Development of a mix design methodology for asphalt mixtures with analytically formulated aggregate structures

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DEVELOPMENT OF A MIX DESIGN METHODOLOGY FOR ASPHALT MIXTURES WITH ANALYTICALLY FORMULATED AGGREGATE STRUCTURES

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ABSTRACT

This research documents an extensive study on the design and characterization of asphalt mixtures for use as road pavement material. Several aspects of asphalt mixtures were addressed using the state of the art laboratory test equipment and technical literature from different information sources. The research was divided into two phases. Phase one included the design and detailed analysis of compaction and performance characteristics of asphalt concrete mixtures with aggregate structures that were designed using an analytical method of aggregate blending. Three aggregate types were considered in this study: limestone, sandstone, and granite. All the aggregates were crushed aggregates. Three different aggregate structures were designed for each aggregate type using the Bailey method of aggregate gradation evaluation. The Bailey method is a comprehensive gradation. Sandstone and Granite mixtures had a nominal maximum aggregate size (NMAS) of 12.5mm and were designed for high traffic level, while two types of Limestone mixtures were designed (25.4 mm and 12.5 mm NMAS) for two traffic levels (high and low traffic volumes). For the heavy traffic mixtures the binder type selected was PG 76-22M while PG70-22 was used for low volume mixtures. The outcome of this research suggests that suitable mixes can be developed with dense aggregate structures using the Bailey method of aggregate gradation that provides good resistance to permanent deformation while still maintaining adequate levels of durability. A systematic, simplified design approach was recommended in which asphalt mixtures are designed based on the locking point concept, analytical aggregate gradation method and fundamental mechanistic properties that describe the behavior of asphalt mixtures based on sound engineering principles.
CHAPTER 1. INTRODUCTION

1.1 Report Organization

This dissertation documents the methodology and findings of the research conducted to study and evaluate asphalt mixtures with analytically formulated aggregate structures. Chapter 1 is an introductory chapter that presents brief background information on asphalt mixtures and highlights the research motivations. It also describes the objectives and the scope of work. Chapter 2 presents a detailed literature review conducted on some important aspects of the research area. Chapter 3 describes the materials used and the test procedures followed to conduct this research. Chapter 4 presents the findings of the first phase of the research. Chapter 5 presents and discusses the findings of phase 2 of the study. Finally, chapter 6 summarizes the key findings and the conclusions that can be drawn based on the results obtained from this research.

1.2 Background

Hot mix asphalt (HMA) is the most common material used for paving applications in the United States. It primarily consists of asphalt cement binder and mineral aggregates. The binder acts as a gluing agent that binds aggregate particles into a cohesive mass. When bound by asphalt binder, mineral aggregate acts as a stone framework that provides strength and toughness to the system. The behavior of HMA depends on the properties of the individual components and how they react with each other in the system.

Several mixture design methods have been developed over time, the purpose of which is developing a mixture that is capable of providing acceptable performance based on certain predefined set of criteria. This is normally achieved by selecting an optimum design asphalt cement content that will achieve a balance among the desired volumetric
properties. The desired properties may include durability, permeability, strength, stability, stiffness, fatigue resistance, and workability. It should be emphasized however, that there is no single asphalt cement content that will maximize all of these properties. Instead, the design asphalt cement content is selected on the basis of optimizing the properties necessary for a specific condition (Asphalt Institute SP-2, 2001).

Usually, the mixture design process consists of two main parts. The volumetric design portion and either empirical or fundamental mechanical testing to verify the design. In addition, the design method may include other requirements that the mixture must meet in order to satisfy the overall specification standard. Such requirements may include certain aggregate qualities like minimum percent of crushed aggregate, maximum amount of rounded sand materials and specific aggregate gradation requirements (Asphalt Institute SP-2, 2001).

The most recently developed mixture design method is the Superpave method. Superpave stands for Superior Performing Asphalt Pavements and represents a basis for specifying component materials, asphalt mixture design and analysis, and pavement performance prediction. It was the final product of the $50 million Strategic Highway Research Program (SHRP). Several new requirements were proposed as means to improve mixture performance by taking into account the critical factors affecting the behavior of individual mixture components as well as the compacted mixtures (Cominsky et al, 1994). The Superpave mixture design includes several processes and decision points. The system includes an asphalt binder specification that uses new binder physical property tests; a series of aggregate tests and specifications; a hot mix asphalt (HMA) design using the Superpave Gyratory Compactor (SGC); a refined procedures and
requirements for mixture analysis; and computer software to integrate the system components.

In summary, the design compaction levels are established and materials are selected and characterized. Then, mixture specimens are prepared and laboratory test results are compared to specifications.

It was hoped that such a sophisticated system like Superpave may resolve some inherent problems in the previous asphalt mixture design systems. The system however, still suffers from certain shortcomings that need to be addressed and improved. The following section highlights some areas of concern about the Superpave system.

1.3 Problem Statement

Generically, Superpave is a step towards improving previous mixture design procedures because Superpave designs the asphalt mixture for a specific location, climate, and traffic (McGennis, 1995). The system however, still has some shortcomings and imperfections that necessitate more research on the different aspects of it that are of concerns to highway industry.

One of the main shortcomings of the Superpave mixture design method is the fact that the whole process is purely volumetric and solely relies on certain volumetric requirements that are supposed to ensure acceptable performance. The criteria were derived based either on experience of panels of experts or on some research studies that were conducted on certain Superpave mixtures under limited conditions. Mixtures are accepted or rejected based on those criteria at an early stage in the design process without any validation of their expected performance. An example of such criteria is the percentage of voids in the mineral aggregate (VMA). VMA is the total void space between the aggregate particles in compacted asphalt concrete, including air voids and
asphalt not absorbed by the aggregates. It was reported by several researchers and highway agencies that there exist difficulties in meeting the minimum voids in VMA requirements (Haddock et al. 1999, Hinrichsen 1996, Kandhal 1998, Coree et al. 1998-2000, Mallick 2000, Nukunya et al 2000, Anderson 2001). Studies have also shown that the current defined VMA criterion was seen to be insufficient by itself to correctly differentiate well performing mixtures from poor ones. In other words, the design process in the Superpave system does not properly address the expected performance of the designed mixtures in terms of major pavement distresses like permanent deformation and fatigue cracking through laboratory performance testing.

Although aggregate constitutes approximately 95% by weight of asphalt mixtures, the aggregate specifications in the Superpave system were developed based on experience from a number of experts in the field who formulated what is called the Aggregate Expert Task Group (ETG). The ETG did no research on aggregates. They did build on the studies and recommendations of researchers who came before them and the expertise of many practitioners. From this previous research they developed rules and recommendations for the Superpave System.

As a result of the lack of research conducted in developing such aggregate specifications, there are still open windows for improvement on those specifications and requirements especially designing the aggregate structure to improve mixture stability. For example, in the current Superpave system, guidance is lacking in the selection of the design aggregate structure and understanding the interaction of the aggregate structure with mixture design and performance. Furthermore, the trial and error nature of the actual conventional process of formulating the gradation curve, and the use of weight instead of volume when blending aggregates, offer alternatives to evaluate more rational
approaches to design an aggregate structure based on sound principles of aggregate packing concepts.

A key to a successful mixture design is the balance between the volumetric composition and the properties of the raw materials used (binder and aggregates). The interaction between these components coupled with the different types and magnitude of loadings the pavement is subjected to, results in highly complex mixture responses that require more complete understanding of asphalt mixture behavior. The key step to achieve that is to understand how the mechanical performance of asphalt mixtures is affected by different mixture components and properties.

From the above discussion, there is clearly a need to address the issues of concern in the current Superpave mix design system by introducing more rational, systematic steps to the current system for better design and evaluation of asphalt mixtures.

1.4 Objectives of Research

The primary objective of the proposed research was to recommend a systematic, simplified mixture design methodology that is based on mixture performance as evaluated by laboratory mechanistic tests. This was achieved by incorporating an analytical gradation design and evaluation method into the Superpave mixture design procedure and evaluating compaction and performance characteristics of the resulting mixtures.

The second objective of this research was to study the strict VMA requirements that were used in the current Superpave method of mix design. The main question was that, what kind of mixture performance will be obtained if those requirements were violated but with a proper design of aggregate structure that will result in a stable mixture without compromising the durability in terms of age hardening and resistance to moisture
damage. A well designed aggregate structure will result in less interconnected air voids that will minimize the penetration of air and water through the pavement structure.

A third objective of this research was to understand the effect of identifiable variables on mixture mechanical responses. Such variable may include nominal maximum particle size, aggregate gradation parameters, aggregate type and compaction level.

1.5 Scope

To achieve the stated objectives, the proposed a test factorial was developed that covers the following controlled parameters:

- Aggregate Type: Three aggregate types were used in this study that are commonly used by Louisiana Department of Transportation and Development (LADOTD) Louisiana. The types are:
  - Hard aggregates (crushed granite),
  - Water absorptive, high friction aggregate (Sandstone), and
  - Low friction, low water absorption aggregate (Limestone aggregate).

- Mixture Types: Two mix types were designed; 12.5mm Nominal Maximum Aggregate Size (NMAS) and 25mm NMAS. The 12.5mm mix was designed for all the three aggregate types mentioned above. The 25 mm mix was designed using only limestone aggregates. Within the same NMAS, Three aggregate structures (coarse, medium, and fine) were designed using the Bailey method of aggregate gradation evaluation. The coarse aggregate structure has the highest volume of coarse particles. This volume decreases as the structure becomes finer.
- Binder type: Two asphalt cement types were used. PG 76-22M was used with the high volume mixture type while PG70-22 was used with mixtures designed for low volume traffic.

- Compaction Level: Two compaction levels were used to manufacture test specimens using the Superpave Gyratory Compactor (SGC). These levels were 125 and 75 Gyrations. These two compaction levels correspond to high and low traffic levels respectively in the Superpave system.

The study was divided into two phases. Phase one involved designing the aggregate structures and performing Superpave mixture design to determine the design asphalt content that provides four percent air void that is currently being used by the Superpave system as an acceptable design parameter for dense graded mixtures. Following that, the first suite of mixture evaluation tests was conducted in order to determine the best performing aggregate skeleton for each aggregate type and size combination. This evaluation included the following:

- Determining compaction properties and frictional resistance of the mixtures.
- Measuring the permeability of each mixture as an important physical parameter for a successful performance of asphalt mixture.
- Conducting simulative test (Hamburg Wheel Tracking Test) on the mixtures to determine their stability under harsh environment of moisture and high temperature.
- Conducting Fundamental mechanistic tests to evaluate the performance of the designed mixtures. These tests include: Indirect Tensile Strength Test (ITS) and Fracture Energy Test. Those tests were conducted at 25°C on both aged and
unaged specimens as part of the durability evaluation of the mixtures. Figure 1.1 is a flow chart of phase 1 of this study.

Figure 1.1 Phase 1 of the Research
Phase 2 involved utilizing the data from phase 1 in selecting mixtures with specific attributes for further evaluation. The locking point concept is introduced in this phase and used in modifying the current Superpave mixture design methodology in order to improve the durability of the mixtures without compromising the stability. It also involved conducting more fundamental engineering tests in order to include performance related parameter(s) that can be added to the current volumetric mixture design process. Figure 1.2 shows the main tasks of phase 2 of this research.

![Flowchart of Phase 2 tasks](image)

The following terms were used in the flow charts in Figure 1 and Figure 2:

- **SGC**: Superpave Gyratory Compactor, **PDA**: Pressure Distribution Analyzer
- **HWT**: Hamburg Wheel Tracking, **E**\(^*\): Dynamic Modulus, **ITS**: Indirect Tensile Strength
- **Jc**: Critical J-integral, **DCSE**: Dynamic Creep Strain Energy
CHAPTER 2. LITERATURE REVIEW

2.1 Introduction

Hot mix asphalt (HMA) is the most common material used for paving applications in United States. It primarily consists of asphalt cement binder and mineral aggregates. The binder acts as an adhesive agent that binds aggregate particles into a cohesive mass. When bound by asphalt cement binder, mineral aggregate acts as a stone framework that provides strength and toughness to the system. The behavior of HMA depends on the properties of the individual components and how they react with each other in the system.

2.2 Asphalt Cement Binder Role

Asphalt cement is one of the two principal constituents of HMA pavement. It is a dark brown to black cementitious material that is either naturally occurring or is produced by the distillation of crude oil (Roberts et al., 1996). In the context of asphalt pavements, three asphalt cement binder characteristics are considered very important to the performance of the pavement in service. These are: temperature susceptibility, viscoelasticity, and aging (Roberts et al., 1996, Asphalt Institute MS No. 22, 2003). The properties of the asphalt cement binder are very dependent on its temperature. At high temperatures, asphalt cement binder becomes viscous and displays plastic response when subjected to loads higher than its viscosity at a particular temperature. This behavior under high temperature can be a contributing factor to one of the most common asphalt pavement distresses which is rutting (Figure 2.1a). In extremely cold climates, asphalt binder becomes very stiff and behaves like an elastic solid. Any induced elastic deformation is completely recovered. The extreme stiffening of the asphalt cements under such cold temperatures is the main factor for a pavement distress known as low temperature cracking. In this case, non-load environmentally related internal stresses
accumulate in the pavement due to the brittle nature of the binder as the pavement tries to shrink and is restrained (Figure 2.1b).

Figure 2.1 Distresses in Flexible Pavements a) Rutting b) Low Temperature Cracking

![Figure 2.1 Distresses in Flexible Pavements a) Rutting b) Low Temperature Cracking](image)

At normal intermediate pavement service temperature, the second important asphalt binder characteristic, viscoelasticity, becomes dominant. At these temperatures, the asphalt binder has characteristics of both viscous fluid and elastic solid (Asphalt Institute...)

![Figure 2.2 Temperature Susceptibility of Asphalt Cement Binder](image)
Figure 2.2 shows typical response of asphalt cement binder to change in temperature.

Asphalt cement binder behavior is also dependent on time of loading (Figure 2.3). The same load applied for different durations will result in different behaviors for the same asphalt. The dependency of the asphalt binder on both the temperature and time of loading make it possible to use these factors interchangeably. In other words, a slow loading rate can be simulated by high temperatures and a fast loading rate can be simulated by low temperatures (Roberts et al., 1996).

Chemically, asphalt binder is composed of organic molecules (hydrocarbon) and therefore reacts with oxygen. The result of this reaction is called aging. Aging is the hardening of the asphalt cement as it reacts with oxygen to the extent that it becomes brittle (Alvarez et al., 1994). Pavements with aged asphalt are more susceptible to cracking that will ultimately lead to structural and or functional failure of the pavement.
2.3 Aggregates Role

Aggregates are the second principal material in HMA. They play an important role in the performance of asphalt mixtures. For HMA, they make up about 90 to 95 percent by weight and comprise 75 to 85 percent of the volume (Roberts et al., 1996 Asphalt Institute MS No. 22, 2003). Therefore, knowledge of aggregate properties is crucial to designing high quality HMA mixtures.

Aggregates can either be natural or manufactured. Natural aggregates are generally extracted from larger rock formations through an open excavation. Extracted rock is typically reduced to usable sizes by mechanical crushing. Manufactured aggregate is often the byproduct of other manufacturing industries such as construction and steel industries.

An aggregate’s mineral composition largely determines its physical characteristics and how it behaves in an HMA pavement. Therefore, when selecting an aggregate source, knowledge of the quarry rock’s mineral properties can provide valuable information about the suitability of the resulting aggregate for HMA pavements. Regardless of the source, aggregate are expected to provide a strong, stone skeleton to resist the repeated traffic load applications. When a mass of aggregate is subjected to excessively high loads, a shear plane develops resulting in the aggregate particles sliding or shearing with respect of each others. This behavior produces what is called permanent deformation in asphalt pavement (Asphalt Institute SP-2, 2001). Along this shear plane, the applied shear stress exceeds the shear strength of the asphalt mixture (Figure2.4).
It is known that aggregate has relatively little cohesion (McGennis et al., 1995). The shear strength is mainly dependent on the internal friction provided by the aggregate. Here, the shape and texture of the aggregate play important role in providing the required interlock. Cubical, rough-textured aggregate provide more shear resistance than rounded, smooth-textured aggregate (Figure 2.5). The internal friction provides the ability of aggregate to interlock and create a strong mass that is able to resist the applied traffic load.

**Figure 2.5 Cubical Rough Aggregates vs. Smooth-rounded Aggregates**

### 2.3.1 Aggregate Gradation

The largest portion of the mixture’s resistance to the applied traffic loads is provided by the aggregate structure. Aggregate is expected to provide a strong stone skeleton to resist repeated load applications. One of the key aggregate properties that is related to asphalt mixture performance is gradation. Aggregate gradation is the
distribution of the different particle sizes in a mass of aggregate expressed as a percent of
the total weight (National Stone Association, 1996). Sieve analysis is the process by
which aggregate gradation is determined in the laboratory. Aggregate particles are
passed through a series of sieves stacked with progressively smaller openings from top to
bottom, and weighing the material retained on each sieve. Gradation of an aggregate is
traditionally represented in graphical format by a gradation curve for which the ordinate
is the total percent by weight passing a given sieve on an arithmetic scale, while the
abscissa is the particle size plotted to a logarithmic scale as shown in Figure 2.6 (Roberts
et al., 1996). For asphalt mixtures, it is generally accepted that a well-balanced,
continuous gradation will provide the greatest permanent deformation resistance for any
given type and quality of aggregates (Roberts et al., 1996, Ruth et al. 2002, National

![Figure 2.6 Typical Conventional Aggregate Gradation Curve](image)
Gradation is considered a key factor in the resistance of mixture to permanent deformation (Ervin JR, 1989, Hveem, 1946). The most important concept is that a well-balanced, continuous, gradation will provide the greatest structural strength (resistance to rutting) for any given type and quality of aggregate.

2.3.2 The Concept of Aggregate Packing

The importance of aggregate gradation and the need for understanding the interlocking mechanism of aggregates have been a topic of interest by several researchers. One of the earliest attempts to explain and quantify the packing of a mass of aggregates was carried out by Tons et al. (1968). In their study, the packing volume and rugosity concepts were introduced as the theoretical basis for understanding the bulk behavior and interlocking mechanisms of aggregates. The angularity and texture of an aggregate particle was unified by the term rugosity. The more angular the rock is, the higher its rugosity. The particle volume was defined as the volume which a single rock particle occupies in a mass of mono volume particles. Due to irregular shape of aggregate particles, aggregates usually touch one another at the peaks of the surface roughness. Therefore, the packing includes not only the solid mass and the surface capillaries but also the volume of the surface voids. In other words, the packing volume can be visualized as the volume enclosed by a dimensionless membrane stretching along the peaks of the surface roughness. For a mass of aggregate, this membrane divides voids into interparticle voids and particle surface voids (Tons et al. 1968).

Ishai et al. (1971) demonstrated experimentally that in bituminous mixtures, surface voids of large particles provide sufficient space not only for asphalt, but also for smaller particles. They explained conceptually using a container filled with one-size, coarse, smooth particles. To this container, a certain amount of one-size, fine, smooth
particles were added. The average equivalent sphere diameters for coarse and fine particles were designated as $d_c$ and $d_f$, respectively. If the diameter ratio ($d_f/d_c$) is small enough, the fine particles will be able to filter between the coarse ones and will fill the interparticle voids. Thus, without changing the mass volume (volume of the container) the total packing volume of the blend will increase, while the amount of packing porosity will decrease. Under no dilation of coarse particles, the increase of the total packing volume is equal to the decrease of the volume of interparticle voids. However, when the diameter ratio ($d_f/d_c$) increases, dilation will occur in the structure of coarse particles and the introduction of fine fraction will increase the mass volume. Under constant packing volume of the particles, any additional increase in the mass volume (dilation) will be equal to a change on the volume of interparticle voids. The models are additive in both cases.

The additivity and simplicity of the above models are distorted when aggregates with irregular and rough aggregate fractions are involved. In this case, some of the particles may penetrate through and under the imaginary packing volume membrane of coarse particles. They defined this interaction between coarse and fine aggregates as the fines lost by rugosity. They further observed that less active fine particles will be located between the larger rough particles which will be packed closer together with thinner asphalt films between them exhibiting higher resistance to shear, tensile and compressive deformation. On the other hand, smooth textured particles will be simply pushed apart by the more active fines between them and show low strength.

Khedaywi et al. (1998) studied the effect of aggregate rugosity and size on bituminous mixes. Their hypothesis was that for each coarse aggregate type with different surface characteristics, there is a specific fine aggregate size that contributes to
developing an interlocking mechanism between the surfaces of coarse aggregates when they are combined in a bituminous mix. To test this hypothesis, two types of coarse aggregates, crushed limestone and rounded gravel were used. They concluded that by matching the rugosity and the size of the fines properly, the strength of rounded gravel mixes could be made much closer to the strength of mixes using crushed limestone coarse aggregate.

2.3.3 Research Studies on the Role of Aggregate Gradation

Several research studies investigated the role that aggregate gradation plays in the performance of asphalt mixtures. A summary of their findings is presented below.

Elliot et al. (1991) conducted an investigation to evaluate the effect of different aggregate gradations on the properties of asphalt mixtures. The aggregate blends included: coarse, fine, and medium gradations and two poorly graded. From this investigation, they concluded that:

- Variations in gradation have the greatest effect when the general shape of the gradation curve is changed (i.e., coarse-to-fine & fine-to-coarse gradations).
- Fine gradation produced the highest Marshall stability, while the fine-to-coarse poorly graded gradation (with hump at sand sized) produced the lowest Marshall stability.

Kandhal et al. (1993) studied the effect of aggregate gradation on measured asphalt content. A total of 547 binder coarse mix samples and 147 wearing coarse mix samples were obtained from field projects and the asphalt cement was extracted using ASTM D2172 “Standard Test Methods for Quantitative Extraction of Bitumen From Bituminous Paving Mixtures” procedure. Correlation analysis was performed to determine if the pavement layer density or the percentage passing various sieve sizes
correlate with asphalt cement content. It was concluded that for binder course mixtures, the percent passing the 4.75 mm and 2.36 mm sieves correlated with measured asphalt cement content. Prediction equations were developed to adjust the measured asphalt cement content to account for the change in gradation from the job mix formula on the 12.5 mm sieve and either 4.75 mm or 2.36 mm sieves.

Krutz et al. (1993) evaluated the effects of aggregate gradation on permanent deformation of HMA mixtures for the Nevada Department of Transportation. They utilized four different gradations, two aggregate sources, and two sources of asphalt cement AC20 asphalt cement. Two of the gradations were labeled as extreme fine and extreme coarse with 60 % and 43 % passing sieve No. 4, respectively. The middle gradations had 52 % and 54 % passing sieve No. 4. The Hveem mixture design method was followed to design the asphalt mixtures. Repeated load triaxial test was used to evaluate all the mixtures. The key findings of this research were that the best aggregate gradations is dependent on the type and source of aggregate and that coarse aggregate gradations performed the worst and fine aggregate.

In October 1995, the WesTrack project was initiated as a joint project between Federal Highway Administration (FHWA) and NCHRP (Mitchell et al. 1996). The project was established primarily to develop performance-related specifications based on construction variables. Twenty six pavement test sections were constructed that included two mix designs, one with a gradation above (fine-graded) and the other below (coarse-graded) the Superpave restricted zone. The restricted zone refers to a particular area of the Superpave gradation curve along the maximum density line between either sieve No. 4 or Sieve No. 8 and sieve No. 50. This zone forms a band through which it is not generally recommended that the gradation curve passes. Gradation that passes through
that zone were thought to have compactability problems and might exhibit reduced resistance to permanent deformation. All of the designed mixes were 19 mm nominal maximum size, with an unmodified performance grade (PG) 64-22 asphalt cement binder. The mixtures were designed using the Superpave method and subjected to traffic for two years from 1996 to 1998. The performance data gathered from this project showed that coarse – graded mixtures exhibited the greatest rut depths and percent fatigue cracking. Those mixes had a relatively high VMA. In the Superpave system however, mixes are accepted if they meet the minimum VMA requirement regardless how high that VMA value is.

Anderson et al. (1997) evaluated several trial aggregate gradations, through Superpave mixture design and material tests, to investigate the relationship between mixture properties determined in the design process and material properties from laboratory mechanistic tests. Four aggregate blends were developed using same aggregates to cover a range of gradations allowable in Superpave. Those gradations were:

- Blend 1: a coarse, S-Shaped gradation
- Blend 2: a fine gradation above the restricted zone
- Blend 3: an intermediate gradation passing through the restricted zone
- Blend 4: an S-Shaped gradation similar to blend 1 but with a slightly humped fine gradation.

The four blends were mixed with a PG 64-22 asphalt cement binder and compacted using the Superpave Gyratory Compactor. Volumetric analysis of the four mixtures showed that VMA did respond to the sum of distances from the Superpave maximum density line. Higher VMA values were associated with a greater sum of
The correlation however, was statistically weak (less than 0.2). The gradation structures were also evaluated using the slope of the compaction curve. The hypothesis was that the slope is an indication of an aggregate structure’s resistance to compaction. Higher slopes exhibit stronger aggregate structures and the S-shaped gradation structure was the strongest. The effect of adding natural sand was found to be significant in reducing the compaction slope (i.e. weaker structure). The mixtures were further studied using frequency sweep at constant height test and repeated shear at constant height test. The results from those tests didn’t confirm the general perception that the finer gradations have weaker aggregate structures.

Roque et al. (1997) evaluated the effects of aggregate gradation on shear resistance and volumetric properties of asphalt mixtures. Eighteen 12.5 mm mixtures were studied. Limestone aggregates were blended to produce coarse aggregate gradations ranging from gap graded gradations to very close to the maximum density line resulting in gradation curves that pass through and below the initial Superpave requirement of restricted zone. A gyratory testing machine (GTM) was used to compact and evaluate all mixtures. Gyratory shear index ($G_s$), determined from the GTM test was used as the basis for evaluating the shear resistance of the mixtures. It was found that several aggregate structures ranging from those passing through the restricted zone to gap graded structures provided good shear resistance when suitable gradations were used. It was also shown that the shear resistance values were sensitive to change in gradation properties as defined by a number of gradation parameters calculated to characterize gradation curves. Those parameters included slopes of the gradation curve within specific sieve sizes and the area between the gradation curve and the maximum density.
This was done in an effort to determine what constituted a suitable gradation. The following lists a summary of the findings from this study:

- Shear resistance of an asphalt mixture appeared to be most strongly related to the gradation characteristics of the coarse aggregate fraction of the mixture.
- The shape or curvature and position of the coarse aggregate fraction gradation curve relative to the maximum density line, as well as the coarseness of the aggregate influenced the shear resistance.
- Good shear resistance could be achieved with a broad range of aggregate structure ranging from TRZ to SMA.

Sousa et al. (1998) studied the effect of gradation on the fatigue life of asphalt mixtures using the SHRP-M009 four-point bending fatigue test. Above restricted zone (ARZ), through restricted zone (TRZ) and below restricted zone (BRZ) gradations ranging in NMS from 12.5 to 25.0 mm were evaluated. Six aggregate sources and two PG binder grades were used to produce nine mixtures. Four gradations were designed above the restricted zone (fine), three through the restricted zone (medium) and two below the restricted zone (coarse). All aggregates were 100% crushed granite. The coarse mixtures were designed using the Superpave mix design method ($N_{des} = 143$ gyrations). Five of the nine mixtures were designed using the Marshall method, one using a roller wheel compactor, and one using the Quebec mixture design method. Fatigue test specimens were prepared using a lightweight steel roller compactor with a target air void level of 7%. All tests were performed at $20^\circ$C in strain control mode. Fatigue life was defined as the number of load cycles required to reduce the initial mixture stiffness 50%. Key findings presented from the study were that fine-graded
mixtures exhibited better fatigue performance than mixtures with coarse gradations and the worst fatigue performance was exhibited by one of the Superpave mixtures.

Haddock (1999) evaluated two mixture sizes 19.0 mm and 9.5 mm as part of the National Pooled Study No. 176 to evaluate SHRP mixture specifications. The purpose of the study was to evaluate sensitivity of tests to change of gradations within the framework of Superpave aggregate specifications. Three aggregate structures for each size were used namely above, through, and below the Superpave restricted zone. The mixtures were evaluated using accelerated pavement test facility and a laboratory wheel tracking test. Triaxial test was conducted on the mixtures. This study concluded that fine mixtures showed suffered less rutting in both the prototype accelerated pavement test and the laboratory wheel test and had higher strength from the triaxial compression test. Overall, Fine mixtures showed a superior performance over the coarse mixtures although they tend to have lower design asphalt contents.

A laboratory investigation was performed by El- Basyouny et al. (1999) to study the effect of different aggregate gradations on the rutting potential of Superpave mixtures. Three aggregate gradations, ARZ, TRZ, and BRZ were used with nominal maximum particle sizes of 19 mm and 37.5 mm. The mixtures were compacted using the Superpave gyratory compactor and evaluated using the axial unconfined creep test. The creep parameters obtained from the creep test in the laboratory were used as input into the pavement analysis program VESYS-3 AM software (Kenis, 1978) to estimate the rut depth of a specific pavement section using different asphalt mixtures. The researchers concluded that aggregate gradation and aggregate nominal maximum aggregate size affected the rut depth as estimated by the VESYS software. Mixtures prepared using the aggregate gradations passing below the restricted zone (coarse mixtures) had a predicted
rut depth of 10 mm, while mixtures using gradations through and above restricted zone had similar predicted rut depth of 11 mm.

Kandhal et al. (2000) conducted a study with the objective of evaluating the effect of mixture gradation on rutting potential of dense graded mixtures. The performance of eighteen mixtures was evaluated based on the results from the Asphalt Pavement Analyzer (APA) and Superpave Shear Tester (SST) tests. Two mixture types (12.5 and 19.0 mm), three aggregate types (granite, limestone, and partially crushed gravel), and three gradation types (ARZ, TRZ, and BRZ) were considered. The coarse fraction of the gradation curve (+4.75 mm) was held constant while the fine portion of the gradation was adjusted to produce the different gradation blends. A PG 64-22 binder was used and mixtures were designed in accordance with the Superpave mix design method with N_{des}=76 gyrations corresponding to traffic level of 0.3 to 1.0 million ESAL. Both APA and SST performance test specimens were compacted to four percent air voids with the Superpave Gyratory Compactor (SGC). APA tests were conducted at 64°C and Repeated Shear at Constant height (RSCH) tests were conducted in accordance with AASHTO TP7. Analysis of APA rut depths indicated that aggregate type, gradation, and NMPS, as well as interaction between aggregate type and gradation were significant. Significant difference between rut depths of ARZ, TRZ, and BRZ mixtures was observed. Considering all data, mixes with gravel and limestone aggregates generally show higher rutting than granite. Also, for granite and limestone, mixes with gradation below restricted zone generally showed highest amount of rutting, whereas gradations through restricted zone generally showed the lowest rut depth. The above restricted zone generally showed intermediate rutting. The RSCH test results did not appear to be as sensitive to differences in gradations as the ones obtained from the APA test.
2.3.4 Existing Aggregate Blending and Evaluation Methods

Aggregate blending is proportionately mixing several aggregate gradations to obtain one desired aggregate gradation. Gradation is considered the most important property of an aggregate. It affects the engineering properties of a HMA such as stability, durability, permeability, and fatigue resistance (Roberts et al., 1996). The following sections highlight some of the commonly used aggregate blending methods for HMA.

2.3.4.1 Conventional Method

Traditionally, asphalt mixtures have been designed using a trial and error procedure to select the aggregate gradation. Aggregates are combined in typical percentages that were developed from years of experience. Coarse and fine aggregate are conventionally separated by the 4.75 mm sieve although occasionally 2.36 mm is used. Fine aggregate is considered as material to reduce the voids developed in the coarse aggregate and to reduce the asphalt cement content to a desirable amount without an excessive increase in coarse aggregate voids. In general, the method is based on the maximum density concept proposed by Fuller and Thompson for concrete mixtures (Fuller et al, 1907). The equation for Fuller’s maximum density curve is:

\[ P = 100 \left( \frac{d}{D} \right)^n \]

Where \( d \) is the diameter of the sieve size in question, \( P \) is the total percent passing or finer than the sieve, and \( D \) is the maximum size of the aggregate. Fuller recommended a value of 0.5 for the exponent \( n \) in the above equation for the maximum density to be achieved. A later investigation by Good and Lufsey (Good et al., 1965) applied the maximum density concept to asphalt mixtures which resulted in the selection of 0.45 for the aggregate gradation exponent (Figure 2.7).
In practice, deviation from the maximum density line is desirable when designing asphalt mixtures. Gradations of maximum density may not provide sufficient void space in the aggregate for enough asphalt cement required for the durability of the asphalt mixtures. Aggregates are normally blended by weight instead of volume. This might cause problems where the aggregate blend contains aggregates with large differences in density. Unless volumetric corrections are applied, it can result in excessive amounts of the lower density aggregate being incorporated into the asphalt concrete mixture. It is apparent that conventional gradation specifications provide little guidance for the selection of suitable gradations.

To help specify a proper aggregate gradation, the SHRP initially suggested two additional features to the traditional 0.45 power chart: control points and a restricted zone. The control points perform as ranges through which gradations must pass. Their
functions are: to maximize the size of aggregate; to balance the relative proportion of coarse aggregate and fine aggregate; and to control the amount of dust. The restricted zone is placed along the maximum density gradation between intermediate size and the 0.3 mm size. It was introduced to avoid mixtures that have a high proportion of fine sand relative to the total sand. It also avoids gradations that follow the maximum density, which do not have adequate voids in the mineral aggregate. Several researches however, showed that mixtures with aggregate gradations passing through the restricted zone have similar performance to other mixtures with aggregate gradations above or below the restricted zone (Cooley et al. 2002). Therefore, this requirement was removed from the Superpave specifications. Figure 2.8 shows a typical gradation chart with the SHRP requirements.

2.3.4.2 The Stone on Stone Contact Method

A method for determining when stone-on-stone contact exists was developed by Brown et al. (1997). This method was primarily developed for a specific type of asphalt mixtures; gap graded dense mixtures called the Stone Matrix Asphalt (SMA). The proposed method first determines the voids in the coarse aggregate (VCA) for the coarse aggregate only fraction of the SMA mixture. Secondly, the VCA for the entire SMA mixture is determined. When the two VCA values are compared, the VCA of the SMA mixture should be less than or equal to the VCA of the coarse aggregate only fraction to ensure that stone-on-stone contact exists in the mixture. To develop this method, five different compaction methods were used in combination with five different aggregate types (traprock, granite, limestone, Florida limestone, and silicious gravel). The five compaction methods were the Marshall hammer, the dry-rodded method (AASHTO...
T19), a vibrating table, the Superpave Gyratory Compactor (SGC), and the British vibrating hammer.

Figure 2.8 12.5mm SHRP specifications For Aggregate Gradations

Three replicates were used for each combination of aggregate type and compaction method. After the VCA of the coarse aggregate fraction was determined using each of the five compaction methods, a mixture design was completed for each of the aggregate types using 50 blows per specimen face of a flat-face, static base, mechanical Marshall hammer to compact the specimens. The VCA of each of these mixtures was calculated and compared to the VCA of the coarse aggregate only fraction. The results indicated that the Superpave Gyratory Compactor and dry-rodded methods produced the best results in terms of minimizing aggregate breakdown. Both of these methods were recommended for further evaluation. Those two methods for densifying the
coarse aggregate only fraction resulted in much less coarse aggregate degradation than that experienced by the total SMA mixture.

2.3.4.3 The Power Law Method for Aggregate Evaluation

Ruth et al (2002) suggested an approach to determine the slope and intercept (constant) of the coarse and fine aggregate portions of the conventional gradation curve using power law regression analyses as shown in Figure 2.9. The format of the power law is:

\[ P_{CA} = a_{CA}(d)^{n_{CA}} \] \[ P_{FA} = a_{FA}(d)^{n_{FA}} \]

where,

- \( P_{CA} \) and \( P_{FA} \) = percent by weight passing a given sieve that has an opening of width \( d \)
- \( a_{CA} \) = intercept constant for the coarse aggregate
- \( a_{FA} \) = intercept constant for the fine aggregate
- \( d \) = sieve opening width, mm
- \( n_{CA} \) = slope (exponent) for the coarse aggregate
- \( n_{FA} \) = slope (exponent) for the fine aggregates

A study was conducted to correlate the gradation parameters with mixture performances. Ten limestone mixtures with different gradations were designed according to Superpave procedures and criteria. These mixtures were compacted to 7 percent air void to prepare specimens for indirect tension testing. Tensile strength, fracture energy and failure strain parameters at 10°C were evaluated for test specimens subjected to short and long term oven aging. Aggregate gradation characterization factors were used in multiple linear regression analyses to establish relationships with the fracture energy and failure strain parameters.
The results of these analyses identified gradation characteristics that are detrimental to mixture properties. Specifically, gap graded or gradations with an excess amount of aggregate retained on a sieve did not yield properties equivalent to well balanced, continuously graded, aggregate blends. Greater asphalt content and percent passing the 4.75-mm sieve resulted in greater tensile strength and fracture energy (FE) for coarse graded mixtures and lower FE for fine graded, long term aged, mixtures. The failure strain of fine graded mixture improved with increase in asphalt content and percent passing the 4.75-mm sieve. Another key finding was the surface areas (SA) of the aggregate blends were found to be related to the aggregate characterization factors (percent aggregates passing the 2.36 mm sieve, nCA, nFA). The author suggests using SA predictions based on gradation factors and effective asphalt content to estimate film thickness.
2.3.4.4 The Bailey Method of Aggregate Blending and Evaluation

One of the methods that is attempting to rationalize the aggregate gradation procedure is the Bailey method of aggregate gradation evaluation (Vavrik et al. 2001, TRB Circular 2002). The Bailey method is a comprehensive gradation evaluation procedure to provide aggregate interlock as the backbone for the aggregate skeleton (Vavrik et al. 2001, TRB Circular 2002). In this method, the definition of coarse and fine aggregate is not based on the conventional No. 4 sieve (4.75 mm). Coarse aggregates are defined as the large aggregate particles that when placed in a unit volume, create voids. Fine aggregates are those particles that can fill the voids created by the coarse aggregates. The sieve that separates the coarse and fine aggregates is called the primary control sieve (PCS). It is dependent on the nominal maximum particle size of the aggregate blend. The PCS is mathematically defined as 0.22 of the NMAS based on two and three dimensional analysis of the packing of different shaped particles (Figure 2.10). Furthermore, the aggregate blend below the PCS is divided into coarse and fine portions, and each portion is evaluated (Figure 2.11). The method provides a set of tools that allows the evaluation of aggregate blends.

![Figure 2.10 Two Dimensional Aggregate Packing Model](NMPS= Nominal Maximum Particle Size)
Aggregate ratios, which are based on particle packing principles, and the relative proportions passing certain critical sieves, are used to analyze the particle packing of the overall aggregate structure. The coarse aggregate ratio (CA Ratio) is used to characterize the packing and size distribution of the coarse portion of the aggregate blend. The coarse portion of the fine aggregate is evaluated using the fine aggregate ratio of the coarse portion (FA_{c}), and the fine portion of the fine aggregate is evaluated using the fine aggregate ratio of the fine portion (FA_{f}). All these ratios are calculated using mathematical equations that relates the amount of aggregate passing specific critical sieve sizes. In Summary, the Bailey Method involves the following approach:

- Evaluates packing of coarse and fine aggregates individually
- Contains a definition for coarse and fine aggregate
- Evaluates the ratio of different size particles
- Evaluates the individual aggregates and the combined blend by volume
The end result is an aggregate blend that is packed together in a systematic manner to form an aggregate skeleton. The details of the method is presented in Appendix A.

2.4 Behavior of Asphalt Mixture

The asphalt mixture resulting from blending the previous two materials, asphalt cement and aggregate will display a behavior that has combined characteristics of both materials. Therefore, the response of the HMA to the applied load depends on the thermoplastic properties of the binder and the toughness and interlock characteristics of the aggregate. When load is applied to a pavement, the primary stresses that are transmitted to the asphalt mixture are vertical compressive stress within the asphalt layer, and horizontal tensile stress at the bottom of the asphalt layer (Asphalt Institute SP2, 2001) as shown in Figure 2.12. The HMA must be internally strong and resistant to the compressive and shear stresses to prevent permanent deformation within the mixture. In the same manner, the mixture must also have enough tensile stress at the base of the asphalt layer to resist fatigue cracking after many load applications.

![Figure 2.12 Illustrations of stresses and strains within pavement layers (White et al. 2002)](image)
The asphalt mixture must also resist thermal stresses caused by rabid fluctuation of temperature and extremely cold temperatures. The behavior of asphalt mixture as a structural component of the pavement is normally achieved by analyzing the mixture performance under laboratory controlled conditions of load and temperature.

2.5 Mixture Design Concept

The term asphalt mixture design is being often considered as synonymous with the selection of a specific binder content normally referred to as the optimum asphalt cement content. In reality however, this is only the last step once other important factors are considered. As mentioned earlier, HMA consists of two basic ingredients: aggregate and asphalt cement binder. HMA mix design is the process of determining what aggregate to use, what asphalt cement binder to use and what the optimum combination of these two ingredients should be. HMA mix design has evolved over the years as a laboratory procedure that uses several critical tests to make key characterizations of several trials HMA blends. Although these characterizations are not comprehensive, they are intended to give the mix designer a good understanding of how a particular mix will perform in the field during construction and under subsequent traffic loading.

Mixture design is a laboratory simulation of the actual HMA manufacturing, construction and performance to the extent possible. From this simulation, prediction (with some certainty) can be made of what type of mix design is best for the particular application in question and how it will perform.

Mixture design is also volumetric in nature. Volume measurements however, are made indirectly by determining a material's weight and specific gravity and then calculating its volume.
It is important to realize however that mix design has its limitations. Specifically, there are differences between laboratory and field conditions. Tables 2.1 through 2.4 summarize some of those differences. Certainly, a small laboratory setup consisting of several 100 - 150 mm (4 - 6 inch) samples, a compaction machine and a couple of testing devices cannot fully mimic actual manufacturing, construction and performance conditions. However, despite limitations, mix design procedures can be a cost effective tool that is useful in making mix design decisions.

2.5.1 Desirable Properties for Asphalt Concrete Mixtures

The design of asphalt paving mixture, as with the design of other engineering materials is largely a matter of selecting and proportioning constituent materials to obtain the desired properties in the finished pavement structure. The common requirements for any asphalt mixtures can be summarized as follow (Asphalt Institute MS-2, 1993):

- Resistance to permanent deformation: The mix should not distort or be displaced when subjected to traffic loads. This property is more important at high temperatures.
- Fatigue resistance: the mix should not crack when subjected to repeated loads over a period of time.
- Resistance to low temperature cracking. This mix property is important in cold regions Durability: the mix should contain sufficient asphalt cement to ensure an adequate film thickness around the aggregate particles. The compacted mix should not have very high air voids, which accelerates the aging process.
- Resistance to moisture-induced damage.
- Skid resistance: This is a functional requirement that is related to safe operation of vehicles on the HMA pavement.
• Workability: the mix must be capable of being placed and compacted with reasonable effort.

• Low noise and good drainage properties: If the mix is to be used for the surface (wearing) layer of the pavement structure.

Table 2.1 Laboratory vs. Field Conditions- Binder (NAPA 2001)

<table>
<thead>
<tr>
<th>Laboratory Conditions</th>
<th>Field Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aging is simulated using the TFO, RTFO or PAV. All of these methods are only rough simulations of actual asphalt binder aging.</td>
<td>Aging is much more complex - especially after construction when it is highly dependent upon construction quality and the environment.</td>
</tr>
<tr>
<td>After mixing the loose mix is generally aged to allow for asphalt binder absorption and in increase in viscosity.</td>
<td>After mixing the loose mix can be immediately transported to the construction site or can be placed in storage silos for up to a week.</td>
</tr>
</tbody>
</table>

The aforementioned properties are sometimes very difficult to achieve simultaneously, and therefore it is often necessary to settle for a compromise, when selecting the most appropriate mixture types and composition.

Table 2.2 Laboratory vs. Field Conditions (NAPA 2001)

<table>
<thead>
<tr>
<th>Laboratory Conditions- Aggregates</th>
<th>Field Conditions- Aggregates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gradation is carefully measured and controlled.</td>
<td>During the manufacturing process aggregate gradation will change slightly as it passes through the cold feed bins, aggregate dryer and drum mixer/pugmill.</td>
</tr>
<tr>
<td>Aggregate used is completely dry.</td>
<td>Even after drying, aggregates typically contains between 0.1 - 0.5 percent by weight</td>
</tr>
</tbody>
</table>
Table 2.2 Cont.

<table>
<thead>
<tr>
<th>Laboratory Conditions-Mixing Process</th>
<th>Field Conditions-Mixing Process</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fines are retained during the mixing process.</td>
<td>Some fines are collected in the mix plant baghouse.</td>
</tr>
<tr>
<td>Oven heating of the aggregate usually results in uniform heating of the coarse and fine aggregate.</td>
<td>In a drum plant there is often a distinct temperature difference between the coarse and fine aggregate.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Laboratory Conditions-Compaction</th>
<th>Field Conditions-Compaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>The mixing process occurs on essentially unaged asphalt binder for the Hveem and Marshall methods. The Superpave method roughly simulates short-term aging using the RTFO.</td>
<td>The mixing process can substantially age the asphalt binder. A mixing time of 45 seconds can increase asphalt binder viscosity by up to 4 times.</td>
</tr>
<tr>
<td>Compaction uses a laboratory device and a small cylindrical sample of HMA. This combination attempts to simulate the partial orientation achieved by field compaction with rollers.</td>
<td>Particle orientation and compactive effort can vary widely depending upon roller variables and the environment (e.g., temperature, wind speed).</td>
</tr>
<tr>
<td>Compaction is relatively quick (&lt; 5 minutes) and thus occurs at an almost constant temperature.</td>
<td>Compaction can take a significant amount of time (30 minutes or more in some cases) and thus occurs over a wide range of mix temperatures.</td>
</tr>
<tr>
<td>Compaction occurs against a solid foundation.</td>
<td>Compaction can occur against a range of foundations (solid, soft).</td>
</tr>
</tbody>
</table>
2.5.2 Mixture Design Methods

This section provides an overview of the mixture design methods that have been or being used by the asphalt industry. Generally, most of the mix design methods rely on experience and performance of mixes of known composition. Almost all mixture design methods include specimen fabrication and compaction in the mix design process to determine the mixture composition and volumetric properties.

2.5.2.1 Hubbard-Field Method

This method might be considered as the first formal design method for asphalt mixtures. It was originally developed to design sand-asphalt mixtures and later modified for aggregates (Roberts et al, 2002). The method included the compaction of 50.8 mm in diameter by 25.4 mm high specimens at a range of asphalt contents. Each specimen is heated to 140°F in a water bath and placed in a testing mold which is in turn placed in the 140°F water bath and a compressive load is applied at a rate of 2.4 in/min. The specimen is forced through a restricted orifice 44.5 mm in diameter. The maximum load sustained, in pounds, is the Hubbard-Field stability. After testing all of the prepared specimens, the average stability at each asphalt content is calculated and plotted to determine the optimum asphalt content. One of the problems reported in this method was the size of the test specimens which limit the use of large size aggregates greater than 12.7 mm since that would violate the ratio of 4:1 of mold diameter to maximum aggregate size (Roberts et al, 2002). The relevance of the measured stability to actual mixture performance is also a concern.

2.5.2.2 Hveem Mixture Design Method

This is one of the oldest mix design methods that dates back to 1927 when a California engineer, Francis Hveem began an extensive work to develop a mixture
design method that can be reliably used by asphalt engineers and that does not solely
depends on experience to reach to the optimum asphalt content (Asphalt Institute MS-2).
Hveem used the aggregate surface area concept to develop a methodology for predicting
the amount of asphalt needed for what used to be called oil mix (Hveem, 1942). The basis
of this method is that the proper amount of binder in a mix of different size particles
depends on the surface area of the gradation and that finer mixtures require higher binder
content. A design chart was developed that relates the surface area to what is called
bitumen index. Multiplication of surface area by the bitumen index gives the so called
the oil ratio which is simply the Kilo gram of oil (binder) per Kilo gram of aggregates.

A series of standard test specimens of 64.0 mm height and 102 mm diameter
compacted using a special mechanical kneading compactor, with binder contents that
vary around an estimated optimum value are subjected to several tests in order to arrive at
the actual optimum value. The tests Hveem used to judge the fitness of the compacted
mixtures were the stabilometer, cohesiometer and the swell test. The stabilometer is a
predecessor of the triaxial test that utilizes a special triaxial-type testing cell and used to
determine the stability of a mixture by measuring the radial expansion due to an axially
applied load. Naturally, an over-filled mixture would show relatively large deformations
and thus be judged unstable. The results from this test are expressed in a relative
stabilometer value, where a true liquid was considered to have zero relative stability
(lateral pressure equal to vertical pressure) while a non-deforming solid was the end of
the range (radial deformation of zero). To account for the influence of height versus
diameter ratio’s, Hveem established correction curves for specimens with non-standard
heights.
The second test Hveem used, the cohesiometer test, was basically a force controlled bending test. By dropping a controlled quantity of a material with a known weight per time unit in a container, the applied load steadily increased. The force necessary to arrive at a standard displacement of the loading arm is recorded as the cohesiometer value. If the bond between the aggregate particles is weak due to a lack of binder, the material will perform badly in this experiment. Finally, the swell test was be used to determine the sensitivity of the mixture to water. It measures the permeability and increase in volume due to absorption of water (swell). Hveem advised to aim for the maximum binder content that met the stability criteria and had no less than four percent air voids, to avoid the risk of (locally) over-filled material in the actual construction. The Hveem method entails a density/voids and stability analysis. The mixture’s resistance to swell in the presence of water is also determined. The Hveem method has two primary advantages. First, the kneading method of laboratory compaction is thought to better simulate the densification characteristics of HMA in a real pavement. Second, Hveem stability is a direct measurement of the internal friction component of shear strength. It measures the ability of a test specimen to resist lateral displacement from application of a vertical load. A disadvantage of the Hveem procedure is that the testing equipment is somewhat expensive and not very portable. Furthermore, some important mixture volumetric properties that are related to mix durability are not routinely determined as part of the Hveem procedure. Studies over several years established a relationship between Hveem stability and stable mix performance in the field (Roberts et al., 1996): stabilometer value <30 (unstable under traffic); stabilometer values between 30 and 35 (border line stability under traffic); stabilometer value >35 (stable under traffic). Hveem
stability characterization of asphalt concrete has been used by the road industry and is specified in ASTM D1561 and AASHTO T246.

2.5.2.3 Marshall Mixture Design Method

The basic concepts of the Marshall mix design method were originally formulated by Bruce Marshall of the Mississippi Highway Department around 1939 and then refined by the U.S. Army (Asphalt Institute MS-2, 1993). It was standardized by the American Society for Testing and Materials as ASTM D-1559 “Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus. Standard test specimens of 63.5 mm height by 100 mm diameter are used that are prepared using a standard procedure for heating, mixing, and compaction of the asphalt-aggregate blend. The key features of this method of mixture design are the compaction method, volumetric analysis and the Marshall stability and flow test of the compacted specimens. The compaction is achieved by applying an impact energy using a standard hammer to compact specimens with a number of blows that are related to the expected traffic conditions. These blows are 35, 50, and 75 for low (<10⁴ ESAL), medium (10⁴-10⁶ ESAL), and high (>10⁶ ESAL) respectively.

The Marshall stability is defined as the maximum load the sample can sustain at 60°C. The Marshall flow is defined as the vertical deformation of the sample in 0.01 inch units occurring at the point of maximum load. The Marshall method seeks to select the asphalt binder content at a desired density that satisfies minimum stability and range of flow values (Asphalt Institute, 1993). Basically, the Marshall stability and flow test provides the performance measure for the Marshall mix design method.

The volumetric analysis includes, calculating the volumetric parameters of the mixture specimen. Measured void expressions are: air voids (V_a), sometimes called
voids in the total mix (VTM), Voids in the Mineral Aggregate (VMA), and Voids filled with Asphalt (VFA). In summary, the design asphalt content is determined based up on:

- Minimum stability values based on traffic level
- A flow value based on traffic level
- Air void level in the range of 3-5 %
- A minimum VMA value
- A range of VFA values

A key advantage of Marshall method is that it requires equipment that is relatively inexpensive and very portable. The testing time is relatively short and can be conducted rapidly with little effort compared to the equipment used in the Hveem method. The volumetric analysis in this method also addresses to some extent the durability concern by specifying a range of volumetric parameters that were developed by experience and thought to act as a safeguard against environmental effects.

A major shortcoming of the Marshall method is that the impact compaction used with the method does not simulate mixture densification as it occurs in a real pavement. Furthermore, Marshall stability does not adequately estimate the shear strength of HMA. These two situations make it difficult to assure the rutting resistance of the designed mixture.

Attempts were made to refine the Marshall method by approaches that rationalize some of its parameters such as stability and flow. Baladi et al suggested a modified procedure to determine the optimum asphalt content (Baladi 1988). He defined a new term called equivalent Marshall stiffness as follows:

$$ ES = S/2[F_{0.5}] $$

where,
Baladi et al reported that the parameter ES correlated better with mixture’s resilient modulus (M_r). It was suggested to replace the Marshall stability by the equivalent Marshall stiffness to select the optimum asphalt content. In other words, the plot of Marshall stability versus asphalt content by the equivalent stiffness versus asphalt content. The design asphalt content could be then calculated as in the original procedure.

Lees (1987) also recommended a procedure to determine the design asphalt content using a “range” approach instead of the method of averaging. Selected parameters are plotted against the binder content similar to the original method. Additional plot is constructed on which are drawn the ranges of binder content over which the specified values of each of these parameters are satisfied. The mid point of the common overlap of all ranges is taken as the design binder content. The parameters that were used in Lees’s method are: Stability (a minimum value is set based on traffic), Flow (minimum value), Marshall stiffness (Stability/Flow- minimum values), air voids, and voids filled with asphalt (minimum and maximum limits). the criteria for the parameters used are based on traffic level.

2.5.3 Developments on Empirical Design Methods

Both mixture design procedures Hveem and Marshall are basically meant to compare a series of binder contents to find the optimum one for a specific aggregate gradation. Of the two methods, Hveem’s procedure is more comprehensive and provides a relatively fundamental approach towards mix characterization. The Marshall method determines the optimum binder content on the basis of a single mechanical test. The
Marshall method, which provides some indices with no clear relation to mix characteristics or behavior is more popular however, than the more fundamental Hveem procedure because of its portability and ease of use.

The link to pavement performance is, in both cases, established through ranges for the test parameters (stabilometer value, cohesiometer value, stability etc.). These ranges are based on experience. As a result, their applicability is limited to similar conditions of construction, climate, and traffic conditions.

Mahboub et al. (1990) presented a methodology to design and evaluate asphalt mixture based on some mechanistic tests to evaluate mixtures stiffness and permanent deformation characteristics of the mixtures. The hierarchy of their approach is as follows:

- Mixture design in accordance with a standard procedure. No specific procedure is recommended;
- Mixture stiffness characterization related to threshold resilient modulus for subgrade protection and stiffness/flexibility analysis for flexural fatigue; evaluation.
- Permanent deformation potential analysis; and
- Thermal cracking analysis

A series of charts that relate the traffic level and subgrade condition to the mechanistic parameters from the different tests conducted is used to evaluate the adequacy of the designed mixtures to be used as a road material. Examples of those charts are presented in Figure 2.13.

Cooper et al (1991) proposed a mixture design method that utilizes the percentage refusal density (PRD) apparatus for specimen compaction.
The PRD test was widely used in UK for determining the degree of compaction of materials laid on site. The description of this apparatus is available elsewhere (Cooper, 1991). In brief, cores are taken from the field and their density is evaluated. The same cores are then tested in the PRD apparatus and the density of the material at refusal is measured. The density of the original core should not be less than a certain percentage of refusal otherwise the materials from which the cores have been taken should be removed. The suggested mixture design procedure is based on three factors: gradation, binder content, and compaction level.

The design process involves determining a suitable gradation and binder content for optimum mechanical properties at a target compaction level. For gradation, the concept of equivalent fine content (EFC) was used. To define a target gradation, a
modification of the convention power law method was made. The modification was to enable gradation, and thus EFC, to be varied whilst maintaining filler material (minus 200 sieve) at a pre-selected and practical level. The grading equation used was as follows:

\[ P = \left[ \frac{(100-F)(d^n-0.075^n)}{(D^n-0.075^n)} \right] + F \]

where,

- \( P \)= percentage passing a sieve of size \( d \), mm
- \( D \)= maximum aggregate size, mm
- \( F \)= filler content (passing #200 sieve)
- \( N \)= an exponent between 0 and 1

Cooper et al (1991) found that exponents of 0.5, 0.6, and 0.7 were the most appropriate for base course materials used in his study in terms of having adequate voids in the mineral aggregates VMA. In this design procedure, three compaction levels are recommended: 100% PRD, 97% PRD, and 93% PRD. The 93% PRD was considered as the minimum acceptable level and the 96% as the optimum. The specimens are prepared with three different levels of binder contents; 3.5, 4.1, and 4.7%. Those levels cover the range of binder contents commonly used for base materials in UK. Once the specimens are compacted, the volumetric properties are analyzed and compared to criteria that were developed based on experience for mixtures used as base course material. The criteria include VMA, air voids, and binder volume. Mixtures that do not meet those criteria are removed from further consideration in the mixture design process. Mixtures that meet the volumetric criteria are further tested to determine their mechanical properties. Two tests are conducted on the mixtures; the elastic stiffness using the repeated load indirect tensile test (RLIT) and the resistance to permanent deformation.
using the repeated load axial test (RLT). The design criteria were defined as the minimum acceptable level of performance based on those two tests. For the elastic stiffness, a minimum of 2500 MPa at 20°C with a load rise of approximately 0.12 seconds was recommended. The permanent deformation criteria were set as a maximum of 1.0% axial strain after 3600 applications of an axial stress of 100 KPa at 40°C.

Realizing the importance of incorporating mixture design to structural design and pavement performance variables, NCHRP project 9-6 was initiated (Quintus 1992). The result of the this project was the development of an Asphalt-Aggregate Mixture Analysis System (AAMAS) for evaluating dense-graded asphalt concrete mixtures proposed for use primarily on high-volume roadways and in a mixture design procedure based on performance related criteria (Quintus 1992).

The AAMAS consists of three basic laboratory steps (Quintus 1992). The first step is the initial mixture design phase which can utilize any current mixture design procedure that is in practice without specifically advocating one over another. The second step involves mixing and compacting materials. This step involves age hardening simulations, moisture conditioning and compaction. After the materials have been mixed, compacted, and conditioned, the specimens are tested in the third step to measure critical mixture properties. The third step is basically the mixture evaluation phase which is meant to provide data that can be integrated into pavement design and analysis models to predict pavement performance. Six tests are used as tools for mixture evaluation in AAMAS. These tests are presented in Table 2.5. The method recommends the use of some empirical relationships that were developed between asphalt cement content and VMA, air void content, and film thickness for specific aggregate blends to estimate the
starting or what is called the seed asphalt content that is used as the median of the range of asphalt contents to be used in the design process.

Table 2.5 Laboratory tests in the AAMAS (after Mamlouk et al 1992)

<table>
<thead>
<tr>
<th>Test Name</th>
<th>Loading Pattern</th>
<th>Loading Condition</th>
<th>Test Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diametral resilient modulus</td>
<td>Pulses with 0.1 sec duration and 0.9 sec rest period</td>
<td>Diametral</td>
<td>Indirect resilient modulus</td>
</tr>
<tr>
<td>Indirect tensile strength</td>
<td>Loading at a rate of deformation of 0.05in/min (41°F) or 2in/min (77°F) until failure</td>
<td>Diametral</td>
<td>Indirect tensile strength, strain at failure</td>
</tr>
<tr>
<td>Indirect tensile creep</td>
<td>Static load of a specified magnitude for 60 min and unloading for 60 min</td>
<td>Diametral</td>
<td>Tensile creep modulus</td>
</tr>
<tr>
<td>Uniaxial compression resilient modulus</td>
<td>Pulses with 0.1 sec duration and 0.9 sec rest period</td>
<td>Axial</td>
<td>Total resilient modulus</td>
</tr>
<tr>
<td>Unconfined compressive strength</td>
<td>Loading at a rate of deformation of 0.6in/min until failure</td>
<td>Axial</td>
<td>Compressive strength</td>
</tr>
<tr>
<td>Uniaxial creep</td>
<td>Static load of a specified magnitude for 60 min and unloading for 60 min</td>
<td>Axial</td>
<td>Compressive creep modulus</td>
</tr>
</tbody>
</table>

A summary of the procedure is as follows. Nine test specimens at selected asphalt contents are tested. Three specimens per asphalt content are tested at 77°F (25°C) using the indirect tensile testing techniques to define the fracture characteristics of the specimens. A second test of three specimens at each asphalt content are compacted and tested using the Corps of Engineers Gyratory Testing Machine (GTM). The GTM device is used to estimate the change in shear characteristics under repeated loads at 140°F
(60°C). Using the results of the indirect tensile and the gyratory shear tests as a guide, a final set of three specimens at selected asphalt contents are tested at 140°F using uniaxial compression tests to define the deformation and creep characteristics of the mixture. The design asphalt cement content and the range of allowable values are determined from these test results for the specific aggregate gradation. The design asphalt cement content is defined as being those values that are within the minimum and maximum limits as established by the fatigue, shear, and deformation criteria. These criteria include a minimum creep modulus, a minimum gyratory shear strength, and a minimum tensile strength at failure for fatigue.

Abdulshafi et al (1999) conducted a study to optimize the design of asphalt mixtures used as intermediate layers between the surface and base course layers. 25.0 mm mixtures were designed using two aggregate types of natural gravel and limestone. The asphalt cement binder type used was not reported in the aforementioned reference. For each type of aggregate, five gradations were developed using aggregate blending method utilized in the design of portland cement concrete mixtures. That was a key step in his modified mixture design method. The volumetric gradation charts recommended by two reports from the Strategic Highway Research Program (SHRP) that describes the development of a dry packing models for optimization of volumetric mix design of portland cement concrete mixtures were followed(Roy et al. 1993). The concept is that optimization can be achieved through careful volumetric proportioning of all aggregate sizes and other mixture components to achieve maximum density and stone-on-stone contact in the material matrix. In the volumetric power chart, the y-axis is the volume percentage of all mix components (not only aggregates) that pass through each sieve size. The x-axis is the sieve size raised to 1/3 power. The weight percent of aggregate passing
each sieve size is calculated from the total mix volume percent passing, based on the individual volume percentages and specific gravity values of the various mix components. Using the 37.5 mm gradation line from the 1/3 power and based on experience, gradation limits were established for the mixtures designed. No clear methodology was given to describe how those limits were reached in the above referenced publication. The Marshall mix design method was used to determine the optimum asphalt content. Resilient modulus and indirect tensile strength tests were then performed on the designed mixtures. Limits were set for the asphalt content and the parameters from the mechanical tests based on the results from this study. The author recommends the use of coefficient of curvature ($C_c$) and coefficient of uniformity ($C_u$) as defined by the United Soil Classification System (Bowels 1992) for the aggregate gradation. He recommends that $C_c$ shall be between 1 and 3 and $C_u$ is greater than 30 for the combined gradation to be used in the job mix formula.

2.6 Superpave Mixture Design

Superpave is an acronym for Superior Performing Asphalt Pavements and was introduced as the result of the Strategic Highway Research Program (SHRP) in 1993 (Roberts, 1996). In 1987, SHRP was initiated with an allotted budget of $150 million for a 5-year period. One of the major objectives of this program was to develop an improved mix design procedure that can be applied to various traffic volumes, axle loads and climatic conditions. As a product of the SHRP research in 1993, a new mix design method called “Superpave” was introduced. The main features included a new grading system for the asphalt cement binders named Performance Grading (PG) system, aggregate specifications, a new compaction procedure, and mixture testing and analysis procedures (Huber 1993; McGennis 1995; Roberts et al. 1996).
Figure 2.14 Aggregate Gradation Using 1/3 Power Chart (Abdulshafi et al, 1999)

Figure 2.15 Volumetric 1/3 Power Chart for Asphalt Concrete Mix Design (Abdulshafi et al, 1999)

The Superpave system was meant to address minimization of three pavement distresses: rutting, fatigue cracking and low temperature cracking. Moisture sensitivity and binder aging are also considered in material selection and mix design (Huber 1993; Cominsky 1994; McGennis 1995).
2.6.1 Superpave Performance Grading System

In the PG grading system, the binders are specified based on the climate and the chosen level of reliability. Several new tests were proposed to evaluate asphalt cement binder (Table 2.6). The requirements for the physical properties of the asphalt binders are the same, whereas the temperature at which the binder is expected to achieve the properties changes depending on the climate (McGennis, 1995). The PG binders are specified in the form PG X-Y. The first number ‘X’ is referred to as the high temperature grade and it represents the temperature at which the particular binder should possess adequate physical properties. This temperature would be the average 7-day maximum pavement temperature in degrees Celsius expected for the considered project. The second number ‘-Y’, represents the lowest temperature at which this binder is expected to serve and the temperature at which the binder possess sufficient flexibility to prevent cracking. For example, PG 76-22 can be used with good performance characteristics for climate where maximum temperature of the pavement would be 76°C and the minimum temperature would be -22°C.

<table>
<thead>
<tr>
<th>Procedure/Equipment</th>
<th>Purpose</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dynamic Shear Rheometer (DSR)</td>
<td>Measures properties at high and intermediate service temperature</td>
</tr>
<tr>
<td>Rotational Viscometer (RV)</td>
<td>Measures properties at high construction temperature</td>
</tr>
<tr>
<td>Bending Beam Rheometer (BBR)</td>
<td>Measures properties at low service temperature</td>
</tr>
<tr>
<td>Direct Tension Test (DTT)</td>
<td></td>
</tr>
<tr>
<td>Rolling Thin Film Oven Test (RTFO)</td>
<td>Simulates short-term aging of asphalt cement binder</td>
</tr>
<tr>
<td>Pressure Aging Vessel (PAV)</td>
<td>Simulates aging under long-term service</td>
</tr>
</tbody>
</table>
2.6.2 Superpave Specifications on Aggregate

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

2.6.3 Superpave Laboratory Compaction

A key feature in the Superpave system is laboratory compaction. In the mix design procedure, a Superpave gyratory compactor (SGC) is used to carry out the compaction of the Superpave mixture samples in the laboratory. SGC was found to be effective in simulating the field compaction and ensures that the properties of the samples compacted in the laboratory are to some extent similar to the mix placed in the field (Cominsky, 1994). The Superpave gyratory compactor is capable of monitoring the rate...
of densification during compaction. A hydraulic or mechanical system applies a load to the loading ram, which applies 600 kPa compaction pressure to the specimen (Figure 2.16). The loading ram diameter nominally matches the inside diameter of the mold, which is normally 150 mm for design purposes. The ram pressure is monitored by a pressure gauge during compaction. As the specimen densifies, the pressure gauge and loading ram maintain compaction pressure.

The design number of gyrations depends upon the traffic level for which the mix is designed. Higher compaction energy is applied to mixtures in the heavy traffic category (Table 2.7). The analysis of the compacted samples is done in terms of percent of theoretical maximum specific gravity at three levels of compaction. These levels are (D’Angelo, 1995):

- **N_{initial}**: The number of gyrations used as a measure of mixture compactability during construction. Mixes that compact too quickly (air voids at N_{initial} are too low) may be tender during construction and unstable when subjected to traffic. A mixture with 4 percent air void at N_{design} should have at least 11 percent air voids at N_{initial}.

- **N_{design}**: This is the design number of gyrations required to produce a sample with the same density as that expected in the field after the indicated amount of traffic. A mix with 4 percent air voids at N_{design} is desired in mix design.

- **N_{max}**: The number of gyrations required to produce a laboratory density that should never be exceeded in the field. If the air voids at N_{max} are too low, then the field mixture may compact too much under traffic resulting in excessively low
air voids and potential rutting. The air void content at $N_{\text{max}}$ should never be below 2 percent air voids.

Table 2.6 presents the compaction requirements for different traffic levels. Typical results from the Superpave Gyratory Compactor are shown in Figure 2.17.

The data from the SGC is generally used in computing volumetric properties such as density or air void content as a function of compaction gyrations. However, several attempts were recently made to analyze the densification curve obtained from the SGC in order to evaluate the asphalt mixtures workability and resistance to permanent deformation. The initial number of gyrations ($N_{\text{initial}}$) and the slope of the initial portion of the SGC compaction curve have been hypothesized to reveal certain mixture properties such as tenderness of the mixtures and the strength of aggregate structure (McGennis, 1995).
Bahia et al. (1998) suggested that the current method of interpretation of the results from the SGC and the design criteria are biased toward the performance under traffic and do not adequately consider the constructability of mixtures. He proposed the use of the SGC curve to evaluate the constructability of the mixtures as well as their resistance to traffic loading.

![Figure 2.17 Typical Results from the Superpave Gyratory Compactor](image)

Table 2.7 Superpave Gyration Levels

<table>
<thead>
<tr>
<th>Design ESALs (millions)</th>
<th>N&lt;sub&gt;initial&lt;/sub&gt;</th>
<th>N&lt;sub&gt;design&lt;/sub&gt;</th>
<th>N&lt;sub&gt;maximum&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;0.3</td>
<td>6</td>
<td>50</td>
<td>75</td>
</tr>
<tr>
<td>0.3 to &lt;3.0</td>
<td>7</td>
<td>75</td>
<td>115</td>
</tr>
<tr>
<td>3.0 to &lt;30.0</td>
<td>8</td>
<td>100</td>
<td>160</td>
</tr>
<tr>
<td>≥ 30.0</td>
<td>9</td>
<td>125</td>
<td>205</td>
</tr>
</tbody>
</table>

Bahia introduced the concept of compaction and traffic energy indices (CDI, TDI). The compaction energy index and the traffic densification index are used to relate to construction and in-service performance of HMA mixtures. The CDI is defined as the area under the densification curve from N= 8 to N corresponding to 92% G<sub>mm</sub> (percent maximum theoretical specific gravity). The TDI is defined as the area from the 92% G<sub>mm</sub>
point to the terminal density of 98% $G_{mn}$. Figure 2.18 illustrates the concept of the compaction indices.

Figure 2.18 Energy Indices from the Superpave Gyratory Compactor

Bahia suggested that controlling these indices is expected to allow optimization of HMA construction and traffic requirements. Guler et al. (Guler et al. 2000) later developed a gyratory load cell and plate assembly (GLPA) for measuring HMA shear resistance during compaction with any SGC. It is a thin cylindrical device that is inserted on top of the mixture in the compaction mold that gives continuous measure of shear resistance under gyratory loading during compaction (Figure 2.19). They hypothesized that bulk shear resistance from the GLPA is a good indicator of the compactability of HMA mixtures and their potential resistance to rutting under traffic. They reported that shear resistance is highly sensitive to gradation, asphalt content, and temperature. It was concluded that the device offers potential as a low-cost tool to complement volumetric properties from the SGC by evaluating the compactability of asphalt mixtures as well.
Mallick (1999) found that the gyratory ratio, the ratio of the number of gyrations required to achieve 2 percent voids and 5 percent voids, was suitable for characterizing HMA. He stated that a gyratory ratio of 4 can be used to differentiate between stable and unstable mixes. Anderson (14) evaluated several SGC compaction parameters and found that the best parameter related to asphalt mixture shear stiffness and rutting potential was $N_{-SR_{\text{max}}}$, defined as the number of gyrations at which the stress ratio (shear stress divided by vertical stress) reaches a maximum value. He measured $N_{-SR_{\text{max}}}$ using a Pine model AFG1 SGC modified with a shear measurement system that produces a unitless stress ratio. Several HMA mix variations of gravel and limestone were used to demonstrate the utility of $N_{-SR_{\text{max}}}$ and to identify threshold values for separating mixtures with good and poor expected performance. It was concluded that none of the evaluated SGC parameters appeared to be capable of identifying differences in mixture performance based on asphalt binder stiffness. Also, $N_{-SR_{\text{max}}}$ is not intended to replace the need for actual mechanical property testing but to identify if and when further performance-related testing is needed.
2.6.4 Determining the Optimum Asphalt Content

Before compaction, the loose mixture is kept in an oven at a compaction temperature for two hours. This process is meant to simulate the mixing and placement of asphalt mixture in the field and the absorption of asphalt by the aggregates (Cominsky 1994; Harrigan 1994; McGennis 1995).

The optimum asphalt content is determined according to two sets of criteria: volumetric and densification. Volumetric requirements include: air Voids in Total Mix (VTM), Voids in Mineral Aggregate (VMA) and Voids Filled with Asphalt (VFA) based on traffic and nominal maximum aggregate size in the mixture (Cominsky 1993; Cominsky 1994; D’Angleo 1995). All these parameters are defined in the following section of this chapter. Table 2.8 and Table 2.9 present the VMA and VFA requirements respectively.

<table>
<thead>
<tr>
<th>Nominal Maximum Aggregate Size (mm)</th>
<th>Minimum VMA, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.5</td>
<td>15.0</td>
</tr>
<tr>
<td>12.5</td>
<td>14.0</td>
</tr>
<tr>
<td>19.0</td>
<td>13.0</td>
</tr>
<tr>
<td>25.0</td>
<td>12.0</td>
</tr>
<tr>
<td>37.5</td>
<td>11.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Design ESALs (millions)</th>
<th>Design VFA, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;0.3</td>
<td>70-80</td>
</tr>
<tr>
<td>0.3 to &lt;3.0</td>
<td>65-78</td>
</tr>
<tr>
<td>3.0 to &lt;10.0</td>
<td>65-75</td>
</tr>
<tr>
<td>10.0 to &lt;30</td>
<td>65-75</td>
</tr>
<tr>
<td>≥ 30</td>
<td>65-75</td>
</tr>
</tbody>
</table>
For the densification criteria, the Superpave method requires that the designed mixture, at the design number of gyrations \( N_{\text{des}} \), have a density of 96 percent of \( G_{\text{mm}} \) or 4 percent air voids. The mixture cannot achieve a density of above 89 percent of \( G_{\text{mm}} \) at the initial level of compaction \( N_{\text{ini}} \). Figure 2.20 presents the Superpave densification criteria.

The trial mixes are also subjected to a moisture sensitivity test using the AASHTO T 283 tests or SHRP M-006 Method of test (Cominsky 1994; Harrigan 1994).

2.6.5 Description of the Volumetric Parameters in Superpave

Physical volumetric properties are commonly used when designing asphalt concrete. Basic HMA weight-volume relationships are important to understand for both mix design and construction purposes. Fundamentally, mix design is meant to determine the volume of asphalt binder and aggregates necessary to produce a mixture with the
desired properties (Roberts et al., 1996). However, since weight measurements are typically much easier, they are typically measured then converted to volume by using specific gravities. Figure 2.21 is a schematic illustration of the different terminologies used in asphalt mixture design. The following is a brief discussion of some volume properties of HMA.

### 2.6.5.1 Bulk Specific Gravity

The ratio of the mass in air of a unit volume of a permeable material (including both permeable and impermeable voids normal to the material) at a standard temperature to the mass in air of an equal volume of gas-free distilled water at the same temperature. This value is used to determine weight per unit volume of the compacted mixture. It is very important to measure $G_{mb}$ as accurately as possible. Since it is used to convert weight measurements to volumes, any small errors in $G_{mb}$ will be reflected in significant volume errors, which may be undetected.

When the total aggregate consists of separate fractions of coarse aggregate, fine aggregate, and mineral filler, all having different specific gravities, the bulk specific gravity for the total aggregate is calculated. The following equation is used to calculate the specific gravity of an aggregate blend:

$$G_{sb} = \frac{(P_1 + P_2 + P_3)}{\left( \frac{P_1}{G_1} + \frac{P_2}{G_2} + \frac{P_3}{G_3} \right)}$$

where,

$G_{sb} =$ combined bulk specific gravity of the aggregate blend
\( P_1, P_2, P_3 = \) percent of different aggregates in the mix

\( G_1, G_2, G_3 = \) bulk specific gravities of the aggregate in the mix

The standard bulk specific gravity test is AASHTO T 166: Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface-Dry Specimens

2.6.5.2 Theoretical Maximum Specific Gravity

The theoretical maximum specific gravity, or theoretical maximum density, is the density of an asphalt concrete mix if all air voids were removed, or the highest possible
density of the mix. Theoretical maximum specific gravity is employed in calculating the amount of voids in an asphalt concrete mix and providing target density values for pavement compaction during construction (Roberts et al. 1996). This parameter is measured experimentally using the test procedure described in AASHTO T 209 and ASTM D 2041: “Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures”.

2.6.5.3 Effective Specific Gravity

The effective specific gravity of aggregate, $G_{se}$ includes all void spaces in the aggregate particles, except those that absorb asphalt. It is calculated as follows:

$$G_{se} = \frac{P_{mm} - P_b}{P_{mm} - P_b} = \frac{G_{mm}}{G_b}$$

where,

$G_{se}$ = effective specific gravity of aggregates

$P_b$ = total asphalt content, %

$P_{mm}$ = Percent of total mix, = 100%

$G_{mm}$ = theoretical Maximum Specific Gravity

2.6.5.4 Asphalt Absorption

Absorption is expressed as a percentage by mass of aggregate rather than as a percentage of total mixture. Absorption is determined by the equation:

$$P_{ba} = 100 \times \frac{G_{se} - G_{cb}}{G_{cb} G_{se}} \times G_b$$

where,

$P_{ba}$ = absorbed asphalt, %

$G_{se}$ = effective specific gravity of aggregates
\(G_{sb}\) = bulk specific gravity of aggregates

\(G_b\) = specific gravity of the binder

### 2.6.5.5 Effective Asphalt Content

Effective asphalt content is the total asphalt content minus the amount of asphalt lost to absorption in the aggregates. Any binder that is absorbed into the aggregate particles does not play a part in the performance characteristics of the mix, but has the effect of changing the specific gravity of the aggregate.

It is expressed as in the equation:

\[
P_{be} = P_b - \frac{P_{ba}}{100} \times P_s
\]

where,

\(P_{be}\) = effective asphalt content, %

\(P_b\) = total asphalt content, %

\(P_{ba}\) = absorbed asphalt, %

\(P_s\) = total aggregate in the mix, %

### 2.6.5.6 Percent VMA in Compacted Paving Mixture (VMA)

Voids in the mineral aggregate (VMA) is the total void space between the aggregate particles in compacted asphalt concrete, including air voids and asphalt not absorbed by the aggregates. Voids in the mineral aggregate (VMA) is among the primary factors evaluated in this research. The inter-granular void space between aggregates in a compacted mixture, which includes air voids and effective asphalt content, is considered to be very important for the durability of a compacted paving mixture. The voids are calculated from bulk specific gravity of the aggregate and are expressed as a percentage.
of the bulk volume of the compacted mixture. Thus, the VMA can be calculated by subtracting the volume of the aggregate determined by its bulk specific gravity from the bulk volume of the compacted paving mixture. The calculations are performed as in the following equation:

\[ VMA = 100 - \frac{G_{mb} \times P_s}{G_{sb}} \]

where,

- \( G_{mb} \) = bulk specific gravity of the compacted mix
- \( P_s \) = total aggregate in the mix, %
- \( G_{sb} \) = bulk specific gravity of aggregates

Several researchers have reported that difficulties exist in meeting the minimum voids in VMA requirements (Kandhal et al. 1998, Coree 1999, Anderson et al. 2001). Under current specifications, many otherwise sound mixtures are subject to rejection solely because they failed to meet the VMA requirement. Studies (Musselman 2001, Coree 2001) also show that VMA requirement based on nominal maximum aggregate size (NMAS) does not take into account the gradation of the mixture, ignores the film thickness of the asphalt binder and thus is insufficient by itself to correctly differentiate between good-performing mixtures and poor-performing ones. Meanwhile, higher VMA mixtures cannot guarantee to provide better Superpave mixtures that are durable, fatigue and rutting resistant than the lower ones. Table 2.10 is a summary of findings of some of the research studies that evaluated VMA.

Rather than specifying a minimum VMA requirement based on asphalt content, Kandhal and Chakraborty (1996) directly specify a minimum average film thickness to ensure durability of asphalt mixtures for various types of mixtures and quantified the
relationship between various asphalt film thickness (ranging from 4 to 13 microns) and the aging characteristics of a dense-graded HMA mixture.

Table 2.10 Summary of Some Research Findings on VMA

<table>
<thead>
<tr>
<th>Researcher</th>
<th>Main findings</th>
</tr>
</thead>
</table>
| Hinrichsen, 1996 | Questioned the use of rigid minimum VMA requirements  
                  | There is a considerable variability in the tests performed to determine VMA  
                  | Mixes based on minimum VMA are not always the best in terms of performance and economics  
                  | Proposed using the film thickness instead                                                                 |
| Kandhal et al, 1998 | Meeting VMA requirements is difficult  
                  | Suggested the use of film thickness instead                                                                 |
| Coree, 1998      | Provided a thorough review on the development of the VMA criteria  
                  | Recommended that the minimum VMA criteria need to be validated against pavement performance  
                  | Rigid enforcement of a minimum VMA requirements is discouraged                                                                 |
| Mallick, 2000    | Aggregat gradation rather than the maximum aggregate defines the VMA requirement  
                  | Specifying VMA by percent passing the 2.36 mm sieve                                                                 |
| Coree., 2000     | Currently defined VMA criterion is seen to be insufficient by itself to correctly differentiate between sound and unsound mixtures  
                  | A composite of aggregate size, gradation shape, texture are more influential                                                                 |
Table 2.10 Cont.

<table>
<thead>
<tr>
<th>Authors</th>
<th>Information</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anderson M., 2001</td>
<td>Acknowledged the difficulties faced by asphalt designers in meeting VMA requirements. Higher VMA are not necessary for coarse coarse mixtures and may result in poor performance. Fine mixtures are less sensitive to change in VMA.</td>
</tr>
<tr>
<td>Nukunya et al. 2001</td>
<td>The rate of binder hardening was not related to VMA. Low VMA did not result in durability or cracking problems in fine-graded mixtures. Low VMA was not related to fracture resistance or rutting for coarse-graded mixtures. The relevance of Superpave VMA criterion must be seriously questioned.</td>
</tr>
</tbody>
</table>

They used the SHRP aging procedure to simulate both short- and long-term aging of HMA mixtures and concluded that the optimum film thickness for HMA compacted to 4 to 5% air void content in service should be somewhat lower than 9 to 10 microns because the rate of aging would be considerably lower at 4 to 5% air voids than when compacted to 8% air voids. Based on their past research experience, an average film thickness of 8 microns was recommended.

2.6.5.7 Percent Air Voids in a Compacted Mixture

The total volume of the small pockets of air between the coated aggregate particles throughout a compacted paving mixture, expressed as a percent of the bulk volume of the compacted paving mixture. The amount of air voids in a mixture is extremely important and closely related to stability and durability. Air voids below...
about 3 percent result in an unstable mixture while air voids above about 8 percent result in a water-permeable mixture. The air void in the mix is calculated as follows:

\[ V_a = 100 \times \frac{G_{mm} - G_{mb}}{G_{mm}} \]

VTM contribute to the thermal stability of compacted asphalt concrete by allowing for thermal expansion of asphalt cement between the aggregate particles as well as volumetric strain under repeated heavy traffic loading (Huber 1989). High VTM however, can decrease the durability of a pavement by allowing water and air to permeate the mix, increasing the oxidization and stripping potential, resulting in reduced mixture durability (Linden et al. 1989, Abdullah et al. 1998). Insufficient VTM may cause aggregate particles to lose contact with each other due to asphalt cement expansion at elevated temperatures, resulting in a loss of stability and increased potential for rutting under traffic load. For a given aggregate gradation, VTM are controlled by asphalt cement content, compaction effort during construction, and compaction under traffic loading (D.Angelo 2001).

2.6.5.8 Percent VFA in Compacted Mixture

Voids filled with asphalt cement (VFA) is the percent of the VMA that is filled with asphalt cement (Roberts et al. 1996). VFA is inversely related to air voids: as air voids decrease, the VFA increases.

It is determined using the following equation:

\[ VFA = 100 \times \frac{VMA - V_a}{VMA} \]
2.6.5.9 Effective Film Thickness ($T_{\text{eff}}$)

Asphalt cement film thickness is the average thickness of the asphalt cement layer covering each aggregate particle in the asphalt concrete mix. Film thicknesses are typically calculated based on the effective asphalt content and the surface area of aggregates as determined from the aggregate gradation and surface area factors (Asphalt Institute MS-2 1993). The calculated asphalt film thickness is the volume of the effective asphalt divided by the calculated surface area of the aggregate.

Film thickness has been linked to mix durability (Kandhal et al. 1996, Roberts et al. 1996). Asphalt concrete with relatively insufficient film thickness is generally considered to be more susceptible to oxidation which causes the mix to become brittle, reducing cracking resistance. In addition, thinner asphalt cement films may be more easily penetrated by water causing moisture induced damage. The definition of “insufficient” is somewhat controversial and a consensus has not been reached on the proper level of film thickness. In general, reported film thickness values are often termed “average film thickness” without a proper reference to whether it is the total film thickness or the effective one. That sometimes leads to confusion on what values of film thickness to be considered suitable. For the same asphalt content, film thickness decreases as the surface area of the aggregate is increased (finer aggregates).

2.6.5.10 Dust Proportion

Another Superpave mixture requirement is the dust proportion. This is computed as the ratio of the percentage by weight of aggregate finer than the 0.075 mm sieve to the effective asphalt content expressed as a percent by weight of total mix. Effective asphalt content is the total asphalt used in the mixture less the percentage of absorbed asphalt. Dust proportion is used during the Superpave mixture design as a design criterion. An
acceptable dust proportion is in the range from 0.6 to 1.6, inclusive for all mixtures (Asphalt Institute, 2001). Low dust proportion values are indicative of mixtures that may be unstable, and very high dust proportion values are believed to indicate mixtures that lack sufficient durability.
CHAPTER 3. MATERIALS AND TEST METHODS

3.1 Introduction

This chapter provides detailed information on the materials used and their properties. It also highlights the laboratory procedures for the tests performed.

3.2 Materials

Asphalt mixture is a composite material that is largely made of two main components; aggregate and asphalt cement. This section describes the properties of the aggregates and the asphalt cement binders used.

3.2.1 Aggregates

Sources of aggregate were selected to encompass a wide range of aggregates typically used in the State of Louisiana. Three aggregate types were used. These are:

- Hard aggregates (crushed granite),
- Water absorptive, high friction aggregate (sandstone) and,
- Low friction, low water absorption aggregate (limestone aggregate).

Different stockpiles from each type of aggregates were acquired. Natural coarse sand was used whenever necessary in the final design blends. Aggregates were acquired in 50 gallons barrels and kept properly sealed from any moisture intrusion. Detailed laboratory evaluation procedures of individual stockpiles were conducted to determine the basic aggregate properties such as specific gravity, gradation, and other Superpave consensus properties. The laboratory tests conducted on each aggregate stockpile included:

- washed sieve analysis (AASHTO T 11) to determine as-received gradation
- specific gravity and absorption (AASHTO T 85 for coarse aggregate and AASHTO T 84 for fine aggregate)
o unit weight (AASHTO T19)
o coarse aggregate angularity (ASTM D 5821)
o fine aggregate angularity (AASHTO T 304)
o flat and elongated (ASTM D 4791), and
o sand equivalency test (AASHTO T176).

The results from all of these tests are tabulated as Appendix 2 for all the aggregates acquired for this study.

Larger sizes stockpiles were sieved into individual size fractions. Materials retained on 1”, 3/4”, 1/2”, 3/8”, No. 4, and passing No. 4 sieves were stored in separate containers so that the required gradations could be batched directly from the individual size fractions. This method of aggregate separation, while somewhat time and labor intensive, allows for strict control and exact replication of mixture’s aggregate gradation.

3.2.2 Asphalt Binders

Two binder types were used in this study. SB polymer-modified asphalt binders meeting Louisiana PG specifications of PG76-22M for high-volume traffic mixtures (greater than 30 million equivalent single axle load; EASLs) and PG70-22 for Low volume category (less than 0.3 million equivalent single axle load; EASLs). Tables 3.1 presents the laboratory test results on the selected binders.

3.3 Aggregate Structure Design

The main aim of this task was to design the aggregate structures using an analytical aggregate gradation method that will allow a rational blending of different sizes of aggregate to achieve a densely packed aggregate skeleton in order to minimize the binder content and maximize the volume filled by mineral aggregates for stiffness and bearing capacity purposes. The Bailey method for aggregate gradation evaluation was
utilized for this purpose. Three aggregate structures were designed for each aggregate type (coarse, medium, fine). The structures for the heavy traffic category mixtures were designed to meet the recommended ranges of the Bailey method parameters. Low volume mixtures were designed using limestone aggregates only and have higher amounts of natural sand (>20%).

Table 3.1 LADOTD Performance Graded Asphalt Cement Specification & Test Results

<table>
<thead>
<tr>
<th>Property</th>
<th>AASHTO Test Method</th>
<th>PG 76-22M Spec.</th>
<th>Test Results</th>
<th>PG 70-22M Spec.</th>
<th>Test Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rotational Viscosity @ 135°C, Pa.s</td>
<td>TP 48</td>
<td>3.0-</td>
<td>1.7</td>
<td>3.0-</td>
<td>0.9</td>
</tr>
<tr>
<td>Dynamic Shear, 10 rad/s, G*/Sin Delta, KPa</td>
<td>TP 5</td>
<td>1.00+@82°C</td>
<td>1.29</td>
<td>1.00+@76°C</td>
<td>--</td>
</tr>
<tr>
<td>Flash Point, °C</td>
<td>T 48</td>
<td>232+</td>
<td>305</td>
<td>232+</td>
<td>295</td>
</tr>
<tr>
<td>Solubility, %</td>
<td>T 44</td>
<td>99.0+</td>
<td>99.5</td>
<td>99.0+</td>
<td>99.6</td>
</tr>
<tr>
<td>Force Ductility Ratio (F2/F1, 4°C, 5 cm/min, F2 @ 30 cm elongation)</td>
<td>T 300</td>
<td>0.30+</td>
<td>0.49</td>
<td>0.30+</td>
<td>0.31</td>
</tr>
<tr>
<td>Mass loss, %</td>
<td>T 240</td>
<td>1.00-</td>
<td>0.08</td>
<td>1.00-</td>
<td>0.03</td>
</tr>
<tr>
<td>Dynamic Shear, 10 rad/s, G*/Sin Delta, KPa</td>
<td>TP 5</td>
<td>2.20+@82°C</td>
<td>1.67</td>
<td>2.20+@76°C</td>
<td>1.65</td>
</tr>
<tr>
<td>Elastic Recovery, 25°C, 10 cm, % elongation, %</td>
<td>T 301</td>
<td>60+</td>
<td>70</td>
<td>40+</td>
<td>65</td>
</tr>
<tr>
<td>Dynamic Shear, @ 25°C, 10 rad/s, G* Sin Delta, KPa</td>
<td>TP 5</td>
<td>5000-</td>
<td>2297</td>
<td>5000-</td>
<td>4615</td>
</tr>
<tr>
<td>Bending Beam Creep Stiffness, S, MPa @ -12°C</td>
<td>TP1</td>
<td>300-</td>
<td>162</td>
<td>300-</td>
<td>193</td>
</tr>
<tr>
<td>Bending Beam Creep Slope, m value, @ -12°C</td>
<td>TP1</td>
<td>0.300+@-12°C</td>
<td>0.327</td>
<td>0.300+@-12°C</td>
<td>0.315</td>
</tr>
</tbody>
</table>
Figures 3.1 to 3.6 are the design gradations for all the mixtures in this study.

The Bailey gradation parameters for all the mixtures are summarized in Tables 3.2 and 3.3 for high and low volume respectively.

![Figure 3.1 Aggregate structures for ½” Granite Mixtures](image)

Reasonable separation was maintained between the aggregate gradations within each type of aggregate in order to capture the variation in performance (if any) within the same nominal maximum aggregate size (NMAS) for each type of aggregate. This separation was quantified by the decrease in the volume of coarse aggregate in the structure when moving from coarse to fine gradation. A great effort was made to maintain a practical number of stockpiles for each aggregate blend. A maximum of four different stockpiles of readily available, and commonly used aggregates in the State of Louisiana were used.
Figure 3.2 Aggregate structures for ½” Limestone Mixtures

Figure 3.3 Aggregate structures for ½” Sandstone Mixtures
Figure 3.4 Aggregate structures for 1” Limestone Mixtures

Figure 3.5 Aggregate Structure for ½” Low Volume Mixture
Figure 3.6 Aggregate Structure for 1” Low Volume Mixture

Table 3.2 Bailey Gradation Properties for High Volume Mixtures

<table>
<thead>
<tr>
<th>Mixture</th>
<th>CA Volume</th>
<th>FA Volume</th>
<th>CUW</th>
<th>%PCS</th>
<th>CA Ratio</th>
<th>F_{AC} Ratio</th>
<th>F_{AF} Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>GRF-1/2”</td>
<td>38.3</td>
<td>61.7</td>
<td>70.0</td>
<td>49</td>
<td>0.728</td>
<td>0.352</td>
<td>N/A*</td>
</tr>
<tr>
<td>GRM-1/2”</td>
<td>48.1</td>
<td>51.9</td>
<td>88.0</td>
<td>39.5</td>
<td>0.694</td>
<td>0.377</td>
<td>N/A</td>
</tr>
<tr>
<td>GRC-1/2”</td>
<td>55.2</td>
<td>44.8</td>
<td>101.0</td>
<td>33.2</td>
<td>0.686</td>
<td>0.396</td>
<td>N/A</td>
</tr>
<tr>
<td>LSF-1/2”</td>
<td>41.0</td>
<td>59.0</td>
<td>75.0</td>
<td>46.1</td>
<td>0.797</td>
<td>0.361</td>
<td>N/A</td>
</tr>
<tr>
<td>LSM-1/2”</td>
<td>46.4</td>
<td>53.6</td>
<td>85.0</td>
<td>39.6</td>
<td>0.706</td>
<td>0.374</td>
<td>N/A</td>
</tr>
<tr>
<td>LSC-1/2”</td>
<td>56.0</td>
<td>44.0</td>
<td>103.0</td>
<td>31.5</td>
<td>0.612</td>
<td>0.487</td>
<td>N/A</td>
</tr>
<tr>
<td>SSF-1/2”</td>
<td>40.8</td>
<td>59.2</td>
<td>75.0</td>
<td>48.4</td>
<td>0.792</td>
<td>0.435</td>
<td>N/A</td>
</tr>
<tr>
<td>SSM-1/2”</td>
<td>47.8</td>
<td>52.2</td>
<td>88.0</td>
<td>41.6</td>
<td>0.765</td>
<td>0.471</td>
<td>N/A</td>
</tr>
<tr>
<td>SSC-1/2”</td>
<td>56.0</td>
<td>44.0</td>
<td>103.0</td>
<td>32.8</td>
<td>0.627</td>
<td>0.493</td>
<td>N/A</td>
</tr>
<tr>
<td>LSC-1”</td>
<td>54.7</td>
<td>45.3</td>
<td>103</td>
<td>36.4</td>
<td>0.802</td>
<td>0.434</td>
<td>0.490</td>
</tr>
<tr>
<td>LSM-1”</td>
<td>47.1</td>
<td>52.9</td>
<td>89</td>
<td>43.2</td>
<td>0.803</td>
<td>0.36</td>
<td>0.482</td>
</tr>
<tr>
<td>LSF-1”</td>
<td>40</td>
<td>60</td>
<td>75</td>
<td>50.3</td>
<td>0.801</td>
<td>0.356</td>
<td>0.475</td>
</tr>
</tbody>
</table>
1. CA = Coarse Aggregate Volume
2. FA = Fine Aggregate Volume
3. CUW = Chosen Unit Weight
4. %PCS = Percent Passing Primary Control Sieve
5. CA Ratio = Coarse Aggregate Ratio
6. \( F_{AC} \) Ratio = Coarse Ratio of Fine Aggregate
7. \( F_{AF} \) Ratio = Fine Ratio of Fine Aggregate. This ratio is not calculated for half inch mixtures

Table 3.3 Bailey Gradation Properties for Low Volume Mixtures

<table>
<thead>
<tr>
<th>Mixture</th>
<th>CA Volume</th>
<th>FA Volume</th>
<th>CUW</th>
<th>%PCS</th>
<th>CA Ratio</th>
<th>( F_{AC} ) Ratio</th>
<th>( F_{AF} ) Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>LSF-1&quot;</td>
<td>37.3</td>
<td>62.7</td>
<td>70</td>
<td>52.9</td>
<td>1.134</td>
<td>0.617</td>
<td>0.317</td>
</tr>
<tr>
<td>LSF-1/2&quot;</td>
<td>37.1</td>
<td>62.9</td>
<td>68</td>
<td>49.2</td>
<td>0.842</td>
<td>0.246</td>
<td>N/A</td>
</tr>
</tbody>
</table>

The design gradations were further evaluated using the power-law method suggested by Ruth et al. (2002). The power-law characterizes the slope and the intercept constants of the coarse and fine aggregate portions of the aggregate gradations as described in chapter 2 of this report. Tables 3.4 and 3.5 present the power law gradation parameters for all the aggregate structures in this study. The divider sieve between the coarse and fine aggregate used in the power law analysis was chosen as the primary control sieve as determined from the Bailey method analyses. This was the 2.36 mm (No.8) sieve for the half inch NMAS mixtures and the 4.75mm (No.4) for the one inch NMAS mixtures.

Table 3.4 Power Law Gradation Parameters for High Volume Mixtures

<table>
<thead>
<tr>
<th>Mixture</th>
<th>( a_{CA} )</th>
<th>( n_{CA} )</th>
<th>( a_{FA} )</th>
<th>( n_{FA} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>GRF-1/2&quot;</td>
<td>34.15</td>
<td>0.42</td>
<td>31.07</td>
<td>0.61</td>
</tr>
<tr>
<td>GRM-1/2&quot;</td>
<td>24.15</td>
<td>0.55</td>
<td>24.65</td>
<td>0.60</td>
</tr>
<tr>
<td>GRC-1/2&quot;</td>
<td>18.20</td>
<td>0.66</td>
<td>20.90</td>
<td>0.58</td>
</tr>
</tbody>
</table>
Table 3.4 Cont.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>a</th>
<th>n</th>
<th>a</th>
<th>n</th>
</tr>
</thead>
<tbody>
<tr>
<td>LSF-1/2”</td>
<td>30.93</td>
<td>0.44</td>
<td>30.24</td>
<td>0.57</td>
</tr>
<tr>
<td>LSM-1/2”</td>
<td>24.66</td>
<td>0.53</td>
<td>24.50</td>
<td>0.58</td>
</tr>
<tr>
<td>LSC-1/2”</td>
<td>17.01</td>
<td>0.68</td>
<td>19.53</td>
<td>0.53</td>
</tr>
<tr>
<td>SSF-1/2”</td>
<td>33.77</td>
<td>0.42</td>
<td>33.10</td>
<td>0.50</td>
</tr>
<tr>
<td>SSM-1/2”</td>
<td>26.60</td>
<td>0.51</td>
<td>28.61</td>
<td>0.48</td>
</tr>
<tr>
<td>SSC-1/2”</td>
<td>18.31</td>
<td>0.65</td>
<td>20.93</td>
<td>0.59</td>
</tr>
<tr>
<td>LSC-1”</td>
<td>15.20</td>
<td>0.56</td>
<td>15.40</td>
<td>0.50</td>
</tr>
<tr>
<td>LSM-1”</td>
<td>20.80</td>
<td>0.50</td>
<td>19.40</td>
<td>0.60</td>
</tr>
<tr>
<td>LSF-1”</td>
<td>28.00</td>
<td>0.38</td>
<td>22.40</td>
<td>0.61</td>
</tr>
</tbody>
</table>

Table 3.5 Power Law Gradation Parameters for Low Volume Mixtures

<table>
<thead>
<tr>
<th>Mixture</th>
<th>a&lt;sub&gt;CA&lt;/sub&gt;</th>
<th>n&lt;sub&gt;CA&lt;/sub&gt;</th>
<th>a&lt;sub&gt;FA&lt;/sub&gt;</th>
<th>n&lt;sub&gt;FA&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>LSF-1”</td>
<td>30.60</td>
<td>0.36</td>
<td>23.80</td>
<td>0.68</td>
</tr>
<tr>
<td>LSF-1/2”</td>
<td>34.40</td>
<td>0.41</td>
<td>30.80</td>
<td>0.70</td>
</tr>
</tbody>
</table>

3.4 Asphalt Mixture Design

Mixture design was performed on all the aggregate structures that were formulated using the Bailey method of aggregate gradation and evaluation. The Superpave mixture design method was followed for all the mixtures designed in phase one except for VMA and VFA requirements. The Superpave mixture design method specifies the number of gyrations to which a sample must be compacted with the Superpave Gyratory compactor (SGC). The number of gyrations specified for mixture design is determined according to volume of traffic expected on the road. For every aggregate structure, trial asphalt content was estimated. The aggregates were then
batched out in the appropriate quantities to produce a final mix specimen of approximately 4800g. The aggregate batches, asphalt binder and mixing equipment were heated for four hours at 170°C to achieve appropriate uniform mixing temperature. The binder and the aggregate were then mixed in a mechanical mixing device (Figure 3.5) until a uniform mix is obtained. The resulting mix was then placed in a flat pan and heated for two hours at the compaction temperature of 150°C for short term aging. This aging represents the aging that occurs in the field between mixing and placement and allows for absorption of the asphalt binder into the aggregate pores. The mix was stirred every 30 minutes during the short-term aging process to prevent the outside of the mixture from aging more than the inner side because of increased air exposure.

For each trial, three specimens were compacted at the estimated asphalt content to the target design number of gyrations using the Superpave gyratory compactor. The bulk specific gravity of the compacted specimens was then determined according to AASHTO T 166 standard test procedure. Another set of two identical specimens in the loose condition of the same mix was used for the maximum theoretical density determination which was done using the Rice method according to AASHTO T 209 standard. The air void was then calculated for that mixture at the estimated asphalt content and specified number of gyrations. The process was repeated if necessary until the design asphalt content was reached. The design asphalt content was determined as the asphalt content required to achieve 4.0 percent air voids at $N_{des}$. The mixtures were then further analyzed to determine the rest of volumetric and physical properties at the design asphalt content. Finally, two specimens of the same mixture were then compacted to the maximum number of gyration $N_{max}$ and the volumetric properties were determined.
3.5 Preparation of Test Specimens

Laboratory test specimens were prepared for the different mechanical tests used in this study. Each test has specific requirements in terms of specimen shape and dimensions. Mixtures were prepared in large batches using a bucket mixture shown in Figure 3.6. This is done to minimize production variability and ensure uniformity of the specimens for each test. The specimens were then compacted to $7\% \pm 0.5\%$ air voids for testing. This is typical of the air void percentage in mixtures when they are placed in the field. The mixtures can be compacted in the SGC by number of gyrations or by height. Compacting based on height resulted in more uniform air void values for the test specimens, so this method was used for compaction. Two specimen categories were produced: rectangular slabs and cylindrical specimens. Asphalt concrete slab, 260.8 mm wide by 320.3 mm long and either 40mm thick for half inch mixes or 80mm thick for one inch mixes were manufactured for the Hamburg Wheel Tracking Test (HWT) using a linear kneading compactor shown in Figure 3.7. This is a special compactor that is capable of compacting two specimens simultaneously by rolling a steel drum on the
asphalt mixture until the specimen thickness is reached. The rectangular molds were heated to the compaction temperature before charging them with the loose mix.

Different sizes of cylindrical specimens were produced using the SGC. For the Indirect Tensile Strength Test, 100 mm in diameter by 63.5 mm thickness specimens were compacted directly in the Superpave Gyratory Compactor.

For the, fracture resistance, dynamic modulus, and resilient modulus tests 150 mm diameter specimens were prepared. Those specimens were further treated in order to obtain the actual test specimens with the dimensions and surface characteristics that allow better instrumentation and minimize testing variability.

For fracture test, SGC specimens 150.0mm in diameter by 57.0mm thickness were compacted. The specimens were then sliced perpendicular to the central axis to obtain the semi-circular test specimens. Two test specimens were then cut from each SGC sample. Air void measurements were made again on the cut specimens to ensure that the level of air voids was still within the targeted range. A vertical notch was
introduced along the symmetrical axis of the test specimen using a special saw blade of 3.0mm thickness.

Three notch depths were used; 25.4 mm, 31.8 mm, and 38.0 mm. Two specimens per notch depth were fabricated. Figure 3.8 is a graphical description of the process of preparing the Fracture Test specimens.

The test specimens for the Dynamic Modulus evaluation were 100mm in diameter that were cored from SGC samples of 150mm diameter and 156mm height using a diamond-tipped coring barrel. The specimens were then grinded from both ends to obtain the desired thickness of 150mm. Air void measurements were made on the finished specimens to ensure that the level of air voids was still within the targeted range of 7.0±0.5%. Figure 3.9 describes the process of preparing test specimens for dynamic modulus testing.
For the resilient modulus test, 150mm diameter test specimens were produced. Similar approach to the dynamic modulus test specimen preparation was followed except that the SGC specimens were compacted to 56mm height and then grinded to the desired thickness of 50mm. Table 3.6 summarizes the dimensions of the specimens for the various types of test conducted.

3.6 Laboratory Test Procedures

A comprehensive laboratory evaluation was conducted on the designed mixtures. A suite of mechanistic and simulative tests were performed to study the behavior of asphalt mixtures under various loading and environmental conditions.

This section provides a description of the test methods and procedures used for evaluating asphalt mixtures.
Figure 3.9 Specimen Preparation for the Dynamic Modulus Test

Table 3.6 Summary of Test Specimens Dimensions

<table>
<thead>
<tr>
<th>Test</th>
<th>ITS (25°C)</th>
<th>Jc (25°C)</th>
<th>LWT (50°C)</th>
<th>IT Mr (10°C)</th>
<th>ITS (10°C)</th>
<th>E*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample type</td>
<td>SGC</td>
<td>SGC</td>
<td>SGC</td>
<td>SGC</td>
<td>SGC</td>
<td>SGC</td>
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<tr>
<td>Sample diameter</td>
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<td>320.3 x 260.8°</td>
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<td>150.0</td>
<td>100.0</td>
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<tr>
<td>Sample height</td>
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<td>57.0</td>
<td>40.0, 80.0°</td>
<td>50.0</td>
<td>50.0</td>
<td>150.0</td>
</tr>
</tbody>
</table>

1. ITS = Indirect Tensile Strength
2. Jc = Fracture Resistance Test
3. LWT = Loaded Wheel Test
4. IT Mr = Indirect Tensile Resilient Modulus Test
5. E* = Dynamic Modulus Test
6. Slab Length = 320.3 mm and slab width = 260.8 mm
7. Slab thickness is 40.0 mm for 12.5 mm mixes and 80.0 mm for 25.4 mm mixes
3.6.1 Compactability of Asphalt Mixture

The densification curve obtained from the SGC was used to evaluate mixture resistance to the compaction energy applied by the SGC. The behavior of the mixtures during compaction was also captured using the PDA. This is a simple accessory that measures the force applied to the mixtures using three load cells equally spaced at an angle of 120°. The load-cells allow measuring the variation of forces during gyration such that the position or eccentricity of the resultant force from the gyratory compactor can be determined in real time. The two dimensional distributions of the eccentricity of the resultant force can be used to calculate the effective moment required to overcome the internal shear frictional resistance of mixtures when tilting the mold to conform to the 1.25 degree angle. Based on the data from the load-cells, the two components of the eccentricity of the total load relative to the center of the plate (ex and ey) can be calculated. (The calculations are simply done with general moment equilibrium equations along two perpendicular axes passing through the center of one of the load-cells as shown in Figure 3.10 using the following Equation

\[ \sum M_x = 0 \implies e_y \\
\sum M_y = 0 \implies e_x \\
e = \sqrt{e_x^2 + (r_y - e_y)^2} \]

P1, P2, P3 are load-cell forces; ex and ey are x- and y-components of the eccentricity, e; and ry is location of the plate center point with respect to the x-axis.

The frictional shear resistance of the asphalt mixture can be calculated using the following relationship:

\[ FR = Re/AH \]
where,

\[ FR = \text{the frictional resistance} \]

\[ R = \text{Resultant Force} \]

\[ e = \text{eccentricity} \]

\[ A = \text{cross-section area} \]

\[ H = \text{sample height at any gyration cycle.} \]

Two specimens per mixture were tested for compactability in both SGC and PDA devices.

Figure 3.10 Pressure Distribution Analyzer: (a) The PDA Device (b) Analysis of Forces (c) Inserting the PDA in the Compaction Mold (d) Typical Results
3.6.2 Permeability

Permeability of asphalt mixtures is an important factor in the durability of asphalt pavements. If the mix is too permeable, premature stripping occurs, shortening the life expectancy of the pavement. Mixtures with high permeability are believed to have a greater number of interconnected voids, allowing air and water to penetrate a pavement. Air increases the rate of oxidation of the asphalt binder which can lead to binder hardening and ultimately pavement cracking. The presence of water within the asphalt mixtures leads to weakening the bond between the aggregate and the binder, a phenomenon known as stripping. Water can also weaken the bottom layers of the pavements causing strength reduction in the subgrade which might lead to severe structural failure of the pavement.

A falling head permeability apparatus, as shown in Figure 3.11 was used to determine the rate of flow of water through the specimen.

SGC compacted specimen of 150mm diameter and 63.5mm height was soaked over night to achieve full saturation. The specimen was then removed from the water bath and the side was treated with petroleum jelly in order to prevent any lateral flow of water. The specimen was then placed in the test apparatus and confined using a latex membrane. The air was evacuated from the membrane cavity. The membrane was inflated to 12.5 psi and this pressure was maintained throughout the test. Water was filled to a level above the graduated, upper timing mark. The timing device was started when the bottom of the meniscus of the water reached the upper timing mark. The test was run for 30 minutes and the final water level was recorded at the end of the test. The time was recorded to the nearest second. The coefficient of permeability is determined using the following equation:
\[ K = \left( \frac{L}{T} \right) \times \left( \frac{d^2}{D^2} \right) \times \ln \left( \frac{H_1}{H_2} \right) \]

where,

\( K = \) Coefficient of Permeability (mm/s x 10-4)

\( L = \) average specimen thickness

\( D = \) average specimen diameter

\( d = \) graduated cylinder diameter

\( T = \) total time of test, seconds

\( H_1 = \) initial height of water

\( H_2 = \) final height of water

Figure 3.11 Permeability Test Apparatus
3.6.3 Wheel Tracking Test (HWT)

The Wheel-Tracking device measures rutting by rolling a small loaded wheel device repeatedly across a rectangular asphalt specimen. The test provides information about the rate of permanent deformation from a moving, concentrated load. The potential for moisture damage is also evaluated since the test specimens are submerged in temperature controlled water bath during testing. A Hamburg version of the wheel tracking device was used. The specimens were placed into steel holders and secured in the water bath. The test temperature was 50°C in this study. The specimens were conditioned in the water bath at that temperature for 90 minutes prior to applying the loaded wheels and starting the test. The load applied by the steel wheels is 702 N (158 lbs) and the wheel face is 47 mm wide. The wheels move reciprocally across the top of the specimens at 56 passes per minute. Two Linear Value Displacement Transducers (LVDT’s) were used to measure the deformation at eleven points across each sample. The HWT test was set up to run for a total of 20 000 cycles or end when one of the eleven monitoring points across the surface of the test specimen reaches a depth of 20 mm. Two pairs of specimens were tested for each mix and the data from both specimens were averaged to provide the final value used in the analysis. Figure 3.12 shows the PMW wheel tracker used in the study. Typical test output is shown in Figure 3.13

Two parameters were calculated from the graph of permanent deformation versus wheel passes: The rut depth and the post compaction consolidation. The rut depth is reported as the average of the middle five of the eleven measurement points along the wheel path. The post compaction consolidation is the amount of deformation which rapidly occurs during the first few minutes of the test. The steel wheel has some
compacting effects on the mixes. A point of inflection occurs after this initial consolidation is completed.

Figure 3.12 Wheel Tracking Machine

Figure 3.13 Typical Results from the Wheel Tracking Test
3.6.4 Semi-Circular Fracture Energy Test

The fracture resistance of the mixtures designed in this study was investigated using the J-integral approach. This procedure is based on a fracture mechanics concept - the critical strain energy release rate, also called the critical value of J-integral, or \( J_c \). In this study, the fracture resistance of the designed mixtures was characterized using this test based on notched semi-circular specimens [Mohammad 2002, 2004].

This approach is gaining popularity for characterizing heterogeneous materials such as asphalt mixtures. The method accounts for the flaws as represented by a notch, which in turn, reveals the material’s resistance of to crack propagation or what is so called fracture resistance. During the test, the specimen was loaded monotonically to failure at a constant cross-head deformation rate of 0.5 mm/min in a three-point bend load configuration, as shown in Figure 3.14. The load and deformation were continuously recorded determined as follows:

\[
J_c = -\left(\frac{1}{b}\right) \frac{dU}{da}
\]

where, \( b \) is the specimen thickness, \( a \) is the notch depth, and \( U \) is the total strain energy to failure, i.e. the area up to fracture under the load-deflection plot (Figure 3.15).

FIGURE 3.14 Semi-circular Test Setup
To determine the critical value of J-integral, semi-circular specimens with at least two different notch depths (parameter “a” in Figure 7-2) need to be tested for one mixture. In this study, three notch depths of 25.4 mm, 31.8 mm, and 38 mm were selected based on an a/rd ratio (the notch depth to the radius of the specimen) of between 0.5 and 0.75 (Mohammad 2002, Mohammad 2004). For each notch depth three duplicates were tested. The test temperature was 25° C. This test was conducted on two groups of specimens: unaged and oven aged.

3.6.5 Indirect Tensile Strength Test (ITS)

The indirect tensile stress (ITS) and strain test was used to determine the tensile strength and strain of the mixtures. This test was incorporated in the study to ensure that the durability of the mixtures would not be compromised while the rut resistance of the mixtures was being improved. This test was conducted at 25°C in accordance with AASHTO T245 (Figure 3.16). The test specimen was loaded to failure at a 50.8 mm/min (2 in/min) deformation rate. The load and deformations were continuously recorded. Figure 3.17 shows a typical test output. Indirect tensile strength and strain were computed as follows:
\[ ITS = \frac{2P}{\pi DT} \]

\[ \varepsilon_t = 0.52H_t \]

where,

\( P \) = the peak load, lb

\( D \) = specimen diameter, in

\( T \) = specimen thickness, in

\( H_t \) = horizontal deformation at peak load, in

Figure 3.16 Indirect Tensile Strength Test Setup
Figure 3.17 Typical Results from the Indirect Tensile Strength Test

The toughness index (TI), a parameter describing the toughening characteristics in the post peak region, was also calculated from the indirect tensile test results. Figure 3.18 presents a typical normalized indirect tensile stress and strain curve. A dimensionless indirect tensile toughness index, TI is defined as follows:

$$TI = \frac{A_\varepsilon - A_p}{\varepsilon - \varepsilon_p}$$

where,  

$TI$ – Toughness index,  

$A_\varepsilon$ – Area under the normalized stress-strain curve up to strain, $\varepsilon$,  

$A_p$ – Area under the normalized stress-strain curve up to strain, $\varepsilon_p$,  

$\varepsilon$ – Strain at the point of interest, and  

$\varepsilon_p$ – Strain corresponding to the peak stress.
This toughness index compares the performance of a specimen with that of an elastic perfectly plastic reference material, for which the TI remains a constant of one. For an ideal brittle material with no post-peak load carrying capacity, the value of TI equals zero.

3.6.6 Dissipated Creep Strain Energy (DCSE)

According to Roque et al. (1997) the Dissipated Creep Strain Energy (DCSE) limit is one of the most important factors that control crack performance and hence durability in asphalt concrete mixtures. The dissipated creep strain energy threshold represents the energy that the mixture can tolerate before it fractures. This parameter is determined using two laboratory tests conducted on the same specimen. These tests are: the indirect resilient modulus test and the indirect tensile strength test. Both tests were conducted at 10°C on 150 mm diameter and 50 mm thick specimens. DCSE is defined as the fracture energy, FE, minus the elastic energy, EE (Figure 3.19). The fracture
energy is defined as the area under the stress-strain curve up to the point where the specimen begins to fracture. The elastic energy is the energy recovered after unloading the specimen. The failure strain ($\varepsilon_f$), tensile strength (St) and fracture energy are determined from the IT strength test. From the resilient modulus test, the resilient modulus (Mt) is obtained. The calculation of the DCSE was then determined as follows:

$$\varepsilon_0 = \frac{(M_r \times \varepsilon_f - St)}{M_r}$$

$$EE = \frac{1}{2} St (\varepsilon_f - \varepsilon_0)$$

$$DCSE = FE - EE$$

![Figure3.19 Dissipated Creep Strain Energy Determination](image)

Because of the relatively low test temperature and the dynamic nature of the Mr test, it was decided to use on sample instrumentation in order to accurately capture the small deformations resulting from the repeated load applied. Brass gage points were attached to the test specimens with a strong adhesive (Devcon plastic steel 5 minute...
epoxy putty\textsuperscript{(SF) 10240) using a special template as shown in Figure 3.20. Four gauge points were installed on each face of the specimen along the vertical and horizontal axis. Two units of single integral, bi-axial extensometers model 3910 from epsilon technology that measure both lateral and vertical deformations were clipped onto gage points mounted on each face of the specimen. The tests were performed using an MTS hydraulic loading system with the Teststar II data acquisition system. An environmental chamber kept the temperature constant ± 0.1° C.

The test specimens were conditioned at 10° C for four hours in a fridge before they were transferred to the environmental chamber of the test equipment. The specimens were further conditioned for one hour in the environmental chamber at the test temperature. A 200-cycle haversine load (the load magnitude was 12 percent of the IT strength of the specimen) with 0.1 second loading period and 0.4 second rest period in each loading cycle was the applied along the diametrical plane on the specimen to condition it to obtain uniform measurements in load and deformation. Then, a four-cycle haversine compressive load was applied and load and deformation data were continuously recorded through a data acquisition system. At the test temperature, after one test was finished, the specimen was rotated 90 degrees and tested again. The average of the two test results was reported as the resilient modulus of the specimen. The magnitude of the load applied was chosen so that the resulting deformation is a maximum of a 100 micro strain.

3.6.7 Dynamic Modulus Test

The test is one of the oldest tests and probably the best documented of the axial compression tests. The large amount of data accumulated from this test over the years
has served as the basis for the development of a series of predictive models that are based on large amount of test data (Witczack et al, 1999).

The dynamic modulus test is a triaxial compression test, which was standardized in 1979 as ASTM D3497, “Standard Test Method for Dynamic Modulus of Asphalt Concrete Mixtures.” The test consists of applying a uniaxial sinusoidal (i.e., haversine) compressive stress to an unconfined or confined HMA cylindrical test specimen (Figure 3.21). The stress-to-strain relationship under a continuous sinusoidal loading for linear viscoelastic materials is defined by a complex number called the “complex modulus” ($E^*$). Mathematically, the Complex modulus is defined as:

$$E^* = \frac{|E^*|}{cos \phi} + i\frac{|E^*|}{sin \phi}$$

Where $\phi$ is the phase angle and defined as:

$$\phi = \frac{T_i}{T_p} \times 360'$$

where:

$T_i =$ time lag between stress and strain
$T_p =$ period of applied stress
$i =$ imaginary number.
The phase angle is an indicator of the viscous properties of the material being tested. For a pure elastic material, \( \phi = 0 \), and for pure viscous materials, \( \phi = 90^\circ \).

The absolute value of the complex modulus \( |E^*| \), is defined as the dynamic modulus. The dynamic modulus is mathematically defined as the maximum (i.e., peak) dynamic stress \( \sigma_o \) divided by the peak recoverable strain \( \varepsilon_o \) as follows:

\[
|E^*| = \frac{\sigma_o}{\varepsilon_o}
\]

Figure 3.21 Loading Condition in the Dynamic Modulus Test

According to the AASHTO TP 62-03 “Standard Test Method for Determining Dynamic Modulus of Hot-Mix Asphalt Concrete Mixtures”, the recommended dynamic modulus \( |E^*| \) test consists of testing at –10, 4.4, 20, 37.8, and 54.4°C (14, 40, 70, 100 and 130°F) at loading frequencies of 0.1, 0.5, 1.0, 5, 10, and 25 Hz at each temperature for the development of master curves for use in pavement response and performance analysis. Each specimen should be tested for each of the 30 combinations of temperature and frequency of loading starting with the lowest temperature and proceeding to the highest. Testing at a given temperature should begin with the highest frequency of loading and proceed to the lowest. In this study, the dynamic modulus \( |E^*| \) test was
performed at 4.4, 21.1, 38.8, and 54°C (14, 40, 77, 100, and 130°F) at the six loading frequencies recommended by AASHTO TP 62-03. A haversine compressive stress was applied on each SPT sample to achieve a target vertical strain level of 100 microns in an unconfined test mode. The dynamic modulus test was conducted in a Universal Testing Machine, which includes the loading device, specimen deformation setup, environmental chamber and control and data acquisition system as shown in Figure 3.22. The Dynamic Modulus $|E^*|$ which is a measure of the material stiffness and the phase angle, $\phi$ which is a measure of the viscous properties of the material were determined from this test.

![Figure 3.22 Dynamic Modulus Test Apparatus](image)
CHAPTER 4. PHASE ONE ANALYSIS OF RESULTS

4.1 Introduction

This chapter presents and discusses the results obtained from phase one of the research. That includes mixture design data and gradation analysis, results from the different laboratory tests presented and discussed in the previous chapter, and the effect of the different mixture parameters on those test results.

4.2 Mixture Design

Mixture design was performed on all the aggregate structures that were formulated using the Bailey method of aggregate gradation and evaluation. The Superpave mixture design method was followed except for the VMA requirement. All the mixtures were designed for high-volume traffic ($N_{\text{des}}=125$ gyrations at $1.25^\circ$ angle of gyration). The optimum asphalt content was determined as the asphalt content required to achieve 4.0 percent air voids at $N_{\text{des}}$. Tables 4.1 and 4.2 present the results of the mix design conducted on all the mixtures considered in phase one of this study. A Graphical representation of the data is presented in Figures 4.1 through 4.5. Optimum asphalt contents ranged from 3.0 percent to 5.1 percent. The coarse mixtures had higher optimum asphalt contents for all the aggregate types considered. These mixtures have higher VMA values compared to the other ones, which created more inter-granular void space for the asphalt cement to occupy and hence increased the optimum asphalt content. Voids in mineral aggregates ranged from 8.4 percent to 13.5 percent. Sandstone medium and fine mixtures had the lowest VMA values. The VMA values for all the mixtures were below the minimum requirement of the current Superpave system. It is noted that coarse and fine mixtures with similar NMAS have different VMA values. This observation supports the concern on the validity of the current Superpave VMA
requirement based on the NMAS. It is evident that VMA is sensitive to aggregate 
gradation within the same NMAS. All the mixtures met the Superpave requirements for 
\( \%G_{\text{mm}} \) at \( N_{\text{ini}} \) and \( \%G_{\text{mm}} \) at \( N_{\text{max}} \). The average effective binder film thicknesses ranged 
from 8.8 microns for limestone and sandstone coarse mixtures to as low as 2.5 for 
medium and fine sandstone mixtures. For most medium and fine mixtures, the calculated 
film thickness was below the generally reported range of 6.0-8.0 microns. Film 
thickness is strongly affected by the amount of dust (passing #200 sieve) in relation to 
asphalt content or what is called Dust/Pbeff ratio as shown in Figure 4.6. Medium and 
fine sandstone mixtures with the lowest film thickness values have the highest Dust/Pbeff 
ratio of 4.7.

![Figure 4.1 Design Asphalt Content](image)

4.3 Permeability

The falling head test was used to determine the amount of water head loss through 
a given sample over a given time. For this test, the coefficient of permeability was
calculated as follows:

$$K = \left( \frac{L}{T} \right) \times \left( \frac{d^2}{D^2} \right) \times \ln \left( \frac{H_1}{H_2} \right)$$

where,

$L = \text{average specimen thickness}$

$D = \text{average specimen diameter}$

$d = \text{graduated cylinder diameter}$

$T = \text{total time of test, seconds}$

$H_1 = \text{initial height of water}$

$H_2 = \text{final height of water}$

Figure 4.2 Voids in the Mineral Aggregate
<table>
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<th>Mixture name</th>
<th>LS Coarse</th>
<th>LS Medium</th>
<th>LS Fine</th>
<th>SST Coarse</th>
<th>SST Medium</th>
<th>SST Fine</th>
<th>GR Coarse</th>
<th>GR Medium</th>
<th>GR Fine</th>
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<td>12.5 mm</td>
<td>12.5 mm</td>
<td>12.5 mm</td>
<td>12.5 mm</td>
<td>12.5 mm</td>
<td>12.5 mm</td>
<td>12.5 mm</td>
<td>12.5 mm</td>
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<td>Aggregate blend</td>
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<td></td>
<td></td>
<td></td>
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<tr>
<td>LS: Siliceous Limestone, SST: Sandstone, GR: Granite</td>
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<td></td>
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<td></td>
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<td>Binder type</td>
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<td>PG 76-22M</td>
<td>PG 76-22M</td>
<td>PG 76-22M</td>
<td>PG 76-22M</td>
<td>PG 76-22M</td>
<td>PG 76-22M</td>
<td>PG 76-22M</td>
<td>PG 76-22M</td>
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<td>% G$<em>{mm}$ at N$</em>{i}$</td>
<td>85.1</td>
<td>86.2</td>
<td>88.0</td>
<td>86.0</td>
<td>86.4</td>
<td>88.0</td>
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<td>97.0</td>
<td>97.2</td>
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<td>97.3</td>
<td>97.7</td>
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<td>80.7</td>
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LS: Siliceous Limestone, SST: Sandstone, GR: Granite
Table 4.2 Job Mix Formula- 25.4 mm Mixes- N_{des} = 125 Gyration

<table>
<thead>
<tr>
<th>Mixture name</th>
<th>LS Coarse</th>
<th>LS Medium</th>
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<tr>
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<td>25.4 mm</td>
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<td>Aggregate blend</td>
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<tr>
<td>Binder type</td>
<td>PG 76-22M</td>
<td>PG 76-22M</td>
<td>PG 76-22M</td>
</tr>
<tr>
<td>Design AC content, volumetric properties, and densification</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>% G_{mm} at N_{f}</td>
<td>85.0</td>
<td>88.8</td>
<td>89.1</td>
</tr>
<tr>
<td>% G_{mm} at N_{M}</td>
<td>97.7</td>
<td>97.4</td>
<td>97.4</td>
</tr>
<tr>
<td>Design binder content, %</td>
<td>3.8</td>
<td>3.0</td>
<td>3.3</td>
</tr>
<tr>
<td>Design air void, %</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>VMA, %</td>
<td>11.1</td>
<td>9.6</td>
<td>10.0</td>
</tr>
<tr>
<td>VFA, %</td>
<td>63.5</td>
<td>58.2</td>
<td>60.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Metric (U.S.) Sieve</th>
<th>Gradation, (% passing)</th>
</tr>
</thead>
<tbody>
<tr>
<td>37.5 mm (1½ in)</td>
<td>100</td>
</tr>
<tr>
<td>25 mm (1 in)</td>
<td>100</td>
</tr>
<tr>
<td>19 mm (¾ in)</td>
<td>100</td>
</tr>
<tr>
<td>12.5 mm (⅝ in)</td>
<td>100</td>
</tr>
<tr>
<td>9.5 mm (⅜ in)</td>
<td>100</td>
</tr>
<tr>
<td>4.75 mm (No.4)</td>
<td>100</td>
</tr>
<tr>
<td>2.36 mm (No.8)</td>
<td>100</td>
</tr>
<tr>
<td>1.18 mm (No.16)</td>
<td>100</td>
</tr>
<tr>
<td>0.6 mm (No.30)</td>
<td>100</td>
</tr>
<tr>
<td>0.3 mm (No.50)</td>
<td>100</td>
</tr>
<tr>
<td>0.15 mm (No.100)</td>
<td>100</td>
</tr>
<tr>
<td>0.075 mm (No.200)</td>
<td>100</td>
</tr>
</tbody>
</table>

Figure 4.3 Voids Filled with Asphalt
Figure 4.4 The Ratio of Dust over Effective Binder Content

Figure 4.5 Effective Film Thickness
Figure 4.6 Sensitivity of Effective Film Thickness to the change in Dust/Pbef Ratio

Table 4.3 Permeability Data

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Gradation</th>
<th>Average Permeability (mm/sec*10^-4)</th>
<th>Average Permeability (ft/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2&quot; Limestone</td>
<td>Fine</td>
<td>0.82</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>Medium</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>Coarse</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>1/2&quot; Sandstone</td>
<td>Fine</td>
<td>1.12</td>
<td>0.03</td>
</tr>
<tr>
<td></td>
<td>Medium</td>
<td>3.12</td>
<td>0.09</td>
</tr>
<tr>
<td></td>
<td>Coarse</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>1/2&quot; Granite</td>
<td>Fine</td>
<td>1.91</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td>Medium</td>
<td>1.37</td>
<td>0.04</td>
</tr>
<tr>
<td></td>
<td>Coarse</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>1&quot; Limestone</td>
<td>Fine</td>
<td>0.21</td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>Medium</td>
<td>0.65</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>Coarse</td>
<td>0.30</td>
<td>0.01</td>
</tr>
</tbody>
</table>

Table 4.3 presents the permeability data for all the mixtures. All the mixtures showed very low permeability level and in many cases they were virtually impermeable. The extremely low permeability of the designed mixtures reflects the dense aggregate...
structures that resulted in minimal interconnectivity of the air voids and hence prevented any water flow through the mixtures despite the fact that all the specimens were compacted to a 7.0% air void.

4.4 Asphalt Mixtures Compactability

The compactability of the designed asphalt mixtures was evaluated using results from the Superpave gyratory compactor (SGC) and the pressure distribution analyzer device (PDA). The densification curve obtained from the SGC was used to evaluate mixture resistance to the compaction energy applied by the SGC. The behavior of the mixtures during compaction was also captured using the PDA. The following terms will be used in the analysis of the results from the SGC and the PDA.

4.4.1 SGC Locking Point

The SGC locking point is the number of gyrations after which the rate of change in height is equal to or less than 0.05 mm for three consecutive gyrations (Figure 4.7 and Table 4.4)

![Figure 4.7 Rate of Change of Height During SGC Compaction](image-url)
Table 4.4 Example Data Set for SGC Locking Point Determination

<table>
<thead>
<tr>
<th>Number of Gyrations</th>
<th>Rate of Change</th>
</tr>
</thead>
<tbody>
<tr>
<td>61</td>
<td>0.07</td>
</tr>
<tr>
<td>62</td>
<td>0.06</td>
</tr>
<tr>
<td>63</td>
<td>0.08</td>
</tr>
<tr>
<td>64</td>
<td>0.07</td>
</tr>
<tr>
<td>65</td>
<td>0.06</td>
</tr>
<tr>
<td>66</td>
<td>0.07</td>
</tr>
<tr>
<td>67</td>
<td>0.07</td>
</tr>
<tr>
<td>68</td>
<td>0.06</td>
</tr>
<tr>
<td>69</td>
<td>0.06</td>
</tr>
<tr>
<td>70</td>
<td>0.05</td>
</tr>
<tr>
<td>71</td>
<td>0.05</td>
</tr>
<tr>
<td>72</td>
<td>0.05</td>
</tr>
<tr>
<td>73</td>
<td>0.05</td>
</tr>
<tr>
<td>74</td>
<td>0.05</td>
</tr>
</tbody>
</table>

4.4.2 PDA Locking Point

It is defined as the number of gyrations at which the rate of change of frictional resistance per gyration is less than 0.01 (Figure 4.8 and Table 4.5).

Figure 4.8 Rate of Change of Frictional Resistance During SGC Compaction
Table 4.5 Example Data Set for PDA Locking Point Determination

<table>
<thead>
<tr>
<th>No. of Gyrations</th>
<th>Rate of change of FR</th>
</tr>
</thead>
<tbody>
<tr>
<td>36</td>
<td>0.053</td>
</tr>
<tr>
<td>37</td>
<td>0.049</td>
</tr>
<tr>
<td>38</td>
<td>0.047</td>
</tr>
<tr>
<td>39</td>
<td>0.052</td>
</tr>
<tr>
<td>40</td>
<td>0.045</td>
</tr>
<tr>
<td>41</td>
<td>0.042</td>
</tr>
<tr>
<td>42</td>
<td>0.031</td>
</tr>
<tr>
<td>43</td>
<td>0.035</td>
</tr>
<tr>
<td>44</td>
<td>0.046</td>
</tr>
<tr>
<td>45</td>
<td>0.046</td>
</tr>
<tr>
<td>46</td>
<td>0.036</td>
</tr>
<tr>
<td>47</td>
<td>0.040</td>
</tr>
<tr>
<td>48</td>
<td>0.029</td>
</tr>
<tr>
<td>49</td>
<td>0.032</td>
</tr>
<tr>
<td>50</td>
<td>0.030</td>
</tr>
<tr>
<td>51</td>
<td>0.009</td>
</tr>
</tbody>
</table>

4.4.3 SGC Compaction Densification Index (CDI)

CDI is defined as the area under the SGC densification curve from N=1 to the SGC locking point (Figure 4.9). This index is hypothesized to be related to compactability of asphalt mixtures. Higher values of this index are associated with mixtures that are difficult to compact.

4.4.4 SGC Traffic Densification Index (TDI)

TDI is the area under the SGC densification curve from the SGC locking point to N at 98% G_m or the end of compaction, whichever comes first (Figure 4.9). This index is hypothesized to be related to the stability of mixtures under traffic loading. Theoretically, higher values are supposed to be indicative of better mixtures stability.
4.4.5 PDA Compaction Force Index (CFI)

CFI is the area under frictional resistance vs. No. of gyration curve from N=1 to the SGC locking point. It is analogous to the CDI (Figure 4.10). Higher values are associated with mixtures with poor compaction characteristics.

![Figure 4.9 SGC Compaction Indices Definition](image)

4.4.6 PDA Traffic Force Index (TFI)

TFI is the area under frictional resistance vs. No. of gyration curve from the SGC locking point to N=205 (Figure 4.5). This index is analogous to TDI from the SGC. Higher values are supposed to be indicative of more stable mixtures.

![Figure 4.10 PDA Compaction Indices Definition](image)
The locking point data presented in Figure 4.11 and Figure 4.12 for both the SGC and the PDA respectively suggest that coarse mixtures take a higher number of gyrations to reach to the locking condition. This indicates that it takes more energy to densify coarse mixtures compared to the medium and fine mixtures. As the aggregate gradation becomes finer, the compactability of the mixtures improves with the only exception of fine granite mixture in which locking point was slightly higher than the medium gradation. Locking points are much lower than the design number of gyrations recommended by the current Superpave design system. The highest locking point is less than 70% of the recommended design number of gyrations for the heavy-traffic category ($N_{des}=125$). For half inch NMAS mixtures, the fine limestone mixture had the lowest locking points from both SGC and PDA (62 and 57 respectively). Both medium and fine limestone one inch NMAS mixtures showed similar response to the applied compaction energy in terms of locking point. Figure 4.13 presents the good correlation between the locking points determined from the SGC and those determined from the PDA. On average, the PDA locking points were about 4 gyrations lower than those determined from the SGC data.

![Figure 4.11 SGC Locking Point Results](image)
The concept of energy indices was first introduced by Bahia (Bahia et al., 1998).

In his study, Bahia calculated the energy indices using the region from N=8 to N at 92%
G\textsubscript{mm} for the CDI and from N at 96% G\textsubscript{mm} to N at 98% G\textsubscript{mm} for the TDI. He assumed that the first 8 gyrations represent the constant compaction energy applied by the paver screed. In this study, however, that energy is considered as part of the applied compaction effort and the densification curve is divided into two main regions: the densification region from N=1 to the locking point, which is used to calculate the CDI and CFI from both the SGC and PDA, and the post densification region from the locking point to N= 205, which represents the terminal densification of the mixture at the end of service life and used to calculate the TDI and TFI. Figure 4.14 and Figure 4.15 show the energy indices calculated for all the mixtures in phase one of the study.

![SGC Indices](image)

**Figure 4.14 SGC Densification Indices**

The compaction densification index CDI from the SGC had notable variations across the different gradations within the same NMAS, indicating that it is sensitive to
the size distribution of blends having the same NMAS. For example, for half inch limestone mixtures, the fine mixture required about 48% lower energy to reach the locking condition than the coarse mixture. The sandstone had lower variation in CDI across the different gradations. The fine sandstone mixtures took about 17% less compaction energy than the coarse one to reach to the locking condition. There was about 11% difference in compaction energy between the medium and the fine granite gradations.

The data, therefore, suggests that it will take more energy to compact coarse mixtures in the first region of the densification curve, indicating that those mixtures might be less desirable for construction and more likely to have compactability problems. The same trend was observed with the compaction force index from the PDA. This is
clearly shown in the strong correlation obtained between CDI and CFI ($R^2=0.92$) in Figures 4.16 and 4.17.

![Figure 4.16 Correlation between PDA and SGC Compaction Indices](image1)

![Figure 4.17 Comparison of Traffic Indices from SGC and PDA](image2)
The aggregate resistance to further densification from traffic loading was explored using the TDI from the SGC and the TFI from the PDA. The variation of these two indices, although is still existent, is less than that observed with the compaction indices. This was expected since the behavior of the mixtures beyond their locking points was relatively similar, as shown in Figure 4.18. The mixtures maintained their frictional resistance until the end of compaction without showing noticeable loss in stability under the compaction load. The only mixture that is showing some loss in stability is the fine sandstone mixture. This mixture has the highest amount of fine materials passing the No. 200 sieve (9.1%). In general, the magnitude of the frictional resistance varied in a narrow range between the mixtures at the locking point, suggesting that different aggregate structures can offer similar performances if they are properly designed with the aim of achieving mix stability.

![Frictional Resistance of Asphalt Mixtures](image)

Figure 4.18 Frictional Resistance of Asphalt Mixtures
4.5 Gradation Analysis

As mentioned earlier, the aggregate structures designed using the Bailey method were further evaluated by the power law gradation evaluation method. Both methods look at distinct regions in the gradation curve and describe them using one or more indices that are related to the size distribution of the aggregates in those particular regions. An attempt was made to correlate the parameters from each method, as shown in Figures 4.19 and 4.20. Figure 4.19 shows that there is a good correlation between the parameters describing the coarse portion of the gradation curve from both methods (CA ratio, $a_{CA}$, and $n_{CA}$). From Figure 4.19a it seems that the relationship between the intercept $a_{CA}$ and CA ratio is NMAS dependant. There was a clear distinction between the one inch mixtures and the half inch mixtures trend lines. The correlation between the parameters describing the fine portion of the aggregate gradation curve is, however, relatively weak. This is not unexpected since the $F_{AC}$ from the Bailey method describes the middle portion of the curve only while the parameters from the power law method considers the whole portion of the gradation curve from the divider sieve to the No.200 sieve. In other words, the fine parameters from the two methods describe different regions of the gradation curve and, thus are not expected to correlate well with each others.

4.5.1 Gradation Parameters and Mixture Design

The effect of aggregate gradation on mixture volumetrics was investigated using the gradation parameters obtained from both the Bailey method and the power law method. Two parameters from the Bailey method were used in this investigation. These are the CA Ratio and the $F_{AC}$ Ratio. Since different aggregate types were used, correlating the gradation parameters directly to mixture design might be misleading.
Mixture design parameters are not only a function of the particle size distribution but also are affected by the shape and surface texture characteristics of the aggregates used. Those characteristics are different for different aggregate types.

![Figure 4.19 Relationship of the Coarse Gradation Parameters from the Bailey and the Power Law methods (a) aCA vs. CA Ratio (b) nCA vs.CA Ratio](image)

Figure 4.19 Relationship of the Coarse Gradation Parameters from the Bailey and the Power Law methods (a) aCA vs. CA Ratio (b) nCA vs.CA Ratio

![Figure 4.20 Fine Gradation Parameters from the Bailey Method and the Power Law Method (a) aFA vs F_{AC} Ratio (b)aFA vs. F_{AC} Ratio](image)

Figure 4.20 Fine Gradation Parameters from the Bailey Method and the Power Law Method (a) aFA vs F_{AC} Ratio (b)aFA vs. F_{AC} Ratio

The final blend is also affected by the amount and type of compaction applied to the mix. Therefore, those effects need to be accounted for when trying to study such type of relationships between gradation and mixture design parameters. Regarding mixture compaction, all the mixtures were subjected to the same type and amount of compaction.
energy (SGC compaction, $N_{\text{des}} = 125$). In order to incorporate the variation of shape and surface texture of the different types of aggregates used, the gradation parameters from both the Bailey method and the power law method were normalized by dividing them by the chosen unit weight of the blend for each mixture considered. The chosen unit weight is a percent of the loose unit weight of the aggregates based on the degree of coarse aggregate interlock required. The loose unit weight is the minimum density (mass per volume) required to provide particle-to-particle contact of the coarse aggregates. Shape and surface texture play an important role in the packing of aggregate particles in a unit volume and consequently have an influence on the measured unit weight. Therefore, by incorporating the unit weight in the relationship between the gradation parameters and mixture design properties, the shape and surface texture is indirectly accounted for.

Figure 4.21 illustrates the relationship between the Bailey parameters and mixture physical properties.

CA ratio, which is predominantly a function of the coarse aggregate blend by volume, seems to have the strongest correlations with mixture physical properties. As the CA ratio increases, the smaller size particles in the coarse portion of the aggregate structure become more dominant, creating an inverse effect on the main volumetric parameters VMA, and VFA.

As shown in Figure 4.22 good correlation is also observed between CA ratio and the effective film thickness ($R^2 = 0.68$) in which film thickness is reduced by having high CA ratio. Mixture volumetrics seem to be less sensitive to the change in the $F_{AC}$ ratio. This is illustrated by the low coefficient of determination, $R^2$. No relationship could be established between the $F_{AC}$ ratio and effective film thickness.
The same analysis was conducted on the parameters obtained from the power law method of aggregate evaluation. This analysis is presented in Figures 4.23 through Figure 4.27. Among the mix properties considered, effective film thickness seems to be more sensitive to the gradation parameters from this method. Three of the four
parameters; $a_{CA}$, $n_{CA}$, and $a_{FA}$, had the strongest influence on effective film thickness with $R^2$ of 0.69, 0.63, and 0.70 respectively. The slope of the fine portion of the gradation curve $n_{FA}$ had the least influence on effective film thickness. In general, as the gradation becomes finer, effective film thickness tends to decrease.

A trend is also observed in the relationship of the power-law gradation parameters with VMA and VFA. The slope of the coarse portion of the gradation curve $n_{CA}$ had a stronger relationship with both volumetric parameters VMA and VFA than the rest of the gradation parameters from this method. Again, the finer the gradation, the lower the VMA and VFA become.

![Graphs showing the relationship between FAC ratio and mixture physical properties](image)

Figure 4.22 Bailey Fine Gradation Parameter; $F_{AC}$ Ratio and Mixture Physical Properties: (a) $F_{AC}$ Ratio vs. VMA (b) $F_{AC}$ Ratio vs. VFA (c) $F_{AC}$ Ratio vs. Effective Film Thickness
In summary, the analysis of the gradation parameters from both the Bailey method and the power-law method clearly demonstrates the sensitivity of asphalt mixtures volumetrics to these parameters.

Figure 4.23 Power-Law Coarse Gradation Parameter \( a_{CA} \) and Mixture Physical Properties

4.5.2 Gradation Parameters and Mixture Compactability

It was established earlier that compaction characteristics were different for mixtures with different aggregate gradations. In order to quantify the effect of aggregate gradation on the compactability of the mixtures, the gradation parameters from Bailey and the power law methods were utilized. Figures 4.28 through 4.30 describe the
relationship between mixture compactability, as represented by the SGC compaction densification index CDI, and those parameters from the gradation analysis.

![Graph](image)

**Figure 4.24 Power-Law Coarse Gradation Parameter nCA and Mixture Physical Properties**

![Graph](image)

**Figure 4.25 Power-law Fine Gradation Parameter aFA and Mixture Volumetrics**
Figure 4.26 Power-law Fine Gradation Parameter nFA and Mixture Volumetrics

Figure 4.27 Power-law Fine Gradation Parameters and Effective Film Thickness

Figure 4.28 Relationships of the Bailey Gradation Parameters with Mixture Compactability
CDI clearly does respond to change in the gradation parameters from the Bailey method, indicating that the compactability of the mixtures is a function (among other factors) of the particle size distribution as measured by those parameters. CA ratio and the nCA, and nFA had the best correlation with the CDI.

4.6 Mixture Performance

This section analyzes the performance of the asphalt mixtures as evaluated by the laboratory simulative and mechanistic tests.
4.6.1 Hamburg Wheel Tracking Test

The designed mixtures were evaluated for their performance under severe load and environmental conditions using the Hamburg Wheel Tracking test (HWT). This is a torture test to determine mixture resistance to rutting and moisture damage. The HWT device measures the combined effects of rutting and moisture damage by rolling a steel wheel across the surface of an asphalt concrete slab, 260.8 mm wide by 320.3 mm long and 40.0 mm thick that is immersed in hot water at a temperature of 50°C. Examination of the rut profile from the HWT test (Figure 4.31) shows that rutting at the ends of the specimen should be taken with caution since the end effect of the rigid mold might prevent the lateral flow of the mix under loading. Therefore, it was decided to only use the middle portion of the profile in the determination of the rut depth. The average of the middle six point measurements was ultimately used. Figure 4.32 presents the mean rut depths for all the mixtures in phase 1.

![Figure 4.31 Rut Profile from the HWT Test](image)
All the mixtures had excellent performance with a maximum rut depth of 4.4 mm after 20,000 passes for the one inch limestone coarse mixture. No signs of stripping were found at the end of the test period. The lowest rut depth was measured the sandstone medium mixture with only 1.5 mm rut depth after 20,000 passes.

An analysis of variance (ANOVA) was conducted using Statistical Analysis Software (SAS) to detect the effects of gradation and type of aggregates on the Hamburg rut depths. The ANOVA analyses were performed using “MIXED” procedure available in SAS. The linear model used in these analyses was a completely randomized factorial design (Gradation × Type), as shown in equation 4. The dependent variable used in the analyses was the rut depth in mm.

\[ Y_{ijk} = \mu + \tau_{ij} + \tau_{2j} + \tau_{1i}\tau_{2j} + \epsilon_{ijk} \]

In equation 4, \( \mu \) is the overall mean; \( \tau_{ij} \) is the effect of aggregate gradation; \( \tau_{2j} \) is the effect of aggregate type; \( \tau_{1i}\tau_{2j} \) is effect of the interaction between the gradation and
type; $\epsilon_{ijk}$ is the random sampling variation for observation $k$, at any level of gradation and type $ij$; and $Y_{ijk}$ is the dependent variable.

The results of the ANOVA analyses showed that, at a 95% confidence level, the aggregate gradation and type have significant effect on the measured HWT rut depth. In addition, the results showed that aggregate gradation had more significant effect rut depth than aggregate type, as indicated by the higher F-value. The interaction effect of the aggregate gradation and type ($\tau_1 \tau_{2ij}$) had no significant effect on the measured rut depth. Table 4.6 shows a summary of this analysis.

Based on the result of the ANOVA analyses, post ANOVA Least Square Means (LSM) analyses were conducted to compare the effect of all the different gradation and aggregate types used. Tukey adjustment was used in this analysis. Saxton’s macro (Saxton, 1998) was implemented to convert the results in the MIXED procedure to letter groupings. The results of this grouping are presented in Tables 4.7 and 4.8. In these tables the groups are listed in ascending order from the worst to the best. Groups with same letter are not significantly different. Medium and fine gradations showed similar performance (Same letter group). Among the three aggregate types used, Limestone showed the least rut resistance under the loading and environmental conditions of the HWT test.

The effect of aggregate gradation on HWT results was further investigated using the parameters obtained from the Bailey and the power law methods. Linear multiple regression analysis using SAS software was performed on the data to determine what gradation parameter(s) was contributing to the significance effect of gradation. Table 4.9 summarizes the results of the regression analysis. Two parameters showed significant
correlation with HWT rut depth at 95% confidence level. These are aCA and aFA from the power law method of gradation analysis. Figure 4.33 presents this correlation. As the mixes get finer the resistance to permanent deformation improves.

Table 4.6 Summary of the statistical analysis on HWT Data

<table>
<thead>
<tr>
<th>Effect</th>
<th>Num DF</th>
<th>Den DF</th>
<th>F Value</th>
<th>Pr &gt; F</th>
</tr>
</thead>
<tbody>
<tr>
<td>GRA</td>
<td>2</td>
<td>15</td>
<td>8.89</td>
<td>0.0028</td>
</tr>
<tr>
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<tr>
<td>GRA*TYPE</td>
<td>4</td>
<td>15</td>
<td>2.61</td>
<td>0.0775</td>
</tr>
</tbody>
</table>

Table 4.7 Effect=GRA Method= Tukey-Kramer (P<.05)

<table>
<thead>
<tr>
<th>Obs</th>
<th>GRADATION</th>
<th>Estimate</th>
<th>Standard Error</th>
<th>Letter Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C</td>
<td>3.2725</td>
<td>0.2042</td>
<td>A</td>
</tr>
<tr>
<td>2</td>
<td>M</td>
<td>2.2483</td>
<td>0.2042</td>
<td>B</td>
</tr>
<tr>
<td>3</td>
<td>F</td>
<td>2.1900</td>
<td>0.2042</td>
<td>B</td>
</tr>
</tbody>
</table>

Table 4.8 Effect=TYPE Method=Tukey-Kramer(P<.05)

<table>
<thead>
<tr>
<th>Obs</th>
<th>TYPE</th>
<th>Estimate</th>
<th>Standard Error</th>
<th>Letter Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>LS</td>
<td>3.1108</td>
<td>0.1581</td>
<td>A</td>
</tr>
<tr>
<td>5</td>
<td>GR</td>
<td>2.3833</td>
<td>0.2237</td>
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<td>6</td>
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<td>2.2167</td>
<td>0.2237</td>
<td>B</td>
</tr>
</tbody>
</table>
In summary, the results show that the performance of the mixtures in the HWT test is sensitive to some of the gradation parameters used to analyze the aggregate gradation in this study.

The results from the HWT test were also analyzed using the energy indices obtained from the SGC and the PDA. It was expected that the higher the TDI and TFI, the lower rut depths obtained from the HWT test, if those indices truly provide indication of mixture stability. The data, however, showed an unexpected increase in the rut depth after a certain value of TDI and TFI as shown in Figure 4.34. This raises a question on the suitability of the energy approach used to highlight plastic instability of asphalt mixtures. The inability of those indices to capture that can be attributed to the fact that the mixture is contained within the rigid walls of the compaction mold and the equally rigid top and bottom platens which prevent any type of lateral flow that constitutes the basic mechanism of permanent deformation in asphalt mixtures.

An important volumetric parameter in HMA is the voids in mineral aggregates (VMA). The effect of this parameter on the rutting performance of asphalt mixtures as measured by the HWT test is shown in Figure 4.35. A trend of increasing rut depth with higher VMA values is observed. The correlation however, is not statistically significant.

<table>
<thead>
<tr>
<th></th>
<th>Pr&gt;F</th>
<th>Correlation, α=0.05</th>
</tr>
</thead>
<tbody>
<tr>
<td>CA ratio</td>
<td>0.2699</td>
<td>Not significant</td>
</tr>
<tr>
<td>F_{AC} ratio</td>
<td>0.7362</td>
<td>Not significant</td>
</tr>
<tr>
<td>aCA</td>
<td>0.0064</td>
<td>Significant</td>
</tr>
<tr>
<td>nCA</td>
<td>0.1984</td>
<td>Not significant</td>
</tr>
<tr>
<td>aFA</td>
<td>0.0014</td>
<td>Significant</td>
</tr>
<tr>
<td>nFA</td>
<td>0.2824</td>
<td>Not significant</td>
</tr>
</tbody>
</table>
Figure 4.33 Gradation Analysis on HWT Test Results a) aCA vs. HWT b) aFA vs. HWT

Figure 4.34 Relationship Between the Energy Indices and HWT Results a) TDI vs. HWT Rut Depth. b) TFI vs. HWT Rut Depth

Figure 4.35 Effect of VMA on Rutting from HWT Test
4.6.2 Indirect Tensile Strength (ITS) Test

This test is one of the most popular tests used for characterizing HMA mixtures. It is a fundamental test that describes mixture cohesion. The test was conducted at 25°C according to AASHTO TP09. In this test, a cylindrical specimen is loaded to failure at deformation rate of 50.8 mm/min using a MTS machine. The IDT strength and tensile strain at failure were used in the analysis. Three SGC specimens were tested for each mixture. The test was conducted on two sets of samples, unaged and long-term oven aged. The long-term oven aging protocol recommended by AASHTO PP2 (1994) was followed. The specimens were placed in a force draft oven at 85°C for 5 days. Figure 4.36 presents the mean IT strength results of the unaged mixtures. In this test, high IT strength values at failure are desirable. Figure 4.37 presents the corresponding strain values. The IT strength values ranged from 116 psi to 309.3 psi for the unaged mixes and from 146.4 psi to 357.1 psi for aged ones. The strain results ranged from 0.40 percent to 0.90 percent for the unaged mixes and from 0.30 percent to 0.8 percent for the aged ones. The IT strength and strain values obtained from this study were compared to typical values obtained for Louisiana Superpave mixtures that have shown good field performance (Mohammad et al 2002). For mixtures with PG76-22, the reported IT strength values were in the range of 192 to 369 psi. The corresponding strain values ranged from 0.26 to 0.88 percent. The IT results from this study fall within that reported range indicating that the designed mixtures in this study can offer good field performance despite the relatively lower than recommended mixture volumetrics. The medium sandstone mixture had the highest strength value of 309 psi compared to the other mixtures. The performance of limestone and sandstone fine mixtures was similar. The lowest strength values were obtained for coarse mixtures for all of the three types of
aggregates. The values, however, are still high and fall within the established range for good performing asphalt mixtures.

Figure 4.36 Indirect Tensile Strength Results

Figure 4.37 Indirect Tensile Strain Results
Statistical analysis similar to that conducted on the HWT data was run on the IT strength and strain data in order to determine if the test parameters used in the analysis are sensitive to gradation, type or the combination of these two factors. Table 4.10 summarizes the results of this analysis on the IT strength data. The fixed effect of gradation and type of aggregates was found to be significant at 95% confidence level. The interaction effect however, does not seem to influence the results of the ITS test. The results of grouping the data based on gradation and type are presented in Tables 4.11 and 4.12. In these tables the groups are listed in descending order from the best to the worst. Groups with same letter are not significantly different. Medium and fine gradations showed similar performance in terms of IT strength (Same letter group). Among the three aggregate types used, Limestone had the lowest strength values.

Table 4.10 Summary of the statistical analysis on IT Strength Data

<table>
<thead>
<tr>
<th>Effect</th>
<th>Num DF</th>
<th>Den DF</th>
<th>F Value</th>
<th>Pr &gt; F</th>
</tr>
</thead>
<tbody>
<tr>
<td>GRA</td>
<td>2</td>
<td>27</td>
<td>11.83</td>
<td>0.0002</td>
</tr>
<tr>
<td>TYPE</td>
<td>2</td>
<td>27</td>
<td>17.91</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>GRA*TYPE</td>
<td>4</td>
<td>27</td>
<td>0.82</td>
<td>0.5259</td>
</tr>
</tbody>
</table>

Table 4.11 Effect=GRA Method= Tukey-Kramer (P<.05)

<table>
<thead>
<tr>
<th>Obs</th>
<th>GRADATION</th>
<th>Estimate</th>
<th>Standard Error</th>
<th>Letter Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>F</td>
<td>249.80</td>
<td>9.9026</td>
<td>A</td>
</tr>
<tr>
<td>2</td>
<td>M</td>
<td>247.28</td>
<td>9.9026</td>
<td>A</td>
</tr>
<tr>
<td>3</td>
<td>C</td>
<td>189.58</td>
<td>9.9026</td>
<td>B</td>
</tr>
</tbody>
</table>
Similarly, the IT strain data showed that gradation and type have significant fixed effect on the strain values but the interaction effect was not influencing the results significantly. The analysis is summarized in Tables 4.13 through 4.15. Coarse gradations had the highest strain values among the different gradations used. Sandstone mixtures are less favorable in terms of IT strain since they resulted in the lowest value among the three aggregate types used. In the ITS test, high strength values are desired for better cohesion characteristics while high strain values are desirable for better cracking resistance.

Table 4.13 Summary of the statistical analysis on IT Strain Data

<table>
<thead>
<tr>
<th>Effect</th>
<th>Num DF</th>
<th>Den DF</th>
<th>F Value</th>
<th>Pr &gt; F</th>
</tr>
</thead>
<tbody>
<tr>
<td>GRA</td>
<td>2</td>
<td>27</td>
<td>29.05</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>TYPE</td>
<td>2</td>
<td>27</td>
<td>19.29</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>GRA*TYPE</td>
<td>4</td>
<td>27</td>
<td>0.70</td>
<td>0.5968</td>
</tr>
</tbody>
</table>
Table 4.14 Effect=GRA  Method= Tukey-Kramer (P<.05)

<table>
<thead>
<tr>
<th>Obs</th>
<th>GRADATION</th>
<th>Estimate</th>
<th>Standard Error</th>
<th>Letter Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C</td>
<td>0.8085</td>
<td>0.02715</td>
<td>A</td>
</tr>
<tr>
<td>2</td>
<td>M</td>
<td>0.5658</td>
<td>0.02715</td>
<td>B</td>
</tr>
<tr>
<td>3</td>
<td>F</td>
<td>0.5456</td>
<td>0.02715</td>
<td>B</td>
</tr>
</tbody>
</table>

Table 4.15 Effect=TYPE  Method= Tukey-Kramer(P<.05)

<table>
<thead>
<tr>
<th>Obs</th>
<th>TYPE</th>
<th>Estimate</th>
<th>Standard Error</th>
<th>Letter Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>GR</td>
<td>0.7311</td>
<td>0.02974</td>
<td>A</td>
</tr>
<tr>
<td>5</td>
<td>LS</td>
<td>0.6932</td>
<td>0.02103</td>
<td>A</td>
</tr>
<tr>
<td>6</td>
<td>SS</td>
<td>0.4956</td>
<td>0.02974</td>
<td>B</td>
</tr>
</tbody>
</table>

Figure 4.38 presents the aging index calculated by dividing the aged IT strain by the unaged one. This index represents the amount of change in the IT strain values due to aging. Coarse mixtures of all the aggregate types were the least affected by aging than the fine and medium gradations. Table 4.16 lists all the mixtures and their corresponding aging indices together with the effect of aging as statistically described by the p-value from the t-test. The null hypothesis was that the unaged IT strain is the same as the aged one. The P-value was calculated and compared with the critical value of 0.05 to reject or accept the null hypothesis. The P-value indicates the extent to which a computed test statistic is unusual in comparison with what would be expected under the null hypothesis. A P-value greater than 0.05 indicates that the aged and unaged strain are statistically the same as that from the axial compression test.
<table>
<thead>
<tr>
<th>Mix Type</th>
<th>Unaged</th>
<th>Aged</th>
<th>Aging Index</th>
<th>P-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2” LSC</td>
<td>0.787</td>
<td>0.770</td>
<td>0.98</td>
<td>0.7582</td>
</tr>
<tr>
<td>1/2” LSM</td>
<td>0.620</td>
<td>0.530</td>
<td>0.85</td>
<td>0.0985</td>
</tr>
<tr>
<td>1/2” LSF</td>
<td>0.537</td>
<td>0.400</td>
<td>0.74</td>
<td>0.0765</td>
</tr>
<tr>
<td>1/2” SSC</td>
<td>0.653</td>
<td>0.620</td>
<td>0.95</td>
<td>0.6829</td>
</tr>
<tr>
<td>1/2” SSM</td>
<td>0.407</td>
<td>0.293</td>
<td>0.72</td>
<td>0.1695</td>
</tr>
<tr>
<td>1/2” SSF</td>
<td>0.427</td>
<td>0.326</td>
<td>0.76</td>
<td>0.0301</td>
</tr>
<tr>
<td>1/2” GRC</td>
<td>0.932</td>
<td>0.7</td>
<td>0.75</td>
<td>0.2747</td>
</tr>
<tr>
<td>1/2” GRM</td>
<td>0.843</td>
<td>0.553</td>
<td>0.66</td>
<td>0.0175</td>
</tr>
<tr>
<td>1/2” GRF</td>
<td>0.630</td>
<td>0.410</td>
<td>0.65</td>
<td>0.0057</td>
</tr>
<tr>
<td>1” LSC</td>
<td>0.891</td>
<td>0.679</td>
<td>0.76</td>
<td>0.0011</td>
</tr>
<tr>
<td>1” LSM</td>
<td>0.697</td>
<td>0.454</td>
<td>0.65</td>
<td>0.0065</td>
</tr>
<tr>
<td>1” LSF</td>
<td>0.625</td>
<td>0.445</td>
<td>0.71</td>
<td>0.0006</td>
</tr>
</tbody>
</table>

Table 4.16 Statistical Analyses on the Effect of Aging on IT Strain

Figure 4.38 Aging Index from the ITS Test
Among the mixture physical parameters considered, effective film thickness showed a strong correlation with the IT strain results for both aged and unaged mixtures (Figure 4.39). A trend of increasing aging index with higher film thickness is also observed in Figure 4.40. That explains why some mixtures are more affected by aging than others. Coarse mixtures that were least affected by aging had the highest film thicknesses compared to the other mixtures.

The influence of aggregate gradation on the tensile strength was explored using the different gradation parameters defined in this study. Only two gradation parameters had significant correlation with the IT strength results. These are aCA and aFA as shown in Table 4.17 and Figure 4.41. Similar to the trend observed with the HWT data, the finer the gradation, the higher the IT strength of the mix. The rest of the parameters did not show a significant correlation with the strength values.
Figure 4.40 Film Thickness and Aging Index using IT Strain at Failure

Table 4.17 Statistical Analysis of Gradation Parameters and IT Strength

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Pr&gt;F</th>
<th>Correlation, α=0.05</th>
</tr>
</thead>
<tbody>
<tr>
<td>CA ratio</td>
<td>0.7955</td>
<td>Not significant</td>
</tr>
<tr>
<td>FAC ratio</td>
<td>0.2130</td>
<td>Not significant</td>
</tr>
<tr>
<td>aCA</td>
<td>0.0270</td>
<td>Significant</td>
</tr>
<tr>
<td>nCA</td>
<td>0.4591</td>
<td>Not significant</td>
</tr>
<tr>
<td>aFA</td>
<td>0.0017</td>
<td>Significant</td>
</tr>
<tr>
<td>nFA</td>
<td>0.7955</td>
<td>Not significant</td>
</tr>
</tbody>
</table>

Figure 4.41 Gradation Analysis on IT Strength Test Results: a) aCA vs. ITS  b) aFA vs. ITS
The IT strength results were also correlated with the compaction parameters obtained from the SGC and the PDA as shown Figures 4.42 and 4.43. Although the correlation is not strong, a trend of increasing IT strength with the increase of those indices can be observed.

Figure 4.42 Traffic Densification Index (TDI) and IT Strength

Figure 4.43 Traffic Force Index and IT Strength
4.6.3 Semi-Circular Fracture Energy Test

The fracture resistance of the mixtures designed in this study was investigated using the J-integral approach. This test was conducted on two groups of specimens: unaged and oven aged. Figure 4.44 presents the results of the calculated Jc from the semi-circular notched fracture test for both groups. The Jc values ranged from 0.364 to 1.764 kJ/m$^2$ for unaged mixes and from 0.599 to 1.761 kJ/m$^2$ for aged ones. This Jc data range is on the same order of magnitude as those reported by Mohammad et al. (2005) for well-performing Superpave mixtures in the State of Louisiana. Mohammad et al. studied 13 Superpave mixtures with different gradations and binder types that have satisfactory field performance. He reported a Jc range of 0.57 to 1.53 kJ/m$^2$. The three coarse mixtures with PG76-22M binder in Mohammad’s study had fracture resistance between 0.73 and 0.83 kJ/m$^2$ compared to 0.599 to 1.764 kJ/m$^2$ obtained for the coarse mixtures in this study with the same binder type of PG76-22M. This clearly demonstrates that despite the fact that those mixtures do not meet the Superpave requirements in terms of volumetrics, they show comparable performance to well performing Superpave mixtures.

Data analysis showed that within each aggregate type, coarser mixtures had higher J1c compared to the medium and fine ones except for the one inch limestone in which the coarse mix showed the lowest Jc value. The highest fracture resistance was obtained by the half inch sandstone coarse mixture, which was about 79% higher than that obtained for half inch coarse limestone mixtures and 38% higher than the medium granite mixture. Aging of test specimens resulted in an increase in the fracture energy of the mixtures except for coarse mixtures. Figure 4.45 presents the effect of aging on the fracture resistance of the mixtures in consideration. A good correlation was obtained between the aging index from the Jc test and the mixtures effective film thickness.
(R²=0.7) in which the effect of aging is reduced by having thicker binder film around the aggregates (Figure 4.46).

Figure 4.44 Fracture Energy from the Semi-Circular Fracture Test

Figure 4.45 Effect of aging on J₁c a) aging Index data  b) Comparison of aged and unaged J₁c

Figure 4.46 Effect of Film Thickness on J₁c Aging Index
The relationship of the gradation parameters to fracture resistance is shown in Figures 4.47 and 4.48. Table 4.18 shows the strength of this relationship when analyzed statistically. Two parameters showed a statistically significant correlation with $J_{1c}$: the CA ratio from the Bailey method and $n_{CA}$ from the Power law method. Both describe the coarse portion of the gradation curve. The finer the mixture is, the lower the fracture energy obtained.

Table 4.18 Statistical Analysis of Gradation Parameters and $J_{1c}$ Test Data

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Pr&gt;F</th>
<th>Correlation, $\alpha=0.05$</th>
</tr>
</thead>
<tbody>
<tr>
<td>CA ratio</td>
<td>0.0298</td>
<td>Significant</td>
</tr>
<tr>
<td>$F_{AC}$ ratio</td>
<td>0.7546</td>
<td>Not significant</td>
</tr>
<tr>
<td>aCA</td>
<td>0.2657</td>
<td>Not significant</td>
</tr>
<tr>
<td>nCA</td>
<td>0.0286</td>
<td>Significant</td>
</tr>
<tr>
<td>aFA</td>
<td>0.3528</td>
<td>Not significant</td>
</tr>
<tr>
<td>nFA</td>
<td>0.4569</td>
<td>Not significant</td>
</tr>
</tbody>
</table>

Figure 4.47 Effect of the Gradation Parameter CA Ratio on $J_c$
Figure 4.48 Effect of the gradation parameter nCA on Jc
CHAPTER 5. PHASE 2 RESULTS AND ANALYSIS

5.1 Introduction

From the literature review presented in this document, it was evident that there were mainly two existing approaches for designing hot mix asphalt. The first approach is based on the concept of using adequate VMA while the second one advocates the use of adequate asphalt film thickness. In both cases, the objective has been to determine a systematic way of designing mixes through the specification of desirable levels of volumetric properties, and using specified compactive effort. It was also evident from phase one of this research that neither adequate VMA nor the design number of gyrations is the same for mixes with different aggregate types and structures. Different mixes responded differently to the applied compaction energy which makes the current approach of specifying the same design number of gyrations to all different mixes in the same traffic level questionable.

The results of phase 1 of this research were presented to a number of researchers and field engineers in the asphalt industry at several technical meetings nationwide. There was a concern that the designed mixtures maybe too dry and might have durability problems. Therefore, a test plan was developed to determine if it is appropriate to improve mixtures durability by using a number of gyrations that is mix-specific and lower than that recommended by the current Superpave system. The premise was that using a lower number of gyrations will increase the design asphalt content and hence improve durability. The suggested approach was to utilize the concept of locking point in specifying the design number of gyrations. It was shown that the locking points of all the mixtures designed in this study were different and are lower than the currently specified
single $N_{des}$ for all the mixes in the traffic level considered. The devised plan involved the following tasks:

- Determine the design asphalt content for selected mixtures from phase one using their locking points.
- Evaluate the rutting resistance of the mixtures designed in the previous step to ensure that stability is not compromised by using higher asphalt contents.
- Run a suite of mechanistic tests on the mixtures with more emphasis on the durability aspect of mixture performance.

5.2 Mixture Selection for Phase 2

Phase 2 of this study required the selection of limited number of mixtures from phase one for mixture design using the locking point concept as opposed to the traditional Superpave $N_{des}$. For the one inch limestone mixture, three different aggregate structures were formulated. Medium and fine mixtures showed similar performance that is relatively better than the coarse mixture. The fine mixture was selected for inclusion in phase 2. For half inch mixtures, a scoring system was developed to rationalize the selection process and assist in making objective decision regarding what mixtures to be included in the second phase.

The scoring system is based on some key mixtures properties that are related to mixture performance. The performance is quantified using the laboratory test parameters obtained from the first suite of testing on the mixtures in phase one. The mix attributes used in the scoring system are:

1- Stability
2- Durability
3- Compactability

Table 5.1 lists the attributes used and their corresponding test parameters together with their assigned numerical weights. A weighted score was calculated for each of the three mix attributes. The score is based on a seed value for each mix property considered. This seed value represents the maximum value obtained for that particular test parameter for the mixtures in consideration. For example, the maximum IT strength value obtained was 357.1 psi for the medium sandstone mixture. Therefore, the seed value for the IT strength parameter is 357.1 and therefore, IT strength score for the medium sandstone mixture will be 1.0. The half inch limestone mixture has a strength value of 195.1 psi which results in an IT score of 0.55 (195.1 divided by 357.1). The seed value for the HWT parameter however, was taken as the maximum allowable rut depth specified by the Louisiana Department of Transportation and Development (LADOTD) which is 6.0mm. The final rating assigned to the mix is based on the sum of the three individual scores using the following equation:

\[
Total \ Score = ITS \ score \cdot w_1 + Jc \ Score \cdot w_2 + \left(\frac{1}{HWT \ score}\right) \cdot w_3 + \left(\frac{1}{CDI \ score}\right) \cdot w_4
\]

Table 5.1 Mixture Attributes Used in the Selection Procedure

<table>
<thead>
<tr>
<th>Attribute</th>
<th>Laboratory Test Parameter</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stability</td>
<td>HWT Rutting</td>
<td>33.33</td>
</tr>
<tr>
<td>Durability</td>
<td>Aged Indirect Tensile Strength</td>
<td>16.67</td>
</tr>
<tr>
<td></td>
<td>Aged Critical J-integral</td>
<td>16.67</td>
</tr>
<tr>
<td>Compactability</td>
<td>SGC Compaction Densification Index</td>
<td>33.33</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>100.00</td>
</tr>
</tbody>
</table>
The equation above is additive in nature and was formulated based on the desired mixture performance from each test. Higher ITS and Jc values are desired and therefore, those two parameters were multiplied directly by their weights. On the other hand, lower rutting from HWT is sought for adequate mixture stability and hence the inverse of rut depth from HWT was used to calculate the contribution of this parameter to the final score. Similarly, Lower CDI indicates better compactability and that led to the use of the inverse of this parameter in the calculation of the compactability contribution to the final score. Table 5.3 summarizes the ranking of the mixtures based on this scoring system. The mixtures selected were as follows: The one with the highest ranking (fine granite-rank #1), the mixture with a medium ranking (fine limestone, ranked #5), and the mixture at the bottom of the list (coarse limestone-ranked #9). It was also decided to include a mixture that has a very high Dust/Pbeff ratio as this is considered to be a problematic one in terms of durability (medium sandstone).

5.3 Analysis of Results and Discussion

The data analysis will consist of comparing mixture physical properties and performance test results of phase 2 mixtures to those designed using the Superpave recommended design number of gyrations in phase one.

5.3.1 Mixtures Physical Properties

Graphical comparisons of the mixtures physical properties of both sets of mixtures from phases 1 and 2 are presented in Figures 5.1 through 5.5. As anticipated, compacting mixtures to their locking point yielded higher design asphalt contents than those obtained when Superpave design number of gyrations was used. The design asphalt content for phase 2 mixtures ranged from 3.9% to 5.4% compared to 3.3% to 5.1% for the same mixtures designed in phase 1. It is worth noting that except for half inch coarse
limestone mixture, there was about 0.6% increase in asphalt content for all other mixtures when the mixtures were designed using their locking points at the same level of 4.0% air void.

The Voids in Mineral Aggregates (VMA) values were about 1.1% to 1.2% higher for the mixtures designed in phase 2 except for the medium sandstone mixture in which there was 0.8% increase. Again, this finding clearly indicates that VMA is compaction dependent and specifying it based on NMAS only as currently adopted by the Superpave design system is questionable.

Higher asphalt contents naturally resulted in higher VFA, lower Dust/Pbeff ratio, and hence higher effective film thickness for the mixtures in considerations as shown in Figures 5.3 to 5.5.
Table 5.2 Selection Procedure for half inch mixtures

<table>
<thead>
<tr>
<th>Mixture Type</th>
<th>IT Strength</th>
<th>Score</th>
<th>Jc Score</th>
<th>Durability Weighted Score</th>
<th>HWT Score</th>
<th>Stability Weighted Score</th>
<th>CDI Score</th>
<th>Compaction Weighted Score</th>
<th>Total Weighted Score</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2&quot; LS C</td>
<td>195.1</td>
<td>0.55</td>
<td>0.699</td>
<td>0.40</td>
<td>15.7</td>
<td>0.62</td>
<td>54.1</td>
<td>1067.8</td>
<td>33.33</td>
</tr>
<tr>
<td>1/2&quot; LS M</td>
<td>238.7</td>
<td>0.67</td>
<td>0.817</td>
<td>0.46</td>
<td>18.9</td>
<td>0.35</td>
<td>95.2</td>
<td>721.9</td>
<td>49.30</td>
</tr>
<tr>
<td>1/2&quot; LS F</td>
<td>281.7</td>
<td>0.79</td>
<td>0.768</td>
<td>0.44</td>
<td>20.4</td>
<td>0.45</td>
<td>74.1</td>
<td>556.6</td>
<td>63.95</td>
</tr>
<tr>
<td>1/2&quot; SST C</td>
<td>270.3</td>
<td>0.76</td>
<td>1.106</td>
<td>0.63</td>
<td>23.1</td>
<td>0.53</td>
<td>62.5</td>
<td>916.1</td>
<td>38.85</td>
</tr>
<tr>
<td>1/2&quot; SST M</td>
<td>357.1</td>
<td>1.00</td>
<td>0.842</td>
<td>0.48</td>
<td>24.6</td>
<td>0.25</td>
<td>133.3</td>
<td>800.2</td>
<td>44.48</td>
</tr>
<tr>
<td>1/2&quot; SST F</td>
<td>317.7</td>
<td>0.89</td>
<td>0.807</td>
<td>0.46</td>
<td>22.5</td>
<td>0.33</td>
<td>100.0</td>
<td>762.0</td>
<td>46.71</td>
</tr>
<tr>
<td>1/2&quot; GR C</td>
<td>185.3</td>
<td>0.52</td>
<td>1.279</td>
<td>0.73</td>
<td>20.8</td>
<td>0.43</td>
<td>76.9</td>
<td>963.3</td>
<td>36.95</td>
</tr>
<tr>
<td>1/2&quot; GR M</td>
<td>251.0</td>
<td>0.70</td>
<td>1.761</td>
<td>1.00</td>
<td>28.4</td>
<td>0.47</td>
<td>71.4</td>
<td>682.0</td>
<td>52.19</td>
</tr>
<tr>
<td>1/2&quot; GR F</td>
<td>284.7</td>
<td>0.80</td>
<td>1.699</td>
<td>0.96</td>
<td>29.4</td>
<td>0.28</td>
<td>117.6</td>
<td>609.3</td>
<td>58.42</td>
</tr>
<tr>
<td>Seed Value</td>
<td>357.1</td>
<td>1.761</td>
<td></td>
<td></td>
<td>6.0</td>
<td></td>
<td>1067.8</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 5.3 Overall Ranking of the Mixtures

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Total Score</th>
<th>Ranking</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2&quot; Gr F</td>
<td>205.4</td>
<td>1</td>
</tr>
<tr>
<td>1/2&quot; SST M</td>
<td>202.4</td>
<td>2</td>
</tr>
<tr>
<td>1/2&quot; SST F</td>
<td>169.2</td>
<td>3</td>
</tr>
<tr>
<td>1/2&quot; LS M</td>
<td>163.4</td>
<td>4</td>
</tr>
<tr>
<td>1/2&quot; LS F</td>
<td>158.4</td>
<td>5</td>
</tr>
<tr>
<td>1/2&quot; Gr M</td>
<td>152.0</td>
<td>6</td>
</tr>
<tr>
<td>1/2&quot; GR C</td>
<td>134.6</td>
<td>7</td>
</tr>
<tr>
<td>1/2&quot; SST C</td>
<td>124.4</td>
<td>8</td>
</tr>
<tr>
<td>1/2&quot; LS C</td>
<td>103.1</td>
<td>9</td>
</tr>
</tbody>
</table>

Maximum Possible Score* 226.7

* This score is calculated assuming the mixtures get the best score for all the attributes

Figure 5.2 Voids in the Mineral Aggregate Data

VMA, %

| Mixture | Voids
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>GRF</td>
<td>10.9</td>
</tr>
<tr>
<td>LSF</td>
<td>9.4</td>
</tr>
<tr>
<td>LSC</td>
<td>13.5</td>
</tr>
<tr>
<td>SSM</td>
<td>8.4</td>
</tr>
<tr>
<td>1&quot; LSF</td>
<td>10.0</td>
</tr>
</tbody>
</table>

Ndes

Locking Point
Figure 5.3 Voids Filled with Asphalt Data

Figure 5.4 Dust/Pbeff Results
5.3.2 Performance Tests Results

For comparison and determination of relative performance, phase 2 mixtures were evaluated using similar suite of testing conducted in phase one, mainly Hamburg Wheel Tracking Test (HWT), IT strength test (ITS), and fracture resistance using the notched semi-circular fracture energy test (Jc). In addition, another two fundamental properties were determined for phase 2 mixtures: Stiffness characteristics using the dynamic modulus test ($E^*$) and the cracking resistance using the concept of dissipated creep strain energy.

The performance of the mixtures in the HWT test is shown in Figure 5.6 together with the corresponding data from phase one. There was a slight increase in the amount of rutting from phase one partly due to higher asphalt contents used. The highest rut depth was 4.0 mm for half inch coarse limestone. The results however, are still within the
range of good performing mixtures indicating that stability was not compromised by
designing the mixes using lower compaction levels.

![Hamburg Wheel Tracking Results](image)

Figure 5.6 Hamburg Wheel Tracking Results

The cohesion characteristics of the mixtures were determined using the IT
strength test. Three parameters from this test were used in the analysis. The parameters
are: Aged IT strength, Aged IT strain, and toughness index (TI). Tensile strength values
were slightly lower than those obtained for the mixtures compacted at $N_{des}$. The strength
values ranged from 168.3 for the half inch coarse limestone mixture to 325.0 psi for the
half inch medium sandstone mixture (Figure 5.7). The highest reduction in strength was
observed for the half inch fine limestone mixture which had a strength value of 27.8%
lower than that obtained for the same mix designed using the Superpave recommended
$N_{des}$. The lowest change in strength was observed for the one inch fine limestone with
only 4.1% reduction in strength.
Analyzing the strain data presented in Figure 5.8 clearly indicates that the mixtures now exhibit higher IT strain values at failure which implies that they will retain more flexibility over time compared to phase one mixtures and that makes them relatively less prone to pre-mature failures due to aging.

Both unaged and aged toughness index data are presented in Figures 5.9 and 5.10. The lowest toughness index was obtained for the medium sandstone mixture, followed by the half inch fine limestone. Those two mixtures had the lowest effective film thickness and the highest dust/Pbeff ratio. Their TI values although still not considerably low (>0.5), the fact that they exhibited a lower TI values compared to other mixtures makes them less favorable in terms of their ability to resist aging effect over time. It should be noted that all the mixtures showed better toughness properties at their locking points than at Superpave N_{des}.

![Figure 5.7 IT Strength Comparison](image-url)
Figure 5.8 IT Strain Comparison

Figure 5.9 Unaged Toughness Index
Figure 5.10 Aged Toughness Index

Figure 5.11 presents the calculated J-integral from the semi-circular notched fracture test. The test was conducted on mixtures that were aged for 5 days in a forced-draft oven at 85ºC. All the mixtures exhibited an increase in their fracture resistance when designed using the locking point except for the one inch fine limestone mixture in which there was a drop of about 35% in Jc. Half inch granite fine mixture showed the same fracture resistance under both N<sub>des</sub> and locking point which was the highest among the mixtures tested. The biggest improvement in fracture resistance was observed for the half inch coarse limestone mixture for which there was about 49% increase in Jc when designed using the locking point followed by the half inch fine limestone mixture which had about 35% increase in Jc. The half inch medium sandstone mixture gained about 20% increase in Jc.
5.3.3 Stiffness Characteristics

The stiffness properties of the mixtures designed in phase 2 were evaluated using the dynamic modulus test. The AASHTO TP-62-03 standard was followed. Two parameters were obtained from this test, the dynamic modulus ($E^*$) and the phase angle. Witczak et al. (2002) conducted a detailed study to evaluate candidate mechanistic parameters that correlates with mixtures performance. The major finding of this study was the recommendation of a set of parameters for two distresses in asphalt layer of road pavement which are permanent deformation, load associated cracking. One of the parameters recommended for the permanent deformation was the dynamic modulus term $E^*/\sin\varnothing$ where $\varnothing$ is the phase angle. Higher values of this parameter indicate stiffer mixtures that have good permanent deformation resistance. $E^*$ alone was also suggested.
as an indicator for rutting resistance of asphalt mixtures. For Fatigue, the recommended parameter was $E^*\sin \phi$.

Permanent deformation is a distress that is associated with excessive loading at relatively high pavement temperatures. The stiffness characteristics and ultimately the rutting resistance of the designed mixtures as defined by the parameter described above was evaluated under two loading conditions that are likely to cause the highest damage to the pavement. The first condition is: high temperature-high frequency of loading in which high traffic speed is simulated by a frequency of 10HZ which represents approximately 60mph speed. The second condition is high temperature-low frequency of loading in which slow moving traffic is simulated using a 0.5HZ loading frequency. This frequency approximates slow traffic at intersections. In both cases, the temperature selected was 54.4°C which is the highest test temperature required by the testing protocol used. Figures 5.12 and 5.13 presents the dynamic modulus and the rutting parameter $E^*/\sin \phi$ for the mixtures considered. The sandstone mixture is clearly showing the highest rutting parameter among all the mixtures considered. The lowest rutting parameter was obtained for the coarse limestone mixture which suggests that this mixture will exhibit less rut resistance compared to the sandstone mixture. The coarse limestone mixture showed the highest rut depth as measure by the HWT test as discussed early in this chapter. Both one inch and half inch limestone mixtures showed similar performance. The relative performance of the mixtures was the same under both loading conditions of fast and slow moving traffic.

Figure 5.14 presents the data for the fatigue parameter $E^*\sin \phi$. The mixtures are evaluated under service temperature of 21.1°C and loading frequency of 10HZ. The lower this parameter is, the better the fatigue resistance obtained. It is evident from the
data that Coarse limestone mixture has the best fatigue performance as measured by the fatigue parameter described in here. That was expected since this mixture is relatively rich in asphalt content and has the lowest Dust/Pbeff ratio among all the mixtures considered. That resulted in a mixture with better flexibility characteristics that can tolerate relatively more repetitive loading without fracture. Granite mixture was ranked second in terms of fatigue resistance while the rest of the three mixtures showed similar performance.

Figure 5.12 E* data at 10HZ, 54.4ºC

Figure 5.13 E* data at 0.5HZ, 54.4ºC

5.3.4 Dissipated Creep Strain Energy

One of the main arguments presented in this research is that every HMA mixture is unique in its performance and therefore, setting up general requirements (either
volumetric or densification) that are empirical in nature and heavily rely on personal experiences with specific types of mixtures is very likely to limit the use of good performing mixtures only on the bases of not meeting such empirical requirements. The mixtures designed in this study are likely to be rejected if they were to be judged using the traditional volumetric criteria adopted in the current Superpave design system. It was therefore, imperative to validate the performance of these mixtures by comparing them to good performing field mixtures that were in place for a reasonable amount of time using a fundamental material property that describes the behavior of the mixtures in consideration which is in this case durability in terms of resistance to cracking.

![Figure 5.14 E* data at 10HZ, 21.1°C](image)

It was mentioned earlier in this chapter that there was a concern that the designed mixtures might have durability problems. To address that concern, it was decided to use a mechanistic property that relates to mixtures durability in terms of resistance to load associated cracking after being in service for a period of time within which the mixtures undergo age hardening due to several environmental factors that are supposed to lower the capability of the mixtures to withstand the applied load.

Roque et al (2004) proposed the Dissipated Creep Strain Energy (DCSE) limit as one of the most important factors that control crack performance and hence durability of asphalt concrete mixtures. Roque studied 22 field mixtures that have been in service for
more than 10 years in the State of Florida. In order to determine this parameter, two laboratory tests conducted on the same specimen. These tests are: the indirect resilient modulus test and the indirect tensile strength test. Both tests were conducted at 10° C on 150mm diameter and 50 mm thick specimens. From the strength test, failure strain ($\varepsilon_f$), tensile strength ($S_t$) and fracture energy (FE) were determined. From the resilient modulus test, the resilient modulus ($M_R$) was found (Please refer to Figure 5.15). The calculation of the DCSE was then determined as follows:

$$\varepsilon_0 = \frac{(M_R \ast \varepsilon_f - S_t)}{M_R}$$

$$EE = \frac{1}{2} S_t (\varepsilon_f - \varepsilon_0)$$

$$DCSE = FE - EE$$

Table 5.4 summarizes the DCSE results obtained for all the mixtures in phase 2. The results are also presented graphically in Figure 5.16.

![Figure 5.15 Calculations of the Dissipated Creep Strain Energy](image_url)
The dissipated creep strain energy threshold represents the energy that the mixture can tolerate before it fractures. Therefore, it is logical that mixtures with higher DCSE thresholds will exhibit better cracking performance than mixtures with lower DCSE thresholds when both are exposed to similar environmental and loading conditions. From the data presented, it is clear that the half inch limestone coarse mixture is favorable in terms of cracking resistance since it has the highest DCSE limit followed by the half inch fine granite mixture and then the one inch fine limestone mixture. Half inch fine limestone and medium sandstone had the lowest DCSE. These two mixtures were relatively dry in asphalt and exhibited high stiffness characteristics in terms of E*.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Mr, Gpa</th>
<th>Fracture Energy, kJ/m3</th>
<th>ITS, MPa</th>
<th>Final Strain, microns</th>
<th>Initial Strain, microns</th>
<th>Elastic Energy, kJ/m3</th>
<th>DCSE-kJ/m3</th>
</tr>
</thead>
<tbody>
<tr>
<td>GRF</td>
<td>19.4</td>
<td>1.5</td>
<td>2.8</td>
<td>570</td>
<td>569.86</td>
<td>0.20</td>
<td>1.30</td>
</tr>
<tr>
<td>LSC</td>
<td>18.1</td>
<td>1.62</td>
<td>2.3</td>
<td>713</td>
<td>712.87</td>
<td>0.15</td>
<td>1.47</td>
</tr>
<tr>
<td>LSF</td>
<td>23.2</td>
<td>0.95</td>
<td>3</td>
<td>370</td>
<td>369.87</td>
<td>0.19</td>
<td>0.76</td>
</tr>
<tr>
<td>SST</td>
<td>25.5</td>
<td>0.97</td>
<td>3.4</td>
<td>350</td>
<td>349.87</td>
<td>0.23</td>
<td>0.74</td>
</tr>
<tr>
<td>1&quot;LSF</td>
<td>25.5</td>
<td>1</td>
<td>2.815</td>
<td>431.6</td>
<td>431.49</td>
<td>0.16</td>
<td>0.84</td>
</tr>
</tbody>
</table>

Rouque et al 2004 stated the following as one of his main observations:

“For high traffic pavements, such as interstate pavements having substantial pavement structures, it was found that the dissipated creep strain energy threshold of the mixture was a good indicator of top-down cracking performance. For these types of pavements, top-down cracking was observed when the dissipated creep strain energy threshold of the surface course mixture was less than about 0.75 kJ/m3 at 10°C.”
Examining the DCSE data obtained for phase 2 mixtures it is evident that the half inch fine limestone and medium sandstone mixtures are on the boarder line of the 0.75 kJ/m$^3$ DCSE limit below which cracking might be a problem. The best performing mixtures was the half inch coarse limestone followed by fine granite and finally the one inch fine limestone. It should be noted that all the four mixtures in phase 2 have volumetric properties that are considered inferior using the current Superpave mix design criteria.

![Figure 5.16 Dissipated Creep Strain Energy of the Designed Mixtures](image)

The performance of mixtures as described by the DCSE was compared to their performance when the fatigue parameter $E^*\sin\theta$ from the dynamic modulus test was used. Both parameters gave similar ranking for coarse limestone and fine granite mixtures. The performance of the sandstone mixture and the two limestone mixtures was very similar when evaluated using both parameters although the ranking is different but
that is probably due to the small differences in magnitude of the parameter in question between the mixtures in both cases. Figure 5.17 shows that a reasonable correlation between the DCSE and the fatigue parameter from the dynamic modulus test indicating that both parameters follow the same trend in describing the cracking resistance of the mixtures.

![Graph showing the relationship between DCSE and the Fatigue Parameter from the Dynamic Modulus Test](image)

Figure 5.17 Relationship between DCSE and the Fatigue Parameter from the Dynamic Modulus Test

5.4 Performance of Low Volume Mixtures

Low volume roads, which can be defined as roads with low number of vehicles per day and low cumulative equivalent single axle load (ESAL) in design period are generally built with lower quality materials compared to roads with higher traffic demands. Recipe type mix design is traditionally used when designing asphalt mixtures for this type of application. A small study was conducted as part of this research to evaluate the laboratory performance of asphalt mixtures intended for low volume application and which contain high amount of natural sand. Natural sand is a fine material that is considered a cheap commodity and it is widely available across country. Limestone aggregates was chosen as the type of aggregate to be used since it is one the
most commonly used aggregate types in the State of Louisiana. Two fine limestone mixtures were designed for this study: 1/2” NMAS and 1” NMAS. The aggregate structures in those two mixtures were proportioned using the Bailey method of aggregate gradation calculation procedure. It was decided however, to proportion the aggregates so that the mix contains at least 25% natural sand. Given that constraint, it was impossible to meet all the Bailey method recommended ranges of gradation evaluation ratios. The gradation characteristics of the low volume mixtures were presented in section 3.2 of this report but they are presented in this section for the reader’s convenience. Table 5.5 Presents mixture design data for these two mixtures.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>CA Volume</th>
<th>FA Volume</th>
<th>CUW</th>
<th>%PCS</th>
<th>CA Ratio</th>
<th>F_{AC} Ratio</th>
<th>F_{AF} Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>LSF-1&quot;</td>
<td>37.3</td>
<td>62.7</td>
<td>70</td>
<td>52.9</td>
<td>1.134</td>
<td>0.617</td>
<td>0.317</td>
</tr>
<tr>
<td>LSF-1/2&quot;</td>
<td>37.1</td>
<td>62.9</td>
<td>68</td>
<td>49.2</td>
<td>0.842</td>
<td>0.246</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Table 5.6 Mix Design Properties for Low Volume Mixes

<table>
<thead>
<tr>
<th>Mixture Type</th>
<th>1/2” Limestone-LV</th>
<th>1.0” Limestone-LV</th>
</tr>
</thead>
<tbody>
<tr>
<td>OAC @ 4.0% AV (Ndes=75)</td>
<td>4.9</td>
<td>4.3</td>
</tr>
<tr>
<td>Effective AC content @ 4.0% AV</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>VMA</td>
<td>13.1</td>
<td>12.2</td>
</tr>
<tr>
<td>VFA</td>
<td>69.0</td>
<td>67.2</td>
</tr>
<tr>
<td>Effective Film Thickness @ 4.0% AV and OAC</td>
<td>6.9</td>
<td>7.4</td>
</tr>
<tr>
<td>Dust/Pbeff</td>
<td>1.4</td>
<td>1.2</td>
</tr>
<tr>
<td>Sand Content (%)</td>
<td>25.3</td>
<td>25.2</td>
</tr>
</tbody>
</table>

The low volume mixtures were further evaluated using similar suite of laboratory tests including HWT, Jc, E*, and DCSE. The results are presented in Figures 5.18.
As expected, the high volume mixture outperformed the low volume ones in all the tests conducted. The one inch low volume mix showed a lower rutting performance as described by the rutting parameter from the dynamic modulus (E*) test. The same mixture also was less fatigue resistant using the fatigue parameter from the same E* test. That observation however was contradicted by the results of the DCSE test in which the 1” low volume mixture had a higher DCSE than the half inch mixture.

Mixture design properties, mainly effective film thickness and Dust/Pbeff ratio are more in line with the results from the DCSE test. 1” limestone mixture has higher effective film thickness and lower dust/Pbeff ratio suggesting that it should have better durability than the half inch mixture. The resistance of low volume mixtures to permanent deformation was evaluated using the HWT test. The test was run for 20,000 passes or until specimen fails. Both mixtures failed the 6.0mm rutting criterion generally specified. Figures 5.21 and 5.22 Show the results of the HWT for these two mixtures. It should be noted that applying the same criterion of 6.0mm at 20,000 passes for both high and low volume mixtures is unjustifiable. Low volume mixes are subjected to significantly less amount of traffic than the high volume ones. Therefore, the low volume mixtures might not experience the level of loads that is equivalent to the 20,000 passes used in the HWT test. Examining Figures 5.21 and 5.22, it is clear that the mixtures maintained reasonable rut resistance for about 10,000 passes indicating that those mixtures might still provide adequate performance for the purpose they are intended for. Figure 5.23 shows the number of passes required to cause 6.0 mm rutting for both mixtures. It took about 11226 passes to cause 6.0 mm rut depth in the one inch mixture compared to 7426 passes for the half inch mixture indicating that higher rut resistance is offered by the one inch mixture.
Figure 5.18 Comparison of Rutting Parameter of Both High and Low Volume Limestone Mixtures

Figure 5.19 Comparison of Fatigue Parameter of Both High and Low Volume Limestone Mixtures
Figure 5.20 DCSE Results for Low Volume Mixtures

Figure 5.21 Performance of 1” Limestone Low Volume Mixtures in HWT Test
Figure 5.22 Performance of 1/2” Limestone Low Volume Mixture in HWT Test

Figure 5.23 Performance Comparison of Low Volume Mixes in HWT Test
5.5 Recommended Design Approach

The research presented herein suggests that suitable mixes can be developed with dense aggregate structures using the Bailey method of aggregate gradation that provides good resistance to permanent deformation while still maintaining adequate level of durability. The research also recognizes the limitation of setting strict empirical criteria for mixtures volumetrics that might narrow the options of the design engineer to be more innovative and develop mixtures that are well performing yet economical.

The suggested design approach has the following advantages:

- Utilizes an analytical aggregate blending method that provides a rational and systematic approach to designing aggregate structures instead of the conventional trial and error procedure.
- Acknowledges the fact that every asphalt mixture is unique in its composition and response to compaction loads during construction. The procedure calls for using the concept of locking point that defines a unique compaction level for every mixture in consideration. It provides a predictive equation to estimate the locking point based on some aggregate characteristics.
- Bypasses the controversial empirical design step in the current Superpave design system; mixtures volumetrics requirements that have been the basis for acceptance/rejection of mixtures based only on failing to meet one or more volumetric parameters that are in most cases indirectly calculated from other laboratory test procedures that have high level of subjectivity.
- Checks the mixtures against two important pavement distresses; rutting and fatigue cracking using engineering properties determined from laboratory mechanistic tests that are relatively fast and simple to perform.
5.5.1 Estimating Locking Point

To facilitate the design process, a multiple linear regression model was developed using SAS software to estimate the locking point of the mixture based on certain properties that is thought to influence the performance of the mixture during compaction. The response parameter used was the locking point (LP). Since the compaction process is always performed at elevated temperatures, the influence of aggregate structure is thought to be more pronounced than that of the binder although the binder will still maintain some lubrication effect that might contribute to the mixtures response to the applied compaction energy. Several parameters were first introduced in the model including different characteristics of the gradation curves of the designed aggregate structure as well as binder content. A stepwise variable selection procedure was first performed on a general model that contains those variables. The purpose of such a procedure is to remove insignificant variables from the general model. The regression analysis was then conducted on the reduced model determined using the stepwise variable selection procedure. Three parameters were used in the regression analysis that were significant when included in the model as independent variables. These are:

- Volume of coarse aggregate in the aggregate structure (VCA)
- Percent Passing #200 sieve for the aggregate structure in consideration. This parameter is termed as “P_{200}”.
- Estimated initial asphalt content (AC).

The predictive model used is:

\[
LP=1.38\cdot VCA + 0.62\cdot P_{200} \cdot AC - 6.86 \tag{5.1}
\]

where LP, VCA, DAC are:
LP = Locking Point to be estimated

VCA = Volume of coarse aggregate in the aggregate structure

P_{200} \times AC = the interaction between the effect of the amount of material passing #200 sieve in the aggregate structure and the estimated asphalt content.

The results of the regression procedure are shown in Table 5.7. The F-Value for the model was 45.44 with a P-value of <0.0001. This indicates that the model is significant in describing the relationship between the response variable and the independent variables. All the parameter estimates for the predictor variables in the model were significant at the 95% significance level selected for the analysis. The model was also checked for any collinearity between the predictor variables. When there is a perfect linear relationship among the predictors, the estimates for a regression model cannot be uniquely computed. The term collinearity describes two variables that are near perfect linear combinations of one another. When more than two variables are involved, it is often called multicollinearity, although the two terms are often used interchangeably. The primary concern is that as the degree of multicollinearity increases, the regression model estimates of the coefficients become unstable and the standard errors for the coefficients can get wildly inflated.

The ‘vif’ option was used to check for multicollinearity. The ‘vif’ stands for variance inflation factor. As a rule of thumb, a variable whose ‘vif’ value is greater than 10 may merit further investigation. A comparison between the measured and predicted response variable is shown in Figure 5.24. Figure 5.25 is a flow chart that describes the recommended design approach a systematic way that guides the designer through the different stages of the design process.
### Table 5.7 Linear Regression Analysis to Estimate Locking Point

#### Analysis of Variance

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<tr>
<th>Source</th>
<th>DF</th>
<th>Sum of Squares</th>
<th>Mean Square</th>
<th>F Value</th>
<th>Pr &gt; F</th>
</tr>
</thead>
<tbody>
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<td>Model</td>
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<td>1413.28</td>
<td>706.64</td>
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<td>Error</td>
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<td>171.08</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Corrected Total</td>
<td>13</td>
<td>1584.36</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Root MSE</td>
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<td>R-Square</td>
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<table>
<thead>
<tr>
<th>Source</th>
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<th>Dependent Mean</th>
<th>Adj R-Sq</th>
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<tbody>
<tr>
<td>Intercept</td>
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<td>71.21</td>
<td>0.87</td>
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<tr>
<td>Coeff Var</td>
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<td>5.54</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### Parameter Estimates

| Variable | DF | Parameter Estimate | Standard Error | t Value | Pr > |t| Tolerance | Variance Inflation |
|----------|----|--------------------|----------------|---------|-------|-----------|-------------------|
| Intercept| 1  | -6.86              | 8.43           | -0.81   | 0.4329|          | 0                 |
| VCA      | 1  | 1.38               | 0.15           | 9.04    | <.0001| 0.99     | 1.00              |
| DAC      | 1  | 0.62               | 0.18           | 3.45    | 0.0055| 0.99     | 1.00              |

![Figure 5.24 Accuracy of the Locking Point Estimation Model](image)

Figure 5.24 Accuracy of the Locking Point Estimation Model
Select Mix Type (NMS)/aggregate Structure (Coarse, Medium, Fine)

Design aggregate structure using the Bailey Method based on the mix type selected

Adjust volume of CA and FA in the aggregate blend (see note 1)

Does structure meet Bailey criteria?

Yes

Estimate the Locking Point using equation 5.1 or suggested ranges (see note 2)- Design the mix for 4.0% AV

Evaluate mix stability using HWT test

Is Rut Depth <6.0mm

Yes

Evaluate Mix Durability (DCSE Threshold)

Is DCSE at 10°C >0.75

No

Reduce Dust/Pbeff: higher AC- Lower % Passing #200- Higher CUW

No

Accept Design

Figure 5.25 Recommended Design Methodology
Note 1: If changing volume of coarse and fine aggregates in the mix does not improve the gradation, aggregate stockpiles from different sources that might have surface characteristics that allow the designer to meet the Bailey criteria.

Note 2: The ranges are those obtained from this research for the different types and gradation of aggregates. If the specimen height during compaction is monitored by the operator, the locking point can be identified instantaneously as the specimen is being densified and the compaction process can be then terminated. This will eliminate the need to estimate the locking point although it is highly recommended to get a rough estimate before the start of the compaction process.

Note 3: Please note that changing the Dust/Pbeff ratio by changing the amount of fine passing the #200 sieve or by using a different CUW, requires re-evaluating the blend using the Bailey ratios.

Note 4: List of Abbreviations used in the design flow chart:

- CUW= Chosen Unit Weight
- DCSE= Dynamic Creep Strain Energy
- Pbeff= Effective asphalt Content
CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

This report documents the findings of an extensive research study on design and characterization of asphalt mixtures for use as road pavement material. Several aspects of asphalt mixtures were addressed using the state of the art laboratory test equipments and technical literature from different information sources. The following is a summary of some of the major findings from this research:

- A simplified design approach was recommended in which asphalt mixtures are designed based on:
  - Analytical aggregate gradation method
  - Variable compaction energy levels
  - Fundamental performance tests that describe the behavior of asphalt mixtures based on sound engineering principles.
- The Bailey method provides a rational approach of aggregate blending and evaluation.
- Adhering to the currently recommended Bailey ratios produced results in terms of volumetrics that are more in line with the generally accepted levels for coarse graded mixtures. Fine and medium mixtures however, had lower voids in mineral aggregates (VMA) than the current Superpave recommendations.
- Data from the SGC provides valuable information on the compactability of asphalt mixtures
- Both SGC and PDA results suggest that coarse mixtures are more difficult to compact compared to the medium and fine ones.
The compaction data also suggest that the current recommended Superpave design number of gyrations is too high and subject the mixtures to unnecessary high compaction loads for extended period of time which might have an adverse effect on the final mixture volumetrics.

There was a strong correlation between the data from the SGC and PDA. This suggests the data from the SGC provides good indication of mixture compactability.

CA ratio, a gradation parameter from the Bailey method which is predominantly a function of the coarse aggregate blend by volume, seems to have the strongest correlations with mixtures volumetrics. The strongest correlation was with the voids in mineral aggregates (VMA) ($R^2 = 0.81$). Mixture volumetrics seem to be less sensitive to the change in the other gradation parameter $F_{AC}$ ratio.

The Compaction Densification Index (CDI) does respond to the change in the gradation parameters, indicating that those parameters do describe the actual gradation characteristics of the mixtures and that the compactability of the mixtures is a function (among other factors) of the particle size distribution as measured by those parameters. CA ratio from the Bailey method and $n_{CA}$ and $n_{FA}$ from the power law gradation analysis method had the best correlation with CDI.

Traffic Indices from Superpave Gyratory Compactor (SGC) and the Pressure Distribution Analyzer (PDA) failed to capture plastic instability of asphalt mixtures as measured by the Hamburg Wheel Tracking Test (HWT).

All the mixtures designed using the Bailey method had highly dense aggregate structures that exhibited superior performance in the HWT test with a maximum
rut depth of 4.0 mm after 20,000 passes. No signs of stripping were found at the end of the test period for all the mixtures.

- Designing mixtures to their locking points resulted in improved durability without compromising stability
- The use of strict volumetric requirements is cautioned. Such requirements are likely to eliminates potential well-performing mixtures

### 6.2 Recommendations

The area of asphalt mixture design is a versatile research platform that is evolving as traffic levels and vehicle design is constantly changing. This research provides a foundation for more elaborate work on developing mixture design methodologies that can reliably produce asphalt mixtures with performance characteristics that matches the demand of the transportation industry. It is strongly recommended that the findings of this research are evaluated using large scale testing facilities such as the Louisiana Accelerated Loading Facility (ALF). This will provide the opportunity to monitor the performance of the designed asphalt mixtures over time as a part of a full pavement structure with different structural properties and thickness design.

It is also recommended that wider range of mixture types and gradations be designed and evaluated using the recommended design approach in order to develop well established ranges of performance criteria adopted for this design methodology.
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APPENDIX A. THE BAILEY METHOD

The Bailey Method is a systematic approach to blending aggregates that provides aggregate interlock as the backbone of the structure and a balanced continuous gradation to complete the mixture. The method provides a set of tools that allows the evaluation of aggregate blends. These tools provide a better understanding in the relationship between aggregate gradation and mixture voids. The Bailey Method gives the practitioner tools to develop and adjust aggregate blends. The new procedures help to ensure aggregate interlock (if desired) and good aggregate packing, giving resistance to permanent deformation, while maintaining volumetric properties that provide resistance to environmental distress. In the Bailey Method aggregate interlock is selected as a design input. Aggregate interlock provides a rut-resistant mixture. To ensure that the mixture contains adequate asphalt binder, VMA is changed by changing the packing of the coarse and fine aggregates. In this way asphalt mixtures developed with the Bailey Method can have a strong skeleton for high stability and adequate VMA for good durability.

A.1 Basic Principles

To develop a method for combining aggregates to optimize 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 following section provide a background for understanding the combination of aggregates. The Bailey Method builds on that understanding and provides more insight into the combination of aggregates for use in an asphalt mixture.
The Bailey Method uses two principles that are the basis of the relationship between aggregate gradation and mixture volumetrics: Aggregate packing, and Definition of coarse and fine aggregate. With these principles, the primary steps in the Bailey Method are: Combine aggregates by volume, and Analyze the combined blend.

A.2 Aggregate Packing

Aggregate particles cannot be packed together to fill a volume completely. There will always be space between the aggregate particles. The degree of packing depends on:

- Type and amount of compactive energy. Several types of compactive force can be used, including static pressure, impact (e.g., Marshall hammer), or shearing (e.g., gyratory shear compactor or California kneading compactor). Higher density can be achieved by increasing the compactive effort (i.e., higher static pressure, more blows of the hammer, or more tamps or gyrations).
- Shape of the particles. Flat and elongated particles tend to resist packing in a dense configuration. Cubical particles tend to arrange in dense configurations.
- Surface texture of the particles. Particles with smooth textures will re-orient more easily into denser configurations. Particles with rough surfaces will resist sliding against one another.
- Size distribution (gradation) of the particles. Single-sized particles will not pack as densely as a mixture of particle sizes.
- Strength of the particles. Strength of the aggregate particles directly affects the amount of degradation that occurs in a compactor or under rollers. Softer
aggregates typically degrade more than strong aggregates and allow denser aggregate packing to be achieved.

The properties listed above can be used to characterize both coarse and fine aggregates. The individual characteristics of a given aggregate, along with the amount used in the blend, have a direct impact on the resulting mix properties. When comparing different sources of comparably sized aggregates, the designer should consider these individual characteristics in addition to the Bailey Method principles presented. Even though an aggregate may have acceptable characteristics, it may not combine well with the other proposed aggregates for use in the design. The final combination of coarse and fine aggregates, and their corresponding individual properties, determines the packing characteristics of the overall blend for a given type and amount of compaction. Therefore, aggregate source selection is an important part of the asphalt mix design process.

A.3 Coarse and Fine Aggregate

The traditional definition of coarse aggregate is any particle that is retained by the 4.75-mm sieve. Fine aggregate is defined as any aggregate that passes the 4.75-mm sieve (sand, silt, and clay size material). The same sieve is used for 9.5-mm mixtures as 25.0-mm mixtures. In the Bailey Method, the definition of coarse and fine is more specific in order to determine the packing and aggregate interlock provided by the combination of aggregates in various sized mixtures. The Bailey Method definitions are:

- Coarse Aggregate: Large aggregate particles that when placed in a unit volume create voids.
Fine Aggregate: Aggregate particles that can fill the voids created by the coarse aggregate in the mixture.

From these definitions, more than a single aggregate size is needed to define coarse or fine. The definition of coarse and fine depends on the nominal maximum particle size (NMPS) of the mixture. In a dense-graded blend of aggregate with a NMPS of 37.5 mm, the 37.5-mm particles come together to make voids. Those voids are large enough to be filled with 9.5-mm aggregate particles, making the 9.5-mm particles fine aggregate. Now consider a typical surface mix with a NMPS of 9.5 mm. In this blend of aggregates, the 9.5-mm particles are considered coarse aggregate. In the Bailey Method, the sieve which defines coarse and fine aggregate is known as the primary control sieve (PCS), and the PCS is based on the NMPS of the aggregate blend. The break between coarse and fine aggregate is shown in Figure A.1. The PCS is defined as the closest sized sieve to the result of the PCS formula in Equation 1.

\[
\text{PCS} = \text{NMPS} \times 0.22
\]

where PCS = Primary Control Sieve for the overall blend
NMPS = Nominal Maximum Particle Size for the overall blend, which is one sieve larger than the first sieve that retains more than 10% (as defined by Superpave terminology).

The value of 0.22 used in the control sieve equation was determined from a two-(2D) and three-dimensional (3-D) analysis of the packing of different shaped particles. The 2-D analysis of the combination of particles shows that the particle diameter ratio ranges from 0.155 (all round) to 0.289 (all flat) with an average value of 0.22. The 3-D analysis of the combination of particles gives a similar result with the particle diameter...
ratio ranging from 0.15 (hexagonal close-packed spheres) to 0.42 (cubical packing of spheres). In addition, research on particle packing distinctly shows that the packing of particles follows different models when the characteristic diameter is above or below 0.22 ratio.

While 0.22 may not be exactly correct for every asphalt mixture, the analysis of gradation is not affected if the value ranges from 0.18 to 0.28. The 0.22 factor is the average condition of many different packing configurations.

**A.4 Combining Aggregates by Volume**

All aggregate blends contain an amount and size of voids, which are a function of the packing characteristics of the blend. In combining aggregates we must first determine the amount and size of the voids created by the coarse aggregates and fill those voids with the appropriate amount of fine aggregate. Mix design methods generally are based on volumetric analysis, but for simplicity, aggregates are combined on a weight basis. Most mix design methods correct the percent passing by weight to percent passing by

![Figure A.1 The Break Between Coarse and Fine Aggregate for 19.0 mm Mixture](image-url)
volume when significant differences exist among the aggregate stockpiles. To evaluate the degree of aggregate interlock in a mixture the designer needs to evaluate a mixture based on volume. To evaluate the volumetric combination of aggregates, additional information must be gathered. For each of the coarse aggregate stockpiles, the loose and rodded unit weights must be determined, and for each fine aggregate stockpile, the rodded unit weight must be determined. These measurements provide the volumetric data at the specific void structure required to evaluate interlock properties.

A.5 Loose Unit Weight of Coarse Aggregate

The loose unit weight of an aggregate is the amount of aggregate that fills a unit volume without any compactive effort applied. This condition represents the beginning of coarse aggregate interlock (i.e., particle-to-particle contact) without any compactive effort applied. The loose unit weight is depicted in Figure A.2. The loose unit weight is determined on each coarse aggregate using the shoveling procedure outlined in AASHTO T-19: “Unit Weight and Voids in Aggregate”, which leaves the aggregate in a loose
condition in the metal unit weight bucket. The loose unit weight (density in kg/m$^3$) is calculated by dividing the weight of aggregate by the volume of the metal bucket. Using the aggregate bulk specific gravity and the loose unit weight, the volume of voids for this condition is also determined. This condition represents the volume of voids present when the particles are just into contact without any outside compactive effort being applied.

A.6 Rodded Unit Weight of Coarse Aggregate

The rodded unit weight of aggregate is the amount of aggregate that fills a unit volume with compactive effort applied. The compactive effort increases the particle to particle contact and decreases the volume of voids in the aggregate. Rodded unit weight is depicted in Figure A.3. The rodded unit weight is determined on each coarse aggregate using the rodding procedure outlined in AASHTO T-19: “Unit Weight and Voids in Aggregate”, which leaves the aggregate in a compacted condition in the metal unit weight bucket. The rodded unit weight (density in kg/m$^3$) is calculated by dividing the weight of aggregate by the volume of the metal bucket. Using the aggregate bulk specific gravity and the rodded unit weight, the volume of voids for this condition is also determined. This condition represents the volume of voids present when the particles are further into contact due to the compactive effort applied.

A.7 Chosen Unit Weight of Coarse Aggregate

The designer needs to select the interlock of coarse aggregate desired in their mix design. Therefore, they choose a unit weight of coarse aggregate, which establishes the volume of coarse aggregate in the aggregate blend and the degree of aggregate interlock.
In the Bailey Method, coarse-graded is defined as mixtures which have a coarse aggregate skeleton. Fine-graded mixtures do not have enough coarse aggregate particles (larger than the PCS) to form a skeleton, and therefore the load is carried predominantly by the fine aggregate. To select a chosen unit weight the designer needs to decide if the mixture is to be coarse-graded or fine-graded. Considerations for selecting a chosen unit weight are shown in Figure A.4.

Figure A.3 Rodded Unit Weight Condition

The loose unit weight is the lower limit of coarse aggregate interlock. Theoretically, it is the dividing line between fine-graded and coarse-graded mixtures. If the mix designer chooses a unit weight of coarse aggregate less than the loose unit weight, the coarse aggregate particles are spread apart and are not in a uniform particle-to-particle contact condition. Therefore, a fine aggregate skeleton is developed and properties for these blends are primarily related to the fine aggregate characteristics.

The rodded unit weight is generally considered to be the upper limit of coarse aggregate interlock for dense-graded mixtures. This value is typically near 110% of the loose unit weight. As the chosen unit weight approaches the rodded unit weight, the
amount of compactive effort required for densification increases significantly, which can make a mixture difficult to construct in the field.

For dense-graded mixtures, the chosen unit weight is selected as a percentage of the loose unit weight of coarse aggregate. If it is required to obtain some degree of coarse aggregate interlock (as with coarse-graded mixtures), the percentage used should range from 95% to 105% of the loose unit weight. For soft aggregates prone to degradation the chosen unit weight should be nearer to 105% of the loose unit weight. Values exceeding 105% of the loose unit weight should be avoided due to the increased probability of aggregate degradation and increased difficulty with field compaction.

![Figure A.4 Selection of Chosen Unit Weight of Coarse Aggregate](image)

With fine-graded mixtures, the chosen unit weight should be less than 90% of the loose unit weight, to ensure the predominant skeleton is controlled by the fine aggregate structure. For all dense-graded mixtures, it is recommended the designer should not use a chosen unit weight in the range of 90% to 95% of the loose unit weight. Mixtures designed in this range have a high probability of varying in and out of coarse aggregate interlock in the field with the tolerances generally allowed on the PCS. It is normal for an aggregate blend to consolidate more than the selected chosen unit weight due to the lubricating effect of asphalt binder. Also, each coarse aggregate typically contains some
amount of fine material when the unit weights are determined, which causes both unit weights (loose and rodded) to be slightly heavier than they would have been, had this material been removed by sieving prior to the test. Therefore, a chosen unit weight as low as 95% of the loose unit weight can often be used and still result in some degree of coarse aggregate interlock. In summary, the amount of additional consolidation, if any, beyond the selected chosen unit weight depends on several factors: Aggregate strength, shape, and texture; the amount of fine aggregate that exists in each coarse aggregate when the loose and rodded unit weight tests are performed; combined blend characteristics; relation of the selected chosen unit weight to the rodded unit weight of coarse aggregate; Type of compactive effort applied (Marshall, Gyratory, etc.); and Amount of compactive effort applied (75 versus 125 gyrations, 50 versus 75 blows, etc.). After selecting the desired chosen unit weight of the coarse aggregate, the amount of fine aggregate required to fill the corresponding VCA is determined.

**A.8 Rodded Unit Weight of Fine Aggregate**

For dense-graded mixtures, the voids created by the coarse aggregate at the chosen unit weight are filled with an equal volume of fine aggregate at the rodded unit weight condition. The rodded unit weight is used to ensure the fine aggregate structure is at or near its maximum strength. A schematic of the rodded unit weight of fine aggregate is shown in Figure A.5.

Rodded unit weight is determined on each fine aggregate stockpile as outlined in the rodding procedure in AASHTO T-19: “Unit Weight and Voids in Aggregate”, which leaves the aggregate in a compacted condition in the unit weight container. The rodded unit weight (density in kg/m$^3$) is calculated by dividing the weight of the aggregate by the volume of the mold. In a dense-graded mixture, the rodded unit weight is always used to
determine the appropriate amount of fine aggregate needed to fill the voids in the coarse aggregate at the chosen unit weight condition. A chosen unit weight is not selected. Note that the rodded unit weight is not determined for dust sized material, such as mineral filler (MF) or bag house fines.

A.9 Determining a Design Blend

The only additional information required other than that typically used in a dense-graded mix design is the corresponding unit weight for each coarse and fine aggregate [excluding MF, bag house fines, and recycled asphalt pavement (RAP)]. The following decisions are made by the designer and used to determine the individual aggregate percentages by weight and the resulting combined blend:

- Bulk specific gravity of each aggregate,
- Chosen unit weight of the coarse aggregates,
- Rodded unit weight of the fine aggregates,
- Blend by volume of the coarse aggregates totaling 100.0%,
- Blend by volume of fine aggregates totaling 100.0%, and
- Amount of −0.075-mm material desired in the combined blend, if MF or bag house fines are being used.

An example design is presented below, which provides the step-by-step calculations required to blend a set of aggregates by volume and determine the resulting combined blend by weight. The following steps are presented to provide a general sense of blending aggregates by volume.

- Pick a chosen unit weight for the coarse aggregates, kg/m³.
- Calculate the volume of voids in the coarse aggregates at the chosen unit weight.
o Determine the amount of fine aggregate to fill this volume using the fine aggregates rodded unit weight, kg/m³.

o Using the weight (density) in kg/m³ of each aggregate, determine the total weight and convert to individual aggregate blend percentages.

o Correct the coarse aggregates for the amount of fine aggregate they contain and the fine aggregates for the amount of coarse aggregate they contain, in order to maintain the desired blend by volume of coarse and fine aggregate.

o Determine the adjusted blend percentages of each aggregate by weight.

o If MF or bag house fines are to be used, adjust the fine aggregate percentages by the desired amount of fines to maintain the desired blend by volume of coarse and fine aggregate.

Determine the revised individual aggregate percentages by weight for use in calculating the combined blend.

Figure A.5 Rodded Unit Weight of Fine Aggregate

A.10 Analysis of the Design Blend

After the combined gradation by weight is determined, the aggregate packing is analyzed further. The combined blend is broken down into three distinct portions, and each portion is evaluated individually. The coarse portion of the combined blend is from
the largest particle to the PCS. These particles are considered the coarse aggregates of the blend. The fine aggregate is broken down and evaluated as two portions. To determine where to split the fine aggregate, the same 0.22 factor used on the entire gradation is applied to the PCS to determine a secondary control sieve (SCS). The SCS then becomes the break between coarse particles and fine particles. The fine particles are further evaluated by determining the tertiary control sieve (TCS), which is determined by multiplying the SCS by the 0.22 factor. A schematic of how the gradation is divided into the three portions is given in Figure A.6. The analysis is done using ratios that evaluate packing within each of the three portions of the combined aggregate gradation. Three ratios are defined: Coarse Aggregate Ratio (CA ratio), Fine Aggregate Coarse Ratio (F_{AC} ratio), and Fine Aggregate Fine Ratio (F_{AF} ratio). These ratios characterize packing of the aggregates. By changing gradation within each portion, modifications can be made to the volumetric properties, construction characteristics, or performance characteristics of the asphalt mixture.

**A.11 CA Ratio**

The CA Ratio is used to evaluate packing of the coarse portion of the aggregate gradation and to analyze the resulting void structure. Understanding the packing of coarse aggregate requires the introduction of the half sieve. The half sieve is defined as one half the NMPS. Particles smaller than the half sieve are called “interceptors.” Interceptors are too large to fit in the voids created by the larger coarse aggregate particles and hence spread them apart. The balance of these particles can be used to adjust the mixture’s volumetric properties. By changing the quantity of interceptors it is possible to change the VMA in the mixture to produce a balanced coarse aggregate structure. With a balanced aggregate structure the mixture should be easy to compact in the field and
should adequately perform under load. The equation for the calculation of the coarse aggregate ratio is:

\[ \text{CA Ratio} = \frac{(\%\text{Passing half sieve}-\%\text{Passing PCS})}{(100-\%\text{Passing half sieve})} \]

The packing of the coarse aggregate fraction, observed through the CA ratio, is a primary factor in the constructability of the mixture. As the CA ratio decreases (below ~1.0), compaction of the fine aggregate fraction increases because there are fewer interceptors to limit compaction of the larger coarse aggregate particles. Therefore, a mixture with a low CA ratio typically requires a stronger fine aggregate structure to meet the required volumetric properties. Also, a CA ratio below the corresponding range suggested in Table 1 could indicate a blend that may be prone to segregation. It is generally accepted that gap-graded mixes, which tend to have CA ratios below these suggested ranges, have a greater tendency to segregate than mixes that contain a more continuous gradation.

As the CA Ratio increases towards 1.0, VMA will increase. However, as this value approaches 1.0, the coarse aggregate fraction becomes “unbalanced” because the 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 causes the portion above the PCS to be less continuous. The resulting mixture can be difficult to compact in the field and have a tendency to move under the rollers because it does not want to “lock up.” Generally, mixes with high CA Ratios have a S-shaped gradation curve in this area of the 0.45-power grading chart. Superpave mixtures of this type have developed a reputation for being difficult to compact.

Table 1 Recommended Ranges of Aggregate Ratios

<table>
<thead>
<tr>
<th>NMPS, mm</th>
<th>37.5</th>
<th>25.0</th>
<th>19.0</th>
<th>12.5</th>
<th>9.5</th>
<th>4.75</th>
</tr>
</thead>
<tbody>
<tr>
<td>CA Ratio</td>
<td>0.80–0.95</td>
<td>0.70–0.85</td>
<td>0.60–0.75</td>
<td>0.50–0.65</td>
<td>0.40–0.55</td>
<td>0.30–0.45</td>
</tr>
<tr>
<td>FA&lt;sub&gt;c&lt;/sub&gt; Ratio</td>
<td>0.35–0.50</td>
<td>0.35–0.50</td>
<td>0.35–0.50</td>
<td>0.35–0.50</td>
<td>0.35–0.50</td>
<td>0.35–0.50</td>
</tr>
<tr>
<td>FA&lt;sub&gt;v&lt;/sub&gt; Ratio</td>
<td>0.35–0.50</td>
<td>0.35–0.50</td>
<td>0.35–0.50</td>
<td>0.35–0.50</td>
<td>0.35–0.50</td>
<td>0.35–0.50</td>
</tr>
</tbody>
</table>

As the CA Ratio exceeds a value of 1.0, the interceptor-sized particles begin to dominate the formation of the coarse aggregate skeleton. The coarse portion of the coarse aggregate is then considered “pluggers”, as these aggregates do not control the aggregate skeleton, but rather float in a matrix of finer coarse aggregate particles.

A.12 Coarse Portion of Fine Aggregate

All of the fine aggregate (i.e., below the PCS) can be viewed as a blend by itself that contains a coarse and a fine portion and can be evaluated in a manner similar to the overall blend. 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 these voids with the appropriate volume of the fine portion of the fine aggregate without overfilling the voids. The equation that describes the fine aggregate coarse ratio (FA<sub>c</sub>) is given in as follows:
\[
F_{AC} = \frac{\% \text{Passing SCS}}{\% \text{Passing PCS}}
\]

where, SCS is the Secondary Control Sieve

As this ratio increases, the fine aggregate (i.e., below the PCS) packs together tighter. This increase in packing is due to the increase in volume of the fine portion of fine aggregate. It is generally desirable to have this ratio less than 0.50, as higher values generally indicate an excessive amount of the fine portion of the fine aggregate is included in the mixture. A \(F_{AC}\) ratio higher than 0.50, which is created by an excessive amount of natural sand and/or an excessively fine natural sand should be avoided. This type of a blend normally shows a “hump” in the sand portion of the gradation curve of a 0.45 gradation chart, which is generally accepted as an indication of a potentially tender mixture.

If the \(F_{AC}\) ratio becomes lower than 0.35, the gradation is not uniform. These mixtures are generally gap-graded and have a “belly” in the 0.45-power grading chart, which can indicate instability and may lead to compaction problems.

**A.13 Fine Portion of Fine Aggregate**

The fine portion of the fine aggregate fills the voids created by the coarse portion of the fine aggregate. This ratio shows how the fine portion of the fine aggregate packs together. One more sieve is needed to calculate the \(F_{AF}\), the Tertiary Control Sieve (TCS). The TCS is defined as the closest sieve to 0.22 times the SCS. The equation for the \(F_{AF}\) ratio is as follows:

\[
F_{AF} = \frac{\% \text{Passing TCS}}{\% \text{Passing SCS}}
\]

The \(F_{AF}\) ratio is used to evaluate the packing characteristics of the smallest portion of the aggregate blend. Similar to the \(F_{AC}\) ratio, the value of the \(F_{AF}\) ratio should be less
than 0.50 for typical dense-graded mixtures. VMA in the mixture will increase with a decrease in this ratio.

A.14 Summary of Ratios

- **CA ratio**: This ratio describes how the coarse aggregate particles pack together and, consequently, how these particles compact the fine aggregate portion of the aggregate blend that fills the voids created by the coarse aggregate.

- **FAC ratio**: This ratio describes how the coarse portion of the fine aggregate packs together and, consequently, how these particles compact the material that fills the voids it creates.

- **FAF ratio**: This ratio describes how the fine portion of the fine aggregate packs together. It also influences the voids that will remain in the overall fine aggregate portion of the blend because it represents the particles that fill the smallest voids created.

These ratios are valuable for evaluating and adjusting VMA. Once an initial trial gradation is evaluated in the laboratory, other gradations can be evaluated on paper to choose a second trial that will have an increased or decreased VMA as desired. When doing the paper analysis, the designer must remember that changes in particle shape, strength and texture must be considered as well. The ratios are calculated from the control sieves of an asphalt mixture, which are tied to the NMAS.

A.15 Example Bailey Method Design Calculations

The calculations in Figure A.7 provide an example of a design using two coarse aggregates, one fine aggregate, and MF. This design uses aggregates of different specific gravity to show how aggregates are blended together by volume. The designer will need to collect information including:
• Stockpile gradation, and bulk specific gravity, and Loose and rodded unit weights (AASHTO T-19). In addition the designer will make several decisions that will determine the stockpile splits. These items include:
  o Chosen unit weight as a percentage of the loose unit weight;
  o Desired percent passing 0.075-mm sieve;
  o Blend by volume of coarse aggregates; and
  o Blend by volume of fine aggregates.

A.15.1 Step 1

Determine the chosen unit of weight for each aggregate according to the loose unit weight for each coarse aggregate and the overall coarse aggregate chosen unit weight for the mixture. The chosen unit weight for the fine aggregates is simply the rodded weight of that aggregate.

A.15.1.1 Calculation

Multiply the loose unit weight percent for each coarse aggregate by the coarse aggregate chosen unit weight for the mixture.

A.15.1.2 Equation

Coarse aggregate chosen unit weight = loose unit weight * desired percent of loose unit weight

CA # 1:  Chosen unit weight = 1425 kg/m$^3$ * 103% = 1469 kg/m$^3$  
CA # 2:  Chosen unit weight = 1400 kg/m$^3$ * 103% = 1441 kg/m$^3$

A.15.2 Step 2
Determine the unit weight contributed by each coarse aggregate according to the desired proportions (by volume) of coarse aggregate.

A.15.2.1 Calculation

Multiply the blend percent of coarse aggregate by the chosen unit weight of each aggregate.

<table>
<thead>
<tr>
<th>Material Grade</th>
<th>Coarse Aggregate Number</th>
<th>Fine Aggregate Number</th>
<th>Mineral Filler</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CA-1</td>
<td>FA-1</td>
<td>MF</td>
</tr>
<tr>
<td></td>
<td>CA-2</td>
<td>FA-2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>CA-3</td>
<td>FA-3</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Design Value</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>103</td>
<td>95 – 105</td>
</tr>
</tbody>
</table>

**CA Chosen Weight as % of Loose Weight**

| Desired % Pass 0.075 mm | 4.5          | 3.5 – 6.0    |

**Coarse Aggregate Blend by Volume**

| 25.0 | 75.0 | 100.0 |

Above blending % must sum to 100

**Fine Aggregate Blend by Volume**

| 100.0 | 100.0 |

Above blending % must sum to 100

**Combined Bulk Specific Gravity of All Aggregates**

| 2.889 |

**Total Volume of Coarse Aggregate**

| 52.7 |

**Total Volume of Fine Aggregate**

| 46.3 |

**Aggregate Properties**

| 19.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 |
| 12.5 | 94.0  | 100.0 | 100.0 | 100.0 | 100.0 |
| 9.5  | 38.0  | 90.0  | 100.0 | 100.0 | 100.0 |
| 4.75 | 3.0   | 30.0  | 100.0 | 100.0 | 100.0 |
| 2.30 | 1.9   | 5.0   | 79.9  | 100.0 | 100.0 |
| 1.18 | 1.5   | 2.5   | 48.8  | 100.0 | 100.0 |
| 0.60 | 1.3   | 1.9   | 29.0  | 100.0 | 100.0 |
| 0.30 | 1.6   | 1.4   | 14.2  | 100.0 | 100.0 |
| 0.10 | 1.8   | 1.3   | 8.6   | 98.0  | 98.0  |
| 0.075| 1.7   | 1.2   | 3.0   | 90.0  | 90.0  |

**Bulk Spec. Gr.**

| 2.702 | 2.698 | 2.702 | 2.698 |

**Apparent Gr.**

| 2.812 | 2.812 |

**% Absorp.**

| 1.452 | 1.502 |

**Loose Weight**

| 1428  | 1400  |

**Rooded Weight**

| 1608  | 1592  |

| 2167  | 2167  |
Contribution = percent coarse aggregate * chosen unit weight

CA #1: \[ \text{Contribution} = 25\% \times 1469 \text{ kg/m}^3 = 367 \text{ kg/m}^3 \] (2a)

CA #2: \[ \text{Contribution} = 75\% \times 1441 \text{ kg/m}^3 = 1081 \text{ kg/m}^3 \] (2b)

A.15.3 Step 3
Determine the voids in each coarse aggregate according to its corresponding chosen unit weight and contribution by volume. Then sum the voids contributed by each coarse aggregate.

A.15.3.1 Calculation
First calculate one minus the chosen unit weight divided by the bulk specific gravity and density of water. Multiply the result by the percent of coarse aggregate blend. Then, sum the contribution of each coarse aggregate.

A.15.3.2 Equation

\[
\text{Voids in coarse aggregate} = \left( 1 - \frac{\text{chosen unit weight}}{G_{50} \times 1000} \right) \times \text{Blend}\% \\
\]

Where \( G_{50} \) = bulk specific gravity.

CA #1: \[ \text{Voids in CA #1} = \left( 1 - \frac{1468}{2.702 \times 1000} \right) \times 25.0 = 11.4 \] (3a)

CA #2: \[ \text{Voids in CA #2} = \left( 1 - \frac{1441}{2.698 \times 1000} \right) \times 75.0 = 34.9 \] (3b)

Total: \[ \text{Voids in CA #1 + Voids in CA #2} = 11.4 + 34.9 = 46.3 \] (3c)

A.15.4 Step 4
Determine the unit weight contributed by each fine aggregate according to the desired volume blend of fine aggregate. This is the unit weight that fills the voids in the coarse aggregate.

**A.15.4.1 Calculation**

Multiply the fine aggregate chosen unit weight by the volume percentage of this aggregate in the fine aggregate blend and multiply this by the total percentage of coarse aggregate voids from (3c).

**A.15.4.2 Equation**

Contribution of each fine aggregate = fine aggregate chosen unit weight $\times$ % fine aggregate blend $\times$ % voids in coarse aggregate.

\[
\text{FA #1: Contribution} = 2167 \text{ kg/m}^3 \times 100\% \times 46.3\% = 1002 \text{ kg/m}^3
\]

Note: If there is more than one fine aggregate the calculation is repeated for each fine aggregate.

**A.15.5 Step 5**

Determine the unit weight for the total aggregate blend.

**A.15.5.1 Calculation:**

Sum the unit weight of each aggregate.

**A.15.5.2 Equation**

\[
\text{Unit weight of blend} = (2a) + (2b) + (4)
\]

\[
\text{Unit weight of blend} = 367 \text{ kg/m}^3 + 1081 \text{ kg/m}^3 + 1002 \text{ kg/m}^3 = 2450 \text{ kg/m}^3
\]
Determine the initial blend percentage by weight of each aggregate.

**A.15.6.1 Calculation**

Divide the unit weight of each aggregate by the unit weight of the total aggregate blend.

**A.15.6.2 Equation**

Percent by weight = unit weight of aggregate/unit weight of blend

\[
\begin{align*}
\text{CA #1:} & \quad \% \text{ by weight} = \frac{367 \text{ kg/m}^3}{2450 \text{ kg/m}^3} = 0.150 = 15.0 \% \\
\text{CA #2:} & \quad \% \text{ by weight} = \frac{1081 \text{ kg/m}^3}{2450 \text{ kg/m}^3} = 0.441 = 44.1 \% \\
\text{FA #1:} & \quad \% \text{ by weight} = \frac{1002 \text{ kg/m}^3}{2450 \text{ kg/m}^3} = 0.409 = 40.9 \%
\end{align*}
\]

(6a) \quad (6b) \quad (6c)

These initial estimates of stockpile splits are based on the choice of how much coarse aggregate to have in the mixture. The initial estimates of stockpile splits will be adjusted to account for fine aggregate particles in the coarse aggregate stockpiles and coarse aggregate particles in the fine aggregate stockpiles.

**A.15.7 Step 7**

In a 12.5-mm NMPS mixture, the CA/FA break (PCS) is the 2.36-mm sieve.

**A.15.7.1 Calculation**

For the coarse aggregate stockpiles, determine the percent passing the 2.36-mm sieve.

For the fine aggregate stockpiles, determine the percent retained on the 2.36-mm sieve.

\[
\begin{align*}
\text{CA #1:} & \quad \% \text{ fine aggregate} = 1.9\% \\
\text{CA #2:} & \quad \% \text{ fine aggregate} = 5.0\% \\
\text{FA #1:} & \quad \% \text{ coarse aggregate} = 100.0\% - 79.9\% = 20.1\%
\end{align*}
\]

(7a) \quad (7b) \quad (7c)

**A.15.8 Step 8**
Determine the fine aggregate in each coarse stockpile according to its percentage in the blend.

**A.15.8.1 Calculation**

For each coarse aggregate stockpile determine the percent passing the 2.36-mm sieve as a percentage of the total aggregate blend.

**A.15.8.2 Equation**

Percent fine aggregate in blend = Coarse stockpile percent of blend \( \times \) percent fine aggregate in coarse stockpile.

\[
\begin{align*}
\text{CA #1: } & \quad \text{Percent fine aggregate in blend} = 15.0\% \times 1.9\% = 0.3\% \\
\text{CA #2: } & \quad \text{Percent fine aggregate in blend} = 44.1\% \times 5.0\% = 2.2\%
\end{align*}
\]

**A.15.9 Step 9**

Sum the percent of fine aggregate particles in all the coarse aggregate stockpiles.

\[
\text{All CAs: } \quad \text{Percent fine aggregate in blend} = 0.3\% + 2.2\% = 2.5\%
\]

**A.15.10 Step 10**

Determine the coarse aggregate in each fine stockpile according to its percentage in the blend.

**A.15.10.1 Calculation**

For each fine aggregate stockpile determine the percent retained on the 2.36-mm sieve as a percentage of the total aggregate blend.

**A.15.10.2 Equation:** Percent coarse aggregate in blend = Stockpile percent of blend \( \times \) percent coarse aggregate in fine stockpile.
A.15.11 Step 11

Sum the percent of fine aggregate particles in all the coarse aggregate stockpiles.

All FAs: \[ \text{Percent fine aggregate in blend} = 8.2\% \] (11)

A.15.12 Step 12

Correct the initial blend percentage of each coarse aggregate to account for the amount of fine aggregate it contains and coarse aggregate contributed by the fine aggregate stockpiles.

A.15.12.1 Equation

Adjusted stockpile percent in blend = (initial %) + \left( \frac{\text{initial } \% \times \text{Sum FA in CA}}{\text{Total } \% \text{ of CA}} \right)

CA #1:

\[ \text{Adjusted stockpile percent in blend} = (15.0\%) + (0.3\%) - \left( \frac{15.0\% \times 8.2\%}{15.0\% + 44.1\%} \right) = 13.2\% \] (12a)

CA #2:

\[ \text{Adjusted stockpile percent in blend} = (44.1\%) + (2.2\%) - \left( \frac{44.1\% \times 8.2\%}{15.0\% + 44.1\%} \right) = 40.2\% \] (12b)

A.15.13 Step 13

Correct the initial blend percentage of each fine aggregate to account for the amount of coarse aggregate it contains and fine aggregate contributed by the coarse aggregate stockpiles.

A.15.13.1 Equation

Adjusted stockpile percent in blend

\[ = (\text{initial } \% ) + (\text{CA in FA}) - \left( \frac{\text{initial } \% \times \text{Sum FA in CA}}{\text{Total } \% \text{ of FA}} \right) \]
The next steps will determine whether MF will be needed to bring the percent passing the 0.075-mm sieve to the desired level.

**A.15.14 Step 14**
Determine the amount of –0.075-mm material contributed by each aggregate using the adjusted stockpile percentages.

**A.15.14.1 Calculation**
Multiply the percent passing the 0.075-mm sieve for each aggregate by the adjusted blend percentage for each aggregate.

**A.15.14.2 Equation**
Percent contribution of 0.075-mm sieve for each stockpile = adjusted stockpile percent \( \times \) percent passing 0.075-mm sieve for that stockpile.

\[
\begin{align*}
\text{CA #1:} & \quad \text{Percent contribution 0.075 mm} = 13.2\% \times 1.7\% = 0.2\% \\
\text{CA #2:} & \quad \text{Percent contribution 0.075 mm} = 40.2\% \times 1.2\% = 0.5\% \\
\text{FA #1:} & \quad \text{Percent contribution 0.075 mm} = 46.7\% \times 3.0\% = 1.4\%
\end{align*}
\]

**A.15.15 Step 15**
Determine the amount of mineral filler required, if any, to bring the percent passing the 0.075-mm sieve to the desired level. For this mixture the desired amount of –0.075-mm material is 4.5%.
A.15.15.1 Equation

\[
\text{Percent of MF} = \left( \frac{\% 0.075 \text{ mm desired} - \% 0.075 \text{ mm in blend}}{\% 0.075 \text{ mm in filler}} \right)
\]

\[
\text{MF: Percent MF} = \left( \frac{4.5 - 2.1}{90\%} \right) = 2.7\%
\]

(15)

A.15.16 Step 16

Determine the final blend percentages of fine aggregate stockpiles by adding the percent MF to the fine aggregate. In this step the blend percentage of CA is not changed. The blend percentage of FA is adjusted to account for the MF.

A.15.16.1 Equation

\[
\text{Final blend percent for fine aggregate} = \text{Adjusted blend percent} - \left( \frac{\% \text{FA} \times \% \text{MF}}{\text{Total} \% \text{FA}} \right)
\]

FA #1: Final blend percent = 46.7\% - \left( \frac{46.7\% \times 2.7\%}{46.7\%} \right) = 44.0

(16)

A.16 Results

The final blending percentages are taken from the following equation results:

<table>
<thead>
<tr>
<th>Equation</th>
<th>Results(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CA #1</td>
<td>12a</td>
</tr>
<tr>
<td>CA #2</td>
<td>12b</td>
</tr>
<tr>
<td>FA #1</td>
<td>16</td>
</tr>
<tr>
<td>MF</td>
<td>15</td>
</tr>
</tbody>
</table>
### APPENDIX B. INDIVIDUAL AGGREGATE STOCKPILES PROPERTIES

#### Table B 1. Limestone Stockpile Gradations and Physical Properties

<table>
<thead>
<tr>
<th>Stockpile No.</th>
<th>#57 LS</th>
<th>#67 LS</th>
<th>#78 LS</th>
<th>#8 LS</th>
<th>#11 LS</th>
<th>#10 LS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Metric (U.S.)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sieve</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>37.5 mm (1.5 in)</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>25 mm (1 in)</td>
<td>79.1</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>19 mm (¾ in)</td>
<td>41.6</td>
<td>91.3</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>12.5 mm (½ in)</td>
<td>8.0</td>
<td>44.3</td>
<td>93.2</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>9.5 mm (⅜ in)</td>
<td>2.3</td>
<td>23.1</td>
<td>55.3</td>
<td>94.6</td>
<td>100.0</td>
<td>100.0</td>
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<tr>
<td>4.75 mm (No.4)</td>
<td>1.4</td>
<td>4.6</td>
<td>7.0</td>
<td>24.3</td>
<td>92.1</td>
<td>98.8</td>
</tr>
<tr>
<td>2.36 mm (No.8)</td>
<td>1.3</td>
<td>2.5</td>
<td>3.1</td>
<td>6.0</td>
<td>62.8</td>
<td>77.2</td>
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<tr>
<td>1.18 mm (No.16)</td>
<td>1.3</td>
<td>2.1</td>
<td>2.5</td>
<td>4.1</td>
<td>39.7</td>
<td>56.1</td>
</tr>
<tr>
<td>0.6 mm (No.30)</td>
<td>1.2</td>
<td>1.9</td>
<td>2.3</td>
<td>3.5</td>
<td>25.9</td>
<td>41.3</td>
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<tr>
<td>0.3 mm (No.50)</td>
<td>1.1</td>
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<td>2.1</td>
<td>3.2</td>
<td>18.3</td>
<td>30.4</td>
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<tr>
<td>0.15 mm (No.100)</td>
<td>1.0</td>
<td>1.6</td>
<td>2.0</td>
<td>3.0</td>
<td>14.1</td>
<td>22.6</td>
</tr>
<tr>
<td>0.075 mm (No.200)</td>
<td>0.9</td>
<td>1.5</td>
<td>1.8</td>
<td>2.8</td>
<td>11.6</td>
<td>17.0</td>
</tr>
<tr>
<td>Bulk specific gravity</td>
<td>2.673</td>
<td>2.674</td>
<td>2.658</td>
<td>2.654</td>
<td>2.567</td>
<td>2.496</td>
</tr>
<tr>
<td>Apparent specific gravity</td>
<td>2.701</td>
<td>2.703</td>
<td>2.697</td>
<td>2.688</td>
<td>2.706</td>
<td>2.716</td>
</tr>
<tr>
<td>Absorption, %</td>
<td>0.381</td>
<td>0.401</td>
<td>0.538</td>
<td>0.469</td>
<td>2.007</td>
<td>3.329</td>
</tr>
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</table>
Table B1 Cont.

<table>
<thead>
<tr>
<th></th>
<th>CAA</th>
<th>FAA</th>
<th>Flat &amp; Elongated</th>
<th>SE value</th>
<th>Loose Unit</th>
<th>Rodded Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>FAA</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>46.1</td>
<td>45.1</td>
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<tr>
<td>Flat &amp; Elongated</td>
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<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>N/A</td>
<td>N/A</td>
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<tr>
<td>SE value</td>
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<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
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<td>51.6</td>
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<td>Loose Unit</td>
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<td>Rodded Unit</td>
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<tr>
<td>Weight</td>
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<td></td>
<td></td>
<td>114.4</td>
<td>111.8</td>
</tr>
</tbody>
</table>

Table B 2. Sandstone Stockpiles Gradations and Physical Properties

<table>
<thead>
<tr>
<th>Stockpile No.</th>
<th>#57 SST</th>
<th>#67 SST</th>
<th>#78 SST</th>
<th>#8 SST</th>
<th>#11 SST</th>
<th>1/4 by 0 SST</th>
<th>Coarse Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Metric (U.S.)</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sieve</td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>37.5 mm (1.5 in)</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>25 mm (1 in)</td>
<td>98.6</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>19 mm (¾ in)</td>
<td>68.5</td>
<td>95.9</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
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<tr>
<td>12.5 mm (½ in)</td>
<td>29.0</td>
<td>50.3</td>
<td>93.2</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>9.5 mm (⅜ in)</td>
<td>14.0</td>
<td>27.2</td>
<td>67.4</td>
<td>96.3</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>4.75 mm (No.4)</td>
<td>2.0</td>
<td>5.2</td>
<td>16.1</td>
<td>39.0</td>
<td>99.7</td>
<td>87.4</td>
<td>99.0</td>
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<tr>
<td>2.36 mm (No.8)</td>
<td>1.6</td>
<td>3.0</td>
<td>3.8</td>
<td>7.4</td>
<td>86.6</td>
<td>69.3</td>
<td>92.0</td>
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<tr>
<td>1.18 mm (No.16)</td>
<td>1.6</td>
<td>2.8</td>
<td>3.2</td>
<td>4.7</td>
<td>59.5</td>
<td>57.7</td>
<td>81.7</td>
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<td>0.6 mm (No.30)</td>
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<td>4.2</td>
<td>44.0</td>
<td>50.0</td>
<td>63.8</td>
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<tr>
<td>Table B2 Cont.</td>
<td></td>
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</tr>
<tr>
<td>----------------</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>0.3 mm (No.50)</strong></td>
<td>1.6</td>
<td>2.7</td>
<td>2.9</td>
<td>4.0</td>
<td>32.2</td>
<td>42.0</td>
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<tr>
<td><strong>0.15 mm (No.100)</strong></td>
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<td>2.1</td>
<td>2.4</td>
<td>3.1</td>
<td>11.8</td>
<td>23.6</td>
<td>1.6</td>
</tr>
<tr>
<td><strong>0.075 mm (No.200)</strong></td>
<td>1.0</td>
<td>1.5</td>
<td>1.9</td>
<td>2.4</td>
<td>4.2</td>
<td>14.1</td>
<td>0.6</td>
</tr>
<tr>
<td><strong>Bulk specific gravity</strong></td>
<td>2.555</td>
<td>2.513</td>
<td>2.539</td>
<td>2.520</td>
<td>2.551</td>
<td>2.514</td>
<td>2.595</td>
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<tr>
<td><strong>Apparent specific gravity</strong></td>
<td>2.655</td>
<td>2.644</td>
<td>2.655</td>
<td>2.656</td>
<td>2.678</td>
<td>2.682</td>
<td>2.647</td>
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<tr>
<td><strong>Absorption, %</strong></td>
<td>1.466</td>
<td>1.966</td>
<td>1.721</td>
<td>2.027</td>
<td>1.874</td>
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</tr>
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<td><strong>CAA</strong></td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td><strong>FAA</strong></td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>47.8</td>
<td>-</td>
<td>38.0</td>
</tr>
<tr>
<td><strong>Flat &amp; Elongated</strong></td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
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<td><strong>SE value</strong></td>
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<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>38.8</td>
<td>-</td>
<td>100</td>
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<tr>
<td><strong>Loose Unit Weight</strong></td>
<td>82.9</td>
<td>87.6</td>
<td>86.1</td>
<td>83.8</td>
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<td>N/A</td>
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<tr>
<td><strong>Rodded Unit Weight</strong></td>
<td>93.9</td>
<td>97.5</td>
<td>95.9</td>
<td>95.5</td>
<td>103.6</td>
<td>110.6</td>
<td>109.6</td>
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</table>

Table B 3. Granite Stockpiles Gradations and Physical Properties

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<th>#5 Granite</th>
<th>#78 Granite</th>
<th>#11 Granite</th>
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<tbody>
<tr>
<td>Metric (U.S.)</td>
<td>Sieve</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diameter (mm)</td>
<td>37.5 mm (1.5 in)</td>
<td>25 mm (1 in)</td>
<td>19 mm (¾ in)</td>
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<tr>
<td>--------------</td>
<td>------------------</td>
<td>--------------</td>
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<td>100.0</td>
<td>100.0</td>
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<td>92.2</td>
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<td>100.0</td>
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<tr>
<td></td>
<td>63.3</td>
<td>100.0</td>
<td>100.0</td>
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<tr>
<td></td>
<td>22.1</td>
<td>95.2</td>
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<td>10.8</td>
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<td></td>
<td>9.1</td>
<td>95.4</td>
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<td>2.3</td>
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<td>1.6</td>
<td>43.4</td>
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<td>1.5</td>
<td>27.5</td>
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<td>1.1</td>
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<td>1.2</td>
<td>11.0</td>
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<td>0.8</td>
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<td>0.851</td>
<td>1.957</td>
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<td>100%</td>
<td>N/A</td>
</tr>
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<td></td>
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<td>N/A</td>
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</tr>
<tr>
<td></td>
<td>96.3</td>
<td>91.8</td>
<td>N/A</td>
</tr>
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Table B3 Cont.

<table>
<thead>
<tr>
<th>Weight (lb/ft³)</th>
<th>106.8</th>
<th>101.4</th>
<th>109.6</th>
</tr>
</thead>
</table>

Rodded Unit
VITA

Khalid Alshamsi was born on January 28th, 1969, in a small, beautiful mountainous village called Fanja, The Sultanate of Oman. He grew up enjoying the beauty of the lovely surrounding nature and the company of the wonderful people of Fanja. Khalid finished his high school in 1988 and joined the College of Engineering of the Sultan Qaboos University (SQU), The Sultanate of Oman, as a student in the civil engineering department. He received his bachelor’s degree in civil engineering from SQU in June 1993. Immediately after his graduation, he joined the same department he graduated from, as a teaching assistant, and worked there for a year before he decided to continue his higher studies. Khalid spent 12 months in the United Kingdom pursuing his master’s degree in international highway engineering at the University of Birmingham. He successfully finished his master’s degree in August 1995. Khalid always remembers his time in England where he experienced the rich English culture and heritage. He came to the United States in the fall of 2001 to pursue his doctoral degree in civil engineering at Louisiana State University, Baton Rouge, Louisiana. He spent almost 5 years at LSU and carries with him so far wonderful memories. Khalid expects to receive the degree of Doctor of Philosophy in civil engineering in May 2006. Khalid is married and has three beautiful, adorable, cute, and charming kids; a girl and two boys.