EFFECTS OF SUPERPAVE RESTRICTED ZONE ON PERMANENT DEFORMATION

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ABSTRACT

The purpose of this study is to evaluate the restricted zone effect using four different aggregates: crushed granite, crushed limestone, crushed river gravel, and a mixture of crushed river gravel as coarse aggregate with natural fines. As the restricted zone is a component of Superpave, the blends prepared met most of the Superpave criteria, except the restricted zone in selected mixtures and fine aggregate angularity in three mixtures. Each type of aggregate was used for mixture design of three gradations: above, through, and below the restricted zone. The twelve mixtures designed were tested in the laboratory to evaluate their relative resistance to permanent deformation. Four types of tests were performed using Superpave equipment: simple shear at constant height, frequency sweep at constant height, repeated shear at constant stress ratio, and repeated shear at constant height. Rutting resistance of the mixtures was measured using the Asphalt Pavement Analyzer.

Researchers found that there is no relationship between the restricted zone and permanent deformation when crushed aggregates are used in the mixture design. Superpave mixtures with gradations below the restricted zone were generally most susceptible to permanent deformation while mixtures above the restricted zone were least susceptible to permanent deformation. Recommendations include elimination of the restricted zone from HMA design specifications.

DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the International Center of Aggregate Research (ICAR), Texas Transportation Institute (TTI) or Texas A&M University. The report does not constitute a standard, specification, or regulation, nor is it intended for construction, bidding, or permit purposes.

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CHAPTER 1

INTRODUCTION

1.1 GENERAL

The United States road network has 3.9 million miles of roadway, of which 61 percent are paved *(1)*. In order to improve the performance, durability, and safety of United States roads, the Strategic Highway Research Program (SHRP) was established by Congress in 1987 as a five year research program. Fifty million dollars of the one hundred and fifty million dollars of the SHRP research funds were used for the development of asphalt specifications to directly relate laboratory analysis with field performance. SUPERPAVE™ (Superior Performing Asphalt Pavements) is the final product of the SHRP research effort. Superpave is a complete mixture design and analysis system with three major components:

- Asphalt binder specification,
- Mixture design, and
- Analysis system.

The SHRP research effort mainly concentrated on properties and testing of asphalt binder (*2*). The aggregate and asphalt-aggregate characteristics of Superpave mixtures were developed by a group of 14 experts known as the Aggregate Expert Task Group (ETG). These experts, who were selected by SHRP, used a modified Delphi procedure to select the aggregate and mixture characteristics (*3*). The procedure consisted of three parts:

- Formulate questionnaires concerning the aggregate and mixture characteristics to the experts,
- Compile and compare the responses, and
- Select aggregate and mixture characteristics from consensus responses.

The results of the modified Delphi process are summarized in SHRP-A-408 (*3*). It is noteworthy that, even though aggregate constitutes approximately 95% of hot mix asphalt (HMA) by weight, little effort was devoted to the study of the contribution of aggregates to the pavement performance (*2*).

As a result, some aspects of Superpave aggregate specifications are not universally accepted, being questioned by the agencies, mainly departments of transportation (DOTs), or the industries, or by both.

1.2 PROBLEM STATEMENT

One of the products of the modified Delphi procedure was the Superpave aggregate gradation controls. The components of this gradation control include the following items (*4*):

- Federal Highway Administration (FHWA) 0.45 Power Chart: used to define a permissible gradation,
- Control Points: through which the combined mixture gradation must pass,
- Restricted Gradation Zone: an area lying along the maximum density line extending from the No. 50 (0.30-mm) sieve to the No. 8 (2.36-mm) or No. 4 (4.75-mm) sieve, through which it is undesirable for a mixture gradation to pass (*5*).

One of the most controversial components of the Superpave mixture design is the aggregate gradation restricted zone. The purposes of the restricted zone, as mentioned in Report FHWA-SA-95-003 (*6*), are to limit the inclusion of large amounts of natural sand that cause humps in the gradation curve in the 0.6 mm range and to discourage the gradations which fall on the maximum density line which often lacks adequate voids in mineral aggregate (VMA). This restricted zone was adopted primarily to reduce premature rutting. The apparent increase in HMA pavement rutting in recent years is due to higher traffic volumes, increased loads and tire pressures, poor construction quality control, and decreased quality of HMA mixtures (*7*).

In most cases, a humped gradation indicates an over-sanded mixture and/or a mixture that possesses too much fine sand in relation to total sand. This gradation often results in a mixture that poses compaction problems during construction (tenderness) and offers reduced resistance to permanent deformation during its service life. These gradations are very sensitive to asphalt content and can easily become plastic with an increase in asphalt content within the tolerance allowed by most specifications (*4*). In some publications (*4*, *8*), it is indicated that improved mixture performance is generally achieved from gradations that pass below the restricted zone.

Avoiding the restricted zone was merely a recommendation by Superpave, not a mandatory specification. Superpave encourages design of mixtures with gradations below the restricted zone. Nevertheless, some state highway agencies categorically rejected any mixture passing through the restricted zone (*9*).

On the other hand, some highway agencies question the validity of the restricted zone, stating that specified gradations of many successful mixtures pass through the restricted zone. They further state that these high-performance mixtures contain high quality aggregates, which are normally 100 percent manufactured aggregates with no rounded sands. Strict adherence to the restricted zone may have negative effects on the economy of the mixture. Exclusion of some particular sizes form the combined gradation (even though they are manufactured) jeopardize the balanced aggregate skeleton and, hence, can potentially increase the cost of mixture.

1.3 OBJECTIVES OF STUDY

The objective of this research is to evaluate the relationship between the restricted zone and rutting while the shape and angularity of the aggregate remains unchanged. The concept is to compare properties of HMA containing the same aggregate type with three different gradations, passing through, above, and below the restricted zone. In these three gradations, the coarse side (plus No. 4 sieve) of the grading curve was kept the same, while the fine side (minus No. 4 sieve) was varied in order to pass through, above, or below this restricted zone.

The most interesting aggregate selected for this study was the river gravel because, among the aggregates used, it was assumed to be one of the most sensitive since it presents the least desirable particle shape and surface texture. Although it was crushed, it retained some percentage of rounded faces with rather smooth surface textures. In this study, crushed granite, crushed limestone, and rounded sand were also used in HMA mixtures, and mixture evaluation tests were conducted to examine the influence of the restricted zone.

The origin and grade of binder employed in the different HMA mixtures was kept the same to facilitate comparisons of the different performances obtained with the different aggregates. Except for the mixtures through the restricted zone and fine aggregate angularity (where indicated), the mixtures met all the Superpave criteria.

The twelve mixtures were tested in the laboratory to evaluate their relative resistance to permanent deformation. Four types of fundamental tests, using the Superpave Shear Tester (SST), were performed:

- Simple Shear at Constant Height,
- Frequency Sweep at Constant Height,
- Repeated Shear at Constant Stress Ratio, and
- Repeated Shear at Constant Height.

Although the repeated shear test at constant height is not required by Superpave, it was performed because it is a simplified method to predict premature rutting (*6*). In addition to these four tests, the mixtures were subjected to wheel tracking torture test using the Asphalt Pavement Analyzer (APA).

1.4 ORGANIZATION OF REPORT

This report is divided into five chapters. Chapter I serves as an introduction, stating the nature of the problem to be addressed, objectives of the research, and scope of work accomplished.

Chapter II summarizes an overview of permanent deformation in asphalt pavements. It covers the definition of permanent deformation, its different types and causes, and its characterization. Selected studies conducted in the last five years related to the effect of aggregate gradation, focusing specifically on the restricted zone, are described in this chapter.

Chapter III is a description of the experimental program. The work plan includes the following tasks: plan of study, materials selection and acquisition, tests for characterizing asphalt cement and aggregates, Superpave mixture design, and tests for asphalt concrete mixture evaluation.

Chapter IV covers analysis of the results from different tests which have been conducted to predict premature rutting in asphalt pavement: repeated shear at constant stress ratio, frequency sweep at constant height, simple shear at constant height, repeated shear at constant height, and rutting evaluation using the APA.

Chapter V presents conclusions and recommendations that arise from the study.

CHAPTER 2
LITERATURE REVIEW

2.1 GENERAL

Permanent deformation is the predominant type of distress found in flexible pavement that concerns paving agencies. Three types of permanent deformation are described below.

- Structural deformation: a subsidence in the base, subbase, and/or subgrade accompanied by subsidence and, possibly, distress cracking pattern in the pavement.
- Plastic deformation: a depression in the asphalt pavement near the center of the applied load usually with slight humps on either side of the depression.
- Consolidation or densification: a depression in the asphalt pavement near the center of the applied load without the accompanying humps. This is a result of further compaction of the asphalt pavement by traffic after construction.

2.2 WHEELPATH RUTTING

Wheelpath rutting is the most common form of permanent deformation exhibited in flexible pavements (*8*). It typically occurs in the top 75-100 mm (3-4 inches) of asphalt pavements. Wheel path rutting is produced by one or a combination of the three types of rutting defined previously. This research was focused on the plastic deformation in asphalt layers (the leading cause of permanent deformation). If an asphalt mixture ruts, it is normally because the mixture has insufficient shear strength to support the stresses to which it is submitted. Wheelpath rutting is a function of traffic volume and applied loads.

The purpose of a pavement is to provide a safe, smooth riding surface for vehicular travel. Therefore, when rutting interferes with these purposes, it has become excessive. From a safety point of view, the important factor is cross drainage of surface water. Rutting is not normally a significant safety problem in dry weather unless it is sufficient to interfere with vehicle control. However, when water begins to pond in the wheel path, the rutted pavement poses a hazard because hydroplaning or sliding on ice in cold weather (*6*). The cross slope of the pavement section is the controlling factor in determining when a rut depth is acceptable or not. At speeds of 90 km/hr (55 mph) or more, pavements with crown slopes of the order of 2 percent and rut depths of about 1.25 cm (0.5 inch), ponding is sufficient to cause vehicles hydroplaning.

There are several wheelpath rutting classifications, one of which was provided in 1979 by the Federal Highway Administration, which classified rutting into three levels of severity:

- Low, from 6 to 12.5 mm (0.25 to 0.5 inches),
- Medium, from 12.5 to 25 mm (0.5 to 1.0 inches), and
- High, over 25 mm (1 inch).

For normal cross slope values, a rut depth of 12.5 mm (0.5 inch) is typically accepted as the maximum allowable rut depth (*8*, *12*).

2.3 RUTTING CHARACTERIZATION

The components of asphalt concrete are aggregates, asphalt cement, and air voids. Rutting is a complicated process, affected by the properties and proportions in which these components are mixed. These three components interact to produce HMA properties. Asphalt pavement rutting typically occurs during the summer. When higher pavement temperatures are reached, the viscosity of the asphalt binder is low, and the traffic load is primarily carried by the mineral aggregate structure (*8*). The resistance of HMA to rutting is considered the combined resistance (shear strength) of the mineral aggregate and asphalt cement (*4*). The Mohr-Coulomb equation is often used to illustrate how both materials (asphalt cement and aggregates) contribute to the shear strength of the asphalt mixture:

$$\tau = c + \sigma \tan \varphi$$

where:

- τ is the shear strength of the asphalt mixture,
- c is the cohesion term, in our case, the portion of the mixture shear strength provided by the asphalt cement,

- σ is the normal stress to which the asphalt mixture is subjected,
- φ is the angle of internal friction provided by the aggregate structure.

Of course, air void content plays an important role in the shear resistance of an HMA mixture. Since this study concentrates on the Superpave aggregate gradations, the effect of aggregate-related properties on rutting characterization of HMA will be discussed below.

2.3.1 Aggregates

The largest portion of the resistance to permanent deformation of the mixture is provided by the aggregate structure. Aggregate is expected to provide a strong, stone skeleton to resist repeated load applications. Gradation, shape, and surface texture have a great influence on HMA properties. Angular, rough-textured aggregates provide more shear strength than rounded, smooth-textured aggregates. When a load is applied to the aggregate in an asphalt mixture, the angular, cubical, rough-textured aggregate particles lock tightly together and function as a large, single elastic mass, thus increasing the shear strength of the asphalt mixture. Conversely, instead of locking together, smooth, rounded aggregate particles tend to slide past each other.

If the aggregate provides a high degree of internal friction, φ, the shear strength of the asphalt mixture will be increased and, therefore, the resistance to rutting. This is accomplished by selecting an aggregate that is angular, cubical, has a rough surface texture, and is graded in a manner to develop particle to particle contact (*6*).

2.3.2 Aggregate Gradation

R. P. Elliot et al. (*13*) conducted an investigation to evaluate the effect of different aggregate gradations on the properties of asphalt mixtures. The aggregate blends included: coarse, fine, mid-band (job mix formula - JMF), and two poorly graded materials from coarser than JMF to finer than JMF (coarse-fine gradation), and from finer than JMF to coarser than JMF (fine-coarse gradation). 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.

N. C. Krutz and P. E. Sebaaly (*14*) evaluated the effects of aggregate gradation on permanent deformation of HMA mixtures for the Nevada Department of Transportation and concluded:

- The best aggregate gradation is dependent on the type and source of aggregate.
- Coarse aggregate gradations (bottom of band) performed the worst and fine aggregate gradations (middle and top band) produced better performing mixtures.

R. B. Moore and R. A. Welke (*15*) found that, as the mixture gradation approached the Fuller curve for maximum density, the Marshall stability increased.

T. W. Kennedy et al. (*8*) stated that, in order to prevent permanent deformation of HMA pavements, one should:

- Avoid gradations near the maximum density because, although they theoretically produces the strongest HMA mixtures, due to their relatively low voids in the mineral aggregate, these types of mixtures are very sensitive to asphalt content and present the risk of flushing due to inevitable variations during construction.
- It is better to use aggregates with angular particles because they exhibit greater interlock and internal friction and, hence, result in greater mechanical stability than rounded particles.
- It is better to use aggregates with rough surface texture because they tend to form stronger mechanical bonds when compared to smooth-textured aggregates and provide higher VMA in a compacted mass.

2.3.3 Fine Aggregates

C. Crawford (*16*) concluded from a study related to tender mixtures that particle shape and the amount of material passing the No. 4 sieve (4.75-mm) were major factors contributing to the tenderness of an asphalt concrete mixture. He also stated that rounded, uncrushed aggregates are more likely to contribute to tender mixtures and, therefore, more rutting susceptible, especially as the amount of uncrushed material passing No. 4 sieve increases.

M. Herrin and W. H. Goetz (*17*) found from a laboratory evaluation that the strength of the asphalt mixture, regardless of the type of coarse aggregate, increased substantially when the fine aggregate was changed from rounded sand to crushed fine aggregates.

B. F. Kallas and J. M. Griffith (*18*) studied the influence of fine aggregates on asphalt paving mixtures and demonstrated that an increase in angularity of crushed fines increased the Marshall and Hveem stability values at the optimum asphalt content. An increase in angularity in the fine aggregate also increased the void content at a given compaction level and the optimum asphalt content.

E. Shklarsky and M. Livneh (*19*) found that replacing natural sand materials with crushed fine aggregate increased the stability and strength properties of Marshall specimens, reduced permanent deformation, improved resistance to wear, reduced asphalt content sensitivity, and increased VMA and air voids in the compacted specimen.

R. R. Lottman and W. H. Goetz (*20*) stated that increases in strength of HMA were attributed to the angularity and the roughness of the crushed fine aggregates. The authors recommended that some amount of crushed fine aggregate be used with natural sands in asphalt mixtures to produce sufficient stability for high quality pavements.

J. W. Button and D. Perdomo (*21*) demonstrated that total deformation and rate of deformation increased as the percentage of natural sand increased. Shape and texture of the fine aggregate were major factors controlling plastic deformation in HMA. Replacing natural sand material with manufactured sand increased the resistance of the HMA to permanent deformation.

2.3.4 Coarse Aggregates

M. Yeggoni et al. (*22*) conducted a laboratory study to evaluate the influence of coarse aggregate shape and texture on permanent deformation characteristics of HMA mixtures. The authors concluded that an increase in the percentage of crushed coarse aggregate resulted in increased Hveem stability, Marshall stability, and resistance to permanent deformation. They also found a strong correlation between rutting potential and the shape of the coarse aggregate particles as measured using image analysis.

E. R. Brown et al. (*23*) concluded that the maximum aggregate size greatly affected the pavement performance and that larger maximum aggregate sizes produce higher stability, better skid resistance, and lower optimum asphalt contents.

Y. R. Kim et al. (*24*) demonstrated that aggregate type has significant effects on fatigue resistance and permanent deformation of asphalt concrete. Gradation had no significant effects on permanent deformation. Interactions of aggregate type with gradation, asphalt type, air voids, and temperature were found to be significant for the permanent deformation of asphalt concrete.

C. E. Basset and E. R. Brown (*25*) concluded that:

- Very little change in indirect tensile strength as maximum aggregate size is changed.
- If the maximum aggregate size increases, the mixture will be more resistant to permanent deformation and will have greater resilient modulus values.

2.3.5 Filler

E. R. Brown et al. (*23*), from various laboratory and field studies, concluded that additional minus No. 200 (filler) material produced a lower optimum asphalt content (filler material fills the voids in certain asphalt mixtures and lowers the optimum asphalt content), a higher stability, and a more asphalt sensitive mixture. Some filler is required for stability, but an excessive amount (greater than 6 percent in conventional mixtures) produced unsatisfactory mixtures.

D. A. Anderson (*26*) and J. P. Tarris and D. A. Anderson (*27*) stated that mineral filler characteristics vary with the type and gradation of the filler. Care must be taken to consider not only the amount of filler, but also its particle size distribution in evaluating whether an excessive amount of filler is present in a mixture design. If the size of mineral filler particles is smaller than about 10 microns, i.e., smaller than the asphalt film thickness in the HMA, the filler acts as an extender of the asphalt binder. But, if the mineral filler size is larger than 10 microns, it acts more like an aggregate. If an excessive amount of large size mineral filler is present, the asphalt demand may increase.

2.4 SPECIFICATIONS TO REDUCE RUTTING

Many agencies around the United States have adopted specifications to address rutting distress in asphalt concrete pavements. The main criteria adopted are:

- Increase the percentage of VMA. For instance, Illinois DOT has increased VMA from a minimum of 11-13 percent to a minimum of 15 percent for 1/2- inch mixtures.
- Fix a minimum and maximum percentage of air voids in the asphalt concrete mixture. For instance, Iowa DOT has fixed the limits at 3.5 and 6.0 percent.
- Increase the number of blows in the Marshall compaction (lower binder content).
- Limit the amount of natural sand. FHWA recommends no more than 15 percent.
- Fix a minimum percentage of crushed coarse and fine aggregate.
- Increase the percentage of filler in the mixture.
- Use stiffer asphalt cements binders.
- Use coarser aggregate gradations with appropriate asphalt binder for climate and traffic conditions, as with Superpave.

The authors believe that, with coarser HMA mixtures, the VMA and possibly air void content should likely decrease to improve resistance to rutting. This will be discussed more later. In order to minimize permanent deformation in asphalt pavements, certain aggregate requirements were fixed in Superpave: coarse aggregate angularity, fine aggregate angularity, flat-elongated requirements, and gradation controls. These issues, as well as the other Superpave requirements, will be discussed in the next chapter.

2.5 RECENT STUDIES ON SUPERPAVE RESTRICTED ZONE

Soon after the inception of the Superpave mixture design method, the paving industries and agencies realized some of their successful HMA mixtures did not meet the Superpave specifications. Some mixtures exhibiting good performance pass through the restricted zone. A recent paper presented by Hand et al. (*9*) summarizes results of recent research focusing on the effect of restricted zone on performance of HMA. This review of recent studies indicated that good performance can be achieved with fine graded (ARZ and TRZ) mixtures and, in most cases, fine Superpave mixtures out perform coarse Superpave mixtures. His study concluded that there is no relationship between the Superpave restricted zone and HMA rutting or fatigue performance.

In 1996, David Jahn (*2*) stated that most of the Georgia DOT mixtures which have exhibited good performance pass through the restricted zone. Georgia DOT set a very narrow band of combined aggregate gradation for their 19-mm mixtures. This band is well suited for the local aggregate source, and resulting HMA mixtures have been used successfully for heavy traffic. In order to pass the Georgia DOT specification, one (practically speaking) has to violate the restricted zone. Watson et al. (*28*) indicated similar findings. He mentioned that Georgia DOT's good-performing Type B (19.0-mm nominal maximum size), Type F (9.5-mm NMS), and Type E (12.5-mm NMS) mixtures usually encroach the restricted zone. These mixtures resulted from extensive research and are performing well in high-volume traffic roads. The Type B mixture exhibited exceptional field performance in rutting susceptibility.

T. Kuennen (*29*) discussed two years of field experience with Superpave pavements. He mentioned that, in certain regions of the country, Superpave mixes are performing well even though the mixtures include aggregate fines that fall within the restricted zone.

B. Prowell (*30*) studied the field and laboratory performance of HMA to evaluate the performance of stabilizers and modifiers. Ten test sections were built in 1995 on IH 66, in

Virginia. The test sections were constructed using both dense-graded Marshall and coarse-graded Superpave mixtures. Dense-graded Marshall mixtures followed all the Superpave volumetric requirements except of the gradation, which passed through the restricted zone. Prowell mentioned that all of the test sections are rut resistant and are performing well after 45 months of service.

Anderson et al. (*3*) evaluated a Superpave mixture design data base. In that study, the researchers examined 128 trial aggregate blends used for mixture design during the period of 1992-96. Their objective was to set a guideline for the mixture designers, more specifically, they were focused to identify the gradation or gradation characteristics which can yield adequate VMA for the mixtures. They tried to find a correlation between VMA and the distance from maximum density line on the 0.45 power gradation chart or distance from the restricted zone. They did not find any statistically good correlation between VMA in an asphalt mixture and the sum of the distances from the Superpave maximum density line or the sum of the distances from the restricted zone. In the same study, the researchers designed and evaluated HMA of four different gradations using only one aggregate source. The combined gradations were a S-shaped coarse gradation, a fine gradation above the restricted zone, an intermediate gradation passing through the restricted zone, and a S-shaped coarse but with slightly humped gradation. The asphalt mixtures were evaluated using simple shear at constant height and repeated shear at constant height test using the Superpave shear tester. The researchers noticed that the gradation above restricted zone performed the best and those below restricted zone performed the worst. They concluded that contrary to the common contention, finer gradations have stronger aggregate structure than coarse gradations.

Sousa et al. (*31*) evaluated the effect of aggregate gradation on the fatigue life of HMA. In this study, 100 percent crushed granite with gradations passing above, through, and below the restricted were used. To evaluate fatigue life of HMA, four-point bending fatigue tests were performed according to the SHRP M009 test protocol. Fatigue lives of 230 actual laboratory tests were compared with predictions by Shell, Asphalt Institute, and SHRP-A003 fatigue predictive equations. One of their conclusions was that the fine-graded mixtures (above

(ARZ) and through (TRZ) the restricted zone) out-performed those below (BRZ) the restricted zone, with respect to fatigue life.

Van-de-Ven et al. (*32*) reported on a joint study between University of Stellenbosch, South Africa and South Central Superpave Center (located then at Austin, Texas). The objective of this research was to examine the Superpave aggregate gradation and fine aggregate angularity specification. Basically, three gradations: above, through, and below the restricted zone were used. The nominal maximum size of 100 percent crushed aggregate used in the study was 9.5 mm. For mixtures below restricted zone, researchers used fine aggregates with different FAA values. Even though the mixtures were designed using the Superpave volumetric method, some of the mixtures did not meet all of the Superpave requirements. Obviously, the restricted zone requirement was violated intentionally. SST and Model Mobile Load Simulator (MMLS) were used to evaluate relative properties of HMA. Dynamic creep and indirect tensile tests were also performed. The authors pointed out that a small variation in nominal maximum aggregate size of a mix may change the restricted zone of that mix. Although based on the limited data, one of the conclusions of this research was that the mixtures passing through the restricted zone perform well and sometimes better than those below or above the restricted zone.

Cooley, (*33*) expressed concern that the Superpave coarse mixtures (gradations passing below the restricted zone) are more permeable than pavements previously designed with Marshall hammer.

Rouque et al. (*34*) examined the influence of aggregate gradation on shear resistance and volumetric properties of HMA. Other objectives of that study were to find an optimized aggregate gradation to maximize shear resistance to determine if it is possible to produce dense gradations that provide shear resistance equal to or greater than that of stone matrix asphalt (SMA) mixture. Eighteen mixtures were prepared using different coarse aggregate gradations ranging from SMA to those near the maximum density line. Gradations near the maximum density line can be considered as TRZ (*9*). Shear resistance of mixtures were estimated using the gyratory shear value determined from Corps of Engineers gyratory test machine. This study showed that a broad range of aggregate gradations ranging from TRZ to SMA can yield good shear resistance in HMA. Gradation of the coarse aggregate fraction is the most pronounced factor affecting the shear resistance of the HMA. VMA could not be related to shear resistance of the mixture.

El-Basyouny et al. (*35*) studied the effect of aggregate gradation, nominal maximum size, and binder content on the rutting related volumetric properties of HMA mixtures. In that study, mixtures were prepared using aggregate with different gradations (ARZ, TRZ, and BRZ). Using the results from uniaxial creep tests, VESYS-3AM software predicted their rutting potential. This software predicted a 10-mm rut depth for TRZ mixture and an 11-mm rut depth for ARZ and BRZ mixture.

During 1994-95, a 2.9-km oval test track (WesTrack) was constructed at the Nevada Automotive Test Center near Reno, Nevada under the sponsorship of FHWA (Project No. DTFH61-94-C-00004) and National Cooperative Highway Research Program (NCHRP Project No. 9-20). This full-scale test track contained 26 test sections. Two of the several objectives of these test sections were to examine the effect of variability of construction and materials on the performance of HMA and to establish a field verification of the Superpave mixture design and analysis system. Beginning in 1996, loading was applied with driverless triple trailers/trucks operating at a speed of 65 km/hr. During March 1996 to June 1998, 4.7 million 80-kN ESALs were applied (*9*). Numerous types of pavement performance data were collected bi-weekly and monthly.

The WesTrack test sections originally constructed included three different gradations: fine (ARZ), S-shaped coarse (BRZ), and fine plus (fine gradation with additional bag house fines) graded mixtures. Crushed gravel and PG 64-22 asphalt were used for these sections. The amount of filler, asphalt content, and air voids were varied systematically to simulate construction variability in the field (*9*). The mixtures were designed following the Superpave volumetric mixture design system. Performance of the BRZ mixture sections were unexpectedly poor. These coarse-graded sections exhibited the greatest amount of rutting and fatigue cracking of all mixture variable combinations. All coarse-graded sections were replaced with similar gradations but different aggregate (100 percent crushed granite). Other variables were kept essentially the same. The performance of these replacement sections were even worse. Both types of fine-graded sections exhibited clearly better performance than the coarse-graded sections.

National Pooled Fund Study 176 was conducted in two phases (*36*). One objective of this study was to investigate the effect of aggregate gradation on permanent deformation of HMA and validate the Superpave volumetric specification. Phase I of this study was limited to only six mixtures containing limestone and limestone sand. Phase II of this study was conducted with twenty-one mixtures. These mixtures were composed of two coarse aggregates (granite and limestone) and three fine aggregates (granite, limestone, and natural sand). Two aggregates (nominal maximum size 19.0 mm and 9.5 mm) with gradations ARZ, TRZ, and BRZ were used for mixture design.

Mixture performance was evaluated (*36*) using Superpave volumetric mixture design data, a triaxial test, PURWheel laboratory-scale wheel track test, and Indiana DOT/Purdue University prototype-scale accelerated pavement test (APT). The triaxial test was performed in the dry condition. Specimens compacted using the Superpave gyratory compactor (SGC) were axially loaded at a constant confining pressure to 1.0 percent compressive strain. Stresses obtained at this strain level were plotted against asphalt content. From that plot, the authors suggest that the mixtures could be observed to transition from a stable state to unstable state. PURWheel is a laboratory-scale torture test device. It can be operated on a compacted slab in the dry or wet condition. Twenty thousand wheel passes or 12.5-mm rut (whichever comes first) was applied on the compacted slab at a tire pressure of 793 kPa. The INDOT/Purdue APT is a prototype-scale torture test device where one or two directional wheel (or dual wheel) loads can be applied on compacted mat. The APT is more suitable to simulate the truck traffic than the PURWheel.

The authors (*36*) summarized their observations stating that both laboratory and prototype-scale performance tests indicated that adequate rutting performance can be achieved with gradations ARZ, TRZ, and BRZ. They found that ARZ and TRZ mixtures might provide slightly better performance than BRZ mixtures. APT results did not show clear trend.

Mallick et al. (*37*) conducted a related study. Their objective was to evaluate rutting potential of HMA with gradations both complying with and violating the Superpave restricted zone. In that study, researchers designed HMA mixtures containing granite, limestone, and gravel. All three aggregates were crushed. Gradations used were ARZ, TRZ, and BRZ. Mixtures were designed for wearing courses and binder courses using 12.5-mm and 19.0-mm nominal maximum size of aggregate, respectively. Only one type of asphalt (PG 64-22) was used. Test samples, compacted using the SGC, were tested using the APA and repeated shear at constant height test. APA and RSCH tests were conducted using 8,000 and 5,000 cycles, respectively.

The researchers (*37*) summarized their observations by stating that the statistical analyses of APA rut depth data obtained on all mixtures indicated significant differences in performance among different gradations. They observed that, for granite and limestone, BRZ generally exhibited the highest and TRZ exhibited the lowest rut depths, and ARZ showed intermediate rut depths. For river gravel mixtures, the order from highest to lowest rut depth was ARZ, BRZ, and TRZ. Test results from RSCH was not as definitive as that from APA, but it followed the same general trend. The BRZ limestone mixture yielded the highest peak shearing strain for both wearing and binder courses. TRZ river gravel showed the lowest and ARZ river gravel showed the highest peak shearing strain for both wearing and binder courses.

Very recently, the National Center for Asphalt Technology (NCAT) completed construction of a 2.7-km oval test track near Auburn, Alabama. This track will be used for full-scale accelerated testing of flexible pavements. HMA mixtures with coarse, fine, and through restricted zone gradations mixtures will be used to construct the test sections. This full-scale testing facility will provide an excellent opportunity to examine the effects of gradation on field performance.

On the basis of results from previous research, the authors found that rut resistant HMA can be developed using fine-graded mixtures. Most of the studies indicated that the ARZ and/or TRZ gradation Superpave mixtures exhibit less permanent deformation than coarse-graded BRZ Superpave mixtures. Some researchers concluded that mixtures with adequate rut resistance can be produced with either of the gradations.

CHAPTER 3
EXPERIMENTAL PROGRAM

3.1 PLAN OF STUDY

This research focused on examining the effects of crushed and uncrushed aggregate gradations on permanent deformation in Superpave HMA mixtures. The coarse side (plus No. 4 sieve) of the grading curve remained unchanged, while the fine side (minus No. 4 sieve) was varied in order to pass through, above, and below the restricted zone. Laboratory tests were used to predict pavement rutting (Table 1). The work plan was divided in the following steps:

- Materials selection and acquisition: This phase includes identification and collection of the four aggregate types (partially crushed river gravel, crushed granite, crushed limestone, combination of partially crushed river gravel & rounded natural sand) and one binder to prepare the HMA blends.
- Asphalt cement and aggregate characterization: The individual mixture components were tested to determine if they meet Superpave requirements.
- Superpave mixture design: Several trial blends were prepared to obtain the design aggregate gradation and asphalt content for the different mixtures (4 aggregate types × 3 gradations = 12 HMA designs).
- Asphalt concrete mixture evaluation: Performance tests to establish rut resistance of the HMA mixtures were performed. Performance test of HMA includes the use of Superpave Shear Tester and Asphalt Pavement Analyzer.

The test plan includes preparation of HMA specimens with three aggregate gradations (ARZ, TRZ, and BRZ) with all four aggregates using design or optimum asphalt content. For each of these aggregate gradations and asphalt contents, different sets of specimens were prepared at different degrees of compaction (different air void levels). As specified in Superpave, replicate specimens were tested to improve the reliability of the results.

Since the river gravel mixtures proved more susceptible to permanent deformation, additional specimens were prepared with high and low asphalt contents (Table 1). After the first three mixtures showed no effect of the restricted zone, a fourth low-quality mixture containing river gravel and rounded (uncrushed) fines was designed. This was done to determine if a rut-susceptible mixture containing rounded sand show any effect of the restricted zone. This mixture did not meet all the Superpave criteria (FAA and VMA).

Table 1. Different Mixtures and Test Description

Aggregate	Name of Tests	Asphalt Content	Number of Specimens	Air Voids (%)	Test Temperature (˚C)
Partially Crushed River Gravel	SSCH	Design, high, & low	2*	7	4, 20, and 46
	FSCH	Design, high, & low	2*	7	4, 20, and 46
	RSCSR	High	2	3	46
	RSCH	Design	2	4	46
	APA	Design	3 pair	4	64
100 % Crushed Granite	SSCH	Design	2*	7	46
	FSCH	Design	2*	7	46
	RSCSR	High	2	3	46
	APA	Design	3 pair	4	64
100 % Crushed Limestone	SSCH	Design	2*	7	46
	FSCH	Design	2*	7	46
	RSCSR	High	2	3	46
	APA	Design	3 pair	4	64
Crushed River Gravel & Rounded Sand	SSCH	Design	2*	7	46
	FSCH	Design	2*	7	46
	RSCSR	High	2	3	46
	APA	Design	3 pair	4	64

* Same specimen is used for both SSCH and FSCH test

3.2 MATERIALS SELECTION AND ACQUISITION

This study focused on crushed river gravel because, among the three primary aggregate types, river gravel was assumed to be most prone to permanent deformation, since it presents partially rounded particle shape and relatively smooth surface texture. Although the river gravel was crushed, it retained some rounded faces with smooth surface texture. Studies conducted by R. C. Ahlrich (*38*) and A. Chowdhury (*39*) demonstrated that crushed river gravel aggregates often contain rounded particles with smooth surface texture even after crushing. Granite and limestone were selected because they posses widely different characteristics and are commonly used in asphalt pavements.

The origin of the aggregates used in this study are: partially crushed river gravel from McAllen, Texas (Fordyce); crushed limestone from Brownwood, Texas (Vulcan Materials); and crushed granite from Forsyth Quarry, Georgia (Martin Marietta). The fourth aggregate, i.e., the combination of coarse crushed river gravel and rounded natural sand was selected intentionally to develop a poor HMA mixture. The rounded natural sand was collected from the Brazos river valley in Brazos county, Texas. The crushed gravel, limestone, and granite have demonstrated generally good field performance in HMA. About one ton of each of these aggregates was obtained for mixture design and specimen preparation.

Binder selection was according to the Superpave binder specification (AASHTO MP1, Appendix A). In this specification, the binders are selected on the basis of the climate and traffic in which they are intended to serve. The geographic location selected for this study was Lubbock, Texas, and the traffic level selected was between 3 and 10 million ESALs for limestone, river gravel, and natural sand aggregates, and between 1 and 3 million ESALs for granite. The traffic level for granite is different because its gradation curve passing through the restricted zone is a gradation curve commonly used in Georgia (provided by the Georgia DOT), and researchers could not achieve a Superpave volumetric mix design for 3 to 10 million ESALs. These traffic (1 to 10 million ESALs) levels were selected because they correspond to intermediate levels of analysis in Superpave, and they are anticipated to be the predominant Superpave analysis used in typical highway applications (*6*). The PG grade that corresponds to this geographic location and the traffic levels, obtained from Superpave Software version

2.0 program using a 98% reliability, is PG 64-22. Researchers assumed that the projected pavements will be subjected to fast moving loads, so no adjustment for the binder grade was required.

3.3 TESTS FOR ASPHALT CEMENT CHARACTERIZATION

One of the three major components of the Superpave mixture design process is the asphalt binder performance grading specification (AASHTO MP1). Asphalt binder is tested in conditions that simulate its critical stages during the service, such as:

- During transportation, storage, and handling - original binder is tested.
- During mix production and construction - simulated by short-term aging the original binder in a rolling thin film oven (RTFO).
- After 5 to 10 years of service - simulated by long term aging the binder in the rolling thin film oven test plus the pressure aging vessel (PAV). In the PAV, the RTFO residue is exposed to high air pressure and temperature for 20 hours to simulate the effect of long-term pavement aging.

Results of the binder tests are included in Appendix A.

3.3.1 Dynamic Shear Rheometer (DSR)

Researchers used the DSR to characterize viscous and elastic behavior of asphalt binders at high and intermediate service temperatures. The DSR measures the complex shear modulus (G^*) and phase angle (δ) of asphalt binders at desired temperature and frequency of loading. Complex modulus is a measure of the total resistance of a material to deformation when repeatedly sheared. It consists of two components:

- Storage modulus (G') or the elastic (recoverable) part,
- Loss modulus (G'') or the viscous (non recoverable) part.

The lag time between the applied peak stress and resulting peak strain is the phase angle (δ). For perfectly elastic materials the phase angle is 0 degrees, and for perfectly viscous fluid materials it is 90 degrees. Asphalt binders behave like elastic solids at very low temperatures and like viscous fluids at high temperatures. However, at typical pavement service temperatures, it behaves like a viscoelastic material, therefore, δ will be greater than zero but smaller than 90 degrees (*4*).

The DSR is used to determine the rutting parameter of the asphalt binder at high temperatures for unaged binders and short-term aged binders. For rutting resistance, a high complex shear modulus (G^*) value and low phase angle (δ) are both desirable. Higher G^* values indicate stiffer binders that are more resistant to rutting. Lower δ values indicate more elastic asphalts that are more resistant to rutting. Therefore, a larger $G^*/\sin\delta$ signifies more resistance to permanent deformation by the asphalt binder.

The DSR is also used to determine the fatigue resistance of the asphalt binder at intermediate temperatures for long-term aged binders. For fatigue resistance, a low complex modulus value and a low phase angle are both desirable. Therefore, smaller values of $G^*\sin\delta$ indicate more resistance to fatigue cracking.

3.3.2 Bending Beam Rheometer (BBR)

The BBR measures a binder's resistance to thermal cracking. Thermal cracking may occur in asphalt pavements when the temperature drops rapidly at low temperatures. The BBR uses a transient creep bending load on the center of an asphalt beam specimen held at a constant low temperature. This test is performed on asphalt binder that has been subjected to long-term aging. From this test, two parameters are obtained:

- Creep stiffness - a measure of how the asphalt binder resists the constant creep loading.
- m-value - which is a measure of the rate at which the creep stiffness changes with time of loading.

If creep stiffness increases, the thermal stresses developed in the pavement due to thermal shrinking also increase, and thermal cracking becomes more likely. If m-value decreases (the curve flattens) the ability of the asphalt binder to relieve thermal stresses decreases, and the propensity of thermal cracking increases.

3.3.3 Direct Tension Tester (DTT)

The DTT measures the low temperature ultimate tensile strain of the binder. This test is performed using binder subjected to long-term aging. The DTT is performed only when the asphalt creep stiffness obtained from the BBR is greater than 300 MPa but smaller than 600 MPa. The DTT is performed because there are some asphalt binders which may have high creep stiffness but do not crack because they can stretch further before breaking. Larger failure strain indicates more ductile binders and, therefore, more resistant to cracking.

3.3.4 Rotational Viscometer (RV)

The rotational viscometer was adopted in Superpave for determining the viscosity of asphalt binder at high temperatures, primarily to ensure that it is sufficiently fluid for pumping or mixing. Rotational viscosity is determined by measuring the torque required to maintain a constant rotational speed of a cylindrical spindle. HMA mixing and compaction temperature ranges are determined using the rotational viscometer

3.3.5 Mixing and Compaction Temperature

Superpave HMA mixtures are mixed and compacted under equiviscous temperature conditions corresponding to 0.17 Pa-s and 0.28 Pa-s, respectively (*6*). Viscosity of the asphalt was tested using Brookfield rotational viscometer at 135˚C and 175˚C. Plotting the result in a viscosity versus temperature graph (log-normal), the mixing and compaction temperature ranges were determined.

Details of the binder testing results and determination of the mixing and compaction temperatures are described in Appendix A.

3.4 TESTS FOR AGGREGATE CHARACTERIZATION

Superpave specifications contain two categories of aggregate properties: consensus properties and source properties (*6*). Consensus properties are those aggregate characteristics which are critical to well performing asphalt mixtures. These properties include:

- Coarse aggregate angularity,
- Fine aggregate angularity,
- Flat and elongated particles, and
- Clay content.

The specific criteria for these consensus aggregate properties are based on traffic level and position of the layer within the pavement structure.

Source properties are those aggregate properties that, although important for the asphalt mixture performance, they were not considered critical, and no critical values for those properties were defined by Superpave (*4*). Criteria for the aggregate source properties are left for the local agencies. Those properties include:

- Toughness,
- Soundness, and
- Deleterious materials.

Only the consensus aggregate properties were considered in this study because they can be related to permanent deformation in HMA mixtures. The source aggregate properties were not examined since these tests do not correlate particularly well with pavement deformation *(40, 41)*.

3.4.1 Coarse Aggregate Angularity (CAA)

CAA is the percent by weight of aggregates larger than No. 4 (4.75-mm) with one or more fractured faces. Higher CAA enhances coarse aggregate internal friction and thus HMA rutting resistance (*6*). CAA was measured following ASTM D 5821-95. A fractured face is defined as an angular, rough, or broken surface of an aggregate particle created by crushing, by other artificial means or by nature. A face will be counted as fractured only if it has a projected area at least as large as one quarter of the maximum projected area (maximum cross-sectional area) of the particle and the face has sharp and well defined edges (*42*).

Superpave has a required minimum value for CAA as a function of traffic level and position within the pavement. The traffic level selected was less than 3 million ESALs for granite blends and less than 10 million ESALs for limestone and river gravel blends. The depth from the surface selected was less than 100 mm primarily because the study is focused on plastic deformation in the asphalt layers, and this type of rutting occurs mainly in the uppermost asphalt layers.

Tables 2, 3, and 4 show that the aggregates meet the coarse aggregate angularity requirements except for the 19-mm river gravel, which does not meet either of the fractured faces criteria. However, this material can be used as long as the selected blend of coarse aggregate meets the design criterion.

Table 2. Coarse Aggregate Angularity for River Gravel (also used with Rounded Sand)

Aggregate Size	1 + Fractured Faces	Minimum Criterion	2 + Fractured Faces	Minimum Criterion
19 mm	82	85	72	80
12.5 mm	89	85	84	80
9.5 mm	93	85	90	80
4.75 mm	98	85	95	80
Coarse Gradation	93*	85	89*	80

*Each of the three river gravel blends has the same coarse aggregate proportions.

Table 3. Coarse Aggregate Angularity for Granite

Aggregate Size	1 + Fractured Faces	Minimum Criterion	2 + Fractured Faces	Minimum Criterion
19 mm	90	75	85	-
12.5 mm	95	75	91	-
9.5 mm	97	75	94	-
4.75 mm	99	75	98	-
Coarse Gradation	97*	75	93*	-

*Each of the three granite blends has the same coarse aggregate proportions.

Table 4. Coarse Aggregate Angularity for Limestone

Aggregate Size	1 + Fractured Faces	Minimum Criterion	2 + Fractured Faces	Minimum Criterion
19 mm	92	85	88	80
12.5 mm	96	85	93	80
9.5 mm	98	85	95	80
4.75 mm	99	85	97	80
Coarse Gradation	97*	85	94*	80

*Each of the three limestone blends has the same coarse aggregate proportions.

3.4.2 Fine Aggregate Angularity (FAA)

FAA is the percent air voids present in loosely compacted aggregates of a specified gradation smaller than 2.36 mm. Higher void contents generally mean more fractured faces. This criterion is designed to ensure a high degree of fine aggregate internal friction and thus rutting resistance (*6*). The test procedure followed was ASTM C 1252, Method A. Superpave has a required minimum value for fine aggregate angularity as a function of traffic level and layer position within the pavement. Design traffic level and depth have been stated in Section 3.4.1. Test results are shown in Table 5.

Although limestone and crushed river gravel do not meet the FAA criteria (see Table 4), the values obtained were accepted because the measured values are very close to the minimum criterion, and the aggregate has demonstrated good performance in HMA. Chowdhury (*39*) demonstrated that FAA values for aggregate containing 100 percent crushed but cubical particles were often lower than those for aggregates containing rounded particles.

Table 5. Fine Aggregate Angularity

	FAA	Minimum criterion (%)
River gravel	44.3	45
Granite	48.0	40
Limestone	43.5	45
River Gravel + Rounded Sand	39.0	45

3.4.3 Flat and Elongated Particles (F&E)

According to Superpave, F&E is the percentage by mass of coarse aggregate particles larger than 4.75 mm that have a maximum to minimum dimension ratio greater than five. This criterion is an attempt to avoid particles with a tendency to break during construction and under traffic. The test procedure followed was ASTM D 4791 (Table 6).

Superpave has a required maximum value for F&E coarse aggregate particles as a function of traffic level.

3.4.4 Clay Content

Clay content is the percentage of clay material (by volume) contained in the aggregate fraction finer than 4.75 mm. Superpave has a required minimum value for clay content of fine aggregate particles as a function of traffic level. This property ensures that the relative proportion of clay-like or plastic fines in granular soils and fine aggregates is not too high. The test procedure followed was ASTM D 2419-95 (Table 7).

Table 6. Flat and Elongated Particles for Aggregates

Aggregate Type	Aggregate Size	Percent by Weight Flat and Elongated Particles	Requirement (maximum percent)
River Gravel	19.0 mm	6	N/A
	12.5 mm	4	
	9.5 mm	3	
	4.75 mm	2	
	Coarse gradation	3*	10
Granite	19.0 mm	8	N/A
	12.5 mm	9	
	9.5 mm	8	
	4.75 mm	5	
	Coarse gradation	8*	10
Limestone	19.0 mm	6	N/A
	12.5 mm	4	
	9.5 mm	4	
	4.75 mm	2	
	Coarse gradation	4*	10
River Gravel + Rounded Sand	Coarse gradation	3* (Same as River Gravel)	10

* Three blends (ARZ, TRZ, and BRZ) of each aggregate has the same coarse aggregate proportions.
** F&E criterion for all traffic levels is a maximum of 10% F&E particles by weight of total particles > 4.75 mm.

Table 7. Clay Content

Aggregate Type	Sand Equivalent (%)	Minimum Criterion (%)
River gravel	88	45
Granite	79	40
Limestone	78	45
River Gravel + Rounded Sand	89	45

3.4.5 Specific Gravity

The specific gravity of the aggregates is required to determine fine aggregate angularity as well as for the Superpave mixture design. The test procedure followed was ASTM C 127 for coarse aggregates and ASTM C 128 for fine aggregates.

The criteria for distinguishing between coarse and fine aggregates is the 4.75-mm (No. 4) sieve. Table 8, 9, and 10 provide the specific gravity values for each aggregate used in this study. The specific gravity of the coarse portion of the rounded natural sand mixture was the same as that listed for river gravel of river gravel.

Table 8. Crushed River Gravel and Rounded Natural Sand Specific Gravity

Aggregate Size	Bulk Specific Gravity (oven dry)	Bulk Specific Gravity (SSD)*	Apparent Specific Gravity
+19 mm (coarse)	2.578	2.591	2.613
+12.5 mm(coarse)	2.603	2.617	2.642
+9.5 mm(coarse)	2.604	2.619	2.643
+4.75 mm(coarse)	2.597	2.616	2.647
−4.75 mm (fine)	2.578	2.609	2.662
Rounded Natural Sand (Fine)	2.572	2.592	2.643

* saturated surface dry.

Table 9. Granite Specific Gravity

Aggregate Size	Bulk Specific Gravity (oven dry)	Bulk Specific Gravity (SSD)	Apparent Specific Gravity
+19 mm (coarse)	2.706	2.719	2.743
+12.5 mm(coarse)	2.704	2.718	2.743
+9.5 mm(coarse)	2.704	2.717	2.743
+4.75 mm(coarse)	2.705	2.718	2.742
−4.75 mm (fine)	2.672	2.701	2.752

Table 10. Limestone Specific Gravity

Aggregate Size	Bulk Specific Gravity (oven dry)	Bulk Specific Gravity (SSD)	Apparent Specific Gravity
+19 mm (coarse)	2.668	2.689	2.729
+12.5 mm(coarse)	2.664	2.687	2.726
+9.5 mm(coarse)	2.667	2.687	2.723
+4.75 mm(coarse)	2.668	2.682	2.671
−4.75 mm (fine)	2.633	2.668	2.729

3.5 SUPERPAVE MIXTURE DESIGN

One of the three major components of Superpave is the mixture design procedure. Once the aggregates and asphalt materials have been selected and tested, the following steps are followed to develop the mixture design:

- Develop aggregate trial blends,
- Prepare mixtures,
- Compact specimens,

- Conduct volumetric analysis, and
- Determine the optimum asphalt content.

Superpave mixtures were designed using AASHTO PP 28-97 standards. The mixture design data for the different blends is provided in Appendix B.

3.5.1 Aggregate Blends

To properly develop the aggregate blends, the aggregates were sieved and separated into bins then recombined in preparing asphalt mixtures. This separation was very important in order to obtain accurate mixture gradations, because the differences between the three gradations developed (ARZ, TRZ, and BRZ) for the different aggregates were fairly small.

The Superpave aggregate gradation controls were maintained using the FHWA 0.45 power chart (Figure 1). This chart is used to define a permissible gradation. This chart uses a unique graphing technique where the ordinate shows the percent passing and the abscissa is an arithmetic scale of sieve size in millimeters, raised to the 0.45 power (*4*). The maximum density gradation theoretically plots as a straight line from the maximum aggregate size (two sieve sizes larger than the first sieve size to retain more than 10 percent) to the origin. The mix gradation must pass between certain control points. These control points function as master ranges serving three purposes: to control the top size of the aggregate, to control the relative proportion of coarse and fine aggregate, and to control the proportion of dust (*4*).

There is an area called 'restricted zone' lying along the maximum density line extending from the 0.30-mm (No. 50) sieve size to the 2.36-mm (No. 8) or 4.75-mm (No. 4) sieve size, through which it is undesirable for a mix gradation to pass (*5*). The zone terminates at the 2.36-mm or 4.75-mm sieve, depending on the nominal maximum size of aggregate used. This restricted zone was established in an attempt to minimize the risk of poor volumetric properties; to minimize the amount of rounded, fine sands; and to encourage the development of a strong aggregate structure in the mixture.

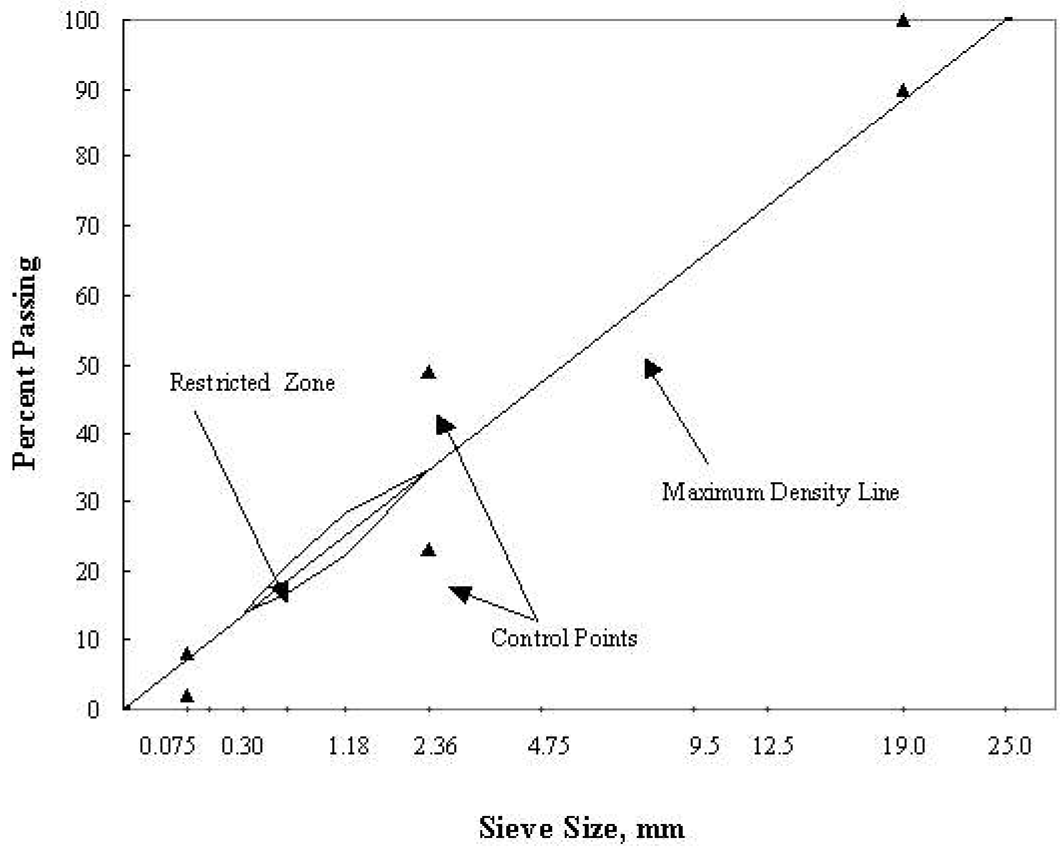

Figure 1. Superpave Gradation Control for 19.0-mm Mixtures

The restricted zone was adopted by Superpave to reduce premature rutting. However, it is one of the most controversial components of the Superpave mixture design procedure.

The location of the control points as well as the restricted zone depend on the aggregate nominal maximum size. In Superpave, the nominal maximum size is defined as one sieve size larger than the first sieve to retain more than 10 percent of the aggregates. Maximum size is defined as one sieve size larger than the nominal maximum size. Each one of the aggregate blends selected has the same nominal maximum size, which is 19 mm.

Detailed characterizations of the different aggregate blends is provided in Appendix B. During the development of these aggregate blends, several trial blends were tested in order to select viable blends.

3.5.2 Preparation of HMA Mixtures

Once the aggregate blends were selected and the initial trial asphalt binder content was calculated, the HMA mixtures were prepared. This phase consists of the following main steps:

- Heating the aggregates and asphalt binder to the mixing temperature (159 ± 3˚C).
- Mix both components and short-term age the mixture for 4 hours at 135˚C.
- Compaction the mixture at a temperature of 145 ± 3˚C.

3.5.3 Compaction

All specimens were compacted using the SGC manufactured by Industrial Product Corporation, Inc., Australia (Figure 2). The SGC was developed by SHRP researchers to achieve the following objectives:

- Obtain realistic compaction of specimens,
- Be an effective method of compaction for aggregate gradations with particle sizes up to 37.5 mm,
- Be able to monitor compactability during the process of compaction, and
- Be portable;

The SGC was based on the Texas gyratory compactor and the French gyratory compactor.

In Superpave, as with other mixture design procedures, asphalt mixtures are designed using a specific compactive effort. Compactive effort is a function of the design number of gyrations, N_{des}. N_{des} is used to vary the compactive effort of the design mixture as a function of climate and traffic level. Two other compaction levels are of interest: the initial number of gyrations (N_{ini}) and maximum number of gyrations (N_{max}).

$$\text{Log } N_{ini} = 0.45 \times \text{Log } N_{des}$$
$$\text{Log } N_{max} = 1.10 \times \text{Log } N_{des}$$

Climate is represented by the average design high air temperature. For Lubbock, Texas, it is <39 ˚C. Selected traffic levels were 1-3 million ESALs for granite blends, and 3-10 million ESALs for limestone, river gravel, and the rounded natural sand mixture, as stated before. For the selected traffic levels, $N_{ini,}$ N_{des}, and N_{max} are indicated in Table 11. Specimens for the volumetric analysis were compacted to N_{max}.

Figure 2. Superpave Gyratory Compactor by Industrial Process Control

3.5.4 Volumetric Analysis

To complete the volumetric analysis, determination of the bulk specific gravity of the specimens compacted at N_{max} was required. Bulk specific gravity was determined using the standard test method for non-absorptive compacted bituminous mixtures (ASTM D 2726). The ratio between the measured bulk specific gravity and the estimated bulk specific gravity obtained from the gyratory compactor at N_{max} is the correction factor. This correction factor was applied to the estimated bulk specific gravities of the specimen during the compaction process. With the data obtained from the Superpave gyratory compactor and the bulk specific gravity of the specimens, the volumetric analysis can be completed.

Table 11. Superpave Gyratory Compactive Effort

	N_{ini}	N_{des}	N_{max}
River Gravel	8	96	152
Granite	7	86	134
Limestone	8	96	152
River Gravel + Rounded Sand	8	96	152

Superpave specifies acceptable values for the following volumetric characteristics of the specimen:

- Percentage of air voids at N_{des},
- Percentage of the theoretical maximum specific gravity of the mix (% G_{mm}) at $N_{initial}$ and N_{max},
- Voids in the mineral aggregate (VMA), according to the nominal maximum aggregate size (19-mm for the blends analyzed),
- Voids in the mineral aggregate filled with asphalt (VFA), according to the traffic level,
- Dust proportion, which is the percent by mass of the material passing the 0.075-mm sieve size divided by the effective asphalt binder content in percent.

Volumetric criteria for the different aggregates is given in Table 12.

Table 12. Superpave Mixture Design Volumetric Criteria

Volumetric Parameter	River Gravel	Granite	Limestone	River Gravel + Rounded Sand
Air voids at N_{max} (%)	>2	>2	>2	>2
Air voids at N_{ini} (%)	>11	>11	>11	>11
Air voids at N_{des} (%)	4	4	4	4
VMA at N_{des} (%)	>13	>13	>13	>13
VFA at N_{des} (%)	65-75	65-78	65-75	65-75
Dust proportion	0.6-1.2	0.6-1.2	0.6-1.2	0.6-1.2

Once the volumetric analysis has been conducted in the trial blends, the asphalt content of these trial blends was corrected in order to estimate the asphalt content required to obtain 4% air voids at N_{des} (the most restrictive of all the requirements). With this estimated asphalt content, the other volumetric characteristics of the blends will change, therefore, they are also estimated with the equations provided in the FHWA report tilted "Background of Superpave Asphalt Mixture Design and Analysis" (*6*). After establishing all the estimated properties, the estimated values obtained for the different trial blends are compared with the volumetric requirements to determine if any of the trial blends are acceptable or if more trials are needed.

3.5.5 Design Asphalt Content

The optimum asphalt content was determined by compacting and analyzing two specimens at each of the following four asphalt binder contents:

- Estimated asphalt binder content (obtained previously from the trial blends),
- Estimated asphalt binder content + 0.5%,

- Estimated asphalt binder content - 0.5%, and
- Estimated asphalt binder content + 1.0%

Compaction and volumetric properties were evaluated for the selected blend at the different asphalt binder contents. From these values, graphs of air voids, VMA, and VFA were plotted as a function of asphalt content. The design asphalt binder content was established at 4.0 percent air voids, and the other mixture properties were checked.

3.6 TESTS FOR ASPHALT CONCRETE EVALUATION

The mixtures designed were tested in the laboratory primarily to evaluate their relative resistance to permanent deformation. Three types of Superpave shear tests were performed on all mixtures.

- Simple Shear at Constant Height- A specific shear stress is applied to the sample at a constant rate. This stress value is maintained for 10 seconds, after which it is reduced to zero at a defined rate. The height of the specimen is kept constant throughout the test (*43*).
- Frequency Sweep at Constant Height- A sinusoidal shear strain with an amplitude of ±0.05-mm/mm at different frequencies (from 0.1 to 10 Hz) is applied. The number of cycles applied with each frequency is between 4 and 50 (*44*). The height of the specimen is kept constant throughout the test.
- Repeated Shear Test at Constant Stress Ratio- Repeated shear and axial stresses are applied with a ratio between 1.2 and 1.5 for a certain number of cycles. The objective of this test is to identify whether the mix will exhibit tertiary plastic flow (tertiary creep) (*45*).

For the river gravel mixtures, Repeated Shear at Constant Height was also peformed. This test is not required by Superpave, but it was developed as a simplified method to predict premature rutting. In this test, repeated shear and axial stresses are applied, but the axial stress, in this case, is required to maintain the specimen at constant height. A detailed explanation of the tests and results is provided in the next chapter.

All mixtures were tested using Asphalt Pavement Analyzer (Figure 3). Loaded steel wheels are oscillated over a pneumatic rubber hose located on cylindrical or beam specimens for a specified number of cycles (usually 8,000) at certain test conditions. The depressions formed on the sample are measured and termed as APA rut depth. APA rut depth provides an indication of rut susceptibility of the HMA mixture on a pavement. In this study, only cylindrical specimens compacted by SGC were tested in the dry condition to assess rutting susceptibility (Figure 3).

Figure 3. APA Testing Setup (Rubber Hose is not Shown)

CHAPTER 4
EVALUATING SUPERPAVE MIXTURES

4.1 SUPERPAVE SHEAR TESTER (SST)

The SST is used for both permanent deformation and fatigue testing. It is a closed-loop feedback, servo hydraulic system that consists of four major components.

- Testing apparatus includes a reaction frame and a shear table. The shear table imparts shear loads. The reaction frame is extremely rigid, so that precise specimen displacement can be measured without displacements due to frame compliance.
- Test control unit consists of the system hardware and the software. The hardware is the computer and its peripherals as well as the controllers and signal conditioners. The software are the algorithms required to control the testing apparatus and to acquire data.
- Environmental control unit maintains constant temperature and air pressure inside the testing chamber.
- Hydraulic system provides the required force to load specimens according to the required testing conditions.

The control unit and the testing apparatus are connected through linear variable differential transducers (LVDTs). The LVDTs are fixed to the specimen to measure and control specimen deformations (*4*).

4.2 PERFORMANCE TESTS

According to the traffic levels selected for this study, an intermediate analysis is required for the HMA mixtures made using four aggregates. This analysis prediction of permanent deformation requires:

- frequency sweep at constant height (FSCH),
- simple shear at constant height (SSCH), and
- repeated shear at constant stress ratio (RSCSR).

Figure 4. Cox SST Machine Used for HMA Evaluation

The simple shear at constant height and the frequency sweep tests, for Level II analysis, are performed at a specified temperature called effective temperature for permanent deformation, T_{eff}(PD). T_{eff}(PD) is defined as a single test temperature at which some amount of permanent deformation would occur equivalent to that measured by considering each season separately throughout the year. A detailed procedure for obtaining T_{eff}(PD) is included in the Superpave Mix Design Manual for New Construction and Overlays, Report SHRP-A-407 (*45*). For this study, T_{eff}(PD) was calculated as 46˚C for Lubbock, Texas.

Repeated shear at constant stress ratio test is performed at a control temperature, T_c, which is obtained from the T_{eff}(PD) and the traffic level (*45*). Several specimens were prepared and tested at T_c, but test results were questionable, therefore, new specimens were prepared and tested at T_{eff}(PD). The possible reasons for this inadequate test performance at T_c may be because of the high temperatures (Reference 45 required testing at 62.8˚C), and the corresponding stress values selected to perform this test.

The frequency sweep at constant height and the simple shear at constant height tests must be performed at three different asphalt contents:

- Design Asphalt Content- when 4 percent voids are achieved at the design number of gyrations,
- High Asphalt Content- when 3 percent voids are achieved at the design number of gyrations, and
- Low Asphalt Content- when 6 percent voids are achieved at the design number of gyrations.

As stated previously, the river gravel was assumed to be more sensitive to permanent deformation than the limestone or granite, therefore, in order to better characterize its behavior, additional tests were performed which included additional frequency sweep and simple shear at constant height tests at 4˚C and 20˚C, as well as repeated shear test at constant height. For the granite, limestone and river gravel with rounded natural sand mixtures, a simplified intermediate analysis was performed. Table 1 summarizes the specimen properties and test condition for different mixtures.

4.3 SPECIMEN PREPARATION AND INSTRUMENTATION

Specimens were prepared according to the Superpave procedure, as indicated in Chapter III. The specimens had the following general characteristics (Table 13):

Table 13. Specimen Characteristics

Aggregate mass	4700 gm (approx.)
Asphalt cement mass	250 gm (approx.)
Specimen height	125 mm (approx.)
Specimen diameter	150 mm

The specimens were prepared at different asphalt contents and compacted to different air void contents, depending on the test to be performed:

- Frequency sweep at constant height and simple shear at constant height: River gravel specimens were prepared at three different asphalt contents (high, design, and low) and compacted to 7 percent air voids. All other aggregates specimens were prepared with design asphalt content and 7 percent air voids.
- Repeated shear at constant height: Specimens were prepared at the design asphalt content and compacted to 4 percent air voids;
- Repeated shear test at constant stress ratio: Specimens were prepared at high asphalt content and compacted to 3 percent air voids.

The tolerance adopted for compaction was one percentage point for air voids for the frequency sweep and simple shear at constant height. This is the tolerance suggested in ASTM D 4867 M-96 (*42*) for specimens compacted to evaluate moisture sensitivity of asphalt mixtures. In fact, there was no mention of tolerance for compaction in the AASHTO provisional standard TP7 (until AASHTO Standard, Interim April 2001).

The tolerance for repeated shear tests at constant stress ratio and constant height was reduced to 0.5 percentage points, because with low air voids, the mixtures are more sensitive to permanent deformation. Reducing the tolerance was needed to increase accuracy. A summary of air void content of compacted specimens is listed in Appendix B.

Both ends of all test specimens were sawed. These saw cuts were perpendicular to the longitudinal axis of the specimens such that the height of the specimens was 75 ± 2.5 mm. Both ends have to be smooth and mutually parallel within 2 mm. AASHTO TP7-94 (Standard Test Method for Determining Permanent Deformation and Fatigue Cracking Characteristics of Hot Mix Asphalt Using the SST) (*43*) indicates that, for a nominal maximum aggregate size of 19-mm, the height of the specimen is 50 mm. As the objective in this research is to determine the resistance to permanent deformation of different asphalt mixtures while applying shear stresses, a specimen height of 75 mm was adopted to ensure adequate accuracy. With the greater height, the distance between the ends of the saw cut specimen, which are glued to the platens, and the LVDTs was greater, and therefore, the distortion due to their proximity to the glued platens was smaller.

The specimens were glued to the platens using a Superpave gluing device which compresses the specimen between the platens with a 32-kPa load for 30 minutes, while the glue sets up. This gluing device rigidly holds the platens and specimen to ensure that the platen faces are parallel. Test specimens were glued to the platens using Devcon high strength, 5-minute, fast drying epoxy. After setting, the epoxy was subsequently allowed to cure overnight before testing the specimens.

After marking their locations with a template, mounting screws were attached to the side of the specimen with a cyanoacrylate glue with an accelerator, and, once it set up, the horizontal LVDT holders were attached and the LVDTs were installed. The difference in horizontal displacement was measured between the two LVDTs with a gage length of 38.1 mm. The tests were conducted using the Cox & Sons 7000 SHRP Superpave Shear Tester.

The abbreviations used in this report are indicated in Table 14. For example, RGAd20 means River Gravel blend with the gradation passing Above the restricted zone with the Design asphalt content and tested at 20˚C.

Table 14. Abbreviations Used in the Analysis

RG	River Gravel (Partially Crushed)
GR	Granite (Crushed)
LS	Limestone (Crushed)
NS	Rounded Natural Sand (with RG as coarse portion)
T, A, B	Through, Above, or Below the restricted zone, respectively
d, h, l	design, high, or low asphalt content, respectively
4, 20, 46	Test temperature 4, 20, 46˚C

4.4 FREQUENCY SWEEP AT CONSTANT HEIGHT

FSCH is a shear strain controlled test. The test applies a repeated sinusoidal horizontal shear strain with a peak amplitude of approximately ± 0.005 percent and a variable axial stress to maintain constant the height of the specimen. It is the only SST test which uses dynamic loading. The shear strain is applied at different frequencies, including 10, 5, 2, 1, 0.5, 0.2, 0.1, 0.05, 0.02, and 0.01 Hz. The specified strain level was selected during the SHRP program to ensure that the viscoelastic response of the asphalt mixture is within the linear range. This means that the ratio of stress to strain is a function of loading time and not of the stress magnitude. In some cases within this range of frequencies, it has been observed that, at the high and low frequencies, the behavior becomes nonlinear. In Reference 47, it is shown that the dynamic shear modulus (ratio of stress to strain) of asphalt cement is approximately linear between the frequency range of 0.01to 10 Hz.

Before testing, the specimens were preconditioned by applying a controlled sinusoidal shear strain at a frequency of 10 Hz for 100 cycles and a peak to peak amplitude of 0.0001 mm/mm. A detailed description of this test method is given in AASHTO TP7, Procedure E (*43*) and Superpave Asphalt Mixture Analysis: Lab Notes (*46*). The number of cycles applied, sampled cycles, and data points per cycle for the standard procedure were increased in order to increase accuracy of the results (Table 15).

The axial deformation, shear deformation, axial load, and shear load at each of the ten different frequencies were recorded. The data obtained from the FSCH test was used to calculate the material properties: dynamic shear modulus (with its real and imaginary parts), phase angle, and slope of the dynamic shear modulus versus frequency on a log-log scale.

4.4.1 Dynamic Shear Modulus (G^*) and m-values

Dynamic shear modulus is defined as the ratio of peak stress to peak strain at a given frequency. It is a measure of total stiffness of asphalt mixtures. It consists of two parts:

- G', real part or shear storage modulus, elastic behavior.
- G'', imaginary part or loss storage modulus, viscous behavior.

Table 15. Frequencies, Number of Cycles Applied, and Data Points per Cycle (FSCH)

Frequency (Hz)	Total No. of Cycles	No. of Cycles Sampled	Data Points Per Cycle
10	50	10	60
5	50	10	60
2	20	10	60
1	20	10	60
0.5	10	10	60
0.2	10	10	60
0.1	10	10	60
0.05	5	1	60
0.02	5	1	60
0.01	5	1	60

In the Superpave distress model for permanent deformation, the m-value (slope of the dynamic shear modulus versus frequency on a log-log scale) is used to calculate plastic strain accumulation during N number of load applications. Validity of the model is based on the assumption that the higher the test temperature, the higher the m-value will be; and the higher the m-value, the greater the permanent deformation will be. In Tables 16 and 17 are listed the different m-values for the different asphalt mixtures. As indicated previously, in some cases at extreme frequencies, the behavior was not linear, and, therefore, those values were not considered in obtaining the slope.

In order to compare the asphalt cement rheology with that of asphalt concrete, the complex shear modulus and shear phase angle of unaged asphalt were determined using the DSR machine at different frequencies and at three different temperatures (7, 20, and 46˚C). The test results are provided in Appendix A. Figure 5 exhibits the complex shear modulus (G*) plotted on log-log chart against frequencies tested at three different temperatures. Complex

Table 16. Parameter m for River Gravel Mixtures

Mixture Type	m-value at 4˚C	m-value at 20˚C	m-value at 46˚C
RGTd (average)	0.278	0.407	0.260
RGTh (average)	0.287	0.404	0.270
RGTl (average)	0.242	0.342	0.304
RGAd (average)	0.265	0.427	0.283
RGAh (average)	0.393	0.422	0.226
RGAl (average)	0.261	0.339	0.240
RGBd (average)	0.289	0.394	0.210
RGBh (average)	0.321	0.459	0.152
RGBl (average)	0.283	0.419	0.254

Table 17. Parameter m for Granite, Limestone, and Rounded Natural Sand Mixtures

Aggregate Type	Mixture Type	m-value at 46˚C
Crushed Granite	GRTd (average)	0.362
	GRAd (average)	0.330
	GRBd (average)	0.314
Crushed Limestone	LSTd (average)	0.440
	LSAd (average)	0.419
	LSBd (average)	0.337
Rounded Natural Sand	NSTd (average)	0.355
	NSAd (average)	0.396
	NSBd (average)	0.475

shear modulus of the binder increases with decreasing temperatures and increasing frequencies. The slope for the asphalt binder increases with increasing temperatures and approaches a value of 1.0 indicating its tendency to behave as a Newtonian fluid at high temperatures.

From the complex shear modulus versus frequency chart, the m values of asphalt cement were calculated as 0.613, 0.739, and 0.929 at 7, 20, and 46˚C temperature, respectively.

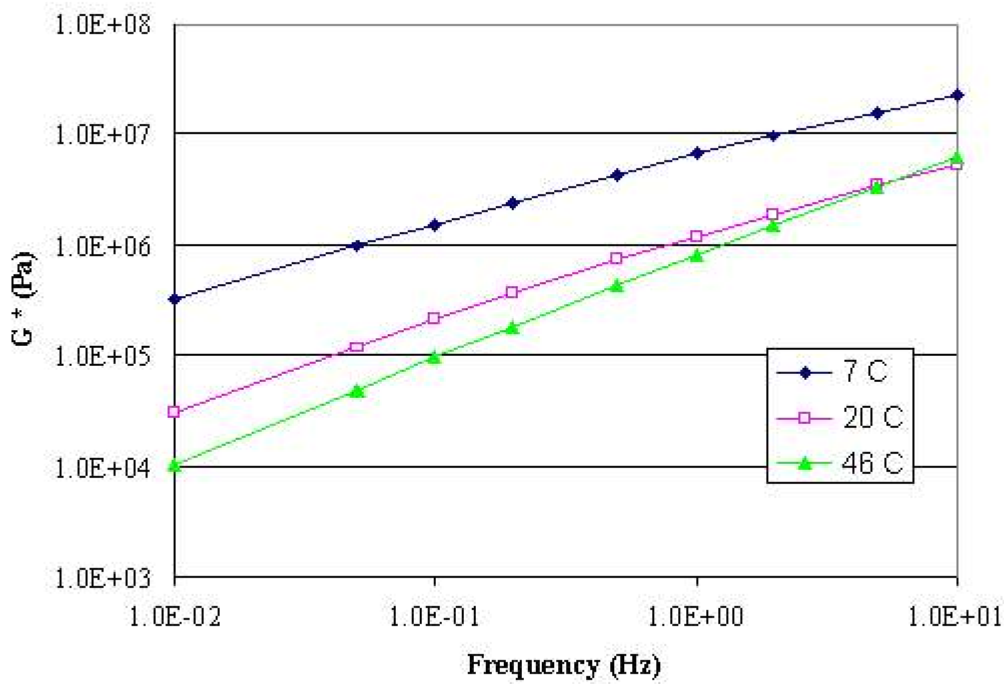

Figure 5 Complex Shear Modulus of Asphalt Cement versus Frequency

In Figures 6 through11, the complex (dynamic) shear modulus of different mixtures are plotted against the testing frequencies on log-log chart. The following conclusions appeared warranted form the graphs:

- Based on Figures 6 to 11 and Tables 16 and 17, the asphalt mixture dynamic shear modulus, G^*, increases when the frequency increases as well as when the temperature decreases (Note: only river gravel mixtures were tested at three different temperatures). But the dynamic shear modulus slope (m-value) in the asphalt mixtures behaves different than in the asphalt cement. The slope in the asphalt mixtures increases with increasing temperatures; it reaches a peak and then decreases. According to Zhang (*47*), this is because the rheology of asphalt mixtures at high temperature is predominantly affected by the aggregate instead of the asphalt binder. On the other hand, at low temperatures, asphalt cement and mixtures show a similar trend, indicating that influence of the asphalt cement in the rheology of the asphalt mixtures is more prominent at low temperatures.

- Validity of the Superpave performance model is based on the assumption that higher test temperature indicate higher m-values and higher m-values indicate greater permanent deformation. This assumption is not correct, because the m-value increases with increasing temperatures, reaches a peak between 4˚C and 46˚C, and then decreases. The 1993 Superpave performance model might yield unreasonable predictions if the parameter m is used as the slope of the permanent deformation performance model equation. As the parameter S of the performance model equation is not related to the parameter m, it should be determined from a repeated load test (*47*).

- Comparing the m-values of the mixtures with different asphalt contents and at different temperatures (Table 16) shows that the m-value is greater with high asphalt contents at low temperatures. But at high temperatures, the m-value is higher at low asphalt contents. Therefore, at low temperatures, the greater the asphalt content, the more sensitive to loading times the asphalt mixture will be, but not at high temperatures.

- Two different "rankings" to characterize the m-values of the asphalt mixtures were examined. In the first, the different blends (TRZ, ARZ, and BRZ) at the same temperature and asphalt content were compared (Table 18). In the second, the

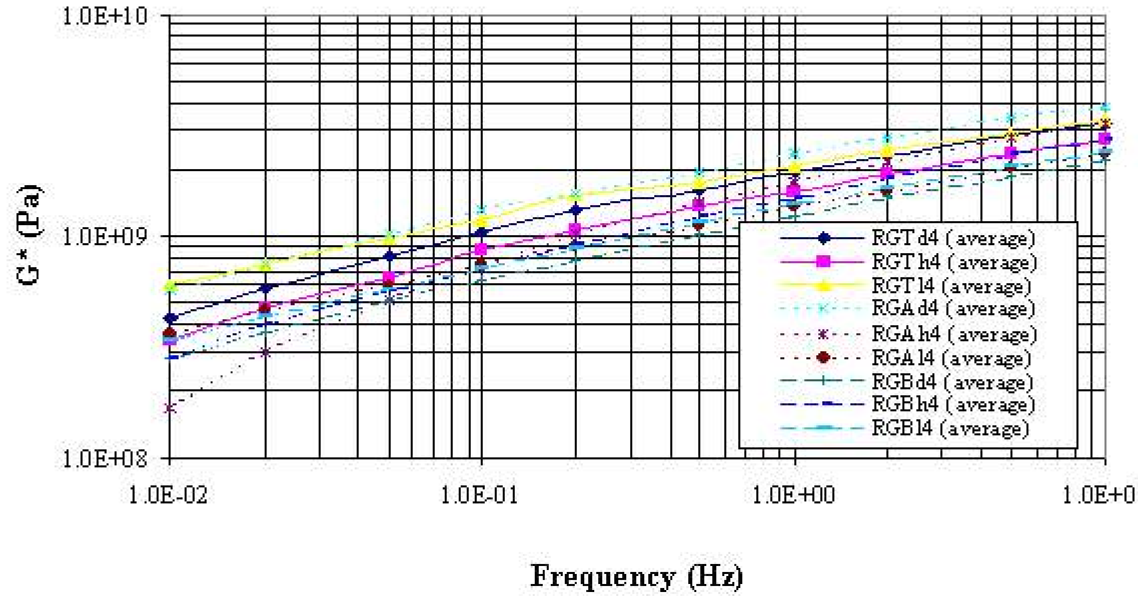

Figure 6 River Gravel, G* versus Frequency at 4˚C

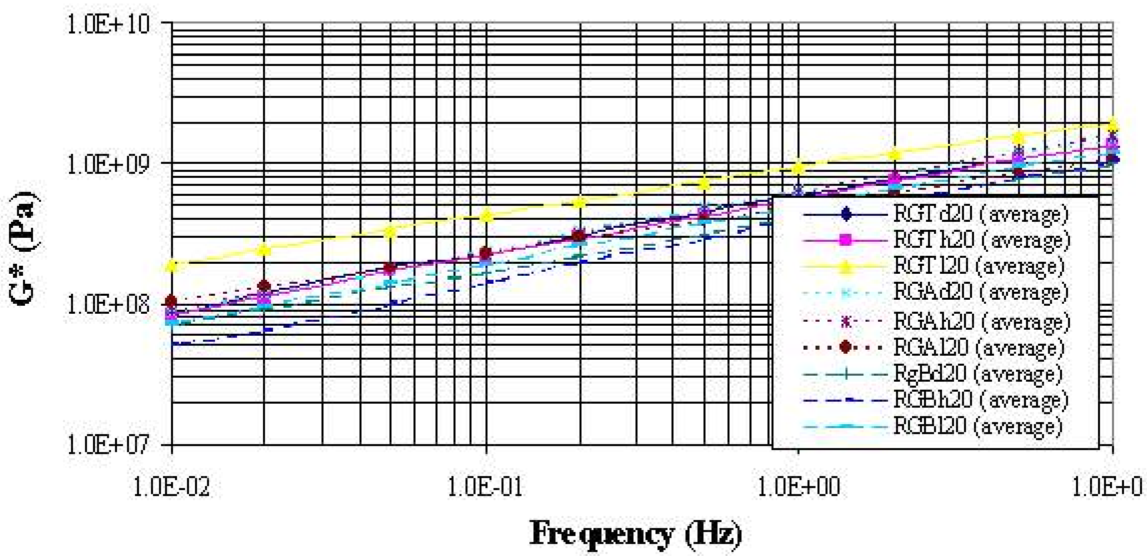

Figure 7 River Gravel, G* versus Frequency at 20˚C

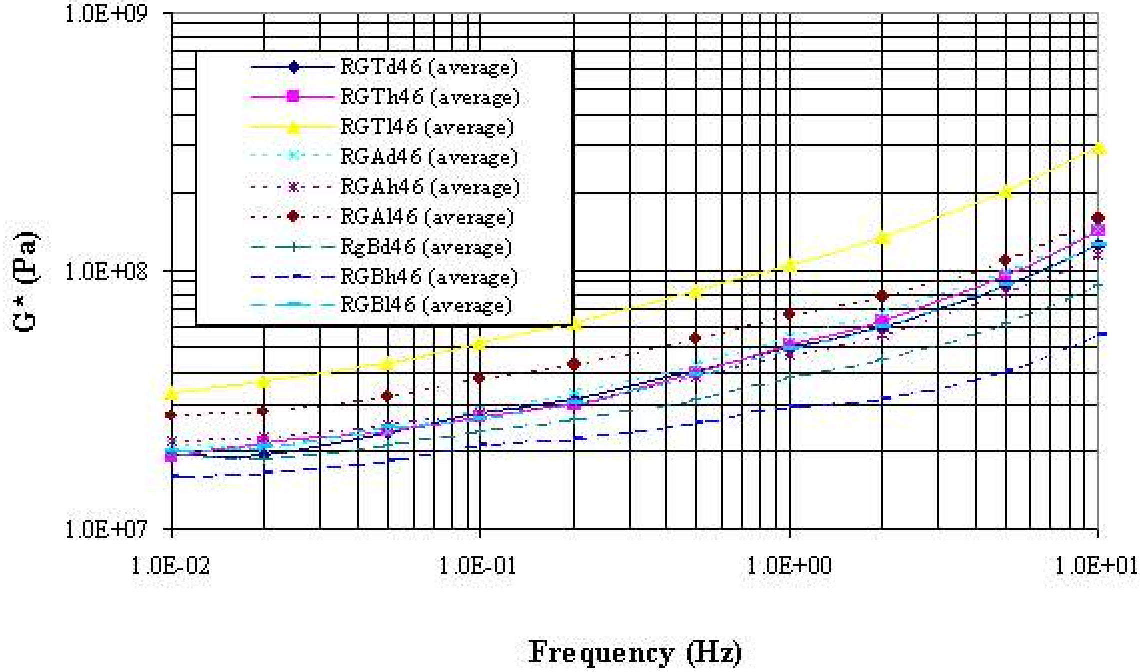

Figure 8 **River Gravel, G* versus Frequency at 46˚C**

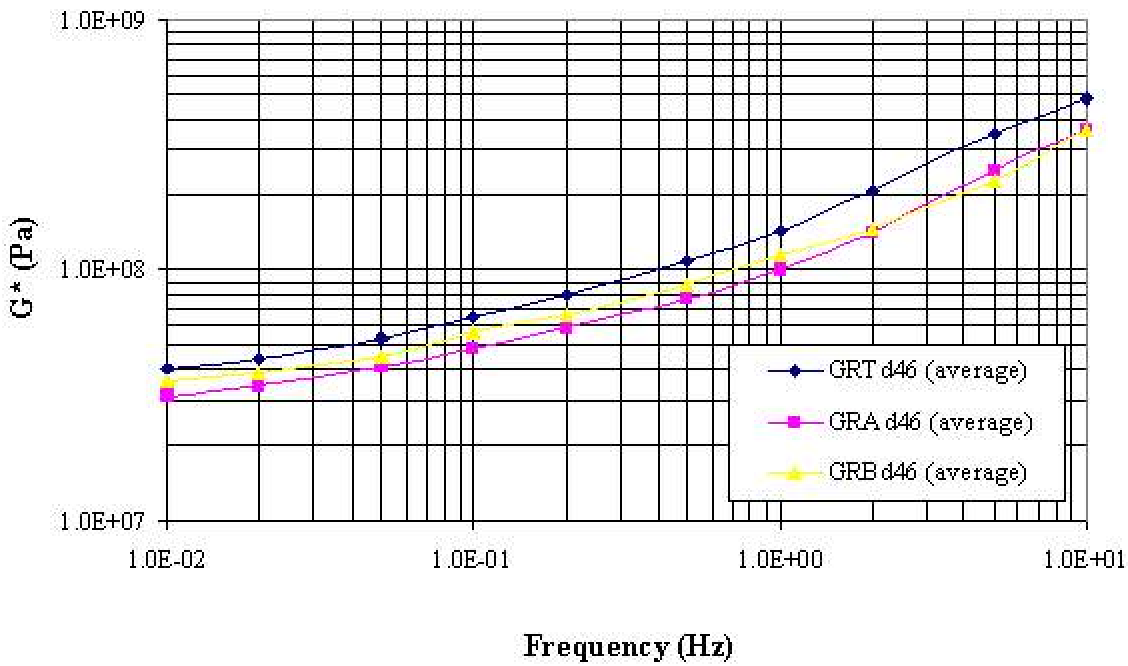

Figure 9 **Granite, G* versus Frequency at 46˚C**

different blends were compared at the three different temperatures (Table 19). It is observed that, at low temperatures, greater m-values correspond to the blends passing below the restricted zone, but, at high temperatures, greater m-values correspond to the blends passing through and above the restricted zone. Higher m-values indicate more sensitivity to loading time for the asphalt mixture.

- G* is highly temperature and frequency dependent. The G* values in the asphalt mixtures at 46˚C, the most concern temperature in permanent deformation ranges from 1.57×10^{7} to 2.96×10^{8} Pa in river gravel (ratio 18.8), from 3.14×10^{7} to 4.86×10^{8} Pa in granite (ratio 14.16), from 1.52×10^{7} to 3.43×10^{8} Pa in limestone (ratio 22.5), and from 8.13×10^{6} to 1.29×10^{8} Pa in rounded natural sand mixture (ratio 15.9). Based on these values, the aggregate most sensitive to gradation is limestone and the least sensitive is the granite. The stiffest aggregate is granite and the least is river gravel with rounded natural sand. That is, mixtures containing rounded natural sand were most prone to rutting.
- Higher G* values indicate more resistance to HMA permanent deformation. Comparing the G* curves for the different blends and temperatures shows that, for the river gravel mixtures, greater G* are obtained with low asphalt contents and lower G* values are achieved with high asphalt contents. Further, blends below the restricted zone tend to have lower G* values. From the data obtained with the granite mixtures, the stiffest blends are those which pass through the restricted zone. For the limestone mixtures, no conclusion regarding which one is stiffer can be achieved; results depend on frequency (see Table 20). G* for the ARZ mixture containing rounded natural sand is higher than that of the TRZ and BRZ mixtures.

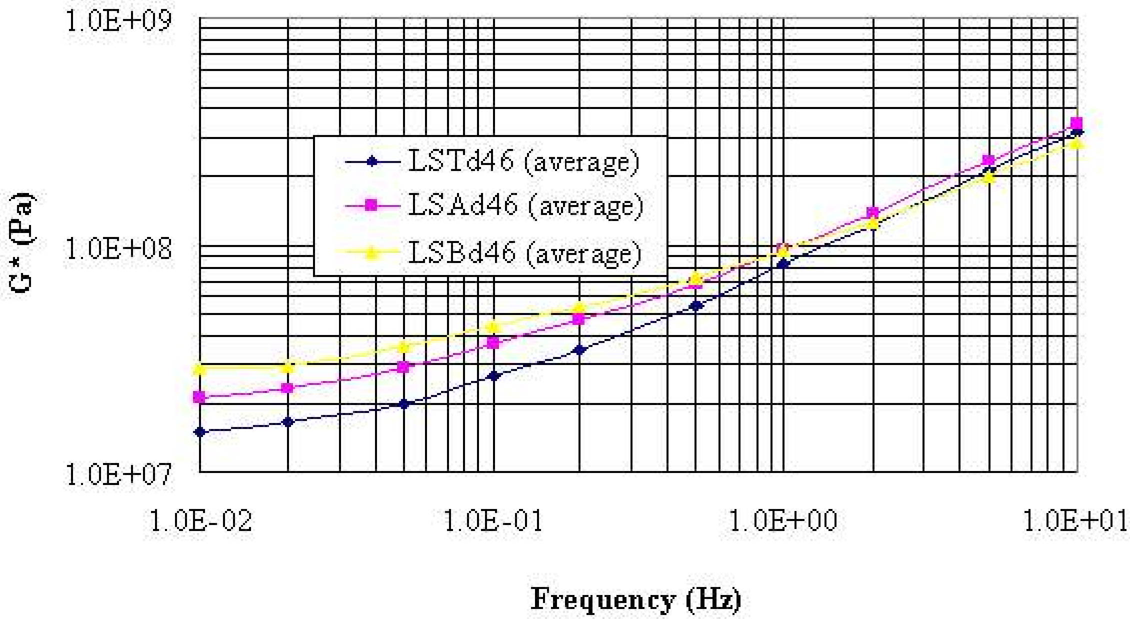

Figure 10 Limestone, G* versus Frequency at 46˚C

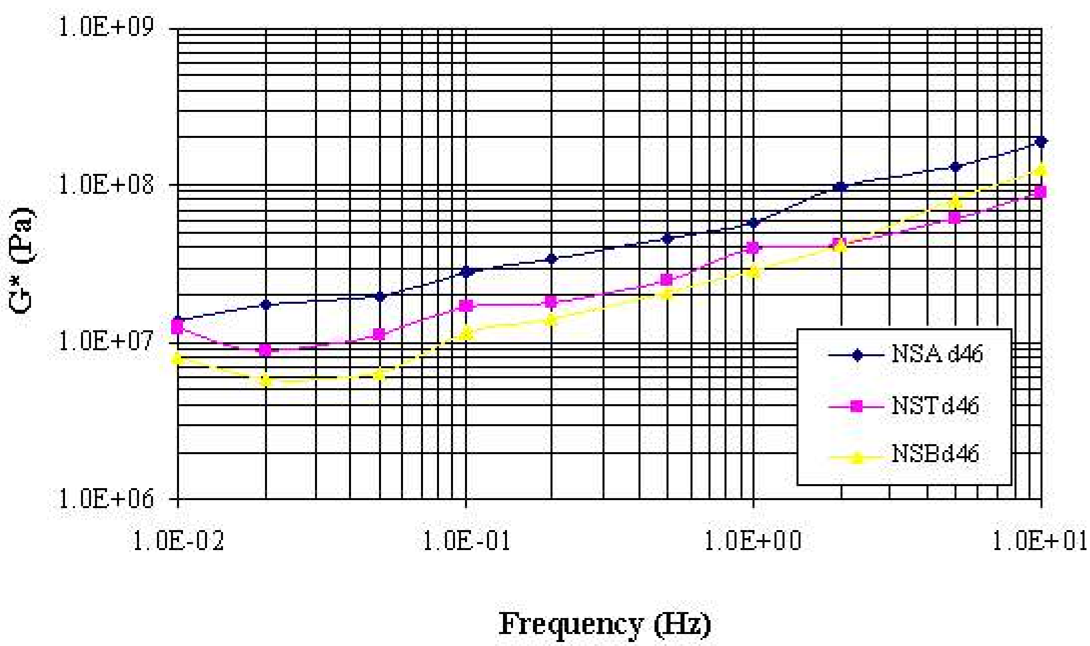

Figure 11 River Gravel + Rounded Natural Sand, G* versus Frequency at 46˚C

Table 18. Comparative Ranking of Asphalt Mixtures Considering m-values

m-value	River Gravel			Granite	Limestone	River Gravel + Natural Sand
	High a/c	Design a/c	Low a/c	Design a/c	Design a/c	Design a/c
4˚C	A>B>T	B>T>A	B>A>T	-	-	-
20˚C	B>A>T	A>T>B	B>T>A	-	-	-
46˚C	T>A>B	A>T>B	T>B>A	T>A>B	A>T>B	B>A>T

Table 19. Ranking of Asphalt Mixtures Considering m-values

m-value	1st (worst)	2nd	3rd	4th	5th	6th	7th	8th	9th (best)
RG at 4˚C	Ah	Bh	Bd	Th	Bl	Td	Ad	Al	Tl
RG at 20˚C	5h	Ad	Ah	Bl	Td	Th	BB	Tl	Al
RG at 46˚C	Tl	Ad	Th	Td	Bl	Al	Ah	Bd	Bh
GR at 46˚C	Td	Ad	Bd						
LS at 46˚C	Ad	Td	Bd						
NS at 46˚C	Bd	Ad	Td						

(*) 1st indicates the greatest m-value and 9th indicates the smallest one.

Table 20. Ranking Asphalt Mixtures Considering G*

G* value	1st (best)	2nd	3rd	4th	5th	6th	7th	8th	9th (worst)
RG at 4˚C	Ad	Tl	Td	*	*	*	Bh	Bd	Ah
RG at 20˚C	Tl	*	*	*	*	*	*	Bd	Bh
RG at 46˚C	Tl	Al	*	*	*	*	*	Bd	Bh
GR at 46˚C	Td	Bd	Ad	-	-	-	-	-	-
LS at 46˚C	*	*	Td	-	-	-	-	-	-
NS at 46˚C	Ad	*	*	-	-	-	-	-	-

* indicates that there is no clear classification.

\- indicates there is no data available.

4.4.2 Shear Phase Angle

Shear phase angle is defined as the lag time between the application of a stress and the corresponding strain.

Figures 12 through 18 show shear phase angle versus frequency for the asphalt cement binder and the HMA mixtures tested. Based on these findings, the following conclusions appear warranted:

- The shear phase angle in the *asphalt cement* is highly temperature and frequency dependent. The phase angle values ranged from 45 to 89 degrees. At the high temperature (46˚C) and low frequency (0.01 Hz), the phase angle is very near 90 degrees. At the high frequency (10 Hz) and low temperature (7˚C), the phase angle was very near 45 degrees (i.e., elastic and viscous components of the asphalt cement are similar).
- In the HMA mixtures, the shear phase angle decreases with increasing frequencies and decreasing temperatures for intermediate (20˚C) and low temperatures (4˚C), as with the asphalt cement. But at high temperatures, the shear phase angle

decreases with decreasing frequencies (opposite from the asphalt cement). In the HMA mixtures, the slope of phase angle as a function of frequency at intermediate and low temperatures is smaller than at high temperatures, which means that it is less dependent on time of loading at low and intermediate temperatures than at high temperatures (opposite from the asphalt cement).

- At low temperatures, the shear phase angle in the HMA mixtures for the frequencies studied, ranged from 22 to 45 degrees; at intermediate temperatures, from 26 to 56 degrees; and at high temperatures, from 32 to 68 degrees. Therefore, at high temperatures, the asphalt mixture exhibit more viscous behavior than at low temperatures, but it will be highly dependent on loading time. At high temperatures, the HMA mixture exhibited predominantly elastic behavior at low frequencies and a viscous behavior at high frequencies. For the HMA mixtures at low and intermediate temperatures, the elastic shear modulus component is generally greater than the viscous component (mainly at high frequencies).

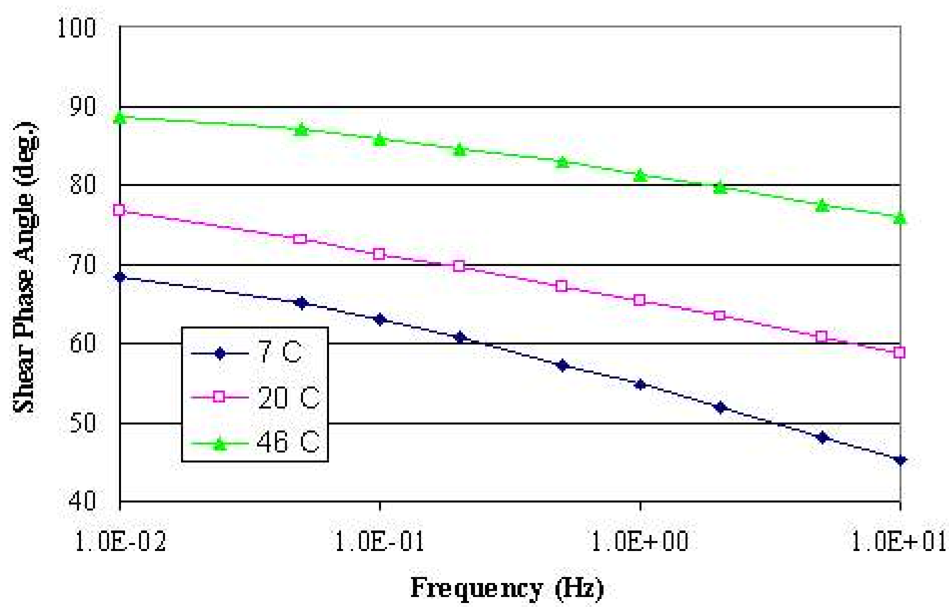

Figure 12 Shear Phase Angle of Asphalt versus Frequency

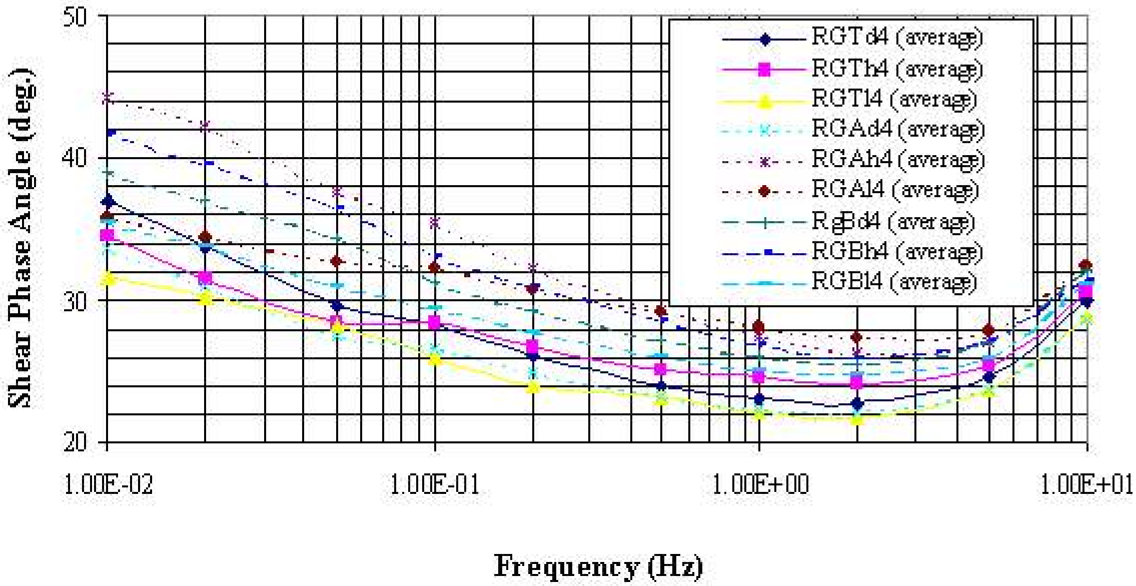

Figure 13 River Gravel, Phase Angle versus Frequency at 4˚C

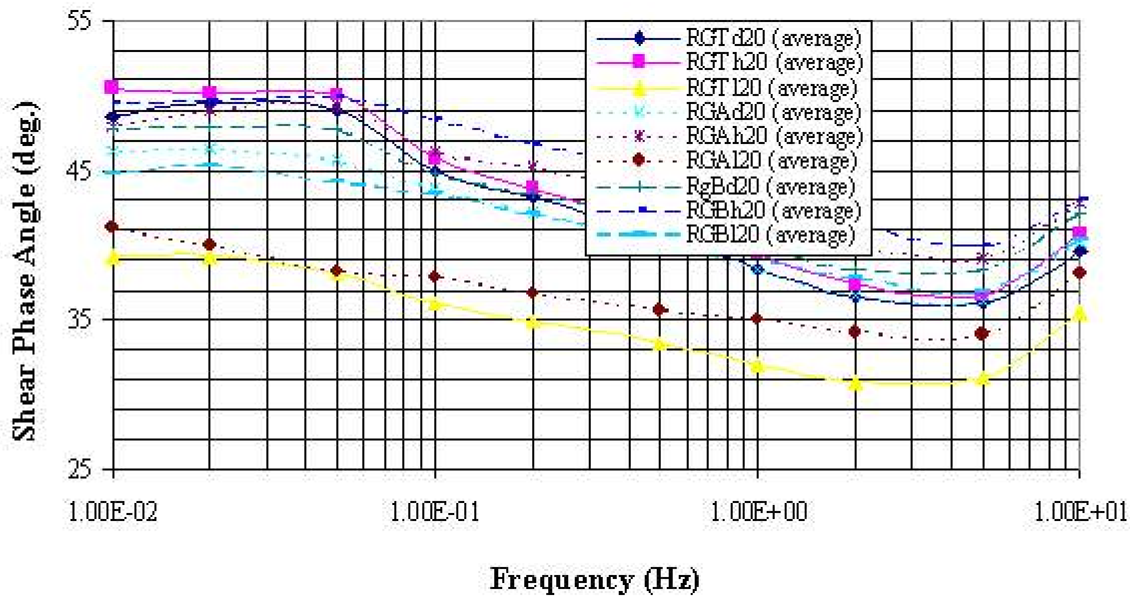

Figure 14 River Gravel, Phase Angle versus Frequency at 20˚C

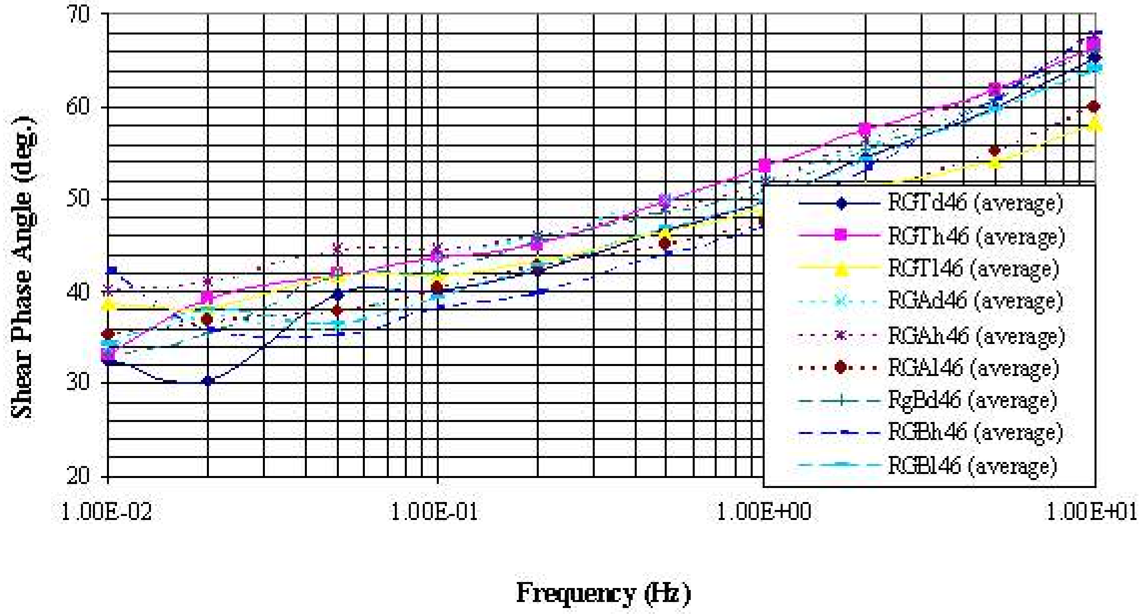

Figure 15 River Gravel, Phase Angle versus Frequency at 46˚C

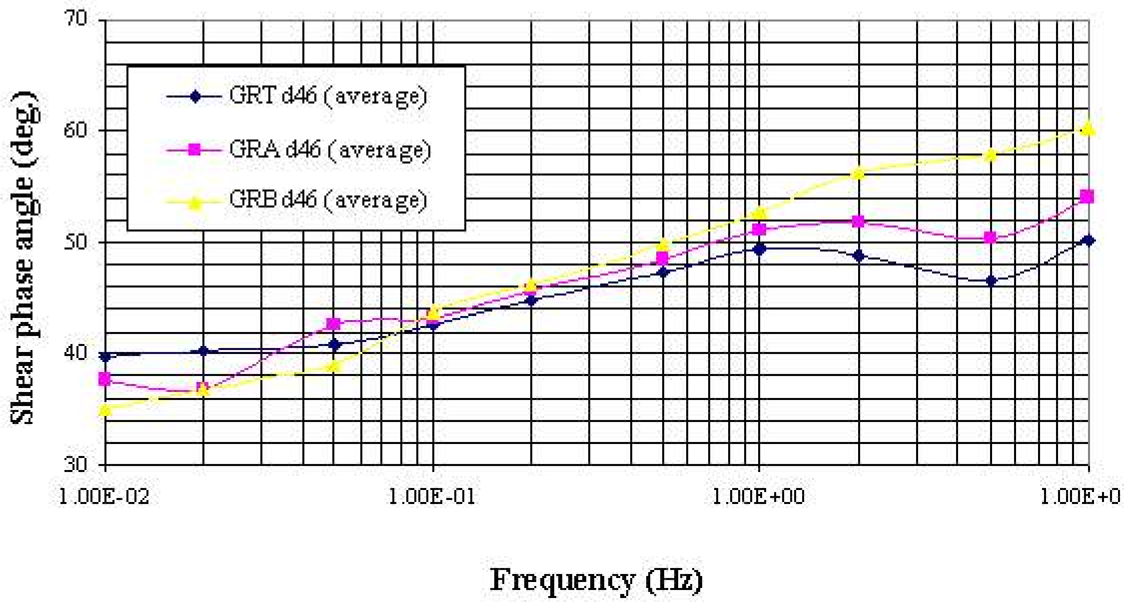

Figure 16 Granite, Phase Angle versus Frequency at 46˚C

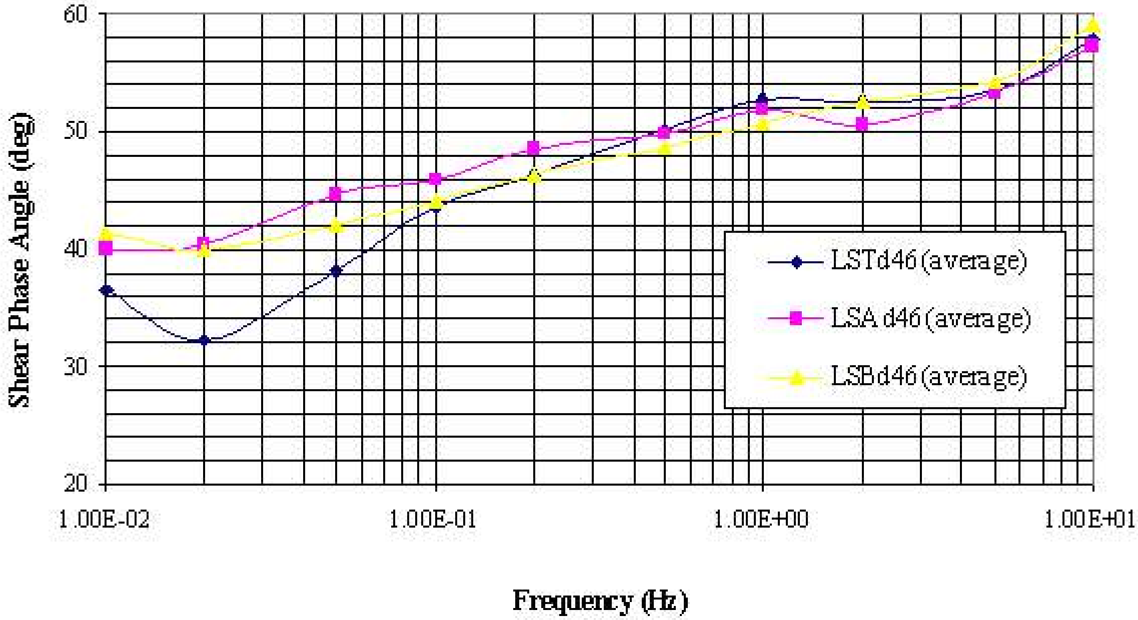

Figure 17 Limestone, Phase Angle versus Frequency at 46˚C

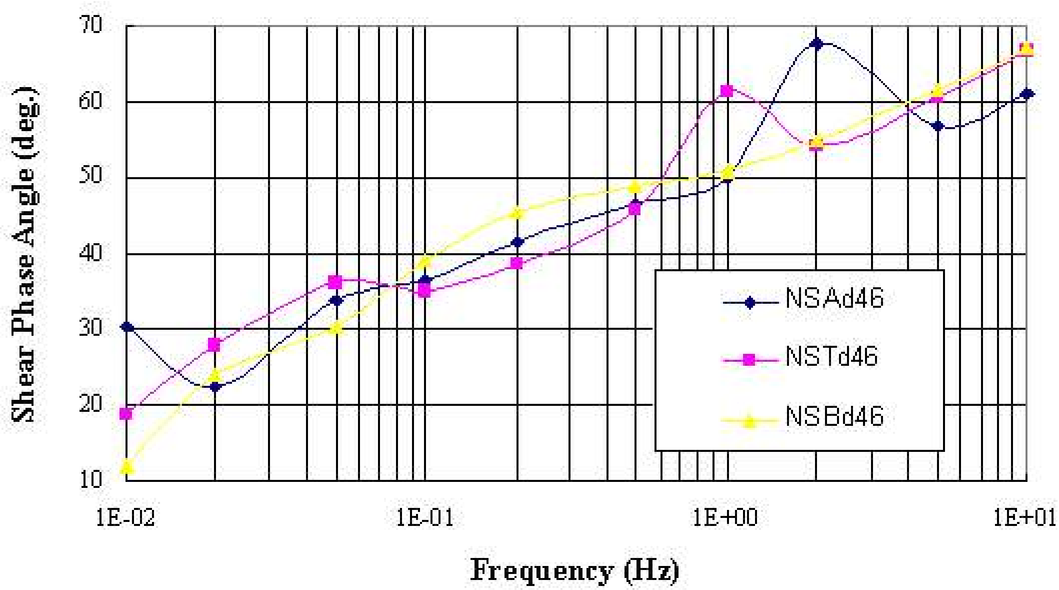

Figure 18 River Gravel + Rounded Natural Sand Phase Angle versus Frequency at 46˚C

- At low and intermediate temperatures, the shear phase angle of the asphalt cement and asphalt mixture follow similar trends with the different frequencies because this property of the asphalt mixture is primarily affected by the binder at these temperatures. But, at high temperatures, asphalt cement and asphalt mixtures exhibit opposite trends because, at high temperatures, the shear phase angle of the mixture is more affected by the aggregate (mainly at low frequencies).
- Lower phase angles indicate more resistance to permanent deformation of the asphalt mixture. Figures 13 through 15 show that high asphalt contents give higher phase angles, and low asphalt contents give lower phase angles. See also Table 21.
- Regarding the restricted zone, no clear trends from the shear phase angle can be obtained.
- The range of shear phase angle values in the HMA mixtures at 46˚C is from 32 to 68 degrees in the river gravel mixtures (ratio 1.75), from 35 to 60 degrees in the granite mixtures (ratio 1.50), from 36 to 58 degrees in the limestone mixtures (ratio 1.44), and from 12 to 67 degree in the gravel plus rounded natural sand mixtures (ratio 5.6). The test results from the rounded natural sand mixture are questionable. Even at the same frequency (0.01 Hz), the phase angle varies 12 to 30 degrees among mixtures of different gradations, which is very unusual. Again, these data indicate that the river gravel mixtures are the most susceptible to permanent deformation (the phase angle values and its range are greater).

4.4.3 G*/sin δ Ratio

For rutting resistance to be contributed by asphalt cement, a high complex modulus, G*, and low phase angle, δ, are both desirable. The ratio used in Superpave to determine the resistance to permanent deformation by asphalt cements is G*/sin δ at different temperatures. The greater this ratio, the more resistant to permanent deformation the asphalt cement will be.

Table 21. Ranking Asphalt Mixtures Considering Phase Angle

δ value	1st (worst)	2nd	3rd	4th	5th	6th	7th	8th	9th (best)
RG at 4˚C	Ah	Bh	Al	*	*	*	*	Ad	Tl
RG at 20˚C	Bh	Th	Ah	*	*	*	*	Al	Tl
RG at 46˚C	Bh	Th	*	*	*	*	Tl	Al	Bh
GR at 46˚C	*	*	Bd	-	-	-	-	-	-
LS at 46˚C	*	*	*	-	-	-	-	-	-
NS at 46˚C	*	*	*	-	-	-	-	-	-

* indicates that there is no clear classification.

- indicates there is no data available.

A comparison of the G*/sin δ values for the different asphalt mixtures was conducted. Figures19 through 25 show G*/sin δ as a function of test frequency for the asphalt cement and different mixtures. Comparing these figures has produced the following conclusions:

- For the river gravel mixtures, the blends through and above the restricted zone were the more resistant to permanent deformation, at the temperatures tested, and the blends below the restricted zone were more prone to rutting (Table 23). For the granite mixtures, the blends through the restricted zone were most resistant to permanent deformation. For the limestone mixtures, no clear trends were observed. Note that G*/sin δ depends on the frequency selected.
- Lower asphalt contents indicate more resistance to permanent deformation of the HMA mixture. The graphs, indicate that low asphalt contents give higher G*/sin δ values than higher asphalt contents. For a constant air void content in all the blends, higher asphalt contents generally yield higher susceptibility to permanent deformation. See Table 22.

- The rankings provided in Table 22 are very similar to those in Table 20 (rankings of the asphalt mixtures considering G*). This indicates that G* has a much greater effect than sin δ in the ratio G*/sin δ.

Table 22. Ranking Asphalt Mixtures Considering G*/sin δ

G*/sin δ	1st (best)	2nd	3rd	4th	5th	6th	7th	8th	9th (worst)
RG at 4˚C	Ad	Tl	Td	Th	*	*	*	*	Bd
RG at 20˚C	Tl	Ad	Al	*	*	*	*	Bd	Bh
RG at 46 ˚C	Tl	Al	*	*	*	*	*	Bd	Bh
GR at 46˚C	Td	Bd	Ad	-	-	-	-	-	-
LS at 46˚C	*	*	Td	-	-	-	-	-	-
NS at 46˚C	Ad	*	*	-	-	-	-	-	-

* indicates that there is no clear classification

- indicates there is no computed data

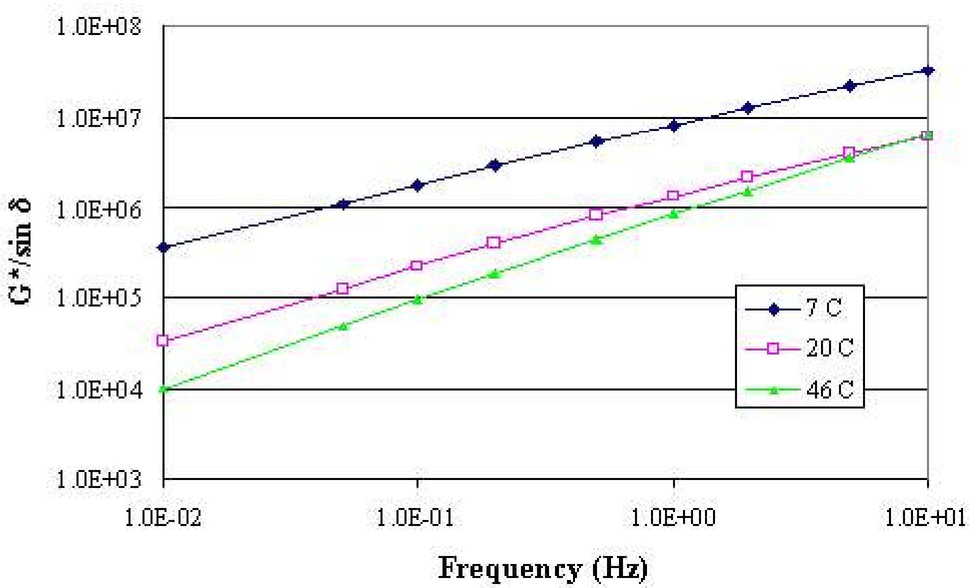

Figure 19 G*/sin δ versus Frequency for Asphalt Cement

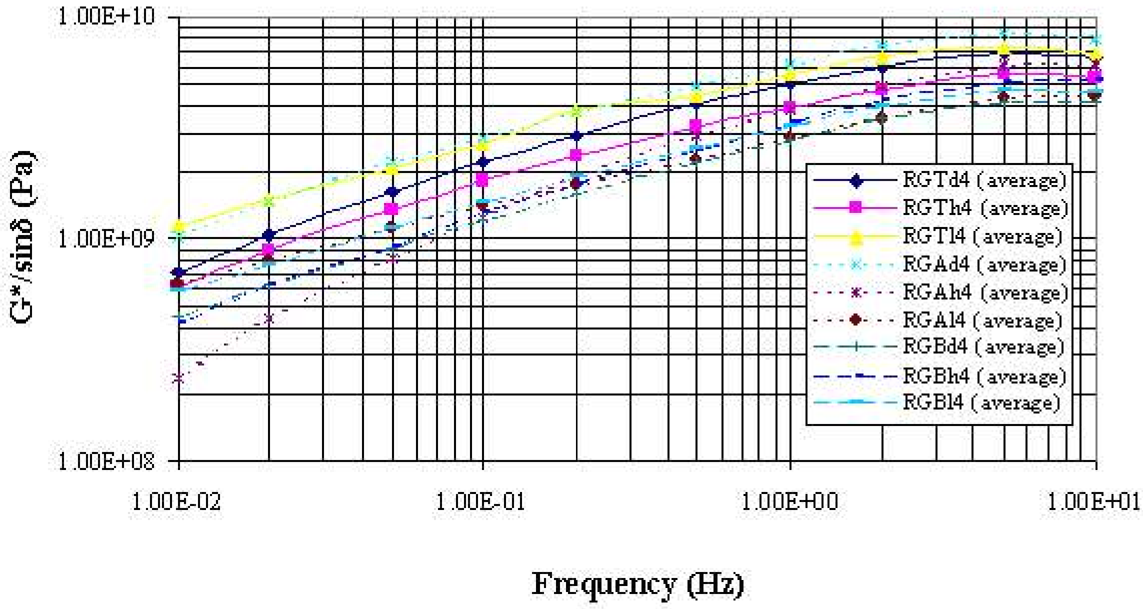

Figure 20 River Gravel, G*/sin δ versus Frequency at 4˚C

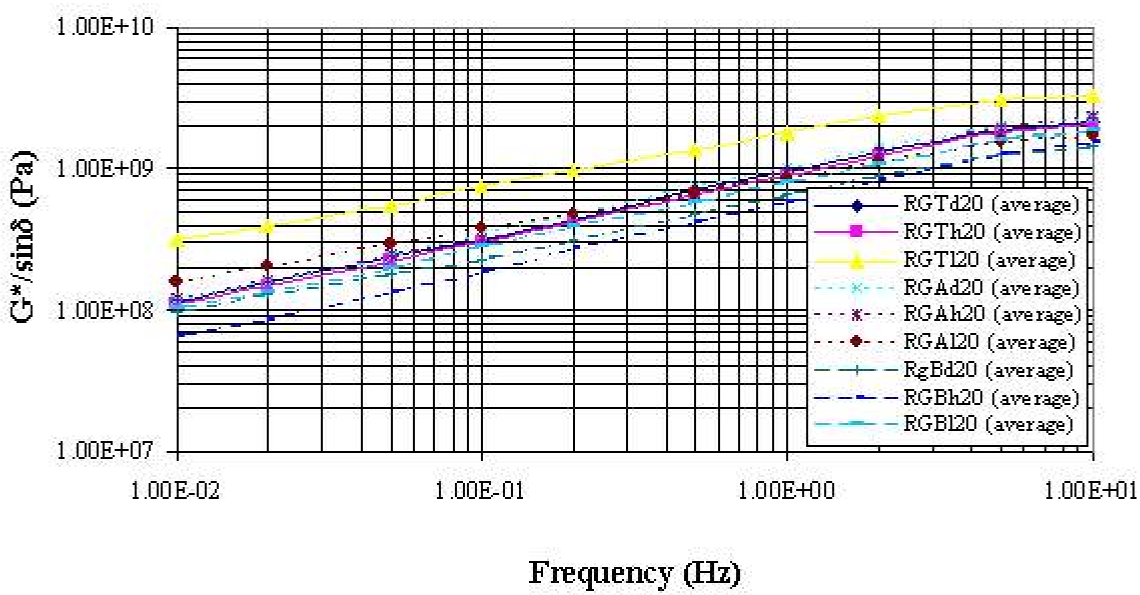

Figure 21 River Gravel, G*/sin δ versus Frequency at 20˚C

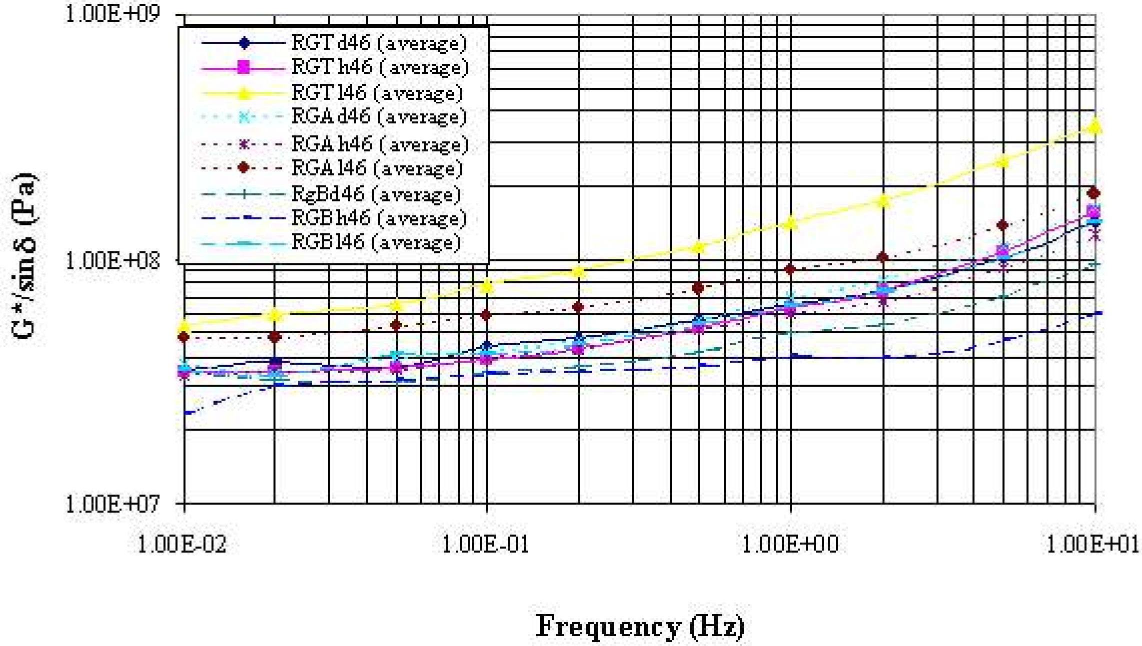

Figure 22 **River Gravel, G*/sin δ versus Frequency at 46˚C**

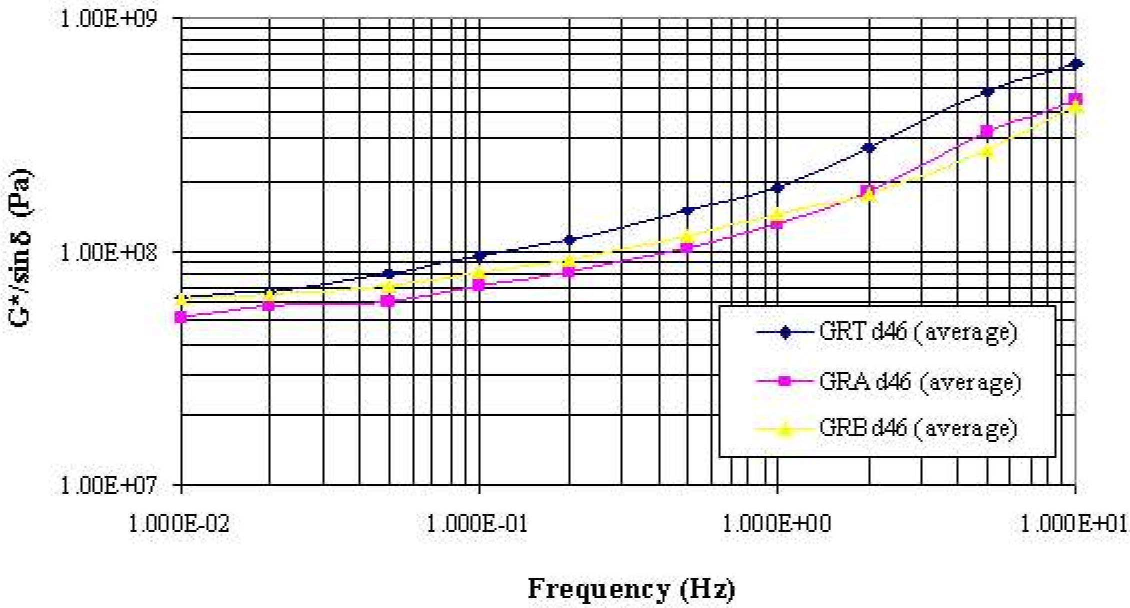

Figure 23 **Granite, G*/sin δ versus Frequency at 46˚C**

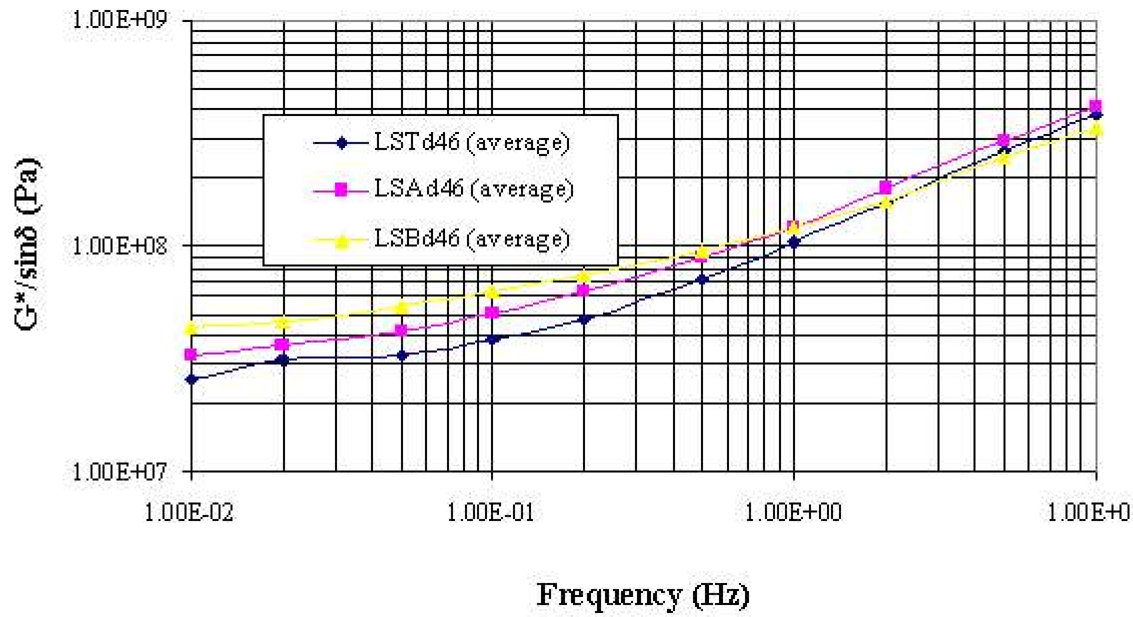

Figure 24 **Limestone, G*/sin δ versus Frequency at 46˚C**

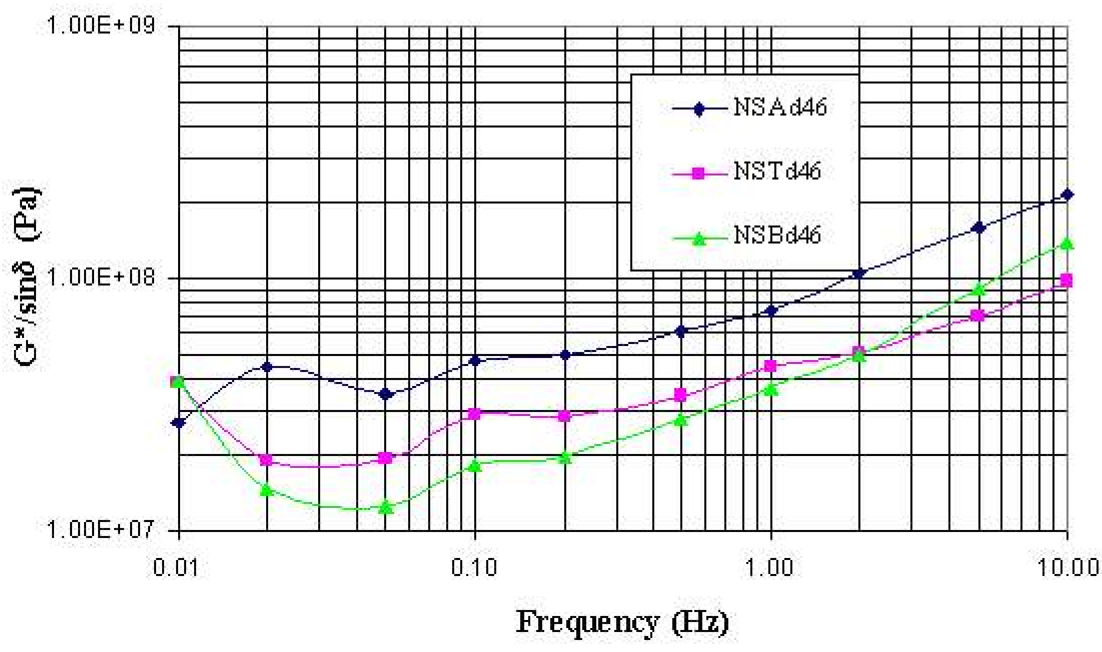

Figure 25 **River Gravel + Rounded Natural Sand, G*/sinδ versus Frequency at 46˚C**

4.5 SIMPLE SHEAR AT CONSTANT HEIGHT

The SSCH is a shearing stress controlled test. The test is performed at different stress levels depending on the test temperature. Shear stress was applied at a rate of 70 ± 5 kPa per second up to the stress level indicated in Table 23. The stress level was maintained for 10 seconds, and, afterwards, it was reduced to 0 kPa at a rate of 25 kPa/s. As the specimen is sheared, it tries to dilate (increase in height). A controlled axial load is applied to maintain a constant specimen height.

All specimens were preconditioned for 100 cycles with a shear stress having a peak magnitude of approximately 7 kPa. Each cycle has a duration of 0.7 seconds, consisting of a 0.1-second loading period followed by a 0.6-second rest period in a haversine wave form.

Table 23. Stress Levels Applied in the SSCH Test

Test Temperature	Shear Stress, kPa
4˚C	345
20˚C	105
46˚C	35

The SSCH was performed after the frequency sweep at constant height using the same specimens. The tests at the lowest temperatures were performed first. A detailed description of this test method is provided in AASHTO TP7, Procedure D (*43*) and Superpave Asphalt Mixture Analysis: Lab Notes (*46*).

Material properties obtained from this test are maximum shear strain, plastic and elastic shear strain, and permanent deformation after the first load application.

A summary of the blends and temperatures at which the SSCH test was performed is given in Table 1.

4.5.1 Maximum Shear Strain

The maximum shear stress was the same for all SSCH specimens tested at a given temperature. Therefore, it is possible to compare the expected performance of the different mixtures based on the maximum shear strain resulting from the applied shear stress at a given temperature. Mixtures exhibiting low strains are expected to be more resistant to permanent deformation. From the rankings in Table 24 and Figure 26, the following conclusions are given:

- Blends more susceptible to permanent deformation are those which pass below the restricted zone for river gravel and limestone and those which pass above the restricted zone for granite. Blends more resistant to permanent deformation are those which pass above the restricted zone for river gravel and limestone and those which pass through the restricted zone for granite.
- Mixtures with higher asphalt contents exhibited more susceptibility to permanent deformation.
- Performance of the HMA mixtures at 46˚C and 20˚C was similar, but, at 4˚C, they were different (Table 24).
- Both River gravel blends exhibited greater maximum shear strain than granite or limestone blends, thus indicating more susceptibility to rutting (Figure 26).

Table 24. Ranking Asphalt Mixtures Considering Maximum Shear Strain

Max. Shear Strain	1st (worst)	2nd	3rd	4th	5th	6th	7th	8th	9th (best)
RG at 4˚C	Ah	Al	Bd	Th	Bh	Bl	Td	Ad	Tl
RG at 20˚C	Bh	Bd	Bl	Ah	Td	Al	Ad	Th	Tl
RG at 46˚C	Bh	Bd	Bl	Th	Td	Ah	Ad	Al	Tl
GR at 46˚C	Ad	Bd	Td	-	-	-	-	-	-
LS at 46˚C	Bd	Td	Ad	-	-	-	-	-	-
NS at 46˚C	Bd	Ad	Td	-	-	-	-	-	-

- indicates there is no computed data.

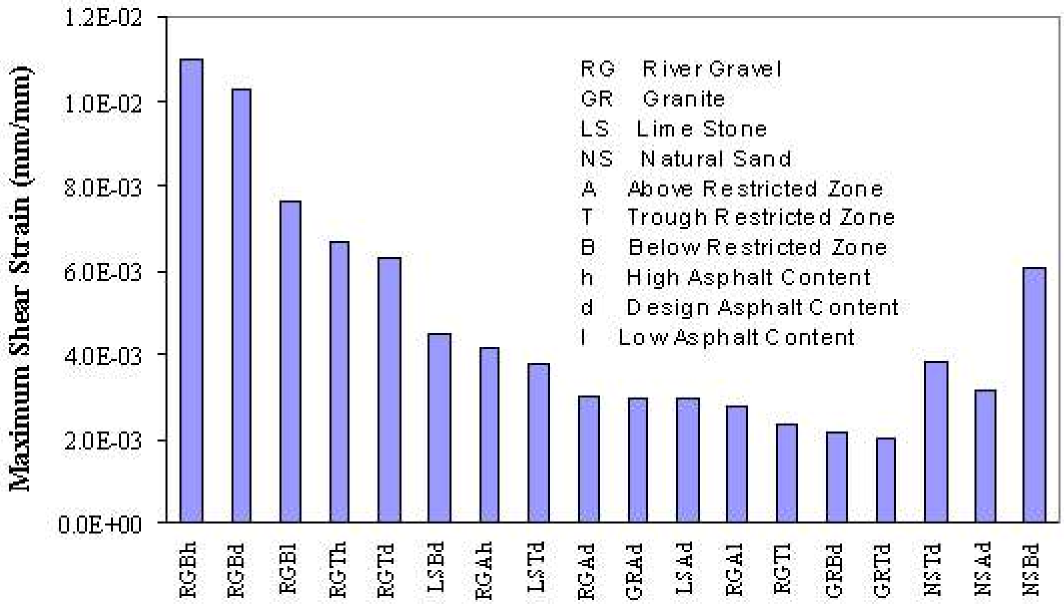

Figure 26 Maximum Shear Strain at 46˚C for Different Mixtures

4.5.2 Permanent Shear Strain

As indicated previously, the maximum shear stress was the same for all SSCH specimens tested at a given temperature. Therefore, it is possible to compare the expected performance of the different mixtures based on the permanent shear strain resulting from the applied shear stress. Mixtures with low permanent strains are expected to be more resistant to permanent deformation. From the rankings presented in Table 25 and Figure 27, the same conclusions as those for the maximum shear strain are supported.

Table 25. Ranking Asphalt Mixtures Considering Permanent Shear Strain

Max. Shear Strain	1st (worst)	2nd	3rd	4th	5th	6th	7th	8th	9th (best)
RG at 4 ˚C	Ah	Al	Bd	Bh	Th	Bl	Td	Ad	Tl
RG at 20 ˚C	Bh	Bd	Bl	Ah	Td	Al	Ad	Th	Tl
RG at 46 ˚C	Bh	Bd	Bl	Th	Td	Ah	Ad	Al	Tl
GR at 46 ˚C	Ad	Bd	Td	-	-	-	-	-	-
LS at 46 ˚C	Bd	Td	Ad	-	-	-	-	-	-
NS at 46 ˚C	Bd	Td	Ad	-	-	-	-	-	-

- indicates there is no computed data.

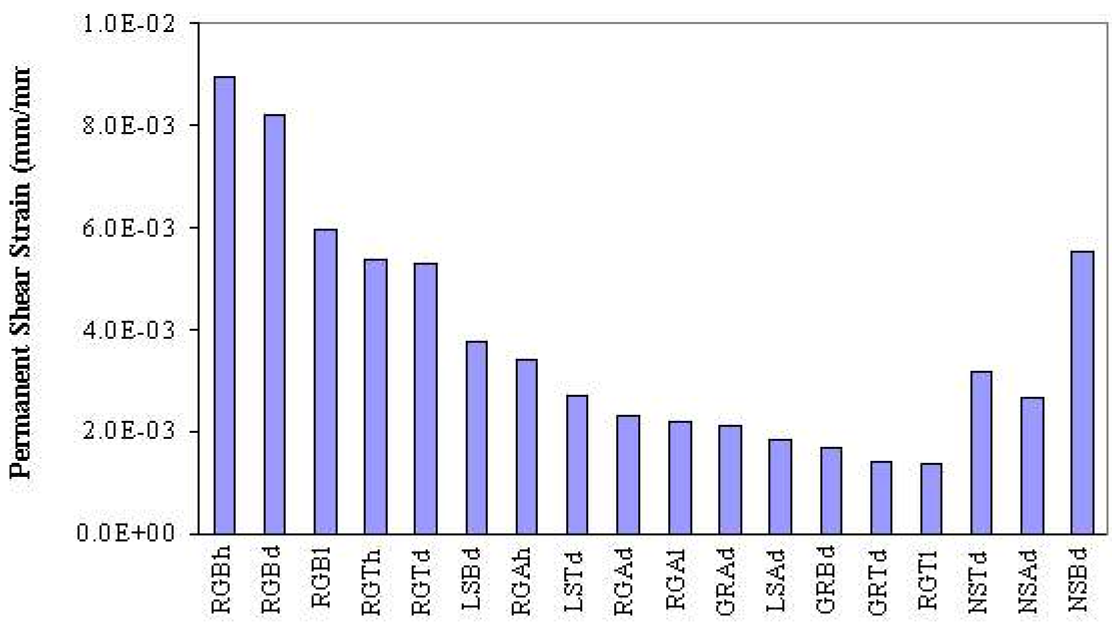

Figure 27 Permanent Shear Strain at 46˚C for Different Mixtures

4.5.3 Recovered Shear Strain

Recovered (elastic) strains of the different SSCH specimens were compared to rankings the blends. Recovered strain was measured 10 seconds after loading the specimen according to AASHTO TP7, Procedure D (*43*). Analyzing the data revealed that at 46˚C, the recoverable strain is about 15 percent to 30 percent of the maximum strain. At 20˚C, the recoverable strain is between a 30 percent to 40 percent of the maximum strain. And, at 4˚C, it is between a 40 percent to 50 percent of the maximum strain. A ratio between recovered and maximum deformation in the different blends can be computed. Smaller recoverable strain in the SSCH test indicates greater permanent and maximum shear strain and thus more rutting (assuming the same temperature). Comparing the rankings prepared in Table 26 and Figure 28 indicate:

- Higher asphalt contents yield greater recoverable strain.
- The percentage of recoverable strain in a specimen increases when the temperature decreases, because the asphalt cement is more elastic at low temperatures.
- For the river gravel mixtures, recoverable shear strain is greater in the blends passing below the restricted zone. For the limestone and granite mixtures, the higher recoverable strains are in the blends passing above the restricted zone.

Table 26. Ranking Asphalt Mixtures Considering Recoverable Shear Strain

Max. Shear Strain	1st (Best)	2nd	3rd	4th	5th	6th	7th	8th	9th (Worst)
RG at 4˚C	Ah	Al	Bd	Th	Bh	Bl	Tl	Td	Ad
RG at 20˚C	Bh	Bd	Bl	Al	Ah	Td	Ad	Th	Tl
RG at 46˚C	Bd	Bh	Bl	Th	Td	Tl	Ah	Ad	Al
GR at 46˚C	Ad	Td	Bd	-	-	-	-	-	-
LS at 46˚C	Ad	Td	Bd	-	-	-	-	-	-
NS at 46˚C	Td	Bd	Ad	-	-	-	-	-	-

- indicates there is no computed data

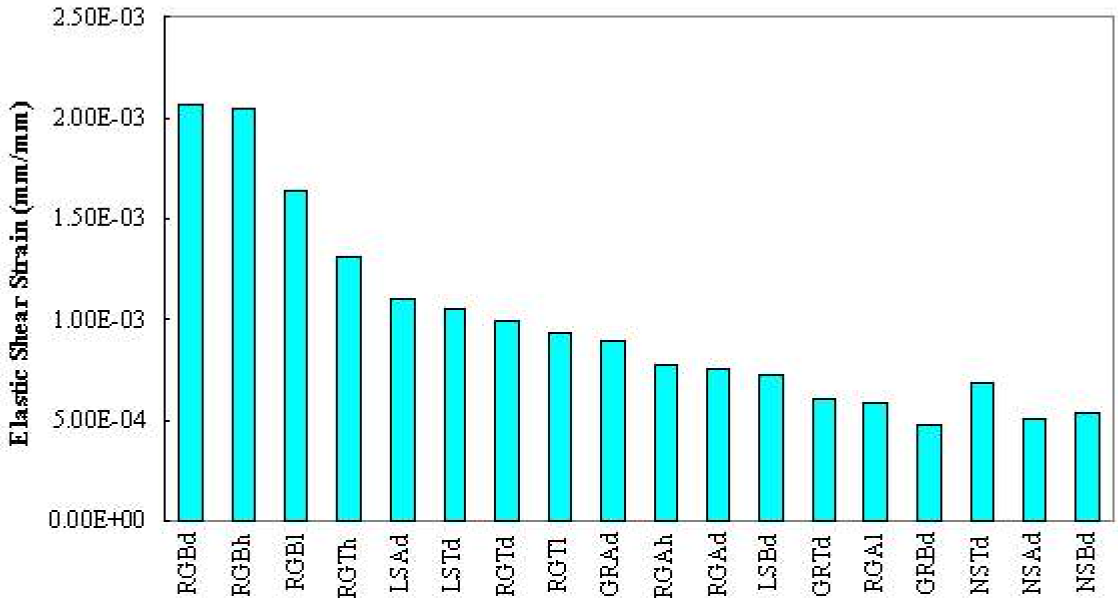

Figure 28 Elastic Shear Strain at 46˚C for Different Mixtures

4.5.4 Permanent Shear Strain After the First Load Application

Permanent strain after the first load application from the SSCH is used as the independent term in the Superpave performance model. To obtain the permanent shear strain after the first load application, a loading time of 0.02 seconds was selected. This is representative of the range of loading times occurring in practice and equivalent to a vehicle speed of 70 km/hr (40 mph) according to McLean's (*49*)square wave loading. This time of loading value has also been suggested by Shell (*50*). In Table 27 and Figures 29 and 30, permanent shear strains after the first load application are provided. These data support the following conclusions:

- The blends which developed the greatest early permanent strains are those which pass below the restricted zone for river gravel and limestone and those which pass above the restricted zone for granite.
- Mixtures with higher asphalt contents usually exhibited greater early permanent deformation.
- Rankings of blends at 46˚C and 20˚C was similar, but, at 4˚C, the rankings were different.
- Blends prepared with river gravel appeared more susceptible to rutting.
- Rankings obtained here are very similar to those obtained for maximum and permanent shear strain.

Table 27. Permanent Deformation after the First Load Application

Max. Shear Strain	Td	Th	Tl	Ad	Ah	Al	Bd	Bh	Bl
RG at 4˚C	1.45E-6	2.99E-6	1.16E-6	1.25E-6	6.82E-6	4.62E-6	3.84E-6	2.93E-6	1.89E-6
RG at 20˚C	2.85E-6	2.34E-6	1.08E-6	2.41E-6	2.91E-6	2.67E-6	4.59E-6	5.53E-6	3.50E-6
RG at 4˚C	1.07E-5	1.23E-5	3.99E-6	7.00E-6	1.22E-5	5.48E-6	1.93E-5	1.97E-5	1.46E-5
GR at 46˚C	2.83E-6	-	-	4.21E-6	-	-	3.37E-6	-	-
LS at 46˚C	5.41E-6	-	-	3.65E-6	-	-	7.50E-6	-	-
NS at 46˚C	-	-	-	-	-	-	-	-	-

- indicates there is no data available

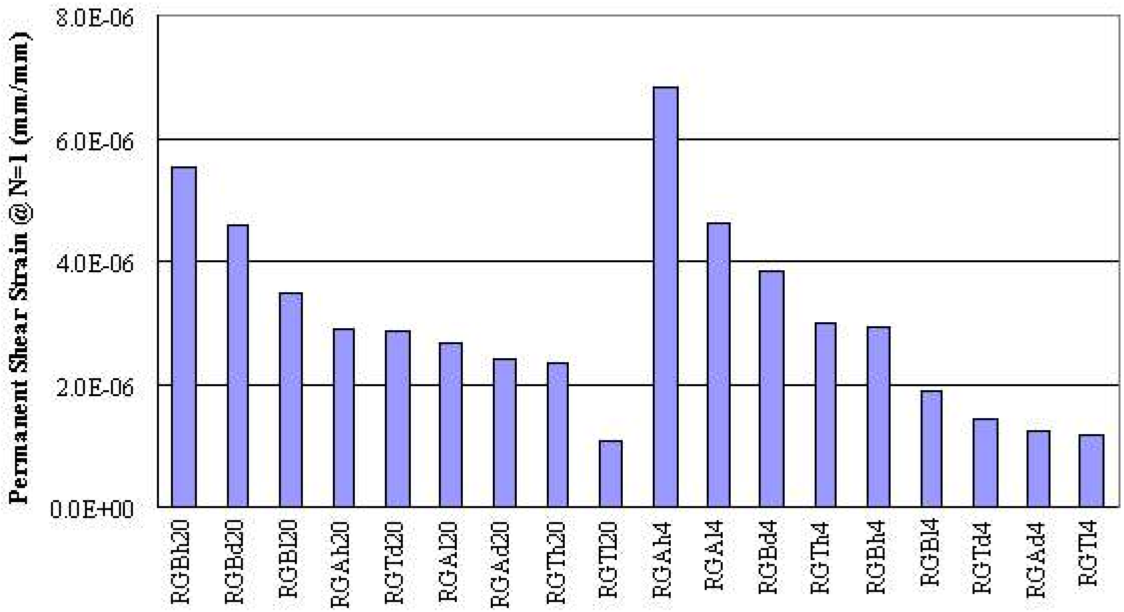

Figure 29 Permanent Shear Strain of RG at 4˚C and 20˚C at First Load Cycle

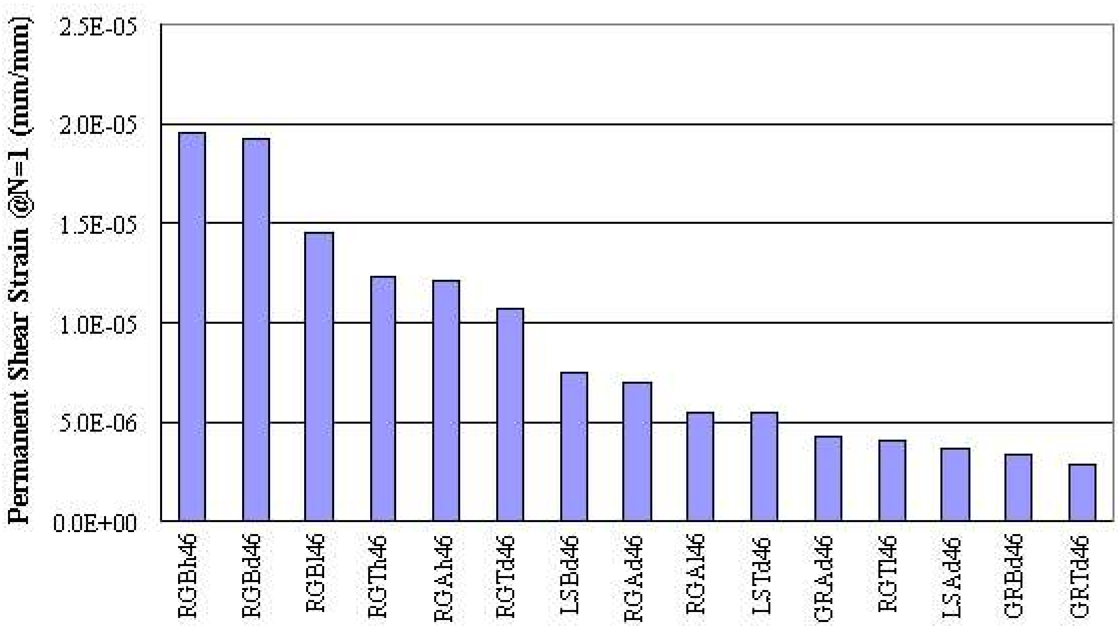

Figure 30 Permanent Shear Strain After the First Load Cycle at 46˚C

4.6 REPEATED SHEAR AT CONSTANT STRESS RATIO

RSCSR evaluates the potential for tertiary flow of asphalt mixtures. This form of rutting normally occurs at low air void contents and is the result of mixture instability.

In the RSCSR test, a repeated synchronized haversine shear and axial load pulses are applied to the specimen. The load cycle requires 0.7 seconds, wherein a 0.1-second load is followed by 0.6-second rest period. The ratio of haversine axial load to shear load was maintained at a constant ratio within the range of 1.2 to 1.5. This test was performed at high asphalt content (asphalt content corresponding to three percent air voids at N_{des}) to enhance tertiary rutting for the aggregate type and gradation. The shear stress and axial stress selected correspond to a strong base condition was 98 kPa and 148 kPa, respectively(*6*, *46*).

The specimens were preconditioned by applying 100 cycles of shear load pulses with a peak magnitude of 7±1 kPa and corresponding axial loads. After preconditioning the specimens, the repeated shear test was initiated. A detailed description of this test method is given in AASHTO TP7, Procedure C (*43*).

Test specimens were subjected to 10,000 load cycles at a temperature of 46˚C. No tertiary flow was observed in any specimen, but the data obtained was analyzed to characterize the asphalt mixtures at the test conditions. From this test and the Superpave performance model (Table 28 and Figure 31), the following is concluded:

- Blends with highest permanent deformation, after 10,000 cycles, are those which pass below the restricted zone for river gravel, limestone, and river gravel + rounded sand and those which pass above the restricted zone for granite.
- Blends most resistant to permanent deformation are those which pass above the restricted zone for river gravel, limestone, and river gravel + rounded sand and those which pass below the restricted zone with granite.
- River gravel mixtures are more prone to permanent deformation than any other mixture.
- Similar S-values (slope of log ε^{p} (N) versus log (N)) were obtained for the different gradations when the same aggregate, test temperature, and loading conditions were used (Table 29).

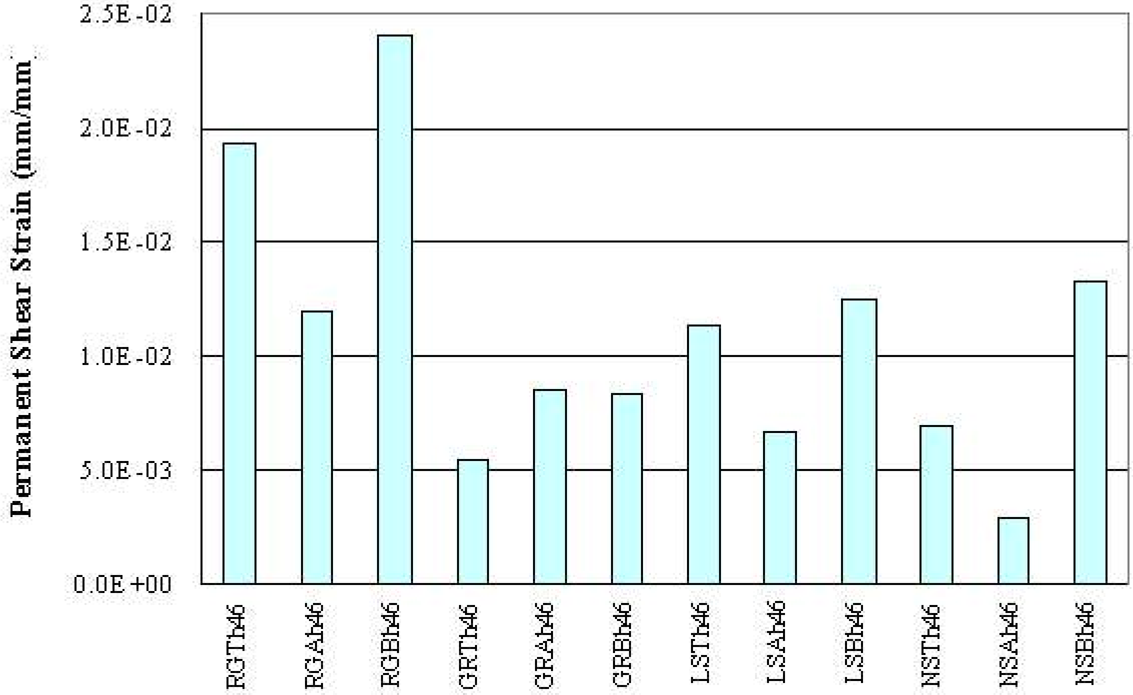

Figure 31 Permanent Shear Strain After 10,000 Cycles for RSCSR Test

Table 28. RSCSR S-Values at 46˚C

Aggregate Type	S-value
River Gravel	0.537-0.571
Granite	0.347-0.406
Limestone	0.387-0.422
Rounded Natural Sand	0.386-0.419

Table 29. Permanent Shear Strain Models from RSCSR

River Gravel	RGTh46	$\log(\varepsilon(N)) = \log(1.371E\text{-}4) + 0.5374 \times \log(N)$
	RGAh46	$\log(\varepsilon(N)) = \log(6.721E\text{-}5) + 0.5625 \times \log(N)$
	RGBh46	$\log(\varepsilon(N)) = \log(1.251E\text{-}4) + 0.5714 \times \log(N)$
Granite	GRTh46	$\log(\varepsilon(N)) = \log(1.605E\text{-}4) + 0.4062 \times \log(N)$
	GRAh46	$\log(\varepsilon(N)) = \log(4.426E\text{-}4) + 0.3476 \times \log(N)$
	GRBh46	$\log(\varepsilon(N)) = \log(3.045E\text{-}4) + 0.3846 \times \log(N)$
Limestone	LSTh46	$\log(\varepsilon(N)) = \log(3.033E\text{-}4) + 0.4226 \times \log(N)$
	LSAh46	$\log(\varepsilon(N)) = \log(2.456E\text{-}4) + 0.3872 \times \log(N)$
	LSBh46	$\log(\varepsilon(N)) = \log(2.641E\text{-}4) + 0.4186 \times \log(N)$
River Gravel + Rounded Natural Sand	NSTh46	$\log(\varepsilon(N)) = \log(2.687E\text{-}4) + 0.3862 * \log(N)$
	NSAh46	$\log(\varepsilon(N)) = \log(8.372E\text{-}5) + 0.4121 * \log(N)$
	NSBh46	$\log(\varepsilon(N)) = \log(3.684E\text{-}4) + 0.4192 * \log(N)$

$\varepsilon(N)$ is the permanent shear strain after N cycles

4.7 REPEATED SHEAR AT CONSTANT HEIGHT

The RSCH test is not required by Superpave. It was developed as a simplified method for Superpave Levels 2 and 3 to estimate rut depth. In the RSCH test, repeated haversine shear load pulses (68 kPa) are applied to the specimen. When the repeated shear load is applied, the test specimen tends to dilate. To prevent vertical dilation, a controlled axial load is applied to keep the specimen at a constant height. The load cycle requires 0.7 seconds, wherein a 0.1-second load is followed by 0.6-second rest period. This test was performed at the design asphalt content (asphalt content corresponding to four percent air voids at N_{des}) using only the mixtures containing river gravel.

Before testing, the specimens were preconditioned by applying 100 cycles of a haversine shear load with a peak magnitude of 7±1 kPa. After preconditioning, the specimens were subjected to 5,000 load cycles at a temperature of 59˚C in accordance with ATS Manual, Version 3.1 (*48*) and AASHTO TP7, Procedure F (*43*). This temperature was obtained from the maximum pavement design temperature expected at a depth of 5 cm and increased by two times the standard deviation to give 97 percent reliability.

The average (of two specimens) peak shearing strain obtained at the end of 5,000 load cycles are reported in Table 30. With this number of cycles, the asphalt mixture was modeled (Table 30). The equivalence between the load cycles applied at the test conditions described and the number of ESALs is estimated to be (*48*):

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

Table 30. Permanent Shear Strain Models from RSCH

Mixture Type	Peak Shear Strain	Model
RGTd46	0.046	$\log(\varepsilon(N)) = \log(8.815E\text{-}4) + 0.4678 \times \log(N)$
RGAd46	0.030	$\log(\varepsilon(N)) = \log(4.733E\text{-}4) + 0.4838 \times \log(N)$
RGBd46	0.057	$\log(\varepsilon(N)) = \log(1.028E\text{-}4) + 0.4724 \times \log(N)$

(*) $\varepsilon(N)$ is the permanent shear strain after N cycles

The following are observed from the test results:

- Blends with higher permanent deformation are those which pass below the restricted zone. The blends more resistant to permanent deformation are those which pass above the restricted zone.

- The difference in performance between the gradation TRZ and that BRZ is small (Table 31).

- Similar S-values (slope of $\log \varepsilon^p(N)$ versus log (N)) were obtained for the three different gradations. The S-values obtained from RSCH test and those obtained from the RSCSR test show that, with the same aggregate type, test temperature, and load conditions, similar S-values were achieved for the different gradations (Table 31).

Table 31. RSCH S-values and Permanent Deformation after 10 Million ESALs

Mixture	S-value	ε_{10M}
RGTd46	0.4678	0.0934
RGAd46	0.4838	0.0590
RGBd46	0.4724	0.1137

4.8 ASPHALT PAVEMENT ANALYZER

All twelve mixtures were tested for rutting using the Asphalt Pavement Analyzer. Four cylindrical specimens for each mixture were prepared using the Superpave gyratory compactor. Specimen size was 150 mm in diameter and 75 mm in height. The APA manufacturer recommends using three pairs of specimens for each mixture. But due to shortage a of materials, only two pairs of specimens were prepared for each mixture. Specimens were prepared with 4% air voids and rutting tests were performed at 64 ˚C. Each set of specimens was subjected to 8,000 load cycles (*51*). The wheel load and hose pressure were 445 N and 700 kPa, respectively. The average of two rut depths measured on two sets of specimens is reported as mixture rut depth.

Table 32 exhibits the rut depths measured for each specimen after 8,000 APA load cycles. There is no indication that mixtures passing through the restricted zone produce highest rutting. The river gravel + rounded sand mixture yielded the highest rut depth and the river gravel mixture yielded the second highest rut depth (Figure 32). Rut depth for river gravel + rounded sand and crushed river gravel mixtures are similar. This phenomenon could be attributed to the fact that the design asphalt contents for rounded sand mixtures were significantly lower than those for the crushed river gravel mixtures. For both gravel mixtures, those gradings passing below the restricted zone produced the highest rut depths.

Granite and limestone yielded much less rutting than the two river gravel mixtures. For these mixtures, the highest rutting was shown for the granite BRZ mixture and the limestone ARZ mixture.

Table 32. APA Rut Depths for Different Mixtures

Mixture Type		Rut Depth (mm)		
		Specimen 1	Specimen 2	Average
River Gravel	Above RZ	8.05	8.09	8.07
	Through RZ	11.11	10.75	10.93
	Below RZ	13.44	14.25	13.85
Granite	Above RZ	4.27	2.74	3.51
	Through RZ	2.88	3.47	3.18
	Below RZ	5.22	4.23	4.73
Limestone	Above RZ	4.75	5.11	4.93
	Through RZ	4.61	3.84	4.22
	Below RZ	4.68	4.09	4.38
Rounded Natural Sand	Above RZ	9.72	10.04	9.88
	Through RZ	10.71	8.46	9.58
	Below RZ	17.12	13.81	15.47

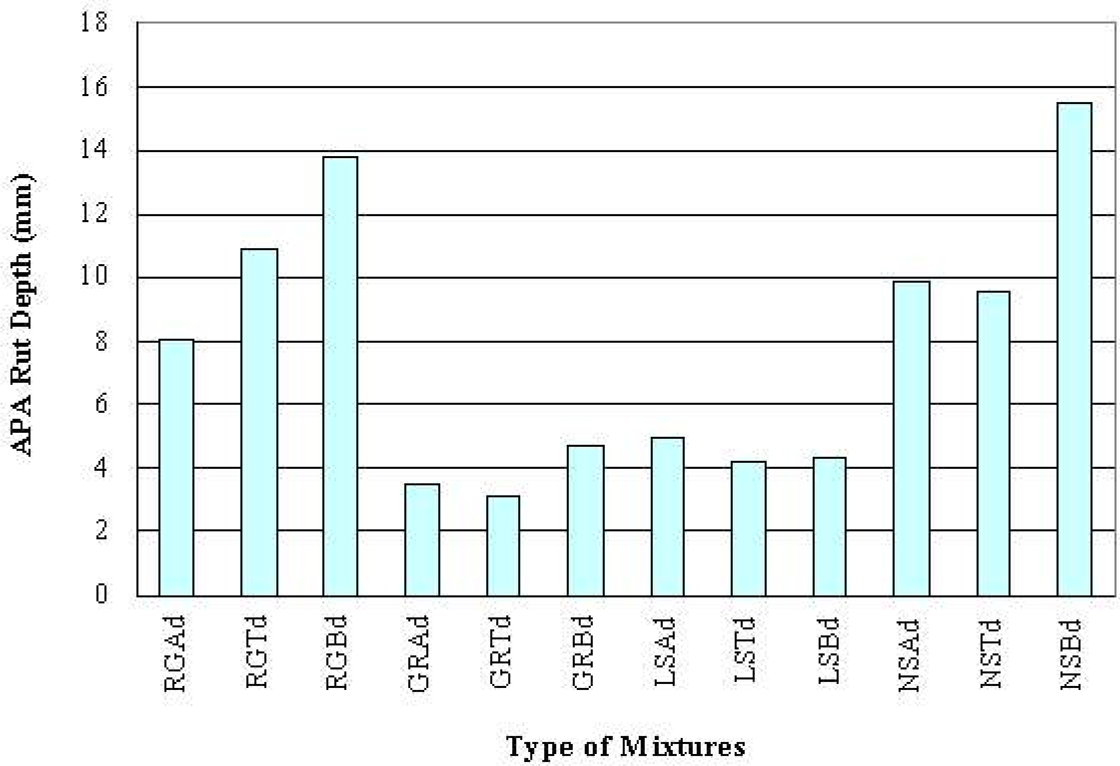

Figure 32 Average Rut Depth Measured by APA after 8,000 Cycles

4.9 SUMMARY OF LABORATORY TEST DATA AND DISCUSSION

4.9.1 General Observation

A summary of all the test data (Table 33) consistently reveals that the restricted zone has no effect on the rutting susceptibility of the Superpave asphalt mixtures tested. The data further reveals that, generally, the coarser mixtures BRZ are the most rut susceptible and the finer mixtures ARZ are the least rut susceptible. Similar findings were reported by Hand et al. (*9*), Hand et al. (*36*), and Mallick et al. (*37*). These findings refute the widely accepted concept that coarse-graded mixtures are normally more resistant to rutting than similar fine-graded mixtures. However, the authors still believe this concept to be generally true.

It should be pointed out that Superpave HMA mixtures are, by design, coarser graded than preceding conventional mixtures, this is particularly true of those gradings passing below the restricted zone, which were advocated by SHRP researchers. The authors believe this was a move in the right direction. However, due to lack of funding to study the fundamentals of the "new" aggregate gradations proposed by the SHRP researchers, the VMA requirement used with former conventional dense-graded mixtures were adopted for the Superpave mixtures. Coarser graded HMA mixtures, such as Superpave mixtures BRZ, possess a greater unit volume of aggregate than conventional dense-graded mixtures and, for optimum rutting performance, may have lower capacity for VMA and even air voids.

It should also be pointed out that this study examined 19-mm nominal maximum size HMA mixtures. In order to meet the Superpave VMA requirements during design of mixtures, the filler content had to be minimized in all the mixtures which, in turn, yielded a relatively low-viscosity mastic and thick asphalt films, particularly for the coarser graded mixtures (BRZ) which possess the lowest specific surface area of the three mixture types (ARZ, TRZ, and BRZ). The low viscosity mastic and thick films may have contributed to the relatively poor rutting performance of the HMA mixtures BRZ.

In conclusion, if VMA requirements for HMA mixtures are excessive, this may:

- cause difficulty in obtaining a mixture design that meets the VMA specification,
- force the use of fine-graded mixtures (ARZ),
- invite the introduction of excessive sand-size particles or the production of gap-graded mixtures,
- disallow sufficient filler (minus No. 200 material),
- promote excessive film thickness,
- needlessly increase the asphalt binder content and thus the cost of the mixture,
- produce a mixture that exhibits tenderness during construction, and/or
- produce a more rut-susceptible mixture (just the opposite of the purpose of VMA requirements)

All of these circumstances have been experienced at one time or another with Superpave mixtures.

Table 33. SST and APA Test Summary

Test Type	Measured Property	Partially Crushed River Gravel	Crushed Granite	Crushed Limestone	Rounded Natural Sand
FSCH	G*	A T > B	T > A > B	A B > T	A > T B
	G*/sinδ	A T > B	T > A > B	A B > T	A > T B
SSCH	Max Shear Strain	A > T > B	A T > B	A > T > B	A > T > B
	Perm Shear Strain	A > T > B	T > A > B	A > T > B	A > T > B
	Elastic Shear Strain	A > T > B	A > T > B	A T > B	A B > T
	Perm Shear Strain @ N=1	A > T > B	T > B > A	A > T > B	-- -- --
RSCSR	Perm Deformation	A > T > B	T > A B	A > T B	A > T > B
RSCH	Perm Deformation	A > T > B	-- -- --	-- -- --	-- -- --
APA	Rut Depth	A > T > B	T > A > B	T > B > A	T > A > B

A= Above, T= Through, B= Below; A > B Means A is more rut resistant than B

-- no data available.

4.9.2 Statistical Analysis

So far, all the tables prepared for ranking were based upon the numerical value of the visually observed nature of graphs (average of two specimens). The ranking data in Table 33 was examined statistically. Each HMA parameter was compared with respect t above, through, and below the restricted zone gradations for each mixture type.

The frequency sweep test properties (e.g., G* and G*/sinδ) were analyzed based on their values at 10 Hz frequency only. This frequency level is comparable with highway speed. Analysis of variance (ANOVA) and least significance difference (LSD) multiple comparison method were used to analyze these parameters at a 5 percent confidence level. No difference was found between the different gradations for any of the four mixtures.

At a 5 percent confidence level, maximum shearing strain measured by SSCH test for granite, limestone, and river gravel mixtures do not show any difference between their three gradations. In the case of the rounded natural sand mixture, below and through gradation and through and above gradation are statistically the same, but above and below are not the same.

The below the restricted zone gradation mixture containing rounded natural sand produces significantly more maximum shearing strain than the corresponding above restricted zone gradation mixture. A similar trend is observed for permanent shearing strain measured by SSCH test. Granite, limestone, and river gravel mixture do not exhibit any significant difference between the different gradations for the permanent shearing strain property. But for the mixture containing rounded natural sand, below the restricted zone gradation produce significantly more strain than that of the above restricted zone gradation. There is no significant differences in elastic shearing strain measured by SSCH test among the three gradations for mixtures containing granite, limestone, and rounded natural sand. Elastic shearing strain for below restricted zone is significantly higher than above and through restricted zone for the mixture containing river gravel.

Above, through, and below gradation do not exhibit any significant difference with respect to the property measured by RSCSR test for the mixtures containing granite, limestone, and river gravel. Permanent shearing strain measured by the RSCSR test on the through restricted zone rounded natural sand mixture is higher than that of the above restricted zone rounded natural sand mixture. Again, the below restricted zone natural sand mixture yields higher permanent shearing strain than the through restricted zone mixture. The RSCH test was performed only with the river gravel mixture. There is no significant difference between the three gradations with respect to the property measured by RSCH test.

The APA results were examined using ANOVA and LSD tests at a 5 percent confidence level. The granite and limestone mixtures do not exhibit any statistically significant differences between the three gradations. For the river gravel mixtures, the below gradation produced more rut depth than the through gradation and the through gradation produce more rut depth than the above gradation. For the mixtures containing rounded natural sand, the above and through gradations produced statistically equivalent rut depths and they are lower than the rut depth produced by the below gradation.

The above statistical analyses confirm that there is no indication that the through restricted zone gradation yields inferior mixtures compared to other gradations, and in some cases, the below restricted zone mixtures were inferior.

CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

Laboratory experiments were performed to predict the permanent deformation of Superpave HMA using four aggregate types with gradations that pass through, below, or above the restricted zone. The aggregates selected for this research were 100 percent crushed granite, 100 percent limestone, 100 percent partially crushed siliceous river gravel, and partially crushed river gravel plus rounded sand. Rutting susceptibility of HMA mixtures was measured using SSCH, FSCH, RSCSR, and RSCH with the Superpave SST and the APA. Based on the findings, the following conclusions and recommendations appear warranted:

- No relationship between the restricted zone and permanent deformation was found using HMA mixtures of high to relatively low quality.
- Superpave HMA mixtures above the restricted zone were generally most resistant to permanent deformation and mixtures below the restricted zone were generally most susceptible to permanent deformation (see data summary in Table 33).
- Aggregate shape and surface texture play a very important role in permanent deformation of HMA. Blends prepared with partially crushed river gravel were more sensitive to permanent deformation than those prepared with quarried limestone or granite. The crushed river gravel retained some of its original rounded surfaces and smooth surface texture.
- Fairly consistent permanent deformation rankings for HMA were obtained using the five different tests performed and nine different test parameters (Table 34).
- Asphalt mixture rheology responds differently at high temperatures than at low temperatures. At high temperatures, HMA rheology is predominantly affected by the aggregate, but at low temperatures, it is predominantly affected by the asphalt cement.

- Similar S-values (slope of accumulated permanent strain versus number of loads applications on a log-log scale) were obtained from the repeated shear test at constant stress ratio and constant height for the different gradations when using the same aggregate type and test conditions. This indicates that grading had little effect on rutting even when the grading passed through the restricted zone.
- The m-value (slope of the dynamic shear modulus versus frequency on a log-log scale) should not be used as the slope of the permanent deformation equation in the Superpave performance model because this might yield unreasonable values of permanent deformation.
- Extreme caution should be exercised when conducting SST procedures at high test temperatures (> 55˚C), because the accuracy of the results decrease at temperatures above 55˚C.
- Until validation of the 1993 Superpave performance model, which was used herein to predict rutting, these tests results should only be used for comparative rankings.

5.2 RECOMMENDATIONS

- The restricted zone should be eliminated form the Superpave specifications.
- Test temperatures above 55˚C should be avoided in the Superpave shear tester. Due to the Superpave shear tester characteristics, it is possible to perform the tests at a greater number of cycles without an excessive test temperature to simulate severe loading conditions.
- Similar S-values (slope of accumulated permanent strain versus number of load applications in a log-log scale) were obtained for HMA with different gradations tested using the same aggregate type, test temperature, and load conditions. A wider range of aggregate gradations should be tested in order to check this relationship.

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APPENDIX A

ASPHALT CEMENT CHARACTERIZATION

The asphalt cement used in the asphalt mixtures was tested according to the Superpave asphalt binder specification (AASHTO MP1). The mixing and compaction temperatures and complex shear modulus at different frequencies and temperatures was determined. A summary of the results is provided in Table A1. These results confirm that the grade of the asphalt cement is PG 64-22.

Table A1. PG 64-22 Requirements

Binder Property	Binder Aging Condition	Test Result	Superpave Requirement
Flash Point (˚C)	Unaged	299	>230
Viscosity at 135˚C (Pa-second)	Unaged	0.41	<3.00
Dynamic Shear, G*/sin δ at 64˚C (kPa)	Unaged	1.045	>1.00
Mass Loss (%)	RTFO aged	0.55	<1.00
Dynamic Shear, G*/sin δ at 64˚C (kPa)	RTFO aged	2.91	>2.20
Dynamic Shear, G*sin δ at 25˚C (kPa)	PAV aged	2842	<5000
Creep Stiffness, S at −12˚C (MPa)	PAV aged	176	<300
m-value at −12˚C	PAV aged	0.301	>0.300

The rheological properties of the asphalt cement were determined according to AASHTO TP5. The test apparatus used was a Bohlin Controlled Stress Rheometer. In Table A2, A3, and A4, test conditions and test results obtained using the Dynamic Shear Rheometer (DSR) are listed.

The asphalt cement was aged using the rolling thin film oven test (ASTM D 2872 or AASHTO T 240); and a pressure aging vessel (AASHTO PP1).

The stiffness of the asphalt cement at very low temperatures was measured according to AASHTO TP1 using a bending beam rheometer. In Table A5 and A6, test results obtained with the bending beam rheometer (BBR) are listed.

Table A2. Test Results on Original Binder from DSR

Test temperature	52˚C	58˚C	64˚C
Complex Shear Modulus (kPa)	5.299	2.257	1.042
Shear phase angle (degrees)	82.5	84.8	86.1
G*/sin δ (kPa)	5.345	2.266	1.045
Test plate diameter (mm)	25.0	25.0	25.0
Plate Gap (mm)	1.0	1.0	1.0
Test Frequency (rad/sec)	10.08	10.08	10.08
Final Temperature (˚C)	52.0	58.0	64.0
Strain amplitude (%)	11.73	11.84	11.95
TEST STATUS	Passed	Passed	Passed

Table A3. Test Results on RTFO Residue from DSR

Test temperature	70˚C	64˚C	58˚C
Complex Shear Modulus (kPa)	1.602	2.887	6.458
Shear phase angle (degrees)	85.7	83.1	80.1
G*/sin δ (kPa)	1.606	2.909	6.556
Test plate diameter (mm)	25.0	25.0	25.0
Plate Gap (mm)	1.0	1.0	1.0
Test Frequency (rad/sec)	10.08	10.08	10.08
Final Temperature (˚C)	70.1	64.0	58.0
Strain amplitude (%)	10.04	9.90	9.99
TEST STATUS	Failed	Passed	Passed

Table A4. Test Results on PAV Residue from DSR

Test temperature	19˚C	22˚C	25˚C
Complex Shear Modulus (kPa)	8275.8	6190.0	4511.7
Shear phase angle (degrees)	42.4	41.36	39.05
G*sin δ (kPa)	5580.4	4090.3	2842.4
Test plate diameter (mm)	8.0	8.0	8.0
Plate Gap (mm)	2.0	2.0	2.0
Test Frequency (rad/sec)	10.08	10.08	10.08
Final Temperature (˚C)	18.9	22.0	25.0
Strain amplitude (%)	1.01	1.01	1.03
TEST STATUS	Failed	Passed	Passed

Table A5. Test Results at −12˚C from BBR

Time (sec)	Force (mN)	Deflection (mm)	Measured Stiffness (MPa)	Estimated Stiffness (MPa)	Difference (%)	m-value
8	993	0.262	306	305	-0.327	0.249
15	994	0.309	259	260	0.386	0.265
30	994	0.374	214	215	0.467	0.283
60	**995**	**0.457**	**176**	**176**	**0.000**	**0.301**
120	995	0.565	142	142	0.000	0.318
240	1000	0.716	113	113	0.000	0.336

A = 2.69 B = -0.196 C = -0.0295 R^2 = 0.999965

Table A6. Test Results at -18°C from BBR

Time (sec)	Force (mN)	Deflection (mm)	Measured Stiffness (MPa)	Estimated Stiffness (MPa)	Difference (%)	m-value
8	994	0.179	448	447	-0.223	0.230
15	994	0.209	383	385	0.522	0.246
30	994	0.249	322	322	0.000	0.264
60	994	0.300	267	267	0.000	0.282
120	995	0.367	219	218	-0.457	0.301
240	997	0.458	176	176	0.000	0.319

A = 2.83 B = -0.175 C = -0.0302 R^2 = 0.999945

The flash point temperature was determined according to ASTM D 92. High temperature viscosity was measured using ASTM D 4402. The viscosity at 135 ˚C was 410 cP (0.41 Pa-s). See Figure A1.

In order to compare the asphalt cement rheology with that of the asphalt concrete, the complex modulus and the shear phase angle were determined at different frequencies and temperatures (Tables A7, A8, and A9).

Table A7. Shear Complex Modulus at 46˚C

Temperature (˚C)	Frequency (Hz)	Phase angle	Shear Complex Modulus (Pa)
46	10	76.11	6.16E6
46	5	77.57	3.43E6
46	2	79.80	1.54E6
46	1	81.28	8.25E5
46	0.5	82.92	4.36E5
46	0.2	84.61	1.85E5
46	0.1	85.82	9.51E4
46	0.05	87.06	4.87E4
46	0.02	88.64	1.01E4

Table A8. Shear Complex Modulus at 20˚C

Temperature (˚C)	Frequency (Hz)	Phase angle	Shear Complex Modulus (Pa)
20	10	58.77	5.19E6
20	5	60.64	3.46E6
20	2	63.37	1.91E6
20	1	65.40	1.22E6
20	0.5	67.19	7.36E5
20	0.2	69.67	3.71E5
20	0.1	71.09	2.17E5
20	0.05	73.15	1.20E5
20	0.02	76.67	3.13E4

Table A9. Shear Complex Modulus at 7˚C

Temperature (˚C)	Frequency (Hz)	Phase angle	Shear Complex Modulus (Pa)
7	10	45.24	2.27E7
7	5	48.12	1.61E7
7	2	51.83	9.82E6
7	1	54.86	6.69E6
7	0.5	57.32	4.38E6
7	0.2	60.62	2.48E6
7	0.1	62.85	1.55E6
7	0.05	65.13	9.78E5
7	0.02	68.49	3.34E5

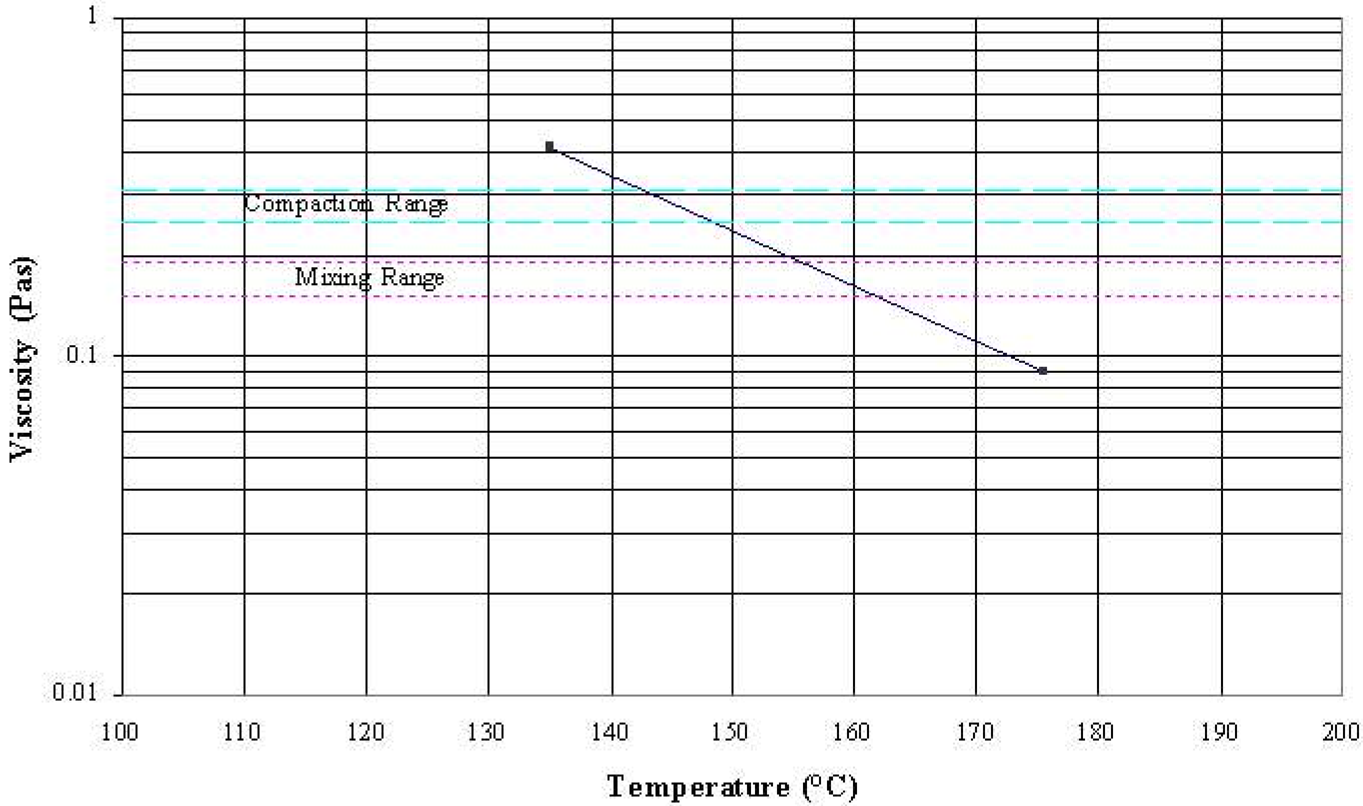

Figure A1. **PG 64-22 Brookfield Viscosity versus Temperature**

APPENDIX B

SUPERPAVE MIXTURE DESIGN

Table B1. River Gravel Gradations

								RGT	Through	RGA	Above	RGB	Below
	Sieve	Sieve Size	Maximum	Control Points		Restricted Zone		Total %	Individual	Total %	Individual	Total %	Individual
	Size (mm)	(0.45 power)	Density line	upper	lower	upper	lower	Passing	% Retained	Passing	% Retained	Passing	% Retained
	25	4.256699613	100					100	0	100	0	100	0
	19	3.762176102		100	90			96	4	96	4	96	4
	12.5	3.116086507			90			86	10	86	10	86	10
	9.5	2.754074109						77	9	77	9	77	9
4	4.75	2.016100254	47.36299					55	22	55	22	55	22
8	2.36	1.47166988		49	23	34.6	34.6	38	17	38	17	32	23
16	1.18	1.07732541				28.3	22.3	25	13	30	8	20	12
30	0.6	0.794635682				20.7	16.7	16	9	22	8	14	6
50	0.3	0.581707368				13.7	13.7	10	6	14	8	9	5
100	0.15	0.425834718						5	5	6	8	5.5	3.5
200	0.075	0.31172926		8	2			3	2	3	3	3.5	2
pan	0	0	0						3		3		3.5

Table B2. Granite Gradations

	Sieve Size (mm)	Sieve Size (0.45 power)	Maximum Density line	Control Points upper	Control Points lower	Restricted Zone upper	Restricted Zone lower	GRT Through Total % Passing	GRT Through Individual % Retained	GRA Above Total % Passing	GRA Above Individual % Retained	GRB Below Total % Passing	GRB Below Individual % Retained
	25	4.256699613	100					100	0	100	0	100	0
	19	3.762176102		100	90			98	2	98	2	98	2
	12.5	3.116086507			90			84	14	84	14	84	14
	9.5	2.754074109						67	17	67	17	67	17
4	4.75	2.016100254	47.36299					44	23	44	23	44	23
8	2.36	1.47166988		49	23	34.6	34.6	33	11	34.6	9.4	33	11
16	1.18	1.07732541				28.3	22.3	25	8	30	4.6	21	12
30	0.6	0.794635682				20.7	16.7	20	5	23	7	15.7	5.3
50	0.3	0.581707368				13.7	13.7	15	5	15	8	12.7	3
100	0.15	0.425834718						9	6	9	6	8	4.7
200	0.075	0.31172926		8	2			2	7	2	7	2	6
pan	0	0	0						2		2		2

Table B3. Limestone Gradations

								LST	Through	LSA	Above	LSB	Below
	Sieve	Sieve Size	Maximum	Control Points		Restricted Zone		Total %	Individual	Total %	Individual	Total %	Individual
	Size (mm)	(0.45 power)	Density line	upper	lower	upper	lower	Passing	% Retained	Passing	% Retained	Passing	% Retained
	25	4.256699613	100					100	0	100	0	100	0
	19	3.762176102		100	90			96	4	96	4	96	4
	12.5	3.116086507			90			86	10	86	10	86	10
	9.5	2.754074109						77	9	77	9	77	9
4	4.75	2.016100254	47.36299					55	22	55	22	55	22
8	2.36	1.47166988		49	23	34.6	34.6	38	17	36	19	32	23
16	1.18	1.07732541				28.3	22.3	25	13	30	6	20	12
30	0.6	0.794635682				20.7	16.7	16	9	22	8	14	6
50	0.3	0.581707368				13.7	13.7	10	6	14	8	9	5
100	0.15	0.425834718						5	5	4.5	9.5	5	4
200	0.075	0.31172926		8	2			3	2	3	1.5	3	2
pan	0	0	0						3		3		3

Table B4. River Gravel + Rounded Natural Sand Gradations

	Sieve Size (mm)	Sieve Size (0.45 power)	Maximum Density line	Control Points upper	Control Points lower	Restricted Zone upper	Restricted Zone lower	NST Total % Passing	Through Individual % Retained	NSA Total % Passing	Above Individual % Retained	NSB Total % Passing	Below Individual % Retained
	25	4.256699613	100					100	0	100	0	100	0
	19	3.762176102		100	90			96	4	96	4	96	4
	12.5	3.116086507						86	10	86	10	86	10
	9.5	2.754074109						77	9	77	9	77	9
4	4.75	2.016100254	47.36299					55	22	55	22	55	22
8	2.36	1.47166988		49	23	34.6	34.6	38	17	38	17	32	23
16	1.18	1.07732541				28.3	22.3	25	13	30	8	20	12
30	0.6	0.794635682				20.7	16.7	16	9	22	8	14	6
50	0.3	0.581707368				13.7	13.7	10	6	14	8	9	5
100	0.15	0.425834718						4	6	5	9	4	5

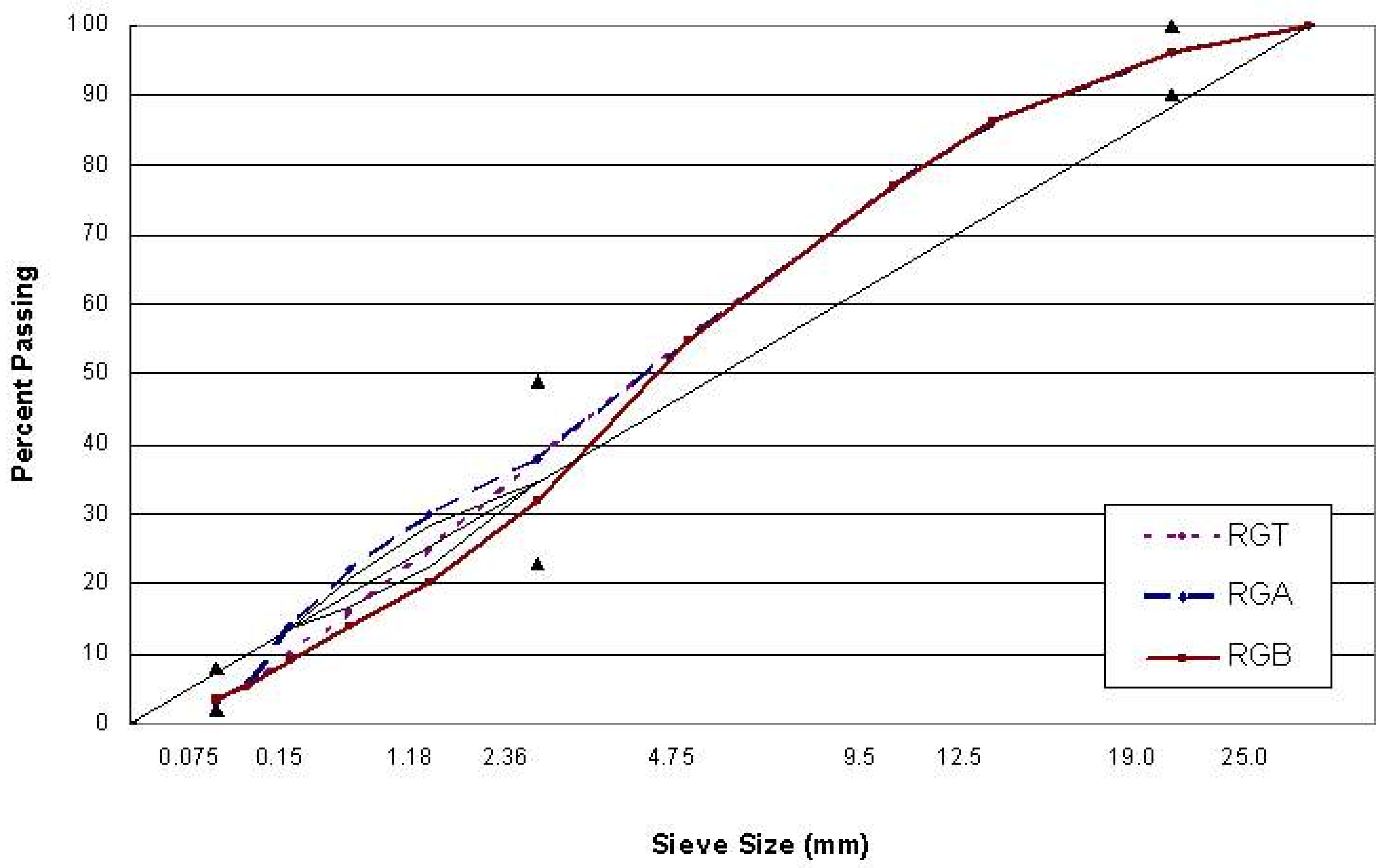

Figure B1. **River Gravel Gradation Curves**

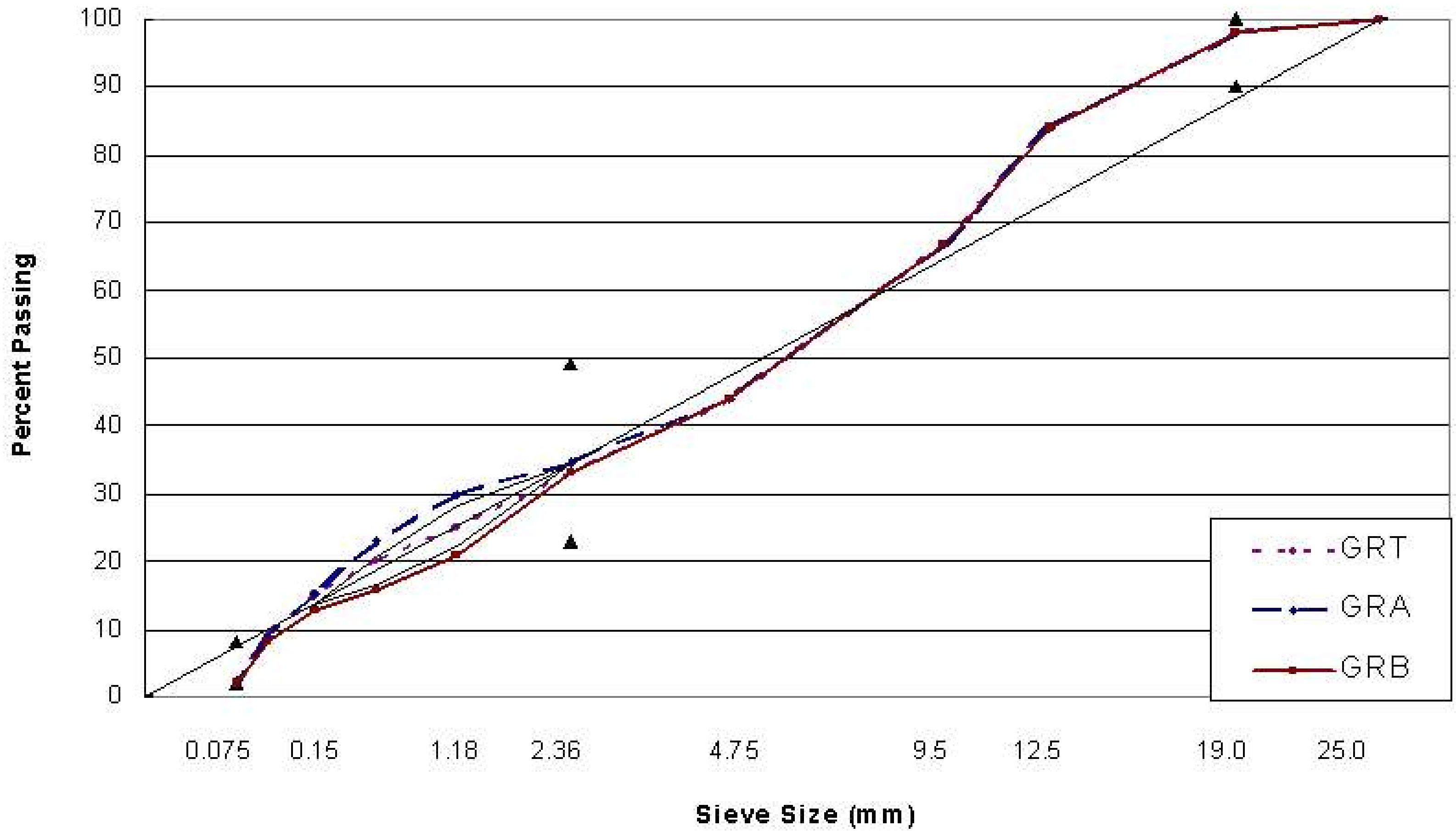

Figure B2. Granite Gradation Curves

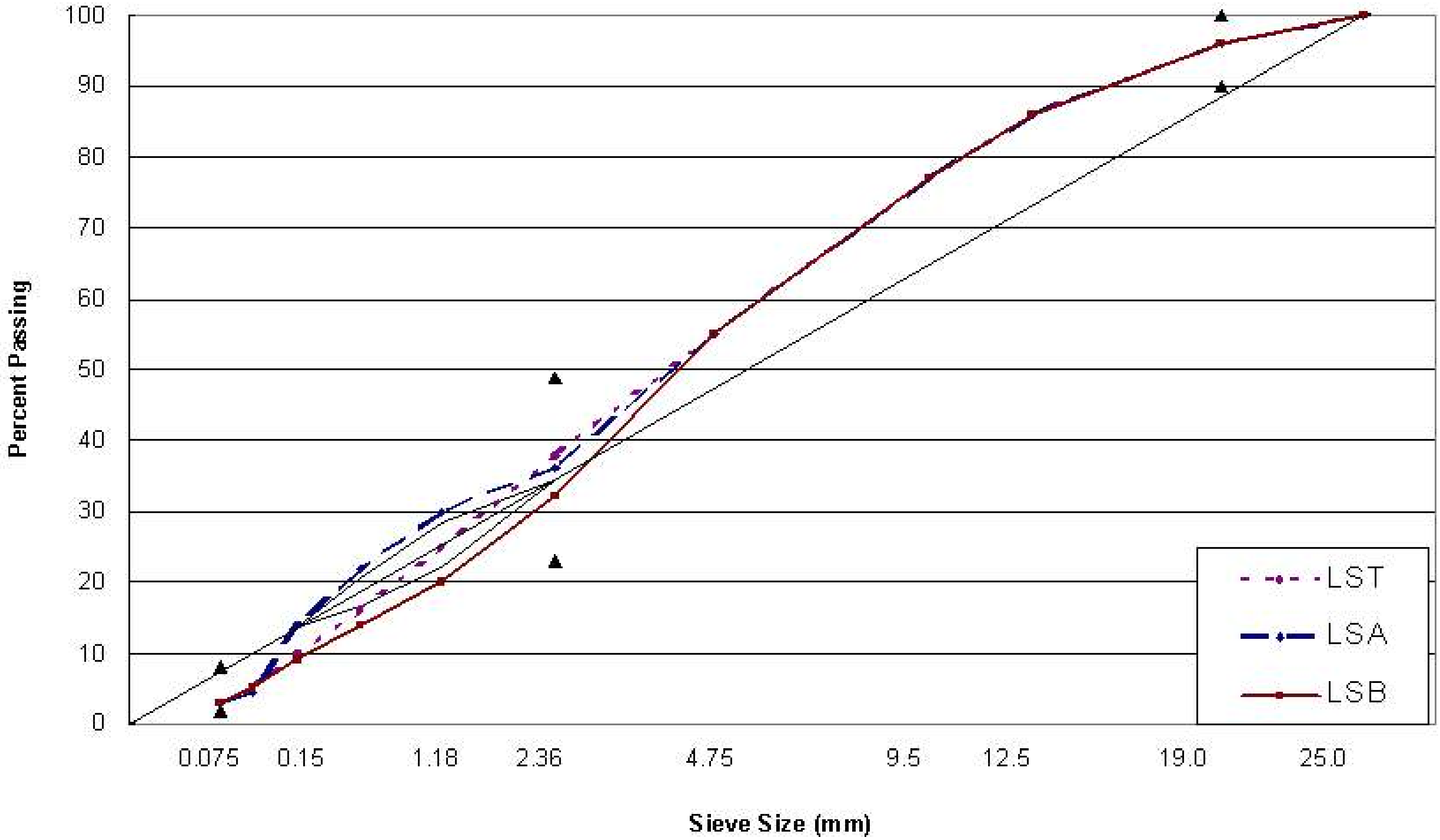

Figure B3. **Limestone Gradation Curves**

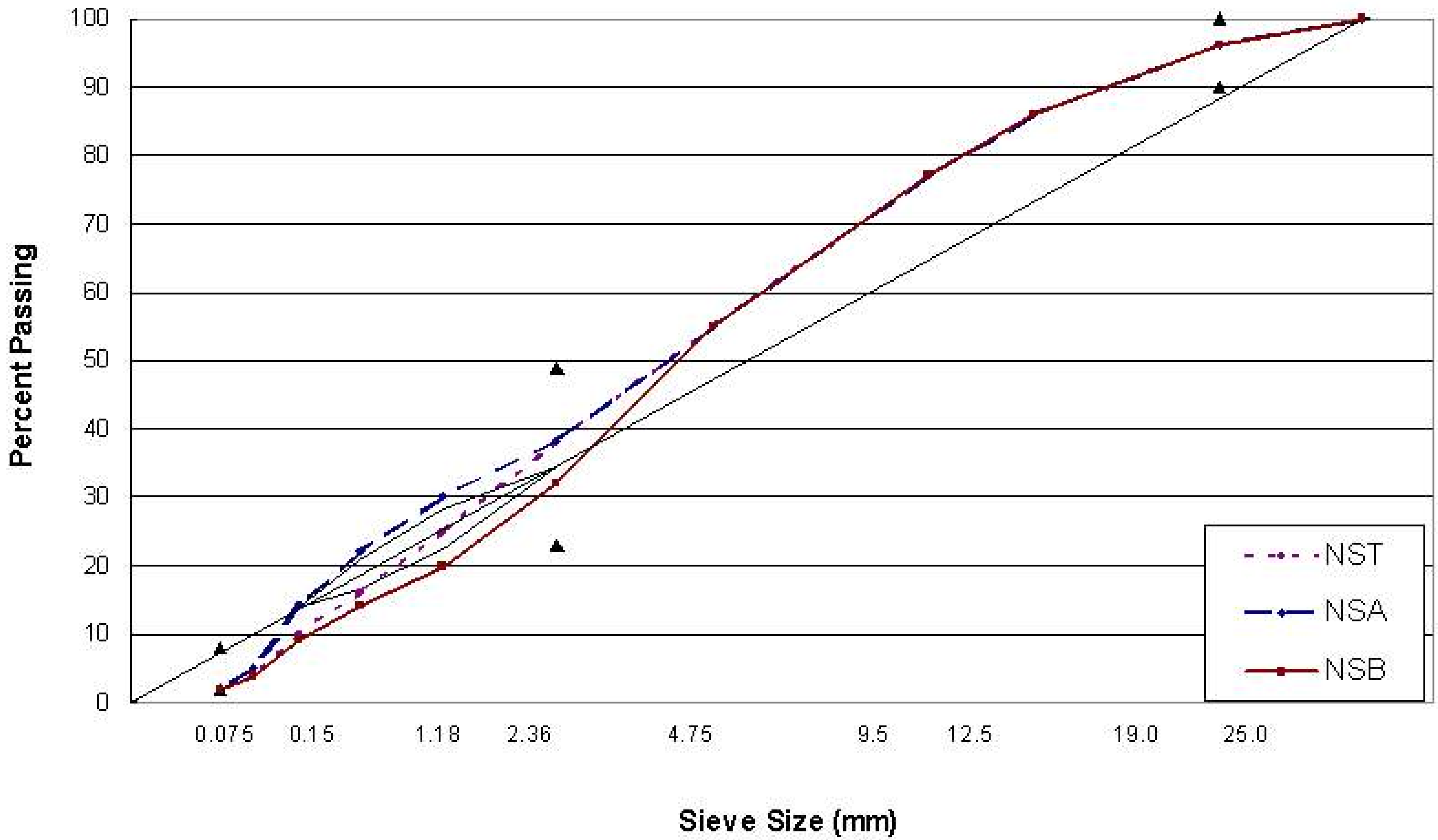

Figure B4. **River Gravel + Rounded Sand Gradation Curves**

Table B5. River Gravel Mixture Design Data

Aggregate Bulk Specific Gravity: 2.5881 gm/cc

RGT (through the restricted zone)

5.3 % Design Asphalt Content	Spec. 1	Spec. 2	Average	
Air Void at Nmax% (152 gyrations)	2.83	2.80	2.815	>2
Air Void at Nini % (8 gyrations)	11.72	11.72	11.72	>11
Air Void at Ndesign % (96 gyrations)	3.97	3.90	3.94	4
VMA at Ndesign, %	14.75	14.69	14.72	>13
VFA at Ndesign, %	73.10	73.45	73.275	65-75
Dust Proportion			0.69	0.6-1.2

RGA (above the restricted zone)

5.0% Design Asphalt Content	Spec. 1	Spec. 2	Average	
Air Void at Nmax% (152 gyrations)	3.07	3.35	3.21	>2
Air Void at Nini % (8 gyrations)	11.08	11.33	11.205	>11
Air Void at Ndesign % (96 gyrations)	4.03	4.31	4.17	4
VMA at Ndesign, %	13.83	14.08	13.96	>13
VFA at Ndesign, %	70.90	69.42	70.16	65-75
Dust Proportion			0.73	0.6-1.2

RGB (below the restricted zone)

5.6% Design Asphalt Content	Spec. 1	Spec. 2	Average	
Air Void at Nmax% (152 gyrations)	2.56	2.91	2.735	>2
Air Void at Nini % (8 gyrations)	12.16	12.42	12.29	>11
Air Void at Ndesign % (96 gyrations)	3.91	4.08	3.995	4
VMA at Ndesign, %	14.97	15.28	15.125	>13
VFA at Ndesign, %	75.12	73.29	74.205	65-75
Dust Proportion			0.76	0.6-1.2

Table B6. Granite Mixture Design Data

Aggregate Bulk Specific Gravity: 2.6941 gm/cc

GRT (through the restricted zone)

4.3 % Design Asphalt Content	Spec. 1	Spec. 2	Average	
Air Void at Nmax % (134 gyrations)	3.07	2.90	2.985	>2
Air Void at Nini % (7 gyrations)	11.75	11.91	11.83	>11
Air Void at Ndesign % (86 gyrations)	4.01	3.98	3.995	4
VMA at Ndesign, %	13.06	13.01	13.035	>13
VFA at Ndesign, %	69.34	69.38	69.36	65-75
Dust Proportion			0.63	0.6-1.2

GRA (above the restricted zone)

4.3% Design Asphalt Content	Spec. 1	Spec. 2	Average	
Air Void at Nmax % (134 gyrations)	3.30	2.43	2.865	>2
Air Void at Nini % (7 gyrations)	11.62	10.51	11.065	>11
Air Void at Ndesign % (86 gyrations)	4.24	3.34	3.79	4
VMA at Ndesign, %	13.68	12.64	13.16	>13
VFA at Ndesign, %	68.98	73.55	71.265	65-75
Dust Proportion			0.63	0.6-1.2

GRB (below the restricted zone)

4.3 % Design Asphalt Content	Spec. 1	Spec. 2	Average	
Air Void at Nmax % (134 gyrations)	2.74	2.62	2.68	>2
Air Void at Nini % (7 gyrations)	12.43	12.29	12.36	>11
Air Void at Ndesign % (86 gyrations)	3.81	3.73	3.77	4
VMA at Ndesign, %	13.04	12.97	13.005	>13
VFA at Ndesign, %	70.79	71.21	71	65-75
Dust Proportion			0.63	0.6-1.2

Table B7. Limestone Mixture Design Data

Aggregate Bulk Specific Gravity: 2.6575 gm/cc

LST (through the restricted zone)

4.5 % Design Asphalt Content	Spec. 1	Spec. 2	Average	
Air Void at Nmax % (152 gyrations)	2.60	2.61	2.605	>2
Air Void at Nini % (8 gyrations)	13.94	13.81	13.875	>11
Air Void at Ndesign % (96 gyrations)	4.05	4.10	4.075	4
VMA at Ndesign, %	13.01	13.08	13.045	>13
VFA at Ndesign, %	68.83	68.69	68.76	65-75
Dust Proportion			0.82	0.6-1.2

LSA (above the restricted zone)

4.0 % Design Asphalt Content	Spec. 1	Spec. 2	Average	
Air Void at Nmax % (152 gyrations)	2.79	2.63	2.71	>2
Air Void at Nini % (8 gyrations)	12.42	12.39	12.405	>11
Air Void at Ndesign % (96 gyrations)	3.99	3.78	3.885	4
VMA at Ndesign, %	12.06	11.87	11.965	>13
VFA at Ndesign, %	66.94	68.14	67.54	65-75
Dust Proportion			0.91	0.6-1.2

LSB (below the restricted zone)

4.8 % Design Asphalt Content	Spec. 1	Spec. 2	Average	
Air Void at Nmax % (152 gyrations)	2.74	2.31	2.525	>2
Air Void at Nini % (8 gyrations)	14.73	14.36	14.545	>11
Air Void at Ndesign % (96 gyrations)	4.16	3.79	3.975	4
VMA at Ndesign, %	13.91	13.60	13.755	>13
VFA at Ndesign, %	70.00	72.13	71.065	65-75
Dust Proportion			0.77	0.6-1.2

Table B8. River Gravel + Rounded Natural Sand Mixture Design Data

Aggregate Bulk Specific Gravity: gm/cc 2.556

NST (through the restricted zone)

3.8 % Design Asphalt Content	Spec. 1	Spec. 2	Estimated	
Air Void at Nmax % (152 gyrations)	3.07	3.39	3.23	>2
Air Void at Nini % (8 gyrations)	9.90	8.70	9.3	>11
Air Void at Ndesign % (96 gyrations)	3.84	4.20	4.02	4
VMA at Ndesign, %	9.86	9.95	9.9	>13
VFA at Ndesign, %	61.23	60.79	61.02	65-75
Dust Proportion			0.52	0.6-1.2

NSA (above the restricted zone)

3.5% Design Asphalt Content	Spec. 1	Spec. 2	Estimated	
Air Void at Nmax % (152 gyrations)	3.53	3.30	3.42	>2
Air Void at Nini % (8 gyrations)	10.22	8.10	9.16	>11
Air Void at Ndesign % (96 gyrations)	4.30	3.95	4.13	4
VMA at Ndesign, %	9.82	9.41	9.62	>13
VFA at Ndesign, %	56.21	57.09	56.63	65-75
Dust Proportion			0.57	0.6-1.2

NSB (below the restricted zone)

4.3% Design Asphalt Content	Spec. 1	Spec. 2	Estimated	
Air Void at Nmax % (152 gyrations)	2.96	3.42	3.19	>2
Air Void at Nini % (8 gyrations)	10.54	11.98	11.26	>11
Air Void at Ndesign % (96 gyrations)	3.73	3.90	3.82	4
VMA at Ndesign, %	10.48	11.02	10.75	>13
VFA at Ndesign, %	64.40	63.47	63.93	65-75
Dust Proportion			0.47	0.6-1.2

Table B9. River Gravel Through Restricted Zone SST Specimen Data

RGT(through the restricted zone)
Design asphalt content 5.3 % asphalt content.

RGTd, 4% voids		RSCH
Design Asphalt content: 5.3 %		
Rice specific gravity: 2.4282		
	Bulk Sp. Gravity	% voids
RGTd Spec. 1	2.3318	3.97
RGTd Spec. 2	2.3335	3.9
Average	2.3327	3.94

RGTh, 3% voids		RSCS
High Asphalt content: 5.6 %		
Rice specific gravity: 2.4118		
	Bulk Sp. Gravity	% voids
RGTh Spec. 1	2.3346	3.12
RGTh Spec. 2	2.3383	2.97
Average	2.3365	3.05

RGTl, 7% voids		SS/FS
Low Asphalt content: 4.80 %		
Rice specific gravity: 2.4525		
	Bulk Sp. Gravity	% voids
RGTl Spec. 1	2.2896	6.64
RGTl Spec. 2	2.2902	6.61
Average	2.2899	6.63

RGTd, 7% voids		SS/FS
Design Asphalt content: 5.30 %		
Rice specific gravity: 2.4282		
	Bulk Sp. Gravity	% voids
RGTd Spec. 1	2.2530	7.12
RGTd Spec. 2	2.2531	7.13
Average	2.2531	7.13

RGTh, 7% voids		SS/FS
High Asphalt Content: 5.60 %		
Rice specific gravity: 2.4118		
	Bulk Sp. Gravity	% voids
RGTh Spec. 1	2.2477	6.80
RGTh Spec. 2	2.2507	6.68
Average	2.2492	6.74

Table B10. River Gravel Above Restricted Zone SST Specimen Data

RGA(above the restricted zone)
Design asphalt content 5.0 % asphalt content.

RGAd, 4% voids		RSCH
Design Asphalt content: 5.0 %		
Rice specific gravity: 2.4384		
	Bulk Sp. Gravity	% voids
RGAd Spec. 1	2.3401	4.03
RGAd Spec. 2	2.3333	4.31
Average	2.3367	4.17

RGAh, 3% voids		RSCS
High Asphalt content: 5.30 %		
Rice specific gravity: 2.4296		
	Bulk Sp. Gravity	% voids
RGAh Spec. 1	2.3546	3.12
RGAh Spec. 2	2.3551	3.10
Average	2.3549	3.11

RGAl, 7% voids		SS/FS
Low Asphalt content: 4.40 %		
Rice specific gravity: 2.4740		
	Bulk Sp. Gravity	% voids
RGAl Spec. 1	2.3101	6.62
RGAl Spec. 2	2.3096	6.64
Average	2.3099	6.63

RGAd, 7% voids		SS/FS
Design Asphalt content: 5.00 %		
Rice specific gravity: 2.4384		
	Bulk Sp. Gravity	% voids
RGAd Spec. 1	2.2725	6.86
RGAd Spec. 2	2.2814	6.50
Average	2.2770	6.68

RGAh, 7% voids		SS/FS
High Asphalt content: 5.30 %		
Rice specific gravity: 2.4296		
	Bulk Sp. Gravity	% voids
RGAh Spec. 1	2.2711	6.52
RGAh Spec. 2	2.2703	6.56
Average	2.2707	6.54

Table B11. River Gravel Below Restricted Zone SST Specimen Data

RGB(below the restricted zone)
Design asphalt content 5.6 % asphalt content.

RGBd, 4% voids — RSCH
Design Asphalt content: 5.6 %
Rice specific gravity: 2.4247

	Bulk Sp. Gravity	% voids
RGBd Spec. 1	2.3298	3.91
RGBd Spec. 2	2.3258	4.08
Average	2.3278	4.00

RGBh, 3% voids — RSCS
High Asphalt content: 5.90 %
Rice specific gravity: 2.3996

	Bulk Sp. Gravity	% voids
RGBh Spec. 1	2.3343	3.26
RGBh Spec. 2	2.3295	3.46
Average	2.3319	3.36

RGB, 7% voids — SS/FS
Low Asphalt content: 4.90 %
Rice specific gravity: 2.4496

	Bulk Sp. Gravity	% voids
RGBl Spec. 1	2.289	6.56
RGBl Spec. 2	2.2898	6.52
Average	2.2894	6.54

RGB, 7% voids — SS/FS
Design Asphalt Content: 5.60 %
Rice specific gravity: 2.4247

	Bulk Sp. Gravity	% voids
RGBd Spec. 1	2.2288	8.12
RGBd Spec. 2	2.2369	7.79
Average	2.2329	7.96

RGB, 7% voids — SS/FS
High Asphalt content: 5.90 %
Rice specific gravity: 2.3996

	Bulk Sp. Gravity	% voids
RGBh Spec. 1	2.2352	6.86
RGBh Spec. 2	2.226	7.23
Average	2.2306	7.05

Table B12. Granite SST Specimen Data (Through, Above, and Below Restricted Zone

GRT(through the restricted zone)
Design asphalt content 4.3 % asphalt content.

GRTd, 7% voids		SS/FS
Design Asphalt content: 4.30 %		
Rice specific gravity: 2.5494		
	Bulk Sp. Gravity	% voids
GRTd Spec. 1	2.3681	7.11
GRTd Spec. 2	2.3701	7.03
Average	2.3691	7.07

GRTh, 3% voids		RSCS
High Asphalt Content: 4.60 %		
Rice specific gravity: 2.5377		
	Bulk Sp. Gravity	% voids
GRTh Spec. 1	2.4454	3.51
GRTh Spec. 2	2.4524	3.36
Average	2.4489	3.44

GRA(above the restricted zone)
Design asphalt content 4.3 % asphalt content.

GRBd, 7% voids		SS/FS
Design Asphalt content: 4.30 %		
Rice specific gravity: 2.5433		
	Bulk Sp. Gravity	% voids
GRAd Spec. 1	2.3574	7.31
GRAd Spec. 2	2.3614	7.15
Average	2.3594	7.23

GRBh, 3% voids		RSCS
High Asphalt Content: 4.60 %		
Rice specific gravity: 2.5362		
	Bulk Sp. Gravity	% voids
GRAh Spec. 1	2.4596	3.02
GRAh Spec. 2	2.4624	2.91
Average	2.4610	2.97

GRB(below the restricted zone)
Design asphalt content 4.3 % asphalt content.

GRBd, 7% voids		SS/FS
Design Asphalt content: 4.30 %		
Rice specific gravity: 2.5467		
	Bulk Sp. Gravity	% voids
GRBd Spec. 1	2.3791	6.58
GRBd Spec. 2	2.3717	6.87
Average	2.3754	6.73

GRBh, 3% voids		RSCS
High Asphalt Content: 4.60 %		
Rice specific gravity: 2.5369		
	Bulk Sp. Gravity	% voids
GRBh Spec. 1	2.4519	3.35
GRBh Spec. 2	2.4547	3.24
Average	2.4533	3.30

Table B13. Limestone SST Specimen Data (Through, Above, and Below Restricted Zone)

LST(through the restricted zone)
Design asphalt content 4.5 % asphalt content.

LSTd, 7% voids	SS/FS	
DESIGN Asphalt content: 4.50 %		
Rice specific gravity: 2.5220		
	Bulk Sp. Gravity	% voids
LSTd Spec. 1	2.3363	7.36
LSTd Spec. 2	2.3406	7.19
Average	2.3385	7.28

LSTh, 3% voids	RSCS	
HIGH Asphalt Content: 4.80 %		
Rice specific gravity: 2.5110		
	Bulk Sp. Gravity	% voids
LSTh Spec. 1	2.4339	3.07
LSTh Spec. 2	2.4319	3.15
Average	2.4329	3.11

LSA(above the restricted zone)
Design asphalt content 4.0 % asphalt content.

LS3d, 7% voids	SS/FS	
DESIGN Asphalt content: 4.00 %		
Rice specific gravity: 2.5355		
	Bulk Sp. Gravity	% voids
LSAd Spec. 1	2.3403	7.69
LSAd Spec. 2	2.3411	7.66
Average	2.3407	7.68

LSAh, 3% voids	RSCS	
HIGH Asphalt Content: 4.30 %		
Rice specific gravity: 2.5235		
	Bulk Sp. Gravity	% voids
LSAh Spec. 1	2.4506	2.88
LSAh Spec. 2	2.4453	3.09
Average	2.4480	2.99

LSB(below the restricted zone)
Design asphalt content 4.8 % asphalt content.

LSBd, 7% voids	SS/FS	
DESIGN Asphalt content: 4.80 %		
Rice specific gravity: 2.5467		
	Bulk Sp. Gravity	% voids
LSBd Spec. 1	2.3268	7.21
LSBd Spec. 2	2.3307	7.05
Average	2.3288	7.13

LSBh, 3% voids	RSCS	
HIGH Asphalt Content: 5.10 %		
Rice specific gravity: 2.4961		
	Bulk Sp. Gravity	% voids
LSBh Spec. 1	2.4303	2.63
LSBh Spec. 2	2.4324	2.55
Average	2.4314	2.59

Table B14. River Gravel + Rounded Sand SST Specimen Data (Through, Above, and Below Restricted Zone)

NST(through the restricted zone)
Design asphalt content 3.8 % asphalt content.

NSTd, 7% voids SS/FS
Design Asphalt content: 3.8 %
Rice specific gravity: 2.4930

	Bulk Sp. Gravity	% voids
NSTd Spec. 1	2.3240	6.78
NSTd Spec. 2	2.3170	7.06
Average	2.3205	6.92

NSTh, 3% voids RSCS
High Asphalt Content: 4.10
Rice specific gravity: 2.4930

	Bulk Sp. Gravity	% voids
NSTh Spec. 1	2.412	3.24
NSTh Spec. 2	2.427	3.56
Average	2.4195	3.40

NSA(above the restricted zone)
Design asphalt content 3.5 % asphalt content.

NSAd, 7% voids SS/FS
Design Asphalt content: 3.5 %
Rice specific gravity: 2.5050

	Bulk Sp. Gravity	% voids
NSAd Spec. 1	2.3520	6.13
NSAd Spec. 2	2.3420	6.52
Average	2.3407	6.33

NSAh, 3% voids RSCS
High Asphalt Content: 3.80
Rice specific gravity: 2.4940

	Bulk Sp. Gravity	% voids
NSAh Spec. 1	2.417	3.06
NSAh Spec. 2	2.427	2.64
Average	2.4220	2.85

LSB(below the restricted zone)
Design asphalt content 4.3 % asphalt content.

NSBd, 7% voids SS/FS
Design Asphalt content: 4.3 %
Rice specific gravity: 2.4810

	Bulk Sp. Gravity	% voids
NSBd Spec. 1	2.3190	6.52
NSBd Spec. 2	2.3090	6.94
Average	2.3140	6.73

NSBh, 3% voids RSCS
High Asphalt Content: 4.60 %
Rice specific gravity: 2.4800

	Bulk Sp. Gravity	% voids
NSBh Spec. 1	2.416	2.62
NSBh Spec. 2	2.402	3.20
Average	2.4090	2.91