ASPHALT MIXTURE DESIGN CONCEPTS TO DEVELOP AGGREGATE INTERLOCK

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THESIS

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ABSTRACT

From the beginning of asphalt mixture design it was desired to understand the interaction of aggregates, asphalt , and the voids created during their compaction. In asphalt mixture design, guidance is lacking in the selection of the design aggregate structure and understanding the interaction of that aggregate structure and mixture volumetric properties.

This paper presents mixture design concepts that utilize aggregate interlock and aggregate packing to develop an aggregate blend that meets volumetric criteria and provides adequate compaction characteristics. The presented concepts rely on coarse aggregate for the skeleton of the mixture with the proper amount of fine aggregate to provide a properly packed aggregate structure. The objective is to utilize aggregate packing concepts to analyze the combined gradation and relate the packing characteristics to the mixture volumetric properties and compaction characteristics.

The presented concepts include an examination of aggregate packing and aggregate interlock, blending aggregates by volume, a new understanding of coarse and fine aggregate, and an analysis of the resulting gradation.

This study presents comprehensive mix analysis concepts for developing and analyzing hot mix asphalt gradations. It is presented through a rational approach to the selection of relative amounts of coarse and fine aggregate.

Evaluation of gradation with aggregate ratios provides a new tool for examining aggregate gradations. These ratios, based on particle packing, provide distinct relationships with the resulting mixture volumetrics and compaction characteristics.

The results of this study improve the state-of-the-art in asphalt mix design and production by providing a method to characterize HMA mixture volumetrics and

compaction characteristics through the fundamental principles of particle packing. The design concepts outlined in this study provide the foundation for a comprehensive asphalt mixture design method: *The Bailey Method of Gradation Analysis and Asphalt Mix Design.*

This Thesis is
Dedicated to:

My Parents
Whose wise instruction and loving guidance in life have prepared me for the challenges and joys yet to come

Liz
Who stuck by me through the research, writing, and completion of this thesis
and after it all
has agreed to be with me forever

Grandma Vavrik
Whose value of education has driven my efforts to not only complete my education but to strive for excellence in all I do

Grandpa Amore
Whose pride and honor in family has helped me become as good as I can be

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2-D	Two Dimensional
3-D	Three Dimensional
AASHTO	American Association of State Highway Transportation Officials
APA	Asphalt Pavement Analyzer
ATREL	Advanced Transportation Research and Engineering Laboratory
CA	Coarse Aggregate
CA Ratio	Coarse Aggregate Ratio
CMHB	Coarse Matrix High Binder
DUW	Design Unit Weight
E_C	Compressive Modulus
E_E	Extension Modulus
ESAL	Equilivant Single Axle Load
FA	Fine Aggregate
FAc	Fine Aggregate Coarse Fraction Ratio
FAf	Fine Aggregate Fine Fraction Ratio
FSCH	Frequency Sweep Constant Height
G*	Complex Shear Modulus
G_{mb}	Bulk Specific Gravity of Asphalt Mixture
G_{mm}	Maximum Specific Gravity of Asphalt Mixture
G_{sb}	Bulk Specific Gravity of the Aggregate
GTM	Gyratory Test Machine
IDOT	Illinois Department of Transportation
IPC	Industrial Process Controls, Inc.
LUW	Loose Unit Weight
MF	Mineral Filler
MPCS	Mixture Primary Control Sieve
N_{DES}	Number of Design Gyrations
N_{LP}	Number of Gyrations to the Locking Point
NMPS	Nominal Maximum Particle Size

P_a	Air Voids
P_b	Binder Content
P_{be}	Effective Binder Content
PCS	Primary Control Sieve
PG	Performance Graded
RaTT	Rapid Triaxial Test
RSCH	Repeated Shear Constant Height
RUW	Rodded Unit Weight
SCS	Secondary Control Sieve
SGC	Superpave Gyratory Compactor
SHRP	Strategic Highway Research Program
SST	Superpave Shear Test
TCS	Tertiary Control Sieve
UVCATA	Uncompacted Void Content of Aggregate Test Apparatus
VCA_{DRC}	Voids in the Coarse Aggregate – Dry Rodded Coarse Condition
VCA_{MIX}	Voids in the Coarse Aggregate of the Mix
VFA	Voids Filled with Asphalt
VMA	Voids in the Mineral Aggregate
VTM	Voids Total in the Mixture

CHAPTER 1 INTRODUCTION

The design of asphalt mixtures has been studied since the early 1900's. From the beginning of mixture design it was desired to understand the interaction of aggregates, asphalt, and the voids created during compaction. There have been many mixture design methods over the previous century that have furthered the understanding of asphalt mixtures. These mixture design methods rely on the experience of seasoned mix designers and their understanding of local materials. Increased understanding of the effect of aggregate gradation in asphalt mixtures is necessary to advance the design, construction, and performance of these mixtures.

1.1 Asphalt Mix Design

Hot mix asphalt is a combination of aggregate that is mixed with asphalt cement. The objective in the design of asphalt mixtures is to optimize the properties of the mixture with respect to the stability, durability, flexibility, fatigue resistance, skid resistance, permeability, and workability. This is often accomplished only with the evaluation of the volumetric properties of the mixture.

1.1.1 Asphalt Mixture Composition

The volumetric properties of concern in the performance of the asphalt mixture are air voids (P_a), voids in the mineral aggregate (VMA), and voids filled with asphalt (VFA). Air voids in an asphalt mixture are the small air spaces between the coated aggregate particles. VMA is the inter-granular void space between the aggregate particles in a compacted asphalt mixture, including the air voids and the effective asphalt content. VFA is the percentage of the VMA that is filled with asphalt cement. Figure 1-1 shows the density-voids and volume relationships of a compacted asphalt sample.

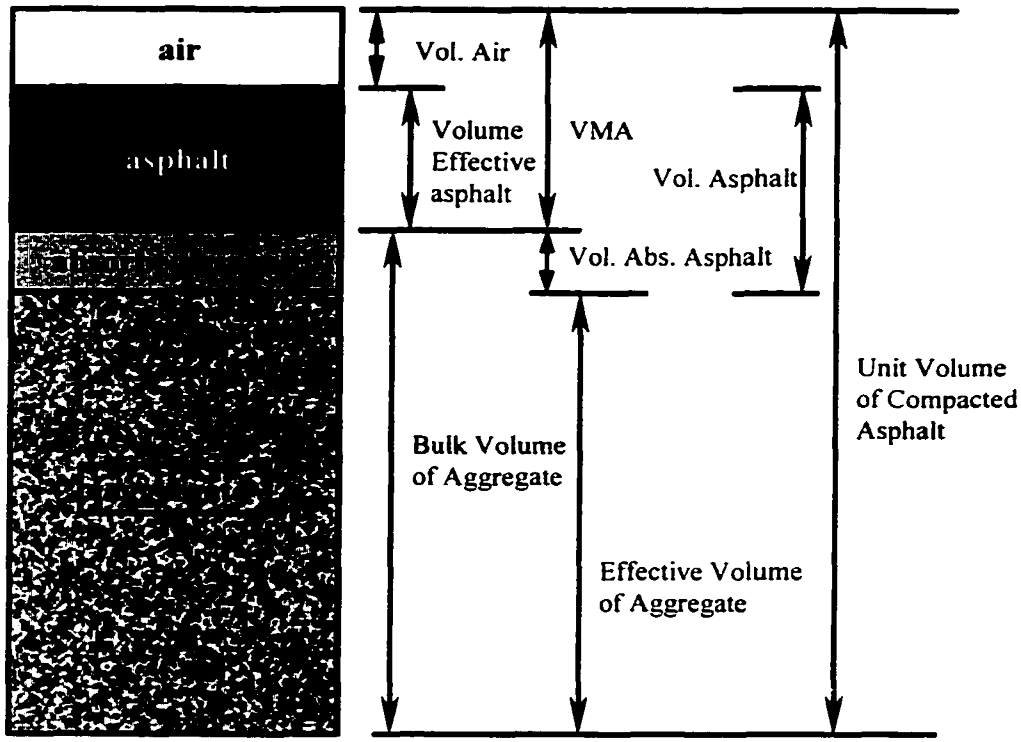


Figure 1-1 Density-Voids and Volume Relationships of a Compacted Asphalt Sample

Current asphalt mixture design methods are empirical design procedures that have been established on the basis of observed field performance. Unfortunately, the specification of volumetric properties in these design methods does not guarantee the performance of the asphalt mixture. A method to achieve the desired volumetric properties by the combination of aggregates is not provided in any of the current asphalt mixture design methods.

1.2 "STATE OF THE ART" IN ASPHALT MIX DESIGN

Asphalt mix design is in a state of change with the advent of the Superpave volumetric mix design method and the phasing out of the Marshall and Hveem Mix design methods. It is envisioned that in the next 5 years the majority of asphalt mixtures in the United States will be design using Superpave based procedures.

1.2.1 Marshall Mix Design

The Marshall mix design procedure involves selecting a trial aggregate gradation and a compaction level, number of blows. The compaction is applied by blows of a falling weight hammer. The trial aggregate gradation is mixed with varying percentages of asphalt cement and then compacted at a specified temperature level, fixed by the viscosity-temperature relationship for the asphalt cement. The voids developed in the compacted samples are then determined and compared to the specification values. Normally, four percent air voids is desired with a VMA requirement that is based on the nominal maximum size of the aggregate blend. If the specified voids cannot be achieved by merely varying the asphalt content, a new aggregate gradation, or even new aggregate materials must be examined.

With the volumetric properties within specification the compacted asphalt sample is tested for stability and flow. The stability is the maximum load carried by the compacted sample at a temperature of 140 F (60 C). The stability is affected significantly by the internal friction of the aggregates. The stability of a mixture can be changed by using different viscosity of asphalt cement or by changing the gradation or quality of aggregate. The flow is the vertical deformation of the sample at the point where the stability starts to decrease. High flow values can indicate a plastic mix that will deform under traffic and low values my point to deficiencies in durability. The values of stability and flow are typically specified.

This design methodology lends no guidance to the selection of the aggregate gradation or to the changes in gradation that may be necessary to produce the appropriate volumetric properties.

1.2.2 Superpave Volumetric Mix Design

Superpave volumetric mix design procedures were developed as part of Strategic Highway Research Program (SHRP) to be a comprehensive system for the design and modeling of asphalt materials. Asphalt binder testing was implemented to tune the performance of the binder to the climate and traffic level. Aggregate quality specifications were established in an effort to improve the performance of the asphalt materials. The control of gradation was through control points and a restricted zone. This restricted zone was included to prevent mixtures from being designed with excessive amounts of natural sand, which can lead to tender mixtures. The gyratory compactor was developed as a laboratory tool that more closely simulates field compaction of asphalt mixtures.

Superpave volumetric mix design is conducted using a trial-and-error aggregate blending process to find a mixture with the appropriate properties at the design compactive effort. The design compactive effort is selected based on the design equivalent single axle loads (ESAL). The laboratory compaction is with the Superpave gyratory compactor (SGC). The SGC is a fixed angle (1.25 deg), fixed pressure (600 kPa), and fixed rate of gyration (30 rev/min) compactor that creates samples 150-mm in diameter. The compacted samples are measured for specific gravity and the volumetric properties are calculated. If the design blend does not meet the volumetric criteria additional gradations are tested.

The design of asphalt mixtures in Superpave remains a trial and error process that relies on local experience. No direct guidance is given to the selection of aggregate gradation for achieving the volumetric design criteria.

1.3 Problem Statement

The Superpave asphalt mixture design procedure provides the tools necessary to design and construct excellent mixtures. This improvement is realized through the use of the Superpave Gyratory Compactor (SGC), aggregate quality specifications, and the mechanical property tests. However, guidance is lacking in the selection of the design aggregate structure and understanding the interaction of aggregate structure with mix design, construction, and performance. It is necessary to develop a method for designing asphalt mixtures that utilizes aggregate interlock and aggregate packing to develop a mixture that meets all volumetric criteria, is easy to construct, and gives excellent performance. This thesis will present mix design concepts that accomplish these tasks and outline a testing scheme to validate the presented concepts.

1.4 Mix Design Philosophy

The mixtures designed and evaluated in this study are based on the following philosophy:

- The strength and rut resistance of an asphalt mixture is best derived from coarse aggregate interlock and proper aggregate packing
- The durability of mixtures is insured with proper mixture design volumetrics, including air voids, VMA, asphalt film thickness, and dust proportion

1.5 Project Objective

The objective of this thesis is to utilize aggregate packing concepts to validate the asphalt mixture design concepts presented by the experience of seasoned asphalt mix designers and develop new procedures and technologies, leading to an improved asphalt mixture design specification. The new design procedure would help provide aggregate interlock, giving resistance to permanent deformation, while maintaining volumetric properties that provide resistance to environmental distress.

1.6 ACKNOWLEDGEMENTS

The concepts and methods validated in this report are based on the years of experience in designing mixtures of Mr. Robert Bailey (retired) Materials Engineer of the Illinois Department of Transportation (IDOT). These ideas were used by Mr. Bailey in the design of asphalt mixtures in District 5 of IDOT and are based on his life experience from farming to the design of concrete and asphalt mixtures.

The concepts put forth by Mr. Bailey were continued by Mr. William J. Pine, formerly Mixture Control Engineer of the IDOT District 5, in the design of asphalt mixtures. The methods have been continued by Mr. Pine and in combination with the author provide the background for this study.

CHAPTER 2 GRADATION ANALYSIS IN ASPHALT MIX DESIGN

A historical description of asphalt mix design and gradation analysis in asphalt mixtures was developed through a literature review. This literature review highlights historical and current research projects, which lend validity and show need for improvement on the current state of the art in selection of aggregate gradation in asphalt mixture design. It presents literature that identifies the current lack of support provided for the selection of aggregate gradation in the design and control of asphalt mixtures.

2.1 Gradation Analysis in Mixture Design

The design of asphalt mixtures has been studied since the 1860's when tar was used in the first bituminous pavements[1]. In these early mixtures the aggregate proportioning was not a major concern, nor was it understood. These tar mixtures did not perform well and the process for the design and construction was not mechanized. The need to examine the asphalt mixture was realized.

In the early 1900's Clifford Richardson examined the mixtures placed in the late 1800's and realized the importance of material selection. Richardson published a book, "The Modern Asphalt Pavement[2]," in 1905 and it is considered an original work in the study of asphalt mixtures. In this book Richardson points out the significant role of aggregate, particularly fine aggregate. Richardson examined the voids in the mineral aggregate (VMA) and air voids as important in the design of these mixtures.

The study of improved aggregate gradation was moved forward by the development of a method for specifying the aggregate gradation. Roy Green developed a procedure for obtaining an ideal dense gradation for large stone asphalt mixtures.[18] Green connected the percent passing the # 200 (0.075-mm) sieve with the top aggregate sieve size on a gradation chart. The intermediate points were then selected as the values of the straight line. This method of aggregate selection was similar to that proposed by Goode and Lufsey[3]. Their method puts forth that the 0.45 power line is the most appropriate method for the graphing of aggregate gradations in asphalt mixtures. Goode and Lufsey omitted a method of drawing a maximum density line. Controversy on the intercept and termination point of this line prompted a study by Huber and Shuler[4] where the most appropriate definition of nominal maximum particle size and maximum particle size was recognized. The maximum density line is now drawn from the origin to the 100 percent passing point of the maximum aggregate size.

These methods for selection of aggregate gradation were initially only used to determine the asphalt demand of the mixture. The need for minimum amount of asphalt binder was recognized and formulas were applied to the gradation for the determination of this amount of asphalt binder to provide adequate durability.

The Hubbard Field method of mix design was an original mixture design method that recognized the importance of air voids in the design and construction of the asphalt pavement[5,6]. Hubbard Field mix design procedures were based on a requirement for air voids and the establishment of a minimum amount of asphalt binder. Specifications were developed to govern the total voids in the mixture and the voids in the aggregate.

The Michigan State Highway Department developed a mixture design method in the early 1930's. This method examined the shape of the gradation curve and evaluated this shape on the basis of a "Gradation Modulus."[7] This "Gradation Modulus" was used to understand the "Bituminous Capacity of Aggregate." This method of gradation analysis is not an optimization of aggregate gradation, rather a method for determining the asphalt content needed for durability.

Several researchers examined the problem of improved gradation by the 1940's. Nijboer[8] experimentally showed that the ideal gradation for maximum packing of aggregate solids occurred when the slope of the log-log gradation chart was 0.45. This agrees with the later work by Goode and Lufsey[3] in their establishment of the "0.45 chart."

Nijboer also pointed out the importance of aggregate particle shape in an asphalt mixture. While he found that the 0.45 slope gave the most dense packing, he also recognized that the combination of round and angular aggregate may lead to decreased voids when compared to angular aggregate alone. He also stated that the most dense state would be for the gradation that contains all round particles. This fact is verified by Huber and Shuler[4] who concluded that rounded gravels produce mixtures with lower VMA than crushed aggregates for the same gradation.

Nijboer stresses the importance of the quantity of coarse aggregate in developing required mechanical properties and plastic deformation in an asphalt mixture. He states that with increasing quantities of coarse aggregate the system will change into one in which the coarse particles form a skeleton. This aggregate skeleton is independent of the largest aggregate size and is only a factor of the amount of coarse aggregate, where coarse aggregate is the largest size of aggregate included in the mixture. Nijboer found that the interlocking resistance provided by coarse aggregate is the best mechanism for resisting permanent deformation in an asphalt mixture. The interlocking resistance was found to increase with the volume concentration of coarse aggregate in the mixture.

When a mixture contains small quantities of coarse aggregate these particles can be considered to be solids moving in a liquid formed by the asphalt and fine aggregate material. Nijboer concluded that this type of mixture will have a decreased resistance to deformation. He concluded that coarse aggregate affects the properties mainly through the quantity present in the mix, but not through the maximum particle size or gradation. Nijboer states that study of middle size aggregates is not necessary if an adequate quantity of coarse aggregate and improved filler-binder ratio.

A later study of gradation of asphalt mixtures and the resulting voids included an examination of aggregate packing. This study by Hudson and Davis[9] points to the fact that gradation specification bands are arbitrarily determined as a result of typical experience. These gradation bands are not necessarily related to the quality of the resulting mixture. Hudson and Davis state that if the criteria based on aggregate bands are to be fully met, aggregate voids must be maintained within definite limits. This study recognizes that the most important characteristic of a gradation is the resulting aggregate voids in the compacted mixture.

Recent studies on the combination of aggregates for asphalt mixture design recognize the importance of the volume of coarse aggregate for improved mixture performance. Ideas put forth by Davis[10] advocate the use of gap graded asphalt mixtures. This gap gradation is desired because the resulting mixture would be considered a high yield strength pavement because of the high amount of coarse aggregate. Similar work by Seward et. al.[11] advocates using increased volume of coarse aggregate for improved performance. This work proposes using a standard load, applied by the SGC, to compact the coarse aggregate, determine the volume of voids remaining, and filling those voids with the dry compacted volume of fine aggregate. Based on the field experience of seasoned mixture control personnel, the resulting mixtures from these proposed design methods may be considered difficult to construct and may have high permeability because of a lack of fine aggregate in the mixture.

Other studies of aggregate gradation optimization concentrate on the component materials to be added to the blend. Ruth[12] has shown that improved, continuous, and "well-balanced" gradation of the coarse aggregate in combination with an increased volume of coarse aggregate will lead to improved mixtures. The fine aggregate and mineral filler contents are limited in these mixtures by design. These mixtures have performed well in high traffic situations.

Studies from the concrete industry on the combined gradation of aggregate in a concrete mixture have yielded similar results to those currently being presented in the asphalt industry. For concrete it has been found that the largest possible volume fraction of coarse aggregate is advantageous with regard to strength and stiffness, creep, drying shrinkage, and permeability[13]. For concrete mixtures the cement paste may be considered the weakest part of the concrete. It is necessary to hold the skeleton of aggregate particles together. If the aggregate is sound and of high quality it is advantageous to ensure that the aggregate skeleton is as closely packed as possible and to bind it with just the right amount of high-quality cement paste to fill the voids between the aggregate particles.

A satirical paper presented by Kight and Crockford[14] presents a method for aggregate gradation selection that is similar to the ideas presented in this thesis. The paper entitled "Tailgates, Beer Mugs, Napkins and No. 2 Pencils – Mix Design on a Budget" presents a method for determining the coarse aggregate volume in a compacted state and filling the remaining voids with fine aggregate in a compacted state. These mixtures have been placed in very heavy truck traffic load applications and have out-performed the Texas Coarse Matrix High Binder (CMHB) materials and a SHRP Superpave mix. The mechanical properties of these "tailgate" mixtures are improved over the CMHB and Superpave mixtures when evaluated in the Rapid Triaxial Test device outlined in Section 6.5 of this report.

A paper by Monismith et. al.[15] presents desirable aggregate characteristics for design. He states that for resistance to permanent deformation in thick lift asphalt sections, aggregates with rough surface texture and dense gradation will give the best performance, but when compacted in thin lifts the surface texture and gradation are of little influence. For fatigue resistance in the thick lift of asphalt pavement it is desirable to have a dense gradation and high stiffness. The fatigue resistance for the thin lift mixtures is improved with the use of more open mixtures, containing low percentages of - #200 (0.075-mm) material, or gap gradations with lower stiffness.

2.1.1 Summary of Gradation Analysis

The ideas presented on the relation of aggregate gradation to the desired properties of an asphalt mixture give mixed recommendations on the appropriate gradation for an asphalt. Many have concentrated on the development of the most dense condition for the entire gradation, feeling that this close packing of the aggregate particles will give the best performance. The realization that adequate void structure is necessary was an early conclusion that continues into present day mixture design concepts. The modification of gradations to achieve the most dense aggregate particle orientation with the desired volumetric properties dominated the design of early asphalt mixtures.

With the advent of increased traffic loading and vehicle weight it became necessary to reexamine the aggregate structure properties that give good performance. With this reexamination the importance of coarse aggregate in the performance of mixtures under the increased load was demonstrated. An increase in the volume of coarse aggregate has not been totally embraced because of the difficulty in construction of these mixtures, and the perceived problems with their durability.

2.2 AGGREGATE STRUCTURE IN ASPHALT MIX DESIGN METHODS

Several mixture design methods have been developed to maximize the material properties of an asphalt mixture. The Hveem Method and Marshall Method were the predominate method for the design of asphalt mixtures until the 1990's with Marshall method being used in 38 states[16]. Since completion of the Strategic Highway Research Program (SHRP) Superpave asphalt mixture design procedures are becoming widely accepted and implemented in the United States with full implementation of expected in the early 2000's. Each of these methods presents ideas on the proper design of aggregate structure and asphalt content to give good field performance.

2.2.1 Hveem Method for Asphalt Mix Design

The Hveem method for asphalt mix design was developed by a resident engineer in California, Francis Hveem[17]. The design method began working with oil mixes in the late 1920's. With the introduction of mechanical paving equipment there was a change in the grade of asphalt cement available for the design of these mixtures. Hveem realized that there was a relationship between the gradation of the aggregate and the amount of asphalt to maintain a consistent color and appearance of the mixture. Hveem originally worked to optimize the asphalt content for the asphalt mixtures.

Later, Hveem realized that the proper asphalt content would not guarantee the performance of the pavement, especially with respect to rutting. He then developed anther test to evaluate the stability of the mixture: the Hveem Stabilometer. This device was intended to measure the ability of the mixture to resist the shear forces applied by wheel loads.

By 1959 the Hveem Stabilometer procedure for the design of asphalt mixtures had evolved into its final form. This procedure was adopted by several states, primarily in the western United States.

The basic philosophy of the Hveem method is summarized as follows:

- The designed mixture should provide sufficient asphalt cement to allow for absorption into the aggregate and to produce an optimum film of asphalt cement on the aggregate
- The designed mixture should produce a compacted aggregate-asphalt cement mixture with sufficient stability to resist traffic
- The designed mixture should contain enough asphalt cement for durability from weathering including effects of oxidation and moisture

These elements of mixture design can be put into a summary statement: "Use a dense, well-graded aggregate with high internal friction without an excess of fines and as much asphalt cement as the mixture will tolerate without losing stability."[18] This summary statement and mix design method represents the extent of guidance on the aggregate structure desired to accomplish these goals of mixture voids and stability and is woefully deficient.

2.2.2 Marshall Method for Asphalt Mix Design

The development of the Marshall Method for the design of asphalt mixtures is well documented.[19,20] This method was developed at the Mississippi Highway Department by Bruce Marshall around 1939. The Army Corps of Engineers at the Waterways Experiment

Station adopted the Marshall procedures for the design of airport asphalt mixtures and standardized the procedures.

The development and evolution of the Marshall Method is based on three criteria in the design mixture: asphalt content, density, and a structural test. Field performance will depend on the highest satisfactory asphalt content at an acceptable density achieved under traffic. In the laboratory the determination of this asphalt content under an appropriate design compactive effort is desired. The Marshall method was developed with a controlling idea that the voids achieved in the laboratory during design must correspond with the density achieved in the field under traffic.

The Marshall Hammer, a fixed weight dropped through a fixed distance, was developed as the tool for the compaction of the asphalt mixtures. After compaction, the samples have their volumetric properties measured and are then tested for their stability (peak strength) and flow (deformation at peak strength). The specification exists to govern the resulting voids and stability.

The Marshall mix design procedure does require that quality aggregate be used in the mixture however it does not give any guidance in the selection of aggregate structure. It is desired in the Marshall method to achieve optimum mixture volumetrics and stability, however no guidelines exist for selecting the aggregate structure to accomplish this goal.

2.2.3 Superpave Method for Asphalt Mix Design

The Superpave mix design method is a direct result of SHRP[21,22]. It was developed to properly design **SU**perior **PER**forming Asphalt **PAVE**ments and to give highway engineers and contractors the tools necessary to design mixtures for different temperatures and traffic loadings. The Superpave system was developed to address the lack of test directly related to mixture performance.

The Superpave system recommends a new compaction method in the Superpave Gyratory Compactor (SGC). This method was selected because it was felt to orient the aggregate particles in a way similar to that observed in the field. This compactor was developed based on evaluation of existing gyratory compactors and is a unique piece of equipment. This compactor operates at a constant vertical pressure, angle of gyration, rate of gyration, and number of gyrations. The number of gyrations is selected based on the traffic level calculated for the pavement.

The Superpave system specifies material quality for all components in the asphalt mixture. The aggregates are evaluated on coarse and fine aggregate angularity, flat and elongated particles and sand equivalent results. These tests were selected from existing tests as best characterizing aggregate quality, as it relates to asphalt mixtures. The asphalt binders are selected based on the high and low service temperature for the pavement. These binders are tested with new testing procedures and upon meeting specification are considered performance graded (PG).

The basis for the selection of the aggregate blend is based on achieving proper mixture volumetrics. The gradation of the aggregate is designed to ensure 1) the maximum aggregate size is appropriate for the application, 2) VMA requirements are met, and 3) a satisfactory aggregate skeleton is obtained.

These goals for the aggregate blend are achieved with a very loose control system for the aggregate blend. The Superpave system controls gradation on the nominal maximum sieve size, the #8 (2.36-mm) sieve and the #200 (0.075-mm) sieve. The requirement for the nominal maximum size is established from 90% to 100%. The control of the #8 (2.36-mm) sieve and the #200 (0.075-mm) sieve is based on the nominal maximum particle size for the mixture.

The procedure for determining the optimum aggregate blend is a trial and error procedure. Superpave states that it is desirable to evaluate 2 to 3 aggregate gradations prior to performing a mix design. If an adequate blend is not found with the 2 to 3 initial blends more blends should be evaluated to find a blend that provides the necessary volumetric properties.

The design of a mixture in the Superpave system is based primarily on the achievement of adequate mixture volumetrics. No guidance is given in the selection of gradation, rather a trial and error process is proposed. Trial and error procedures can be time consuming and costly, therefore an improvement in the method to combine aggregates is necessary to improve the state of the art in asphalt mixtures.

2.2.4 French Method for Asphalt Mix Design

The French have recently developed a new asphalt mixture design system that has evolved into a set of performance related specifications[23]. This mixture design procedure is set up in four levels of mixture design depending on the traffic load of the pavement and the requirements of the mixture. The first of these levels is a volumetric mix design procedure with performance related testing accompanying the other levels of the performance related specifications.

The development of aggregate structure in French asphalt mixtures has developed and changed since the 1950's. After some considerable pavement failures in the mid 1950's and early 1960's the French developed a strengthened asphalt mixture. These mixtures had a considerable increase in the volume of coarse aggregate to make a very coarse graded asphalt concrete that would provide improved performance. This very coarse mixture was used in the entire asphalt pavement thickness. As these pavements reached their design life and required resurfacing a new problem was discovered in that the previously designed very coarse mixtures could only be constructed in thick lifts. The function of the new mixture was no longer to increase the structural capacity of the pavement, but to restore the surface characteristics. The French then turned to the use of gap graded surface mixtures with high binder content. A gap in the gradation with increased asphalt content increased the compactability and surface texture. These mixtures, with their high binder content, performed well in medium traffic, but showed rutting under heavy traffic loads. A further improved ultra-thin asphalt concrete has come to the forefront as the premium surface mixture. This mixture incorporates the use of polymer modified asphalt and is viewed as a porous asphalt concrete with a large portion of coarse aggregate.

The French method for asphalt mixture design is similar to the overall Superpave system in that the first level of mixture design is a volumetric design, followed by increased levels that include mechanical property testing. The guidance for the selection of gradation also appears to be similar in that no specified method for selecting the correct aggregate blend is given in the method. The French still continue to use the very coarse mixtures developed in the 1960's as the structural asphalt material in the lower layers of the pavement structure and have moved to an open graded friction course as the premium surface mixture. The development of these aggregate blends still follows experience and no structured procedure exists to design the aggregate structure in the mixture.

2.2.5 Australian Method for Asphalt Mix Design

Noticing problems with the Marshall designed mixtures used since the turn of the century, the transport authorities joined with transport research and industry to develop and implement a new Australian asphalt mix design procedure[24]. The Marshall and Hubbard Field procedures had given good performance for years, but the tests in those procedures are not related directly to road conditions and do not reliably predict performance under traffic. It is believed that good mixtures under these procedures were derived from the years of experience with local materials rather than from the tests themselves. These methods fall short where the composition of the proposed mixture is outside the limits of local experience, or where traffic is increased beyond local experience. Because of these problems, work began in the late 1980's on a new procedure for asphalt mixture design. That procedure is currently being implemented.

The new design procedure is based on a three level procedure that uses volumetric mix design as the first level and adds performance testing to improve the volumetric design in the later levels. The level one gyratory mix design utilizes the Gyropac gyratory compactor in 100-mm and 150-mm diameters. The level one procedure is based on selecting a target aggregate gradation and materials combination and preparing the samples over a range of asphalt contents expected for the final design. The procedure compacts samples using several levels of gyratory compaction and a level of compaction and resulting voids is selected, thereby giving a design asphalt content. The determination of these parameters includes a refusal density test for the proposed design to verify the void structure after 350 cycles in the compactor. The second level of mixture design adds tests for moisture sensitivity, modulus, and creep with the possibility of testing for fatigue, when necessary. The third level adds a wheel tracking laboratory test and compliance check of all mechanical property tests.

This new procedure for the design of asphalt mixture is improved over the Marshall and Hubbard Field methods because of the improved technology. The use of laboratory performance tests will help assure the performance of the in-place pavement by providing better material characterization prior to construction. Criteria have been developed to establish the limits on the material properties, which gives a reliability to the design.

This new design method, however, still does not provide a method for determining the aggregate structure that will result in adequate test results. The selection of aggregate structure, which will effect the results of the structural tests, is still based on local experience and generalized target gradations. This new mix design procedure will give improved performance, but does not give the mix designer guidance on the necessary aggregate structure to meet the specification.

2.2.6 Summary of Mix Design Concepts

The mixture design procedures that have been used since the inception of asphalt pavements have had the same eventual goal, provide a paving material that resists deformation and cracking to provide an adequate service life. Improvements in materials testing have lead to new design procedures that measure the engineering properties of the asphalt materials and attempt to relate those properties to field performance.

Improvements have also been made in the laboratory compaction of asphalt mixtures. From the early impact compactor used by Marshall to the newly recognized gyratory compactor the technology has improved to provide a product that more closely simulates the compaction and orientation of the asphalt mixture to the field condition. Acceptance of the gyratory compactor as the appropriate tool for laboratory compaction appears to be universal. Procedures developed in the United States, France, and Australia have all recommended the use of gyratory compactors for lab compaction of mixtures and preparation of materials for mechanical property testing.

No procedure is outlined for understanding the interaction of aggregate blending and the resulting mixture volumetrics. The lack of guidance in the selection of the blend of aggregates requires that local experience guide the mix designer to the appropriate design. If local experience is not available, the trial and error process must be used to develop an appropriate mixture. This trial and error process can be time consuming and costly, therefore improvements in the understanding of the combination of aggregates and the resulting mixture would provide a considerable improvement in the state of the art.

2.3 SUMMARY OF GRADATION ANALYSIS IN ASPHALT MIX DESIGN

Clearly the study of aggregate gradation is important in the characterization of an asphalt mixture. The relative amounts of the component aggregate materials will govern the material properties of the resulting mixture. Several researchers have realized the importance of increasing coarse aggregate, yet no well accepted design procedure that establishes the appropriate volume of coarse aggregate exists. This study will examine the importance of the volume of coarse aggregate as well as the effect in change of gradation on the resulting mixture volumetric properties. The study will provide a foundation for a systematic design procedure for the selection of the proper volume of coarse aggregate and continuous gradation that is necessary to give improved performance of dense graded asphalt mixtures, while also providing tools for the evaluation of aggregate gradation.

CHAPTER 3 REVIEW OF AGGREGATE AND PARTICLE PACKING

The study of particle packing is necessary to understand the basis for the combination of aggregates in an asphalt mixture. Considerable work has been recorded on the combination of particles and the resulting voids without a solution that provides the answer to the problem of particle packing. This review will examine the portions of particle packing that are relevant to asphalt mixture design methodology in an effort to validate the proposed methods.

This literature review will present information on the size of particles that fill the voids created by the packing of larger particles, which leads to an overall gradation. The ability of aggregates to fit together in a manner that can be captured by a gradation specification is the central issue of this thesis. This will be accomplished by examination of the packing of spheres and through the examination of experiments carried out on the optimization of aggregate packing as it impacts the behavior of an asphalt mixture.

3.1 PACKING OF SPHERES

The idealized packing of spheres is often used as a starting point to evaluate the packing of aggregate particles. The theoretical models for the packing of spheres are used to understand the general concepts that govern the packing of aggregate particles.

The study of uniform spheres is used as background material in rock physics, seismic analysis and ceramics[25, 26]. In the geometric packing of single sized spheres several particle orientations exist for ordered packing including, cubical, orthorhombic, tetragonal, pyramidal, hexagonal, and tetrahedral. Of interest in this study is the size of the maximum sphere fitting in the narrowest channel created by the packing of the unit sphere. These sizes can be used to calculate a particle diameter ratio

($\frac{\text{Particle Fitting in Void}}{\text{Large Particle Creating Void}}$). This particle diameter ratio has a range from 0.42 for the simple cubical packing to 0.155 for the tetrahedral, or hexagonal close packed, packing[26, 27].

Several models exist for evaluating the densities resulting from combining two sizes (binary packing) of spheres. These models are generally divided in to two distinct groups, those with particle diameter ratios below 0.22 and those with particle diameter ratios above 0.22[13]. Experimental results with the packing of spheres show that the model by Aim and Goff[28] gives the best fit to the experimental data for particle diameter ratios below 0.22. The model by Toufar et al.[29, 30] gives the best fit for particle diameter ratios above 0.22. Toufar states that the smaller particles, for diameter ratios greater than 0.22, will actually be too large to be situated within the interstices between the larger particles. Based on the split in applicability of the models naturally occurring at a particle diameter ratio of 0.22 it is felt that this ratio is applicable for describing a particle size that fills the void, rather than a particle size that is larger than the created void.

In the study of gap gradations for asphalt Davis[10] has suggested that the proper size of a sphere to perfectly fill the void created by the intersection of other spheres would have a diameter ratio of 0.3. In this paper Davis does not give the background for such an assumption, but does state the there is a considerable increase in the volume concentration of aggregate when in a binary mixture of spheres the second size particles has a diameter ratio of 0.3 or smaller.

3.2 Optimization of Aggregate Packing

The study of optimization of packing has primarily been undertaken in the concrete industry[31, 32, 33, 34, 35]. Work in the optimization of aggregate gradation has improved the state of the art and the rheological properties of cement and concrete, which has lead to improved performance in many applications.

The development of ultra high performance concrete has also utilized the ideas of particle packing.[32] It is necessary when developing an ultra high performance concrete to include aggregate to create some void space for the ultra high performance mortar. The use of properly sized aggregate is important for the creation of this material. Experimental data shows that an aggregate with a characteristic size of 250 μm is optimized with a cement mortar created by cement with a continuous gradation and nominal maximum size of 63 μm. The particle diameter ratio for the sized materials is 0.25.

Work by Shilstone in the design of concrete mixtures has been received with mixed reaction[33, 35]. The analysis of gradation for general use concrete that was introduced is not in line with the traditional design of concrete mixtures. The Portland Cement Association method for design of concrete mixtures advocates blending aggregates that meet the quality and gradation specifications given by ASTM but do not analyze the resulting gradation[36]. Shilstone uses the idea of blending the aggregates by volume and adds an analysis of gradation for the design of mixtures with improved performance[33, 35]. Shilstone advocates the use of a percent retained graphical analysis procedure to ensure balance in the gradation. The primary purpose of this analysis is to avoid a gap graded mixture, which would decrease the concrete's rheological characteristics.

Work conducted in Wisconsin validates the principles put forth by Shilstone. A study by the University of Wisconsin and the Wisconsin Department of Transportation has shown that with an optimized aggregate gradation an increase in strength of 10 to 20 percent is observed. They also noticed decreased segregation after extended vibration, which leads to quality construction and long pavement life.

Testing of different aggregates has revealed that the percentage of voids in the compacted state range from 41% to 32% depending on the maximum size and gradation of the aggregate[34]. These values are typically 33% to 38% when continuous gradation of sands are examined[37]. When combining two aggregates it is possible to achieve voids as low as 23%. The addition of a third aggregate source will reduce the voids to values as low as 18%. These results on the voids in aggregate are very dependent on the particle shape of the aggregates[38].

Recent research on the theoretical close packing of spheres shows that the random close packing of spheres is an elusive topic. The definition of close packing is arguably difficult to describe mathematically[39]. This research, which applies previously unused packing philosophies, may change the way that engineers design composite materials from solid chemicals for industrial use to portland cement concrete and hot mix asphalt[40].

3.3 Summary of Particle and Aggregate Packing

The overall goal of optimization of aggregate packing has a very important result. If a space is to be filled with stone and a structural strength is required, it is evident that a piece of stone that entirely fills the space would have the greatest structural strength. Smaller pieces of stone, no matter how well packed, will never achieve the stability of the solid piece of stone. If there is no piece of stone that will completely fill the space, it is evident that the largest piece of stone that can be fitted into the space will give greater structural strength than a smaller piece. If the largest space left is filled with the largest piece that can be fitted into it, the strength is further increased.

The studies conducted to date on this topic lend credence to the use of the results of sphere packing studies as an appropriate estimate for the geometric relations of the packing of granular materials. The review of literature has presented evidence that particle packing concepts provide an adequate background for the continued study of aggregate gradations. Further, a particle diameter ratio of 0.22 would appear to be an appropriate value to study for evaluating the gradation of asphalt concrete. This particle diameter ratio is considered be the most appropriate value based on the current state of the art for the examination of aggregate gradations in HMA mixtures.

CHAPTER 4 DESIGN CONCEPTS

Asphalt mixtures have traditionally been designed using a trial and error procedure to select the aggregate gradation. Aggregates are combined in "typical" percentages that were developed from an empirical database of information extending back to the beginning of asphalt mix design. No mix design method has been presented that provides a means to design the aggregate structure in the asphalt mixture.

New mix design concepts will be presented that are based on the following philosophy:

- The strength and rut resistance of an asphalt mixture should be derived from aggregate interlock and proper aggregate packing
- The durability of those mixtures will be insured with proper mixture design volumetrics, including air voids, VMA, asphalt film thickness, and dust proportion

4.1 COMBINATION OF AGGREGATES

The primary components in asphalt mixtures are typically defined as coarse aggregate, fine aggregate, mineral filler, and asphalt binder. These aggregate components are ostensibly combined to provide an aggregate skeleton that will resist permanent deformation and cracking. It is necessary to examine the packing of aggregates and the characteristics of the components in order to understand their behavior as a mixture and to generalize on their performance properties.

4.1.1 Aggregate Packing Characteristics

Naturally occurring aggregate particles can not be packed together to completely fill a unit volume. Void space will always exist between the particles. With an understanding of aggregate packing it should be possible to design the void space in the aggregate structure to accommodate the asphalt cement and produce desirable voids in an asphalt mixture. Aggregate packing to fill a unit volume will depend on:

- Compactive Energy
- Shape of Aggregate Particles
- Surface Texture of Aggregate Particles
- Gradation of Aggregate Particles

Several methods for inputting compactive energy exist for the compaction of aggregate particles and asphalt mixtures. Static pressure, Impact loading (e.g. Marshall Hammer), shearing (e.g. Gyratory Compactor), or kneading are all methods that have been used to compact asphalt mixtures. Under each of these compaction methods, increased density can be achieved with increased compactive energy.

Characteristics of individual aggregate particles can influence the final density. The shape of the aggregate particles alters the level of aggregate packing and resulting density of the mixture[38]. Rounded particles tend to arrange in a more dense configuration than do irregular or elongated particles. The surface texture of the particles can also alter the final density and aggregate packing because of the friction created between the particles during the compaction. Particles with smooth textures may more easily reorient into a dense configuration, while particles with rough textured surfaces may resist sliding against one another into the low density configuration.

Changing the aggregate gradation changes the particle packing. Single sized particles will not pack as dense as a mixture of two sizes. The packing of a continuous gradation is effected by the overall gradation of the mixture. The relationship between density and change in gradation is currently not fully explained, and is a subject of this study.

4.1.2 Coarse Aggregate

Coarse aggregate is considered the primary deformation-resisting component in asphalt mixtures. The coarse aggregate particles, through their interlock, provide a path for the applied pavement stresses to be carried within the asphalt mixture and transmitted to the lower pavement layers. The aggregate interlock also provides a skeleton that resists deformation.

In order to resist deformation it is necessary to provide aggregate interlock. This interlock occurs in various levels in an asphalt mixture. Mixtures possessing high levels of aggregate interlock will resist deformation under high load repetitions, increased ESAL loading, while mixtures containing lower levels of aggregate interlock will only resist deformation at lower load levels. Therefore, it is necessary to characterize and quantify this interlock to determine the load carrying capacity provided by the mixture. By understanding the load carrying capacity needed for the pavement application during design, it is possible to properly design mixtures to satisfy the requirements of the pavement.

In order to quantify the amount of coarse aggregate interlock in a mixture an investigation of coarse aggregate packing is necessary. To explain coarse aggregate interlock, the volumetric properties of the coarse aggregates will be examined.

4.1.3 Fine Aggregate

In asphalt mixtures, the fine aggregate completes the aggregate structure by creating a support structure for the void spaces in the coarse aggregate. Without the inclusion of this fine aggregate, the mixture would remain open with a high percentage of voids. This type of mixture, with high coarse aggregate interlock and little fine aggregate to fill the voids, is characteristic of an open graded friction course.

Because the fine aggregate is viewed as filler material to the coarse aggregate, it is desired that this fine aggregate be in a compacted state. With the fine aggregate in this compacted state the amount of permanent deformation due to shear flow in this fraction of the aggregate blend should be minimized. This maximization of the fine aggregate consolidation makes the amount of permanent deformation accumulated in the mixture a function of the coarse aggregate interlock.

Fine aggregate is viewed as a filler material for the voids in the coarse aggregate. This fine aggregate is added to the coarse aggregate in a densified state to minimize the densification of the fine aggregate of the mixture. The fine aggregate in a compacted state reduces the amount of permanent deformation due to shear flow in this fraction of the aggregate blend. With the densification and deformation of the fine aggregate minimized, it becomes possible for the coarse aggregate to carry the bulk of the applied load. Deformation resistance must come from the coarse aggregate because the fine aggregate structure, when not densified correctly, is typically subject to increased volumetric densification and is typically not strong enough to resist deformation and shear flow under load.

4.1.4 Mineral Filler

Mineral filler is used in the aggregate blend for several purposes. One purpose is to develop mastic. The properties of the mastic contribute to the properties of the mixture, especially as it relates to mixture stiffness and low temperature performance. From an aggregate combination perspective, mineral filler is used to fill the voids in the mixture that are created by the fine aggregate. The total combination of aggregates to develop a dense graded asphalt mixture must include all of the particle sizes down to and including the filler size aggregates.

4.1.5 Putting them Together

For the purposes of blending aggregates, consider filling a unit volume with aggregate. If the coarse aggregate is to compose the primary component in this unit volume and provide coarse aggregate interlock the appropriate amount of coarse aggregate must be determined, between a minimum and maximum value. To ensure aggregate interlock the minimum volume of coarse aggregate that can be added to the mixture is the amount to fill the unit volume with the coarse aggregate in its loosest state. This amount of coarse aggregate will provide a considerable amount of void space, which must be filled by the smaller fine aggregate. The maximum volume of coarse aggregate that can be added to the unit volume is the amount to fill the unit volume with the coarse aggregate in a compacted state under a specified compactive effort.

Asphalt mixtures with coarse aggregate volumes between the loose state and the compacted state would be considered to have a degree of aggregate interlock relative to this proportion of coarse aggregate. Mixtures that fall between these values could then be considered deformation-resisting mixtures, which based on their relative amount of coarse aggregate, could withstand various levels of traffic with varying permanent deformation.

It is possible to have a mixture with a volume of coarse aggregate less than the amount in the loose state with the excess volume then being filled with more fine aggregate. This mixture would derive its deformation resistance from the fine aggregate structure, which is not desirable. This study will focus on mixtures where the coarse aggregate provides for the aggregate skeleton in the mixture.

Upon establishing the volume, determined as a percentage of the unit volume and selected as a unit weight, of coarse aggregate desired in the mix, it becomes necessary to determine the relative percentages of the other aggregates. The amount of fine aggregate and the amount of mineral filler must be determined so that the voids established by the coarse aggregate are filled with the appropriate volume of filler aggregate and the proper mastic properties are achieved.

4.2 Design Method to Develop Aggregate Interlock

In order to develop a method for combining aggregate to optimize the aggregate interlock and provide the proper volumetric properties, it is necessary to understand some of the controlling factors that affect the design and performance of these mixtures. The explanation of coarse and fine aggregates given in the previous section has provided a background for understanding the combination of aggregates. The design method presented below builds on that understanding and provides additional insight into the combination of aggregates for use in an asphalt mixture.

This new method to combine aggregates to produce a high quality mix design requires the understanding of two concepts:

1. Developing a more fundamental definition to distinguish between coarse and fine aggregate
2. Combining aggregates by volume to ensure coarse aggregate interlock

4.2.1 Coarse vs. Fine

It is necessary to understand that the previous discussion of coarse aggregate and fine aggregate was developed from traditional analysis techniques. Tradition offers that coarse aggregate is the larger size particles typically greater than #4 sieve size material (gravel size material). Fine aggregate is any aggregate that is less than the #4 sieve size material (sand, silt, and clay size material). For the purposes of this study, it is necessary to change those definitions in order to properly analyze a mixture gradation and determine the packing and aggregate interlock provided by the combination of all aggregates in the mixture.

In this study analysis of aggregate blending for asphalt mixtures will use the following definitions of coarse and fine aggregate.

Coarse Aggregate: Large aggregate particles, which when placed in a unit volume, creates voids.

Fine Aggregate: Aggregate particles that fill the voids created by the coarse aggregate.

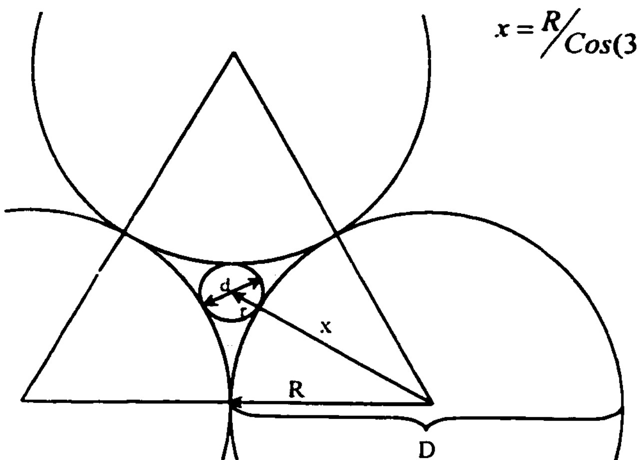
It can be seen from this definition of coarse and fine aggregate that there is no aggregate size associated with the words coarse or fine. Therefore, it is possible to have a fine aggregate in a traditional "coarse" aggregate fraction as well as coarse aggregate in a traditional "fine" aggregate fraction. This definition of the size difference between coarse particles and fine particles provides a fundamental relationship that helps understand the interaction between particles of various sizes..

4.2.1.1 2-D Analysis of Particle Size

The two-dimensional analysis of aggregate shape is based on four combinations of geometry, with the following dimensional relationships:

1. All round faced particles, shown in Figure 4-1, produces a ratio of 0.15
2. 2 round faces, 1 flat face, shown in Figure 4-2, produces a ratio of 0.20
3. 1 round face, 2 flat faces, shown in Figure 4-3, produces a ratio of 0.24
4. All flat faced particles , shown in Figure 4-4, produces a ratio of 0.29

A 0.22 size ratio is the average of these four different combinations of two-dimensional particle combinations, and appears to best represent an average condition. While the actual size ratio would vary depending on the particles included in the mixture, an average value is certainly typical and would seem to be applicable to particle arrangements of randomly shaped particles as are found in an asphalt mixture.



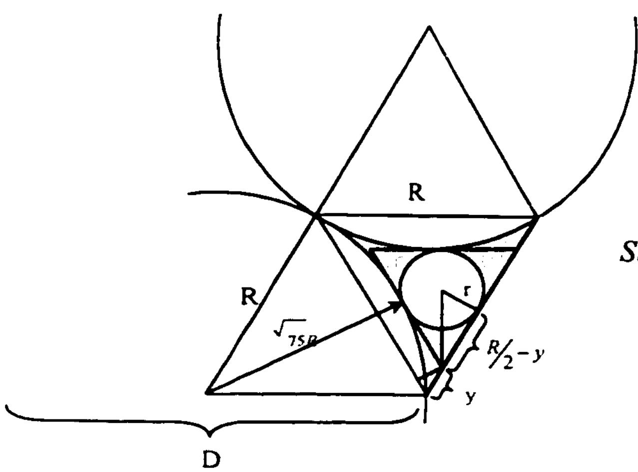
$$x = R/Cos(30)$$

$$\begin{aligned} d &= 2\left(R/Cos(30) - R\right) \\ &= 2R\left(1/Cos(30) - 1\right) \\ &= D\left(1/Cos(30) - 1\right) \end{aligned}$$

$$\left.\begin{aligned} r &= x - R \\ d &= 2r \end{aligned}\right\} \quad d = 2(x - R)$$

$$d = 0.155\,D$$

Figure 4-1 Two-Dimensional Packing of All Round Particles



$$Tan(30) = r/0.5\,(R - 2\,y)$$

$$Tan(30) = r/0.5\,(0.5\,D - 0.15470\,D)$$

$$Tan(30) = r/0.17265\,D$$

$$Sin(60) = \frac{\left(1 - \sqrt{.75}\right)R}{y} \qquad y = \frac{\left(1 - \sqrt{.75}\right)R}{Sin(60)}$$

$$y = 0.07735\,D$$

$$r = 0.09968\,D$$

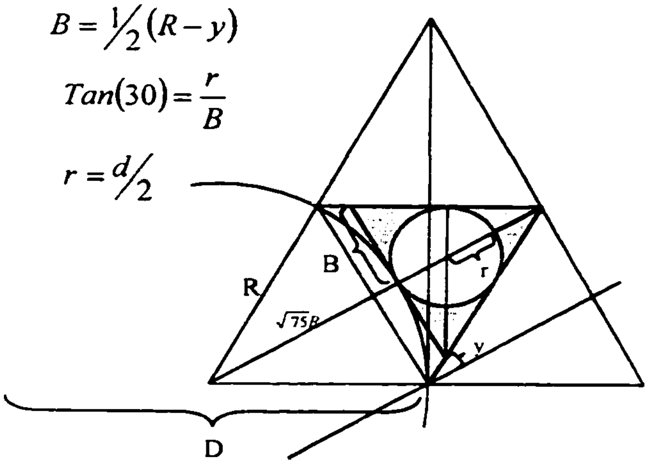
$$d = 0.199\,D$$

Figure 4-2 Two-Dimensional Packing of 2-Round and 1-Flat Particle

$$B = \frac{1}{2}(R - y)$$

$$Tan(30) = \frac{r}{B}$$

$$r = \frac{d}{2}$$



$$Tan(30) = \frac{d/2}{\frac{1}{2}\left(\frac{D}{2} - \frac{\left(1 - \sqrt{.75}\,\frac{D}{2}\right)}{Tan(60)}\right)}$$

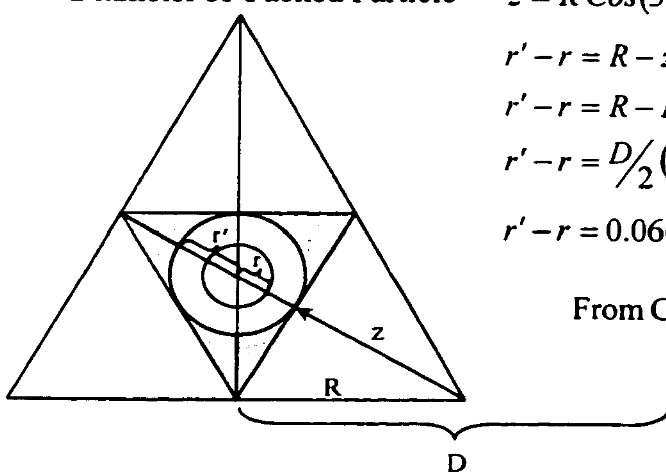
$$Tan(60) = \frac{\left(1 - \sqrt{.75}\,R\right)}{y} \quad \therefore y = \frac{\left(1 - \sqrt{.75}\,R\right)}{Tan(60)}$$

$$Tan(30) = \frac{d}{0.42264\,D}$$

$$d = 0.244\,D$$

Figure 4-3 Two-Dimensional Packing of 1-Round and 2-Flat Particle

d' = Diameter of Packed Particle



$$z = R\,Cos(30)$$

$$r' - r = R - z$$

$$r' - r = R - R\,Cos(30)$$

$$r' - r = \frac{D}{2}\left(1 - Cos(30)\right)$$

$$r' - r = 0.06699D$$

$$R = \frac{D}{2}$$

$$d' = 2r'$$

From Condition #1: $d = 2r = 0.1547D$

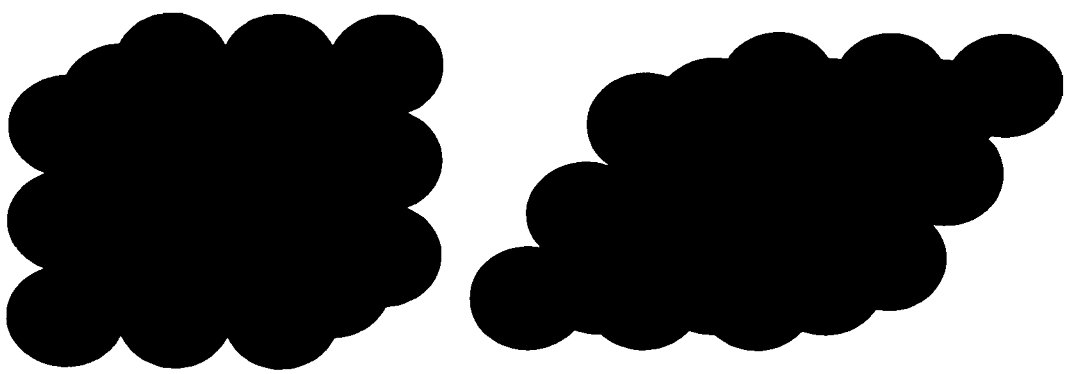
$$d' = 0.289\,D$$

Figure 4-4 Two-Dimensional Packing of All Flat Particles

4.2.1.2 3-D Analysis of Particle Size

Results of the literature review, provided in Chapter 3, provide guidance in the selection of the characteristic size of the void created by combining spheres but do not solve the problem. It is apparent that further study into the packing of particles is necessary to precisely determine the void relationships of packed particles. It has been determined that the characteristic diameter of the void in a packed system would be in the range from 0.15, from the tetrahedral packing of spheres, to 0.42, from the cubical packing of spheres.[25, 26, 27] These packing configurations are shown in Figure 4-5. Because the packing of the aggregates is desired to be between cubical and tetrahedral, yet more similar to the tetrahedral packing, providing the more stable configuration, the 0.22 particle size ratio is reasonable for use in this study.

The literature has shown that the 0.22 size ratio that was presented in two-dimensions is also validated through the use of three-dimensional analysis. Theoretical models of aggregate packing have shown that different models are necessary to describe the behavior of binary mixtures when the component sizes are similar and when one is much smaller than the other.[13] From the literature review it is seen that the 0.22 particle diameter ratio is most commonly used as this characteristic size.



Cubical Packing of Spheres

Tetrahedral Packing of Spheres

Figure 4-5 Ordered Packing Arrangements of Uniform Spheres

4.2.1.3 Primary Control Sieve

Determining the exact particle size ratio developed in a mixture is an important problem that has been studied by many researchers but never adequately resolved. This project does not attempt to solve the theoretical problem of particle packing, rather to use most appropriate recommendations from the current research as a guide to directing the combination of aggregates for asphalt concrete mixture design.

The analysis of gradation for asphalt mixtures has been standardized in the United States through the use of a standard set of sieves. These sieves are 1½-in., 1-in., ¾-in., ½-in., ⅜-in., #4, #8, #16, #30, #50, #100, and #200 (37.5-mm, 25-mm, 19-mm, 12.5-mm, 9.5-mm, 4.75-mm, 2.36-mm, 1.18-mm, 0.600-mm, 0.300-mm, 0.150-mm, 0.075-mm). Application of the particle diameter ratio to the standard set of sieves gives the primary control sieve (PCS). The primary control sieve is then defined as the closest sieve to the nominal maximum sieve size in millimeters multiplied by 0.22. The complete list of standard sieve sizes for asphalt mixtures and the corresponding PCS size is given in Table 4-1. There is a standard sieve size matching each PCS reasonably well, therefore the standard set of sieves is adequate in analyzing an asphalt gradation using the 0.22 size ratio.

Table 4-1 Standard Sieve Sizes and Associated Primary Control Sieves

Particle Size		Particle Size x 0.22	Primary Control Sieve	
US Std.	*mm*	*mm*	*US Std.*	*mm*
1-1/2"	37.5	8.25	3/8"	9.5
1"	25	5.5	#4	4.75
3/4"	19	4.18	#4	4.75
1/2"	12.5	2.75	#8	2.36
3/8"	9.5	2.09	#8	2.36
#4	4.75	1.05	#16	1.18
#8	2.36	0.52	#30	0.600
#16	1.18	0.26	#50	0.300
#30	0.600	0.13	#100	0.150
#50	0.300	0.07	#200	0.075

Because the true particle diameter ratio will change in every mixture, an analysis was performed to determine if an adjusted ratio would change the PCS when using the standard set of aggregate sieves. The result is that ratios in the range from 0.27 to 0.20 give the same PCS as the 0.22 size ratio for the standard set of sieves.

For the purposes of asphalt concrete mixture design the 0.22 relationship adequately defines the size difference between coarse aggregate and fine aggregate with sufficient accuracy applicable to current sieve sizes. On average it can be expected that a particle with a characteristic dimension of 0.22 will fill the void created by the larger particle. This fine aggregate particle with the 0.22 dimensions is then referred to as a filler aggregate, as it serves to fill the created voids.

4.2.1.4 Examination of Standard Sieve Sets

Examination of the PCS for the standard set of sieves used in asphalt mixture design, shown in Table 4-1, shows that the PCS is not unique for all standard sieve sizes. For 1/2" (12.5-mm) and 3/8" (9.5-mm) sized aggregates the associated primary control sieve is the #8 (2.36-mm). This shared primary control sieve is also seen in the 1" (25-mm) and 3/4" (19-mm) sized aggregate particles. With the void characteristics of aggregates governed by particle packing principles a question of the adequacy of the standard sieve sizes is raised.

The selection of standard sieve sizes did not follow the 0.22 particle diameter ratio derived from aggregate packing. The standard sieve sizes follow a 0.5 particle diameter ratio for the #4 (4.75-mm) sieve and smaller, and an alternating 0.76 and 0.66 pattern for the larger size material. The particle size ratios for the standard set of sieves is shown in Table 4-2. The standard sieve sets do not appear to be based on particle packing principles, however they provide adequate sieves for the traditional characterization of aggregate gradation.

Table 4-2 Standard Sieve Sizes and Associated Particle Size Ratio

Particle Size		Particle Size Ratio
US Std.	*mm*	
1-1/2"	37.5	
1"	25	0.67
3/4"	19	0.76
1/2"	12.5	0.66
3/8"	9.5	0.76
#4	4.75	0.50
#8	2.36	0.50
#16	1.18	0.50
#30	0.600	0.51
#50	0.300	0.50
#100	0.150	0.50
#200	0.075	0.50

4.2.2 Combination by Volume

Current practice involves combining aggregates on a weight basis; however, aggregates must be combined by volume if aggregate interlock is to be achieved. Furthermore, combining aggregates by weight does not offer the mix designer the information necessary to develop a numerical parameter to evaluate the degree of aggregate interlock because differing specific gravities will produce different quantities of each particle size for the same weight. In order to accomplish this volumetric combination of aggregates additional information must be gathered. For each of the coarse aggregates the loose and rodded unit weights must be determined, and for the fine aggregate the rodded unit weight is necessary. These measurements provide the volumetric data at the specific void structure required to evaluate interlock properties.

In an effort to keep the nomenclature in line with current practices the aggregate volumes will be expressed as unit weights. It is easier for mix designers and quality control personnel to understand increasing the unit weight of aggregate than to understand a change in aggregate volume.

4.2.2.1 *Loose Unit Weight of Coarse Aggregate*

The loose unit weight of an aggregate is determined using the Uncompacted Void Content of Aggregate Test Apparatus (UVCATA)[41]. This apparatus is as described in Chapter 6. This test deposits a representative sample of the whole coarse aggregate in a standard dimension bucket from a standard fall, putting the aggregate in a standard loose condition in the bucket. By knowing the volume of the bucket, the weight of aggregate deposited from a standard fall, and the aggregate bulk specific gravity, the volume of voids in the coarse aggregate can be determined. This volume of voids is the volume present when the particles are just into contact without any outside compactive effort being applied. The reported value from this test is the loose unit weight, converted to pounds per cubic foot, of the aggregate. Figure 4-6 shows the loose unit weight of the coarse aggregate.

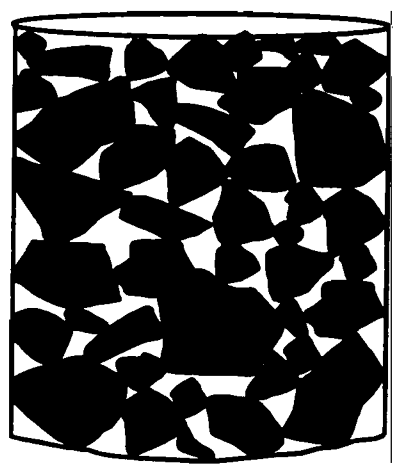


Figure 4-6 Loose Unit Weight of Coarse Aggregate

4.2.2.2 *Rodded Unit Weight of Coarse Aggregate*

The rodded unit weight of an aggregate is determined using procedures similar to the Unit Weight and Voids in Aggregate Procedure outlined in AASHTO T 19. A modification to this procedure will be used for this experiment. As an alternative to the $^1/_4$ cubic foot bucket that is typically used for the unit weight test the bucket from the

UVCATA will be used. This allows direct comparison of the results from the loose unit weight test. Because the test apparatus and the volume of material is scaled down from the ¼ cubic foot bucket to the UVCATA bucket no difference is observed in the results of testing from these different apparatus. It is customary to use the same test apparatus for the loose unit weight and rodded unit weight measurements, because use of different apparatus can result in different compactive energy applied to the sample.

The rodded unit weight is determined by dropping the complete gradation of coarse aggregate into the bucket in three equal lifts applying 25 rods with a ⅝" diameter steel rod per lift. The rodded unit weight is combined with the volume of the bucket and the bulk specific gravity of the aggregate to determine the volume of voids of the coarse aggregate when the particles have undergone compaction and consolidation. The reported value from this test is the rodded unit weight, converted to pounds per cubic foot, of the aggregate. Figure 4-7 shows the rodded unit weight of the coarse aggregate.

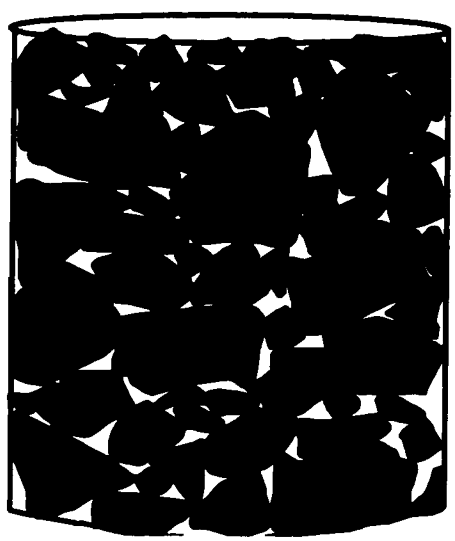


Figure 4-7 Rodded Unit Weight of Coarse Aggregate

4.2.2.3 *Design Unit Weight of Coarse Aggregate*

The design unit weight of the coarse aggregate sets the level of coarse aggregate interlock that will result in the aggregate blend by fixing the volume of coarse aggregate in the mixture. The design unit weight is selected by the mix designer and under the proposed method is typically chosen to be near the loose unit weight because the loose unit weight is considered the lower limit of coarse aggregate interlock. The rodded unit weight is considered the upper limit of coarse aggregate interlock. The design unit weight can be below the loose unit weight for a mixture, however as the design unit weight is lowered beyond the loose unit weight, theoretically, the mixture has no coarse aggregate interlock. However, it has been found that the densest gradation that can exist is with the design unit weight approximately 5% below the loose unit weight of coarse aggregate[42]. The most dense condition could change if the testing methods for determining the loose and rodded unit weights was modified. Figure 4-8 shows the design unit weight.

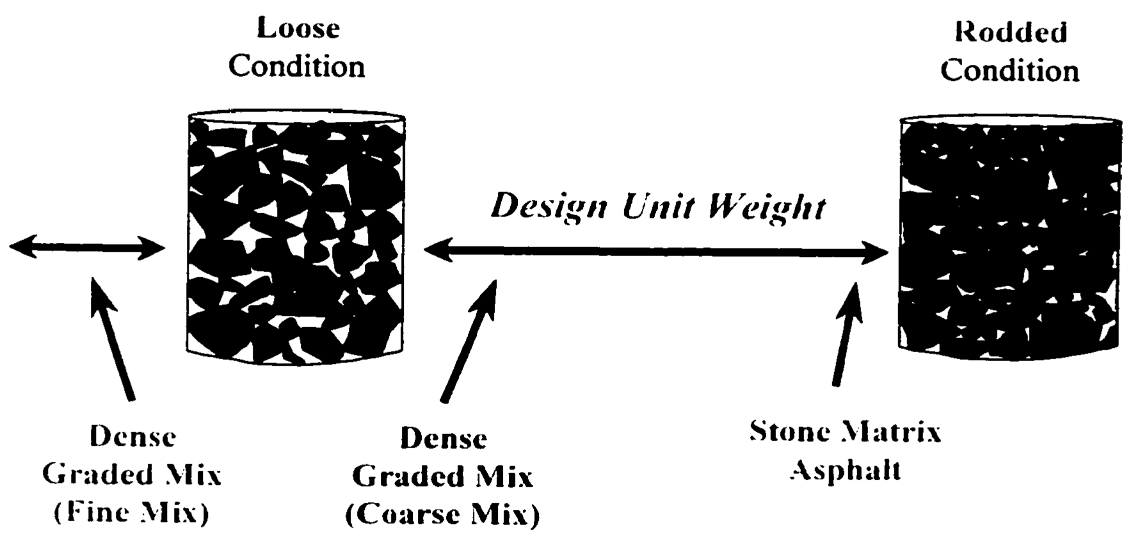


Figure 4-8 Design Unit Weight

The design unit weight also affects the ability to construct a pavement with the mixture. With the design unit weight increased to close to the rodded unit weight the amount of compactive effort required for densification increases. This may make a mixture difficult to construct. If increased voids in design are desired, strengthening the fine aggregate structure may provide a more appropriate method for changing the volumetric properties, as opposed to increasing the design unit weight of the coarse aggregate.

Upon selection of the appropriate design unit weight it is necessary to fill the voids created by the coarse aggregate. These voids are filled with the appropriate volume of fine aggregate in its compacted state.

4.2.2.4 Fine Aggregate Rodded Unit Weight

Selecting the amount of fine aggregate used to fill the voids created by the coarse aggregate done with fine aggregate in a state of dry compaction. With the fine aggregate in a compacted state, the densification of the mixture due to the compaction of fine aggregate is minimized. The fine aggregate in a compacted state reduces the amount of permanent deformation due to shear flow in this fraction of the aggregate blend. This state of dry compaction is the rodded unit weight of the fine aggregate.

The rodded unit weight is determined by dropping a representative sample of the whole aggregate into the standard bucket in three equal lifts with 25 rods per lift, using a 5/8" diameter steel rod. The rodded unit weight, when combined with the volume of the bucket and the bulk specific gravity of the aggregate, are used to determine the volume of voids of the fine aggregate when the particles have undergone compaction and consolidation. The compacted voids in the fine aggregate are useful in the evaluation of different fine aggregate sources, but is not required in the determination of the amount of aggregate required to fill the voids in the coarse aggregate. The reported value from this test is the rodded unit weight, converted to pounds per cubic foot, of the aggregate, which is used to determine the appropriate amount of fine aggregate in the mixture. Figure 4-9 shows the rodded unit weight of fine aggregate.

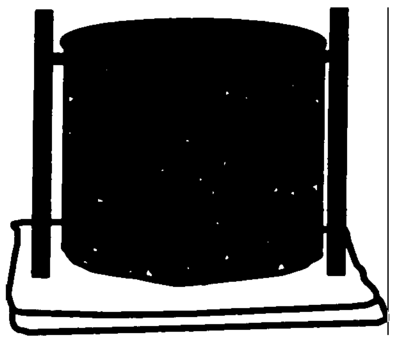


Figure 4-9 Rodded Unit Weight of Fine Aggregate

4.2.3 Summary of Design for Aggregate Interlock

The proposed procedure for the blending of aggregates to achieve aggregate interlock is as follows:

1. Determine loose and rodded unit weights of coarse aggregate
2. Determine rodded unit weight of fine aggregate
3. Select the design unit weight of coarse aggregate
4. Fill the remaining void space created by the coarse aggregate with the rodded unit weight of fine aggregate
5. Include the appropriate amount of mineral filler for proper mastic properties and mixture volumetrics

The procedures outlined here provide the direct ability to proportion aggregates to achieve varying degrees of aggregate interlock using measurable properties of the individual aggregates used in the mixture.

4.3 Ratios for Evaluation of Gradation

The combined gradation of an asphalt mixture can be analyzed using the concepts of particle packing. The "traditional examination" of packing density is not used in this design method. An alternate examination of aggregate packing is more appropriate for the design and quality control of asphalt mixtures. This use of particle packing involves applying the appropriate particle size ratio (0.22) to the gradation to illustrate the void relationships that result from the filling of voids with different size particles. Studying the filling of voids with the appropriate amount and size of filler material will illustrate the resulting void structure in the combined mixture. The understanding of aggregate blending will lead to improvements to the design of mixtures.

When attempting to break down a continuous aggregate gradation for analysis the mixture must be examined from the top down. The largest size material that is in an asphalt mixture is the nominal maximum particle size. The nominal maximum particle size (NMPS) is defined as the first sieve larger than the first sieve that retains more than 10 percent, and can be assumed to be the largest size of included particles. The PCS for the NMPS will then be the break between what is considered coarse and fine aggregate in the total aggregate structure. This sieve will be termed the Mixture Primary Control Sieve (MPCS). Further breakdown of the fine portion of the combined gradation is accomplished by using the PCS for the MPCS, thereby giving a secondary control sieve (SCS) for the total aggregate structure. Voids are created in the fine aggregate making it necessary to further break down the gradation for analysis. The PCS for the SCS will provide information on the fine portion of the fine aggregate. This sieve will be termed the tertiary control sieve (TCS).

Breaking down a continuous gradation using the MPCS, SCS, and TCS does not allow a complete explanation of the coarse portion of the aggregate structure. Examining the packing of coarse aggregate requires the introduction of the half sieve. The half sieve is defined as the NMPS x 0.5; the particle passing this sieve is termed an "interceptor." Interceptors keep single sized aggregate from achieving their optimum density, hence aggregate interlock. This relationship for the size selection to use in establishing ratios for evaluation of gradation is pictured in Figure 4-10.

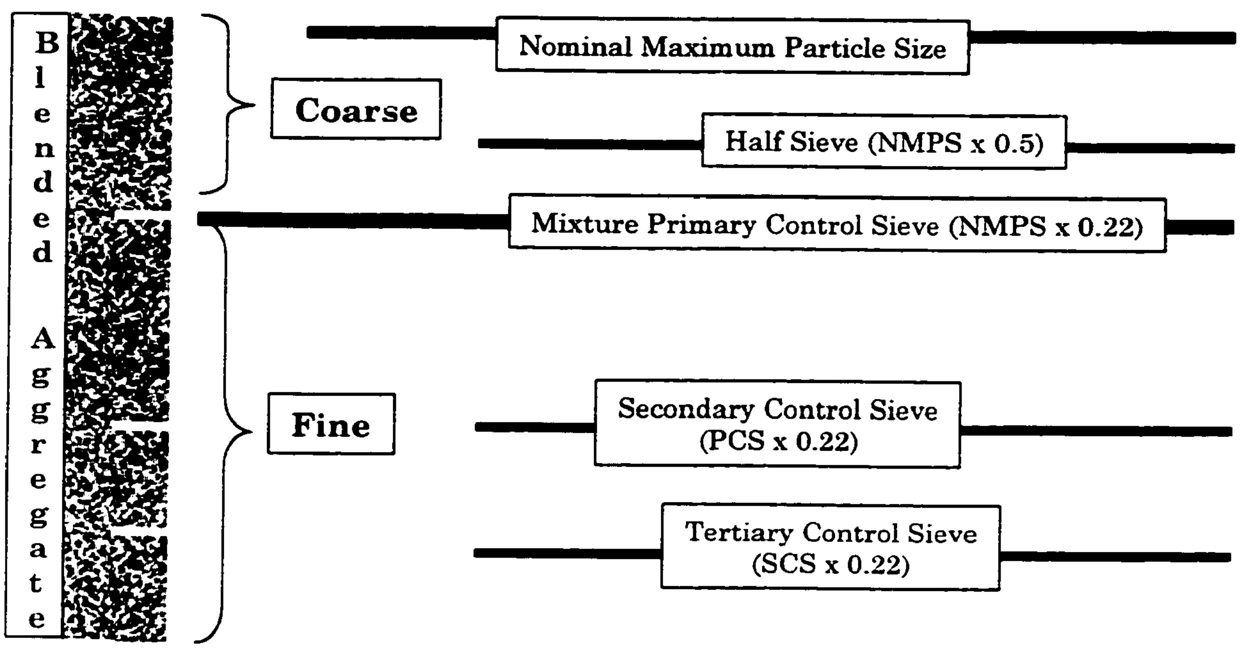


Figure 4-10 Schematic Drawing of Ratios for Evaluation of Gradation

4.3.1 Coarse Aggregate Ratio

The coarse aggregate ratio (CA Ratio) is used to evaluate the packing of the coarse portion of the aggregate gradation. The volume of coarse aggregate is controlled by the design unit weight, which can be selected to provide coarse aggregate interlock. The coarse aggregate ratio is therefore used to analyze the resulting void structure in the coarse aggregate.

The interceptor size aggregates can be used to adjust the mixture's volumetric properties. "Interceptor" aggregates are particles that are smaller than the half sieve (NMPS x 0.5) and larger than the MPCS material. Changing the quantity of interceptors changes the voids in the mixture, primarily by changing the size of the voids. The proper selection of the quantity produces a balanced coarse aggregate structure. With a balanced coarse aggregate structure the mixture should be easy to compact in the field and should adequately resist deformation under load.

Use of the half sieve to characterize an aggregate mixture is not based on traditional analysis of aggregate or particle packing. The use of the half sieve is proposed for use in the characterization of aggregate mixtures. Preliminary experience suggests that a coarse aggregate ratio based on the half sieve is appropriate for characterizing aggregate voids. The experimental activities and results from this study will prove or disprove the appropriateness of CA Ratio and the use of the half sieve.

The equation for the calculation of the CA Ratio is given in Equation 4-1.

Equation 4-1 Coarse Aggregate Ratio

$$\text{CA Ratio} = \frac{(\text{\% Passing Half Sieve} - \text{\% Passing Primary Control Sieve})}{(100\% - \text{\% Passing Half Sieve})}$$

The packing of the coarse aggregate fraction is a primary factor in the constructability of the mixture, which can be directly shown using the CA Ratio. For dense graded mixtures this ratio is desired to be between 0.40 and 0.80 to ensure "balance" in the coarse portion of the aggregate structure. A CA Ratio below 0.40 leads to an aggregate blend that allows the over compaction of the fine aggregate fraction because the size of the voids created in the coarse aggregate are smaller. These smaller voids may exhibit problems with proper mixing and particle distribution which can cause cavities or bridged voids in the aggregate structure. Experience with mixtures designed under these concepts have shown that mixtures with a low CA ratio also tend to segregate during construction[43]. A mixture with a low CA Ratio requires a strong fine aggregate structure in order to maintain adequate mixture volumetrics.

As the CA Ratio increases and approaches 1.0 the coarse aggregate fraction becomes "unbalanced" because the increased amount of interceptor size aggregates are attempting to control the coarse aggregate skeleton. Although this blend may not be as prone to segregation, it contains such a large quantity of interceptors that the coarse aggregate fraction packs differently than desired. The voids in the coarse aggregate are larger than necessary resulting in decreased compaction of the fine aggregate. The result can be a mixture that is difficult to compact in the field, as it tends to move under the rolling compaction.

It is possible for the CA Ratio to increase considerably above 1.0, causing problems in design and construction. With this high CA Ratio the fine portion of the coarse aggregate actually dominates the formation of the aggregate skeleton. At this point the fine portion of the coarse aggregate creates the aggregate structure and the larger aggregates in the coarse aggregate are considered "pluggers." Pluggers do not make up part of the aggregate structure, rather they float in a matrix of finer particles.

4.3.2 Coarse Portion of Fine Aggregate (FA_c)

The fine aggregate portion of any blend is defined as that portion passing the MPCS. The fine aggregate portion can also be viewed as a blend of aggregates which contains a fine and coarse portion, given our definition of coarse and fine. The coarse portion of the fine aggregate creates voids that will be filled with the fine portion of the fine aggregate. As with the coarse aggregate, it is desired to fill the voids with the appropriate volume of the fine portion of the fine aggregate without overfilling the voids.

Equation 4-2 gives the equation that describes the fine aggregate coarse fraction ratio (FA_c). The FAc ratio is used to characterize the packing characteristics of the coarse portion of the fine aggregate. The ratio does not give the volume of coarse aggregate in the mix, rather it examines the gradation of the fine aggregate and its packing. A very low value for this ratio, less than 0.4 is characteristic of a gap gradation in the coarse portion in the fine aggregate. It is generally desirable to have this ratio less than 0.50, as greater values will mean that the gradation of the fine aggregate is unbalanced in the combination of particles. As this ratio increases it becomes possible for the smaller fine aggregate particles to push apart the larger fine aggregate particles. As mixtures approach the 0.55 value in the FA_c ratio they may be considered tender mixes, which have been shown to over-densify and give early failure under traffic. This FAc ratio can be used to identify tender mixtures and mixtures with high amounts of natural sand. Specifications in the FAc could eliminate the need for a restricted zone or natural sand volume requirements.

Equation 4-2 Fine Aggregate Coarse Portion Ratio

$$FAc = \frac{\%\ \text{Passing Secondary Control Sieve}}{\%\ \text{Passing Primary Control Sieve}}$$

This ratio can become too low and create a gradation that is not uniform. These mixtures would then to be gap graded in the fine portion of the blend. This non-uniformity can cause instability and may lead to compaction problems due to the difficulty in compacting this portion of the aggregate structure. This ratio has a considerable impact on the VMA of a mixture due to the blending of sands and the creation of voids in the fine portion of the aggregate. The voids in the mixture will increase with a decrease in this ratio.

4.3.3 Fine Portion of Fine Aggregate (FA_f)

The fine portion of the fine aggregate is used to fill the voids created by the coarse portion of the fine aggregate. This ratio behaves similarly to the FA_c ratio and changes in this ratio produce changes similar to those in the FA_c ratio. This ratio shows how the fine portion of the fine aggregate packs together. The equation that describes the fine aggregate fine fraction ratio (FA_f) is given in Equation 4-3.

Equation 4-3 Fine Aggregate Fine Portion Ratio

$$FA_f = \frac{\%\ \text{Passing Tertiary Control Sieve}}{\%\ \text{Passing Secondary Control Sieve}}$$

This ratio is used to further evaluate the blend in regards to the packing characteristics of the smallest portion of the aggregate blend. Like the FA_c ratio the value of the FA_f ratio should be less than 0.50 for typical dense graded mixtures because the voids in the mixture will increase with a decrease in this ratio.

A complete listing of the sieve sizes and calculations for the CA Ratio, FA_c ratio, and FA_f ratio are given in Table 4-3. This table is given in an effort to clarify the previously presented information on ratios for the evaluation of aggregate gradation.

Table 4-3 Summary of Ratios for Evaluation of Aggregate Gradation

Nominal Maximum Particle Size	1 1/2 - in.	1 - in.	3/4 - in.	1/2 - in.	3/8 - in.	#4
Primary Control sieve	3/8 - in.	#4	#4	#8	#8	#16
½ sieve of the CA	3/4 - in.	1/2 - in.	3/8 - in.	#4	#4	#8
CA Ratio	$\frac{3/4" - 3/8"}{100\% - 3/4"}$	$\frac{1/2" - \#4}{100\% - 1/2"}$	$\frac{3/8" - \#4}{100\% - 3/8"}$	$\frac{\#4 - \#8}{100\% - \#4}$	$\frac{\#4 - \#8}{100\% - \#4}$	$\frac{\#8 - \#16}{100\% - \#8}$
Secondary Control Sieve	#8	#16	#16	#30	#30	#50
CA Ratio of the FA (FA_C)	$\frac{\#8}{3/8"}$	$\frac{\#16}{\#4}$	$\frac{\#16}{\#4}$	$\frac{\#30}{\#8}$	$\frac{\#30}{\#8}$	$\frac{\#50}{\#16}$
Tertiary Control Sieve	#30	#50	#50	#100	#100	#200
FA Ratio of the FA (FA_F)	$\frac{\#30}{\#8}$	$\frac{\#50}{\#16}$	$\frac{\#50}{\#16}$	$\frac{\#100}{\#30}$	$\frac{\#100}{\#30}$	$\frac{\#200}{\#50}$

4.4 Summary of Design Concepts

The concepts given in this chapter serve as an outline for a design procedure that will ensure coarse aggregate interlock through measurable properties of the individual component aggregates.. The establishment of this coarse aggregate interlock is expected to provide the deformation resisting portion of the aggregate structure and guide the development of proper volumetrics in a final mix design.

The evaluation of aggregate gradation using the aggregate ratios will help provide insight into packing of the aggregate structure. These ratios include:

- Coarse Aggregate Ratio – CA Ratio
- Fine Aggregate Coarse Portion Ratio – FAc
- Fine Aggregate Fine Portion Ratio - FAf

The control of aggregate packing should give a designer the ability to specify the mixture properties and eliminate the trial and error process normally used in the determination of aggregate gradation.

These procedures and evaluation tools can also be used in the quality control process during construction to direct any changes that may be necessary during production to meet the quality requirements of density and air voids. The mixture quality control personnel will have a method to adjust mixtures to improve quality because of the understanding of the effects of aggregate gradation and aggregate packing in the asphalt material.

The tools provided through these concepts will also provide a valuable tool in the forensic evaluation of asphalt materials. It becomes possible, with knowledge of the actual aggregate components, to determine the degree of aggregate interlock and balance of the asphalt mixture; leading to a more fundamental understanding of mixture performance.

CHAPTER 5 EXPERIMENTAL TESTING PLAN

The new procedures to design and evaluate aggregates provide the basis for an improved design method for HMA. Blending aggregates by volume provides the ability to design a level of coarse aggregate interlock of the mixture. The ratios for evaluation of gradation provide the tools to understand the combined blend and the resulting void structure in the mixture. A comprehensive testing plan to validate these concepts is necessary.

A testing plan that examines aggregates and their properties as well as the combination of aggregates and the resulting effect on mixtures was conducted. Aggregate testing was performed in order to understand the change in the aggregate packing properties with changes in aggregate shape, surface texture and gradation. Mixture testing was performed to evaluate combinations of two aggregates in order to understand the interaction of the aggregates as it relates to aggregate interlock and change in aggregate gradation with respect to mixture volumetrics and mechanical properties.

5.1 Aggregate Testing

Fine and coarse aggregates were tested to illustrate the effect of combined gradation, shape, surface texture, and particle size of aggregate materials. All aggregate testing was performed on aggregates with precisely controlled gradations that meet the IDOT specification for fine and coarse aggregate. The gradations were varied from the finest to coarsest allowed by the appropriate specification.

Aggregate testing was performed using the Uncompacted Voids in Coarse Aggregate Test Apparatus (UVCATA) as described in Chapter 6 Testing Methods.

5.1.1 Coarse Aggregates

Typical coarse aggregates including dolomite, gravel, and slag were sampled and tested to show the aggregate structure and particle packing of typical coarse aggregates as it relates to aggregate properties. These aggregates were selected to provide information relative to the aggregate particle packing and void property changes with changes in the gradation, surface texture, and maximum particle size of the coarse aggregate.

Samples of IDOT aggregate CM-16 (3/8-in. stone) were collected from certified sources of dolomite, limestone, and gravel. A sample of CM-13 (1/2-in. stone) was collected from a certified source of steel slag. A sample of CM-11 (3/4-in. stone) was collected from a certified source of dolomite; this source corresponds with one of the sources of CM-16. Aggregates were divided into individual standard sieve sizes and carefully recombined to match target gradations. Target gradations were chosen as the coarse limit of the specification, medium value of the specification, and the fine limit of the specification and are given for the CM-11, CM-13, and CM-16 in Table 5-1, Table 5-2, and Table 5-3, respectively.[44] These aggregate gradations were then evaluated in the UVCATA. The modified tests as described in Chapter 6 Testing Methods were performed to include the uncompacted voids, voids with 10 rods compaction, and voids with 25 rods compaction (similar to unit weight). The reported values for these tests are the average result of 10 repeated tests. The results from this testing are given in Chapter 7.

Table 5-1 Aggregate Test Gradations for CM-11 Coarse Aggregate

Sieve	Sieve	Percent Passing			
mm	U.S.	Coarse	Medium	Fine	Specification
25.0	1-in.	100	100	100	100
19.0	3/4-in.	84	92	100	84 - 100
12.5	1/2-in.	30	45	60	30 - 60
4.75	#4	0	6	12	0 - 12
1.18	#16	0	3	6	0 - 6

Table 5-2 Aggregate Test Gradations for CM-13 Coarse Aggregate

Sieve	Sieve	Percent Passing			
mm	U.S.	Coarse	Medium	Fine	Specification
19.0	3/4-in.	100	100	100	100
12.5	1/2-in.	94	97	100	94 - 100
9.5	3/8-in.	70	80	90	70 - 90
4.75	#4	15	30	45	15 - 45
1.18	#16	0	3	6	0 - 6

Table 5-3 Aggregate Test Gradations for CM-16 Coarse Aggregate

Sieve	Sieve	Percent Passing			
mm	U.S.	Coarse	Medium	Fine	Specificaiton
12.5	1/2-in.	100	100	100	100
9.5	3/8-in.	94	97	100	94 - 100
4.75	#4	15	30	45	15 - 45
1.18	#16	0	2	4	0 - 4

5.1.2 Fine Aggregates

Fine aggregate natural sand was evaluated to determine the void structure that remains when blending is conducted. Evaluation of the amount of fine aggregate necessary to fill the voids created by the coarse aggregate is possible with this information. The gradation of the fine aggregate was varied to illustrate the effect of changing gradation on the voids in the fine aggregate.

Samples of IDOT aggregate FA-01, natural sand, were collected from an IDOT certified source of fine aggregate. This fine aggregate was sieved into standard sized materials and recombined to precise gradations. These gradations were the coarse limit of the specification, medium value of the specification, and the fine limit of the specification as given for FA-01 in Table 5-4.[44] These aggregate gradations were then evaluated in the UVCATA. The modified tests as described in the Testing Methods section for this proposal were performed. This testing included determining the uncompacted voids, voids with 10 rods of compaction, and voids with 25 rods of compaction. The reported values for these tests are the average result of 10 repeated tests. The results from this testing are given in Chapter 7.

Because the design concepts presented are focused on the development of coarse aggregate interlock and the fact that testing methods exist for the characterization of voids in sand size fine aggregate, the study of aggregate voids in the fine aggregate with multiple sources was not performed. Only one fine aggregate source was used in this study.

Table 5-4 Aggregate Test Gradations for FA-01 Fine Aggregate

Sieve	Sieve	Percent Passing			
mm	U.S.	Coarse	Medium	Fine	Specification
9.5	3/8-in.	100	100	100	100
4.75	#4	100	97	94	94 - 100
2.36	#8	84	82.5	81	
1.18	#16	68	65	62	45 - 85
0.600	#30	60	45	30	
0.300	#50	29	16	3	3 - 29
0.150	#100	2	2	2	0 - 10

5.2 Mixture Testing

With selection and testing of the individual aggregate components complete, the aggregates were combined in precise percentages to produce asphalt mixtures which should exhibit controlled levels of coarse aggregate interlock. These combinations were tested to relate the compaction characteristics, mixture volumetrics, and mechanical properties with changes in the coarse and fine aggregate gradations and the corresponding ratios. Mechanical property testing of these mixtures was performed to generate information on the performance properties of the mixtures.

Typical coarse and fine aggregates, selected from the previously tested materials, were selected from stockpiles of IDOT accepted aggregates. The selected aggregates are a

CA-16, dolomite crushed 3/8-in stone, and a FA-01, natural sand, and limestone mineral filler. The aggregate information and properties for these individual components are given in Table 5-5 and Table 5-6. These aggregates were selected because of their typical use for Illinois highway pavements in Type 2 primary route mixtures. The selected CA-16 contains a small percentage of flat and elongated aggregates and contains 100% crushed faces. The selected fine aggregate is typical natural sand that is used for the construction of asphalt pavements in Illinois. The selected aggregates will provide the starting point for future studies, which may examine the combination of aggregates with differing shape and texture.

Table 5-5 Aggregate Information for Aggregates Used In Mixture Testing

	Aggregate Information		
Name	JSG Chips	Nat Sand	Min. Filler
Size	CA-16	FA-01	MF-01
Producer	Joliet S&G	Urban Cravel Co.	Fine Grind Lime
Type	Dolomite	Natural Sand	Limestone
Location	ATREL Stock	ATREL Stock	ATREL Stock
Bulk Specific Gravity	2.692	2.572	2.755
Apparent Specific Gravity	2.79	2.7	2.755
Absorbtion Capacity	1.5	1.8	

The coarse aggregate was separated and recombined to a coarse, medium and fine gradation that remains within CA-16 specification. The fine aggregate was similarly assembled to a coarse, medium, and fine gradation that remains within the FA-02 specification. For each combination of coarse and fine aggregate listed in Table 5-6, the percentage of coarse aggregate and fine aggregate was varied to produce different levels of aggregate interlock and aggregate ratios, which should produce changes in the resulting volumetric properties and mechanical properties. The gradations were developed using the principles outlined in Chapter 4. These gradations were developed by selecting the design unit weight to be at the following five levels:

- loose unit weight - 10%
- loose unit weight - 5%
- loose unit weight
- loose unit weight + 5%
- loose unit weight + 10%

The overall test matrix is given in Table 5-7.

Table 5-6 Aggregate Gradations for Aggregates Used In Mixture Testing

Sieve		CA - 16			FA - 02			Minneral
mm	U.S.	Coarse	Medium	Fine	Coarse	Medium	Fine	Filler
12.5	1/2"	100	100	100	100	100	100	100
9.5	3/8"	94	97	100	100	100	100	100
4.75	#4	15	30	45	94	97	100	100
2.38	#8	7	16	25	81	82.5	84	100
1.18	#16	0	2	4	62	65	68	100
0.600	#30	0	1	1	30	45	60	100
0.300	#50	0	0	0	3	16	29	100
0.150	#100	0	0	0	1.9	1.9	1.9	99
0.075	#200	0	0	0	0.2	0.2	0.2	88

Samples were combined using precise gradation control. Aggregates were mechanically sieved to individual sieve sizes from ½" to #50 sieve for each aggregate. The aggregates were then hand sieved before preparation of the mixture to verify correct sizing and increased control of the aggregate gradation. Dust correction was performed in accordance with the IDOT procedure for dust correction. The percentage of passing #200 material remained constant for all mixtures. All asphalt samples were mixed with 5.5 %, by weight of total mix, of PG 64-22 asphalt cement from an IDOT approved supplier of asphalt cement. After mixing, all samples were aged according to the Superpave short-term aging procedure and prepared for compaction.

Asphalt samples were compacted in the Superpave Gyratory Compactor to 75 gyrations and the volumetric properties were measured. The samples were prepared to achieve a height of 150-mm +/- 2-mm, for subsequent structural testing. Volumetric properties were measured on all samples to determine the change in volumetrics produced by a change in aggregate gradation and aggregate interlock. The results from this testing are given in Chapter 7 Aggregate Test Results and Discussion.

Table 5-7 Test Matrix for Experimental Design

Block	Coarse Aggregate Gradation	Fine Aggregate Gradation	Design Unit Weight
Block 1	Med	Med	LW - 10%
Block 1	Med	Med	LW - 5%
Block 1	Med	Med	LW
Block 1	Med	Med	LW + 5%
Block 1	Med	Med	LW + 10%
Block 2	Coarse	Med	LW - 10%
Block 2	Coarse	Med	LW - 5%
Block 2	Coarse	Med	LW
Block 2	Coarse	Med	LW + 5%
Block 2	Coarse	Med	LW + 10%
Block 3	Fine	Med	LW - 10%
Block 3	Fine	Med	LW - 5%
Block 3	Fine	Med	LW
Block 3	Fine	Med	LW + 5%
Block 3	Fine	Med	LW + 10%
Block 4	Med	Coarse	LW - 10%
Block 4	Med	Coarse	LW - 5%
Block 4	Med	Coarse	LW
Block 4	Med	Coarse	LW + 5%
Block 4	Med	Coarse	LW + 10%
Block 5	Med	Fine	LW - 10%
Block 5	Med	Fine	LW - 5%
Block 5	Med	Fine	LW
Block 5	Med	Fine	LW + 5%
Block 5	Med	Fine	LW + 10%
6	Med	Med	LW - 40%

5.2.1 Tests Conducted on HMA

Sample preparation and testing was performed on mixtures in the above described fractional factorial experiment according to the following schedule:

- Two - 2,000 gram samples from each mixture were mixed and short term aged for maximum specific gravity testing (G_{mm})
- Five Gyratory samples were prepared and short term aged for each mixture

- The five gyratory samples were compacted to 75 gyrations in the IPC Servopac SGC for volumetric analysis
- Because data was desired to further understand the development of shear stress and capture the locking point in the SGC, in Blocks 1, 2, 4, and 5 one sample was compacted to 125 gyrations

The mechanical property testing was performed on four of the gyratory compacted samples according to the following schedule:

- Two of the gyratory compacted samples were tested at 50 °C in the IPC rapid triaxial test apparatus (RaTT) in a QC/QA frequency sweep test. This test will provide the measurement of compressive and extension modulus as well as deformation indexes that have been linked to permanent deformation performance
- One of the gyratory compacted samples was cut into two 150-mm x 50-mm samples for testing in the Superpave Shear Tester. The frequency-sweep-constant-height test protocol was used to determine the complex shear modulus (G*) for the mixture at 50 °C. After completion of the frequency sweep test, the repeated shear constant height test was conducted for evaluation of resistance to permanent deformation
- One of the gyratory compacted samples was cut into two 150-mm x 50-mm samples for indirect tension resilient modulus testing at 25 °C

5.3 Summary of Experimental Testing Plan

The experimental testing plan provides an investigation of aggregate properties and aggregate combinations that provide a backbone for a comprehensive mixture design and control system. The testing of dry aggregates, both coarse and fine, should characterize packing of the individual components for use in the aggregate blend. These aggregates are then combined to produce asphalt mixtures. The void characteristics of these mixtures are the primary result of this experimental testing plan.

A comprehensive testing matrix was developed to study the effect of change in gradation of the coarse and fine aggregate portions of an aggregate blend while changing the volume of coarse and fine aggregates. Figure 5-1 shows this test matrix and test variables in the experimental plan.

	Coarse	Medium	Fine
Coarse		LUW - 10% LUW - 5% LUW LUW - 5% LUW + 10%	
Medium	LUW - 10% LUW - 5% LUW LUW - 5% LUW + 10%	LUW - 40% LUW - 10% LUW - 5% LUW LUW - 5% LUW + 10%	LUW - 10% LUW - 5% LUW LUW - 5% LUW + 10%
Fine		LUW - 10% LUW - 5% LUW LUW - 5% LUW + 10%	

Figure 5-1 Experimental Testing Matrix

Through this test matrix 26 mixtures were developed. For each of the 26 mixtures 5 samples were produced in the SGC for volumetric and mechanical property tests.

The demonstration of the change in mixture volumetrics and mechanical properties that is developed in this experiment provides unique data connecting aggregate gradation to the state of the art in mixture deign, quality control, and performance. This testing scheme offers a systematic method for evaluation of aggregate interlock that when analyzed should produce an improvement in the understanding of blended aggregate gradations.

CHAPTER 6 TESTING METHODS AND DATA INTERPRETATION

The testing methods used in this experiment to understand the void characteristics of aggregates in an asphalt mixture include mixture testing on several types of accepted testing apparatus including a triaxial test apparatus and the Superpave Shear Tester. The test equipment included in the evaluation of these mixes includes:

- Uncompacted Void Content of Aggregate Test Apparatus
- IPC Servopac Gyratory Compactor
- IPC Rapid Triaxial Tester (RaTT)
- IPC Universal Testing Machine
- Superpave Shear Tester (SST)

6.1 AGGREGATE UNIT WEIGHTS

Loose and rodded weights will be determined for all of the coarse and fine aggregates. This information will be used to demonstrate the effect of gradation, compaction, surface friction, and particle shape on the development of voids in the aggregates.

This test was performed in the Uncompacted Void Content of Aggregate Test Apparatus (UVCATA). This apparatus, shown in Figure 6-1, was constructed to the specifications as described in NCHRP Report 405[41], Appendix D. The test method used is as described in that document, following method C.

Figure 6-1 Uncompacted Voids in Coarse Aggregate Test Apparatus

Method C of the proposed test procedure requires use of the stockpile gradation. For the aggregates used in this experiment the test was performed on controlled gradations as outlined in Table 5-1, Table 5-2, Table 5-3 for the coarse aggregates and Table 5-4 for the fine aggregate. These aggregates were mechanically sieved into component sieve sizes and recombined into achieve the target gradation.

In addition to the standard testing, all of the aggregates for this study were tested under two levels of compactive effort. The standard container used in the UVCATA served as the standard container for the compaction of the aggregates and determination of unit weight. Each aggregate was tested with 10 rods and 25 rods of compaction.

For each of the rodded tests the aggregates were dropped into the bucket in a similar manor. The aggregates, which are of the specified stockpile gradation, were dropped into the container in three lifts with each lift being rodded the appropriate amount. The rodding proceeded as outlined in AASHTO T 19, Unit Weight and Voids in Aggregate.

The reported results from this testing include:

- Loose Unit Weight and uncompacted voids of aggregate

- Rodded Unit Weight and rodded voids of aggregate (10 rods)
- Rodded Unit Weight and rodded voids of aggregate (25 rods)

6.2 Preparation of Asphalt Mixtures

All mixture samples were carefully prepared to ensure precise gradation control. The individual aggregate samples were dried and then sieved into their component sizes by mechanical sieve machine. The aggregates were then hand sieved for 1 minute to ensure proper size characterization of the aggregate materials. These materials were combined to precisely controlled gradations. A dust correction procedure as outlined in the IDOT Manual of Test Procedures[45] was performed. The use of this dust correction ensured that the dust that remains on the coarse aggregate particle is included in the overall calculation of minus #200 (0.075-mm) material. After determination of the dust correction factor the materials were batched based on the adjusted blending percentages. Table 6-1 illustrates an example of the blending recipe sheet.

Table 6-1 Example Blending Recipe Sheet for Batching of Asphalt Mixtures

Aggregate Blending Amounts				
Sieve Size (mm)	(in)	CA-16	FA-02	M.F.
12.5	1/2"	0.0	0.0	0.0
9.5	3/8"	123.4	0.0	0.0
4.75	#4	2755.7	52.6	0.0
2.38	#8	575.8	254.3	0.0
1.18	#16	575.8	306.9	0.0
0.600	#30	41.1	350.7	0.0
0.300	#50	41.1	508.6	0.0
< 0.300	< #50	0.0	280.6	228.6
	Total	4113.0	1753.7	228.6

Binder %	5.50	Agg. Wt.	6100 g
Binder Weight	355.0 g	Dust Corr.	1.75

6.2.1 Steps In Preparing Specimens

1- For each specimen to be prepared, the batch weights for different aggregate fractions were weighed into a pan

2- The aggregate was heated in an oven at a temperature of approximately 10° C higher than the established mixing temperature. Mixing bowls, spatulas and the other tools used were also heated in the oven

3- The asphalt binder was heated to the proper mixing temperature

4- The heated mixing bowl was placed on the scale and scale was zeroed. Heated aggregate were charged into the bowl and dry mixed for several seconds. Hot asphalt binder was added to the aggregate to achieve the desired batch weight.

5- The mixing was done with a "J" hook type mixer, shown in Figure 6-2 for up to 1-minute for proper mixing and coating of the aggregate

6- The mix was then placed in a shallow pan and aged in a forced draft oven at compaction temperature for 2 hours

7- Approximately 1-hour before compaction of the first specimen, the compaction molds and base plates were placed in an oven at the compaction temperature

8- The mold and base plate was removed from the oven and a paper disk was placed at the top of the base plate. The mixture, at the proper compaction temperature, was then placed in the mold and another paper disk was placed on the top of the material

9- The mold containing the specimen was loaded into the compactor and was placed in the compactor. The gyratory compactor was set for a compaction pressure of 600 kPa, gyration angle of 1.25, and 30 rpm. The specimens were compacted to 75 gyrations

10- After compaction was completed the specimen were removed from the mold, labeled, and stored at room temperature

Figure 6-2 Bowl and "J" hook For Mixing Asphalt Samples

6.3 Gyratory Compaction

The Superpave Gyratory Compactor (SGC) is currently accepted as the most appropriate device to assess the compaction characteristics of asphalt mixtures. This acceptance is derived from the assumption that gyratory compacted samples more closely produce material characteristics that match those found in the compacted pavement.[46, 47] The gyratory compactor also provides the ability to investigate the material properties at void levels representing construction and throughout the life of the pavement.

In this investigation the Servopac SGC will be unitized exclusively. The Servopac SGC is a fully functional feedback controlled testing machine, which was designed to meet and exceed the specification for SGC compaction. The compactor is fully automated, servo-controlled and designed to compact asphalt mixes by means of the fixed angle and vertical pressure gyratory compaction technique. The simultaneous action of static compression and the shearing action resulting from the mold being gyrated through an angle about its longitudinal axis achieve the gyratory compaction. The compactor is pictured in Figure 6-3.

The Servopac SGC includes a feature to measure the gyratory shear stress during the compaction process. This measurement of shear stress will give insight into the developing aggregate structure of the mixture through an examination of the energy required to accomplish the compaction.

6.3.1 Shear Stress Measurement During Compaction

The Servopac gyratory compactor is a second-generation gyratory compactor from Australia. The closed loop feedback control electronics allow for precise control of the critical parameters involved in the gyratory compaction process. The compactor is fitted with a pressure transducer in the pressure lines of the three vertical actuators that control the gyration. This pressure, when combined with the other gyratory inputs, allows calculation of the shear resistance of the asphalt material during compaction. The algorithm for determining this shear resistance is similar to that of the Gyratory Test Machine of the Corps of Engineers. In the GTM, as the angle decreased it had the effect of compressing an 'air-roller' at a distance L (lever arm distance) from the vertical axis of rotation to generate an air pressure P. The gyratory shear stress (G_s) in a specimen of area A and height h is calculated through the following relationship:

$$G_s = \frac{2PL}{Ah}$$

The measurement of shear stress in the Servopac uses a similar algorithm where P is the average pressure measured in gyratory actuators and L is the distance to the midpoint of the actuators.

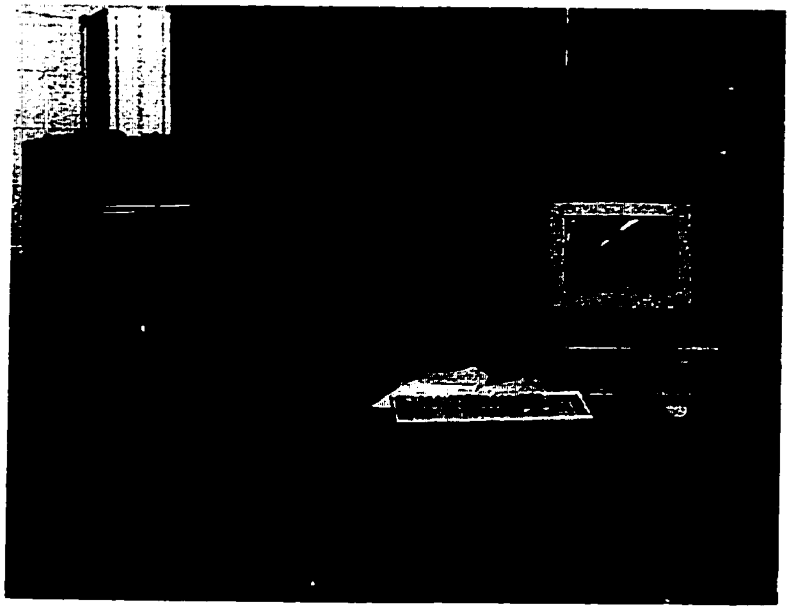


Figure 6-3 Servopac SHRP Gyratory Compactor

The gyratory compaction data will be examined to determine if a relationship exists between coarse aggregate interlock, aggregate ratios, mixture volumetrics, the compaction curve (height vs. gyrations), and the shear resistance characteristics in the gyratory compactor. It is presumed that a relationship exists with these variables that will show the gyratory compactor to be an improved tool in the examination of asphalt mixtures.

During gyratory compaction it is noticed that the shear resistance typically increases during compaction until some point of maximum shear resistance. After that point the shear resistance decreases. This peak in shear stress is felt to be tied to the development of a stable aggregate structure under the boundary conditions imposed by the SGC. Gyratory compaction beyond this point is felt to lead to over-compaction, breaking of aggregates, and a resulting degradation of the mixture. The volumetric and structural properties at this peak are an important representation of how the aggregate particles are orienting to mobilize shear resistance.

6.3.1.1 *The "Locking Point" Concept*

The typical SGC does not collect this shear resistance information. It does however collect the height data with each gyration, which may be used to describe the compaction characteristics of the mixture during compaction in a manner that allows some inference of the development of aggregate structure.

The "locking point" was developed as a visual method to infer the compaction of asphalt mixtures in the SGC. It was proposed as the point in the compaction curve where the aggregate structure begins to develop and resist compaction, and was related to a specific decrease in the compaction rate, viewed as a change in height, common to all mixtures. This point was originally proposed by William J. Pine while working with the Illinois Department of Transportation. The idea of the "locking point" was to tie the maximum density achieved in a growth curve obtained during construction to the compaction of the mixture in the SGC. It was determined that this "locking point" is a most appropriate point in the gyratory compaction for this comparison given proper field lift thickness and construction techniques.

This locking point is defined as the first gyration in which three gyrations are at the same height preceded by two sets of two gyrations at the same height. The locking point is the first of those three consecutive height gyrations. Gyrations beyond this point exhibit a deviation from a uniform densification curve. The following gyratory height data in Table 6-2 shows a typical SGC height printout with the locking point as it has been defined.

It is proposed that the "locking point" of a mixture can be used to prevent over-compacting in the Superpave gyratory compactor As the mixture is compacted the aggregate particles are forced together, and they lock up and develop a structure as compaction proceeds. Figure 6-4 plots the gyratory height vs. number of gyrations and shows this locking of the aggregate structure. All mixtures will lock up, but they will do it at different air voids and different gyration levels. It is felt that an appropriate interpretation of the compaction curve should provide some indication of how the aggregates are locking and developing structure as they densify. This definition of locking point is a subjective interpretation of the compaction curve that could possibly represent a consistent level of aggregate structure in all mixtures. This study will examine the applicability of the locking point and any relationship that may exist with the shear resistance. The objective of defining a true locking point is to provide an indication of where over-compacting begins in the SGC process.

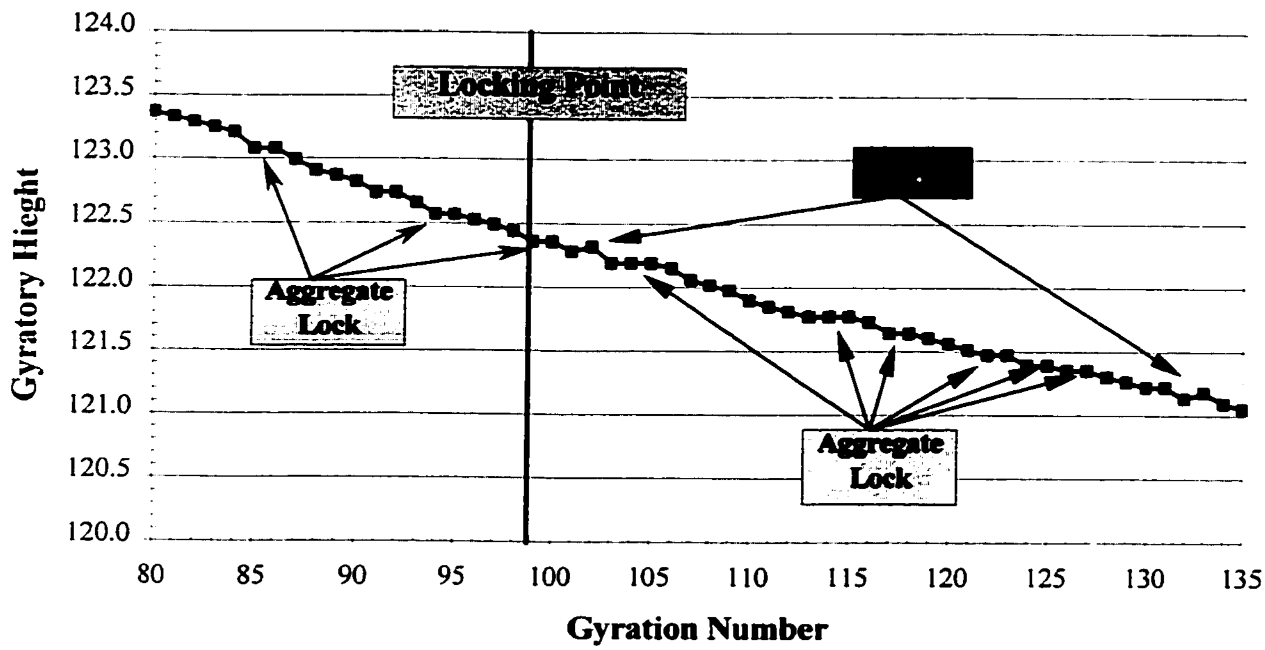


Figure 6-4 Gyratory Height Plot Showing the Locking Point

Table 6-2 Sample of Gyratory Height Data Showing the "Locking Point" Gyration

	1	2	3	4	5	6	7	8	9	10
80	111.9	111.9	111.8	111.8	111.7	111.7	111.6	111.6	111.5	111.5
90	**111.4**	**111.4**	**111.3**	**111.3**	***111.2*** ^{LP}	**111.2**	**111.2**	111.1	111.1	111.0
100	111	110.9	110.9	110.8	110.8	110.8	110.7	110.7	110.7	110.6

6.3.1.2 Slope of the Densification Curve

The slope of the densification curve from the SGC has been offered as a possible compaction characteristic that relates to densification of an asphalt mixture. The densification slope is computed as the slope of the percent of maximum density (%Gmm) versus the log number of gyrations from gyration 10 to the end of compaction. The traditional interpretation of the compaction slope data gives that stronger mixtures give increased compaction slope in the SGC, however documentation of this interpretation is scarce.

The interpretation of compaction slope data is counterintuitive. An increased compaction slope results in an increased densification rate, which is typically associated with a poor mixture. For low strength mixtures the initial densification in the first 10 gyrations is considerable, giving much of the densification in the compaction process. This low strength mixture then has a low compaction slope as measured from gyration 10 to the end of compaction. High strength mixtures do not exhibit as much initial densification and the densification rate, compaction slope, from gyration 10 to the end of compaction is then higher.

6.3.2 Summary of SGC Compaction

The IPC Servopac SGC will be used in this project to prepare all samples for volumetric and mechanical property testing. The data will be collected and analyzed to help demonstrate if the SGC has the ability to differentiate the compaction characteristics of mixtures with different aggregate skeletons. If this ability is realized the designer would be capable of designing a mix for an appropriate air void level when the aggregate structure is stable.

The data reported from this compaction will include the height and shear stress from each gyration. This data will be used to calculate the following compaction properties:

- Locking Point
- Slope of the Densification Curve

6.4 VOLUMETRIC PROPERTIES

The maximum specific gravity (Gmm) for each of the 26 mixtures was determined using AASHTO T 209. Samples for the determination of the maximum specific gravity were prepared as outlined in 6.2 Preparation of Asphalt Mixtures. Figure 6-5 shows the equipment used in the testing for maximum specific gravity. The maximum specific gravity is reported for each mixture in the Chapter 7 Aggregate Test Results and Discussion. This value is use to determine the volumetric properties of the asphalt mixtures.

The bulk specific gravity (Gmb) of each of the compacted samples was measured using AASHTO T 166 after compaction and prior to structural testing. The Gmb test setup is shown in Figure 6-6. The average bulk specific gravity of all similarly compacted samples is reported for each mixture in Chapter 7 Aggregate Test Results and Discussion. This value is use to determine the volumetric properties of the compacted asphalt mixtures.

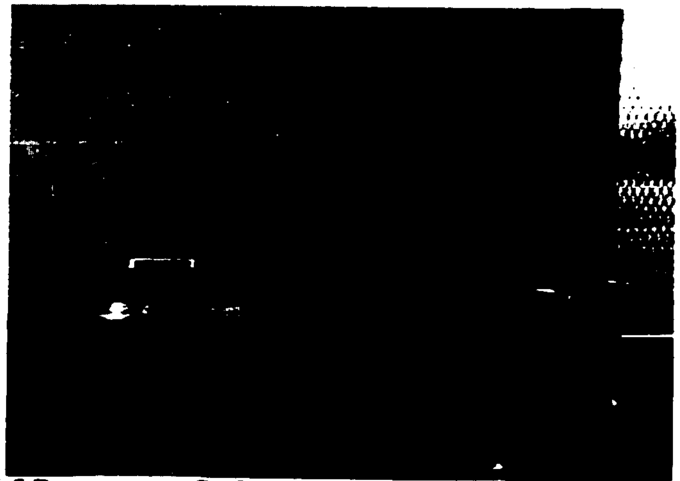


Figure 6-5 Pycnometer, Scale, And Water Bath For Determination Of G_{mm}

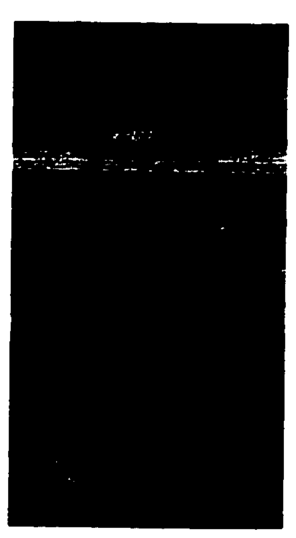


Figure 6-6 Water Bath, Basket, And Scale For Determination Of G_{mb}

6.5 Rapid Triaxial Test (RaTT)

Dynamic triaxial testing to determine material properties provides the ability to characterize the time dependent response, and the stress dependent response of the material. The triaxial test has been historically used to characterize materials for geotechnical, earthquake as well as pavement applications. In this project, the triaxial testing of the asphalt mixtures will be conducted at elevated temperatures for material characterization under multiple stress states and frequencies to emphasize aggregate effects.

The testing will be conducted under a modified procedure as outlined in the report from NCHRP Project 9-7.[48] The IPC RaTT test device, shown in Figure 6-7, will be used to conduct this test. This test apparatus is a closed loop, servo controlled pneumatic test machine with independent control of the vertical and horizontal axis. The control system has two channels of feedback control and is capable of producing a sinusoidal wave shape at 15-Hz in the vertical axis while maintaining a constant confining pressure. The feedback system is capable of dynamically controlling the amplitude of the axial waveform to within 0.5% of the input command value. The system also simultaneously ramps the vertical load and confining pressure to maintain a hydrostatic condition prior to the initiation of the test.

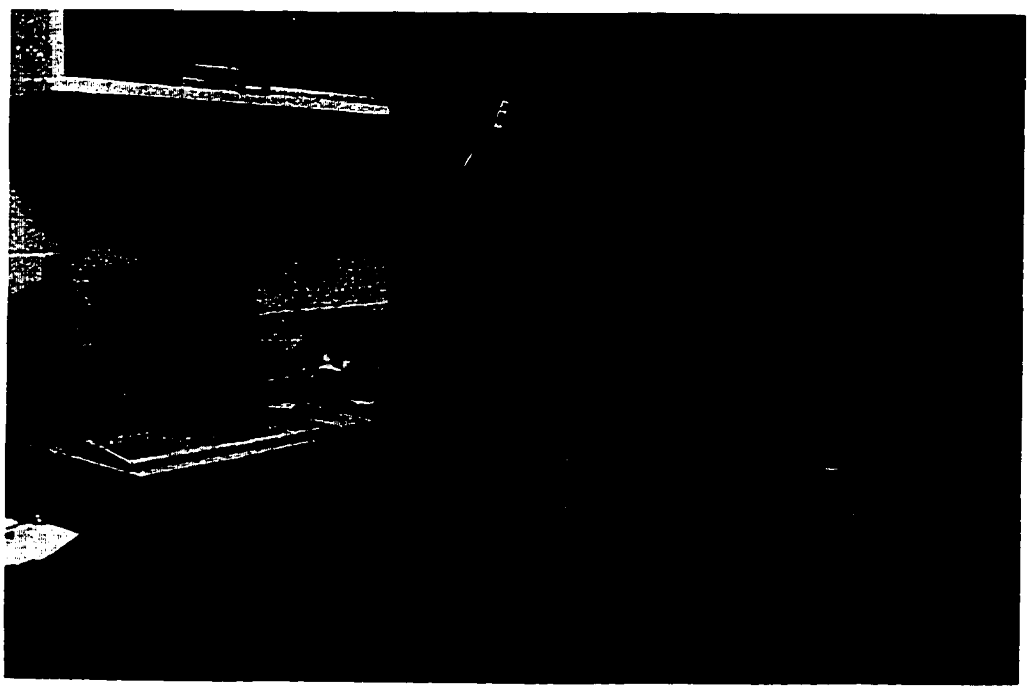


Figure 6-7 Rapid Triaxial Test Device By Industrial Process Controls

The data acquisition system is integral to the operation of the test system. The vertical load is measured with an electronic load cell, which is inline with the vertical applied load. The confining pressure is measured with an electronic pressure transducer.

The deformations, both axial and radial, are monitored using LVDT's. The axial deformations are measured from a fixed point on the test frame to the on-sample load plate. The radial deformations are measures via on-sample through-the-bladder LVDT's. This test setup is shown in Figure 6-8. The sample is maintained at a constant test temperature by enclosure in an environmental chamber. This temperature is monitored using on-sample temperature transducer.

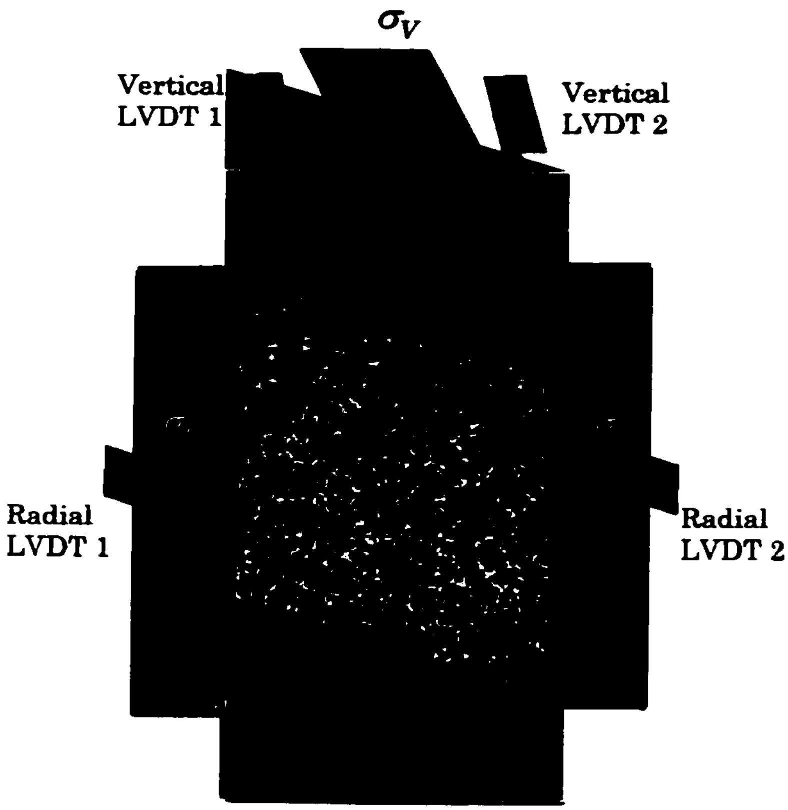


Figure 6-8 Rapid Triaxial Test Sample Configuration

The RaTT test is viewed as an excellent fundamental material property test to determine mechanical properties and characterize the performance of an asphalt mixture. The asphalt sample is tested through a range of stresses that start at a hydrostatic condition, move from a confined compression to a confined extension and terminate at the hydrostatic condition, thereby giving a stress reversal. This cycle is repeated at different frequencies and the deformations in the axial and radial direction are measured. Because the test is conducted in a stress reversal mode it is felt to be representative of the stresses observed under a moving wheel. The strength and deformation data measured during the stress reversal may be more influenced by aggregate structure and load resistance than a single compressive load.

The axial deformations resulting from this test are given in Figure 6-9. The upper and lower regression lines are drawn at the peak axial strains for the applied stress. These lines are indicative of the development of permanent deformation. The upper line is the deformation in compression, while the lower line represents the deformation in extension. The middle regression line is drawn through the strain value where the stress changes from compression to extension. This crossover point is felt to represent a response of the aggregate structure and its rearrangement under the stress reversal. From this plot the slope and intercept is recorded. Also recorded are the modulus and Poisson's ratio in compression and extension for each frequency.

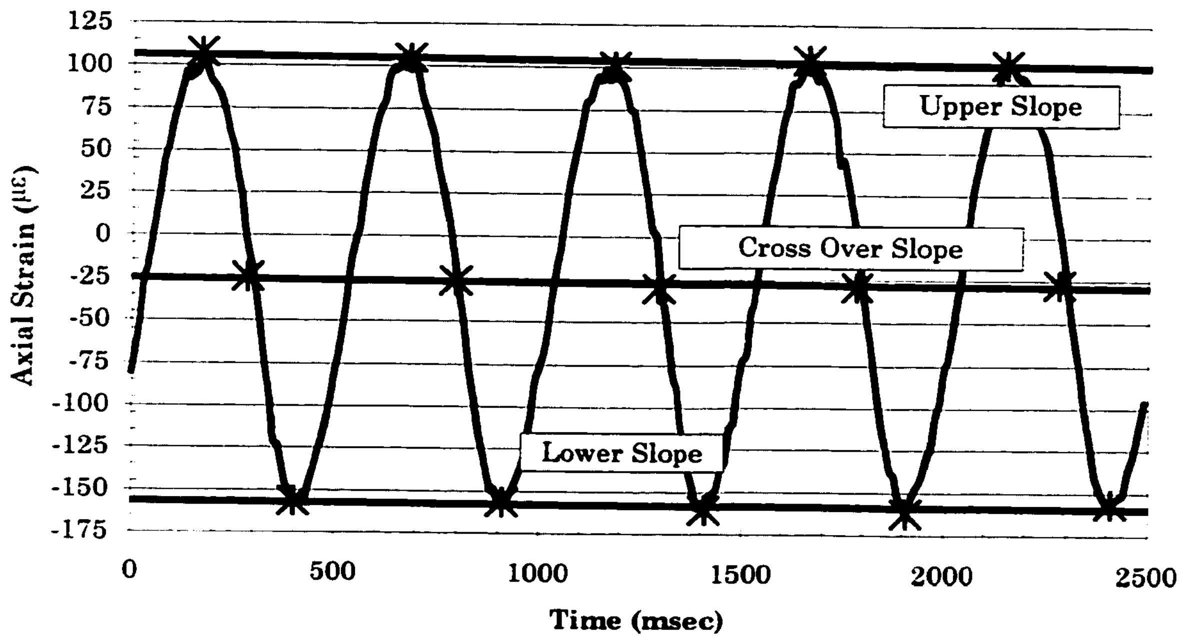


Figure 6-9 Axial Deformations From RaTT Stress Reversal Test

Work by Carpenter[49] has shown that the results from the 2-Hz. testing in the RaTT can be correlated to the results from the Asphalt Pavement Analyzer; previously known as the Georgia Loaded Wheel test. The results of this correlation are considered a rut index. This correlation exists for samples compacted to 7% +/- .5% air voids. The applicability of this rut index is unknown for samples that are not prepared to the specified air voids as different aggregate structures are developed at different air void levels.

The stress state and frequencies given in Table 6-3 were used in this testing in an effort to understand the mechanical properties of the various mixtures. The reported results from this test include the following values for each stress state and frequency when applicable:

- Compression Modulus
- Extension Modulus
- Poisson's ratio in Compression

- Poisson's ratio in Extension
- RaTT rutting index

Table 6-3 Triaxial Stress States and Frequencies for RaTT Testing

Stress State	Starting Hydrostatic State (kPa)	Axial Deviation from Hydrostatiec (kPa)
Extension / Compression	75	+50 / -50
Test Frequencies (Hz)	10, 5, 2, 1, 0.1	

6.6 SUPERPAVE SHEAR TESTER (SST)

The Superpave Shear Tester (SST) was developed under the SHRP Research Program and was designed to evaluate the shear strength of asphalt mixtures. The SST is a closed-loop feedback, servo hydraulic system with a test chamber designed to impart repeated shear loads to 150 mm test specimens, shown in Figure 6-10. The SST is designed to perform a number of test that include:

1. Volumetric test
2. Uniaxial strain test
3. Repeated shear test at constant stress ratio
4. Repeated simple shear test at constant height (RSST-CH)
5. Simple shear at constant height
6. Frequency sweep at constant height

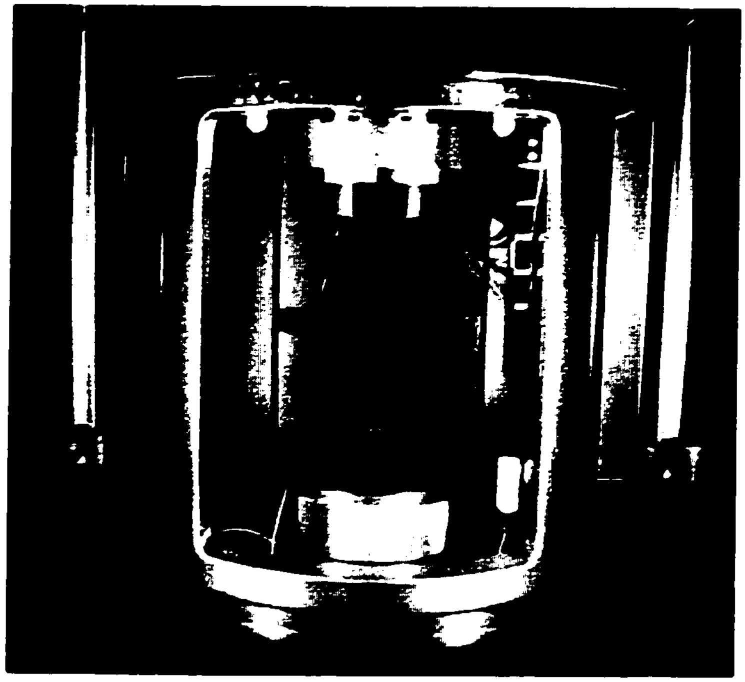


Figure 6-10 Superpave Shear Tester sample chamber

6.6.1 Frequency Sweep Constant Height (FSCH)

The Frequency Sweep Constant Height (FSCH) test (AASHTO TP-7, Procedure E) is used to estimate the mixture stiffness at high temperatures. This test will be conducted in the Superpave Shear Tester (SST). Greater stiffness is considered desirable at high temperature for resistance to permanent deformation.[50, 51] The FSCH test generates the complex shear modulus, G^*, at different frequencies. The sample is subjected to constant strain at very low levels (0.01%) with a sinusoidal shear force. During this test the sample is maintained at constant height. The frequency used in this testing are 10, 5, 2, 1, 0.5, 0.2, 0.1, 0.05, 0.02, and 0.01 Hz. The G^* value is determined from the stress-strain data.

The samples prepared for testing in the FSCH test were not prepared in accordance with the standard test procedure, which calls for a standard air void level. The air voids in this experiment were allowed to vary because a constant compaction level was used. The mechanical property testing was performed on the resulting samples.

The G* value at 10 Hz is often used as a reference point for comparison of different mixtures. This point is felt to accurately rank mixtures for rutting. The reported result from this test is the value of the complex shear modulus (G*) at each test frequency.

6.6.2 Repeated Shear Constant Height (RSCH)

The Repeated Shear Test at Constant Height (RSCH) is performed at high temperature to indicate the rutting susceptibility of a mixture. The test is performed in accordance of AASHTO TP-7, Procedure F. In this test the sample is loaded using a 69-kPa shear stress pulse. Each cycle consists of 0.1 seconds loading time and 0.6 seconds rest period. The pulse is repeated for 5000 cycles and the resulting deformations are recorded.

The response variable of the test is the permanent shear strain after 5000 loading cycles. Lower permanent shear strain in the RSCH test has equated to less rutting susceptibility. The trigger value of 5% shear strain has been developed by SHRP researchers as a value that indicates a mixture is prone to rutting.[52]

The samples prepared for testing in the RSCH test were not prepared in accordance with the standard test procedure, which calls for a standard air void level. The air voids in this experiment were allowed to vary because a constant compaction level was used. The mechanical property testing was performed on the resulting samples.

The reported result from this test is the value of the accumulated permanent strain at 5000 load cycles.

6.7 RESILIENT MODULUS

The resilient modulus of each mixture will be measured according to ASTM D 4123 at 25 °C. The testing will be conducted on the IPC 5P test frame, Figure 6-11. This test apparatus is a servo-pneumatic closed loop feedback control test apparatus. The test device is able to apply a 0.1 second load pulse in a 3 second load cycle with monitoring of the vertical and radial deformations.

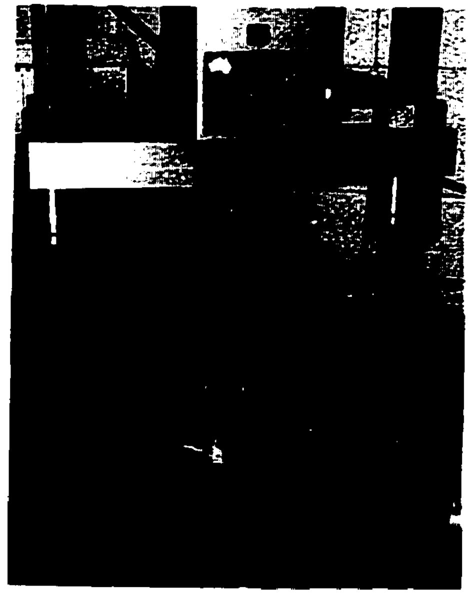


Figure 6-11 Resilient Modulus Test Apparatus by Industrial Process Controls

In this test a diametral loading force will be applied to a 50-mm by 150-mm sample, which is cut from a standard gyratory sample. The resulting total recoverable diametral strain under each load is measured from axes 90-degrees to the applied force and the resilient modulus is calculated. The equation used for calculating resilient modulus is:

$$M_R = (P \ / \ \Delta H \, t) \ (0.27 + \mu)$$

where:

M_R = resilient modulus of elasticity (psi)

P = applied repeated load (lb.)

t = thickness (in)

ΔH = recoverable horizontal deformation (in)

μ = Poisson's ratio

The reported data from this test is the average resilient modulus of at least five tests.

6.8 Summary

This testing program provides data that directly illustrates the degree of aggregate interlock achieved though the proposed blending scheme. This engineering validation provides a direct indication of the volumetric changes that can be expected with a change in aggregate gradation and degree of aggregate interlock. The mechanical testing provides insight into the mechanical properties (modulus, rut resistance, time and temperature sensitivity, etc.) that can be expected with a change in aggregate gradation and degree of aggregate interlock. The RaTT and SST testing is provided as an evaluation of the rutting potential of the asphalt mixtures, while the resilient modulus provides information that can be used for pavement design.

It is expected that the test data will provide a direct correlation back to the volume of coarse aggregate and the aggregate ratios used to develop the gradations. This correlation would indicate the control of volumetrics achievable through the use of these principles. The mechanical property tests may provide an indication of the structural characteristics achievable through use of these aggregate ratios.

CHAPTER 7 AGGREGATE TEST RESULTS AND DISCUSSION

The testing plan given in the Chapter 5 outlines the testing scheme for the evaluation of individual aggregates, both coarse and fine, and asphalt mixtures with changing aggregate interlock and aggregate component gradation. This testing plan was conducted, with the results presented in the following.

7.1 COARSE AGGREGATE TESTING

The coarse aggregate testing results are the loose and rodded unit weights for the coarse aggregates. These test results characterize the aggregates and provide the limits for aggregate interlock in asphalt mixtures.

7.1.1 Coarse Aggregate Test Results

Five aggregate Sources were selected and sampled for testing in the UVCATA under the loose, 10 rods, and 25 rods conditions. This study utilizes many of the typical aggregates found in Illinois. The aggregate sources and aggregate types are given in Table 7-1.

Table 7-1 Aggregate Source and Aggregate Type for Coarse Aggregate Testing

Aggregate Name	Aggregate Source	Aggreagate Type	Characteristic Size
ATREL CM-11	Midwest	Dolomite	3/4-in. (19-mm)
Slag	Levy	Steel Slag	1/2-in. (12.5-mm)
ATREL Chips	Midwest	Dolomite	3/8-in. (9.5-mm)
JSG Chips	Joliet Sand & Gravel	Dolomite	3/8-in. (9.5-mm)
Dolomite Chips	Vulcan - McCook	Dolomite	3/8-in. (9.5-mm)
Gravel Chips	Thelen Sand & Gravel	Crushed Gravel	3/8-in. (9.5-mm)

The results for unit weight and the voids in the coarse aggregate are given for all of the tested aggregates in Table 7-2 and Figure 7-1.

Table 7-2 Unit Weight and Voids in the Aggregate for Coarse Aggregate Testing

	Unit Weight (pcf)			Voids in CA (%)		
	Loose	10 Rod	25 Rod	Loose	10 Rod	25 Rod
ATREL CM-11						
Fine	86.4	95.0	98.0	47.2	42.0	40.2
Medium	86.2	96.1	98.1	47.3	41.3	40.1
Coarse	85.7	94.9	96.7	47.7	42.0	40.9
Slag						
Fine	119.1	128.5	131.7	47.2	43.0	41.5
Medium	118.9	129.0	131.6	47.2	42.7	41.6
Coarse	114.4	126.9	129.6	49.2	43.7	42.5
ATREL Chips						
Fine	83.7	93.9	96.7	48.2	41.8	40.1
Medium	84.1	94.3	96.7	47.9	41.6	40.1
Coarse	83.5	94.3	97.2	48.3	41.6	39.8
JSG Chips						
Fine	88.4	98.3	100.9	46.1	40.1	38.6
Medium	88.7	97.9	100.1	46.0	40.4	39.0
Coarse	87.5	97.9	100.6	46.7	40.4	38.7
Dolomite Chips						
Fine	88.8	97.3	100.1	46.1	40.9	39.3
Medium	88.5	97.7	100.0	46.3	40.7	39.3
Coarse	87.6	97.3	99.4	46.9	41.0	39.7
Gravel Chips						
Fine	82.5	92.2	96.5	51.1	44.0	41.8
Medium	80.0	90.6	95.1	51.1	45.1	42.1
Coarse	80.7	92.4	96.1	50.0	44.2	41.5

Upon Completion of the standard testing, as outlined in Chapter 5, further testing was conducted to evaluate the effect of change in gradation and the resulting voids in the coarse aggregate. Samples were prepared using the JSG chips that evaluated the following:

- A constant percentage of material passing the #4 (4.75-mm) and retained on the #8 (2.36-mm) sieve with increasing amount of material passing the #8 (2.36-mm) sieve [Constant #4 to #8]
- A constant percentage passing the #4 (4.75-mm) sieve with changing amount of material passing the #8 (2.36-mm) sieve [Constant < #4]

- Tests performed on only the material retained on the #4 (4.75-mm) sieve [> #4 Material]

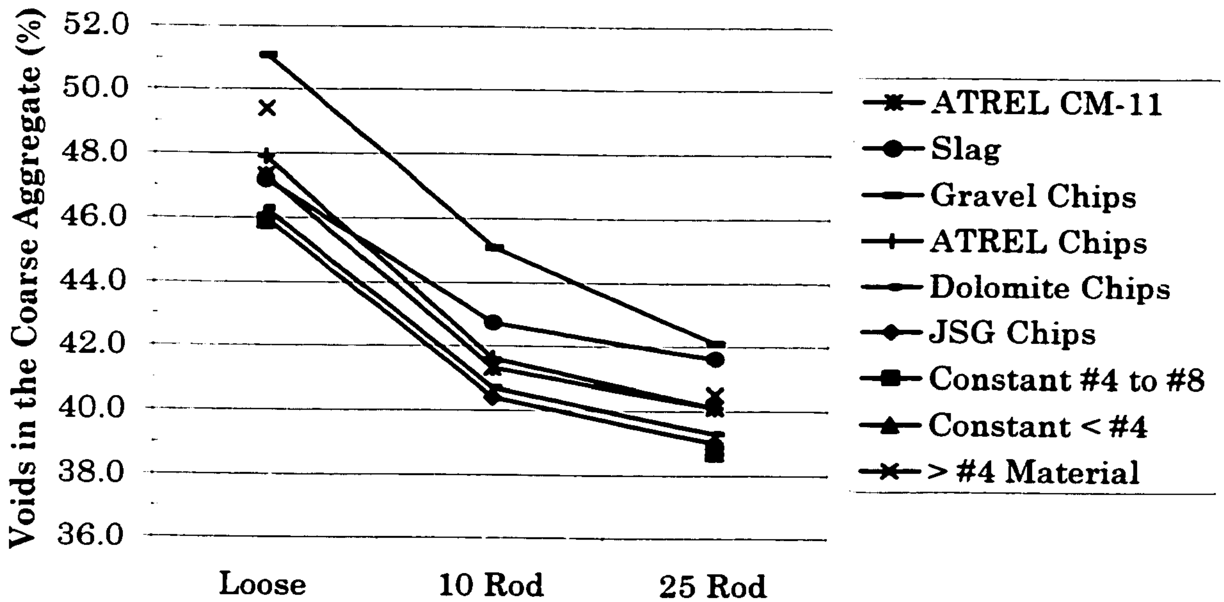


Figure 7-1 Voids in Coarse Aggregate Plot for Coarse Aggregate Testing

The results from this testing and the original test results from the JSG Chips are given in Table 7-3.

Table 7-3 Additional Samples for Unit Weight and Voids in the Aggregate

	Unit Weight (pcf)		Voids in CA (%)	
	Loose	25 Rod	Loose	25 Rod
Original Testing JSG Chips				
Fine	88.4	100.9	46.1	38.6
Medium	88.7	100.1	46.0	39.0
Coarse	87.5	100.6	46.7	38.7
Constant #4 to #8				
Fine	92.8	105.7	44.9	37.2
Medium	91.1	103.2	45.9	38.7
Coarse	90.0	102.6	46.5	39.0
Constant < #4				
Fine	92.3	103.6	45.2	38.4
Medium	91.1	103.2	45.9	38.7
Coarse	90.1	103.2	46.5	38.7
> #4 Material				
Fine	85.4	100.3	49.3	40.4
Medium	85.1	100.2	49.4	40.5
Coarse	85.2	100.5	49.4	40.3

7.1.1.1 Variability of Test Methods

The repeated testing of aggregates can cause degradation of the aggregate material, which results in changing in test results. This aggregate degradation can be especially prominent with increased compactive effort. The analysis of residuals with the tests taken in order will identify the existence of any change test value after repeated tests. Figure 7-2, Figure 7-3, and Figure 7-4 give the residual analysis for the uncompacted voids, 10 rods, and 25 rods testing.

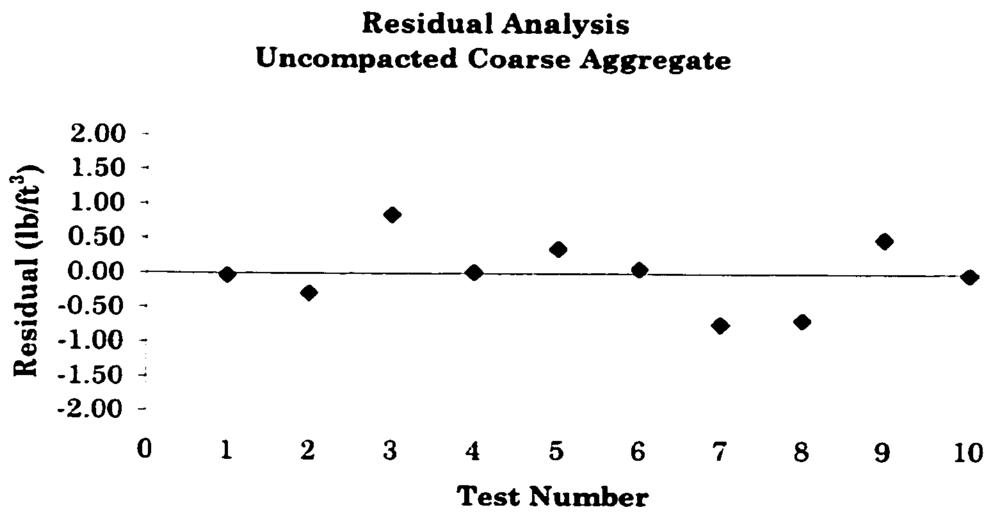


Figure 7-2 Residual Analysis for Uncompacted Unit Weight of Coarse Aggregate

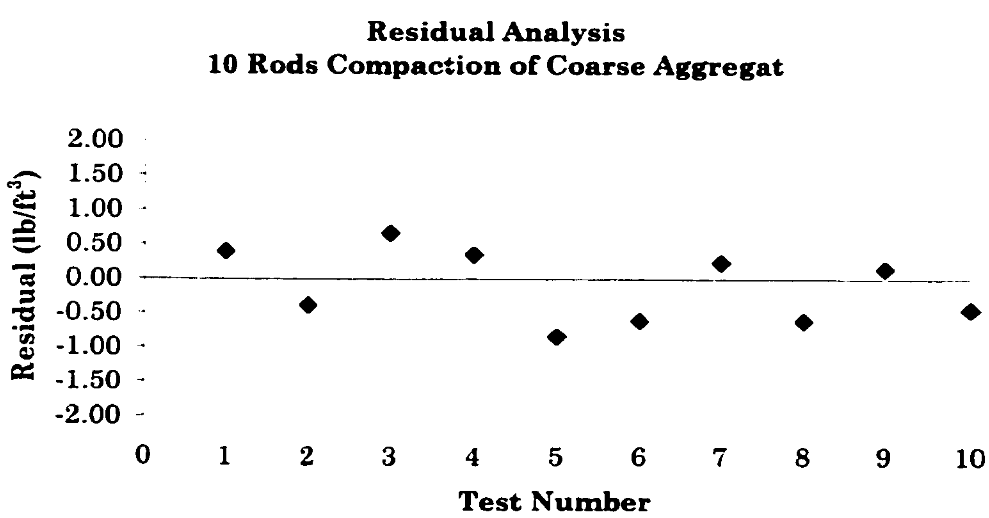


Figure 7-3 Residual Analysis for 10 Rods Compaction Unit Weight of Coarse Aggregate

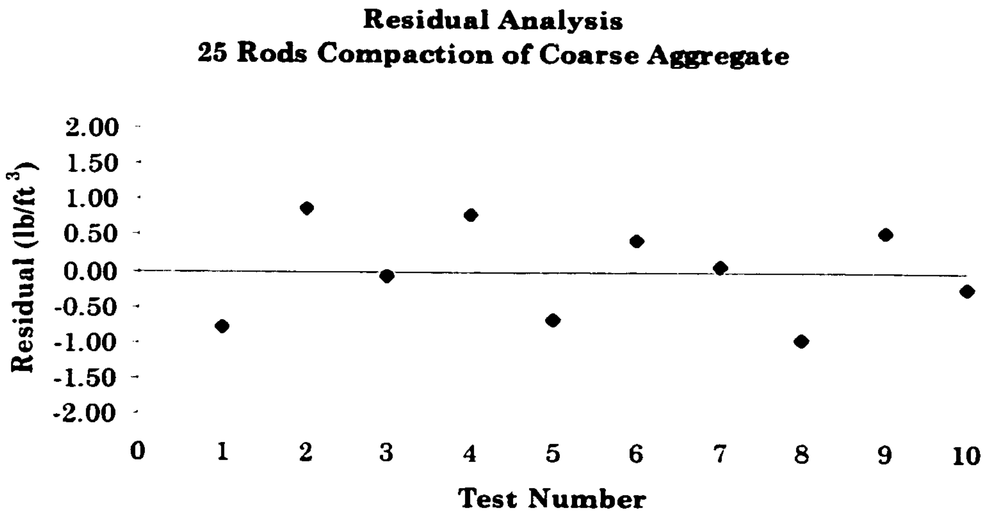


Figure 7-4 Residual Analysis for 25 Rods Compaction Unit Weight of Coarse Aggregate

The analysis of residuals is a visual identification of trends in the ordered test residual output. This analysis of ordered test residuals does not show any large change in test value with repeated testing of the aggregate materials. The unit weight tests are normally scattered about the average (0 on the Y axis of residual plot) with little to no trend in the scatter plot, which signifies the normality of the testing method. There is no noticed effect in the change in unit weight after several tests that can be seen in the test data. This is not to say that no breakdown is taking place, rather the breakdown that does occur with repeated testing is not large.

7.1.1.2 *Precision of Test Data*

Each of the reported unit weights in is the average of 10 repeated tests. Analysis of the test data for uncompacted unit weight, 10 rods unit weight, and 25 rods unit weight show that the maximum standard deviation of the test is 0.85 lb/ft^3. The average standard deviation for the test is 0.52 lb/ft^3 for all samples tested.

7.1.2 *Discussion on Coarse Aggregate Testing*

The comparison of aggregate tests requires normalization of the aggregate test data with the specific gravity. Examination of the voids in the coarse aggregate first requires that the unit weights are normalized by the bulk specific gravity, thereby allowing the direct comparison between aggregate types, and sizes. Although in the development of an aggregate blend the first selection is the design unit weight, this unit weight is converted to voids in the coarse aggregate. The use of voids in the coarse aggregate is therefore the first piece of information that is used in the proposed aggregate bending procedures. The following analysis of coarse aggregate is based on comparison of voids in the coarse aggregate.

7.1.2.1 *Effect of Aggregate Type*

Analysis on the change in aggregate properties with changing aggregate type utilizes the data from Table 7-2. The comparison in aggregate type can be made across the 3 types of aggregate that are presented in this data set.

There is a difference in the uncompacted voids in coarse aggregate for different types of aggregate. The average value for the uncompacted voids in the coarse aggregate for the dolomite, slag, and gravel are 46.2%, 47.2%, and 51.1% respectively. Knowing that the standard deviation of this test is 0.4%, these differences are significant at 95% confidence. Therefore, aggregate type has an effect on the uncompacted voids in the coarse aggregate.

The result of the aggregate type effect is expected, however question is called to the gravel having the highest voids. A visual inspection of the aggregates for textures shows that the slag has the most visible texture with the gravel to follow and finally the dolomite. It would be expected that this order would translate to the uncompacted voids in the coarse aggregate.

The analysis of the compacted voids shows similar results, with the aggregate types having different compacted voids. The average voids in the compacted condition are 39.3%, 41.6%, and 42.1% for the dolomite, slag, and gravel.

Based on the results of aggregate type it is seen that aggregates from different sources, which have different particle texture, pack together differently. This difference in packing would require a change in the aggregate blend with the use of each of these coarse aggregates to produce similar packing results. Increased voids in the coarse aggregate will require more fine aggregate to fill the increased volume of voids.

7.1.2.2 Effect of Aggregate Shape

The aggregates used in this study were 100 percent crushed materials on 2 or more faces, which does not allow a complete analysis of aggregate shape. The shape of the crushed particle is classified by examining the flat and elongated aggregate percentages for the coarse aggregate. It has been shown that changing the percentage of flat and elongated aggregate in an asphalt mixture will change the amount of particle breakdown in the design of those mixtures, however will not change the mixture volumetrics outside of typical testing variation[53, 54].

The comparison of particle shape can best be accomplished by examining the JSG Chips and the Dolomite Chips. This comparison is most appropriate because of the source location and sizing of these two materials. These aggregates are both taken from the south and southwest suburbs of Chicago, giving similar geological properties. The aggregates are both graded as CA-16 aggregates and, due to the testing procedures used in this study, are combined to the exact same gradation. The difference in these aggregates is the percentage of flat and elongated particles in each material. The percentage of flat and elongated for the JSG Chips is 11.9 percent while the Dolomite Chips tested for flat and elongated at 35.9 percent, tested with a 5 to 1 maximum to minimum particle dimension.

It is seen in examining these materials, shown in Table 7-2, that no difference is noticed between these two aggregates. The JSG Chips have 46.0 percent voids while the Dolomite Chips have 46.3 percent voids in the uncompacted aggregate. In the compacted state the JSG Chips have 39.0 percent voids with the Dolomite Chips having 39.3 percent voids.

The effect of crushed particle shape is not significant, therefore the voids in the coarse aggregate that exist with the flat and elongated particles are similar to those experienced with more cubical particles. This agrees with the findings by Vavrik et. al. in the study of flat and elongated particles which shows some difference in volumetric properties, however the difference is within typical limits for design and construction of asphalt pavements.

7.1.2.3 Effect of Maximum Aggregate Size

A direct comparison of maximum aggregate size can be performed by examining the ATREL CM-11 and the ATREL Chips. These aggregates were from the same source, were crushed in the same crusher, and are analyzed utilizing the median value of the aggregate specification, thereby providing the basis for comparison.

Data given in Table 7-2 shows that there is no effect of the maximum aggregate size. The larger ATREL CM-11 shows 47.3% uncompacted voids with the smaller ATREL Chips showing 47.9% uncompacted voids. The compacted voids for the two aggregate sizes show the same volume of compacted voids at 40.1%.

The effect of maximum aggregate size is not significant in the volume of resulting voids, which lends credibility to the applicability of the proposed design concepts for different maximum aggregate sizes. If the volume of voids were significantly different, there may be a change in the method desired for filling those voids.

7.1.2.4 Effect of Compactive Effort

The compactive effort applied to the aggregate sample has a significant effect on the resulting voids in the coarse aggregate. Examination of the percent change in voids in the aggregate from uncompacted to 10 rods of compaction shows an average decrease in voids of 12%. The additional change in voids from 10 rods to 25 rods of compaction is 4% on average, giving an average change in voids of 16% from the uncompacted voids to the 25 rods compaction. In all cases more densification is realized in the first 10 rods of the coarse aggregate than the next 15 rods necessary to reach 25 rods total.

The percentage change in voids between the uncompacted state and the 25 rods state shows similar results no mater what type of aggregate is tested. The average void reduction is 16% with a standard deviation of 1.8% for the 30 tests conducted on coarse aggregates of different type, shape, maximum size, and gradation.

The slag and gravel aggregate sources, being a different aggregate type than the remainder of the aggregates, give decreased reduction in voids in the coarse aggregate. Figure 7-1 shows that the slope of the line, which indicates the reduction in voids, is flatter for the slag and gravel aggregates. This change in voids with compactive effort is a result of the different surface texture with the different aggregate source. Changes in compactive effort will change the compaction of different aggregates differently.

The reduction in voids of the coarse aggregate gives guidance about the relative amount of fine aggregate required to fill those voids. With the limits for coarse aggregate interlock near the uncompacted condition of the coarse aggregate and the maximum practical limit near the 25 rods of compaction the mix designer can easily recognize the amount of change in coarse aggregate allowed in the mixture while maintaining coarse aggregate interlock in the mixture.

7.1.2.5 Effect of Change in Gradation

The change in aggregate gradation is examined using the data in Table 7-2 and Table 7-3. The data in Table 7-2 is used to examine the typical results of materials that are found within the specification, while Table 7-3 gives more information on changes in the fine portion of the coarse aggregate gradation. Table 5-1, Table 5-2, and Table 5-3 give the specification for the coarse aggregate gradation.

The standard testing, which varied the gradation from the coarse to the fine limit of the appropriate specification, gives the typical limits of expected aggregates for Illinois conditions. This data shows that there is a difference in the uncompacted voids from the coarse to fine gradation for the majority of the aggregates tested. The analysis gives a result that changing from the medium to the coarse or fine gradation does not give a difference, but moving from the coarse to the fine gradation does change the volume of voids in the uncompacted state.

The 25 rod voids for these aggregates show that less difference is observed between the coarse and fine gradation. The addition of compactive effort reduces the difference in compacted voids between the aggregate gradations, but does not change the significance of the change in gradation.

The additional test data, given in Table 7-3, shows that the change in gradation from the medium to the fine gradation does not change the voids in the uncompacted or rodded coarse aggregate. Because there is no change in the volume of voids and smaller particles are included in the fine mixture the size of the resulting voids must be decreasing when the aggregate becomes fine.

Typical unit weight and voids testing for coarse aggregate are performed on coarse aggregates with the material passing the #4 (4.75-mm) sieve. The testing of aggregates for this study and the proposed design concepts test the whole aggregate, without removing any of the material. Examining the > #4 Material from Table 7-3 shows that the voids in the coarse aggregate are 3 percent higher on average. The values produced by the rodded voids of the whole aggregate can not be directly compared with the ASTM standard unit weight and voids in the coarse aggregate and if used in an analysis of a mixture would produce erroneous results.

7.2 Fine Aggregate Testing

Fine aggregate is used as a filler material in the asphalt mixture and therefore must be packed in to the voids created by the coarse aggregate. Because the primary deformation resistance is derived from the coarse aggregate, under the proposed mix design concepts, the fine aggregate packing is less emphasized in the testing of dry aggregates.

7.2.1 Fine Aggregate Test Results

One aggregate source for IDOT FA-01 was sampled, broken down into component aggregate sizes, and recombined for aggregate voids testing. This aggregate was produced to the fine, medium, and coarse gradation specification. Testing of this aggregate was performed in the UVCATA as well as the standard test apparatus for fine aggregate angularity.

The results for unit weight and the voids in the fine aggregate are given in Table 7-4 and Figure 7-5 for tests conducted in the UVCATA. The results for fine aggregate angularity are given in Table 7-5.

Table 7-4 Unit Weight and Voids in the Aggregate for Fine Aggregate Testing

	Unit Weight (pcf)			Voids in FA (%)		
	Loose	10 Rod	25 Rod	Loose	10 Rod	25 Rod
FA-01 Natural Sand						
Fine	106.1	111.9	113.6	33.0	29.4	28.3
Medium	105.8	111.1	112.8	33.2	29.9	28.8
Coarse	104.4	109.2	109.9	34.1	31.1	30.6

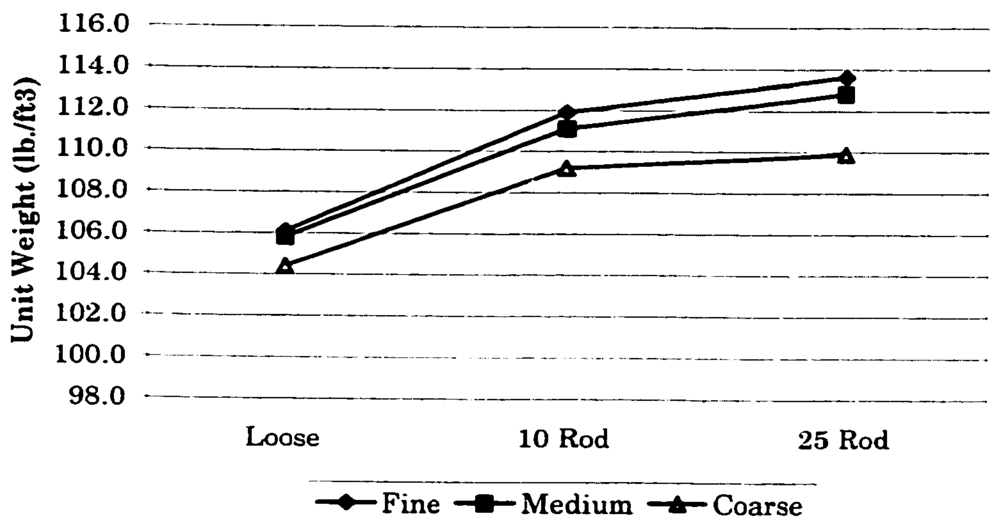


Figure 7-5 Unit Weight Test Result Plot for Fine Aggregate Testing

Table 7-5 Fine Aggregate Angularity Results for FA-01

	Fine Aggregate Angularity
Method A AASHTO Standard Gradation	40.8
Method C Medium Stockpile Gradation	38.1

7.2.1.1 Variability of Test Methods

The repeated testing of aggregates can cause degradation of the aggregate material, which results in changing in test results. This aggregate degradation can be especially prominent with increased compactive effort. The analysis of residuals with the tests taken in order will identify the existence of any change test value after repeated tests. This analysis looks for visual trends in the residual analysis plots that would show trends in the testing results. Figure 7-6,
Figure 7-7, and Figure 7-8 give the residual analysis plots for the uncompacted voids, 10 rods, and 25 rods testing.

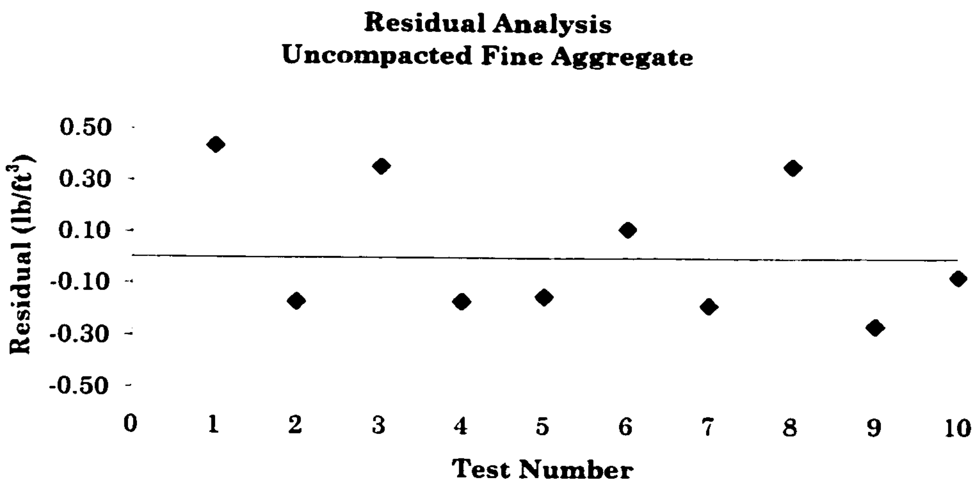


Figure 7-6 Residual Analysis for Uncompacted Unit Weight of Fine Aggregate

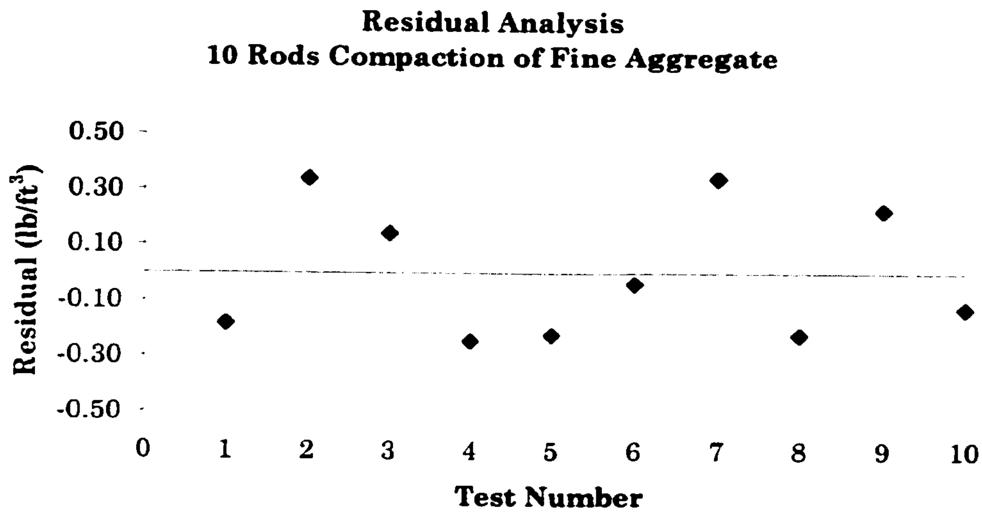


Figure 7-7 Residual Analysis for 10 Rods Compaction Unit Weight of Fine Aggregate

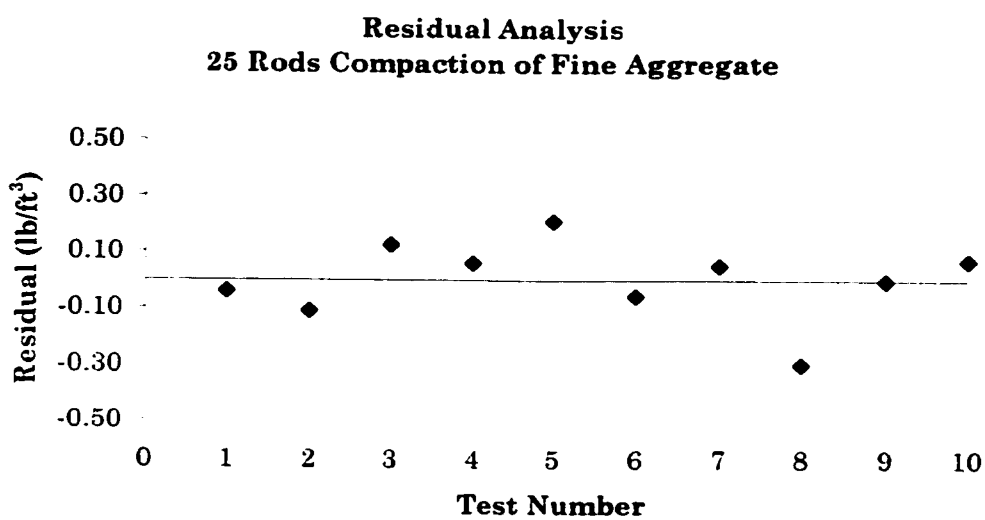


Figure 7-8 Residual Analysis for 25 Rods Compaction Unit Weight of Fine Aggregate

The analysis of ordered residuals does not show any changing trend in test value with repeated testing of the aggregate materials. The unit weight tests are normally scattered about the average (0 on the Y axis of residual plot) with no trend in the scatter plot, which signifies the normality of the testing method. There is no noticed effect in the change in unit weight after several tests that can be seen in the test data. This is not to say that no breakdown is taking place, rather the breakdown that does occur with repeated testing is not significant in this test procedure.

The variability in the test results decreases with increasing compactive effort. The residual analysis shows the error of the estimate for each of the modes of aggregate compaction. Comparison of Figure 7-6,
Figure 7-7, and Figure 7-8 shows that the maximum error of the estimate for the uncompacted state is 0.44 lbs./ft^3 while the maximum error of the estimate for the 25 rods of compaction is 0.21 lbs./ft^3.

7.2.2 Discussion on Fine Aggregate Testing

Comparison of change in compactive effort and change in gradation will give guidance in the volume of aggregate necessary to fill the voids created in the coarse aggregate. The discussion of the fine aggregate testing will be based on the unit weight values given testing in the UVCATA. The design procedure for mixture design only utilizes the rodded weight of the fine aggregate, therefore the analysis based on unit weight is appropriate for the evaluation of aggregate.

7.2.2.1 Effect of Compactive Effort

The change in unit weight of the fine aggregate is significant when applying compactive effort. The data in Table 7-4 shows that there is considerable densification, increase in unit weight, with increasing the compactive effort. In all cases more densification is realized in the first 10 rods of the coarse aggregate than the next 15 rods necessary to reach 25 rods total.

7.2.2.2 *Effect of Change in Gradation*

Changing the gradation of the fine aggregate effects both the unit weight and the change in unit weight with compaction. The coarse gradation of fine aggregate has a lower unit weight, fewer voids in the fine aggregate, than a medium or fine gradation (Table 7-4, Figure 7-5). The fine and medium gradation s are similar in the loose condition, but show some difference in the 25 rods compacted state.

The change in densification with increased compactive effort is affected by the gradation of the fine aggregate. The more fine the gradation of the fine aggregate, the more densification between the uncompacted and 25 rods of compaction. The fine gradation of fine aggregate showed a 6.6% change in unit weight, while the coarse gradation of fine aggregate only showed a 5.0% change in unit weight.

7.2.2.3 *Comparison of UVCATA to Fine Aggregate Angularity Test*

The tests performed in the UVCATA give similar results to the standard fine aggregate angularity (FAA) test established in Superpave.[21] The tests conducted in the UVCATA are used to determine the dry rodded weight of fine aggregate that will be used to fill the voids in the coarse aggregate. The FAA test was adopted by Superpave as a quality indicator for fine aggregate.

The data from method C of the fine aggregate angularity test can be compared to the voids in the coarse aggregate in the loose condition. The voids in the fine aggregate for the medium gradation is 33.2% and the FAA test under method C is 38.1%. These test values are significantly different. This difference is based in large part on the size of the testing apparatus and the test parameters. Because the voids in the fine aggregate in the loose condition will show similar trends to the FAA test this test in the UVCATA can be substituted for evaluation of different fine aggregates. The development of a specification for the test in the UVCATA would require additional study to determine the appropriate limits and procedures for the test.

The dry rodded weight is used to determine the volume of fine aggregate in the combined blend. This dry rodded weight is determined from the test outlined in Chapter 4 utilizing the standard bucket for the UVCATA.

7.3 Summary Aggregate Testing

Examination of aggregate packing for coarse and fine aggregates validates the previous claim that the packing of an aggregate to fill a unit volume is dependant on the characteristics of the aggregate material and the test method. The results from this testing show that the shape, texture, gradation, and aggregate size have change the resulting voids in the aggregate. Change in the test procedure and compactive energy also effect the resulting voids of dry aggregate testing.

The testing methods utilized in this experiment are acceptable for the evaluation of coarse and fine aggregates. The use of the UVCATA is appropriate for determining the voids in an aggregate in the uncompacted and rodded conditions. Repeated testing of the same sample does not change the test result and a minimum of three tests should be use to determine the average unit weight.

The effect of aggregate type was noticeable and significant in the change in voids for coarse aggregates. A change in aggregate source material will change the aggregate packing. Aggregates such as gravel and steel slag, which have improved surface texture, have increased voids in the coarse aggregate.

The shape of a crushed aggregate particle as measured by the flat and elongated aggregate percentage does not significantly change the voids in the coarse aggregate.

Literature has shown that examination of different aggregate shapes from rounded to cubical crushed will change the voids in the coarse aggregate.[38]

Gradation has a significant effect on the voids in an aggregate, however small changes in gradation may not be noticed in the testing for voids. Results of aggregate testing by Hossain et. al.[55] are similar to the results of this study. Changing gradation will change the voids in an aggregate structure, this result provides the basis for continued evaluation of aggregate gradation in asphalt mixtures.

The effect of maximum aggregate size is not significant in the volume of resulting voids, validating to the applicability of the proposed design concepts for different maximum aggregate sizes.

CHAPTER 8 PRELIMINARY VOLUMETRIC RESULTS AND DISCUSSION

A small preliminary experiment was performed to examine the presented mix design concepts. This small preliminary experiment preceded the primary experiment in this study and was used to investigate the design concepts and the need for further investigation. The objective was to examine the volumetric differences in mixtures where the design unit weight was varied from 10% below the loose weight to 10% above the loose weight.

Typical aggregates for surface mixtures (9.5-mm NMPS), including a CA-16 (3/8" Crushed Stone), FA-20 (Manufactured Sand), FA-01 (Natural Sand), and MF-01 (Mineral Filler), were selected and broken down into their component sizes for recombination into precisely controlled gradations. The coarse aggregate was tested to determine its loose and rodded unit weight. The fine aggregates were tested to determine their rodded unit weights.

The gradations were established using 5.0% passing the #200 sieve, a 50%-50% blend by volume of manufactured and natural sand, and the following design unit weights (DUW) of coarse aggregate: 80, 85, 88(Loose Unit Weight), 93, 95, 97(Rodded Unit Weight). Table 8-1 gives the final blending gradations for the samples and Figure 8-1 shows these gradations on the standard gradation plot. All samples were mixed with 5.4% asphalt and aged for 2-hours.

Two samples from each gradation were compacted in a Troxler Model 4140 Gyratory Compactor to 100 gyrations. The volumetric properties including maximum specific gravity (G_{mm}) and bulk specific gravity (G_{mb}) were determined for each mixture. The samples were then analyzed to evaluate the volumetric and compaction properties of the various mixtures. A summary of the volumetric and compaction data is given in Table 8-2.

Table 8-1 Gradation Data for Preliminary Experiment Samples

Sieve Size		Design Unit Weight					
mm	US	80	85	88	93	95	97
12.5	1/2"	100	100	100	100	100	100
9.5	3/8"	98	98	98	98	98	98
4.75	#4	64	62	60	58	57	56
2.38	#8	39	37	35	33	32	31
1.18	#16	27	25	24	23	22	22
0.6	#30	18	17	17	16	15	15
0.3	#50	10	10	9	9	9	9
0.15	#100	7	7	7	7	7	7
0.075	#200	4.9	4.9	4.9	5.0	5.0	5.0

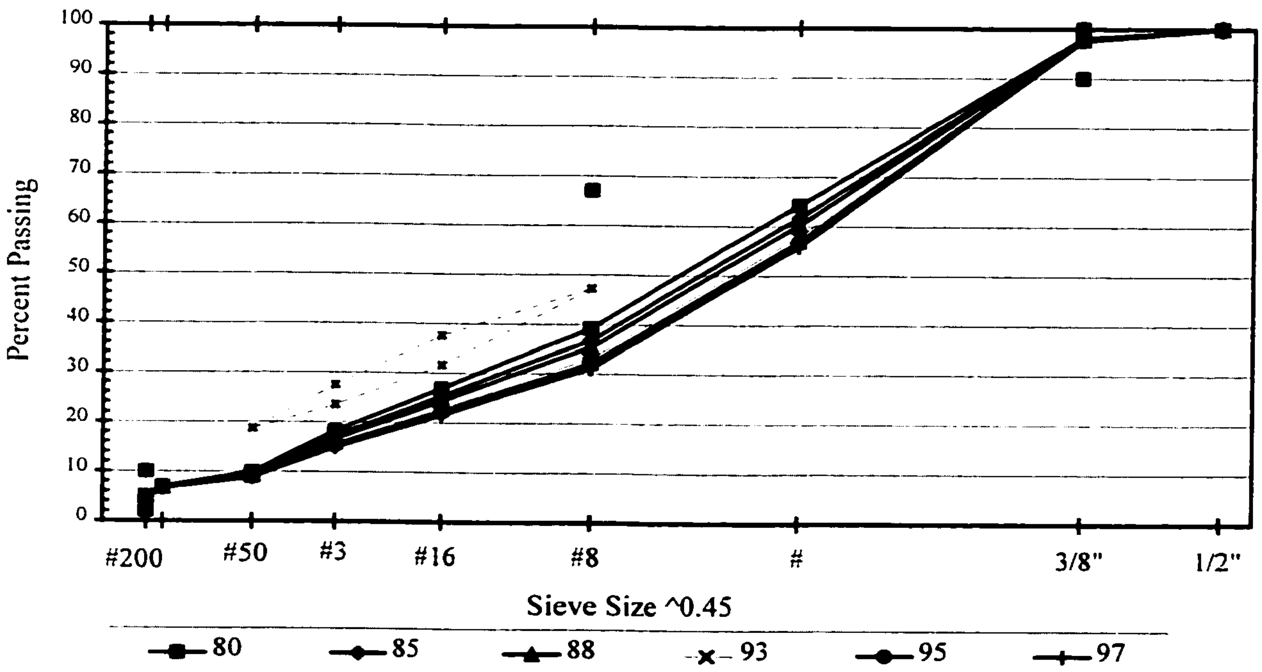


Figure 8-1 Gradation Plot for Preliminary Experiment Samples

Table 8-2 Volumetric and Compaction Data for Preliminary Experiment Samples

		Design Unit Weight					
		80	85	88	93	95	97
Maximum Specific Gravity	G_{mm}	2.509	2.509	2.51	2.513	2.518	2.514
Bulk Specific Gravity	Gmb	2.406	2.429	2.415	2.403	2.408	2.414
Air Voids	VTM	4.1%	3.2%	3.8%	4.4%	4.4%	4.0%
Voids in Minneral Aggregate	VMA	14.7%	14.0%	14.4%	14.9%	14.7%	14.5%
Voids Filled with Asphalt	VFA	72.2%	77.2%	73.9%	70.6%	70.3%	72.7%
Locking Point	N_{lp}	87	93	96	96	97	102
Voids @ N_{lp}		5.2%	4.2%	4.6%	5.3%	5.2%	4.7%
Number of Gyrations to 4%	$N_{4\%}$	115	93	107	124	124	115
Slope		8.96	9.29	9.61	9.56	9.79	10.04

8.1 VOLUMETRIC RESULTS

The effect of aggregate interlock on compaction can be observed by examining the volumetric data from these mixtures. Figure 8-2 shows that as the design unit weight is taken above the loose weight there is more coarse aggregate structure, which requires higher compactive effort to compact the mixture. This is evident by the increase in air voids for the DUW=93 and DUW=95 samples, which have design unit weights increasing above the loose unit weight. Conversely, as the design unit weight is lowered below the loose unit weight, DUW=85, the sample is lacking the coarse aggregate structure necessary to resist compaction that gives lower air voids. The DUW=80 sample is considerably out of coarse aggregate interlock and therefore the void structure is governed by the fine aggregate structure. Examination of the fine aggregate structure for this mixture shows that significant voids exist in the fine aggregate portion of the combined blend. Because of this fine aggregate structure and use of crushed stone sand as the fine aggregate, the air voids are acceptable even though coarse aggregate interlock is never developed.

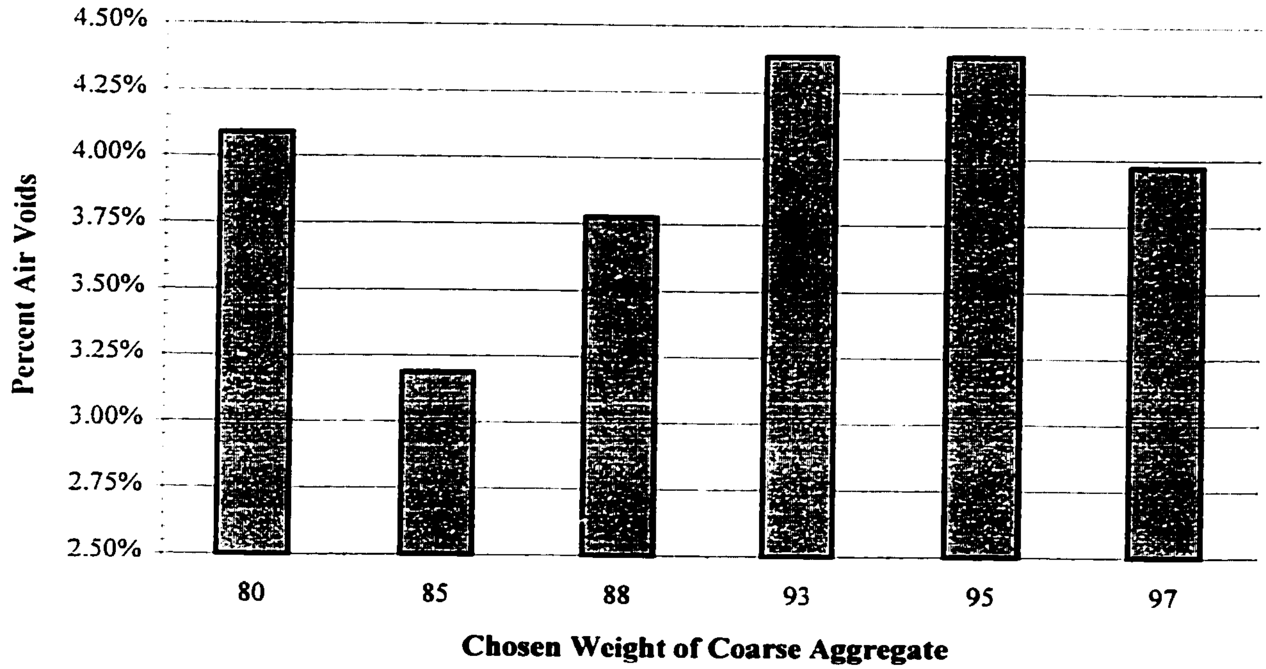


Figure 8-2 Air Void Plot for Preliminary Experiment Samples

Figure 8-3 shows the plot for VMA in these compacted samples. The trends in the data are very similar to those of the air voids. This is expected because the design of the experiment maintained a constant asphalt content and percentage of dust, therefore the VMA will follow the air voids. Also showing the similar trend is the number of gyrations to 4% air voids (N4%). These results are given in Figure 8-4. This result is also expected due to the design of the experiment where the volume of coarse aggregate in the mixture is increasing for each mixture.

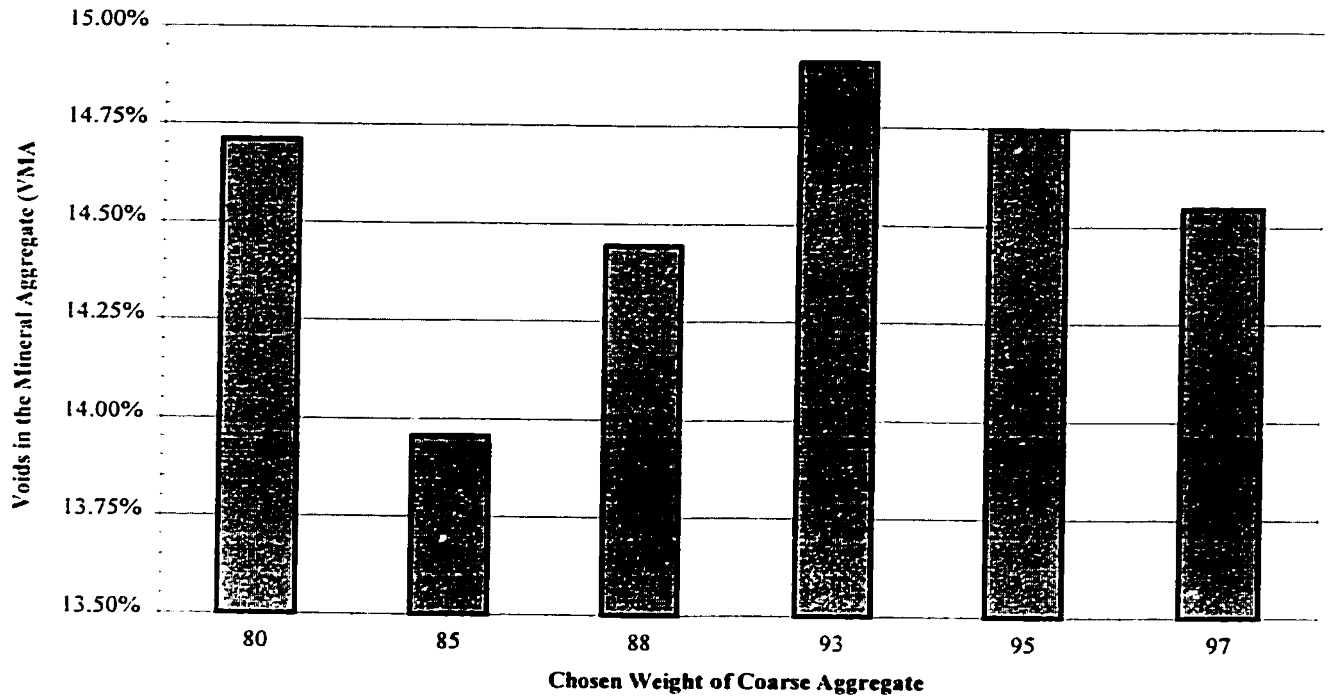


Figure 8-3 Voids in the Mineral Aggregate for Preliminary Experiment Samples

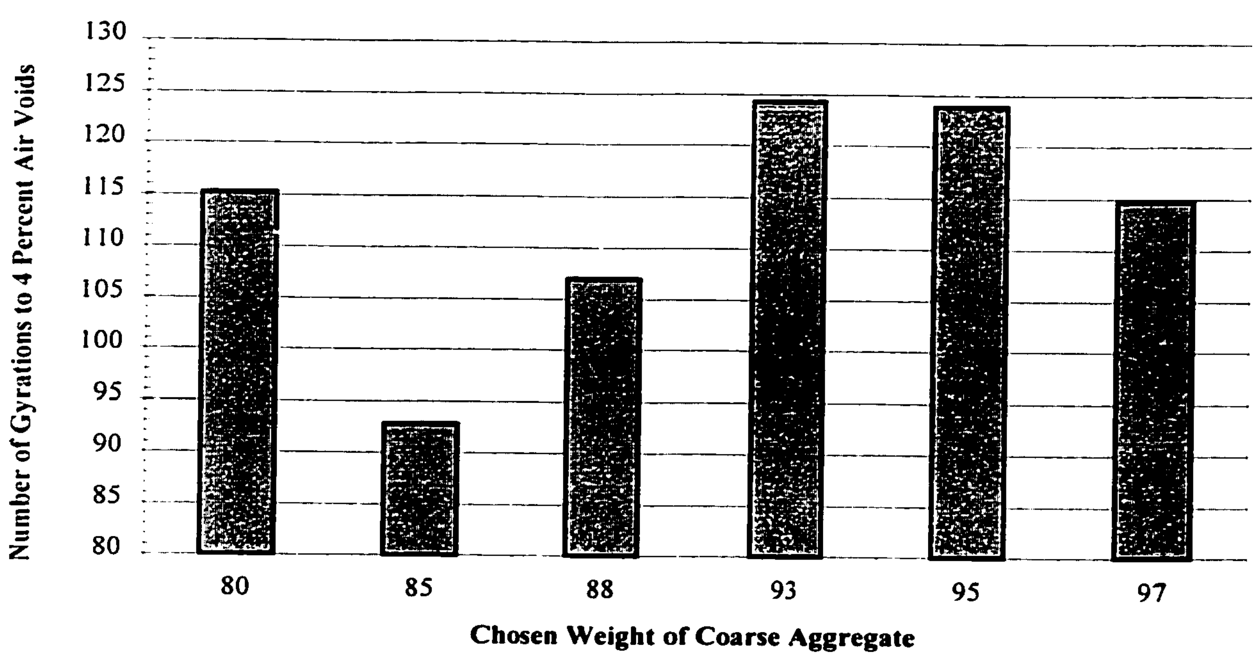


Figure 8-4 Number of Gyrations to 4% Air Voids for Preliminary Experiment Samples

8.2 COMPACTION RESULTS

Densification slope (%G_{mm} vs. Log Gyrations) in the gyratory compactor has been suggested as an indicator of the resistance of a mixture to compaction, and thus an indicator of the quality of a mixture. It is seen in these samples that as the design unit weight increases the densification slope also increases (Figure 8-5). This lends some credence to the use of the densification slope as an indicator of aggregate interlock. A more accurate evaluation would utilize the fact that this curve is not linear and actually has different slopes at different compaction levels.

The locking point was determined for each of the compacted samples and is shown in Figure 8-6. This indicator of aggregate lockup shows similar results to the trend in the densification slope; as the design unit weight increases the locking point also increases. This data indicates that the locking point may be an adequate indicator of the coarse aggregate interlock that is established in each of these mixtures.

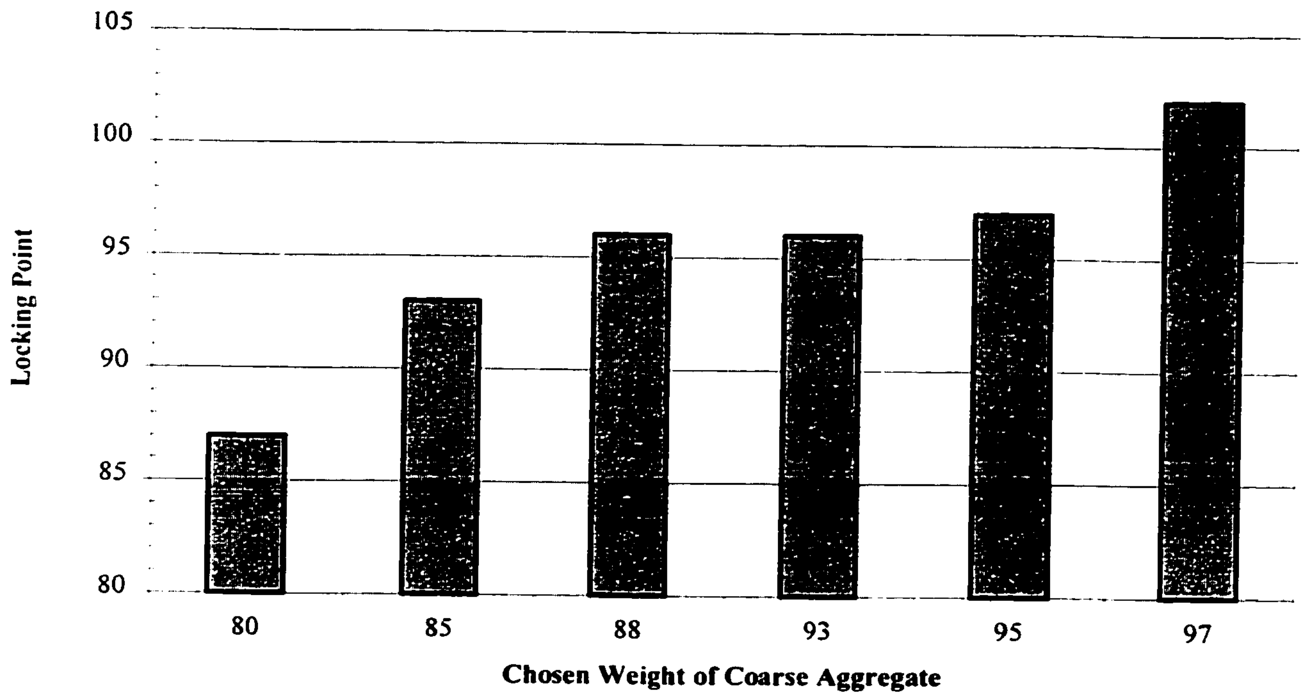


Figure 8-5 Gyratory Densification Slope for Preliminary Experiment Samples

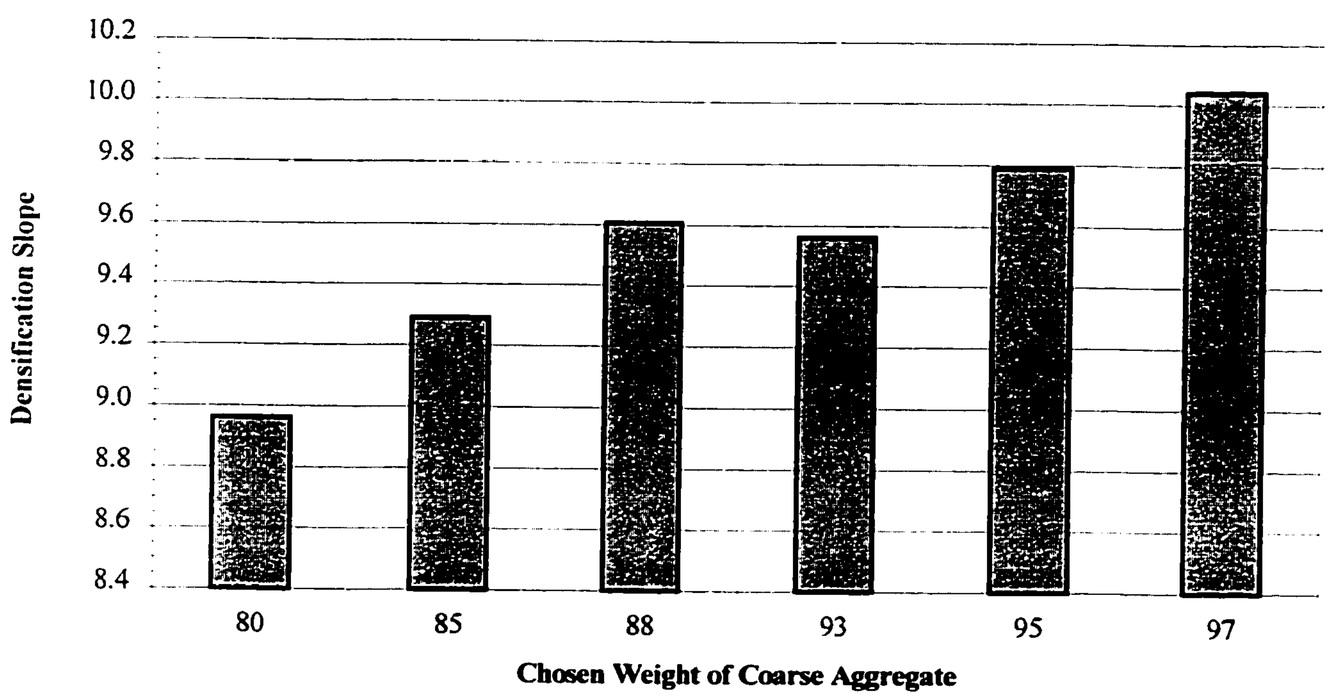


Figure 8-6 Locking Point for Preliminary Experiment Samples

8.3 Summary of Preliminary Mixture Volumetric Results

The preliminary experimental mixture results show further experimentation into the development of asphalt mixtures with coarse aggregate interlock would provide a valuable improvement into the design of asphalt mixtures. Clear trends exist that show an increase in voids in an asphalt mixture with an increase in coarse aggregate. This trend is evident when the volume of coarse aggregate is near to the minimum value necessary for coarse aggregate interlock. The understanding of the relationship between the volume of coarse aggregate in and asphalt mixture and the resulting mixture volumetrics will improve the design and performance of asphalt mixtures.

Aggregate interlock can be captured through analysis of the volumetric properties and compaction characteristics of asphalt mixtures. The change in volume of coarse aggregate has an effect on the resulting volumetric properties of a compacted asphalt mixtures. The understanding of this change in mixture volumetrics provides an opportunity to design a mixture with aggregate interlock. The design of the compaction characteristics and resulting mixture volumetrics allow a mixture to be easily designed for the application in the pavement.

CHAPTER 9 HMA VOLUMETRIC TEST RESULTS AND DISCUSSION

The testing plan in Chapter 5 outlines an experiment to improve the understanding of aggregate interlock and the design of asphalt mixtures. This experiment utilizes one coarse and one fine aggregate assembled to the coarse limit, median value, and fine limit. For each combination of coarse and fine aggregate, mixtures will be created with selected coarse aggregate volume near the point where aggregate interlock is developed. The results from this experiment provide an understanding of the change in aggregate gradation and the effect on resulting mixture volumetric property changes.

9.1 Volumetric Property Test Results

9.1.1 Asphalt Mixtures and Volumetric Properties

An experimental test matrix is given in Table 5-7 for the study of aggregate interlock and change in aggregate gradation. Utilizing the mixture design concepts outlined in Chapter 4 and the test matrix given in Table 5-7 26 individual mixtures were developed. These mixtures contained varied relative percentages of coarse and fine aggregate while keeping the material passing the #200 (0.075-mm) sieve and asphalt cement content constant.

Table 9-1 gives the gradation of each component aggregate and volume of coarse aggregate through the design unit weight. Table 9-2 provides the loose unit weight, rodded unit weight, and design unit weight for the coarse aggregate for each mixture in the test scheme. Table 9-2 also gives the rodded unit weight of the fine aggregate for each of the tested mixtures.

Using the concepts outlined in Chapter 4 and the test values given above blending percentages for the coarse aggregate, fine aggregate and mineral filler were calculated. These calculated blending percentages are presented in Table 9-3.

The gradations for the individual mixtures are developed using the blending percentages and the stockpile gradations for the coarse and fine aggregate. These gradations are tabulated using standard sieve sizes in Table 9-4 and are plotted on the standard 0.45 power curve in Figure 9-1.

Figure 9-1 plots all 26 aggregate gradations in one figure, which creates confusion in understanding the difference between the blocks and levels of coarse aggregate in the experiment. Figure 9-2 through Figure 9-6 show the plots for each block of the experiment, thereby allowing a more individual comparison of the change of shape and location of the gradation for each individual block of the experiment.

Table 9-1 Test Matrix for Experimental Design with Aggregate Gradation and Design Unit Weight of Coarse Aggregate

Sample Name	Coarse Aggregate Gradation	Fine Aggregate Gradation	Design Unit Weight
Block 1 -10	Med	Med	LUW - 10%
Block 1 -5	Med	Med	LUW - 5%
Block 1 LW	Med	Med	LUW
Block 1 +5	Med	Med	LUW + 5%
Block 1 +10	Med	Med	LUW + 10%
Block 2 -10	Coarse	Med	LUW - 10%
Block 2 -5	Coarse	Med	LUW - 5%
Block 2 LW	Coarse	Med	LUW
Block 2 +5	Coarse	Med	LUW + 5%
Block 2 +10	Coarse	Med	LUW + 10%
Block 3 -10	Fine	Med	LUW - 10%
Block 3 -5	Fine	Med	LUW - 5%
Block 3 LW	Fine	Med	LUW
Block 3 +5	Fine	Med	LUW + 5%
Block 3 +10	Fine	Med	LUW + 10%
Block 4 -10	Med	Coarse	LUW - 10%
Block 4 -5	Med	Coarse	LUW - 5%
Block 4 LW	Med	Coarse	LUW
Block 4 +5	Med	Coarse	LUW + 5%
Block 4 +10	Med	Coarse	LUW + 10%
Block 5 -10	Med	Fine	LUW - 10%
Block 5 -5	Med	Fine	LUW - 5%
Block 5 LW	Med	Fine	LUW
Block 5 +5	Med	Fine	LUW + 5%
Block 5 +10	Med	Fine	LUW + 10%
Block 6 -40	Med	Med	LUW - 40%

Table 9-2 Loose, Rodded, and Design Unit Weights of Aggregates in Mixture Testing

Sample Name	Coarse Aggregate			Fine Aggregate
	Loose Unit Weight (pcf)	Rodded Unit Weight (pcf)	Design Unit Weight (pcf)	Rodded Unit Weight (pcf)
Block 1 -10	88.7	100.1	79.8	112.8
Block 1 -5	88.7	100.1	84.3	112.8
Block 1 LW	88.7	100.1	88.7	112.8
Block 1 +5	88.7	100.1	93.1	112.8
Block 1 +10	88.7	100.1	97.6	112.8
Block 2 -10	87.5	100.6	78.8	112.8
Block 2 -5	87.5	100.6	83.1	112.8
Block 2 LW	87.5	100.6	87.5	112.8
Block 2 +5	87.5	100.6	91.9	112.8
Block 2 +10	87.5	100.6	96.3	112.8
Block 3 -10	88.4	100.9	79.6	112.8
Block 3 -5	88.4	100.9	84.0	112.8
Block 3 LW	88.4	100.9	88.4	112.8
Block 3 +5	88.4	100.9	92.8	112.8
Block 3 +10	88.4	100.9	97.2	112.8
Block 4 -10	88.7	100.1	79.8	109.9
Block 4 -5	88.7	100.1	84.3	109.9
Block 4 LW	88.7	100.1	88.7	109.9
Block 4 +5	88.7	100.1	93.1	109.9
Block 4 +10	88.7	100.1	97.6	109.9
Block 5 -10	88.7	100.1	79.8	113.6
Block 5 -5	88.7	100.1	84.3	113.6
Block 5 LW	88.7	100.1	88.7	113.6
Block 5 +5	88.7	100.1	93.1	113.6
Block 5 +10	88.7	100.1	97.6	113.6
Block 6 -40	88.7	100.1	53.2	112.8

Table 9-3 Blending Percentages of Coarse Aggregate, Fine Aggregate, and Mineral Filler for Asphalt Mixtures

Sample Name	Coarse Aggregate	Fine Aggregate	Mineral Filler
Block 1 -10	59.1	35.3	5.6
Block 1 -5	62.5	31.9	5.6
Block 1 LW	65.9	28.5	5.6
Block 1 +5	69.1	25.3	5.6
Block 1 +10	72.4	22	5.6
Block 2 -10	53.2	41.2	5.6
Block 2 -5	56.3	38.1	5.6
Block 2 LW	59.4	35	5.6
Block 2 +5	62.5	31.9	5.6
Block 2 +10	65.4	29	5.6
Block 3 -10	64.1	30.3	5.6
Block 3 -5	67.7	26.7	5.6
Block 3 LW	71.3	23.1	5.6
Block 3 +5	74.7	19.7	5.6
Block 3 +10	78.1	16.3	5.6
Block 4 -10	59.3	35.1	5.6
Block 4 -5	62.8	31.6	5.6
Block 4 LW	66.1	28.3	5.6
Block 4 +5	69.4	25	5.6
Block 4 +10	72.7	21.7	5.6
Block 5 -10	59.5	34.9	5.6
Block 5 -5	62.9	31.5	5.6
Block 5 LW	66.2	28.2	5.6
Block 5 +5	69.4	25	5.6
Block 5 +10	72.7	21.7	5.6
Block 6 -40	37	57.4	5.6

Table 9-4 Blended Aggregate Gradations for Asphalt Mixtures

	Percent Passing									
Sample Name	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	Asphalt Content
Block 1 -10	100	98	58	44	30	22	11	6	5.0	5.5
Block 1 -5	100	98	55	42	28	21	11	6	5.0	5.5
Block 1 LW	100	98	53	40	25	19	10	6	5.0	5.5
Block 1 +5	100	98	51	38	23	18	10	6	5.0	5.5
Block 1 +10	100	98	49	35	21	16	9	6	5.0	5.5
Block 2 -10	100	97	54	43	32	24	12	6	5.0	5.5
Block 2 -5	100	97	51	41	30	23	12	6	5.0	5.5
Block 2 LW	100	96	48	39	28	21	11	6	5.0	5.5
Block 2 +5	100	96	46	36	26	20	11	6	5.0	5.5
Block 2 +10	100	96	44	34	24	19	10	6	5.0	5.5
Block 3 -10	100	100	64	47	28	20	10	6	5.0	5.5
Block 3 -5	100	100	62	45	26	18	10	6	5.0	5.5
Block 3 LW	100	100	60	42	23	17	9	6	5.0	5.5
Block 3 +5	100	100	58	41	21	15	9	6	5.0	5.5
Block 3 +10	100	100	57	39	19	14	8	6	5.0	5.5
Block 4 -10	100	98	56	44	29	17	7	6	5.0	5.5
Block 4 -5	100	98	54	41	26	16	7	6	5.0	5.5
Block 4 LW	100	98	52	39	24	15	6	6	5.0	5.5
Block 4 +5	100	98	50	37	22	14	6	6	5.0	5.5
Block 4 +10	100	98	48	35	21	13	6	6	5.0	5.5
Block 5 -10	100	98	58	44	31	27	16	6	5.0	5.5
Block 5 -5	100	98	56	42	28	25	15	6	5.0	5.5
Block 5 LW	100	98	54	40	26	23	14	6	5.0	5.5
Block 5 +5	100	98	51	38	24	21	13	6	5.0	5.5
Block 5 +10	100	98	49	35	22	19	12	6	5.0	5.5
Block 6 -40	100	99	72	59	44	32	15	7	5.0	5.5

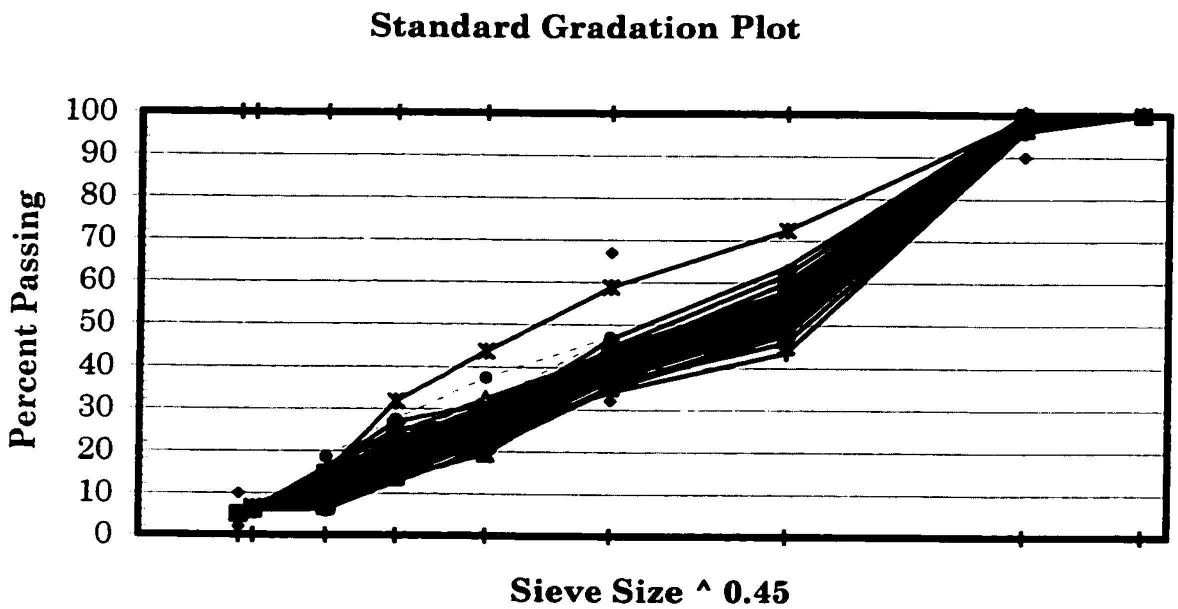


Figure 9-1 Standard Gradation Plot for All Asphalt Mixtures

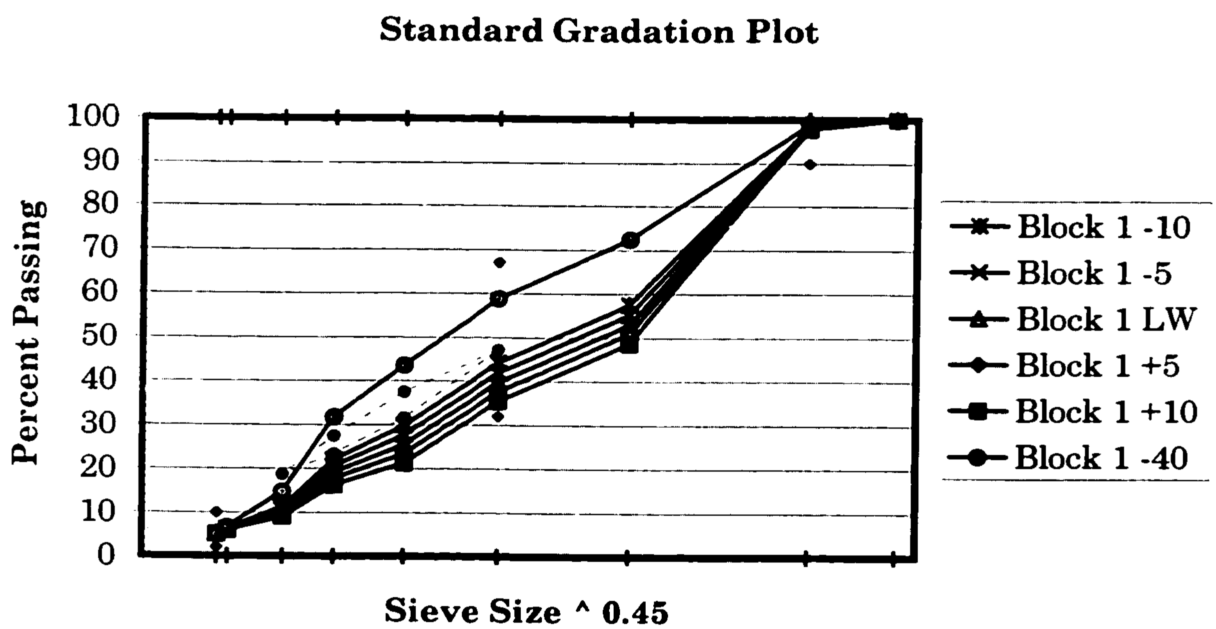


Figure 9-2 Block 1 and Block 6 Standard Gradation Plot

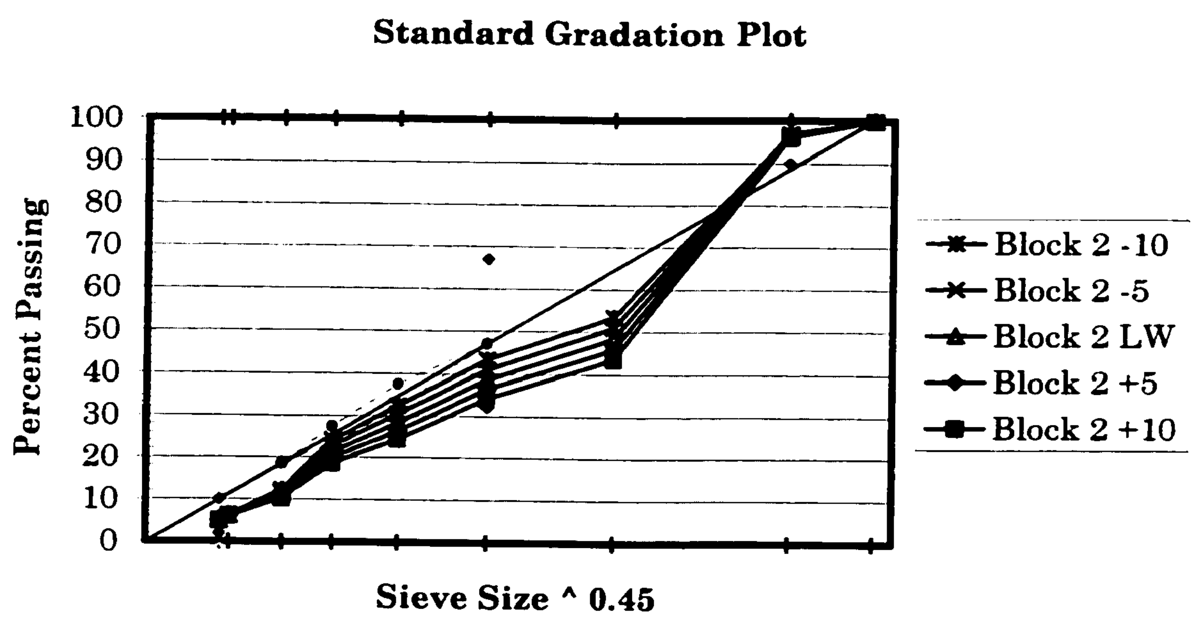


Figure 9-3 Block 2 Standard Gradation Plot

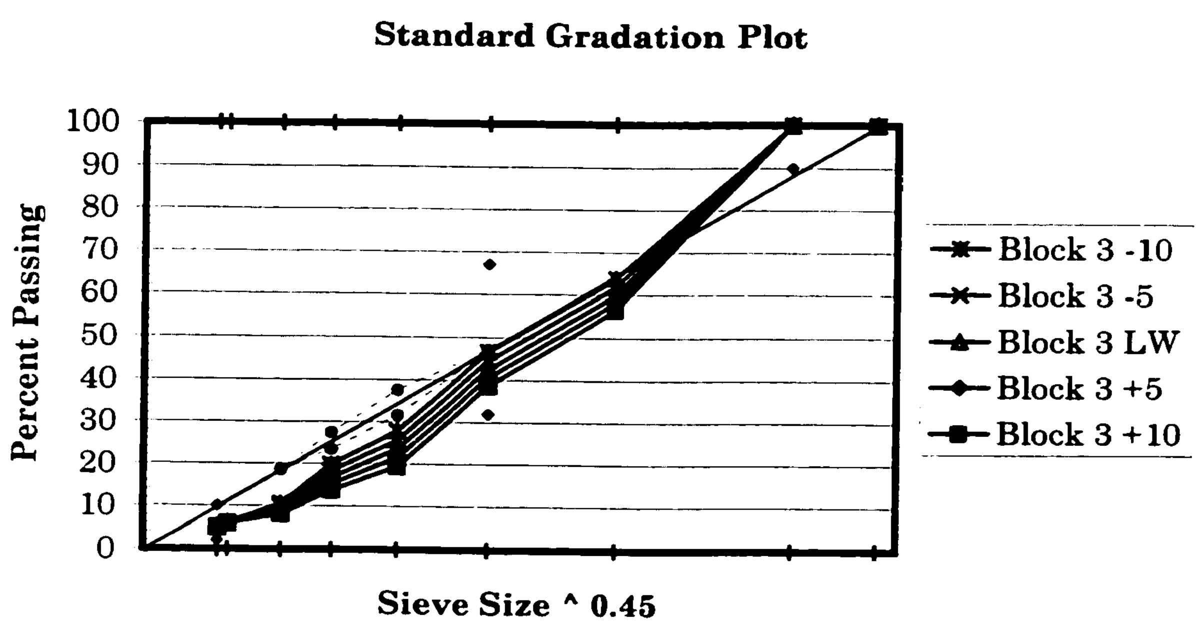


Figure 9-4 Block 3 Standard Gradation Plot

Standard Gradation Plot

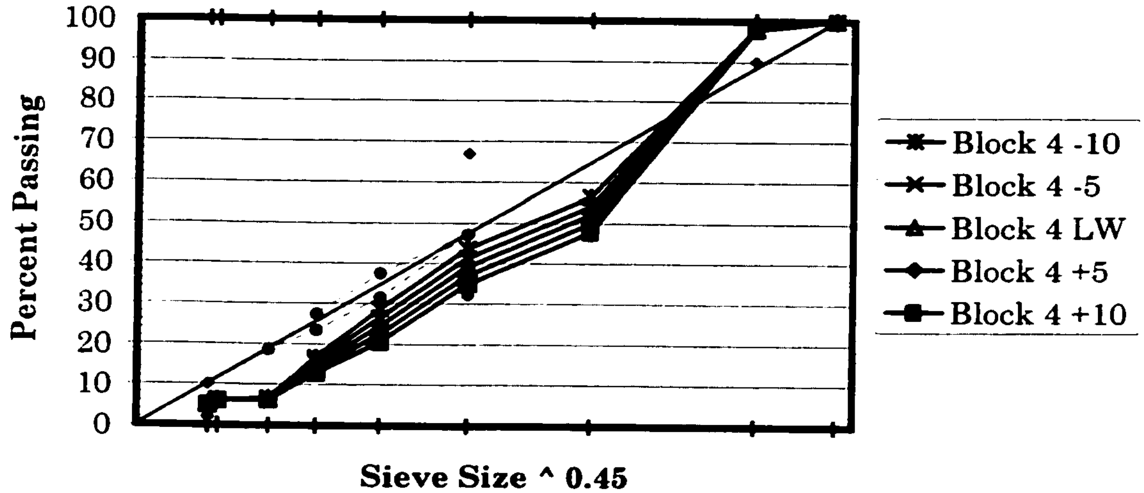


Figure 9-5 Block 4 Standard Gradation Plot

Standard Gradation Plot

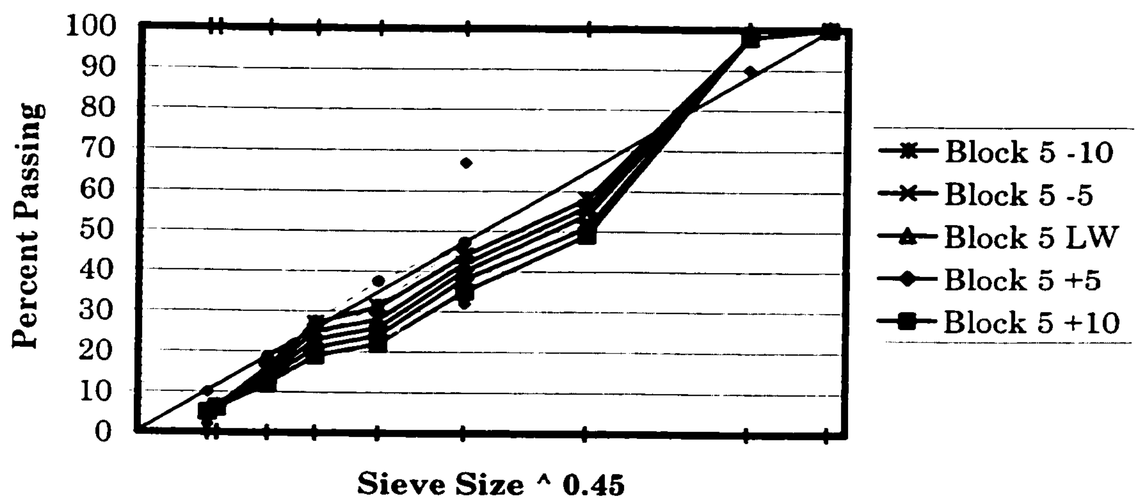


Figure 9-6 Block 5 Standard Gradation Plot

This experiment is very comprehensive in coverage of dense graded asphalt mixtures. The gradations cover the Superpave gradation limits below the maximum density line, with one mixture over the maximum density line. This complete coverage of aggregate gradations below the maximum density line provides an all-inclusive study of aggregate gradation and mixture volumetric properties.

Because the experimental design used one coarse aggregate, one fine aggregate, one mineral filler, and a constant asphalt content many of the volumetric results show identical trends. Analysis of the volumetric results where the trend is duplicated with another volumetric result is not given. Those properties include bulk specific gravity of the combined aggregate, effective specific gravity of the combined aggregate, percentage of absorbed asphalt, percentage of effective asphalt, voids filled with asphalt, dust proportion, surface area factor, and film thickness. These volumetric properties are given in Table 9-5 and Table 9-6.

Table 9-5 Additional Volumetric Properties of Combined Aggregate

Sample Name	Combined Specific Gravity of Aggregate (Gsb)	Effective Specific Gravity of Aggregate (Gse)	Surface Area Factor
Block 1 -10	2.652	2.721	25.5
Block 1 -5	2.656	2.730	24.8
Block 1 LW	2.660	2.740	24.0
Block 1 +5	2.664	2.743	23.3
Block 1 +10	2.668	2.747	22.6
Block 2 -10	2.645	2.732	26.2
Block 2 -5	2.648	2.734	25.5
Block 2 LW	2.652	2.737	24.9
Block 2 +5	2.656	2.742	24.2
Block 2 +10	2.659	2.743	23.5
Block 3 -10	2.658	2.738	24.9
Block 3 -5	2.662	2.742	24.2
Block 3 LW	2.667	2.745	23.5
Block 3 +5	2.671	2.748	22.8
Block 3 +10	2.675	2.759	22.1
Block 4 -10	2.652	2.720	23.2
Block 4 -5	2.656	2.725	22.7
Block 4 LW	2.660	2.730	22.2
Block 4 +5	2.664	2.733	21.7
Block 4 +10	2.668	2.742	21.2
Block 5 -10	2.652	2.734	27.7
Block 5 -5	2.656	2.739	26.7
Block 5 LW	2.660	2.740	25.8
Block 5 +5	2.664	2.742	24.9
Block 5 +10	2.668	2.742	24.0
Block 6 -40	2.625	2.694	30.2

Table 9-6 Additional Volumetric Properties of Asphalt Mixtures

Sample Name	% Absorbed Asphalt (Pba)	% Effective Asphalt (Pbe)	VFA	Dust Proportion	Film Thickness
Block 1 -10	1.0	4.560	68.630	1.1	9.0
Block 1 -5	1.1	4.502	61.726	1.1	9.1
Block 1 LW	1.1	4.428	60.860	1.1	9.2
Block 1 +5	1.1	4.449	58.746	1.1	9.6
Block 1 +10	1.1	4.456	56.989	1.1	9.9
Block 2 -10	1.2	4.328	67.642	1.2	8.3
Block 2 -5	1.2	4.348	68.744	1.2	8.5
Block 2 LW	1.2	4.367	71.289	1.1	8.8
Block 2 +5	1.2	4.354	72.557	1.1	9.0
Block 2 +10	1.2	4.387	69.058	1.1	9.4
Block 3 -10	1.1	4.430	56.152	1.1	8.9
Block 3 -5	1.1	4.442	57.109	1.1	9.2
Block 3 LW	1.1	4.454	60.306	1.1	9.5
Block 3 +5	1.1	4.479	55.243	1.1	9.9
Block 3 +10	1.2	4.390	52.042	1.1	10.0
Block 4 -10	1.0	4.580	67.228	1.1	9.9
Block 4 -5	1.0	4.573	64.807	1.1	10.1
Block 4 LW	1.0	4.562	63.329	1.1	10.3
Block 4 +5	1.0	4.585	60.350	1.1	10.6
Block 4 +10	1.0	4.526	55.141	1.1	10.7
Block 5 -10	1.2	4.402	84.027	1.1	8.0
Block 5 -5	1.2	4.394	82.167	1.1	8.3
Block 5 LW	1.1	4.433	80.514	1.1	8.6
Block 5 +5	1.1	4.471	78.461	1.1	9.0
Block 5 +10	1.0	4.526	72.081	1.1	9.5
Block 6 -40	1.0	4.554	60.796	1.1	7.6

9.1.2 Ratios for Analysis of Aggregate Gradation

Chapter 4 presented concepts for the analysis of aggregate gradations with the use of ratios for the coarse and fine aggregate. These ratios were developed from aggregate packing principles and allow the gradation to be sectioned from the most coarse to the most fine part of the aggregate blend. The ratios calculated for the analysis of gradation are given in Table 9-7. These ratios will be used in later analysis of the volumetric test results and the mechanical property test results.

Table 9-7 Aggregate Ratios for the Evaluation of Aggregate Gradation on Asphalt Mixtures

Sample Name	CA Ratio	FA_c Ratio	FA_f Ratio
Block 1 -10	0.32	0.50	0.28
Block 1 -5	0.30	0.49	0.30
Block 1 LW	0.28	0.48	0.32
Block 1 +5	0.27	0.47	0.34
Block 1 +10	0.26	0.46	0.37
Block 2 -10	0.22	0.56	0.26
Block 2 -5	0.20	0.56	0.28
Block 2 LW	0.19	0.55	0.29
Block 2 +5	0.18	0.55	0.31
Block 2 +10	0.17	0.55	0.33
Block 3 -10	0.48	0.43	0.31
Block 3 -5	0.46	0.41	0.33
Block 3 LW	0.44	0.39	0.36
Block 3 +5	0.43	0.38	0.39
Block 3 +10	0.41	0.36	0.43
Block 4 -10	0.29	0.38	0.37
Block 4 -5	0.28	0.38	0.39
Block 4 LW	0.27	0.38	0.41
Block 4 +5	0.26	0.37	0.44
Block 4 +10	0.25	0.37	0.46
Block 5 -10	0.33	0.61	0.23
Block 5 -5	0.32	0.60	0.24
Block 5 LW	0.30	0.58	0.27
Block 5 +5	0.27	0.55	0.29
Block 5 +10	0.27	0.54	0.32
Block 6 -40	0.49	0.54	0.21

The use of aggregate gradation ratios is explored through this experiment. However, due to the design of experiment a full analysis of the power of the aggregate gradation ratios is not possible, because the method for developing the aggregate gradations produces ratios that are confounded with on another. In all blocks as the design unit weight is increased, the CA ratio decreases, the FAc ratio decreases and the FAf ratio increases. A full evaluation of the aggregate gradation ratios would require a prohibitively enlarged experiment where these values changed independent of each other.

9.1.3 Maximum Specific Gravity (G_{mm})

The maximum specific gravity (G_{mm}) of an asphalt mixture is a fundamental volumetric property. The Gmm is used as the point where the complete volume is taken by aggregate and asphalt and forms the basis for many volumetric properties. Most important of these volumetric properties is the air voids, the percentage difference between the Gmm and the bulk specific gravity (G_{mb}).

It is expected that the trend for all Gmm measurements would be consistent with a change in volume of coarse aggregate. Because the coarse aggregate has a higher specific gravity samples with increased coarse aggregate would have increased Gmm for the mixture. This trend is observed in all of the blocks of the experiment as shown in Table 9-8 and shown in Figure 9-7.

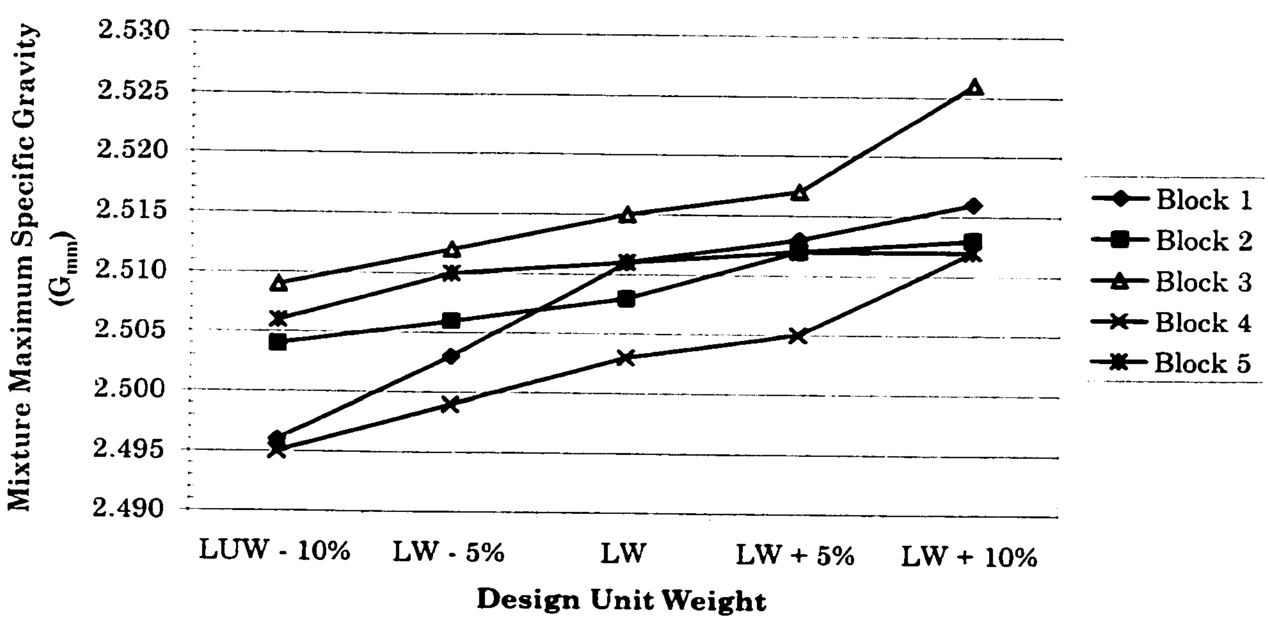


Figure 9-7 Maximum Specific Gravity Plot for Asphalt Mixtures by Experimental Block

Table 9-8 Maximum Specific Gravity (G_{mm}) of Asphalt Mixtures

Sample Name	Mixture Maximum Specific Gravity (G_{mm})
Block 1 -10	2.496
Block 1 -5	2.503
Block 1 LW	2.511
Block 1 +5	2.513
Block 1 +10	2.516
Block 2 -10	2.504
Block 2 -5	2.506
Block 2 LW	2.508
Block 2 +5	2.512
Block 2 +10	2.513
Block 3 -10	2.509
Block 3 -5	2.512
Block 3 LW	2.515
Block 3 +5	2.517
Block 3 +10	2.526
Block 4 -10	2.495
Block 4 -5	2.499
Block 4 LW	2.503
Block 4 +5	2.505
Block 4 +10	2.512
Block 5 -10	2.506
Block 5 -5	2.510
Block 5 LW	2.511
Block 5 +5	2.512
Block 5 +10	2.512
Block 6 -40	2.474

The amount of change in G_{mm} with a change in gradation is important in the quality control of asphalt mixtures. Changes in G_{mm} effect the volumetric properties measured in the lab for quality control, and effect the resulting measured in place density of the compacted pavement. Based on a test precision given by AASHTO of 0.011, results show that a change in coarse aggregate volume of 5% will not produce a change to the mixture G_{mm} that can be measured. In some cases a change in mixture G_{mm} will occur with a change in coarse aggregate volume of 10% or greater, which can be measured.

9.1.4 Bulk Specific Gravity (G_{mb})

The G_{mb} of the compacted asphalt mixtures in this study show the compactability of the mixtures. Because the mixtures have consistent asphalt content, preparation technique, and compactive effort, different G_{mb} indicates different resistance to densification of the mixture. Table 9-9 gives the G_{mb} data for all of the compacted samples, while Figure 9-8 shows this data plotted for each experimental block.

Table 9-9 Mixture Bulk Specific Gravity (G_{mb}) of Asphalt Mixtures

Sample Name	Mixture Bulk Specific Gravity (G_{mb})
Block 1 -10	2.376
Block 1 -5	2.344
Block 1 LW	2.348
Block 1 +5	2.335
Block 1 +10	2.325
Block 2 -10	2.384
Block 2 -5	2.391
Block 2 LW	2.405
Block 2 +5	2.415
Block 2 +10	2.398
Block 3 -10	2.314
Block 3 -5	2.323
Block 3 LW	2.347
Block 3 +5	2.312
Block 3 +10	2.298
Block 4 -10	2.367
Block 4 -5	2.357
Block 4 LW	2.352
Block 4 +5	2.334
Block 4 +10	2.305
Block 5 -10	2.456
Block 5 -5	2.453
Block 5 LW	2.447
Block 5 +5	2.439
Block 5 +10	2.409
Block 6 -40	2.311

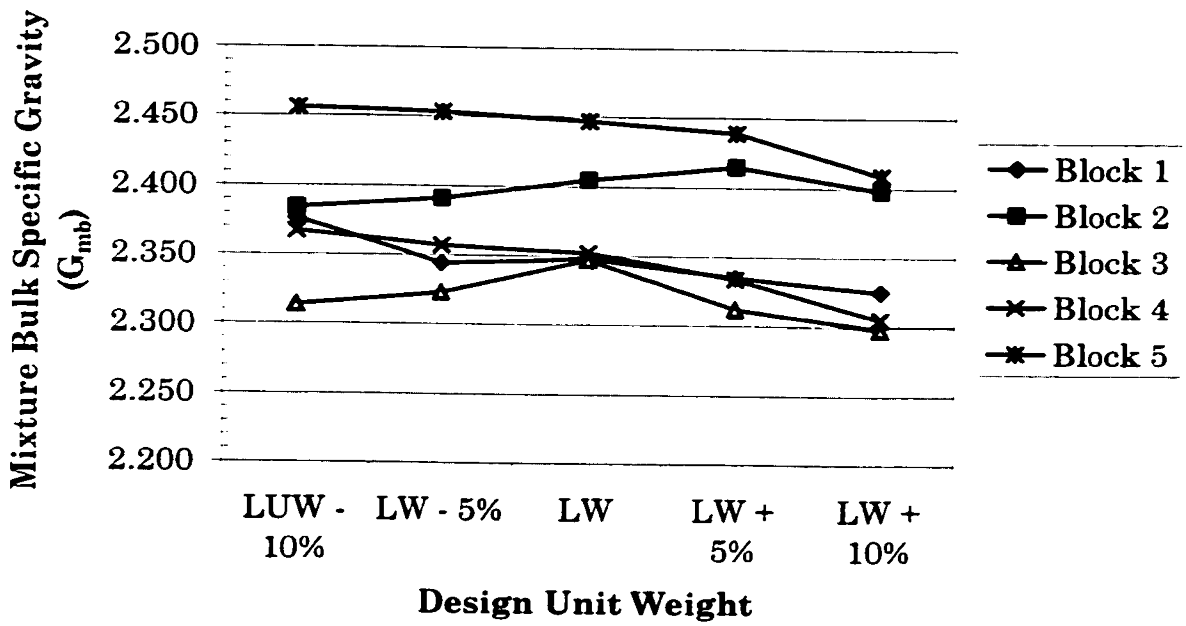


Figure 9-8 Mixture Bulk Specific Gravity (G_{mb}) Plot for Asphalt Mixtures by Experimental Block

Because the G_{mb} and G_{mm} are to calculate the mixture air voids and the air voids are used a mixture design criteria, the analysis of G_{mb} data is given through a discussion of mixture air voids.

9.1.5 Air Voids

The design of asphalt mixtures specifies the air voids in the laboratory compacted sample, therefore the understanding of change in air voids with change in aggregate gradation is essential. The air voids in an asphalt mixture allow void space for expanding asphalt binder during temperature cycling. A properly designed asphalt mixture will contain enough air voids to allow for this expansion of the asphalt. If the total air void structure in a mixture is not sufficient for the total expansion of the asphalt cement the mixture will be lubricated by the additional asphalt binder and will flow. This flow in the asphalt material is seen as rutting.

If the total voids in the asphalt mixture is greater than necessary problems can develop with permeability, stripping, and rutting. With an increasing volume of air voids in the mixture the voids will become interconnected. Interconnected air voids allow water to permeate through the material. With water trapped in the compacted the asphalt mixture can strip, where the asphalt binder is scoured off of the aggregate, leaving a degraded material. This degraded material is considerably weaker than an asphalt pavement and will rut under traffic.

Because air voids is a fundamental property in the design of asphalt mixtures an understanding of the relationship between aggregate gradation and air voids is important in the development of proper mixture designs. The air void data for the 26 mixtures of this experiment are given in Table 9-10. Figure 9-9 gives a plot of the air void data for each experimental block.

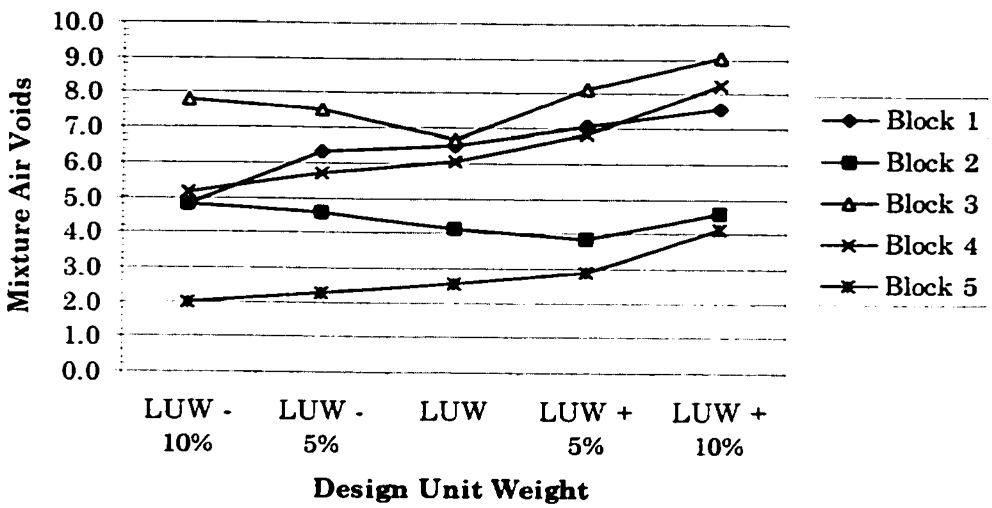


Figure 9-9 Air Void Plot for Asphalt Mixtures by Experimental Block

Table 9-10 Air Voids of Asphalt Mixtures

Sample Name	Air Voids (%)
Block 1 -10	4.8
Block 1 -5	6.4
Block 1 LW	6.5
Block 1 +5	7.1
Block 1 +10	7.6
Block 2 -10	4.8
Block 2 -5	4.6
Block 2 LW	4.1
Block 2 +5	3.9
Block 2 +10	4.6
Block 3 -10	7.8
Block 3 -5	7.5
Block 3 LW	6.7
Block 3 +5	8.1
Block 3 +10	9.0
Block 4 -10	5.1
Block 4 -5	5.7
Block 4 LW	6.0
Block 4 +5	6.8
Block 4 +10	8.2
Block 5 -10	2.0
Block 5 -5	2.3
Block 5 LW	2.5
Block 5 +5	2.9
Block 5 +10	4.1
Block 6 -40	6.6

9.1.5.1 Discussion of CA Volume, Gradation, and Air Voids

The experimental blocks in the testing scheme show a similar, but not identical, trend with an increase in coarse aggregate. Experimental blocks one, four, and five show an increase in air voids for with an increase in design unit weight for all levels of design unit weight. The air voids in block two show a minimum at five percent above the loose unit weight, while the most dense point in block three is when the design unit weight is equal to the loose unit weight.

The change in gradation of the coarse aggregate changes the void structure in the coarse aggregate, which changes the way the coarse aggregate compacts the fine aggregate.

Figure 9-10 shows the change in air voids with an increasing volume of coarse aggregate in block one mixtures. Because the medium gradation of coarse aggregate contains a balanced gradation of coarse aggregate it is the most dense in the uncompacted state. This balance in the coarse gradation allows the coarse aggregate to compact the fine aggregate in the mix. With an increase in coarse aggregate volume the coarse aggregate interlocks and is no longer able to compact the fine aggregate, yielding increased air voids.

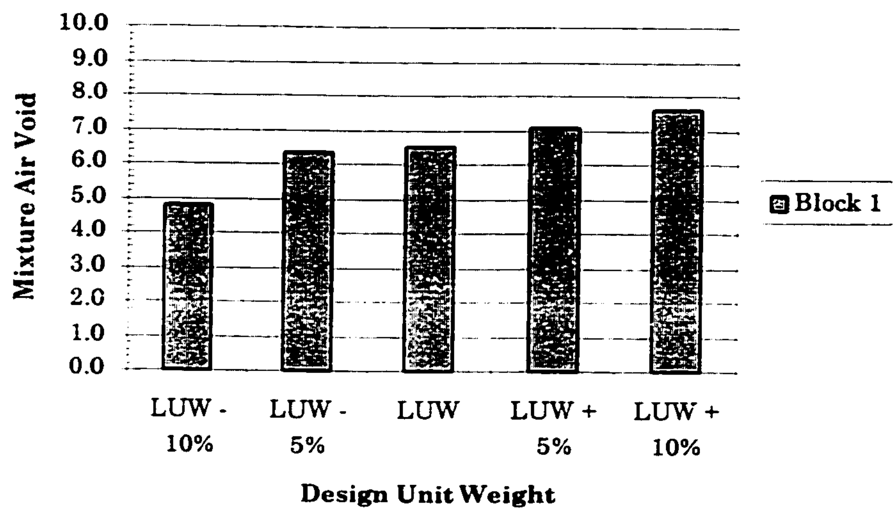


Figure 9-10 Air Void Plot for Experimental Block 1 Mixtures

Block two of the mixture testing uses a coarse gradation of coarse aggregate in the mixtures that produces fewer voids in the coarse aggregate that are larger in size. This larger void size in the coarse aggregate and fewer "interceptor" aggregates give a larger space for the fine aggregate to occupy. Figure 9-11 shows the air void plot for block two. With the larger void space in the coarse aggregate and fewer "interceptor" aggregates it appears that the fine aggregate is compacted because appreciable coarse aggregate interlock develops until the design unit weight is 5 percent above the loose unit weight and the air voids begin to increase, indicating increased resistance to compaction. The coarse gradation of coarse aggregate appears to desensitize air voids to the change in coarse aggregate volume.

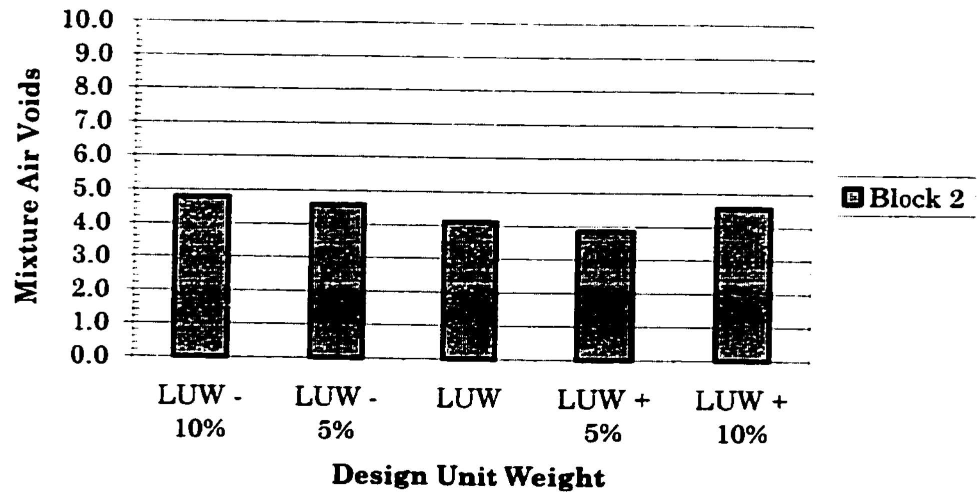


Figure 9-11 Air Void Plot for Experimental Block 2 Mixtures

Using a fine gradation of coarse aggregate, in Block 3, the amount of "interceptor" aggregates is increases, which increased the voids in the coarse aggregate. This increase in coarse aggregate voids is noticed in the increased voids in the mixture. The average voids for block one is 6.5% and the average for block 3 is 7.8%. Figure 9-12 shows the air void plot for block three. The change in volume of coarse aggregate for block three produces a more sensitive response with air voids than is seen in other blocks. The change in air voids from the loose unit weight to the rodded unit weight is 2.3%. A fine gradation of coarse aggregate appears to accentuate the change in coarse aggregate volume when analyzed by the resulting air voids.

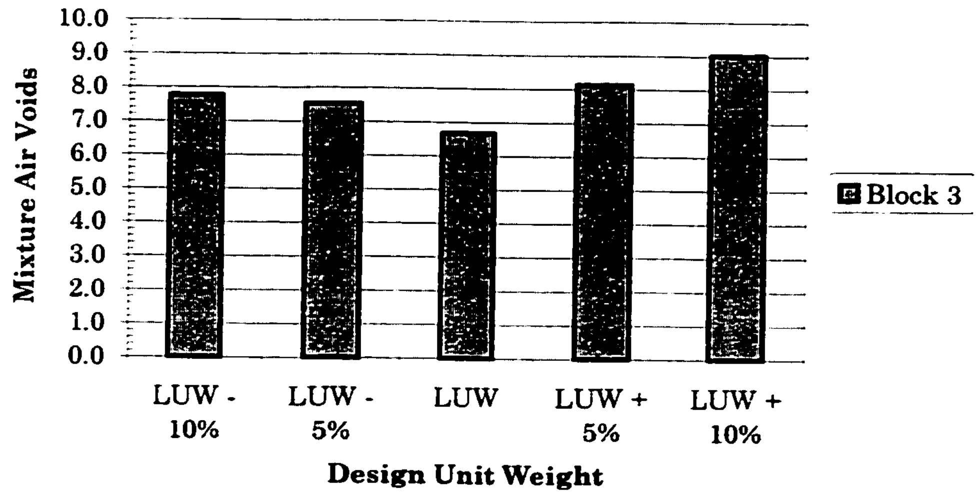


Figure 9-12 Air Void Plot for Experimental Block 2 Mixtures

Blocks one, four and five are used in comparison of fine aggregate gradation, and show a similar trend in change in air voids with a change in coarse aggregate volume. Figure 9-13 shows the air void data from block one, four, and five with a trend line for each block. The trend lines for these blocks are approximately parallel, indicating a similar trend in the response to change in coarse aggregate volume.

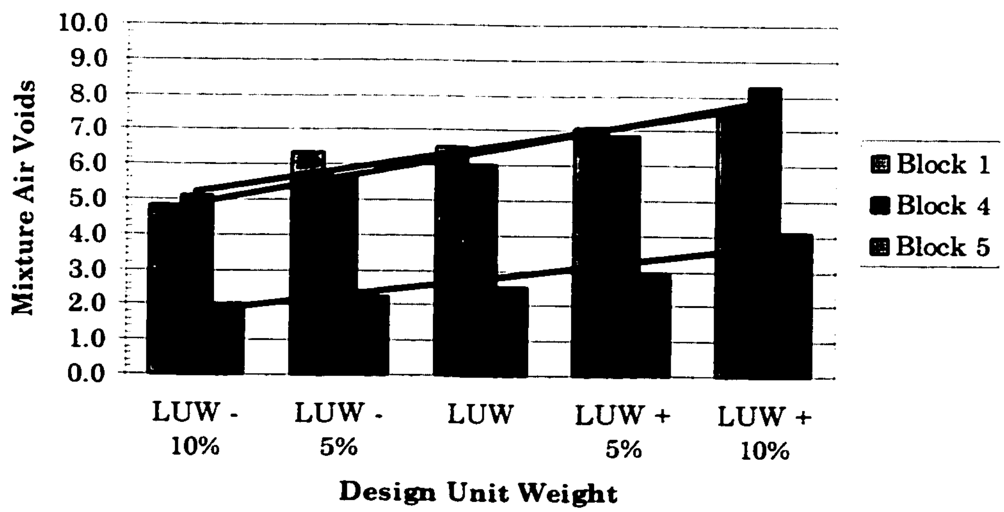


Figure 9-13 Air Void Plot for Experimental Blocks 1, 4, and 5 Mixtures

The coarse gradation of fine aggregate has increased voids in the fine aggregate structure and provide more resistance to deformation than a fine gradation of fine aggregate. Similar to the results of block one, the influence of volume of coarse aggregate is observed by an increase in air voids with increased volume of coarse aggregate. The coarse gradation of fine aggregate compacts to give similar voids to the medium gradation of fine aggregate.

The use of a fine gradation of fine aggregate also shows the same trend as the medium and coarse gradation with a change in coarse aggregate volume, however the resulting voids in the mixture are considerably decreased. The fine gradation of fine aggregate packs with increased density in the asphalt mixture, giving decreased air voids in the mixture.

An analysis of variance (ANOVA) was performed to understand the effect of changing coarse aggregate gradation, fine aggregate gradation, and volume of coarse aggregate on the resulting air voids of the mixture. The results from that ANOVA are given in Table 9-11. The ANOVA shows, through the Pr > F less than 0.01, that statistical differences in the air voids exist with a change in gradation of the component aggregates or a change in volume of coarse aggregate. Changing the gradation of the fine aggregate has the largest effect on changing air voids, with the change of coarse aggregate gradation and change in design unit weight as also significant. The relevant effect of each of the treatments is seen through the magnitude of the F value.

Least square difference (LSD) analysis of the data set is given in Table 9-12. This LSD provides a t-grouping for the mixtures with change in aggregate gradation and design unit weight. The changing of design unit weight gives the LUW –5% and LUW +5% to be the same, the LUW and LUW –10% to be the same grouping, however they are different from one another and from LUW +10%. The coarse, medium, and fine aggregate gradations give different air voids for both the coarse and fine aggregate.

Table 9-11 ANOVA for Air Voids

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	24	306.5347171	12.7722799	100.37	0.0001
Error	63	8.0168283	0.1272512		
Corrected Total	87	314.5515455			

R-Square	C.V.	Root MSE	VOIDS Mean
0.974513	6.367468	0.356723	5.602273

Source	DF	Type I SS	Mean Square	F Value	Pr > F
DESIGN	4	38.5942203	9.6485551	75.82	0.0001
CA	2	100.2148284	50.1074142	393.77	0.0001
FA	2	147.6909422	73.8454711	580.31	0.0001
DESIGN*CA	8	16.5934168	2.0741771	16.30	0.0001
DESIGN*FA	8	3.4413094	0.4301637	3.38	0.0027

Table 9-12 Least Square Difference T-Grouping for Air Voids

Design Unit Weight

T Grouping	Mean	N	DUW
A	6.7458	19	10
B	5.7282	17	5
B	5.4873	15	-5
C	5.0322	18	0
C	4.9768	19	-10

Coarse Aggregate

T Grouping	Mean	N	CA
A	7.5633	18	Fine
B	5.3702	51	Medium
C	4.3674	19	Coarse

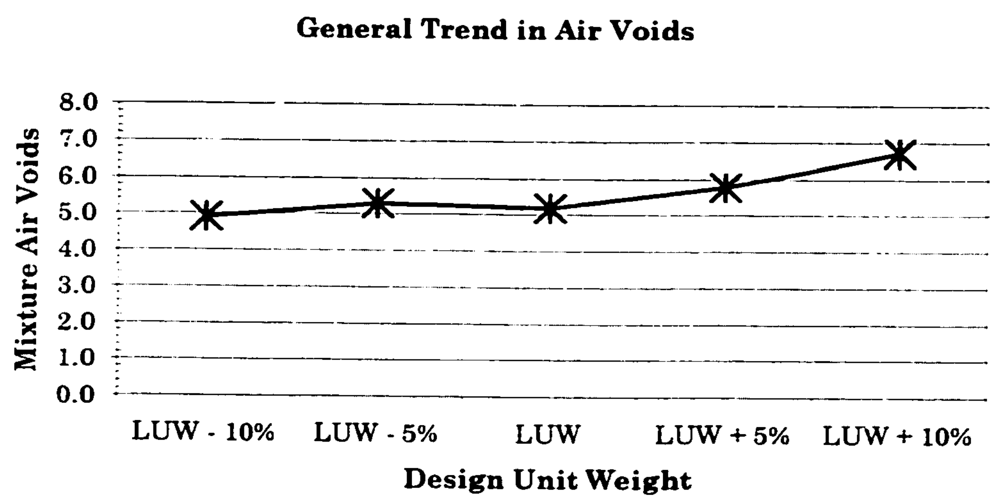
Fine Aggregate

T Grouping	Mean	N	FA
A	6.4650	16	Coarse
B	6.1380	56	Medium
C	2.8644	16	Fine

The ANOVA analysis of the entire data set shows the variables that effect the air voids in an asphalt mixture. This analysis is taken on the entire data set, in order to understand the overall trend. Additional analysis, through the use of ANOVA, of the points from Block one, two, and three give the same result as the overall ANOVA. An ANOVA on the points from Block one, four and five also gives the same result as the overall ANOVA.

Taking the mean values from the ANOVA the general trend in air voids is established for a change in design unit weight. This general trend is shown in Figure 9-14. The general trend shows that with an increase in volume of coarse aggregate the air voids in the mixture increase. This increase in voids is derived from aggregate interlock. With an increased volume of coarse aggregate the large aggregate particles interlock and resist deformation, thereby giving increased voids in the mixture.

Figure 9-14 General Trend in Air Voids for Asphalt Mixtures



9.1.5.2 Discussion of Gradation Analysis and Air Voids

The use of ratios for the analysis of gradation is proposed as a method for understanding the aggregate packing and resulting voids. A multiple regression was performed using the aggregate ratios to fit a model with the resulting air voids. All 26 mixtures were used in the creation of the model. A model using only the CA ratio, FAc, and FAf produced a model with an R-Square value of 0.69. While this signifies a good fit improvements in the model were evaluated. An additional model was developed using the

CA ratio, FAc, FAf, and the square of these values. This model describes the air voids with an R-Square of 0.87 and Standard Error of 0.80. The predicted air voids and measured air voids from this improved model are shown in Figure 9-15. The model relating the aggregate ratios to the air voids is given in Equation 9-1.

Equation 9-1 Model Relating Aggregate Ratios to Mixture Air Voids

$$\text{Air Voids} = -31.9 + 35.5\ CA^2 - 14.3\ CA - 199.6\ FA_c^2 + 187.1\ FA_c + 98.66\ FA_f^2 - 45.4\ FA_f$$

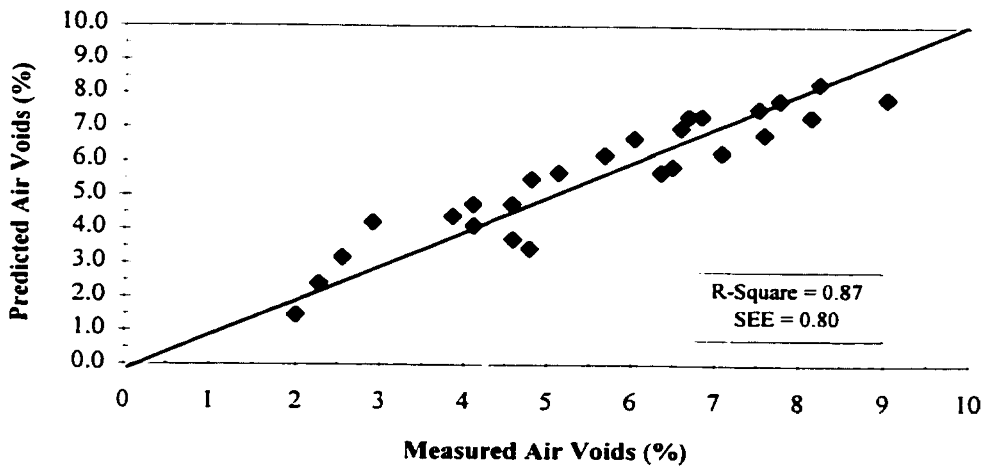


Figure 9-15 Measured and Predicted Air Voids Using Aggregate Ratio Model

The model was created using a multiple regression optimization routine that will display the order that variables were allowed into the model. From the original and improved models the FAc ratio had the highest correlation with the resulting air voids. The coarse portion of the fine aggregate appears to have the most influence on the resulting air voids. The coarse aggregate ratio has the second greatest influence on the resulting voids in the mixture with the fine portion of the fine aggregate having the least influence on voids in the resulting mixture.

9.1.6 Voids in the Mineral Aggregate (VMA)

Voids in the Mineral Aggregate are desired in an asphalt mixture to ensure durability. A requirement for adequate VMA will ensure that a mixture contains the asphalt binder content necessary to form a thick film of asphalt over the aggregate particles that will resist environmental effects and create a durable material.

The calculation of VMA is based on the mixture bulk specific gravity (Gmb), the percentage of asphalt binder in the mixture, and the bulk specific gravity of the aggregate in the mixture. Because the percentage of asphalt binder in the mixture was held constant in the experiment the VMA shows the same trend as the air voids. The factors that change the air voids are the same factors that change the VMA of these mixtures because of the design of the experiment.

9.1.6.1 Discussion of Gradation Analysis and VMA

A multiple regression analysis was used to develop a model that predicted the VMA in a mixture based on the aggregate ratios. The simple regression with the ratios yielded a ratio with an R-Square of 0.79. The improved model, including the squared values of the aggregate ratios, yielded an improved model with an R-Square of 0.91 and SEE of 0.63. This model is given in Equation 9-2 and the actual and predicted values are plotted in Figure 9-16.

Equation 9-2 Model Relating Aggregate Ratios to Mixture VMA

$$VMA = -24.6 + 20.1\ CA^2 - 3.8\ CA - 191.6\ FA_c^2 + 181.0\ FA_c + 87.3\ FA_f^2 - 36.6\ FA_f$$

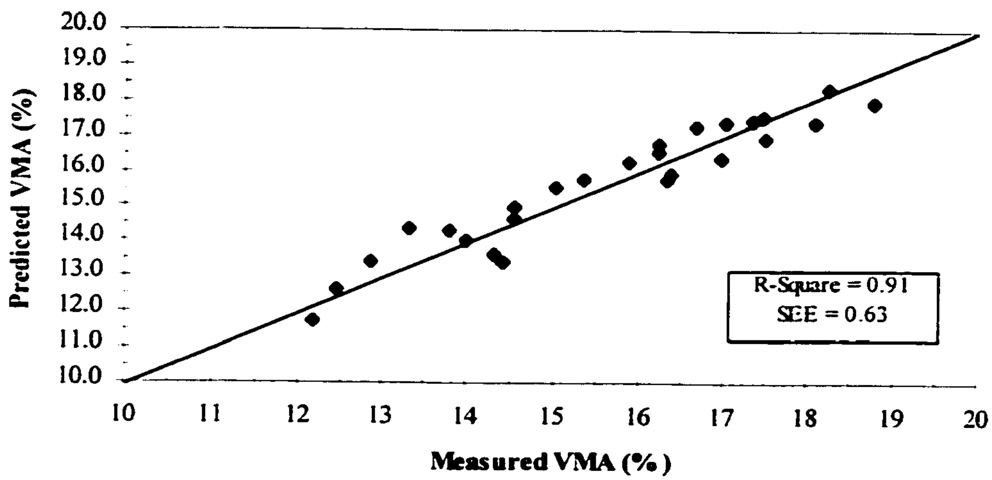


Figure 9-16 Measured and Predicted VMA Using Aggregate Ratio Model

Although the data set used in the generation of this model is not comprehensive in the independent changing of all of the aggregate ratios, the resulting model is appropriate for the prediction of VMA with the combination of the given aggregates.

9.1.7 Voids in the Coarse Aggregate (VCA)

Voids in the coarse aggregate of the mixture (VCA_{mix}) are determined for the coarse potion of the aggregate blend as a property that identifies the existence of a coarse aggregate skeleton with stone-on-stone contact. A coarse aggregate skeleton is developed when the VCA_{mix} in the mixture is equal to or less than the VCA of the coarse aggregate in the loose unit weight configuration. It is used as a design criteria in SMA mixtures as the identifier of coarse aggregate skeleton when compared with the rodded unit weight of coarse aggregate.

9.1.7.1 *Discussion of Gradation Analysis and VCA*

The use of aggregate ratios to understand the packing of aggregates can be extended to predict the VCA. A model was developed in the same manor as for the VMA or air voids to predict the VCA_{mix} in the compacted sample. All 26 mixtures were included in the prediction model including the outlier point, which is a fine graded mixture above the maximum density line.

The predictive model developed from the ratios accurately describes the coarse aggregate skeleton as characterized by the VCA_{mix}. The model gives an R-Square value of 0.98 with a standard error of the estimate of 0.67. The model is given in Equation 9-3 and predicted vs. actual from the model is plotted in Figure 9-17.

Equation 9-3 Model Relating Aggregate Ratios to Voids in the Coarse Aggregate

$$VCA_{mix} = 169.6 + 59.0\ CA^2 - 41.5\ CA - 110.2\ FA_c^2 + 3.2\ FA_c + 428.3\ FA_f^2 - 423.3\ FA_f$$

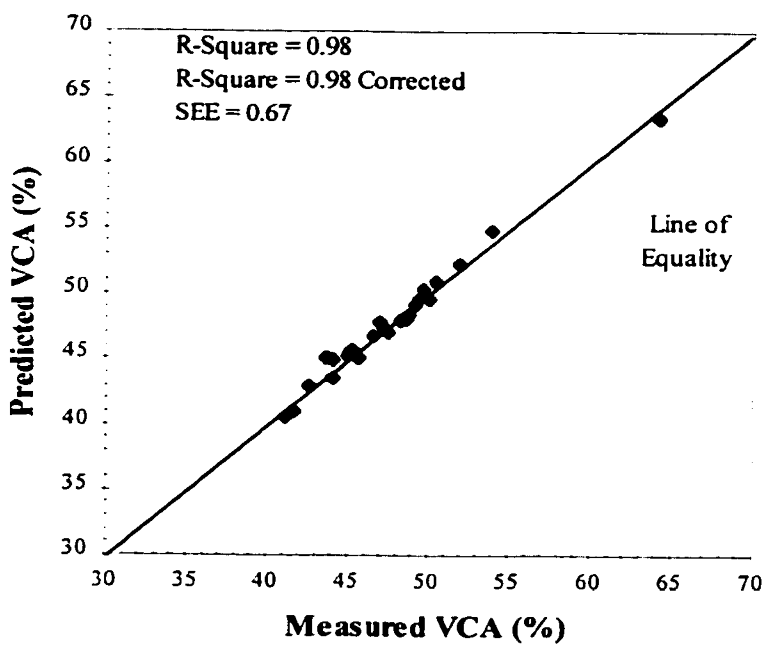


Figure 9-17 Measured and Predicted VCAmix Using Aggregate Ratio Model

9.2 SUMMARY OF VOLUMETRIC TEST RESULTS

Voids in an asphalt mixture, which are fundamental in mixture design, are highly influenced by changes in the volume percentage of coarse aggregate in the mixture. With mixtures controlled by the concepts presented in this study, this volume change in aggregate can be designed by changing the design unit weight of the coarse aggregate in the mixture. For mixtures with a design unit weight at or above the loose unit weight the coarse aggregate dominates the development of voids in the mixture. For mixtures with a design unit weight below the loose unit weight the fine aggregate becomes dominate in the aggregate structure and the resulting voids are dependent on the fine aggregate structure.

Changing the gradation of the coarse aggregate changes the size of the voids in the coarse aggregate, in turn effecting the resulting voids in the mixture. Mixtures with a coarse gradation of coarse aggregate, indicated by a low CA ratio, are able to provide a larger void space in the coarse aggregate. The fine aggregate in this void space is compacted by the coarse aggregate more than mixtures with a higher CA ratio. Mixtures with a high CA ratio contain interceptor size aggregates. These interceptors change the void structure in the coarse aggregate and lead to less compaction of the fine aggregate by the coarse aggregate.

The gradation of the fine aggregate will change the resulting voids in the mixture as the compactability of the fine aggregate changes. Since the coarse aggregate provides the aggregate skeleton when mixtures are designed according to the concepts presented in this study, the fine aggregate will compact in a similar manor regardless of its gradation. The change in fine aggregate gradation changes the degree to which the fine aggregate compacts, thereby changing the resulting air voids.

The ratios used for the analysis of gradation are improved tools for understanding the voids in an aggregate mixture. In dense graded asphalt mixtures changes in the FAc ratio have the largest effect on the resulting air voids. The CA ratio can be used to evaluate the coarse aggregate structure, which has been shown to change the voids in a mixture. The FAf ratio is also believed to change the mixture volumetrics, however because of the design of experiment the sensitivity of the FAf can not be observed in this data set.

Based on the data from this experiment it is possible to create an adequate prediction model, based on the aggregate ratios, to describe the resulting air voids and VMA in the mixture. This model is limited by the data set used to create it, but shows promise for expansion to the larger set of asphalt mixtures. The tools provided by the aggregate ratios improve the state of the art in analysis of aggregate gradation for asphalt mixtures.

9.3 Gyratory Compaction Results

The resistance to compaction of these asphalt mixtures is an indicator of the strength of the aggregate structure. The slope of the densification curve is used as an indicator of the aggregate skeleton strength. The locking point of these mixtures provides an indication of the beginning of over compaction in the SGC process. The development of shear stress in the SGC provides a true measure of the resistance of the aggregate structure to compaction.

The Results for the compaction slope, locking point, shear stress slope, and maximum shear stress are given in Table 9-13.

Table 9-13 Gyratory Compaction Test Results for Mixtures

Sample Name	Locking Point	Compaction Slope	Shear Stress Slope	Max Shear Stress
Block 1 -10	81.0	7.7	4.4	390
Block 1 -5	79.0	7.4	3.9	378
Block 1 LW	84.0	7.9	3.6	381
Block 1 +5	92.0	8.3	4.4	376
Block 1 +10	93.0	8.7	5.4	381
Block 2 -10	80.0	6.6	3.5	389
Block 2 -5	90.0	7.0	4.5	375
Block 2 LW	97.0	7.8	3.9	374
Block 2 +5	108.0	8.2	5.0	375
Block 2 +10	111.0	9.0	4.9	376
Block 3 -10	N/A	8.0	4.7	371
Block 3 -5	N/A	8.1	4.3	366
Block 3 LW	N/A	8.8	4.9	377
Block 3 +5	N/A	8.7	4.4	365
Block 3 +10	N/A	8.9	4.9	373
Block 4 -10	104.0	7.8	4.3	379
Block 4 -5	99.0	8.1	4.5	373
Block 4 LW	104.0	8.3	4.9	374
Block 4 +5	109.0	8.5	4.7	369
Block 4 +10	108.0	8.7	4.3	363
Block 5 -10	94.0	7.4	2.1	375
Block 5 -5	92.0	8.3	3.7	378
Block 5 LW	99.0	8.9	4.3	377
Block 5 +5	105.0	9.5	5.3	374
Block 5 +10	N/A	9.6	5.3	371
Block 6 -40	73.0	5.7	2.9	363

9.3.1 Compaction Slope

The compaction slope can be used to evaluate the development of aggregate structure in the SGC. Strong aggregate structure will resist deformation, which will be evident by a large value in compaction slope. Mixtures which easily densify will show a low value of compaction slope.

The compaction slope is the rate change in densification during compaction in the SGC. It is a log-log slope of the density taken from the fifth gyration to the end of the gyratory compaction. The computation of compaction slope in this study uses the densification data for all points to the locking point of the mixture. After the locking point of the mixture the slope of the densification curve changes, therefore using gyrations after this point will skew the result. The compaction data is given for all data points in Table 9-13 and is shown in Figure 9-18.

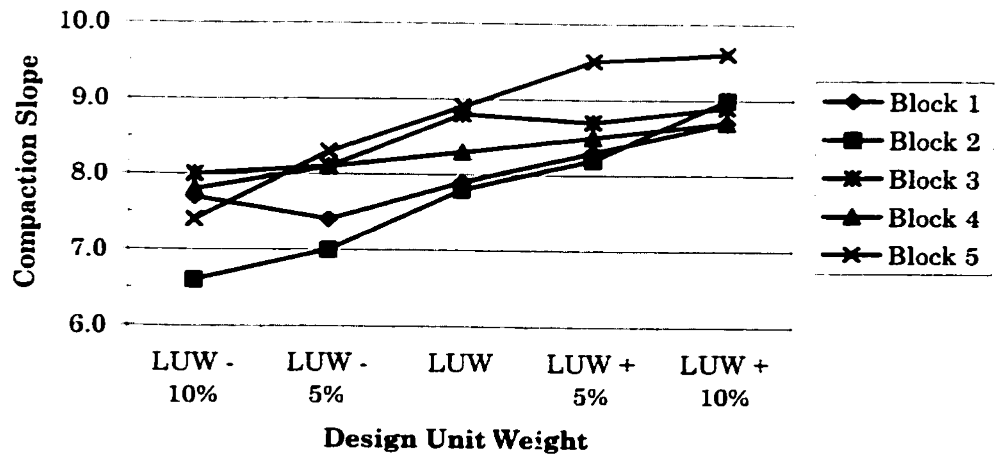


Figure 9-18 Compaction Slope for Asphalt Mixtures

The compaction slope is highly dependent on the design unit weight used for the design of the asphalt mixture. With increased design unit weight, increased volume of coarse aggregate, the compaction slope increases. Figure 9-19 shows the compaction slope from the block six point and the block one points; block one and block six materials are from the same component aggregates and have different design unit weights. The increase in coarse aggregate volume causes an increase in the compaction slope.

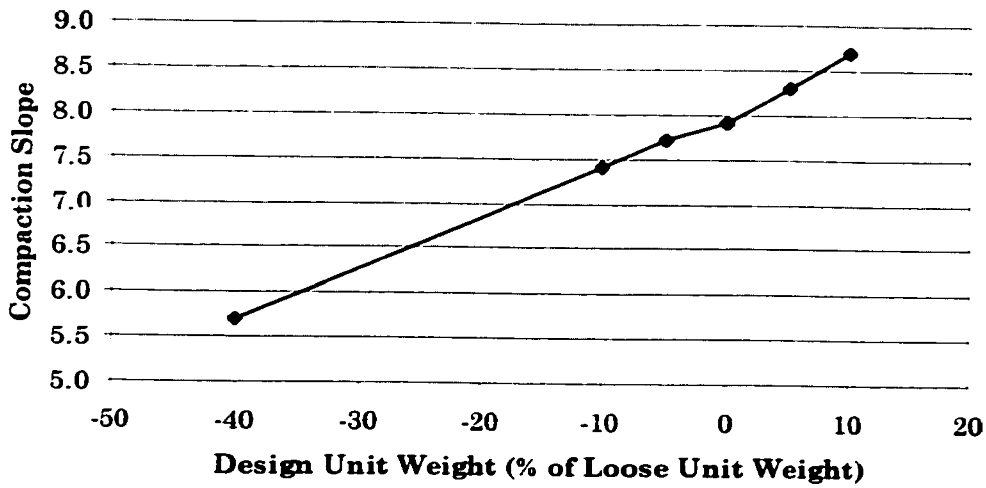


Figure 9-19 Compaction Slope Plot for Changing Volume of Coarse Aggregate

The change in gradation of the coarse or fine aggregate has no definite effect on the compaction slope. Changes in coarse aggregate void structure from blocks 1, 2, and 3 do not reflect a change in the compaction slope. The gradation of the fine aggregate, as observed through blocks 1, 4, and 5, does not predictably change the compaction slope.

Aggregate ratios for analysis of aggregate gradation provide a tool for the prediction of voids in an asphalt mixture through an indication of the aggregate structure. The compaction slope in the SGC is also used as an indicator of the aggregate structure and aggregate packing. Using the aggregate ratios to predict the compaction slope gives a model, shown in Figure 9-20, that is adequate in predicting the locking point of a mixture in the SGC with an R-Square of 0.85 and SEE of 0.38. The equation for this model is given in Equation 9-4.

Equation 9-4 Model Relating Aggregate Ratios to Compaction Slope

$$\text{Comp. Slope} = -31.9 + 35.5\ CA^2 - 14.3\ CA - 199.6\ FA_c^2 + 187.1\ FA_c + 98.7\ FA_f^2 - 45.4\ FA_f$$

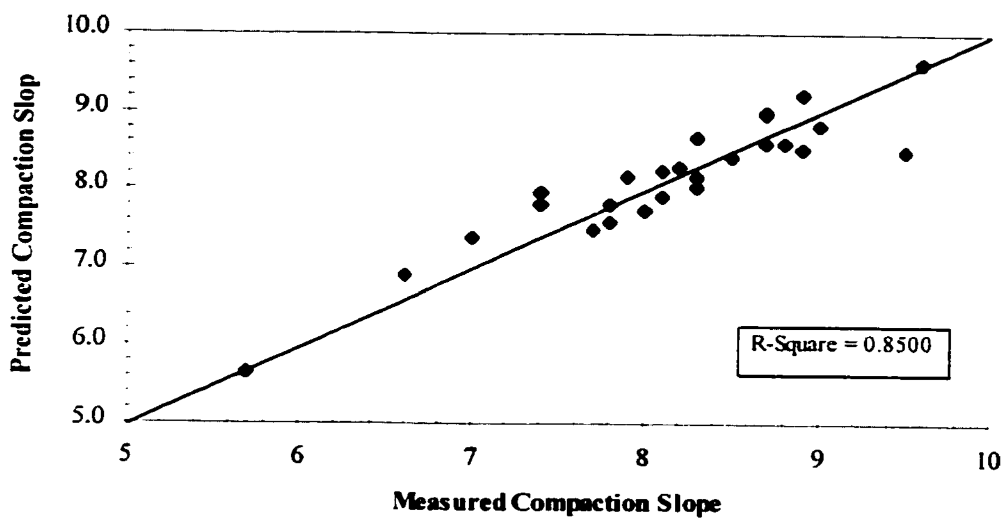


Figure 9-20 Measured and Predicted Compaction Slope Using Aggregate Ratio Prediction

9.3.2 Locking Point

The locking point of an asphalt mixture in the SGC is proposed as the point in the compaction curve where the aggregate structure begins to develop and resist compaction. This point is proposed as a point in the compaction curve that relates to a specific decrease in the change in height in the SGC that is common to all mixtures. The objective of defining the locking point is to provide an indication of where over compacting begins in the SGC.

The locking point data from this study shows similar tends to the developing air voids in the SGC during compaction. The increase in coarse aggregate volume results in an increase in the locking point for the mixture. Figure 9-21 shows the locking point for all of the mixtures where it was measured. Because additional samples were required to determine the locking point of the mixtures, experimental block three does not have locking point information; all samples in block three were compacted to 75 gyrations, and the locking point is above 75 gyrations for all of these mixtures.

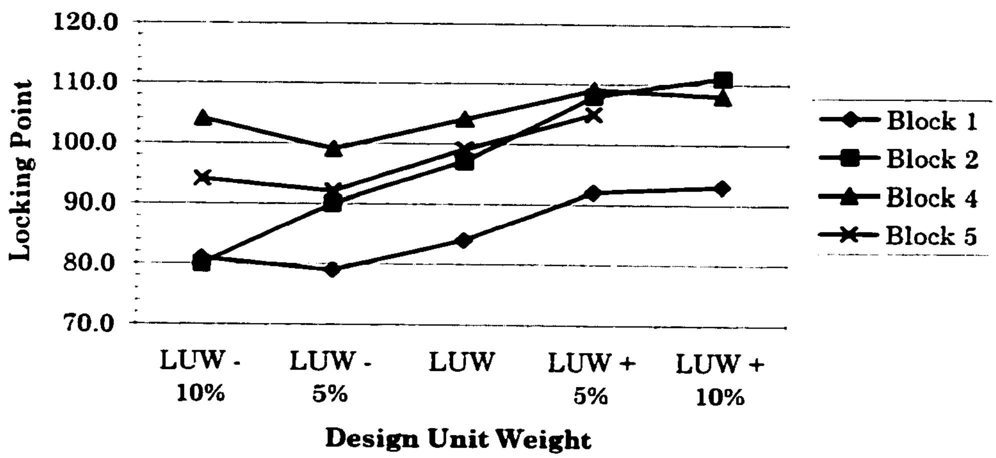


Figure 9-21 Locking Point Plot for All Asphalt Mixtures

The locking point in an asphalt mixture is primarily related to the volume of coarse aggregate in the mixture. By examining the samples from experimental block one and the mixture from experimental block six a large sweep in coarse aggregate volume is observed, Figure 9-22. This plot illustrates how a change in design unit weight changes the locking point of a mixture. The change in locking point when the design unit weight is below the loose unit weight is not large, while design unit weight above the loose unit weight changes the locking point considerably. The development of coarse aggregate interlock to create a coarse aggregate skeleton can be captured by examining the locking point of the mixture.

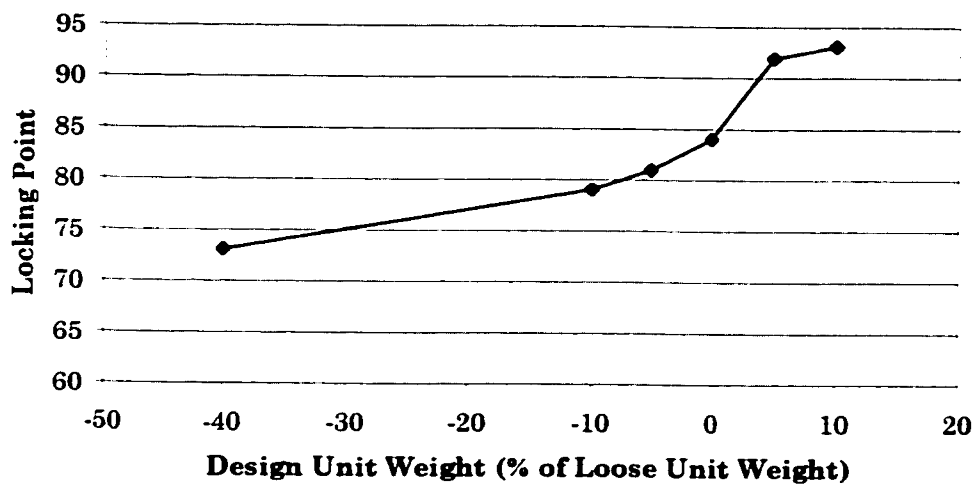


Figure 9-22 Locking Point Plot for Changing Volume of Coarse Aggregate

The change in gradation of the coarse aggregate appears to have a small effect on the locking point. With a coarse gradation of coarse aggregate, low CA ratio, the locking point is very sensitive to changes in design unit weight. Because in these low CA ratio mixtures the coarse aggregate compacts the fine aggregate effectively the locking point appears to be an indicator of the coarse aggregate skeleton. The shape of curve for changing design unit weight is a function of the gradation of the coarse aggregate or CA ratio.

The trend in locking point with change in design unit weight does not change with a change in the gradation of the fine aggregate. However, as the fine aggregate gradation changes the value of the locking point also changes. Because the coarse and fine gradation of fine aggregate are similar in locking point and the medium gradation is lower, the effect of change in fine aggregate gradation is not clear. Further study, which allows the fine aggregate structure to vary will provide insight into the relation of locking point to fine aggregate gradation.

The aggregate ratios for analysis of aggregate gradation provide a tool for the prediction of voids in an asphalt mixture. This prediction is possible because the aggregate ratios indicate the aggregate structure and aggregate packing of the mixture. The locking point in the SGC is also used as an indicator of the aggregate structure and aggregate packing. Using the aggregate ratios to predict the locking point gives a model, shown in Figure 9-23, that is adequate in predicting the locking point of a mixture in the SGC with an R-Square of 0.806 and SEE of 6.2. Equaiton XX gives the model.

Equation 9-5 Model Relating Aggregate Ratios to Locking Point

$$L.P. = 50.1 + 526.3\ CA^2 - 240.8\ CA + 1579.2\ FA_c^2 - 1283.7\ FA_c - 1734.3\ FA_f^2 - 1543.0\ FA_f$$

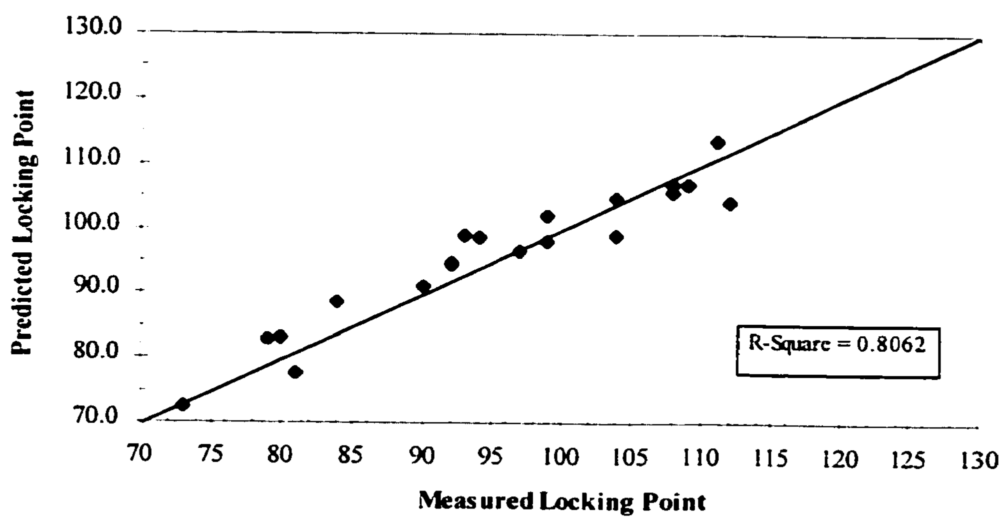


Figure 9-23 Measured and Predicted Locking Point Using Aggregate Ratio Prediction

The typical design of asphalt mixture using the concepts outlined in this study will have a design unit weight within five percent of the loose unit weight. For these typical mixtures the locking point can be most effected by a change in the design unit weight. A mixture design that reaches a stable void structure at the point in compaction where the aggregate is packed but not over compacted is desired. Knowing that the locking point can be changed with a change in design unit weight allows the design to change the aggregate blend to meet the volumetric properties at the desired locking point.

9.3.3 Shear Stress Development During Compaction

Shear stress data was collected for all mixtures through use of the IPC Servopac SGC. This shear stress is measured during compaction as the resistance of the asphalt mixture to the shearing forces applied by the SGC. Testing of designed asphalt mixtures in the IPC Servopac SGC have shown that asphalt mixtures which perform well display increased resistance to shear stress in the SGC. Data collected for the development of shear stress, including the shear stress slope and the maximum shear stress, in the SGC is given in Table 9-13.

No relationship can be derived between the development of shear stress in the SGC and the volumetric properties or the locking point. The maximum shear stress for all of the samples occurred near 75 gyrations, where compaction was stopped. For samples that were tested beyond 75 gyrations the shear stress increased for all gyrations past 75 with no more than 3 kPa. of increased shear stress for any sample. No break-over was observed in the shear stress measurement of any sample.

The shear stress slope was computed for each of the samples to compare with the compaction slope with the result that no trend is noticed between these two values. The shear stress slope has no correlation with any volumetric property or compaction property.

9.4 Summary of Gyratory Compaction Results

Measurements taken during the compaction of asphalt mixtures in the SGC can indicate the strength of the aggregate structure. Coarse aggregate interlock provides a strong aggregate skeleton that resists deformation. The height data collected in the SGC can be interpreted to show the strength of an aggregate blend in an asphalt mixture.

The compaction slope can be used to evaluate aggregate structure and aggregate interlock. The change in coarse aggregate volume has a direct effect on the compaction slope from the SGC. Changes in the gradation of the coarse and fine aggregate in a mixture will effect the resulting compaction slope. The compaction slope appears to be an indicator of the aggregate structure developed in the gyratory compactor, independent of the resulting air voids in the mixture.

The locking point can be used to evaluate the aggregate structure for asphalt mixtures compacted in the SGC. The locking point is an indicator of the volume of coarse aggregate and resulting aggregate structure in an asphalt mixture. Mixtures with high volume of coarse aggregate will have high locking points. The locking point appears as an improved tool over the compaction slope in that it not only provides an indicator of the rate of densification, but also provides a point in the compaction where the aggregates lock together. From field experience with mixtures designed to provide coarse aggregate interlock, the maximum density achieved under normal construction procedures is comparable to the density achieved in the SGC at the locking point.

The use of shear stress measurements to validate the locking point is not promising with the asphalt mixtures used in this study. Shear stress measurements taken during the compaction of these mixtures do not appear to relate to the volumetric properties of the resulting mixtures. The asphalt mixtures in this study did not show a significant decrease in shear resistance with as many as 125 gyrations in the SGC. Validation of the locking point with shear resistance measurement is not achievable with the mixtures in this study.

CHAPTER 10 MECHANICAL PROPERTY TEST RESULTS AND DISCUSSION

The mechanical properties of an asphalt mixture will indicate the performance of the mixture under traffic. The measurement of the engineering properties, including strength and modulus, is used in evaluation of material properties and mechanistic pavement design. The measurement of engineering properties can also be used to determine indices that can rank the performance of asphalt materials.

A testing scheme, as outlined in Chapter 5, was proposed to investigate the mechanical properties of the asphalt mixtures. This testing scheme includes triaxial, shear, and resilient modulus testing. The mechanical property testing is to determine the effect of changing the volume of coarse aggregate and the gradation of the coarse and fine aggregate on the mechanical properties of the resulting mixtures.

10.1 Rapid Triaxial Test Results

Rapid triaxial testing (RaTT) was performed on all asphalt mixtures. Two of the SGC compacted test samples were tested at 50 °C in the IPC QC/QA triaxial test apparatus. A frequency sweep of 10, 5, 2, 1, and 0.1 Hz was performed at a stress state that induces a stress reversal in the sample during testing. The modulus and Poisson's ratio for compression and extension are given in Table 10-1.

Table 10-1 Modulus and Poisson's Ratio in Compression and Extension for Asphalt Mixtures at 2 Hz.

	Modulus (kPa)		Poisson's Ratio		
Sample Name	Compression	Extension	Compression	Extension	Phase Angle (deg)
Block 1 -10	476	397	0.35	0.36	30.2
Block 1 -5	435	351	0.29	0.23	29.3
Block 1 LW	360	277	0.32	0.25	30.9
Block 1 +5	308	223	0.40	0.30	32.7
Block 1 +10	370	279	0.34	0.26	32.2
Block 2 -10	385	291	0.34	0.25	29.9
Block 2 -5	362	278	0.34	0.27	31.3
Block 2 LW	337	236	0.36	0.25	32.4
Block 2 +5	371	276	0.36	0.27	32.0
Block 2 +10	391	302	0.37	0.28	32.1
Block 3 -10	419	327	0.31	0.24	29.9
Block 3 -5	402	320	0.29	0.24	29.6
Block 3 LW	367	282	0.32	0.26	30.8
Block 3 +5	328	237	0.35	0.27	31.9
Block 3 +10	338	247	0.35	0.28	31.3
Block 4 -10	362	276	0.31	0.25	31.4
Block 4 -5	368	281	0.35	0.28	31.4
Block 4 LW	337	245	0.39	0.30	32.2
Block 4 +5	333	238	0.36	0.29	31.2
Block 4 +10	342	244	0.29	0.24	29.1
Block 5 -10	322	202	0.27	0.19	28.6
Block 5 -5	291	195	0.34	0.24	31.8
Block 5 LW	327	213	0.32	0.20	30.4
Block 5 +5	282	199	0.37	0.25	33.5
Block 5 +10	336	198	0.31	0.18	31.4
Block 6 -40	404	321	0.24	0.19	27.0

Using the modulus, slope, and intercept of the sample deformation in extension and compression the rut index proposed by Carpenter is calculated. The equation for this rutting index is given in Equation 10-1 and the resulting rutting index is given in Table 10-2 for the tested asphalt mixtures. All values used in this rutting index are from the testing at 2 Hz.

Equation 10-1 Rutting Index Equation from RaTT Test Data

$$\begin{aligned}\text{Rut Index} = &-15.149 - 147.9\,(\text{Lower Slope}) + 0.0936(\text{Lower Intercept}) + 0.0248(\text{Upper Intercept})\\ &- 0.131(\text{Crossover Intercept}) + 0.0289(\text{Compressive Modulus})\end{aligned}$$

Table 10-2 Rut Index for Asphalt Mixtures

Sample Name	RaTT Rut Index
Block 1 -10	5.3
Block 1 -5	4.7
Block 1 LW	4.2
Block 1 +5	3.2
Block 1 +10	3.5
Block 2 -10	4.4
Block 2 -5	4.1
Block 2 LW	3.3
Block 2 +5	4.6
Block 2 +10	4.6
Block 3 -10	4.3
Block 3 -5	5.0
Block 3 LW	3.8
Block 3 +5	3.2
Block 3 +10	3.6
Block 4 -10	3.7
Block 4 -5	3.7
Block 4 LW	4.1
Block 4 +5	3.5
Block 4 +10	3.5
Block 5 -10	2.3
Block 5 -5	3.5
Block 5 LW	2.8
Block 5 +5	3.6
Block 5 +10	1.5
Block 6 -40	4.3

10.1.1 Discussion of Modulus and Poisson's Ratio

The modulus and Poisson's ratio data collected appears to be confounded with testing parameters and ineffective for evaluation of mechanical properties of the asphalt mixtures. Figure 10-1 shows the effect of change in design unit weight on the compressive modulus for the asphalt mixtures. No clear tend is observed with a change in the volume of coarse aggregate or change in the gradation of the coarse or fine aggregate. In examining the data from LUW – 5% to LUW +5%, where mixtures are typically designed under the proposed concepts, the change in design unit weight will cause a decrease or increase in compressive modulus.

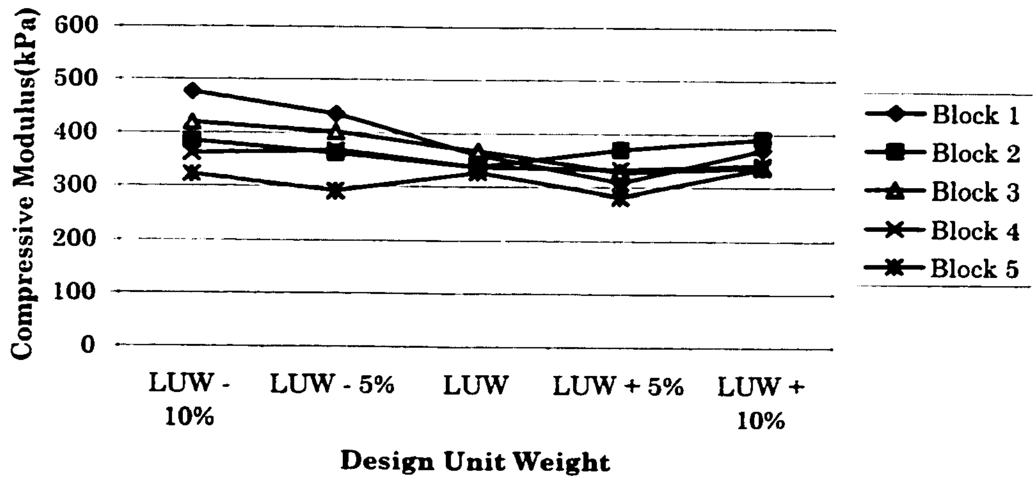


Figure 10-1 Compression Modulus Plot for Asphalt Mixtures

Using the compressive modulus data a ranking of the asphalt mixtures is given in Table 10-3, showing no clear ranking of the asphalt mixtures. In many cases the mixtures with no developed coarse aggregate interlock are ranked higher than mixtures with a strong coarse aggregate skeleton. Because these are not designed mixtures the mechanical property testing is confounded with mixture volumetrics, including the air voids and VMA.

Table 10-3 Ranking of Asphalt Mixtures by Compressive Modulus

Block 1	Block 2	Block 3	Block 4	Block 5	
- 10%	+ 10%	+ 10%	- 5%	+ 10%	*Better*
- 5%	- 10%	- 5%	- 10%	LUW	
+ 10%	+ 5%	LUW	+ 10%	- 10%	↕
LUW	- 5%	10	LUW	- 5%	
+ 5%	LUW	+ 5%	+ 5%	+ 5%	*Worse*

The frequency sweep data gives the appropriate trend in change in mixture modulus with test frequency. Figure 10-2 shows the plot of compressive modulus for the materials in block one over the entire frequency sweep. This viscoelastic behavior of the asphalt mixtures expected and typical.

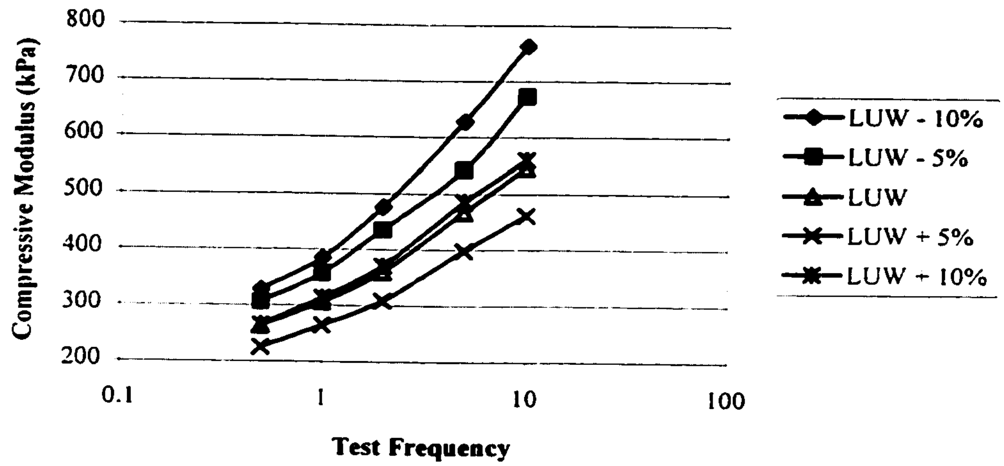


Figure 10-2 Frequency Sweep for Experimental Block 1 Samples

The values resulting from the RaTT testing are appropriate values for Poisson's ratio at 50 °C. Given that the Poisson's ratio values are appropriate and that they are based on the measured axial and radial deformations during testing , the test results from the RaTT testing are appropriate and accurate test results for the asphalt mixtures.

10.1.2 Discussion of RaTT Rutting Index

Examination of the RaTT test data through calculation of the RaTT Rutting Index does not capture the change in coarse aggregate volume or change in aggregate gradation. The rut index, plotted in Figure 10-3, appears as the modulus data from the RaTT test; without significant correlation to the change in aggregate gradation. A ranking of the mixtures based on the RaTT rutting index does not show any conclusive relationship between the aggregate structure and the rutting performance. This ranking is given in Table 10-4

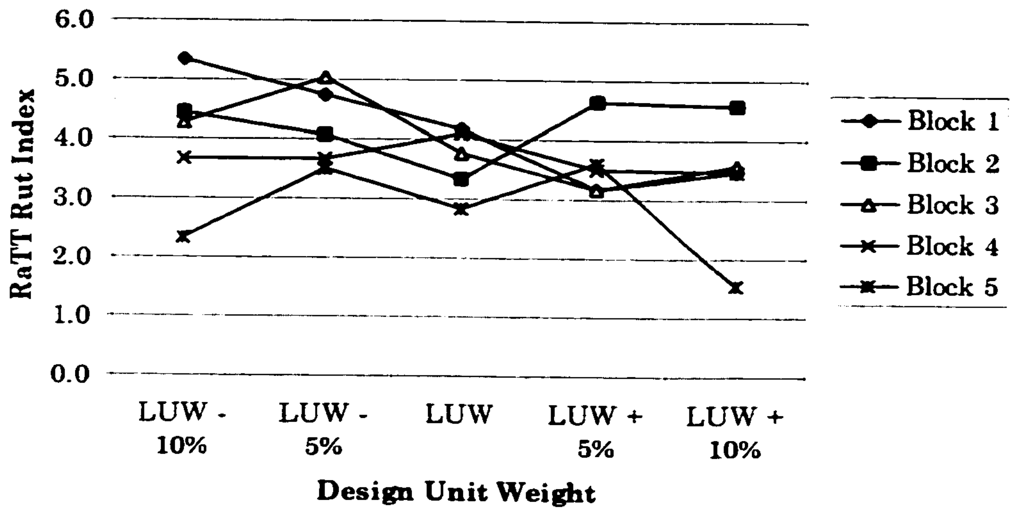


Figure 10-3 RaTT Rutting Index Plot for Asphalt Mixtures

Table 10-4 Ranking of Asphalt Mixtures by RaTT Rutting Index

Block 1	Block 2	Block 3	Block 4	Block 5	
+ 5%	LUW	+ 5%	+ 10%	+ 10%	*Better*
+ 10%	- 5%	+ 10%	+ 5%	- 10%	↑
LUW	- 10%	LUW	- 5%	LUW	↕
- 5%	+ 10%	- 10%	- 10%	- 5%	↓
- 10%	+ 5%	- 5%	LUW	+ 5%	*Worse*

The RaTT rutting index is based on a relationship between the asphalt pavement analyzer and the rapid triaxial test that was developed on testing of controlled materials. The air voids for test samples in the determination of the RaTT rutting index is 7% +/- 0.5%. Because the asphalt samples in this study have air voids from 2.0% to 9.0% the applicability of the RaTT rutting index is suspect and does not provide insight into the behavior of these mixtures.

10.2 Frequency Sweep Constant Height (SST Testing)

Testing of asphalt mixtures in the SST is performed to obtain the material properties of asphalt materials that relate to their performance under normal traffic loading. The frequency sweep test at constant height applies a constant low strain levels through a sweep of test frequencies to captures the complex modulus. The frequencies are varied to relate to traffic speeds from interstate highway to parking lot speeds. The results from the Frequency Sweep Constant Height SST testing are given in Table 10-5.

Table 10-5 Complex Modulus Test Data for Asphalt Mixtures

Sample Name	G Star 10 Hz (psi)	G Star 1 Hz (psi)
Block 1 -10	28,284	8,363
Block 1 -5	20,095	6,236
Block 1 LW	19,630	6,102
Block 1 +5	23,993	8,985
Block 1 +10	27,186	8,264
Block 2 -10	25,506	7,742
Block 2 -5	22,984	7,268
Block 2 LW	23,000	6,823
Block 2 +5	21,749	7,086
Block 2 +10	24,165	7,463
Block 3 -10	27,676	8,584
Block 3 -5	21,018	6,545
Block 3 LW	20,860	6,587
Block 3 +5	16,637	5,193
Block 3 +10	16,241	4,925
Block 4 -10	28,084	8,715
Block 4 -5	22,330	6,794
Block 4 LW	22,924	7,417
Block 4 +5	23,303	7,066
Block 4 +10	16,618	5,351
Block 5 -10	29,594	10,271
Block 5 -5	27,157	8,840
Block 5 LW	26,195	8,673
Block 5 +5	24,621	8,055
Block 5 +10	22,089	7,131
Block 6 -40	28,783	9,108

The FSCH data does not provide insight into the change in performance of the asphalt mixtures in this study. Figure 10-4 shows the complex modulus data at 1 Hz. This complex modulus at 1 Hz is used to evaluate mixtures for rutting potential. The change of coarse aggregate volume is not observed in the FSCH test data. The change in coarse and fine aggregate gradation does change the FSCH test results, but the data appears to be confounded with testing protocol problems and is therefore not indicative of the change in material properties that is expected with a change in aggregate gradation. An examination of the ranking of the mixtures by the complex modulus is given in Table XX. This ranking does not provide insight into the change of aggregate gradation on the performance of the resulting mixture.

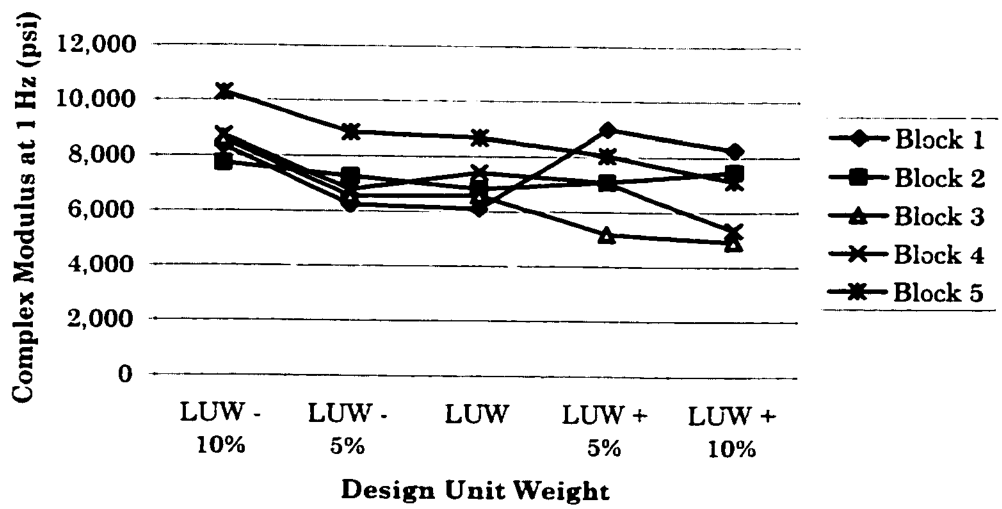


Figure 10-4 Complex Modulus at 1 Hz Plot for Asphalt Mixtures

Table 10-6 Ranking of Asphalt Mixtures by Complex Modulus at 1 Hz.

Block 1	Block 2	Block 3	Block 4	Block 5	
+ 5%	- 10%	- 10%	- 10%	- 10%	*Better*
- 10%	+ 10%	LUW	LUW	- 5%	↑
+ 10%	- 5%	- 5%	+ 5%	LUW	↕
- 5%	+ 5%	+ 5%	- 5%	+ 5%	↓
LUW	LUW	+ 10%	+ 10%	+ 10%	*Worse*

Because the testing was performed on samples that were not prepared to the test specification an evaluation of the FSCH and air voids is provided. Figure 10-5 shows that a definite trend exists between the complex modulus at 1 Hz. and the air voids of the tested samples. This air void interaction will effect the results of the testing, rendering them ineffective in the evaluation of aggregate gradation for this study.

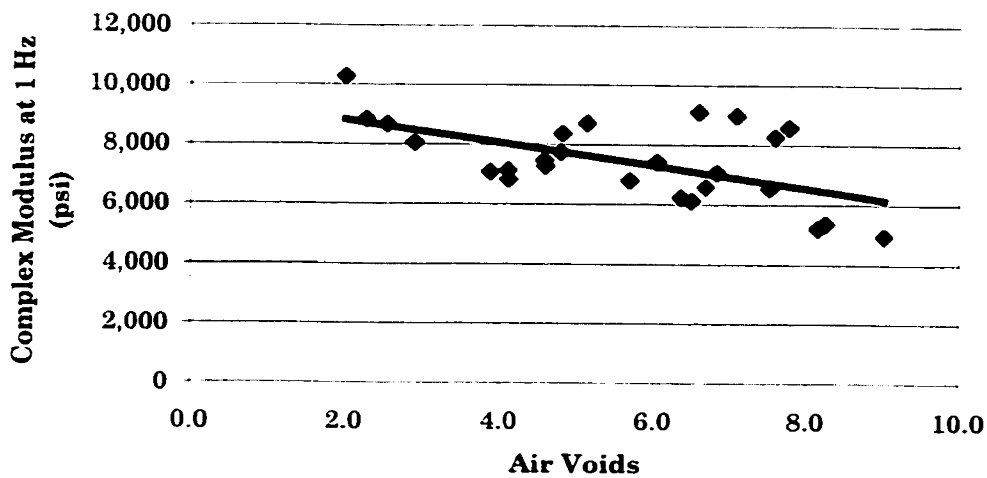


Figure 10-5 Air Void Interaction with Complex Modulus at 1 Hz.

10.3 Repeated Shear Constant Height (SST Testing)

The repeated shear constant height (RSCH) test is a destructive test performed on samples after the FSCH test for the evaluation of rutting potential. A constant shear stress is applied to the test specimen for 5000 cycles and the resulting permanent deformation is recorded. This permanent deformation is accumulated through the 5000 cycles and the accumulated deformation is used to rank asphalt mixtures for rutting potential. The test results for RSCH testing are given in percent of permanent strain and are shown in Table 10-7.

Table 10-7 Repeated Shear Constant Height Test Results for Asphalt Mixtures

Sample Name	Accumulated Strain at 5000 Cycles
Block 1 -10	
Block 1 -5	2.8
Block 1 LW	2.7
Block 1 +5	2.3
Block 1 +10	1.4
Block 2 -10	1.4
Block 2 -5	2.6
Block 2 LW	2.4
Block 2 +5	2.2
Block 2 +10	2.9
Block 3 -10	1.4
Block 3 -5	2.6
Block 3 LW	2.4
Block 3 +5	2.2
Block 3 +10	2.9
Block 4 -10	1.8
Block 4 -5	2.2
Block 4 LW	2.1
Block 4 +5	1.7
Block 4 +10	3.7
Block 5 -10	1.4
Block 5 -5	1.7
Block 5 LW	1.6
Block 5 +5	2.0
Block 5 +10	2.1
Block 6 -40	2.2

The RSCH test data does not provide insight into the performance of the asphalt mixtures in this study. Figure 10-6 shows the permanent deformation at 5000 cycles for the asphalt mixtures. No significant conclusion can be drawn from these test results on the effect of aggregate gradation of the asphalt materials. The test results are confounded with the preparation of the samples and the resulting air voids of the asphalt mixtures. Figure 10-7 shows the relationship between the RSCH test results and air voids.

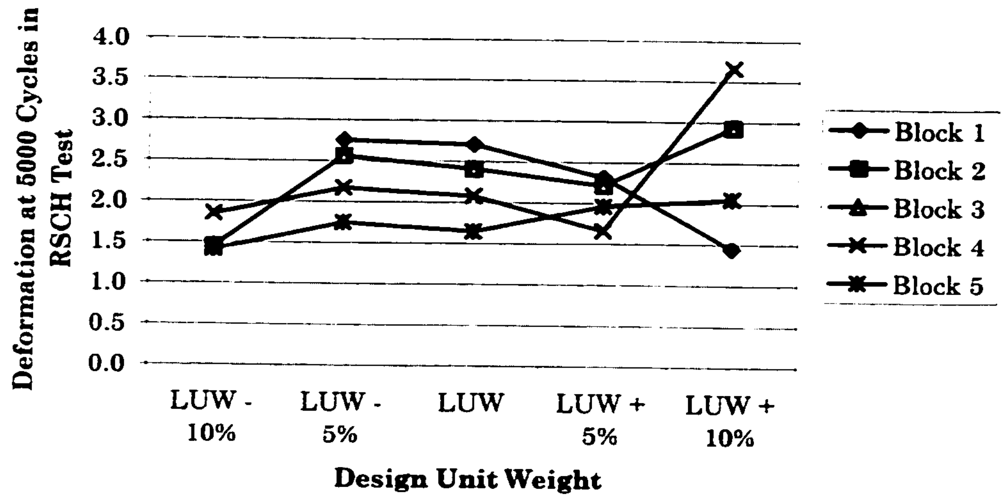


Figure 10-6 Permanent Deformation at 5000 Cycles in RSCH Test for Asphalt Mixtures

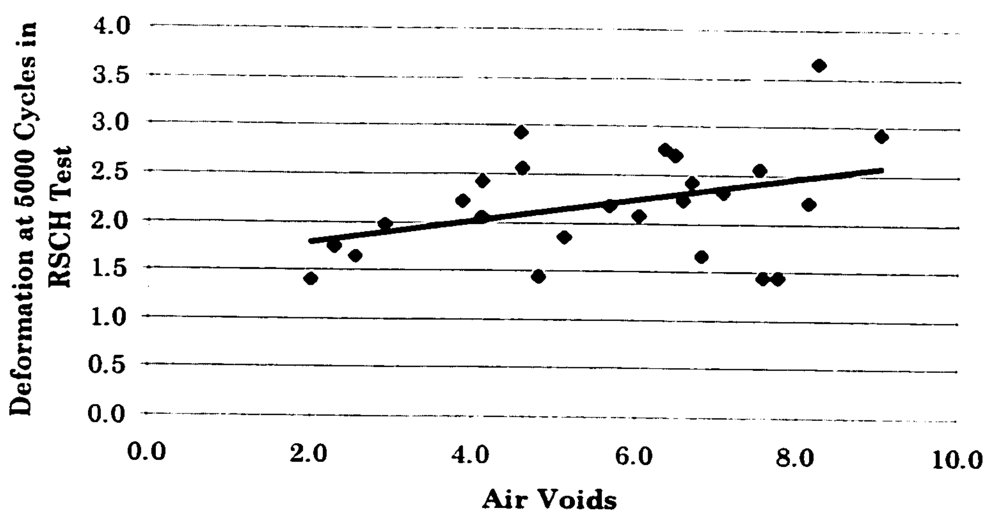


Figure 10-7 Permanent Deformation at 5000 Cycles in RSCH Test Interaction with Air Voids

10.4 Resilient Modulus

The resilient modulus of an asphalt mixture is used in mechanistic pavement design procedures to evaluate the ability of a mixture to carry load in the pavement structure. The test results from the resilient modulus testing are given in kPa. and are shown in Table 10-8. Similarly to the RaTT modulus values and the FSCH complex modulus the resilient modulus data does not provide any insight into the change in mechanical properties and change in aggregate gradation. Figure 10-8 shows the interaction of resilient modulus with air voids. This interaction is strong and confounds the test data beyond use for evaluation of aggregate gradation.

Table 10-8 Resilient Modulus Test Results for Asphalt Mixtures

Sample Name	Resilient Modulus (Mpa)
Block 1 -10	6,001
Block 1 -5	5,011
Block 1 LW	5,383
Block 1 +5	4,634
Block 1 +10	4,201
Block 2 -10	5,456
Block 2 -5	5,481
Block 2 LW	5,885
Block 2 +5	5,134
Block 2 +10	5,601
Block 3 -10	5,989
Block 3 -5	5,245
Block 3 LW	4,669
Block 3 +5	4,100
Block 3 +10	5,251
Block 4 -10	5,918
Block 4 -5	5,388
Block 4 LW	4,986
Block 4 +5	5,059
Block 4 +10	3,392
Block 5 -10	6,475
Block 5 -5	6,006
Block 5 LW	7,070
Block 5 +5	6,382
Block 5 +10	5,508
Block 6 -40	5,301

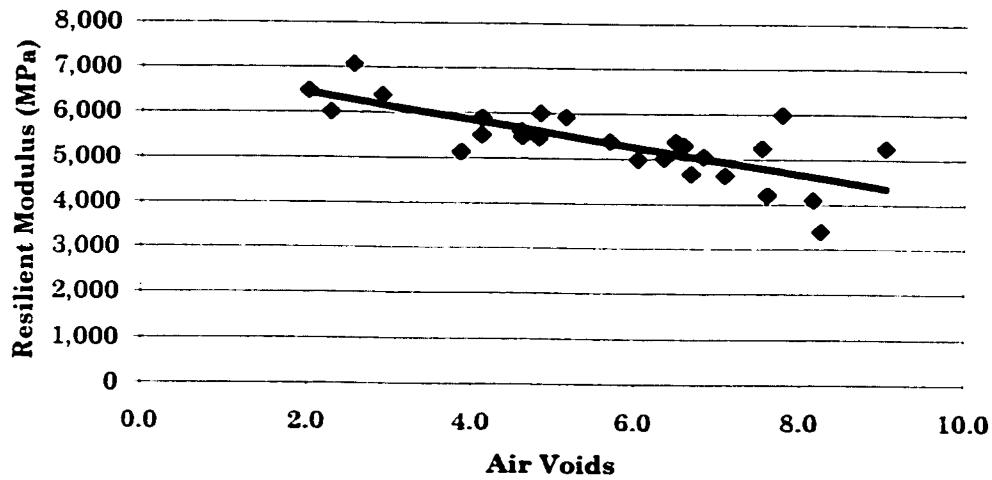


Figure 10-8 Resilient Modulus Interaction with Air Voids

10.5 SUMMARY OF MECHANICAL PROPERTY TEST RESULTS

The mechanical property testing in this study does not follow the accepted test procedures for mechanical property testing, which confounds the results with outside variables. Typical mechanical property testing is performed on mixtures that are designed to reach design air voids at an established level of compactive effort. These designed mixtures are then compacted to a standard percentage of air voids and tested in accordance with the proper test protocol. The tested asphalt mixture are not designed mixtures, rather they have an established asphalt content and compactive effort and allow the air voids to fluctuate considerably. The irregular air void levels have provided insight into the change in mixture volumetrics with a change in aggregate gradation, but confound the mechanical property tests.

CHAPTER 11 CONCLUSIONS

The state of the art in asphalt mixture design is changing with the implementation of Superpave volumetric mixture design. Full implementation of Superpave is expected in the United States in the next five to ten years. The aggregate quality and asphalt cement specifications included in Superpave provide a standard for quality in the component materials for asphalt mixtures. The Superpave Gyratory Compactor is an improved method for the compaction of asphalt mixtures and acceptance of the gyratory compactor as the appropriate tool for laboratory compaction appears nearly universal. While Superpave offers improvements to the state of the art for asphalt mixture design it does not provide a technique for the combination of aggregates. Of the current asphalt design procedures no system outlines the interaction of aggregate blending and the resulting mixture. The results of this study provide such a system.

The following are specific conclusions of this research:

- This study presents the comprehensive mix analysis concepts for developing and analyzing HMA gradations. The method to combine aggregates gives a rational approach to select the appropriate volume of coarse and fine aggregate to develop aggregate interlock. The ratios for the analysis of gradation provide a method to quantify the packing of the aggregates in the HMA that relates to the void structure in the material.
- Aggregate Gradation is important in material properties of HMA. The gradation of the coarse aggregate changes the size of voids, which translates to different compactability of the mixture. The gradation of the fine aggregate changes the compactability of the fine aggregate and the resulting compactability of the mixture.

- Particle packing concepts are important in understanding the combination of aggregates. The packing of spheres provides a background for the study of aggregate packing. From the packing of spheres an estimate of the size of void in the coarse aggregate is given by the characteristic particle diameter ratio of 0.22.
- Aggregate packing is dependent on the shape, texture, and gradation of the aggregate particles and the type and amount of compactive effort applied to the particles.
- Evaluation of gradation with aggregate packing ratios provides a new tool for examining aggregate gradations. These ratios, based on particle packing, provide distinct relationships with the resulting voids and compaction characteristics of the asphalt mixture.

The results of this study improve the state-of-the-art in asphalt mix design and production by providing a method to characterize HMA mixture voids (Air Voids, VMA, VCA) and compaction characteristics through the fundamental principles of particle packing. The design concepts outlined in this study provide the foundation for a comprehensive asphalt mixture design method: *The Bailey Method of Gradation Analysis and Asphalt Mix Design.*

CHAPTER 12 RECOMMENDATIONS FOR FUTURE RESEARCH

The concepts outlined in this study provide the foundation for a comprehensive mixture design procedure. This procedure would allow the aggregate structure to be designed to create coarse aggregate interlock providing an aggregate skeleton to resist deformation. The resulting aggregate blend packing characteristics can be evaluated with the aggregate ratios to understand the void structure in the mixture. However, as discussed in the conclusions, there are crucial elements in a mix design procedure that were not possible to study in this project. Further study is required in the following:

1. The performance of mixtures, measured by mechanical property tests, designed using the concepts given in this study should be evaluated. These would be completely designed mixtures, designed with the Bailey Method, with various levels of coarse aggregate interlock. The effect of coarse aggregate volume and change in aggregate gradation on the mechanical properties of the mixtures can then be combined with the understanding of change in volumetric properties from this study to give a more complete understanding of gradation and the asphalt mixture.

2. An evaluation of mixtures with more than one coarse and fine aggregate should be performed to illustrate the effect of changing aggregate shape and surface texture on the volumetric properties and performance of the asphalt mixtures

3. Mixtures must be tested with larger nominal maximum particle size to verify that the mixture design concepts presented are valid for asphalt mixtures with larger aggregate particles.

4. A comprehensive evaluation of the aggregate ratios is necessary to understand the true relationship between aggregate packing and the resulting volumetric

properties in the asphalt mixture. This evaluation must provide independent changing of the aggregate ratios to understand their individual and combined effects.

With continued research, the concepts provided in this study will become the state of the art in asphalt mixture design; establishing the next mixture design method, "The Bailey Method for Asphalt Mix Design."

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