas Asphalt Pavement Association for an intersection project had also the lowest permeability of 1 x 10^{-6} cm/sec.

Table 4.1 Summary of Water Permeability Test Results
* Measured Air Voids

Mix	Description	Air Voids* (%)	Permeability (x 10^{-6}) cm/sec	% Passing 4.75mm Sieve	% Passing 600 micron Sieve	% Passing 75 micron Sieve	P_b (%)	Film Thickness (microns)	Number of Gyrations
12.5AI	Henningsen K-56	6.5	190.6	67	21	6	6.2	9.66	19
12.5AII	Shilling K-4	6.4	21.9	72	28	5.2	6.4	8.33	17
12.5AIII	Venture K 140	6.2	136.0	76	26	5.4	4.9	7.33	8
12.5AI	Henningsen K-56	8.2	319.9	67	21	6	6.2	9.66	11
12.5AII	Shilling K-4	8.1	199.7	72	28	5.2	6.4	8.33	9
12.5AIII	Venture K 140	7.8	303.8	76	26	5.4	4.9	7.33	5
12.5AI	Henningsen K-56	10.0	1000.9	67	21	6	6.2	9.66	8
12.5AII	Shilling K-4	9.7	711.6	72	28	5.2	6.4	8.33	6
12.5AIII	Venture K 140	7.9	159.9	76	26	5.4	4.9	7.33	2
12.5BI	APAC Shears US-50	6.6	217.1	51	11	4.2	4.85	9.84	27
12.5BII	KAPA Junction City	6.5	1.0	58	10	0.5	6.25	21.08	19
12.5BIII	Shillling K-54	6.7	109.0	56	16	4.3	5.9	8.44	35
12.5BI	APAC Shears US-50	8.5	882.1	51	11	4.2	4.85	9.84	18
12.5BII	KAPA Junction City	8.3	52.4	58	10	0.5	6.25	21.08	16
12.5BIII	Shillling K-54	8.5	651.5	56	16	4.3	5.9	8.44	23
12.5BI	APAC Shears US-50	10.1	2410.2	51	11	4.2	4.85	9.84	11
12.5BII	KAPA Junction City	10.4	355.9	58	10	0.5	6.25	21.08	13
12.5BIII	Shillling K-54	10.3	1818.6	56	16	4.3	5.9	8.44	13
19AI	KDOT Research Special	6.7	1.0	56	18	3.9	6.2	9.43	40
19AII	Venture US 169 – 4A	7.7	97.0	59	19	2.6	4.6	10.41	36
19AIII	Venture US 169- 1A	7.2	38.8	69	24	3.9	4.96	8.74	29
19AI	KDOT Research Special	8.7	22.9	56	18	3.9	6.2	9.43	23
19AII	Venture US 169– 4A	8.6	174.5	59	19	2.6	4.6	10.41	14
19AIII	Venture US 169 -1A	7.9	569.3	69	24	3.9	4.96	8.74	7
19AI	KDOT Research Special	10.6	133.1	56	18	3.9	6.2	9.43	15
19AII	Venture US 169– 4A	10.1	511.0	59	19	2.6	4.6	10.41	7
19AIII	Venture K 169 - 1A	9.6	256.6	69	24	3.9	4.96	8.74	5
19BI	Ritchie K-42	6.9	68.9	57	11	4	5.5	12.09	27
19BII	Shilling US 75 6C	6.9	1.0	38	13	3.4	5.0	11.25	53
19BIII	Shilling US 75 9C	6.3	37.4	38	12	3.2	5.1	12.60	25
19BI	Ritchie K-42	8.9	1273.1	57	11	4	5.5	12.09	11
19BII	Shilling US 75 6C	8.6	61.1	38	13	3.4	5.0	11.25	57
19BIII	Shilling US 75 9C	9.6	142.7	38	12	3.2	5.1	12.60	19
19BI	Ritchie K-42	10.0	813.0	57	11	4	5.5	12.09	8
19BII	Shilling US 75 6C	10.5	318.5	38	13	3.4	5.0	11.25	38
19BIII	Shilling US 75 9C	11.6	2779.3	38	12	3.2	5.1	12.60	15

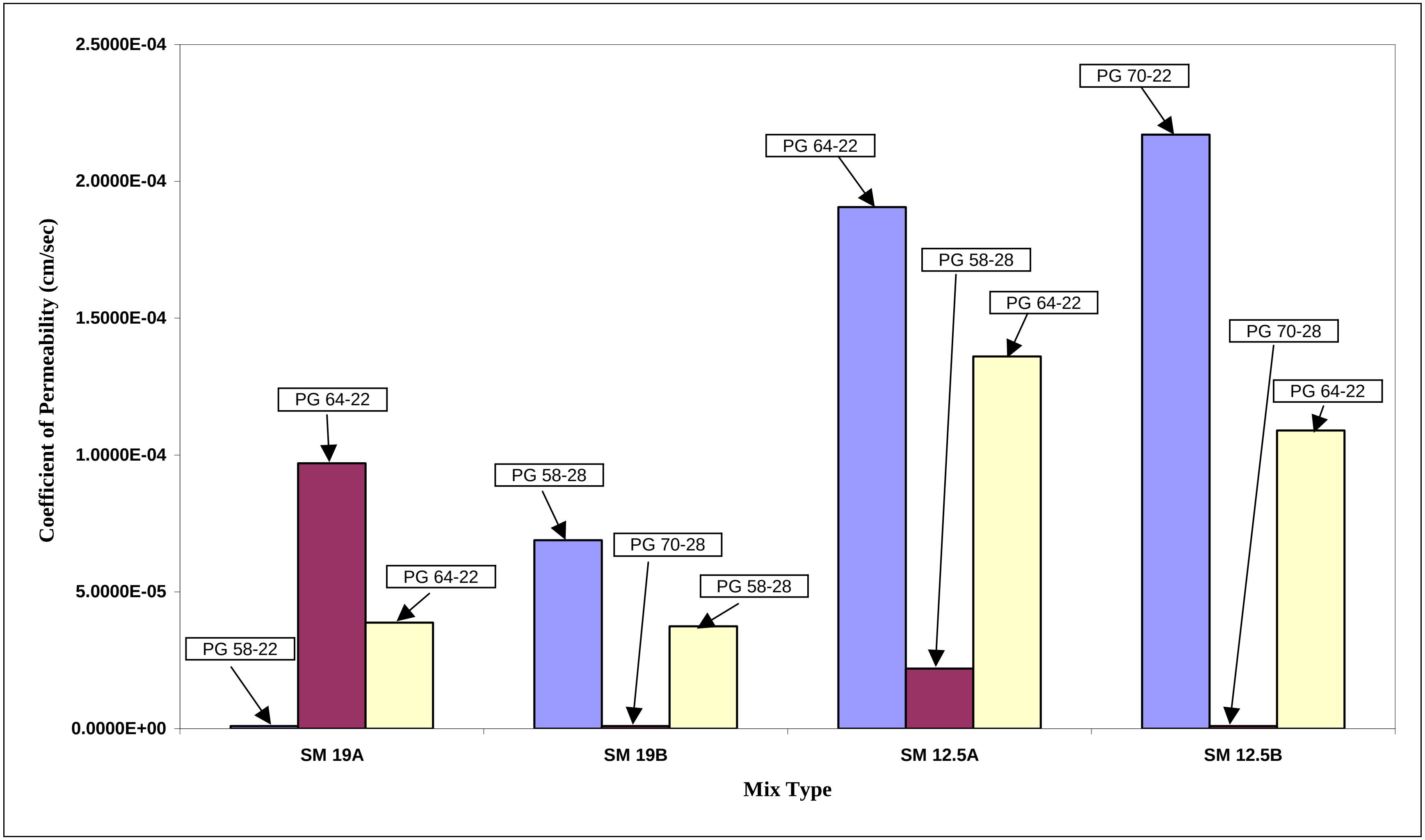


Figure 4.1 Water Permeability Test Results for 7% Target Air Void Samples

4.3 Results of Air Permeability Testing

Air permeability tests were performed on the samples that were used for water permeability testing earlier. Two samples of Ritchie K-42 mix could not be tested due to damaged samples. Table 4.2 tabulates the results of these tests. The results indicate that air permeability values are higher for mixtures with higher air voids. In general, for a given air void, higher nominal maximum size resulted in higher permeability as illustrated in Figure 4.2 for 7% air voids. The Shilling US 75 9C mixture (19 mm NMAS coarse mixture) had the highest air permeability (15,924 x 10^{-10} cm^2) and the Venture 169-1A mixture (19 mm NMAS fine) had the lowest air permeability (40 x 10^{-10} cm^2). In general, the finer mixtures had lower air permeabilities than the coarser mixtures for both nominal maximum aggregate sizes.

Figure 4.3 shows the scatter plot of the air and water permeabilities at all air void levels for all mixtures. The plot shows in general a definite trend that when the water permeabilities increase, the air permeabilities also increase. However, the large scatter of the data indicates that any definite relationship cannot be developed at any air void level.

Table 4.2 Summary of Air Permeability Test Results

Mix	Description	Target Air Voids (%)	Air Voids* (%)	Water Permeability (x 10^{-6}) cm/sec#	Air Permeability (cm^2)#
12.5AI	Henningsen K-56	7	6.5	190.6	231
12.5AII	Shilling K-4	7	6.4	21.9	155
12.5AIII	Venture K 140	7	6.2	136.0	247
12.5AI	Henningsen K-56	9	8.2	319.9	664
12.5AII	Shilling K-4	9	8.1	199.7	1234
12.5AIII	Venture K 140	9	7.8	303.8	612
12.5AI	Henningsen K-56	11	10.0	1000.9	579
12.5AII	Shilling K-4	11	9.7	711.6	7318
12.5AIII	Venture K 140	11	7.9	159.9	345
12.5BI	APAC Shears US-50	7	6.6	217.1	1417
12.5BII	KAPA Junction City	7	6.5	1.0	441
12.5BIII	Shillling US-54	7	6.7	109.0	573
12.5BI	APAC Shears US-50	9	8.5	882.1	3947
12.5BII	KAPA Junction City	9	8.3	52.4	452
12.5BIII	Shillling US-54	9	8.5	651.5	4421
12.5BI	APAC Shears US-50	11	10.1	2410.2	10120
12.5BII	KAPA Junction City	11	10.4	355.9	2484
12.5BIII	Shillling US-54	11	10.3	1818.6	7325
19AI	KDOT Research Special	7	6.7	1.0	108
19AII	Venture US 169 – 4A	7	7.7	97.0	372
19AIII	Venture US 169 –1A	7	7.2	38.8	40
19AI	KDOT Research Special	9	8.7	22.9	201
19AII	Venture US 169– 4A	9	8.6	174.5	1633
19AIII	Venture US 169 – 1A	9	7.9	110.6	2184
19AI	KDOT Research Special	11	10.6	133.1	438
19AII	Venture US 169– 4A	11	10.1	511.0	3513
19AIII	Venture US 169 – 1A	11	9.6	256.6	2039
19BI	Ritchie K-42	7	6.9	68.5	349
19BII	Shilling US 75 6C	7	6.9	1.0	2205
19BIII	Shilling US 75 9C	7	6.3	37.4	2294
19BI	Ritchie K-42	9	8.9	1273.1	**
19BII	Shilling US 75 6C	9	8.6	61.1	2565
19BIII	Shilling US 75 9C	9	9.6	142.7	15771
19BI	Ritchie K-42	11	10.0	813.0	**
19BII	Shilling US 75 6C	11	10.5	318.5	4617
19BIII	Shilling US 75 9C	11	11.6	2779.3	15924

* Measured Air Voids
** Damaged Sample
# average of two replicate measurements

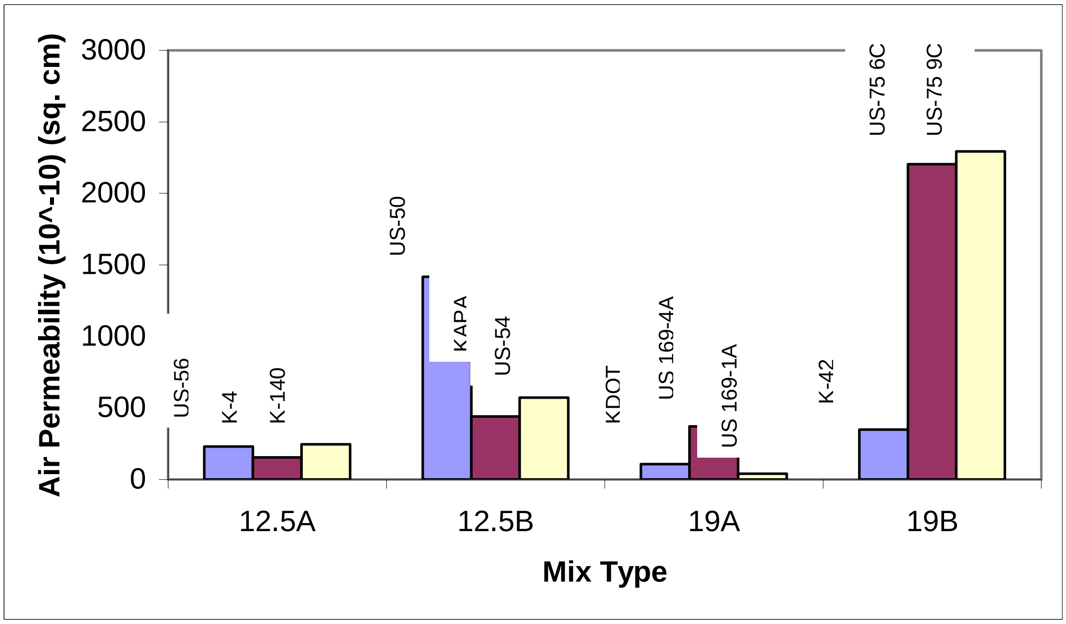
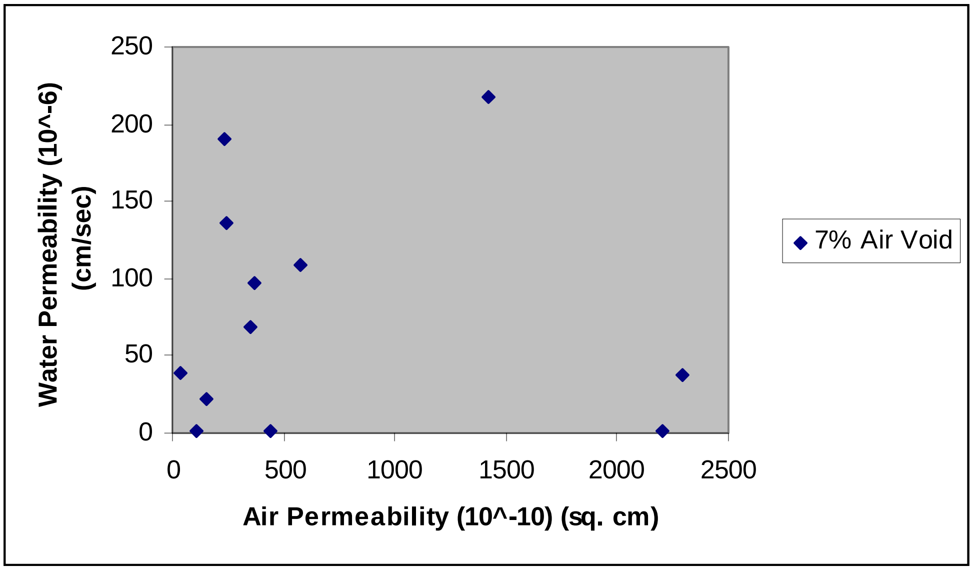


Figure 4.2 Air Permeability Test Results for 7% Target Air Void Samples



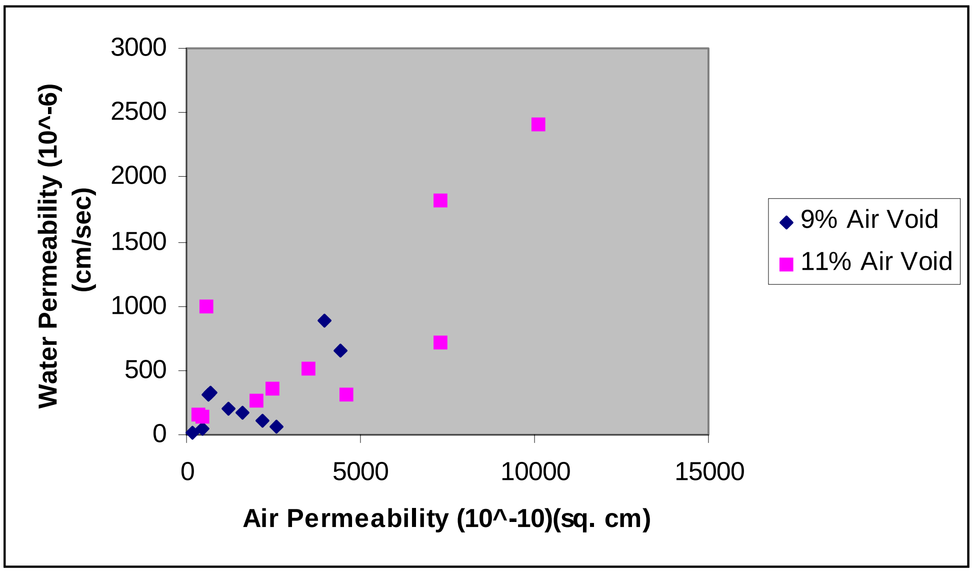


Figure 4.3 Correlation Between Air and Water Permeability Test Results

4.4 Evaluation of Rutting and Stripping Potential of the Mixes

Table 4.3 shows the Hamburg rut tester results, ranked by the average number of passes, for some of the Superpave mixes in this study. From Table 4.3 it can be observed that the best performing mixtures, in terms of number of repetitions to reach 20 mm rut depth and average creep slope, are both 19 mm NMAS mixtures. The stripping performance of these mixtures is also the best. The SM-19A (with PG 58-22) mix has been designed by the KDOT Research Section and the SM-19B (PG 70-28 binder) mix has been used as a binder course in an overlay project on US-75. It is to be noted that these mixtures also had the lowest permeability among all mixtures. For 12.5 mm NMAS mixtures, the coarse or "B" mixtures, out performed the "A" mixtures. Both coarse graded mixtures had higher binder grade than the "A' mixtures. However, the SM-12.5 A mixture on Highway K-140 with a PG 64-22 binder exhibited very similar rutting as the "B" mixtures, but stripped early. This mixture also had higher permeability. The worst performing mixture was SM-19B (PG 58-28) of Ritchie Paving Corporation on Highway K-42. This mixture had the fifth highest permeability of all mixtures tested in this study. The mixture had the highest amount of river sand (39% by total mixture) in it.

The bad performance of this mixture can be explained as follows: During Hamburg wheel tests, the applied heat (45^0 C) during test tends to soften the asphalt coating of the aggregates. Simultaneously the moisture weakens the bond between the asphalt and the aggregates. Although the traditional concept of stripping applies mainly to coarser aggregates, it was noticed that samples under wheel load lost the fines rapidly (test water became muddy) when the stripping slope increased rapidly. At that point, the Superpave mixture disintegrated and the rut depth increased significantly. Finally the mixture reached the maximum threshold rut depth (20 mm) with a lower number of wheel repetitions.

Table 4.3 Summary of Hamburg Wheel Test Results (Ranked by Average Number of Passes)

Mix Type	Description	Number of Passes		Average Number of Passes to 20 mm Rut Depth	Average Creep Slope	Average Stripping Inflection Point	Average Stripping Slope
		Specimen 1 (Left)	Specimen 2 (Right)				
SM 19BI	Ritchie K-42	1440	1320	1380	117	755	66
SM 12.5AII	Shilling K-4	5421	5890	5656	544	3696	430
SM 12.5AIII	Venture K-140	8861	15701	12281	1333	8923	420
SM 12.5BI	APAC Shears US 50	13640	11560	12600	1270	10240	551
SM 12.5BII	KAPA Junction City	13120	12321	12721	954	10311	788
SM 19AII	Venture K-140-4A	12941	13721	13331	1214	8347	501
SM 19AI	KDOT Research Special	20000	16161	18081	2667	14521	1333
SM 19 BII	Shilling US 75 6C	19981	20000	19991	12413	14614	6667

This may indicate that, at higher temperatures, the structural integrity of the coarse Superpave mixture depends not only upon the stone-on-stone contact but also on the cohesion of the matrix provided by the asphalt binder. Thus, the mixtures that had lower permeability and higher PG binder grade performed well, irrespective of their gradation (coarse or fine). These observations were based on the Hamburg Wheel tests, which were conducted at 45^0. Also, it may be desirable to limit natural sand in Superpave mixtures even if the mixture with higher natural sand satisfies all volumetric, compactibility and dust-to-binder ratio requirements. KDOT has already revised the specifications for the Superpave mixtures to include higher fine aggregate angularity requirements for the fine aggregates. This effort was due partly to the concern about natural sand.

4.5 Statistical Analysis of Test Results

In order to study the different factors that affect the permeability of Superpave mixtures a statistical analysis was done. Multiple regression analysis was used to identify different factors influencing permeability. The following sections describe the multiple regression analysis used.

4.5.1 Background

Regression Analysis is a statistical tool, which uses the relation between two or more quantitative variables so that one variable can be predicted from the other or others [Neter, 1974]. Multiple regression analysis is helpful for developing predictive equations consisting of a dependent variable and several independent variables. It mainly identifies and isolates those independent variables, which have the largest impact on the dependent variable. Each variable is given an impact level (regression coefficient), which signifies the independent variable's level of influence on the dependent variable.

The main use of multiple regression analysis is to find a correlation between the independent and the dependent variables. In its elementary form, positive correlation between an independent and a dependent variable means that as the independent variable increases, the dependent variable also increases. The correlation between more than one independent variable and a dependent variable is determined in multiple correlation and an equation known as a regression model is developed as a result of this analysis [Boyer, 1999].

In order to find the factors that influence the permeability of Superpave mixtures, the following independent variables that may affect the permeability of Superpave mixtures were considered in the regression analysis:

(i) Percent air void in the compacted Superpave sample,
(ii) Percent material passing 4.75 mm (No. 4) sieve,
(ii) Percent material passing 600 micron (No. 30) sieve,
(iii) Percent material passing 0.075 mm (No. 200) sieve,
(iv) Effective asphalt content,
(v) Percent asphalt absorption (P_{ba}),
(vi) Asphalt film thickness, and
(vii) The number of gyrations required to reach the target air void content (7%).

The general form of the multivariable linear regression model that was considered is:

$$\text{Permeability} = a + bX_1 + cX_2 + dX_3 + \ldots\ldots. \quad (4.1)$$

In Equation (4.1), permeability is the dependent variable and; X_1, X_2 and X_3 are independent variables; and a, b and c are the linear correlation coefficients. It may be observed in many cases that two or more independent variables may have correlation between them.

4.5.2 SAS Analysis

SAS (Statistical Analysis System) was used to conduct the statistical analysis in this study. SAS is a computer program for statistical analysis of data. The system is capable of information storage and retrieval, data modification and programming, report writing, statistical analysis and file handling. The statistical analysis procedures, which were used in this study, are one of the most widely used and finest available. The procedures range from simple descriptive statistics to complex multiple variable techniques. The flagship of the SAS program is its capability to handle linear model procedures [Helwig et al, 1979]. SAS has the ability to perform multiple regression analysis on large data sets and is designed to extract the maximum amount of information from the data set. It will determine the relationship between a dependent variable and one or more independent variables (X's). With the help of the information provided by SAS a model can be assembled. Other features that SAS is capable of are [Helwig et al., 1979]:

1. Distinguish independent variables which most significantly impact the dependent variable form those that do not (superfluous variables);
2. Determine an operative relationship which quantifies how the significant independent variables impact the dependent variables;
3. Determine the accuracy of the predicted variable;
4. Determine the certainty of the linear coefficients;
5. Determine the total variation of the data which is described by the model built (R^2); and
6. Provide simple statistics of the data set.

4.5.3 Model Selection Criteria

The models in this study were selected on the basis of the following criteria:

1. **R^2 Value:** The R^2 value is also known as the coefficient of multiple determination and has values ranging from zero to one. The R^2 value reflects the amount of total variation of the dependent variable explained

by the model. A value of one indicates that all variation is represented and explained by the model while a value of zero indicates none of the variation is explained or represented. Variation not explained by the model could indicate the results of variables not included in the data set, errors in the data, or different uncontrollable effects. The coefficient of multiple determination is calculated as follows:

$$R^2 = \frac{SSR}{SST} = 1 - \frac{SSE}{SST} \qquad (4.2)$$

Where,

SSE = Error or Residual Sum of Squares;

SSR = Regression Sum of Squares; and

SST = Total Sum of Squares.

R^2 is used as a criterion to check how well a model will predict a dependent variable. The value of R^2 is used for linear regression models and it depends on the information that has been sampled. However, it is important to note that, R^2 always increases as the number of independent variables in the models increases, even though each additional variable may have very little predictive power. Consequently, it is also imperative to evaluate the Mean Square Error (MSE) when determining the quality of the model [Neter, 1974].

2. **Mean Square Error (MSE):** Each linear regression model has an associated MSE or variance (σ^2). MSE is used to produce confidence intervals and test statistics. Small MSE values will result in narrow confidence intervals and large test statistics. Narrow confidence intervals and large test statistics distinguish good models from poor models. However, the model with the smallest MSE may not provide a model that is explainable and sensible. A model with a small MSE and small number of relevant variables is usually more desirable than a model with a very small MSE and a large number of unexplained variables [Ott, 2001]. The

MSE and R^2 values can be related to each other using the following relation:

$$R^2 = 1 - \frac{n(MSE)^2}{SST} \qquad (4.3)$$

Where,

R^2 = Coefficient of Multiple Determination

MSE = Mean Square Error;

SST = Total Sum of Squares; an

n = Sample Size

3. **Model Utility Test (F Test):** The best approach to test the overall effectiveness of a model is to conduct a test involving all linear coefficients simultaneously. The F test will test the hypothesis that all of the linear coefficients are zero simultaneously. If at least one of the linear coefficients cannot be zero then the predictor model obtained from the model selection will generally predict a dependent variable accurately. The general form of the F statistic is:

$$F = \frac{MeanSquareforModel(MSModel)}{MeanSquareforError(MSE)} \qquad (4.4)$$

The calculated F statistic is compared to that of a tabular F value built around an alpha confidence interval [Neter,1974].

4. **t Statistic:** The "t" statistic for a regression coefficient represents the relative assurance that the corresponding independent variable has an effect on the dependent variable. The t statistic is calculated based on the given formula:

$$t = \frac{b_i}{S(b_i)} \qquad (4.5)$$

Where,

t = t statistic for a linear regression coefficient, indicating the significance of the ith independent variable;

b_i= linear coefficient of the ith independent variable; and

$S(b_i)$= Standard deviation of linear regression coefficient.

Variables with a t value less than two are considered insignificant variables [Ott, 2001]. The p value can also be used to determine whether a particular variable plays a significant part in the model. The p values are calculated from the t statistic, with a degree of freedom of one. For a given α (alpha) value, if p is greater than alpha the variable is considered to be insignificant, otherwise, the variable is considered to be significant.

5. **Correlation Coefficient:** The correlation coefficient reflects the magnitude and sign of the correlation between the independent variable and the dependent variable. A positive correlation coefficient predicts an increase in the dependent variable as the independent variable is increased.
6. **Practicality:** From the regression analysis it may be found that some of the models developed may not be practical, they are unexplainable or illogical. Engineering judgment is used to decipher which models are practical and which models are not. Studying the different parameters used in this study, on increasing the percent air voids in the gryratory compacted HMA sample it can be observed that the permeability would increase, similarly if the percent air voids is decreased, the permeability would decrease. Including a higher percent of finer material in the Superpave mixture, would decrease the permeability, since the air void spaces would be filled with fine material and hence be impermeable to water. A higher asphalt film thickness value would also result in a lower permeability. Increasing the compactive effort, that is, by increasing the number of gyrations, the percent air voids and hence the permeability would be expected to decrease.

4.5.4 Model Development

Different selection methods exist to determine which model best explains the data. The backward selection method available in SAS was used in this study to select the optimum model along with the R^2 method [Helwig, 1979]. This model development process starts with a complete model with all independent variables entered and eliminates one variable at a time until a reasonable regression model is found. One of the advantages of using the backward selection model is that it shows the analysts the implications of models with many variables [Neter, 1979]. Regression models were developed separately for mixes with nominal maximum aggregate sizes of 19 mm and 12.5 mm.

A logarithmic transformation was done on the dependent variable, i.e., permeability, to obtain regression models for NAMS 19 mm and 12.5 mm mixes. The general form of the equation is:

$$\text{Log}_{10}(\text{Permeability}) = a + bX_1 + CX_2 + DX_3 + \ldots\ldots \qquad (4.6)$$

Where Log_{10}(Permeability) is the dependent variable and is the measured permeability value expressed as a logarithm to the base 10; X_1, X_2 and X_3 are independent variables; and a, b and c are linear correlation coefficients.

4.5.5 Models Obtained

Table 4.4 shows the results of the regression analysis obtained for the Superpave mixes used in this study. Typical SAS code and outputs are included in the Appendix A.

For SM 12.5 A and B mixes it was found that the percent air voids in the compacted sample, the amount of material passing 600-micron sieve and the film thickness were the significant variables affecting water permeability. For SM 19 A and B mixes, the significant variables were percent air void in the compacted sample, the amount of material passing 600-micron sieve and the number of gyrations required to reach the target air voids. It appears that compaction effort, in addition to end result of it (air voids), is important for large-sized aggregates. This may indicate, as expected, that higher compactive effort for large mixture sizes not only reduces air voids but also decreases permeability.

Table 4.4 Models Derived for Permeability

Variable	Description	Parameter Estimate	R^2 Value
SM 12.5 A and SM 12.5 B Mixes			
Intercept	Vertical Intercept	1.334	0.858
AIR	Percent Air Voids present	0.3753	
PASS30	Percent Passing the #30 (600 Micron) Sieve	-0.0365	
THICK	Film Thickness (microns)	-0.1262	
SM 19 A and SM 19 B Mixes			
Intercept	Vertical Intercept	0.9089	0.6939
AIR	Percent Air Voids present	0.2693	
PASS30	Percent Passing the #30 (600 Micron) Sieve	-0.0298	
GYRATS	Number of Gyrations required to achieve a target Air Void content	-0.0308	

Table 4.4 shows that for both NMAS 19 mm and 12.5 mm mixes, the permeability increases with the increase in percent air voids. This observation, however, is long established. Figures 4.4 and 4.5 show the quantitative relationship between the permeability and the percent air voids. It appears that for the 12.5 mm NMAS mixtures, there is a high increase in permeability at air voids higher than 9.0%. For 19 mm NMAS, that critical air void appears to be about 8%. Thus achieving better compaction is more important for higher nominal maximum size aggregates.

The regression coefficient estimates for the percent material passing 600-micron sieve and film thickness are negative. This indicates that as the amount of material passing 600-micron sieve increases, i.e. when more fine sand is present in the mix, the permeability decreases. This could be due to the fact the interconnected void spaces between the larger aggregate particles are filled up with this finer material, preventing percolation of water through the mix. This was observed for both 12.5 mm NMAS mixes and 19 mm NMAS mixes. Similarly, as the asphalt film thickness increases, it decreases interconnected air void spaces and prevents the flow of water through the interconnected voids. Figures 4.6 and 4.7 show the relationships between permeability, percent material passing 600-micron sieve and the film thickness for the 12.5 mm NMAS mixes. When the percent material passing 600-micron sieve increases from 10% to about 20%, a large decrease in permeability is observed, for the 12.5 mm NMAS mixes. For the 19 mm NMAS mixes, a large decrease in permeability was observed when the amount of material passing the 600-micron sieve increased from 12% to about 18%. Figure 4.6 shows the relationship between the permeability and the percent material passing 600-micron sieve for the 19 mm mixtures. Similar decrease in permeability is observed when the asphalt film thickness increases from eight microns to about twelve microns. It would be expected that as the film thickness increases the permeability would decrease, as the thick asphalt coating around the aggregates would close up the air voids between the aggregates with asphalt and prevent water from flowing thorough these void spaces. However, for the 19 mm NMAS mixtures this was not the case. Figure 4.8 shows the quantitative relationship between the permeability of the mixtures and the film thickness in microns. It can be observed that the permeability increased in spite of the film thickness increasing and this could be due to the large stone sizes. As the NMAS of the aggregate gradation increases, the asphalt film thickness ceases to play a role in closing the interconnected air voids. It is unlikely that all the particles in the asphalt mix would have the same film thickness of asphalt coating and finer particles would have a much thicker coating of asphalt compared to the coarser particles in the mix.

The parameter estimate for the number of gyrations required to reach the target air voids in the regression models for NMAS 19 mm mixtures was also found to be negative. Figure 4.9

illustrates the general relationship between the permeability and number of gyrations required to achieve a target air void content. Sharp decrease in the permeability was observed when the number of gyrations increases from five to about 25. This may indicate that the mixtures that get compacted very fast (although it may satisfy %Gmm at Nini criteria) are more likely to have higher permeability. These mixtures are probably similar to the mixtures with "humped gradations" identified during Superpave research. All the graphs show in Figures 4.2 to 4.8 were plotted with a log scale on the y-axis and a linear trend line was drawn.

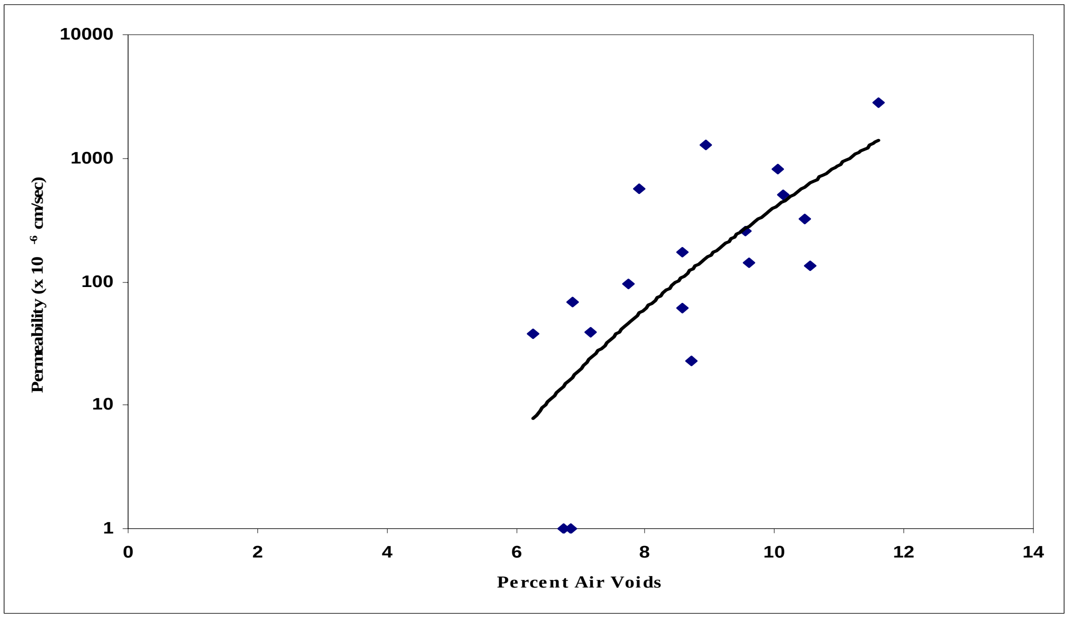


Figure 4.4 Relationship between Permeability and Percent Air Voids for NMAS 19 mm Mixtures

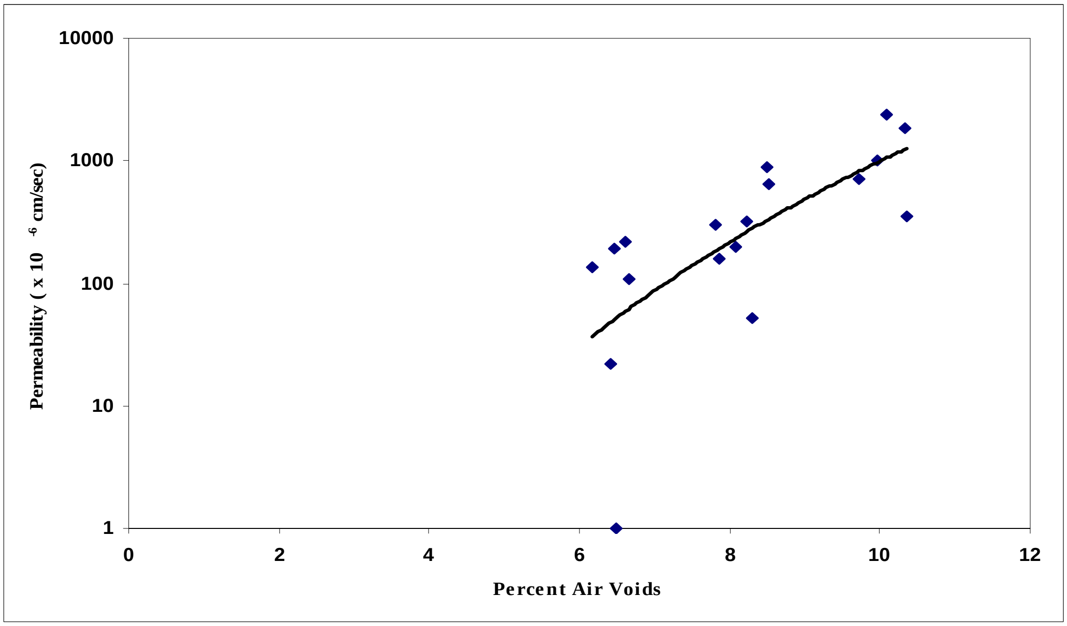


Figure 4.5 Relationship between Permeability and Percent Air Voids for NMAS 12.5 mm Mixtures

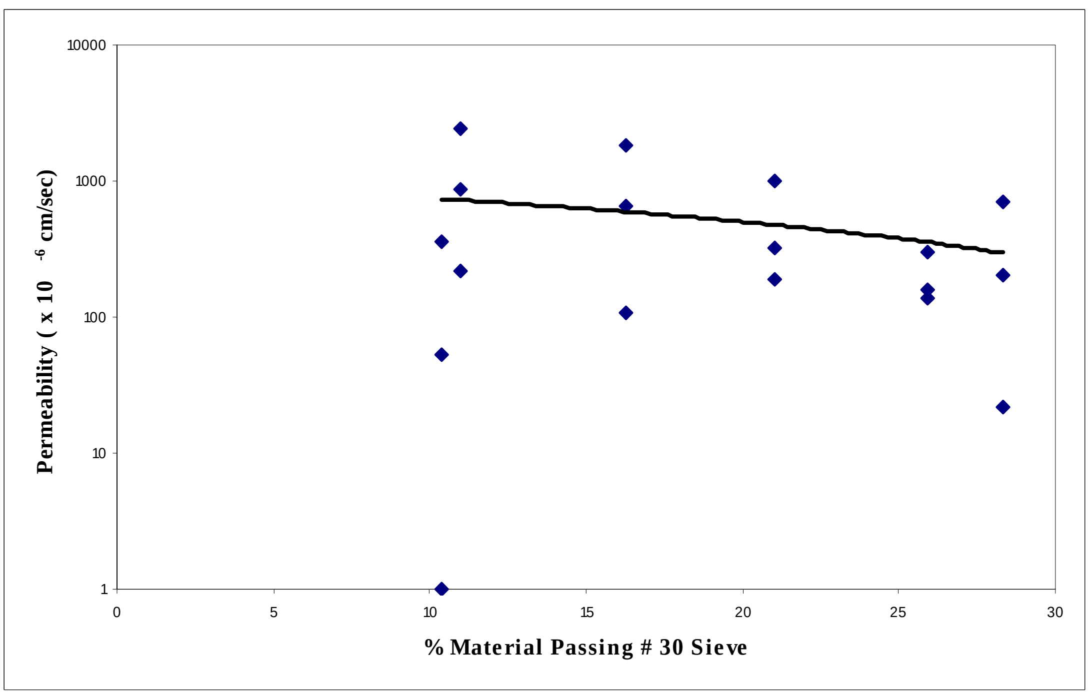


Figure 4.6 Relationship between Permeability and % Passing 600 micron Sieve for NMAS 12.5 mm Mixtures

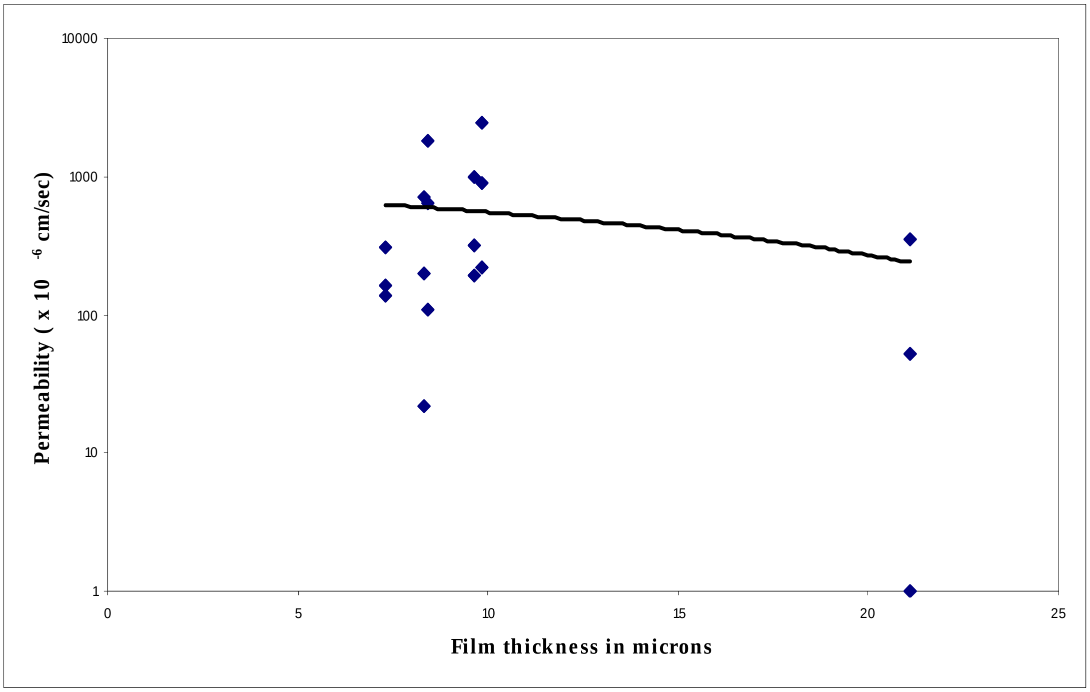


Figure 4.7 Relationship between Permeability and Film thickness for NMAS 12.5 mm Mixtures

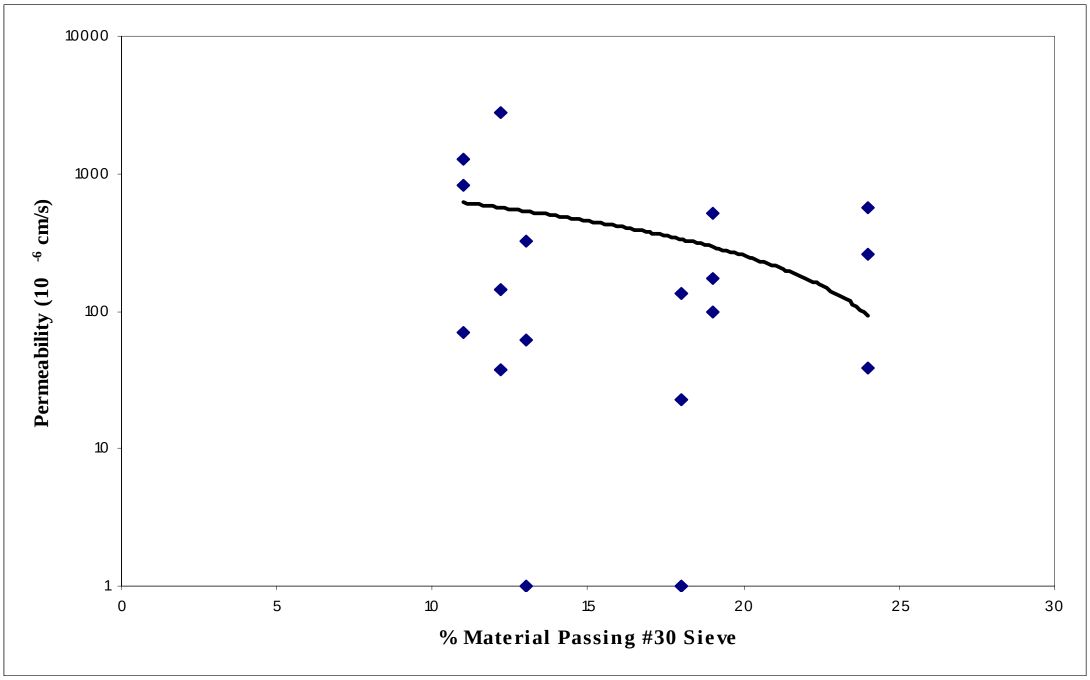


Figure 4.8 Relationship between Permeability and % Passing 600 micron Sieve for NMAS 19 mm Mixtures

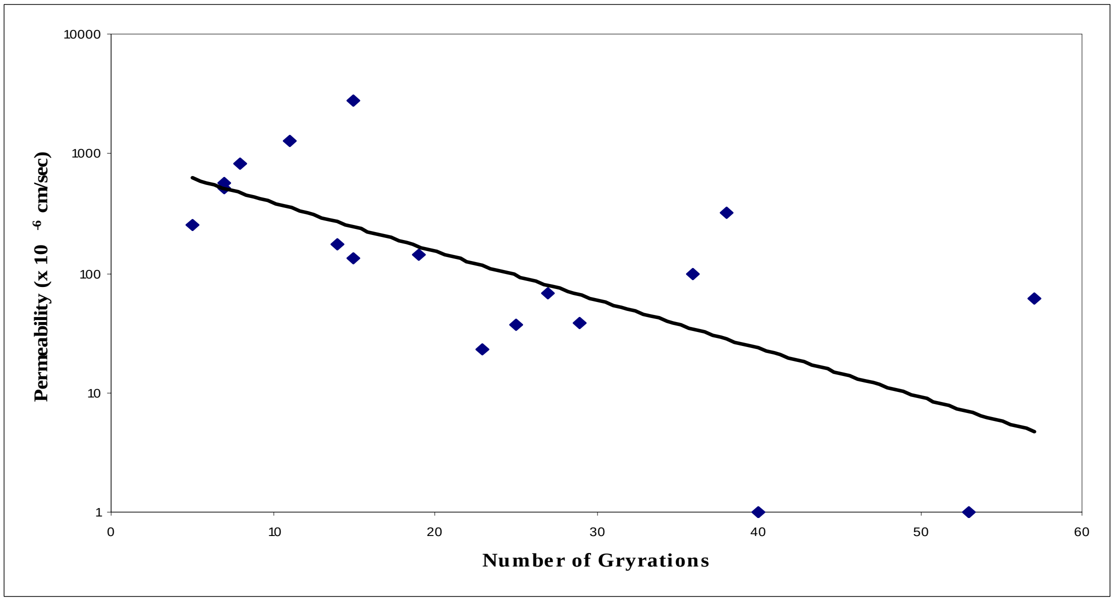


Figure 4.9 Relationship between Permeability and Number of Gyrations for NMAS 19 mm Mixes

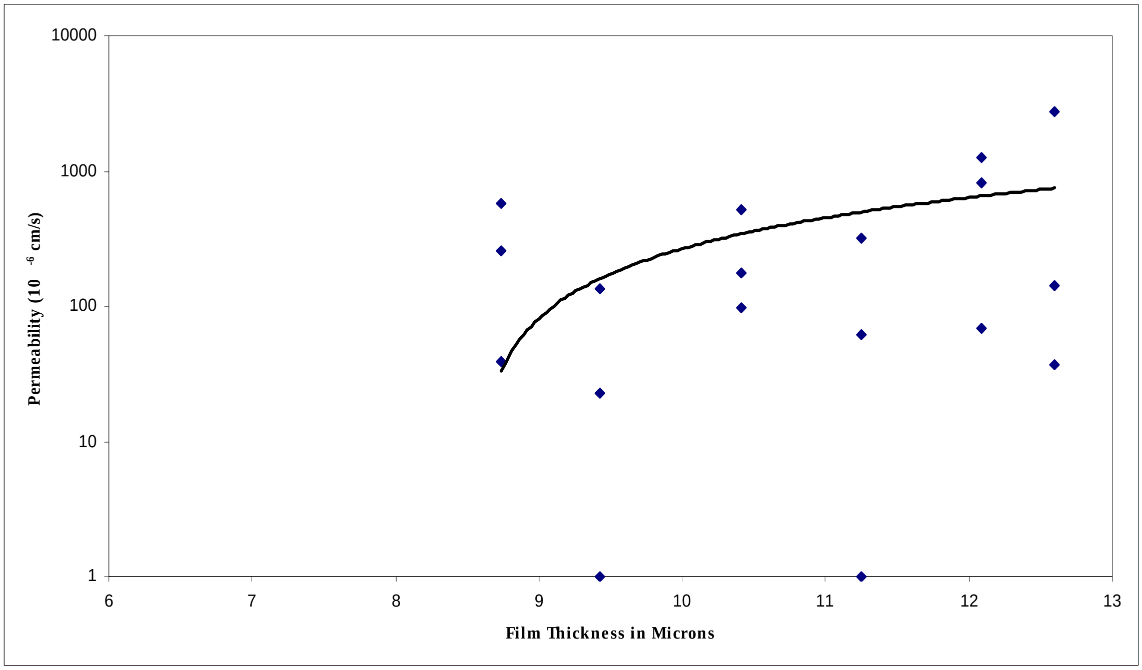


Figure 4.10 Relationship between Permeability and Film Thickness for NMAS 19 mm Mixtures

4.6 Optimum Mixture Water Permeability

A statistical technique, known as Multiple Property Optimization (MPO), was used to find out the optimum water permeability limits for the NMAS 12.5 mm and 19 mm mixtures. Multiple Property Optimization technique discussed here is an unconstrained optimization problem in the mathematical sense of maximization or minimization [MPO, 1991]. The problem involves choosing values in the feasible regions, for different control variables $x_1,\ldots x_n$, which are known as decision variables. This is done so as to maximize or minimize real-valued function *f* of those variables. The structure of the problem may be expressed as:

$$\max_{\{x_1,\ldots,x_n\}} f(x_1,\ldots x_n) \qquad (4.7)$$

The function $f(x_1,\ldots x_n)$ is the total goodness of equation. The variables $x_1,\ldots x_n$ are used to obtain the predictive correlation equations for the permeability. Such variables in the current problem include percent air void, percent material passing a certain sieve size, asphalt film thickness, number of gyrations required to achieve a target air void content, etc.

4.6.1 Information needed for MPO Analysis

A commercial software available from Harold S. Haller & Co. was used in this study. The following information are needed for MPO analysis [MPO, 1991]:

1. **Correlation Equations**: The correlation equations quantify how the independent variables affect the dependent variables. The dependent variables are the variables that are being optimized. In our case, the dependent variable is the permeability. The properties can be adjusted indirectly only by adjusting the independent variables. Table 4.5 shows the correlation equations between different variables that were used in this study.

2. **Goodness Relationships**: A goodness equation is a rating system that describes the various levels for the properties being optimized. In this case, the property is the permeability. The equation describes the target and specification limits for the properties and how much it hurts the goodness for being off target value. A rating system of zero to ten is generally used. Desirable regions between the target and the unacceptable value are scaled accordingly. In this problem, a two-step goodness relationship was used for permeability. Since we would like to have a mix which is impermeable to water, a value of permeability from 0 to 1.0×10^{-6} cm/sec was taken to have a goodness of 10. A permeability value greater than 100.0×10^{-6} cm/sec was taken to have a goodness of zero. Similar results were obtained when the permeability values were chosen from 0 to 1.0×10^{-6} cm/sec. Hence the minimum permeability was chosen to be 1.0×10^{-6} cm/sec, since it is impossible to obtain a Superpave mix which is impermeable to water or having zero permeability. The average permeability obtained during testing by considering the NMAS 19 mm and 12.5 mm mixtures was found to be 467.81×10^{-6} cm/sec. About one-fourth of this value was chosen as the upper limit in the rating system. Hence a permeability value greater than 100.0×10^{-6} cm/sec was taken to have a goodness of zero. Goodness linearly decreases when the permeability increases from 1.0×10^{-6} cm/sec to 100.0×10^{-6} cm/sec. The rating system for permeability is presented in Figure 4.11.

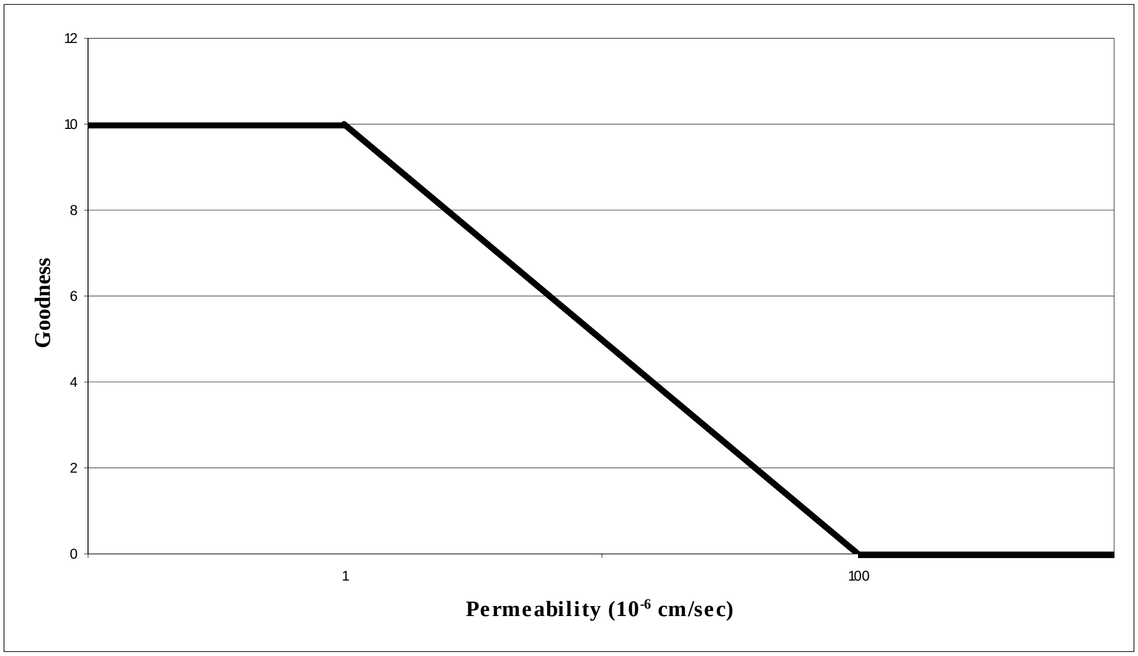


Figure 4.11 Rating System for Permeability

3. **Levels of the Independent Variables**: The maximum and minimum values for each independent variable depend upon the data set used to develop the correlation equations. In addition to the lowest and the highest levels, additional intermediate levels were selected for the MPO analysis.

The total goodness is calculated using the arithmetic sum method as follows:

$$G_{total} = W_1G_1 + W_2G_2 + \ldots\ldots + W_nG_n \quad (4.8)$$

Where, G = goodness,

W = Weighting factor, and

n = number of properties being optimized.

In our problem n is equal to one. The default weighting factor for any goodness equation is one [MPO, 1991]. If all weighting factors are one, then every equation holds equal importance in the goodness calculation. The weighting factor can be changed according to the importance of the evaluated goodness equation compared to others. The calculation steps are as follows:

1. Enter each combination of the process variable levels into the correlation equations and compute the resulting properties;
2. For each resulting property, determine the corresponding goodness; and
3. Calculate the G_{Total} using equation (4.8).

The optimum combination is the combination of the variables that gives the highest G_{Total} [MPO, 1991]. The results of the optimization are shown in Table 4.4 for NAMS 19 mm and 12.5 mm mixes.

4.6.2 Water Permeability Limits

As mentioned earlier, the objective of the MPO analysis was to find a combination of independent variable levels to achieve the maximum total goodness or in other words, to obtain a permeability value as low as possible. Using the regression equations obtained earlier and the KDOT limits for different parameters found to be significant in the regression analysis, acceptable permeability limits for the mixes in this study were determined. The analysis was done separately for NMAS 12.5 and 19 mm mixtures. A value of 4% was used for air voids, and it was assumed that air void above 6 % or below 2 % is unacceptable.

For NMAS 19 mm mixtures, the lowest permeability was found to be 1.003×10^{-6} cm/sec (Table 4.5). The different parameters involved had the following optimum values: air void content, 3.0%; percent material passing 600 micron (No. 30 sieve), 13 %; and the number of gyrations required to reach the target air voids, 43.

For NMAS 12.5 mm mixtures, the lowest value of permeability was 6.5×10^{-6} cm/sec, about 6.5 times higher than the NMAS 19 mm mixtures. The optimum values of different parameters were: air void content, 3.5 %; film thickness, 8 microns; and percent material passing 600 micron (No. 30 sieve), 24 percent.

These results may indicate that in order to achieve lower permeability, higher compactive effort and lower amount of fine sand would be needed for higher NMAS mixtures. However, for lower NMAS mixtures higher fine sand content is desirable.

Table 4.5 MPO Results of 19.0 mm and 12.5 mm Superpave Mixes

19.0 mm Superpave Mixtures					
G_{Total}	Percent Goodness	Permeability (x 10^{-6} cm/sec)	% Air Voids	% Passing the #30 Sieve	Number of Gyrations
10	99.997	1.0030	3.00	13.00	43
10	99.996	1.0040	3.50	17.50	43
10	99.995	1.0050	4.00	22.00	43
10	99.993	1.0070	4.50	12.00	57
10	99.992	1.0080	4.25	18.00	49
10	99.991	1.0090	5.50	21.00	57
10	99.990	1.0100	3.00	16.00	40
10	99.988	1.0120	4.50	14.00	55
10	99.987	1.0130	5.00	18.50	55
10	99.986	1.0140	3.25	12.00	46
10	99.985	1.0150	3.75	16.50	46
10	99.984	1.0160	4.25	21.00	46
10	99.982	1.0180	3.00	19.00	37
10	99.981	1.0190	3.50	23.50	37

12.5 mm Superpave Mixtures					
G_{Total}	Percent Goodness	Permeability (x 10^{-6} cm/sec)	% Air Voids	Film Thickness (microns)	% Passing the #30 Sieve
10	94.947	6.4955	3.5	8.00	24.0
10	94.438	6.9935	3.5	8.00	22.5
10	94.283	7.1495	3.5	7.75	23.5
10	94.200	7.2310	3.5	8.00	22.5
10	94.021	7.4025	3.5	8.00	21.5
10	94.017	7.4110	3.5	8.00	22.0
10	93.916	7.5130	3.5	7.75	23.5
10	93.910	7.5195	3.5	8.00	22.5
10	93.904	7.5210	3.5	7.75	22.5
10	93.829	7.5985	3.5	7.75	21.5
10	93.817	7.6345	3.5	8.00	21.5
10	93.359	8.0635	3.75	8.00	24.0
10	93.166	8.2525	3.75	8.00	23.5
10	92.956	8.4580	3.75	8.00	23.0

4.7 Comparison with KDOT Specifications

From the optimization results, the lowest permeability was found to be 1.003×10^{-6} cm/sec for the NMAS 19 mm mixtures. The optimum values of the different parameters involved were: air void content, 3.0%; percent material passing 600 micron (No. 30 sieve), 13 %; and the number of gyrations required to reach the target air voids, 43. The optimization results for the NMAS 12.5 mm mixtures, gave a lowest value of permeability of 6.5×10^{-6} cm/sec, which was about 6.5 times higher than the NMAS 19 mm mixtures. The optimum values of different parameters were: air void content, 3.5 %; film thickness, 8 microns; and percent material passing 600 micron (No. 30 sieve), 24 percent.

The Kansas Department of Transportation (KDOT) does not have any current specifications for the acceptable permeability limits of Superpave mixtures. All mixtures used in the study were actual mixtures obtained from the contractors and hence they passed the KDOT mix design criteria. The current KDOT criterion for the percent air voids in a compacted mix is 4 ± 2%. In this study the air void contents for optimum permeability obtained for the NMAS 19 mm and 12.5 mm mixes were 3% and 3.5% respectively. These values are within the current KDOT specified limits. KDOT also does not have any current specifications for percent material passing the 600 micron sieve, the film thickness and the number of gyrations required to achieve a certain target air void content. The 600 micron sieve lies in between the 2.36 mm sieve and the 75 micron sieve. Based on the current KDOT gradation criteria, it is observed that the percent material passing the 600 micron sieve for optimum permeability is 24 % for the NMAS 12.5 mm mixtures and 13% for the NMAS 19 mm mixtures, which is in between the KDOT defined criteria for the percent materials passing the 2.36 mm and the 75 micron sieves. This is a good representation of the percent fines present in the mixture. A study done by Kandhal [1996] has shown that the average film thickness in a HMA mixture ranges from 7 to 14 microns. The optimum value of film thickness obtained in this study was 8 microns for the NMAS 12.5 mm mixtures. It can be said that in order to obtain a Superpave mixture with less permeability a film thickness value of 8 microns would be the optimum for mixtures with NMAS 12.5 mm.

CHAPTER 5

FIELD PERMEABILITY

5.1 Introduction

It is a generally accepted that the proper compaction of Superpave mixtures is vital for a stable and durable pavement. Low in-place air voids cause problems such as, rutting and shoving, while high in-place air voids reduce the pavement durability through moisture damage and excessive oxidation of the asphalt binder. For dense graded mixtures, numerous studies have shown that initial in-place air void contents should not be below three percent or above approximately eight percent [Roberts, 1996]. As the in-place air voids increases, the permeability also increases. Past studies have also shown that mixtures with different NMAS have different permeability characteristics [Mallick, 1999]. Coarse-graded Superpave mixtures have a different internal air void structure than the dense-graded mixes used prior to Superpave [Cooley, 2000]. As the NMAS increases, the size of individual air voids also increases. This increase in air void size leads to an increased potential for interconnected air voids. These interconnected air voids cause permeability in the Superpave pavements. Interconnected voids are the paths through which water can flow and hence mixtures with higher NMAS would be expected to be more permeable at a given air void content compared to the mixtures with a lower NMAS. The gradation characteristics of the aggregate structure also affect the permeability of the Superpave mixtures. Gradations that pass below the maximum density line or the “B mixtures” would be expected to be more permeable than mixtures, which pass over the maximum density line, “A mixes” [Choubane, 1998]. The lift thickness of the Superpave mixture layer is another factor that affects the permeability [Mallick, 1999]. In normally constructed asphalt pavements all void spaces are not necessarily interconnected. Voids that are not interconnected do not allow water to flow thorough them. As the thickness of the pavement increases, the chance for voids being interconnected with a sufficient length to allow water to flow decreases. Because of this thinner pavements may have more potential for permeability.

During the mix design process it is not possible to know the actual permeability of a Superpave mix in the field without actually placing, compacting and measuring the field permeability value. In this part of the study, in-place permeability testing was conducted on different Superpave pavements in Kansas to study the correlation between the laboratory and the field permeability values, so that the field permeability values could be predicted during the mixture design process and hence adjustments made to the mix design depending upon the degree of permeability desired. Correlations between the field permeability and the lab permeability and percent air voids were also investigated.

5.2 Study Approach

This study was conducted in two parts: field-testing of Superpave mixtures and laboratory testing of gyratory compacted mixes obtained from the field. Figure 5.1 shows the outline of this study. Six pavements with a NMAS value of 19 mm and three projects with a NMAS of 12.5 mm were chosen. Table 5.1 summarizes the characteristics of the Superpave mixes obtained from different Superpave projects. For each test project, in-place field permeability tests were conducted at three different locations on newly compacted HMA, before opening to traffic. A commercially available field permeability-measuring device, available from Gilson and Company, Inc., was used in this study. Tests were conducted at three locations on the pavement, spaced at about 300 mm apart, and at each test location three permeability tests were conducted. All permeability measurements were made at a distance of about 600 mm from the pavement edge. The permeability device uses a rubber sealant to help seal the permeameter to the pavement surface. After the first test at a given test location, the device was lifted off the pavement and re-sealed immediately to conduct the second replicate test. Each replicate test was conducted at a spacing of approximately 250 mm.

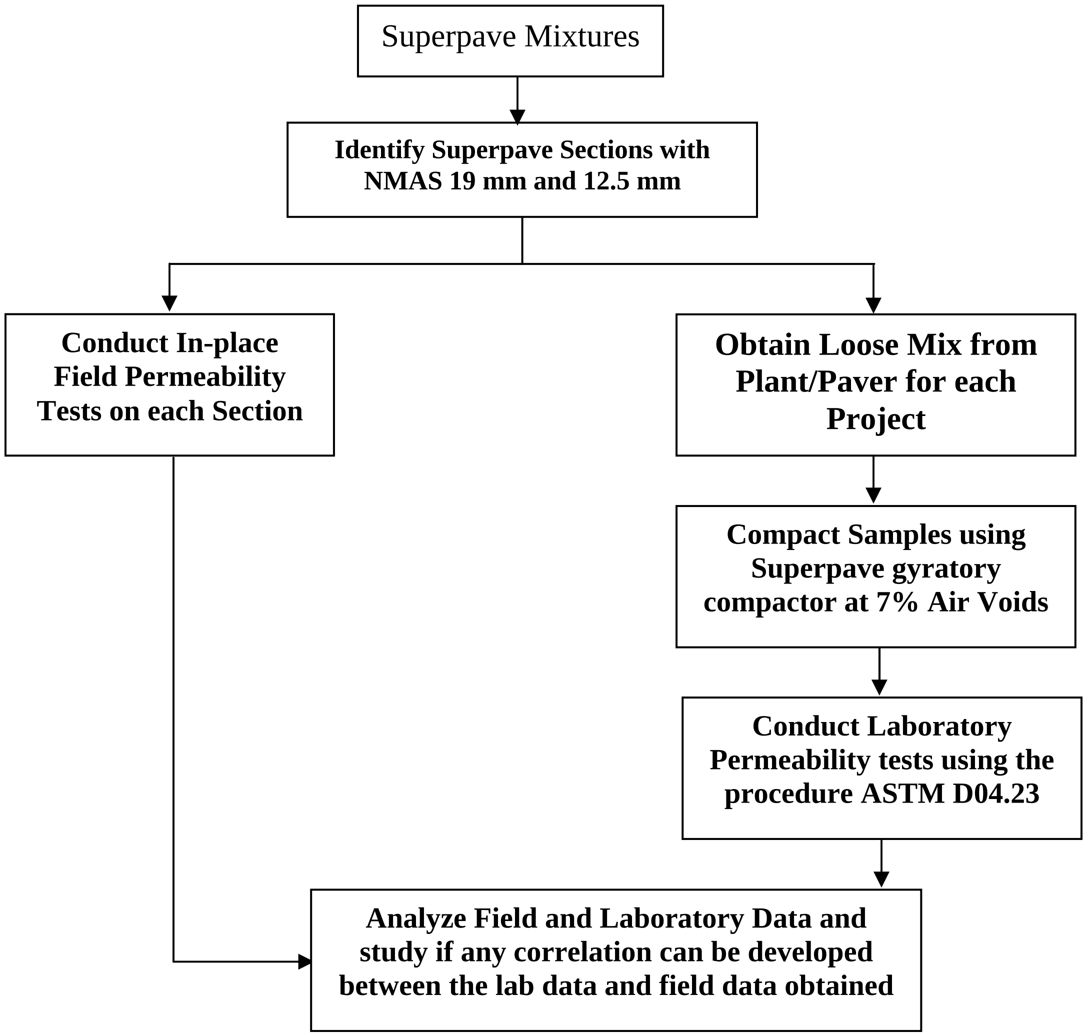


Figure 5.1 Field Permeability Test Plan

Table 5.1 Properties of Superpave Mixes used in Field Study

Mixture/Aggregate Blend Property	Description	Design ESALs (millions)	N_{design}	PG Binder Grade	Asphalt Content (%)	Air Voids (%) at Ndes	VMA (%)	VFA (%)	Dust Binder Ratio	%Gmm at Nini	%Gmm at Nmax
SM 19A(I)	Ritchie K-77	1.4	75	PG 58-28	5.2	4.26	13.96	69.56	1.2	89.1	96.6
SR 19A(II)	Venture I-70 Russell Shld	15	100	PG 64-23	3.6	4.7	14.79	68.21	0.73	90.66	96.08
SR 19A(III)	Venture I-70 Russell ML 1st Lift	15	100	PG 58-28	3.8	4.37	14.81	70.48	1.05	89.66	97.24
SR 19A(IV)	Venture I-70 Russell ML 2nd Lift	15	100	PG 70-28	4.2	4.15	14.12	70.53	0.96	89.71	97.26
SR 19A(V)	Venture I-70 Ellis ML 1st Lift	17.2	100	PG 64-22	3.9	4.4	14.68	76.43	1.2	90.65	97.55
SR 19A(VI)	Venture I-70 Ellis ML 2nd Lift	17.2	100	PG 70-28	4.3	3.6	13.5	73.36	1.4	88.8	97.6
SM 12.5A(I)	Hamm US-73	1.2	75	PG 64-22	6.5	4.74	14.58	67.49	1.29	87.5	96.4
SM 12.5A(II)	Ritchie K-4	0.7	75	PG 64-22	5.4	3.32	13.89	75.15	1.1	89.7	97.7
SM 12.5A(III)	Heckert US-56	1.2	75	PG 64-22	5.25	4.14	13.02	70.78	0.73	88.9	96.7

Field permeability testing was done in a direction longitudinal to the pavement, since the pavement density tends to be more uniform longitudinally than transversely. Also, plant produced mix was sampled from the paver for each project, to carry out permeability testing on laboratory compacted Superpave specimens. Figures 5.2 and 5.3 show the aggregate gradation charts for the 19 mm and 12.5 mm Superpave mixes used.

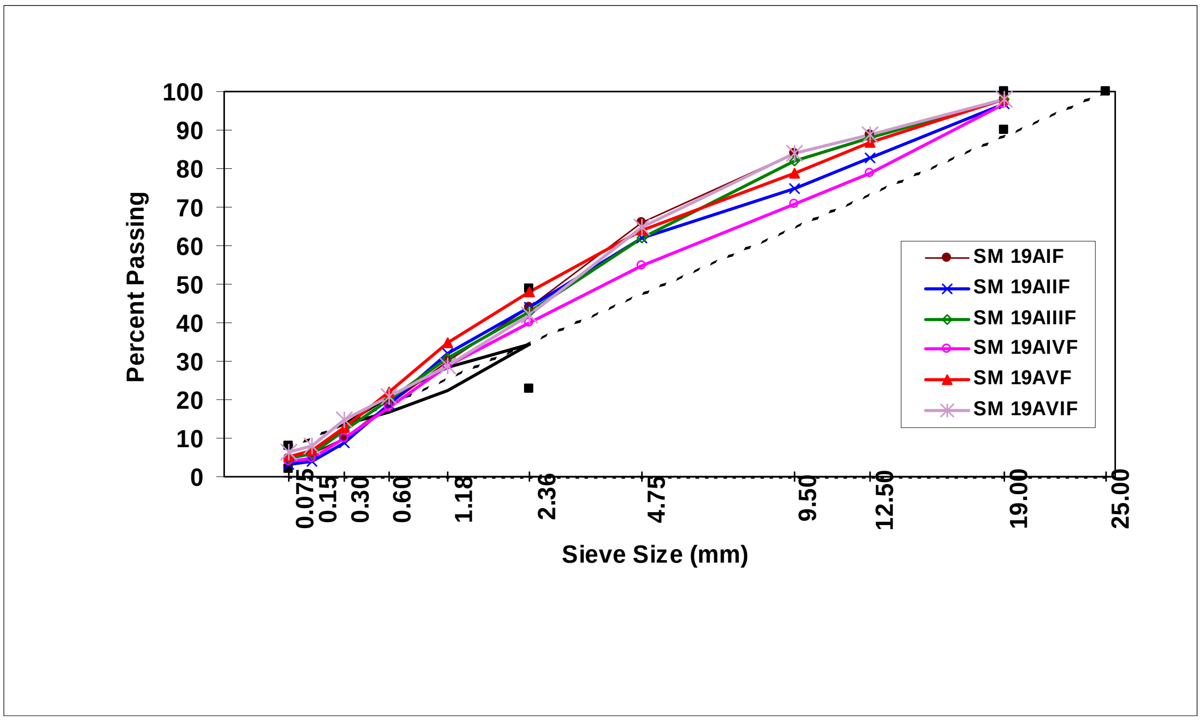


Figure 5.2 Aggregate Gradation Chart (SM 19 mm Mixes)

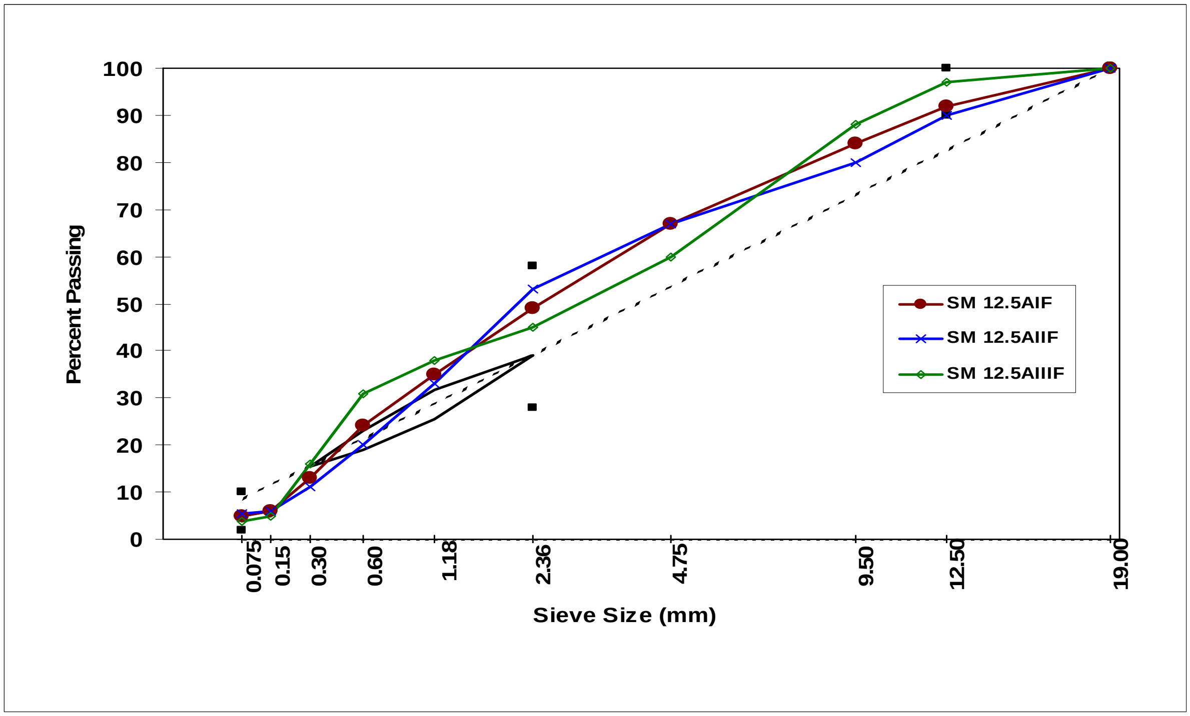


Figure 5.3 Aggregate Gradation Chart (SM 12.5 mm Mixes)

5.3 Field Permeability Testing Device

The field permeability-testing device used in this study is based upon the work by the National Center for Asphalt Technology (NCAT) [Cooley, 2000]. The NCAT permeameter is manufacture commercially by Gilson Company, Inc. A schematic of the permeameter used is shown in Figure 5.4. It is based on the falling head principle of permeability. The coefficient of permeability using this device is calculated as follows:

$$K = \frac{al}{At}\ln\left(\frac{h_1}{h_2}\right) \qquad (5.1)$$

Where,

K = coefficient of permeability in cm/sec;
a = inside cross sectional area of inlet standpipe no.1,2,3 or 4 in cm^2;
l = thickness of the HMA sample, cm (thickness of the pavement);

A = cross-sectional area of the permeameter through which water can penetrate pavement, cm^2;
t = time taken for water to flow from h_1 to h_2, seconds;
h_1 = initial head of water, cm; and
h_2 = final head of water, cm.

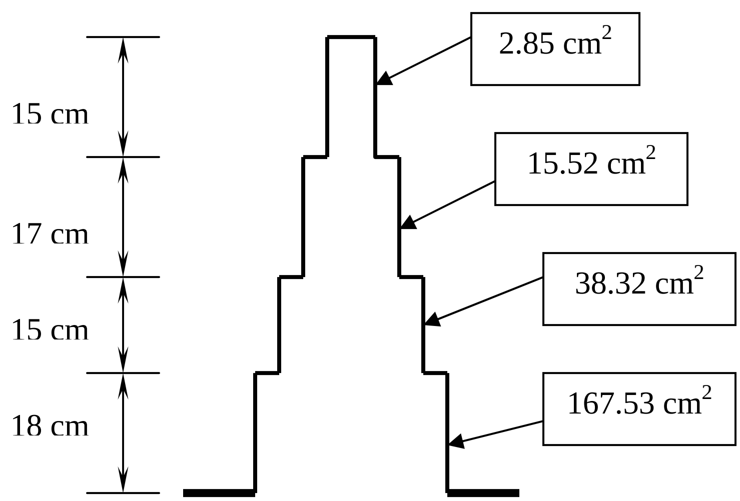


Figure 5.4 Schematic of the Field Permeameter

The device consists of four conjoined segments or "tiers" of clear acrylic plastic. Each of the segments is of variable cross sectional area so that testing of pavements with a wide range of permeability, and hence different rates of flow can be done. The area where the permeability test is conducted is cleaned thoroughly to remove the surface dust and then the permeameter is fitted with a rubber gasket sealant, which ensures a watertight seal between the base of the permeameter and the pavement surface. By stepping gently on the base of the permeameter the sealant can be forced into the surface of the pavement providing a watertight seal. About 2500 grams weight is added to each corner of the permeameter to compensate for the head of pressure exerted by the water column. Without this counter weight, the water pressure can break the seal between the permeameter and the surface of the pavement. After the sealing is ensured, the filling tube assembly is inserted all the way to the bottom of the permeameter. The permeameter is then filled with water at a steady rate, and as the water level nears the top, filling is continued as the tube is withdrawn. Careful filling of the permeameter will ensure minimum entrapped air in the water column. The permeameter vessel can hold about 3.8 liters of water. After the permeameter is filled to the top with water, the rate at which the water flows into the asphalt pavement is monitored and a tier is selected for monitoring the permeability which is neither too slow nor too fast, for accurate observation. The start time and the initial head of water are noted. Because of the large diameter of the second tier, the flow of water would be slow enough for efficient recording of data for most Superpave pavements tested in this study. The time taken for the water level to fall by 100 mm was noted. Figure 5.5 shows the permeameter used for field testing.

Figure 5.5 Field Permeability Testing Device

5.4 Potential Problems in Measuring In-Place Permeability

A majority of the recent research work in permeability testing has been conducted on the core specimens that have been cut from the pavements or on the specimens compacted in the Superpave gyratory compactor. This is important since Darcy's law is applicable for one-dimensional flow as would be encountered in a laboratory test. Measuring the in-situ permeability of an in-place pavement is theoretically more difficult, because water can flow in two dimensions. Other potential problems include degree of saturation, boundary conditions of the flow and the type of flow. When using Darcy's law to compute permeability, one of the assumptions made is that the medium being tested is saturated. As the degree of saturation decreases so does the measured permeability. Unlike laboratory testing, the degree of saturation cannot be accurately identified in the field. When performing laboratory permeability tests, the dimensions of the sample on which testing is done is always known. In the absence of a core, the sample thickness has to be estimated for field tests or the thickness of the pavement from construction data may be used in the analysis. Sample thickness in this case, defines the length of the flow paths. The flow of water in the laboratory permeability test is one dimensional. Another potential boundary condition problem is the flow of water across (through) pavement layers. Without some type of destructive test, such as coring, there is no way of knowing whether water flows across the layers [Cooley, 2000]. Darcy's law was based on testing clean sands and the flow of water through the sands was determined to be laminar. Within a pavement section, it cannot be determined whether the flow of water is laminar or turbulent. Since we cannot use Darcy's law to calculate permeability if the flow is turbulent, hence the flow of water through the pavement *is assumed to be laminar*. While conducting the field permeability tests, it was observed that the first drop in water level usually took lesser time compared to the second or consecutive tests. One possible explanation is that during the first test, the water fills up the voids, including some that were not interconnected, and during the second and third tests, the water cannot go through these non interconnected voids, and only flows through the interconnected voids [Mallick, 2001].

5.5 Results and Discussion

Permeability testing was done in the field for the Superpave mixes that had a NMAS of 19 mm and 12.5 mm as shown in Table 5.1. From the aggregate gradations, shown in Figures 5.2 and 5.3 it can be seen that all gradations pass through the restricted zone. The Kansas Department of Transportation has recently discontinued using restricted zone in Superpave mixture gradation. Table 5.2 summarizes the field and laboratory permeability values.

All laboratory test samples were compacted to a target air void content of 7%. Table 5.2 shows the actual percent air voids in the samples. Laboratory testing was done on the samples as described in Chapter 3. Figure 5.6 is a comparison of the Field permeability values and the laboratory permeability values. From this figure it can be observed that there is a large variation in the permeability values measure in the field and in the laboratory. Figure 5.7 shows the relationship between the laboratory permeability values and the percent air voids for different mixtures tested in this study. Due to absence of coring facilities at all these projects, it was not possible to take in-situ cores of the compacted pavement to measure the actual percent air voids in the field. These results show that there was a significant difference in the values obtained in the field and laboratory permeability values; the field permeability values are always much higher than the laboratory permeability values. This higher permeability for the field data can be explained in terms of flow of water in the field. The flow is not confined to one-dimensional flow. Water entering the pavement can flow in any direction (vertical and/or horizontal). Therefore, it would be expected that the field permeability data should be higher than the laboratory permeability values since both the values were calculated based on the falling head permeability test principles. The difference in the values of permeability obtained would be most likely dependant upon the NMAS, aggregate gradation and the interconnectivity of air voids.

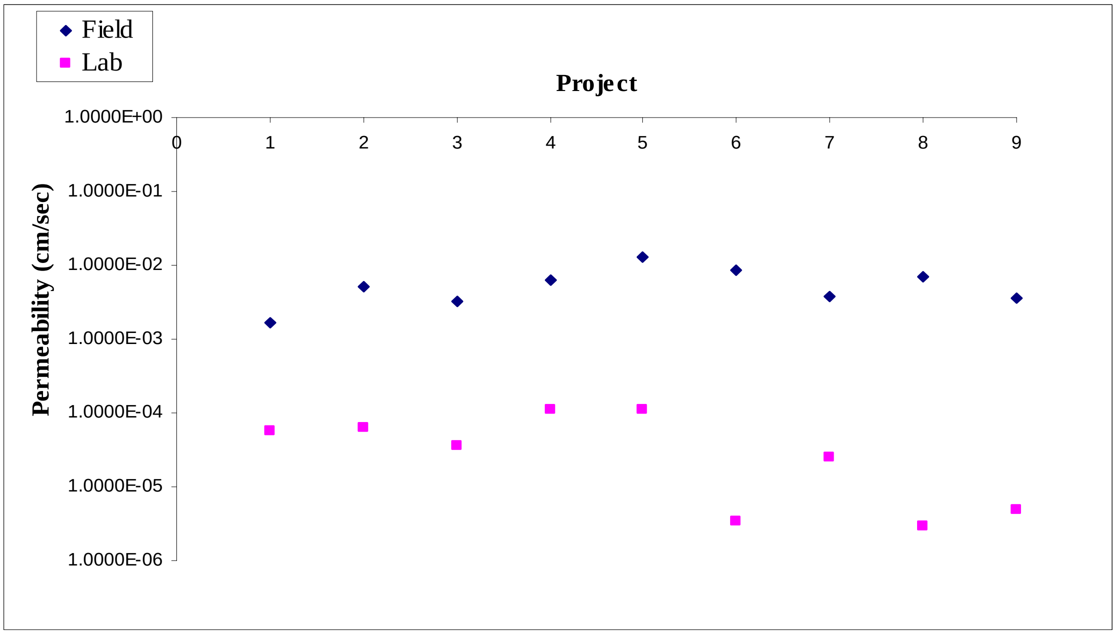


Figure 5.6 Comparison of Field Permeability and Laboratory Permeability Values for the Different Projects.

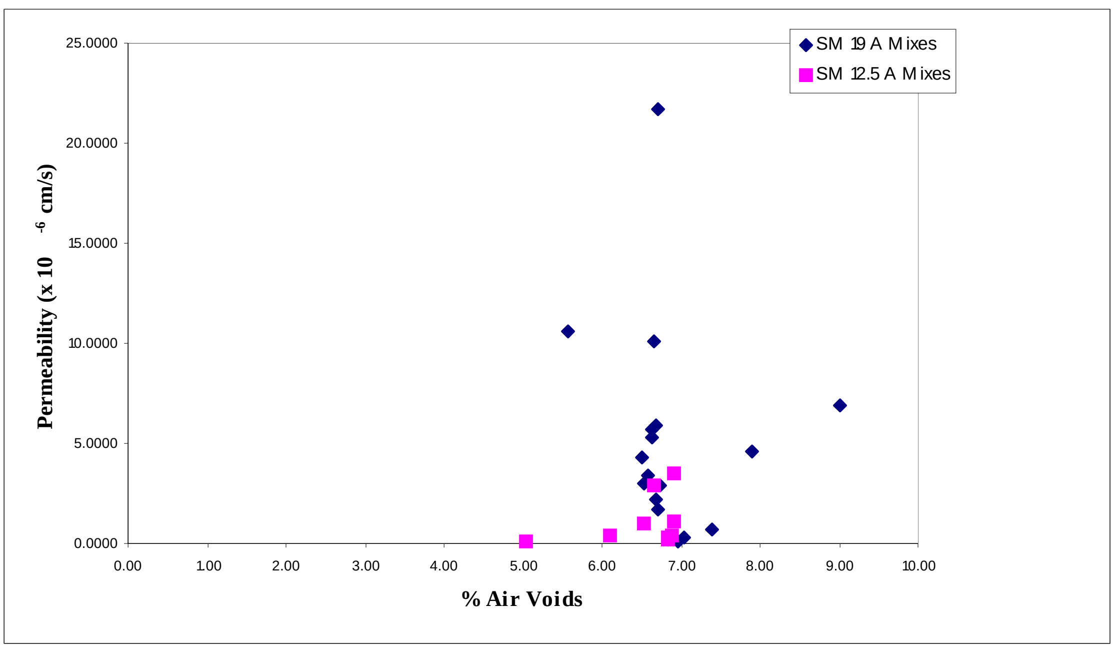


Figure 5.7 Relationship between Permeability and % Air voids for the NMAS 19 mm and 12.5 mm Mixes in this Study

Table 5.2 Summary of Field and Laboratory Permeability Results

Field Permeability Results					Lab Permeability Results			
	Project	**Location**	**Field Permeability (cm/sec)**	**Average Permeabilty (cm/sec)**	**Sample**	**% Air Voids**	**Lab Permeability (cm/sec)**	**Average Permeabilty (cm/sec)**
1	Venture Russel County 1st Lift	1	1.8612E-03		1	9.02	6.8814E-05	
	SR 19A	2	1.2279E-03	1.6454E-03	2	7.90	4.6064E-05	5.7950E-05
		3	1.8471E-03		3	6.68	5.8970E-05	
2	Venture Russel County 2nd Lift	1	3.6090E-03		1	6.74	2.8985E-05	
	SR 19A	2	5.5294E-03	5.0507E-03	2	6.63	5.6808E-05	6.24124E-05
		3	6.0136E-03		3	6.65	1.0144E-04	
3	Venture Russel County Shoulder	1	3.4803E-03		1	6.51	4.2530E-05	
	SR 19A	2	3.0712E-03	3.1900E-03	2	6.54	3.0287E-05	3.55285E-05
		3	3.0185E-03		3	6.59	3.3768E-05	
4	Venture Ellis County 1st Lift	1	5.8534E-03		1	7.04	2.5835E-06	
	SR 19A	2	7.5438E-03	6.1797E-03	2	6.70	2.1712E-04	1.0846E-04
		3	5.1419E-03		3	5.56	1.0568E-04	
5	Venture Ellis County 2nd Lift	1	1.4095E-02		1	6.64	5.2743E-05	
	SR 19A	2	1.3210E-02	1.3008E-02	2	6.70	1.7263E-05	1.0846E-04
		3	1.1719E-02		3	6.68	2.2201E-05	
6	KDOT Marion County	1	8.6681E-03		1	6.97	6.8525E-07	
	SM 19A	2	8.6832E-03	8.3794E-03	2	6.85	2.1315E-06	3.37308E-06
		3	7.7867E-03		3	7.40	7.3025E-06	
7	Ritchie Paving K-4	1	3.4972E-03		1	6.91	1.06047E-05	
	SM 12.5 A	2	3.7896E-03	3.7523E-03	2	6.91	3.53998E-05	2.5104E-05
		3	3.9701E-03		3	6.66	2.93075E-05	
8	Hamm Const. US 73	1	5.1538E-03		1	6.88	3.64946E-06	
	SM 12.5 A	2	7.4717E-03	7.0719E-03	2	6.83	3.06385E-06	2.98136E-06
		3	8.5902E-03		3	6.83	2.23077E-06	
9	Heckert US 56	1	3.8079E-03		1	6.54	1.04708E-05	
	SM 12.5 A	2	3.6109E-03	3.6451E-03	2	5.03	5.26915E-07	5.00212E-06
		3	3.5164E-03		3	6.1	4.00864E-06	

Past experience has shown that a large amount of flow took place in the coarser Superpave mixes with thick lifts in the horizontal direction, whereas finer mixes with thinner lifts tend to have more of a vertical flow [Mallick, 2001]. On most field tests in this study, water was observed to come up through the mat a few centimeters away from the base of the permeameter. This could be due to the horizontal flow of water in the underlying layers of the pavement. In this study, correlation equations between the field permeability, the laboratory permeability values and the percentage of air voids in the laboratory compacted specimens were tried. No meaningful correlation among these three parameters was obtained as shown in Figures 5.6 and 5.7.

5.6 Mat Tearing and Breakdown Rolling

As described in Chapter 2, the weight of the breakdown roller is one of the factors that influences the permeability of HMA pavement during construction. Mat tearing was observed on one of the projects in the field study. The mixture was a SR-19A mix on Interstate 70 in Ellis county, Kansas. The first lift of the pavement exhibited mat tearing at several locations as shown in Figure 5.8. The permeability values at these locations were also very high. The average field permeability value on the sections which did not show mat tearing was found to be 6.1797×10^{-3} cm/sec. The field permeability values on the torn mat was found to be 25.3570×10^{-3} cm/sec which was nearly four times the average field permeability value for the locations with no mat tearing. The thin cracks appeared on the surface of the HMA pavement, due to the heavy roller compacting a thin lift of HMA. In the Ellis County project the thickness of the mat was 65mm (2.5 inches).

Figure 5.8 Mat Tearing/Segregation Observed on I-70, Ellis County Project

5.7 Limitations of the Field Permeability Test

The foremost limitation of the field permeability test is that the conditions, assumed to be valid in order to calculate the coefficient of permeability from Darcy's law, are not valid. Darcy's law is valid for one-dimensional flow, whereas the flow of water through a pavement is both horizontal and in the vertical directions [Mallick, 2001]. Hence, it is difficult to compare or correlate the permeability values obtained in the laboratory and in the field testing. The laboratory test method has flow conditions that are more similar to those assumed for application of Darcy's law and hence in calculating the coefficient of permeability.

The in-place permeability test is best suited for comparative evaluation of permeability of different mixes, and the same mixtures with different properties, such as, in-place density. Since the limitations of the in-place permeability test will be present in each and every test, as long as similar equipment is used and the test procedure is consistent, the results can be used for comparative evaluation purposes only.

CHAPTER 6

CONCULSIONS AND RECOMMENDATIONS

6.1 Conclusions

This study was conducted to identify different factors that influence the permeability of Superpave mixtures in Kansas. Twelve different Superpave mixtures were obtained from different paving projects, and laboratory permeability tests were conducted on the samples compacted using a Superpave gyratory compactor. Multiple regression analysis was used to identify different mixture parameters that influence permeability. Multiple Property Optimization was used to find the optimum permeability limits for the different mixtures used in this study. A field permeability study on a number of Superpave mixes was conducted in an attempt to correlate permeability values measured in the field, laboratory and the percent air voids of samples compacted in the laboratory.

Based on this study, the following conclusions can be made:

1. In general, for any given nominal maximum size Superpave mixture, the fine- graded mix was found to be less permeable than the coarse-graded mix.
2. Percent material passing 600-micron sieve, asphalt film thickness and air voids play significant roles in determining permeability of 12.5 mm nominal maximum aggregate size Superpave mixtures. The permeability of such mixes can be decreased by increasing percent material passing 600-micron sieve and asphalt film thickness. Of course, lesser air voids would also produce a less permeable mix.
3. For 19 mm nominal maximum size Superpave mixtures, decreasing air voids in the mixture decreases water permeability. Also, permeability decreases when more compactive effort, in terms of gyrations in the

Superpave Gyratory compactor, is applied. Permeability of the 19 mm mixes also decreases as the percentage of material passing the 600-micron sieve increases.

4. Mixes that are less permeable, irrespective of gradation (fine or coarse) in the ranges tested, performed very well under the Hamburg wheel rut tester indicating that less permeable mixes are also less susceptible to stripping and rutting.
5. The optimum permeability for the Superpave 19 mm nominal maximum size mix under study was 1.003×10^{-6} cm/sec and for the Superpave 12.5 mm mix was 6.5×10^{-6} cm/sec.
6. There is a large increase in permeability at air voids greater than 9% for mixes with a NMAS of 12.5 mm, whereas for mixes with a NMAS of 19 mm, the critical air voids appears to be about 8%.
7. A large decrease in the permeability values is observed when the percent of material passing the 600-micron sieve is in between 10% and 20% for NMAS 12.5 mm mixtures and in between 12% to 18% for the NMAS 19 mm mixtures.
8. There is a significant difference between the permeability values obtained from the laboratory and field testing. The field permeability values are much higher than the laboratory permeability values.

6.2 Recommendations

Based on the study, the following recommendations can be made:

1. Field permeability testing should be done on a regular basis as quality control and quality assurance procedures on different asphalt pavements, to ensure that the pavements have less permeability values or are within the acceptable permeability limits.
2. Superpave mixes should be designed with a gradation on the finer side of the maximum gradation line. Higher amounts of fines in the mix decreases

the amount of inter connected void spaces and hence decreases the permeability of the mix.

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APPENDIX A

(Typical SAS Input and Output File)

INPUT FILE

```
options ls=80 ps=60 nodate nonumber;
dm 'output;clear;log;clear';
data permall;
infile "permlogd.dat";
input mix$ origin$ air mair nom logperm perm pass4 pass30 pass200 pbe pba
thick gyrats;
run;
proc sort data=permall;
by nom mair pbe;
run;
proc print data=permall;
run;
proc reg data=permall;
model logperm = mair pass4 pass30 pass200 thick gyrats/selection =
rsquare p clm cli vif
corrb;
by nom;
run;

proc sort data=permall;
by nom;
proc reg data=permall;
model logperm = mair pass4 pass30 pass200 pba thick gyrats/ selection=
backward;
run;


proc sort data=permall;
by nom mair;
run;
proc reg data=permall;
model logperm = mair pass30 thick / p clm cli vif corrb;
by nom;

proc sort data=permall;
by nom mair;
run;
proc reg data=permall;
model logperm = mair pass30 gyrats / p clm cli vif corrb;
by nom;

proc corr;
var air mair nom logperm perm pass4 pass30 pass200 pbe pba
thick gyrats;
run
```

OUTPUT FILE

The SAS System

Obs	mix	origin	air	mair	nom	logperm	perm	pass4
1	SM12A	Vent140	7	6.18	12	2.1335	135.99	76.2
2	SM12A	Shil4	7	6.41	12	1.3411	21.93	71.6
3	SM12A	Henn56	7	6.46	12	2.2802	190.63	67.0
4	SM12B	KAPA2425	7	6.50	12	0.0000	1.00	57.8
5	SM12B	Shears50	7	6.61	12	2.3366	217.06	51.0
6	SM12B	Shil54	7	6.65	12	2.0373	108.97	55.9
7	SM12A	Vent140	9	7.81	12	2.4825	303.77	76.2
8	SM12A	Vent140	11	7.86	12	2.2038	159.86	76.2
9	SM12A	Shil4	9	8.07	12	2.3003	199.67	71.6
10	SM12A	Henn56	9	8.22	12	2.5050	319.90	67.0
11	SM12B	KAPA2425	9	8.30	12	1.7196	52.44	57.8
12	SM12B	Shears50	9	8.49	12	2.9455	882.11	51.0
13	SM12B	Shil54	9	8.52	12	2.8139	651.52	55.9
14	SM12A	Shil4	11	9.73	12	2.8522	711.59	71.6
15	SM12A	Henn56	11	9.96	12	3.0004	1000.86	67.0
16	SM12B	Shears50	11	10.09	12	3.3820	2410.16	51.0
17	SM12B	Shil54	11	10.33	12	3.2597	1818.57	55.9
18	SM12B	KAPA2425	11	10.35	12	2.5514	355.95	57.8
19	SM19B	Shil9C	7	6.27	19	1.5727	37.39	37.7
20	SM19A	KDOT	7	6.74	19	0.0000	1.00	56.0
21	SM19B	Shil6C	7	6.86	19	0.0000	1.00	38.1
22	SM19B	Ritch42	7	6.87	19	1.8383	68.92	57.0
23	SM19A	VentB1A	7	7.16	19	1.5883	38.75	69.0
24	SM19A	Vent14	7	7.74	19	1.9868	97.00	59.0
25	SM19A	VentB1A	9	7.91	19	2.7553	569.29	69.0
26	SM19B	Shil6C	9	8.56	19	1.7862	61.12	38.1

Obs	pass30	pass200	pbe	pba	thick	gyrats
1	25.9	5.4	4.3661	0.5614	7.3260	8
2	28.3	5.2	5.1276	1.3594	8.3250	17
3	21.0	6.0	5.4717	0.7763	9.6562	19
4	10.4	0.5	5.7397	0.5442	21.0834	19
5	11.0	4.2	5.1276	1.0901	9.8415	27
6	16.3	4.3	3.8123	1.2502	8.4365	35
7	25.9	5.4	4.3661	0.5614	7.3260	5
8	25.9	5.4	4.3661	0.5614	7.3260	2
9	28.3	5.2	5.1276	1.3594	8.3250	9
10	21.0	6.0	5.4717	0.7763	9.6562	11
11	10.4	0.5	5.7397	0.5442	21.0834	16
12	11.0	4.2	5.1276	1.0901	9.8415	18
13	16.3	4.3	3.8123	1.2502	8.4365	23
14	28.3	5.2	5.1276	1.3594	8.3250	6
15	21.0	6.0	5.4717	0.7763	9.6562	8
16	11.0	4.2	5.1276	1.0901	9.8415	11
17	16.3	4.3	3.8123	1.2502	8.4365	13
18	10.4	0.5	5.7397	0.5442	21.0834	13
19	12.2	3.2	4.2538	0.8917	12.6002	25
20	18.0	3.9	4.2767	2.1049	9.4253	40
21	13.0	3.4	4.2531	0.7862	11.2539	53
22	11.0	4.0	4.6623	0.8864	12.0927	27
23	24.0	3.9	4.4850	0.4998	8.7373	29
24	19.0	2.6	4.6721	0.8760	10.4132	36
25	24.0	3.9	4.4850	0.4998	8.7373	7
26	13.0	3.4	4.2531	0.7862	11.2539	57

The SAS System

------------------------------------ nom=12 ------------------------------------

The REG Procedure
Model: MODEL1
Dependent Variable: logperm

R-Square Selection Method

Number in Model	R-Square	Variables in Model
1	0.4703	mair
1	0.2399	thick
1	0.1977	pass200
1	0.0601	gyrats
1	0.0199	pass4
1	0.0009	pass30
2	0.7949	mair thick
2	0.7512	mair pass200
2	0.4983	mair pass30
2	0.4714	mair gyrats
2	0.4709	mair pass4
2	0.3853	pass4 pass200
2	0.3765	pass30 thick
2	0.3666	pass4 thick
2	0.3658	pass30 pass200
2	0.2699	thick gyrats
2	0.2409	pass200 thick
2	0.2242	pass4 gyrats
2	0.2151	pass200 gyrats
2	0.1754	pass4 pass30
2	0.0739	pass30 gyrats
3	0.8580	mair pass30 thick
3	0.8382	mair pass30 pass200
3	0.8350	mair pass4 thick
3	0.8312	mair pass4 pass200
3	0.8206	mair thick gyrats
3	0.8046	mair pass200 gyrats
3	0.7949	mair pass200 thick
3	0.6638	pass4 thick gyrats
3	0.6636	pass4 pass200 gyrats
3	0.5960	mair pass4 pass30
3	0.5936	pass30 thick gyrats
3	0.5383	mair pass30 gyrats
3	0.5178	pass30 pass200 gyrats
3	0.5029	pass4 pass30 gyrats
3	0.4820	mair pass4 gyrats
3	0.3951	pass30 pass200 thick
3	0.3912	pass4 pass200 thick
3	0.3867	pass4 pass30 pass200
3	0.3769	pass4 pass30 thick
3	0.2791	pass200 thick gyrats
4	0.8716	mair pass30 pass200 thick
4	0.8704	mair pass4 pass30 thick
4	0.8588	mair pass30 thick gyrats

The SAS System

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The REG Procedure
Model: MODEL1
Dependent Variable: logperm

R-Square Selection Method

Number in Model	R-Square	Variables in Model
4	0.8470	mair pass4 pass200 thick
4	0.8408	mair pass30 pass200 gyrats
4	0.8395	mair pass4 pass30 pass200
4	0.8354	mair pass4 thick gyrats
4	0.8314	mair pass4 pass200 gyrats
4	0.8258	mair pass200 thick gyrats
4	0.6808	pass4 pass200 thick gyrats
4	0.6724	pass4 pass30 pass200 gyrats
4	0.6644	pass4 pass30 thick gyrats
4	0.6014	mair pass4 pass30 gyrats
4	0.5984	pass30 pass200 thick gyrats
4	0.3986	pass4 pass30 pass200 thick
5	0.8787	mair pass4 pass30 thick gyrats
5	0.8784	mair pass4 pass30 pass200 thick
5	0.8717	mair pass30 pass200 thick gyrats
5	0.8474	mair pass4 pass200 thick gyrats
5	0.8409	mair pass4 pass30 pass200 gyrats
5	0.6820	pass4 pass30 pass200 thick gyrats
6	0.8855	mair pass4 pass30 pass200 thick gyrats

The SAS System

---------------------------------- nom=12 ----------------------------------

The REG Procedure
Model: MODEL1
Dependent Variable: logperm

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	6	9.14710	1.52452	14.18	0.0001
Error	11	1.18267	0.10752		
Corrected Total	17	10.32977			

Root MSE	0.32790	R-Square	0.8855
Dependent Mean	2.34139	Adj R-Sq	0.8231
Coeff Var	14.00433		

Parameter Estimates

Variable	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|	Variance Inflation
Intercept	1	-2.33293	2.89379	-0.81	0.4372	0
mair	1	0.45371	0.10261	4.42	0.0010	3.59620
pass4	1	0.05353	0.04643	1.15	0.2733	29.18686
pass30	1	-0.10120	0.05291	-1.91	0.0822	22.11515
pass200	1	0.10920	0.13474	0.81	0.4349	9.80439
thick	1	-0.11650	0.05625	-2.07	0.0627	11.63170
gyrats	1	0.01998	0.02416	0.83	0.4259	6.36762

Correlation of Estimates

Variable	Intercept	mair	pass4	pass30
Intercept	1.0000	-0.8633	-0.8413	0.6505
mair	-0.8633	1.0000	0.7113	-0.4998
pass4	-0.8413	0.7113	1.0000	-0.9120
pass30	0.6505	-0.4998	-0.9120	1.0000
pass200	-0.1429	-0.1094	-0.2347	0.1341
thick	0.0775	-0.2587	-0.5286	0.5337
gyrats	-0.8953	0.8355	0.7653	-0.5241

Correlation of Estimates

Variable	pass200	thick	gyrats
Intercept	-0.1429	0.0775	-0.8953
mair	-0.1094	-0.2587	0.8355
pass4	-0.2347	-0.5286	0.7653
pass30	0.1341	0.5337	-0.5241
pass200	1.0000	0.8454	-0.0766
thick	0.8454	1.0000	-0.2275
gyrats	-0.0766	-0.2275	1.0000