2000

Fundamental Characterization and Numerical Simulation of Large Stone Asphalt Mixtures.

Baoshan Huang
Louisiana State University and Agricultural & Mechanical College

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FUNDAMENTAL CHARACTERIZATION AND NUMERICAL SIMULATION OF LARGE STONE ASPHALT MIXTURES

A Dissertation

Submitted to the Graduate Faculty of the Louisiana State University and Agricultural and Mechanical College
in partial fulfillment of the requirements for the degree of Doctor of Philosophy

in

The Department of Civil and Environmental Engineering

by

Baoshan Huang
B.S. Tongji University, Shanghai, China, 1984
M.S. Tongji University, Shanghai, China, 1988
December, 2000
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<tbody>
<tr>
<td>A</td>
<td>= cross-sectional area</td>
</tr>
<tr>
<td>AC</td>
<td>= asphalt cement</td>
</tr>
<tr>
<td>ALF</td>
<td>= accelerated loading facility</td>
</tr>
<tr>
<td>(A_p)</td>
<td>= area under the normalized stress-strain curve up to strain (\varepsilon_p)</td>
</tr>
<tr>
<td>APA</td>
<td>= asphalt pavement analyzer</td>
</tr>
<tr>
<td>(A_e)</td>
<td>= area under the normalized stress-strain curve up to strain (\varepsilon)</td>
</tr>
<tr>
<td>(C_u)</td>
<td>= coefficient of uniformity</td>
</tr>
<tr>
<td>D</td>
<td>= diameter of the specimen</td>
</tr>
<tr>
<td>d</td>
<td>= diameter of the particles; Drucker-Prager parameter</td>
</tr>
<tr>
<td>(D_{ca})</td>
<td>= density of coarse aggregate</td>
</tr>
<tr>
<td>(D_{cm})</td>
<td>= density of coarse aggregate in the compacted LSAM</td>
</tr>
<tr>
<td>(D_h)</td>
<td>= effective particle diameter</td>
</tr>
<tr>
<td>(d_{mb})</td>
<td>= bulk specific gravity of the compacted LSAM</td>
</tr>
<tr>
<td>(d_w)</td>
<td>= density of water</td>
</tr>
<tr>
<td>DOT</td>
<td>= department of transportation</td>
</tr>
<tr>
<td>E</td>
<td>= Young’s elastic modulus</td>
</tr>
<tr>
<td>(e_{ij})</td>
<td>= deviatoric strain tensor</td>
</tr>
<tr>
<td>FEM</td>
<td>= finite element method</td>
</tr>
<tr>
<td>FHWA</td>
<td>= Federal Highway Administration</td>
</tr>
<tr>
<td>FSCH</td>
<td>= frequency sweep at constant height</td>
</tr>
<tr>
<td>g</td>
<td>= acceleration of gravity</td>
</tr>
<tr>
<td>(G_{ca})</td>
<td>= specific gravity of coarse aggregate</td>
</tr>
<tr>
<td>(G_{mb})</td>
<td>= bulk specific gravity</td>
</tr>
<tr>
<td>(G_{mm})</td>
<td>= maximum (Rice) specific gravity</td>
</tr>
<tr>
<td>GTM</td>
<td>= gyratory testing machine</td>
</tr>
<tr>
<td>(G')</td>
<td>= dynamic complex shear modulus</td>
</tr>
<tr>
<td>h</td>
<td>= hydraulic head</td>
</tr>
<tr>
<td>(H_T)</td>
<td>= horizontal deformation at peak load</td>
</tr>
<tr>
<td>HMA</td>
<td>= hot mix asphalt</td>
</tr>
<tr>
<td>i</td>
<td>= hydraulic gradient</td>
</tr>
<tr>
<td>ITS</td>
<td>= indirect tensile strength</td>
</tr>
<tr>
<td>(K)</td>
<td>= coefficient of permeability; bulk elastic modulus; ratio of yield stress in triaxial tension to yield stress in triaxial compression</td>
</tr>
<tr>
<td>(k)</td>
<td>= intrinsic permeability</td>
</tr>
<tr>
<td>(K')</td>
<td>= pseudo coefficient of permeability</td>
</tr>
<tr>
<td>L</td>
<td>= length</td>
</tr>
<tr>
<td>LaDOTD</td>
<td>= Louisiana Department of Transportation and Development</td>
</tr>
<tr>
<td>LSAM</td>
<td>= large stone asphalt mixtures</td>
</tr>
<tr>
<td>LTRC</td>
<td>= Louisiana Transportation Research Center</td>
</tr>
<tr>
<td>LWT</td>
<td>= loaded wheel tester</td>
</tr>
<tr>
<td>m</td>
<td>= shape factor</td>
</tr>
<tr>
<td>(M_R)</td>
<td>= indirect tensile resilient modulus</td>
</tr>
<tr>
<td>N</td>
<td>= number of pipes</td>
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n = porosity
n_e = effective porosity
P = applied vertical load
P_ult = peak load
Q = rate of flow
q = Mises equivalent stress
R = percent of coarse aggregate in LSAM; Raynolds number; over stress ratio
r = radius; third invariant of deviatoric stress
RAP = recycled asphalt pavement
RSCH = repetitive shear at constant height
RSCSR = repetitive shear at constant stress ratio
S_{ij} = deviatoric stress tensor
SGC = Superpave gyratory compactor
SSC = degree of stone-on-stone contact
SST = Superpave shear tester
S_T = tensile strength
S_{tc} = average tensile strength of the control samples
S_{tm} = average tensile strength of the moisture-conditioned samples
t = sample thickness; time
TI = toughness index
TSR = tensile strength ratio
TTI = Texas Transportation Institute
v = velocity of flow
VCA = voids in coarse aggregates
VMA = volume mix asphalt
VFA = volume filled with asphalt
\delta H = horizontal deformation
\delta V = vertical deformation
\alpha = coefficient of shape factor
\epsilon = strain
\epsilon_p = strain corresponding to the peak stress
\epsilon_T = horizontal tensile strain at failure
\gamma = specific weight
\gamma = viscosity parameter
\eta = viscosity of the fluid
\phi = friction angle
\mu = poisson's ratio; dynamic viscosity
\nu = kinematic viscosity
\sigma_{ij} = stress tensor
ABSTRACT

Large stone asphalt mixtures (LSAM) are mixtures that contain maximum aggregate sizes between 25 and 63 mm. LSAMs are used to improve the mixtures’ resistance to rutting and also improve the durability of pavements. However, due to historical reasons, LSAM has been rarely used in pavement constructions.

The objective of this study was to determine the fundamental engineering properties of LSAM for potential use in Louisiana and to conduct numerical simulations of pavements that contain LSAMs. The scope of this evaluation included two types LSAMs: an open-graded and a dense-graded 37.5-mm Superpave mix, and three types of asphalt binders: an SB polymer modified PG 70-22M, a conventional PG 64-ss, and a gelled asphalt, PG 70-22MAlt. The two LSAMs were compared to their corresponding conventional mixtures: Type 508 and Type 5A. Laboratory performance tests were conducted to characterize the rut susceptibility, durability, moisture susceptibility and permeability of these mixtures.

A three dimensional dynamic finite element procedure was developed during this study. Advanced material models of viscoplasticity and elastoplasticity were incorporated into the 3-D dynamic finite element procedure. This procedure was used to compare the structural performance of two groups of pavements, each with two pavements, one with conventional mixtures and one with the LSAM developed in this study.

The results indicated that the open-graded LSAM developed in this study exhibited better rut-resistance, durability and moisture susceptibility than the
conventional LADOTD Type 508 drainable base mixture, whereas the dense-graded LSAM showed the similar laboratory characteristics to the conventional LADOTD Type 5A base mixture. Similarly, the numerical simulation indicated that the pavement containing open-graded LSAM provided increased structural support when compared to the pavement containing conventional Type 508 drainable mixture, whereas, the pavement containing the dense-graded LSAM showed no appreciable increase in structural support comparing to the pavement containing conventional Type 5A base mixture.
CHAPTER 1.
INTRODUCTION

This document describes the research work and findings of a laboratory characterization and numerical analysis of large stone asphalt mixtures. Chapter 1, the introduction, includes the problem statement and background information for the research project. Chapter 2 presents the objective and scope of the research. Chapter 3 describes the research methodology for the mixture characterization that includes a brief description of test equipment, development of test factorials, the materials used in the study, development of mixture designs, and mixture performance test procedures. Chapter 4 presents the analysis of mixture performance test results. Chapter 5 describes the development of numerical simulation procedures for asphalt pavements, in which asphalt pavement materials were modeled by non-linear visco-plastic models for the three dimensional dynamic finite element analyses. Chapter 6 provides the finite element comparisons of pavements containing large stone asphalt mixtures and conventional asphalt mixtures. Chapter 7 is the summary and conclusion of the whole study.

1.1 PROBLEM STATEMENT

Flexible pavements are widely used in the United States and all over the world. Most flexible pavements consist of asphalt concrete wearing and binder course layers, the base course layer(s) (granular materials, cement or bitumen treated aggregates), and the subgrade (Figure 1.1). Three primary modes of structural distress occur in asphalt pavements: fatigue cracking, permanent deformation and thermal cracking. In addition, hot mix asphalt (HMA) is subjected to moisture damage from stripping, which usually
weakens material integrity and strength, and accelerates the occurrence of fatigue cracking and permanent deformation (rutting).

Figure 1.1. A Typical Pavement Section of Flexible Pavement

1.1.1 Rutting in Asphalt Pavements

Rutting is the deviation from the plane section placed at construction and is the surface evidence of permanent deformation within layers of flexible pavements. It develops gradually with increasing number of load applications, usually appearing as longitudinal depressions in the wheel paths. Pavement uplift may occur along the sides of the rut, but, in many instances, ruts are noticeable after rainfall when the depressions are filled with water. The biggest problem produced by rutting is hydroplaning, a phenomenon in which fast moving vehicles lose contact between the wheels and the pavement surface, resulting in loss of control of the vehicles. In addition, the retention of water on the pavement surface provides the potential for weakening the pavement structure, which leads other
Rutting in asphalt pavement involves two different mechanisms (Figure 1.2) and is a combination of densification (volume change) and repetitive shear deformation (plastic flow with no volume change). Densification can occur in any part of pavement structure including the asphalt surface layer(s), base course(s) and subgrade. Shear stress and strain however, are concentrated near the surface of the pavement. Monismith (1992) stated that shear deformation is the primary cause of rutting.

**Figure 1.2** Mechanisms of Asphalt Pavement Rutting

Significant rutting occurs in the asphalt concrete layers of flexible pavements on many occasions. After a comprehensive national survey of forty-eight heavily traveled flexible pavements in twelve states, Brown and Cross (1992) concluded that rutting primarily occurs in the top three to four inches of the pavement layers, and the thicker the asphalt layer, the deeper the rutting.
Hofstra and Klomp (1972) of the Shell Laboratory in the Netherlands contradict Brown and Cross (1992). After a study with a Laboratory Test Track (LTT, 3.25m in diameter and 0.7m of track width), they conclude that asphalt pavement rutting will be significantly reduced with the increase in thickness of asphalt concrete layer.

It has been generally agreed that rutting reduces road serviceability and causes serious traffic safety problems. As wheel loads and tire pressures of truck traffic on highways have increased in recent years, rutting has become more serious. Many state DOTs pay special attention to minimize rutting when designing and constructing asphalt concrete pavements. The use of large stone asphalt mixture is one way to reduce rut susceptibility of asphalt concrete.

1.1.2 Moisture Damage or Stripping in Asphalt Pavements

Stripping (often called moisture induced damage) is defined as the weakening or eventual loss of the adhesive bond between the aggregate surface and the asphalt cement in an asphalt pavement or mixture in the presence of moisture (water) (Roberts et al, 1994). Although many factors contribute to the degradation of asphalt concrete pavements, water is a key element in the deterioration of the asphalt mixture. According to Terrel and Al-Swailmi (1994), there are three mechanisms by which water can degrade the integrity of an asphalt concrete matrix. These are: 1) loss of cohesion (strength) and stiffness of the asphalt film due to several mechanisms; 2) failure of the adhesion (bond) between the aggregate and asphalt, and 3) degradation or fracture of individual aggregate particles when subjected to freezing. When the aggregate tends to have a preference for absorbing water, the asphalt is “stripped” away (Figure 1.3). Stripping causes premature pavement distress and ultimately the failure of asphalt pavement.
Stripping typically begins at the bottom of the HMA layer and progresses upward. It is difficult to identify this distress without opening up the pavement structure because the surface manifestations can take numerous forms such as excessive rutting, shoving, corrugations, raveling, or cracking. In addition to improved asphalt binder adhesion (such as with an anti-strip agent), appropriate mixture design and adequate drainage in the pavement structure should be maintained in order to prevent stripping.

The use of open-graded large stone asphalt mixtures provides positive drainage system in newly designed highways or in reconstruction of existing roadways. Using a standard ASTM No. 57 size stone or large, it has been found that stability of the drainage layer can be maintained during construction while permitting enough air voids in the mix to carry sufficient quantities of water for drainage.
Louisiana currently uses an asphalt treated drainage blanket as a two inch (50.8-mm) lift under Portland concrete pavements as specified in section 508 of the standard specifications (LaDOTD, 1998). An evaluation of the first drainage blanket placed in 1977 indicated that while performing better than pavements without a drainage blanket, the flow rate of water was much less than the design value and significantly less than the flow rates suggested in the FHWA guidelines (FHWA, 1992). The Louisiana specification uses a relatively small nominal size aggregate that produces air void system that does not permit the prescribed flow rate. Currently, these drainage blankets are not used in full depth asphalt concrete pavement design and no structural support is assigned to this layer.

1.1.3 Proposed Louisiana Solution of LSAM

Louisiana has been experiencing rutting and moisture induced damage for many years. With a hot and humid climate, the prevention of these distresses is among the top priorities during the design and construction of asphalt pavements. It has been proposed that large stone asphalt mixtures (LSAM) be used for improved structural support of asphalt pavements. Work by Ameri-Gaznon and Little (1990) indicates that the maximum shear stress due to pavement loading occurs in a zone approximately two to four inches deep in the pavement system. LSAM base courses should provide increased strength in this zone of the pavement to resist rutting potential. Similarly, an open-graded LSAM will provide excellent permeability and at the same time maintain or increase the structural capacity.
1.2 BACKGROUND AND LITERATURE REVIEW

1.2.1 Large Stone Asphalt Mixtures

Large stone asphalt mixtures (LSAM) are defined as HMA paving mixtures containing maximum aggregate sizes between 25 and 63 mm (1 and 2.5 inch) (TTI, 1997). LSAM may be dense-graded, stone-filled or open-graded. The philosophy of using LSAM is to use stone-on-stone contact of the larger stones in order to minimize plastic deformation under heavy traffic load.

The concept of large stone asphalt mixture was introduced when the Warren Brothers Company in May 1903 applied for and obtained a patent that employed large size aggregates in asphalt mixtures (Khosla and Malpass, 1997). The principle of the patent was that traffic loads would be mainly supported by the interlocking effect of the larger aggregates, and that asphalt and smaller mineral aggregates would only provide binding between bigger aggregates and to waterproof the mixture by filling up the voids (Figure 1.4). Use of the Warren Brothers product required paying royalties and as a result, highway departments chose to specify with smaller top size aggregates to avoid infringing on the patent. Such a practice (the use of small stone) has lasted up to this date, largely due to the fact that smaller aggregates are easier to handle in automated machine processing.

With the rapid increase of traffic loads and volume, premature rutting has occurred more and more frequently in recent years. The concept of stone-on-stone contact in large stone asphalt mixtures seems to provide a solution for rut-resistant, durable heavy-duty mixtures. Open-graded large stone asphalt mixtures were advocated in both concrete and asphalt pavement for this purpose. Using a standard ASTM No. 57 size stone (80%
passing 19-mm sieve) or larger, it was found that stability of the drainage layer could be maintained during construction while permitting enough air voids in the mix to carry sufficient quantities of water for drainage.

1.2.2 Review of Recent Applications of LSAM

Although the use of large stone asphalt mixture is not popular, there has been no paucity of research trials among the US state DOTs and foreign countries for this technology. A relatively complete literature review of this topic can be found in the NCHRP 4-18 reports (TTI, 1997 and Von Quintus et al, 1993). According to the survey conducted by the NCHRP 4-18, thirty out of fifty-two state highway agencies in the US had constructed pavements using large stone asphalt mixtures (Figure 1.5). Among the thirty state agencies that had experience with LSAM, fourteen expressed positive effect, six expressed the same performance, while the rest of ten were not sure about the relative performance of their LSAM as compared to conventional mixtures (Figure 1.6). Almost
all the state agencies expressed the interest of considering LSAM in the future (Table 1.1).

Figure 1.5 Number of LSAM Pavements Constructed Between 1987 and 1997

Figure 1.6 Performance of LSAM as Compared to Conventional Mixtures
Table 1.1 Responses from 52 Highway Specifying Agencies on LSAM (TTI, 1997)

<table>
<thead>
<tr>
<th>Is your agency considering the use of LSAMs in the future?</th>
<th>Yes: 41 agencies</th>
<th>No: 1 agency</th>
<th>No Response: 1 agency</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(79%)</td>
<td>(19%)</td>
<td>(2%)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Are you interested in knowing more about LSAM?</th>
<th>Yes: 50 agencies</th>
<th>No: 1 agency</th>
<th>No Response: 1 agency</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(96%)</td>
<td>(2%)</td>
<td>(2%)</td>
</tr>
</tbody>
</table>

Good performance in rut and fatigue crack resistance of dense-graded LSAM compared with conventional mixture was reported in Kentucky (Anderson et al, 1991), Minnesota (Acott et al, 1989), Nevada (TTI, 1997), North Carolina (Khosla and Malpass, 1997), Ohio (Abdulshafi et al, 1992), Tennessee, Texas, Wyoming, South Africa (TTI, 1994), Australia (Vail, 1993), and the former Soviet Union (Gorelyshev and Kononov, 1972). Anderson et al (1991) reported significantly better rut-resistance parameters with LSAM than those with conventional mixtures on heavily trafficked pavements. They also stated that LSAM with angular sands significantly reduced rut depth when compared with mixes using rounded sand. Vail (1993) reported three LSAM trial projects constructed in Australia and concluded that dense-graded LSAM increased both rut and fatigue resistance.

Several sources reported that confined open-graded LSAM provided exceptional rut-resistance. Good performance of open-graded LSAM has been reported in Arkansas (Von Quintus et al, 1993), Indiana (Fehsenfeld and Kriesch, 1988), Tennessee (Acott, 1988) and Wyoming (Von Quintus et al, 1993). Fehsenfeld and Kriesch (1988) reported that, even though the LSAM base layers were highly permeable, no stripping was evident and asphalt aging was minimal even after eight to eighteen years of service. The NCHRP
4-18 report (TTI, 1997) attributed these properties to the relatively thick asphalt films in LSAM.

It should be noted that while most of the published literature reports good results with LSAM; however, a DOT survey reported in NCHRP 4-18 (TTI, 1997) indicates that the performance of LSAM, particularly the dense-graded LSAM, are mixed. Some LSAM have no rutting after the application of very heavy traffic loads. Other pavement structures containing LSAM experience premature rutting and do not show better performance than the conventional mixtures. Coree et al (1997) reports findings from a full-scale rutting test of LSAM as part of the NCHRP 4-18 study. By comparing the rutting performance of three different LSAM designs, they find that LSAM with poor stone-on-stone contact exhibit the same high rut depths found in conventional binder courses. Therefore, correct mix design is the key to ensure quality, rut-resistant LSAM.

1.2.3 Benefits of LSAM

Just as stated in the Warren Brothers' patent, traffic loads are mainly supported by the large size aggregates interlocking in the asphalt concrete. Coarse aggregate stone-on-stone contact is the key that allows LSAM to outperform many conventional mixtures (Figure 1.7). Many researchers (NAPA, 1988) found that asphalt concrete containing 25-mm (1-inch) maximum size aggregates deformed less when subjected to shear load and were denser and stronger than similar mixtures containing 19-mm (3/4-inch) maximum aggregate. Van der Merwe et al (1989), based on South African experience, stated that LSAM could improve both pavement structural capacity and save construction cost. LSAM normally have lower voids in mineral aggregates (VMA) (TTI, 1994). In other words, LSAM have higher relative volumes of aggregates than conventional mixtures.
This characteristic results in higher densities, lower surface areas and lower optimum asphalt contents of HMA mixtures. Lower asphalt content means lower material costs compared to conventional mixtures (Khalifa and Herrin, 1970). The extensive use of coarser aggregates means less crushing energy is spent, which may lead to lower aggregate cost. In addition, LSAM normally have thicker asphalt films than conventional mixtures, which reduces the susceptibility to moisture damage and age hardening (TTI, 1997).

![Stone-on-Stone Contact](Image)

**Figure 1.7 Stone-on-Stone Contact of LSAM**

Khosla et al (1997) conducted a study on LSAM in North Carolina. In that study, a 25-mm (1-inch) dense-graded LSAM binder mixture was compared with a conventional North Carolina DOT H-Binder mixture through laboratory performance tests. They concluded that their LSAM out-performed the conventional mixture in rut-resistance while retaining similar fatigue crack resistance. Davis (1989) stated that the bearing

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capacity of a particular mix could be increased by more than four times when the top size aggregate was changed from 19 to 37 mm (3/4 to 1.5 inch). It was pointed out that LSAM were less sensitive to changes in asphalt cement properties, asphalt cement content, and changes in temperature. Abdulshafi et al (1992) reported a two to three times increase of unconfined compressive strength and significantly lower creep and much higher resilient modulus and fatigue resistance for LSAM when compared with conventional mixtures.

1.2.4 Latest Development of LSAM Research

Most existing LSAM projects in the US were designed by the modified 152-mm (6-inch) Marshall procedure developed by Kandhal (1990). This method produces a cylindrical specimen of 152-mm (6-inch) in diameter by 85-mm (3.4-inch) in height. It was recommended for mixtures containing a nominal maximum aggregate size of 37.5-mm (1.5-inch). Kandhal increased the Marshall hammer mass and number of blows to achieve the same compaction energy per unit volume as in the conventional 102-mm (4-inch) diameter by 63-mm (2.5-inch) high Marshall specimen. Although this procedure was reasonably adequate for determining optimum asphalt content (Anderson et al. 1991), the Marshall mix design method is an empirical procedure that does not address the fundamental engineering properties of asphalt mixtures and Marshall stability is not a good indicator of rutting potential. In addition, Kandhal (1990) reported that during the compaction of the 152-mm (6-inch) Marshall specimens, 75 to 112 blows per face of the specimen often resulted in fracture of the larger aggregates in the mixture.

According to the literature review conducted by the NCHRP 4-18 (TTI, 1997), the Southern African Bitumen and Tar Association (SABITA) conducted a comprehensive
laboratory and field research program over several years, which resulted in an LSAM design manual. The manual recognized that the strength and rut-resistance of LSAM were achieved from coarse aggregate interlocking. Durability was enhanced by thicker asphalt films resulted from using large aggregate. To minimize stripping, the compacted mixtures must either be impermeable or open-graded with interconnected voids so that excessive pore water pressure cannot develop. Mixture designs that trap water should be avoided. This design procedure recommended a 6-inch (150-mm) diameter, rotating-base Marshall hammer with six depressions in the face that provided a certain kneading action. Indirect tensile strength (ITS) and dynamic creep modulus were introduced for mix characterization.

North Carolina Department of Transportation and North Carolina State University conducted a LSAM study during 1993 and 1997. In that study, they designed a dense-graded LSAM with top aggregate size of 25-mm (1-inch) based on modified Marshall mix design method (Khosla and Malpass, 1997). The LSAM was compared with a conventional North Carolina H-Binder mixture through laboratory indirect tensile resilient modulus ($M_r$) tests, axial incremental creep tests, and indirect tensile fatigue tests. The pavement analysis software, VESYS 3AM, was employed to predict performance of the pavement test section on US Highway 70. They concluded that LSAM pavement had lower permanent deformation than the conventional H-Binder mixture under most test conditions. However, at high temperatures and long loading times, the conventional mixture had slightly less permanent deformation than the LSAM. The LSAM had a longer fatigue life than the conventional mixture, but only at initial
strain values of greater than $5 \times 10^{-4}$ mm/mm. At lower initial strain values, the conventional mixture had a longer fatigue life.

The US Army Corps of Engineers conducted a study for the US Air Force to analyzed the effects of increasing the maximum aggregate size of an asphalt mix from 19-mm to 25-mm in order to accommodate the increased tire pressure of modern aircraft (Regan, 1987). The study examined the tensile strength, direct shear, axial creep and aging using different compaction methods and asphalt binders. The gyratory testing machine (GTM) was used with pressures of 0.7, 1.4, 2.1 and 2.8 MPa. Their study concluded that the 25-mm mixtures out-performed the 19-mm mixtures at higher compaction efforts. The study also found that it was the asphalt binder type, instead of top aggregate size, that most influenced the durability of mixtures.

Perhaps the most comprehensive and recent study of LSAM is Project NCHRP 4-18, conducted by the Texas Transportation Institute (TTI, 1997). In that study, a comprehensive literature review and survey of 52 highway agencies about the status of application as well as the performance of LSAM was conducted. The survey found that, while most highway agencies showed interests in applying LSAM in highway construction, the actual experience of design and construction of LSAM had been very limited. Among the constructed LSAM pavements, the performance had been mixed. Some of them (LSAM) exhibited exceptionally good rut-resistance, while the performance of others was similar to that of conventional mixtures. The NCHRP 4-18 report attributed the poor performance to the inadequate mix design. In an effort to overcome this shortcoming, the NCHRP 4-18 study developed a mixture design guide for LSAM. The report recommended the use of Superpave Gyratory Compactor (SGC) or
rolling wheel compaction (AASHTO PP3) during mix design. The six-inch (150-mm) Marshall hammer should only be used when the other two compaction means are unavailable. Stone-on-stone contact was ensured by keeping the voids in coarse aggregates (VCA) above eighty to ninety percent. A number of mixture characterization tests, such as uniaxial creep and Superpave repetitive shear at constant height (RSCH), were recommended. Field projects were constructed as a part of the study to validate the laboratory results. The report concluded that when properly designed, LSAM would perform better than conventional mixtures in rut-resistance (TTI, 1997).

1.2.5 Numerical Simulations of Pavement Structure

Burmister (1943) solved a two-layer linear elastic system problem for stress and strain distribution under a surface load. He used the stress and displacement equations of elasticity for a three-dimensional problem in his solution by assuming the Poisson's ratio to be either 0 or 0.5. Based on Burmister's method, Acum and Fox (1951) presented exact solutions for strength and deflection at the center-line of three-layer systems. Later a number of computer programs such as BISAR and ELSYM5 were developed to calculate stress and strain distribution in a pavement system based on a modified form of the Burmister method.

Layered elastic analytical solutions provided a basis for pavement structural design. However, they over-simplified the material behavior by assuming linear elasticity. Huang (1967) and other researchers (Barksdale, 1967) reformulated the above solutions by introducing viscoelastic models for the asphalt layers of the system. This improves the analytical procedure considerably. Later software such as VESYS and

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MICH-PAVE widely adopted viscoelastic models for the asphalt concrete and linear or nonlinear elastic models for the base course and subgrade materials (Kenis, 1977).

Viscoelasticity and nonlinear elasticity improved the layered elastic solution, but they still failed to capture an important characteristic of paving materials, the plastic behavior under traffic loads. Yandell (1971) and Smith (1984) used elasto-plastic models of the pavement system and introduced a numerical procedure of Mechano-Lattice Analysis (Smith, 1986). They applied the procedure for a number of flexible pavement analyses in the U.S., South Africa and Australia. Chan and other researchers applied elasto-plastic theory for a finite element analysis of a flexible pavement base course rutting study in Nottingham, UK (Chan et al, 1989).

While most analytical methods assumed two dimensional axisymmetrical conditions, Zaghloul and White (1993) applied three dimensional finite element analyses and made possible dynamic analyses to simulate real traffic loads. They used a visco-elastic model for the asphalt concrete, an extended Drucker-Prager model for the granular base course and the Cam Clay model for the clay subgrade soils. The three-dimensional finite element analysis is performed using the commercial finite element software, ABAQUS.

Zaghloul and White's research improved the analytical procedure significantly, but they failed to address one important aspect of asphalt concrete, the visco plasticity. Seibi and Sharma at the Pennsylvania State University (PSU) developed an elastic visco-plastic constitutive relation for asphalt concrete under high rates of loading (Seibi, 1993). The model adds the rate dependent characteristics to the traditional Drucker-Prager plastic model.
1.3 LIMITATION OF EXISTING PROCEDURES

The literature review shows that most LSAM design procedures today still base mix
design on empirical procedures, such as the modified Marshall method. As in most
mixture design, there is little or no mixture performance analysis involved during the
design procedure. A few researchers conducted limited laboratory performance tests on
mixtures they studied and the results varied. The NCHRP Project 4-18 did an excellent
job in summarizing the status of the application of LSAM and in introducing a
performance-based LSAM design procedure. However, at the time of the study, the
Superpave design and analysis procedure was in the development stage. For example,
though the Superpave gyratory compactor (SGC) was recommended to compact
laboratory specimens, the study actually used the Texas gyratory compactor, which uses a
different angle and pressure from the SGC. The types of laboratory performance tests
conducted in that study were also limited. Noticeably, permeability, a very important
property of open-graded LSAM, was not studied in any of the previous research.
Although the North Carolina study conducted a limited structural analysis of LSAM
through VESYS 3AM simulation, none of the other studies conducted numerical analyses
of LSAM pavements to compare the structural performance of LSAM to that of
conventional mixtures. None of the previous research conducted comprehensive
laboratory performance studies on LSAM.

1.4 SIGNIFICANCE OF THIS RESEARCH

The research conducted in this study was designed to overcome the limitations of the
current procedures used to design and analyze LSAM. Table 1.2 lists a summary
comparison of research elements for previous research studies and the research conducted
in this study. This research was aimed at establishing the fundamental engineering properties of LSAM for potential use in Louisiana. Superpave gyratory compactor (SGC) was the standard compaction device used in this study. A dense-graded 37.5-mm (1.5-inch) LSAM was designed according to the Superpave mix design protocol and was intended to replace the Louisiana DOTD conventional Type 5A base mixture. An open-graded LSAM was designed both to provide additional structural support and excellent drainage for the pavement structure. The open-graded LSAM was recommended as a replacement for the Louisiana DOTD conventional Type 508 drainable mixture.

The 3-D dynamic viscoplastic finite element analysis developed in this study was able to predict the pavement responses under dynamic traffic loads.

Table 1.2 Comparisons of Research Elements for Previous Research and This Research

| Research Elements                                    | NCHRP 4-18 | North Carolina | WES | South African | This Research
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix Design Compaction</td>
<td>Texas Gyratory Compactor</td>
<td>6-inch Marshall</td>
<td>GTM</td>
<td>Rolling Marshall Hammer</td>
<td>Superpave Gyratory Compactor</td>
</tr>
<tr>
<td>Quantify Stone-on-Stone Contact</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>Resilient Modulus (M&lt;sub&gt;R&lt;/sub&gt;)</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>Axial Creep</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Indirect Tensile Strength (ITS)</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Indirect Tensile Creep</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>Repetitive Shear at Constant Height (RSCH)</td>
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<td>No</td>
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<td>No</td>
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<td>Moisture Susceptibility</td>
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<td>Yes</td>
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<td>Structural Analysis</td>
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<td>VESYS</td>
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</table>
CHAPTER 2.
OBJECTIVE AND SCOPE

2.1 OBJECTIVE

The objectives of this study were as follows:

- Design two large stone asphalt mixtures (LSAM) as possible alternatives to the conventional Louisiana DOTD Type 5A base mixture and Type 508 open-graded drainable base mixture. The dense-graded large stone (37.5-mm) Superpave mixture and an open-graded large stone asphalt mixture were designed to ensure the stone-on-stone contact and other volumetric criteria are satisfied;
- Perform fundamental engineering property tests on the two LSAM mixtures and the Louisiana DOTD conventional Type 5A base mixture and Type 508 drainable base mixture;
- Develop test equipment and procedures to measure the permeability of both open-graded and dense-graded asphalt mixtures;
- Establish correlations between the volumetric properties and permeability of the asphalt mixtures included in this study;
- Develop a visco-plastic model that can be used to calculate the stress and strain response of asphalt mixtures in the pavement structures;
- Develop a three dimensional dynamic finite element procedure that can reflect the dynamic responses of asphalt pavement under the traffic loads;
- Conduct finite element analysis of an existing pavement to validate the visco-plastic model developed;
• Conduct numerical simulation of four pavements containing the two LSAM designed in this study and two Louisiana conventional mixtures to evaluate the added structural support of the LSAM mixtures.

2.2 SCOPE

The scope of this study included the evaluation of two large stone asphalt mixtures along with their conventional Louisiana DOTD mixtures. The large stone asphalt mixtures were open-graded and dense-graded, whereas, the conventional mixes were LaDOTD Type 508 drainable base mix and Type 5A base mix. Each type of LSAM had three types asphalt cement to study the effects of binder on the performance of large stone asphalt mixtures. Table 2.1 presents the eight mixtures considered in this study.

Table 2.1 Eight Mixtures Designed in this Study

<table>
<thead>
<tr>
<th>Mix Type</th>
<th>Mixture</th>
<th>Symbol</th>
<th>Aggregate</th>
<th>Asphalt Cement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Open-graded Mixes</td>
<td>Type 508 Drainable Base Mix</td>
<td>DT-P</td>
<td>Limestone</td>
<td>PG 70-22M</td>
</tr>
<tr>
<td></td>
<td>Open-graded LSAM</td>
<td>OG-P</td>
<td>Limestone</td>
<td>PG 70-22M</td>
</tr>
<tr>
<td></td>
<td></td>
<td>OG-MG</td>
<td>Limestone</td>
<td>PG 70-22MAlt</td>
</tr>
<tr>
<td></td>
<td></td>
<td>OG-A</td>
<td>Limestone</td>
<td>PG 64-22</td>
</tr>
<tr>
<td>Dense-graded Mixes</td>
<td>Type 5A Base Mix</td>
<td>A-P</td>
<td>Limestone</td>
<td>PG 70-22M</td>
</tr>
<tr>
<td></td>
<td>Dense-graded LSAM</td>
<td>L-P</td>
<td>Limestone</td>
<td>PG 70-22M</td>
</tr>
<tr>
<td></td>
<td></td>
<td>L-MG</td>
<td>Limestone</td>
<td>PG 70-22MAlt</td>
</tr>
<tr>
<td></td>
<td></td>
<td>L-A</td>
<td>Limestone</td>
<td>PG 64-22</td>
</tr>
</tbody>
</table>

The laboratory performance of the mixtures was characterized through fundamental engineering property tests. The engineering property tests conducted in this study included:

• Indirect tensile resilient modulus at 4, 25 and 40 °C;
• Indirect tensile strength and strain at 25 °C;
• Axial creep at 40 °C;
• Indirect tensile creep at 40 °C;

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- Frequency sweep at constant height;
- Repetitive shear at constant height;
- APA rut at 60 °C;
- Permeability;
- Moisture susceptibility;
- Draindown.

In order to utilize the results from fundamental mixture characterization to predict pavement performance, a 3-D dynamic finite element procedure was developed. In the numerical analysis, a visco-plastic model was developed to describe the stress-strain relationship of asphalt mixtures.

A test pavement section from the Louisiana Pavement Test Facilities (LPTF) was selected to validate the material models and the finite element procedures. The numerical simulation of the Accelerated Loading Facility (ALF) test lane was compared to field stress and strain measurement. After model validation, two comparable groups of pavement structures, open-graded LSAM versus conventional Louisiana DOTD Type 508 drainable base and the 37.5-mm dense-graded LSAM versus conventional Louisiana DOTD Type 5A base mixture, were analyzed for their responses to the similar dynamic traffic loading.
CHAPTER 3.
METHODOLOGY

This chapter describes the method and procedures applied in this study. The first part of the section 3.1 describes the equipment and facilities used to conduct laboratory test and numerical analysis of LSAM. The equipment includes specimen preparation facilities, mixture performance test facilities, and the computing facility. Mix specimen preparation facilities include two Superpave gyratory compactors. Mixture performance test facilities include the MTS, Cox & Son 7000 Superpave Shear Tester, Cox & Son Axial Testing System, Asphalt Pavement Analyzer, and LTRC Dual Mode Permeameter. Section 3.2 presents the development of test factorials of the study. Section 3.3 describes the materials (aggregates and asphalt binder) used in this study. Section 3.4 describes the development of mixture design of the 37.5-mm (1.5-inch) Superpave LSAM and the 37.5-mm (1.5-inch) open-graded LSAM. Section 3.5 describes the mixture performance test procedures.

3.1 FACILITIES

3.1.1 Specimen Preparation Facility

Asphalt mixture specimens were first mixed in a mixing bowl (Figure 3.1), mixing bucket (Figure 3.2) or a mini-pugmill mixer (Figure 3.3). Two Superpave gyratory compactors (SGC), a Pine Instrument Model AFGC125X (Figure 3.4) and a Troxler Model 4140 (Figure 3.5), were used to compact the specimens.

The Superpave gyratory compactor (SGC) is a laboratory compaction device used in the Superpave mix design system. The SGC mold is 150-mm in diameter. The SGC consists of the following main components as shown in Figure 3.6:

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

Figure 3.1 Mixing Bowl

Figure 3.2 Mixing Bucket
Figure 3.3 PTI Double Pugmill Mixer

Figure 3.4 Pine Instrument Superpave Gyratory Compactor
Figure 3.6 Components of Superpave Gyratory Compactor (Asphalt Institute, 1994)

Figure 3.7 shows the configuration of a SGC mold, which has an inside diameter of 150 mm and a nominal height of 250 mm. A base plate fits in the bottom of the mold to confine the specimen during compaction.

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3.1.2 Mixture Performance Test Facility

Mixture performance testing includes loading and non-loading tests. Non-loading testing in this study includes only the permeability test, which was performed in the dual-mode permeameter developed during this study (described in detail in section 3.6). The loading tests can be divided into: 1) axial loading; 2) diametrical loading; 3) shearing; and 4) traffic simulating loading. Axial and diametrical loading tests were performed on the Material Testing System (MTS) 810 and the Cox and Sons CS7500 Axial Testing and Environmental System. Shearing tests were performed on the Cox and Sons CS7000 Superpave Shear Tester. An Asphalt Paving Analyzer was used to perform loaded wheel rut testing.

3.1.2.1 Materials Testing System (MTS Model 810)

The LTRC’s Material Testing System (MTS Model 810, Figure 3.8) device is a closed-loop controlled servo-hydraulic test system. The system is equipped with an
environmental chamber. The machine is rated 244 kN (55,000 pounds). Its state-of-the-art digital controller, that is operated under Microsoft Windows NT4.0 and MTS Test Star software, conducts the data acquisition and equipment control. Figure 3.9 shows a schematic representation of the test system. The closed-loop system consists of a computer and a digital controller acting as the controlling unit over a servo valve, a hydraulic actuator, and the test specimen. The initial loading signal is sent from the digital controller to the servo valve, which applied hydraulic pressure on the specimen, from which the linear variable differential transformers (LVDTs) or the force sensors return the feedback signal to the digital controller. The digital controller compares this feedback signal with the control signal and performs adjustments as necessary.

Figure 3.8 Material Testing System (MTS 810)
If the control signal is commanding a given force (sometimes referred to as "in force control"), the feedback signal is from the force sensor.

If the control signal is commanding a given displacement, the feedback signal is from the LVDT.

Figure 3.9 Closed-loop Controlled Servo-hydraulic Test System

3.1.2.2 Cox and Son CS7500 Axial Testing System

The LTRC's Cox and Sons CS7500 Axial Testing and Environmental System is a versatile, fully automated, single axis, closed-loop hydraulic testing system specifically designed to perform tests on soils and asphalt concrete mixtures over a wide range of stresses and frequencies. The equipment has sufficient flexibility to perform special or standard tests.

The system software features custom test templates that automatically perform SHRP and AASHTO tests, analyze the results and present the data in the report-ready format.

The system software incorporates standard test and data acquisition templates to perform tests that may be required for various research projects including the following tests:

- Dynamic (sine, square and triangular wave);
• Creep;
• Repetitive loading (haversine);
• Constant rate (ramp);
• Fatigue;
• Random loading;
• Custom software templates for other tests are made available for tests that fall within the static and dynamic capabilities of the system.

Figure 3.10 Cox and Son CS7500 Axial Testing and Environmental System

3.1.2.3 Cox and Son CS7000 Superpave Shear Tester

The Superpave Shear Tester (SST) used in this study is a Cox and Sons Model CS7000 (Figure 3.11) manufactured by Cox and Sons, Inc. in Colfax, CA. The Superpave shear test system is used to perform nearly all of the load-related performance tests including: volumetric test, uniaxial strain test, simple shear test at constant height, frequency sweep
test at constant height, repeated shear test at constant stress ratio, and repeated shear test at constant height. The SST system includes the following components:

- loading device (load actuators of hydraulic system);
- specimen deformation measurement equipment (testing apparatus);
- environmental chamber;
- control and data acquisition system.

The following table (Table 3.1) contains the minimum requirements of the system:

Table 3.1. Minimum SST System Requirements

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Range</th>
<th>Resolution</th>
<th>Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load</td>
<td>0-31000 N</td>
<td>2</td>
<td>5 N</td>
</tr>
<tr>
<td>Confining Pressure</td>
<td>0-1000 KPa</td>
<td>0.5 KPa</td>
<td>1.0 KPa</td>
</tr>
<tr>
<td>Vertical or Axial Deformation</td>
<td>0-5 mm</td>
<td>0.0025 mm</td>
<td>0.005 mm</td>
</tr>
<tr>
<td>(Constant Ht., Freq. Sweep)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Horizontal Deformation</td>
<td>0-0.050 mm</td>
<td>0.001 mm</td>
<td>0.002 mm</td>
</tr>
<tr>
<td>(Freq. Sweep, Repetitive Shear)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Radial Deformation</td>
<td>0-1 mm</td>
<td>0.005 m</td>
<td>0.010 mm</td>
</tr>
<tr>
<td>Temperature</td>
<td>-10 to 80°C</td>
<td>0.25°C</td>
<td>0.5°C</td>
</tr>
</tbody>
</table>

Figure 3.11 CS7000 Superpave Shear Tester
3.1.2.4 Asphalt Pavement Analyzer

The Asphalt Pavement Analyzer (APA, Figure 3.12), is a multi-functional Loaded Wheel Tester (LWT), used for evaluating the susceptibility to permanent deformation (rutting), fatigue cracking, and moisture damage of asphalt mixes. It features controllable wheel load and contact pressure that are representative of actual field conditions. Triplicate beam specimens, or six cylindrical (gyratory or roadway cores) specimens in three specially designed specimen molds can be tested under controllable temperature and under dry or submerged-in-water environments.

LTRC’s APA has the Automated Asphalt Pavement Analyzer (AAPA) Software, which allows the user to obtain rutting and fatigue measurements using a personal computer to record and store data. The data acquisition system can take up to five measurements during a single pass over a rectangular specimen and up to two measurements during a single pass over a cylindrical specimen (vibratory, gyratory, marshall specimens, or field cores). This information is stored on the PC and subsequently used for data analysis. The results will not only include rutting at a specific number of cycles but also allows the computation of the rate of change of deformation.

3.1.2.5 LTRC Dual Mode Permeameter

A dual mode permeameter (Figure 3-13) was developed in this study. The initial device was purchased from the Virginia LAB Supply Corporation. Modifications were made to the original device so that it can be used to measure hydraulic conductivity of different materials from dense-graded low permeable mixtures to open-graded drainable mixes under both constant and falling head modes.
The LTRC Dual mode permeameter consists of a flexible wall cell, a top reservoir tube, a bottom constant-head drainage tube, a flexible wall pump, two pressure transducers and a data acquisition system (Figure 3.14). Two pressure transducers
installed at the top and bottom of the specimen give accurate readings of the hydraulic head difference during the test. Data acquisition makes it possible to have continuous readings during a falling head test so that the test can be conducted even at very high flow rates (for drainable mixes). The specimen is placed in an aluminum cell with an anti-scratch rubber membrane that is clamped tightly at both end of the cylindrical cell into which the specimen is placed. A vacuum is applied between the membrane and the cell to facilitate the installation of the specimen. During the test, a confining pressure of up to 103.5 kPa (15 psi) is applied between the membrane and the cell to prevent water from short-circuiting around the perimeter of the specimen. Two different top reservoir tubes have been designed for different materials: a 25-mm (1-inch) diameter tube is used for dense graded or less permeable materials and a 75-mm (3-inch) diameter tube is used for highly permeable materials, both tubes are 90-mm (3-feet) long.

A vacuum is applied on the top of the reservoir tube before the test to saturate the specimen.

Figure 3.14 Diagram of LTRC Dual Mode Permeameter
3.1.3 Computational Facility

Numerical simulation using a finite element analysis was performed on a Sun Microsystem Ultra 10 Workstation. The workstation is powered with a 333 MHz UltraSPARC Iii CPU, which offers the numerical computation speed 14.1 SPECint95 and 18.3 SPECfp95 (One SPEC95 is defined as the speed of a Sun SPARCstation 10/40 with 128 MB of memory). The following is a brief description of the main features of the machine:

- CPU: 333 MHz UltraSPARC Iii, 2MB cache;
- RAM: 512 MB DIMM (50ns);
- Storage: 10 GB SCSI HD, 32xCD, 8-mm Tape Backup;
- Graphics: 24-bit on-board PGXx24, Creator3D graphics, 21” Display;
- Network: 10/100BASE-T Ethernet;
- OS: Sun Solaris 2.6.

The commercial finite element software, ABAQUS version 5.8 was installed on the Ultra 10 workstation.

3.2 MATERIALS

3.2.1 Asphalt Binder

Three types of asphalt cement are included in this study: an SB polymer modified asphalt cement meeting Louisiana Superpave performance grading specification of PG 70-22M, a conventional asphalt cement meeting Louisiana DOTD specification of PG 64-22, and a gelled asphalt cement meeting Louisiana DOTD Superpave performance grading specification of PG 70-22MAlt. One hundred percent crushed siliceous limestone will be used for the design of large stone asphalt mixtures as well as the laboratory samples of
conventional mixtures. Table 3.2 presents the asphalt cement properties and the specifications of the Louisiana Department of Transportation and Development (LaDOTD).

Table 3.2 LaDOTD Performance Graded Asphalt Cement Specification & Test Results

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Rotational Viscosity @ 135°C, Pa.s, TP 48</td>
<td>3.0</td>
<td>1.05</td>
<td>3.0-</td>
<td>2.85</td>
<td>3.0</td>
<td>0.463</td>
</tr>
<tr>
<td>Dynamic Shear, 10 rad/s, G*/Sin Delta, kPa, TP 5</td>
<td>1.0+@64°C 3.7</td>
<td>3.7</td>
<td>1.0+@64°C 2.9</td>
<td>2.9</td>
<td>1.0+@64°C 2.1</td>
<td></td>
</tr>
<tr>
<td>Flash Point, °C, T 48</td>
<td>232+</td>
<td>305</td>
<td>232+</td>
<td>310</td>
<td>232+</td>
<td>295</td>
</tr>
<tr>
<td>Solubility, %, T 44</td>
<td>99.0+</td>
<td>99.6</td>
<td>99.0+</td>
<td>99.6</td>
<td>99.0+</td>
<td>99.6</td>
</tr>
<tr>
<td>Softening Point, Ring &amp; Ball, °C, T 53</td>
<td>N/A</td>
<td>---</td>
<td>70.0+</td>
<td>71.1</td>
<td>N/A</td>
<td>---</td>
</tr>
<tr>
<td>Force Ductility, 4°C, 5 cm/min, 30 cm elongation, kg, T 300</td>
<td>0.234</td>
<td>0.35</td>
<td>N/A</td>
<td>---</td>
<td>N/A</td>
<td>---</td>
</tr>
<tr>
<td>Mass loss %, T 240</td>
<td>1.00-</td>
<td>0.10</td>
<td>1.00-</td>
<td>0.07</td>
<td>1.00-</td>
<td>0.03</td>
</tr>
<tr>
<td>Elastic Recovery, 25°C, 10cm elongation, %, T 301</td>
<td>40+</td>
<td>85</td>
<td>N/A</td>
<td>---</td>
<td>40+</td>
<td>25*</td>
</tr>
<tr>
<td>Dynamic Shear, @ 25°C, 10 rad/s, G* Sin Delta, kPa, TP 5</td>
<td>5000-</td>
<td>3175</td>
<td>5000-</td>
<td>---</td>
<td>5000-</td>
<td>3628</td>
</tr>
<tr>
<td>Bending Beam Creep Stiffness, S, Mpa @ -12°C, TP1</td>
<td>300-</td>
<td>99</td>
<td>300-</td>
<td>---</td>
<td>300-</td>
<td>238</td>
</tr>
<tr>
<td>Bending Beam Creep Slope, m value, @ -12°C, TP1</td>
<td>0.300+</td>
<td>0.452</td>
<td>0.300+</td>
<td>---</td>
<td>0.300+</td>
<td>0.310</td>
</tr>
</tbody>
</table>

* Did not meet Elastic Recovery Criteria

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3.2.2 Aggregates

The aggregates used in this study were siliceous limestone provided by the Vulcan Materials Company from the Reed quarry in Kentucky. Table 3.3 presents the aggregate gradations as provided by the supplier and the results of sieve analysis to verify the gradations.

Table 3.3 Gradations from Stockpile Materials.

<table>
<thead>
<tr>
<th>Sieve</th>
<th>No. 3 Vulcan Reed Limestone</th>
<th>No. 57 Vulcan Reed Limestone</th>
<th>No. 8 Vulcan Reed Limestone</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pass (%)</td>
<td>Spec (%)</td>
<td>Pass (%)</td>
</tr>
<tr>
<td>63.5-mm (2.5&quot;)</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>51-mm (2&quot;)</td>
<td>98.5</td>
<td>90-100</td>
<td>100</td>
</tr>
<tr>
<td>37.5-mm (1.5&quot;)</td>
<td>60.6</td>
<td>50-80</td>
<td>100</td>
</tr>
<tr>
<td>25.4-mm (1&quot;)</td>
<td>9.2</td>
<td>10-35</td>
<td>97.6</td>
</tr>
<tr>
<td>19-mm (3/4&quot;)</td>
<td>2.7</td>
<td>0-10</td>
<td>77.7</td>
</tr>
<tr>
<td>12.7-mm (5/8&quot;)</td>
<td>1.4</td>
<td>0-5</td>
<td>35.3</td>
</tr>
<tr>
<td>9.5-mm (3/8&quot;)</td>
<td>1.2</td>
<td>14.3</td>
<td>14</td>
</tr>
<tr>
<td>4.75-mm (No. 4)</td>
<td>1.0</td>
<td>1.9</td>
<td>0-10</td>
</tr>
<tr>
<td>-4.75-mm (-No.4)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Aggregate properties provided by the manufacturer are presented in Table 3.4. In addition to the supplier provided data, flat and elongation test was performed for a sample of aggregates that has the same gradation as the designed mix (for aggregates that are retained at No. 4 sieve and above). The test method is referred to ASTM D4791-95, "Standard Test Method for Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate," (ASTM, 1998). It is found that 20.3% aggregates exceeded the 1:3 criterion, whereas, 1.7% exceeded the 1:5 (Table 3.5).
### Table 3.4 Aggregate Properties Provided by the Manufacturer

<table>
<thead>
<tr>
<th>Aggregate Properties</th>
<th>Test Protocols</th>
<th>Vulcan Reed Siliceous Limestone</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>No. 3</td>
</tr>
<tr>
<td>Bulk (SSD) Spec.</td>
<td>ASTM C 127</td>
<td>2.69</td>
</tr>
<tr>
<td>Gravity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Absorption</td>
<td>ASTM C 127</td>
<td>0.4</td>
</tr>
<tr>
<td>LA Abrasion</td>
<td>ASTM C 131</td>
<td>22.0</td>
</tr>
<tr>
<td>Sulfate Soundness</td>
<td>ASTM C 88</td>
<td>0.3</td>
</tr>
<tr>
<td>Formation</td>
<td></td>
<td>Fort Payne</td>
</tr>
<tr>
<td>Unit Wt. (lb/ft³)</td>
<td>Loose</td>
<td>ASTM C 29</td>
</tr>
<tr>
<td></td>
<td>Rodded</td>
<td>ASTM C 29</td>
</tr>
</tbody>
</table>

### Table 3.5 Results from the Flat and Elongation Test

<table>
<thead>
<tr>
<th>Ratio</th>
<th>Pass %</th>
<th>Flat %</th>
<th>Elongated %</th>
<th>Flat &amp; Elongated %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:3</td>
<td>79.7</td>
<td>3.9</td>
<td>16.4</td>
<td>20.3</td>
</tr>
<tr>
<td>1:5</td>
<td>98.3</td>
<td>0</td>
<td>0</td>
<td>1.7</td>
</tr>
</tbody>
</table>

### 3.3 DEVELOPMENT OF MIXTURE DESIGN

An asphalt mixture design is an optimization procedure that determines the optimum aggregate gradation and asphalt content for compaction efforts. Table 3.6 presents the job mix formulae for the four types of mixtures designed in this study. Specific details of section of mixture gradations, measurement of specific gravity of large stone asphalt mixtures, determination of degree of stone-on-stone contact, as well as volumetric design procedure for each individual mixtures are discussed in the following sections.

#### 3.3.1 Mixture Gradations

Mixture gradations determine the aggregate proportions of different sieve sizes. Proper aggregate gradations ensure robust aggregate skeletons in HMA mixtures. For large stone asphalt mixture, NCHRP 4-18 (TTI, 1994) report suggests to select gradations should always ensure adequate stone-on-stone contact.
Table 3.6 Job Mix Formulae

<table>
<thead>
<tr>
<th>Job Mix Formula</th>
<th>Mix</th>
<th>OG-LSAM</th>
<th>Type 508</th>
<th>Sup-LSAM</th>
<th>Type 5A</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AC%</td>
<td>2.5%</td>
<td>2.3%</td>
<td>3.8%</td>
<td>3.5%</td>
</tr>
<tr>
<td>Aggregates</td>
<td></td>
<td>70%#3LS</td>
<td></td>
<td>60%#3LS</td>
<td>37%#5LS</td>
</tr>
<tr>
<td></td>
<td></td>
<td>13%#57LS</td>
<td></td>
<td>20%#57LS</td>
<td>11%#78LS</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10%#8LS</td>
<td>100%</td>
<td>21%#11LS</td>
<td>21%#11LS</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7%CS</td>
<td></td>
<td>3%HL</td>
<td>12%CS</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>19%RAP*</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Gradation %Passing</th>
<th>Mix</th>
<th>OG-LSAM</th>
<th>Type 508</th>
<th>Sup-LSAM</th>
<th>Type 5A</th>
</tr>
</thead>
<tbody>
<tr>
<td>63.5 mm (2.5&quot;)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>50.8 mm (2&quot;)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>37.5-mm (1.5&quot;)</td>
<td>79</td>
<td>100</td>
<td>86</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>25-mm (1&quot;)</td>
<td>43.7</td>
<td>98</td>
<td>65.24</td>
<td>96</td>
<td></td>
</tr>
<tr>
<td>19 mm (3/4&quot;)</td>
<td>30.9</td>
<td>78</td>
<td>52.4</td>
<td>84</td>
<td></td>
</tr>
<tr>
<td>12.5 mm (1/2&quot;)</td>
<td>23.0</td>
<td>35</td>
<td>34.1</td>
<td>69</td>
<td></td>
</tr>
<tr>
<td>9.5 mm (3/8&quot;)</td>
<td>18.3</td>
<td>14</td>
<td>25.8</td>
<td>61</td>
<td></td>
</tr>
<tr>
<td>No. 4</td>
<td>9.4</td>
<td>4</td>
<td>21.92</td>
<td>46</td>
<td></td>
</tr>
<tr>
<td>No. 10</td>
<td>6.6</td>
<td>2</td>
<td>20</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>No. 40</td>
<td>3.2</td>
<td>15.6</td>
<td>19</td>
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<td></td>
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<tr>
<td>No. 80</td>
<td>0.4</td>
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<td>8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 200</td>
<td>0.1</td>
<td>5.4</td>
<td>5.3</td>
<td></td>
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</table>

<table>
<thead>
<tr>
<th></th>
<th>Gmb</th>
<th>Gmm</th>
<th>VMA</th>
<th>VFA</th>
<th>Air Voids</th>
<th>Film</th>
<th>Thickness</th>
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<tr>
<td>Mix</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>OG-LSAM</td>
<td>1.915</td>
<td>2.592</td>
<td>30.7</td>
<td>14.9</td>
<td>26.2</td>
<td>30.2</td>
<td></td>
</tr>
<tr>
<td>Type 508</td>
<td>1.760</td>
<td>2.584</td>
<td>36.0</td>
<td>10.8</td>
<td>32.1</td>
<td>38.9</td>
<td></td>
</tr>
<tr>
<td>Sup-LSAM</td>
<td>2.459</td>
<td>2.546</td>
<td>11.7</td>
<td>70.9</td>
<td>3.5</td>
<td>7.3</td>
<td></td>
</tr>
<tr>
<td>Type 5A</td>
<td>2.410</td>
<td>2.507</td>
<td>12.3</td>
<td>67.5</td>
<td>4.0</td>
<td>7.07</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>%Gmm@Ni</th>
<th></th>
<th></th>
<th></th>
<th>%Gmm@Ni</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>OG-LSAM</td>
<td>84.5</td>
<td></td>
<td></td>
<td></td>
<td>97.7</td>
<td></td>
</tr>
<tr>
<td>Type 508</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sup-LSAM</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type 5A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: LS - Limestone; CS - Coarse Sand; HL - Hydrated Lime; DF - Donna Fill.
1 For lab mixtures, aggregates passed 50.1 mm (2 inch) were scalped.
2 Mixtures would be 96.5% passing this sieve size for OG-LSAM;
3 For Mixtures would be 98% passing this sieve size for Sup-LSAM;
4 The Gm used for calculation was 2.68
* Field design used RAP. Lab mixes used existing LS to match the gradation.

In order to use the existing laboratory compacting equipment, an aggregate top size of 37.5-mm (1.5-inch) was selected for both open-graded and dense-graded LSAM. Several gradations were examined in order to ensure that volumetric criteria and stone-on-stone contact was satisfied. It was a trial-and-error procedure. During the trial-and-
error process, mixes with different gradations were examined for their volumetric properties and axial creep characteristics. For the open-graded LSAM, permeability was also considered for the section of gradation and optimum asphalt content.

Aggregate gradations for the conventional mixtures (Type 508 drainable base mix and Type 5A base mix) were adopted from the current Louisiana DOTD specification (LADOTD, 1992.)

Figure 3.15 shows all four gradations together in a semi-log scale and Figures 3.16 and 3.17 show the gradations in 0.45 power chart.

It is noticeable that the gradation of Superpave LSAM (Sup-LSAM) goes through the “Restricted Zone” in the Superpave 37.5-mm gradation chart (Figure 3.15). The NCHRP 4-18 Report (TTI, 1997) suggested that the concept of “Restricted Zone” should not apply to LSAM mixtures due to their relatively coarse nature.

![Figure 3.15 Gradation Chart of Four Mixtures in this Study](image)
Figure 3.16 Gradation Chart of Large Stone Asphalt Mixtures

Legend

- = Sup-LSAM
- = OG-LSAM

Percent Passing

Sieve Sizes

- 0.41 mm (0.016")
- 0.075 mm (0.003")
- 0.060 mm (0.0024")
- 0.045 mm (0.0018")
- 0.030 mm (0.0012")
- 0.025 mm (0.0010")
- 0.019 mm (0.00075")
- 0.016 mm (0.0006")
- 0.012 mm (0.00047")
- 0.010 mm (0.0004")
3.3.2 LSAM Bulk Specific Gravity Test

The test to determine the bulk specific gravity $G_{mb}$ is normally performed by placing the mix samples into water and comparing the submerged weight and SSD weight (ASTM D2726, AASHTO T166). But when the specimens contain open or interconnecting voids, or absorb more than 2% water by volume, the SSD method will not provide an accurate measurement of the bulk specific gravity. There are three alternatives to overcome this problem: 1) put a rubber membrane around the specimen before submerging; 2) coat the specimen with paraffin before submerging (ASTM D1188-89); or 3) replace the water with glass beads and perform the test as in the AASHTO T166 or ASTM D2726. The NCHRP 4-18 proposed using glass beads for the large stone asphalt mixture (TTI, 1997). In this study, 8-mm glass beads were used to determine the bulk specific gravity of both the open-graded large stone asphalt mixtures (OG_LSAM) and the conventional Type 508 drainable mixture.

Figure 3.18 shows the test set-up for measuring bulk specific gravity of water permeable, compacted HMA mixes using glass beads. Test equipment includes a half cubic foot aluminum bucket (unit measure) and a metal cone that is intimately fitted to the top of the unit weight measure to form a large metal pycnometer. The cone must be capable of being securely fastened to the unit weight measure and the cone must attain the same relative position with the unit weight measure each time it is set in position in order to ensure a constant volume for the pycnometer. Using the recommendation of NCHRP 4-18 (TTI, 1997), 8-mm glass beads were used.
The test procedure for measuring the bulk specific gravity of a compacted asphalt specimen using glass beads is as following:

- Record weight of asphalt specimen in air;
- Place 50.8 – 76 mm (2 to 3 inch) of beads in the bottom of the measure;
- Place specimen in the center of the measure and resting on the beads; twist the specimen to seat it in the beads;
- Fill the measure with beads to the top of the specimen. Tap the measure with the rubber mallet at four locations equally spaced around the circumference with five blows per location;
- Fill the measure to overflowing with beads and tap the measure with the rubber mallet at four locations equally spaced around the circumference with five blows per location. Level the glass beads to the top of the measure;
- Record the weight of the measure plus cone plus beads plus specimen;
- Using the following equation (Eq. 3.1), calculate the bulk specific gravity of the compacted asphalt specimen:

\[ G_{mb} = \frac{W_{specimen} \cdot G_{beads}}{W_{specimen} + W_{measure+beads} - W_{specimen+beads+measure+cone}} \]  

(3.1)

where

\[ G_{mb} = \text{Bulk Specific gravity of asphalt specimen}, \]
\[ G_{beads} = \text{Bulk specific gravity of beads}, \]
\[ W_{specimen} = \text{Weight of specimen in air}, \]

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\[ W_{\text{measure+beads}} = \text{Weight of measure plus beads, and} \]

\[ W_{\text{specimen+beads+measure+cone}} = \text{Weight of specimen+beads+measure+cone}. \]

The air voids in the compacted specimen is calculated using the following equation (Eq. 3.2):

\[
V_{TM} = \left(1 - \frac{G_{mb}}{G_{mm}}\right) \times 100
\]

where

\( V_{TM} = \) Percent air voids in the specimen,

\( G_{mb} = \) Bulk specific gravity of the specimen, and

\( G_{mm} = \) Maximum (Rice) specific gravity of specimen.

Figure 3.18 Unit Weight Measure, Cone, Glass Beads, and Specimen

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3.3.3 Open-Graded LSAM Design

Open-graded LSAMs are commonly used for rapid drainage of subsurface water. These mixes contain a small amount of fine aggregate (Table 3.6). When designing the open-graded LSAM (OG_LSAM), it is desirable that mixture should have high air voids, high voids in mineral aggregates (VMA) and sufficient asphalt cement for durability. The angularity of the coarse aggregate in these mixes provides the interlock among particles. The asphalt content typically ranges from 1.5 – 2.5 percent. The film thickness in these mixes generally significantly higher than conventional dense-graded mixes. To prevent runoff of asphalt cement during transportation to the job site, a lower mixing temperature is generally adopted. For each mixture design included in this study, nine sets of specimens (three asphalt content levels and three SGC gyrations) were prepared and tested for their volumetric properties in order to provide data for selecting the optimum design. Table 3.7 presents the air voids, VMA, VFA at different asphalt contents and compaction levels. Figure 3.19 presents the variation of air voids with asphalt content and SGC compaction efforts. Figures 3.20a and 3.20b show the relationship between VMA, VFA and the asphalt content and SGC gyrations. It appeared that 2.5 percent of asphalt and 25 SGC gyrations produced the optimum volumetric design for the open-graded LSAM.

3.3.4 Superpave LSAM Design

The design of this mixture was conducted according to AASHTO TP4 (1997). The $N_{ini}$, $N_{des}$ and $N_{max}$ were 8-, 100-, and 160-gyrations. The optimum asphalt content for this mix was 3.8. The optimum volumetric design satisfied a Superpave 37.5-mm

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(1.5-inch) mixture. Table 3.8 presents the volumetric properties of the four mixes used in this study.

Table 3.7 Volumetric Properties of Open-graded LSAM at Different Asphalt Contents

<table>
<thead>
<tr>
<th>Asphalt Content (%)</th>
<th>SGC Gyrations</th>
<th>Air Voids</th>
<th>VMA</th>
<th>VFA</th>
<th>Permeability K' (mm/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>10</td>
<td>26.7</td>
<td>32.4</td>
<td>18.4</td>
<td>5.25</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>22.4</td>
<td>28.2</td>
<td>20.8</td>
<td>5.01</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>19</td>
<td>25.2</td>
<td>24.6</td>
<td>3.89</td>
</tr>
<tr>
<td>2.5</td>
<td>10</td>
<td>26</td>
<td>30.6</td>
<td>15.1</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>26.2</td>
<td>30.7</td>
<td>14.9</td>
<td>7.19</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>20.6</td>
<td>25.5</td>
<td>19.3</td>
<td>0.935</td>
</tr>
<tr>
<td>3.0</td>
<td>10</td>
<td>13.7</td>
<td>19.3</td>
<td>35.3</td>
<td>7.19</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>11.8</td>
<td>17.5</td>
<td>33</td>
<td>0.437</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>15.8</td>
<td>21.3</td>
<td>25.8</td>
<td>0.174</td>
</tr>
</tbody>
</table>

Note: $K'$ is the pseudo coefficient of permeability. Details of $K'$ are described in section 3.6.

Figure 3.19 Open-graded LSAM Air Voids
Figure 3.20a Open-graded LSAM VMA

Figure 3.20b Open-graded LSAM VFA
Table 3.8 Volumetric Properties of Mixtures

<table>
<thead>
<tr>
<th></th>
<th>Type 508</th>
<th>OG LSAM</th>
<th>Type 5A</th>
<th>Sup LSAM</th>
<th>Superpave Spec (37.5-mm, Level II)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC%</td>
<td>2.3</td>
<td>2.5</td>
<td>3.5</td>
<td>3.8</td>
<td></td>
</tr>
<tr>
<td>N&lt;sub&gt;d&lt;/sub&gt; (SGC)</td>
<td>25</td>
<td>25</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>G&lt;sub&gt;mm&lt;/sub&gt;</td>
<td>2.584</td>
<td>2.592</td>
<td>2.507</td>
<td>2.546</td>
<td></td>
</tr>
<tr>
<td>G&lt;sub&gt;mb&lt;/sub&gt; (average)</td>
<td>1.760</td>
<td>1.915</td>
<td>2.410</td>
<td>2.459</td>
<td></td>
</tr>
<tr>
<td>Air voids (%)</td>
<td>32.1</td>
<td>26.2</td>
<td>4.0</td>
<td>3.5</td>
<td>3 ~ 5</td>
</tr>
<tr>
<td>VMA (%)</td>
<td>36.0</td>
<td>30.7</td>
<td>12.3</td>
<td>11.7</td>
<td>≥ 11.0</td>
</tr>
<tr>
<td>VFA (%)</td>
<td>10.8</td>
<td>14.9</td>
<td>67.5</td>
<td>70.9</td>
<td>65 ~ 78*</td>
</tr>
<tr>
<td>Film Thickness (micron)</td>
<td>38.9</td>
<td>30.2</td>
<td>7.07</td>
<td>7.3</td>
<td></td>
</tr>
</tbody>
</table>

* Louisiana Modified Specification (LADOTD, 1992)

3.3.5 Conventional Louisiana Type 5A and Type 508 Mix Design

These designs were selected from field projects. The optimum asphalt content for the Type 5A base mix was determined from a standard Marshall mix design. A Louisiana DOTD method specification (LADOTD, 1992) was used to determine the asphalt content of the Type 508 mix.

3.3.6 Degree of Stone-on-Stone Contact

The stone-on-stone contact was determined based on the test procedure proposed by NCHRP 4-18 (TTI, 1997). According to the procedure, the dry density of coarse aggregate (above 4.75 mm or No. 4 sieve) was first determined through rodded weight of the aggregates in a 0.028-m<sup>3</sup> (1 cubic foot) container. The voids in coarse aggregate can be calculated through the following equation:

\[
VCA = \left[ \frac{(G_{ca} \cdot d_{w}) - D_{ca}}{(G_{ca} \cdot d_{w})} \right] \cdot 100
\]  \hspace{1cm} (3.3)

where

\[ VCA \] is the voids content of the coarse aggregate;

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Gca is the specific gravity of coarse aggregate;

dw is the density of water; and

Dca is the density of the coarse aggregate.

Stone-on-stone contact is defined as the ratio of the density of the coarse aggregate in the compacted LSAM to the density of coarse aggregate. The density of the coarse aggregate in the compacted LSAM is calculated by the following equation:

\[ D_{cm} = (G_{mb} \cdot d_w) \cdot (1 - AC) \cdot R \]  \hspace{1cm} (3.4)

where

- \( D_{cm} \) is the density of the coarse aggregate in the compacted LSAM;
- \( G_{mb} \) is the bulk specific gravity of the compacted LSAM;
- \( d_w \) is the density of water;
- \( AC \) is the asphalt content as a weight percent of the total LSAM;
- \( R \) is the percent of coarse aggregate in LSAM gradation retained on 12.5 mm (0.5 inch) sieve for nominal maximum aggregate size from 25 to 38 mm or on the 19 mm (3/4 inch) sieve for nominal max. aggregate size from 38 to 64 mm.

The degree of stone on stone contact is expressed by the following equation:

\[ SSC = \left( \frac{D_{cm}}{D_{ca}} \right) \cdot 100 \]  \hspace{1cm} (3.5)

where

- \( SSC \) is the degree of stone-on-stone contact in the compacted LSAM;
- \( D_{cm} \) is the density of the coarse LSAM; and
\( D_{ca} \) is the density of the coarse aggregate.

Table 3.9 presents the results of the voids in coarse aggregates and the degree of stone-on-stone contact. It is shown that there is a 92 percent stone-on-stone contact for the open-graded large stone asphalt mixture whereas for the 37.5-mm Superpave mixture, there is an 85 percent of stone-on-stone contact. The NCHRP 4-18 recommended eighty percent or more in LSAM for degree of stone-on-stone contact. Figure 3.21 presents a visual observation of stone-on-stone contact from the cut mixtures.

Table 3.9 VCA and Degree of Stone-on-Stone Contact

<table>
<thead>
<tr>
<th>Mix Type</th>
<th>Coarse Aggregate Unit Density (g/cm(^3))</th>
<th>Coarse Aggregate Unit Density in LSAM (g/cm(^3))</th>
<th>VCA</th>
<th>SSC (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OG-LSAM</td>
<td>1.476</td>
<td>1.364</td>
<td>44.9</td>
<td>92</td>
</tr>
<tr>
<td>Sup-LSAM</td>
<td>1.576</td>
<td>1.343</td>
<td>41.2</td>
<td>85</td>
</tr>
</tbody>
</table>

Figure 3.21 Cut-Section Showing Stone-on-Stone Contact
3.4 DEVELOPMENT OF TEST FACTORIALS

Indirect tensile resilient modulus \( (M_R) \) test, indirect tensile strength (ITS) and strain test, axial creep test, indirect tensile creep test, Superpave simple shear frequency sweep (FSCH) test, Superpave simple shear repetitive shear at constant height (RSCH), Asphalt Pavement Analyzer (APA) rut test, and moisture susceptibility test were performed to characterize the four mixtures in this study. In addition, a permeability test was performed on the open-graded LSAM and the conventional Louisiana Type 508 drainable mixtures. The specimens were prepared for each mixture combination. Table 3.10 lists the performance tests proposed in this study and the corresponding number of specimens tested.

Table 3.10 Mixture Performance Tests

<table>
<thead>
<tr>
<th>Tests</th>
<th>Protocols</th>
<th>Engineering Properties</th>
<th>Mixtures</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Elastic Properties (( M_R ) and ( \mu ))</td>
<td>OG_LSAM</td>
</tr>
<tr>
<td>MR (5°C, 25°C, 40°C)</td>
<td>ASTM D 4123 (modified)</td>
<td></td>
<td>Type 508</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Permanent Strain (Rut Susceptibility)</td>
<td>Sup_LSAM</td>
</tr>
<tr>
<td></td>
<td></td>
<td>G*, ( \delta ) (Viscoelastic Properties)</td>
<td>Type 5A</td>
</tr>
<tr>
<td>ITs at 25°C</td>
<td>AASHTO T245</td>
<td></td>
<td>3x3</td>
</tr>
<tr>
<td>Axial Creep at 40°C</td>
<td>Tex-231-F (Tex DOT 1993)</td>
<td>Permanent Deformation</td>
<td>3x3</td>
</tr>
<tr>
<td>IT Creep at 40°C</td>
<td>Mohammad et al (1993)</td>
<td>Permanent Deformation</td>
<td>3x3</td>
</tr>
<tr>
<td>RSCH at 60°C</td>
<td>AASHTO TP7</td>
<td>Permanent Strain (Rut Susceptibility)</td>
<td>3x3</td>
</tr>
<tr>
<td>FSCH at 60°C</td>
<td>AASHTO TP7</td>
<td></td>
<td>3x3</td>
</tr>
<tr>
<td>APA Rut at 60°C</td>
<td>Georgia Spec. GDOT (1986)</td>
<td>Rut Susceptibility</td>
<td>3x3</td>
</tr>
<tr>
<td>Draindown Test</td>
<td>ASTM D 6390-99</td>
<td>Draindown Susceptibility</td>
<td>3x3</td>
</tr>
<tr>
<td>Permeability</td>
<td>Huang et al. (1999)</td>
<td>Coefficient of Permeability</td>
<td>3</td>
</tr>
<tr>
<td>Lottman Test</td>
<td>AASHTO TP283</td>
<td>Moisture Susceptibility</td>
<td>2x3x3</td>
</tr>
</tbody>
</table>

1 The number of specimens needed to conduct the test (Total number = 252)
3.5 MIXTURE PERFORMANCE CHARACTERIZATION

3.5.1 Specimen Preparation

Cylindrical specimens were fabricated for fundamental engineering property tests in this study. The specimens were compacted in the Superpave Gyratory Compactors (SGC, Figure 3.4 and 3.5) to a diameter of 150-mm (5.9-inch) and heights of between 120-mm (4.7-inch) to 150-mm (5.9-inch). Samples for SST tests were cut to a height of 75-mm (3.0-inch). The specimens for APA rut test were cut into a height between 70-mm and 75-mm. The specimens were then placed into the APA cylindrical mold and flushed with the top of the mold using plaster of Paris (Figure 3.22).

Figure 3.22 APA Rut Test Cylindrical Samples and Mold
3.5.2 Indirect Tensile Resilient Modulus \((M_R)\) Test

The testing temperatures were 5, 25, and 40 °C, and the test was conducted according to the modified ASTM D 4123 (Mohammad et al, 1993). This test is a repeated load indirect tension test for determining the resilient modulus of the asphalt mixtures. The recoverable vertical deformation \(\delta V\) and horizontal deformation \(\delta H\) were used to calculate the indirect tensile resilient modulus, \(M_R\) and Poisson’s ratio, \(\mu\) according to Equations (3.6) and (3.7):

\[
M_R = \frac{P(\mu + 0.27)}{t \cdot \delta H(T)} \tag{3.6}
\]

\[
\mu = 3.59 \left(\frac{\delta H(T)}{\delta V(T)}\right) - 0.27 \tag{3.7}
\]

where, \(M_R\) – Resilient Modulus, MPa,

\(P\) – applied vertical load, N,

\(t\) – sample thickness, mm,

\(\mu\) – poisson’s ratio

\(\delta H(T)\) – horizontal deformation at time \(T\), mm.

\(\delta V(T)\) – vertical deformation at time \(T\), mm.

In order to successfully conduct the indirect tensile resilient modulus test, the loading strip needs to be exactly centered. Even slight deviation will cause significant errors for the test results. During this study, an indirect tensile loading frame was fabricated to conduct various tests in the indirect tensile mode. Figure 3.23 shows the test configuration of indirect tensile resilient modulus test and Figure 3.24 presents the typical results from a \(M_R\) test.

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Figure 3.23 Test Setup of Indirect Tensile Resilient Modulus ($M_R$) Test
Figure 3.24 Typical IT Resilient Modulus ($M_R$) Test Results
3.5.3 Indirect Tensile Strength (ITS) and Strain Test

The indirect tensile strength (ITS) and strain test was used to determine the tensile strength and strain of the mixtures. This test was incorporated in the study to ensure the durability of the mixtures would not be compromised while the rut resistance of the mixtures was improved. This test was conducted at 25 °C according to AASHTO T245. Each test specimen was loaded to failure at a 50.8 mm/min (2 inch/min) deformation rate. The load and deformations were continuously recorded and indirect tensile strength and strain were computed as follows:

\[ S_T = \frac{2 \cdot P_{\text{ult}}}{\pi \cdot t \cdot D} \]  
(3.8)
\[ \varepsilon_T = 0.0205 \cdot H_T \]  
(3.9)

where

- \( S_T \) — Tensile strength, kPa
- \( P_{\text{ult}} \) — Peak load, N
- \( t \) — Thickness of the specimen, mm
- \( D \) — Diameter of the specimen, mm
- \( \varepsilon_T \) — Horizontal tensile strain at peak load, and
- \( H_T \) — Horizontal deformation at peak load.

Toughness index (TI), a parameter describing the toughening characteristics in the post-peak region, was also calculated from the indirect tensile test results. Figure 3.25 presents a typical normalized indirect tensile stress and strain curve. A dimensionless indirect tensile toughness index, TI is defined as follows:
\[ TI = \frac{A_\varepsilon - A_p}{\varepsilon - \varepsilon_p} \]  

(3.10)

where

TI – Toughness index,

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

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

\( \varepsilon \) – Strain at the point of interest, and

\( \varepsilon_p \) – Strain corresponding to the peak stress.

Figure 3.25 A Typical Normalized ITS Curve for TI Calculation
This toughness index compares the performance of a specimen with that of an elastic perfectly plastic reference material, for which the TI remains a constant of one. For an ideal brittle material with no post-peak load carrying capacity, the value of TI equals zero. Similar analyses were reported by Sobhan, et al (1999). In this study, the values of indirect tensile toughness index were calculated at a tensile strain of three percent.

3.5.4 Axial Creep Test

This test was conducted in accordance with the Test Method Tex-231-F (Texas DOT, 1993). The test was conducted in an axial loading mode as shown in Figure 3.26. A static load of 1.220 kN (274 lbf) was applied for the duration of one hour along the centric longitudinal axis of the specimen (150-mm in diameter). The axial deformation of the specimen was continuously measured and subsequently used to calculate creep properties such as stiffness, slope, and permanent strain. These data were used to evaluate the permanent deformation characteristics of asphalt mixtures. The samples were tested at 40°C. A typical axial creep test result is illustrated in Figure 3.27. The slope and strains are shown in the figure and the creep stiffness is calculated as the compressive stress divided by the total strain at the end of the loading period.

Higher stiffness, lower creep slope and lower permanent strain are desired for rut-resistant mixtures. Texas specification (Texas DOT, 1993) specifies that a minimum stiffness of 41.4 MPa, maximum creep slope of 3.5x10-8 and maximum permanent strain value of 5x10-4 mm/mm for a satisfactory surface mixture.
Figure 3.26 Test Setup of Axial Creep Test
Figure 3.27 Typical Axial Creep Test Results
3.5.5 **Indirect Tensile Creep Test**

This test is performed in the indirect tensile mode with a setup similar to that for the indirect tensile resilient modulus \((M_R)\) and the indirect tensile strength (ITS) test (Figure 3.23). At a testing temperature of 40°C (104°F), a compressive load of 1112.5 N (250 lbf) was applied to the specimen using the stress-controlled mode of the MTS test system. The load was applied for 60 minutes or until specimen failure occurs (Mohammad et al, 1993). The deformations acquired during this time were used to compute the creep modulus as follows:

\[
S(T) = \frac{359P}{t \cdot \delta V(T)}
\]

where,
- \(S(T)\) — creep modulus at time \(T\), MPa,
- \(P\) — applied vertical load, N,
- \(t\) — specimen thickness, mm, and
- \(\delta V(T)\) — vertical deformation at time \(T\), mm.

Figure 3.28 shows the typical results of the load versus time, vertical deformation versus time and creep stiffness versus time graph on a log-log scale for the indirect tensile creep test. From this graph the creep slope was computed and used in the analysis.

3.5.6 **APA Rut Test**

The Asphalt Pavement Analyzer (APA, Figure 3.12) is an enhanced version of the Georgia loaded wheel tester. The APA is capable of evaluating rutting, moisture and fatigue cracking susceptibility of asphalt concrete mixes. Loads, pressure and
Figure 3.28 Typical Results from Indirect Tensile Creep Test
temperature are adjustable, and dry or submerged test conditions can be selected. Test
temperature can be adjusted and maintained from 30°C to 65°C (85°F to 148°F) during
testing.

The APA can test three samples simultaneously (Figure 3.29). The concave
shaped wheels travel back and forth over a stiff, pressurized rubber hose that rests
directly on the specimen. A specimen slab is approximately 127 mm wide, 76 mm
deep and 320 mm long. Specimens can also consist of 150 mm diameter field cores or
Superpave gyratory compacted specimens. In this study, 150-mm diameter laboratory
compacted cylindrical specimens were used in this test. Typical test conditions as set
by the Georgia DOT specification are at temperature of 40°C, 444.4 N load, and 0.7
MPa hose pressure with a failure criterion of no more than 7.6 mm rut depth after
8,000 cycles (16,000 passes) under dry conditions. The wheel speed is approximately
60 cm/sec. In this study, a test temperature of 60 °C was selected based on the
LTPPBIND database for the temperature of the pavement at a depth of 50-mm
(FHWA, 1999). LTPPBIND is a software developed by the FHWA Turner-Fairbank
Highway Research Center for Determining Superpave Performance Grades Based on
LTPP and SHRP Pavement Temperature Models and Data from 7928 Weather
Stations in North America.

The LTRC's APA is capable of recording rut depth automatically. Average rut
depth vs. number of load cycles, slope of rut depth vs. number of load cycles, and
change of slopes vs. number of load cycles, can be drawn to analyze the rutting
potential for an asphalt mixture. These three curves are shown in Figure 3.30, 3.31, and 3.32.

![Figure 3.29 APA Specimens in the Molds for Testing](image)

![Figure 3.30 Rut Depth ~ Load Cycles](image)

Figure 3.30 Rut Depth ~ Load Cycles

Figure 3.29 APA Specimens in the Molds for Testing
Figure 3.31 Slope ~ Load Cycles

Figure 3.32 Change of Slope ~ Load Cycles
3.5.7 Superpave Frequency Sweep at Constant Height (FSCH)

The Superpave frequency sweep at constant height (FSCH) test was performed according to AASHTO TP7. The FSCH uses a specimen with loads applied as shown in Figure 3.33. This test, conducted in the shear mode, is a strain controlled test, that is, a specific amount of deformation is induced in the specimen. Stress generated in the specimen is not controlled but is simply the reaction to the induced strain. The sinusoidal shear strain with peak amplitude of approximately 0.05 μm/mm is applied at frequencies of 10, 5, 2, 1, 0.5, 0.2, 0.1, 0.05, 0.02 and 0.01 Hz (Figure 3.34). This strain level was selected during the SHRP Research 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 (or frequency) and not of the stress magnitude. An axial stress is applied to maintain constant height (Figure 3.35). Frequency is directly related to traffic speed. For example, a frequency of 1 Hz is equivalent to a traffic speed of 63 km/hr. (39 mile/hr.) and 2 Hz is 125 km/hr. (78 mile/hr.). Hence, frequency sweep test can be used to evaluate the performance of an asphalt mixture at different traffic speeds.

Figure 3.33 Specimen Configuration of FSCH

67
The ratio of the stress response of the test specimen to the applied shear strain is used to compute a complex shear modulus ($G^*$) for a given frequency. Because of
viscoelasticity, there is a delay of the peak between the strain and stress. Therefore, the phase angle ($\delta$) is also computed.

The relationship between complex shear modulus ($G^*$) and frequency, shown in Figure 3.36, indicates that the faster an asphalt concrete specimen is loaded (i.e., higher frequency) the stiffer it behaves. This relationship is important in Superpave because it is used to determine a major component needed for the rutting and fatigue cracking predictions.

![Figure 3.36 Complex Shear Modulus ($G^*$) at FSCH Test](image_url)

The phase angle ($\delta$) at FSCH test normally initially increases with frequencies and then gradually decreases after reaching a peak value (Figure 3.37). This can be explained as follows. At lower frequency (or higher temperature), asphalt binder
plays the predominant role in the mixture. The pattern of the phase angle resembles that of the asphalt binder. Whereas at high frequency (or low temperature), aggregate structure plays more and more important role in the behavior of the asphalt mixture. The mixture tends to be more and more “elastic”. Therefore, the phase angle decreases.

The test temperature of FSCH in this study was 60 °C. This temperature was selected based on the LTPPBIND database for the temperature of the pavement at a depth of 50-mm (FHWA, 1999).

![Graph](image)

Figure 3.37 Phase Angle ($\delta$) at FSCH Test

3.5.8 Superpave Repetitive Shear at Constant Height (RSCH)

The Superpave repetitive shear at constant height (RSCH) test is a stress-controlled shearing test. This test is included in AASHTO TP7 as an optional test procedure
used to estimate relative rut depth. In this test a cylindrical specimen is subjected to horizontal shear stress pulses with an amplitude of $68 \pm 5$ kPa. A varying axial load is applied automatically during each cycle to maintain the specimen at constant thickness or height. Figure 3.38 shows the haversian shear and axial stresses during repeated shear test at constant height. This test has the duration of up to 5000 load cycles or until the permanent strain reaches 5 percent. The axial load, shear load, axial deformation (LVDT), and shear deformation (LVDT) are recorded at a sampling rate of 60 data points per second during the same specific ranges of load cycles.

In the development of the repeated shear test at constant height two mechanisms that provide resistance to permanent deformation in an asphalt mixture were hypothesized (Sousa, et al, 1994):

- **Asphalt Binder Stiffness**
  
  Stiffer binders help in resisting permanent deformation as the magnitude of the shear strains is reduced under each load application. The rate of accumulation of permanent deformation is strongly related to the magnitude of the shear strains. Therefore, stiffer asphalt will improve rutting resistance as it minimizes shear strains in the aggregate skeleton.

- **Aggregate Structure Stability**
  
  The axial stresses act as a confining pressure and tend to stabilize the mixture. A well-compacted mixture with a good granular aggregate will develop high axial forces at very small shear strain levels. Poorly compacted mixtures can also generate similar levels of axial forces but only at much higher shear strains.
In the constant height simple shear test, these two mechanisms are free to fully develop their relative contribution to the resistance of permanent deformation as they are not constrained by imposed axial or confining stresses. The development of the repeated shear test at constant height was detailed elsewhere (Sousa, et al, 1994).

Figure 3.38 Haversian Stress Applications in the RSCH Test

3.5.9 Moisture Susceptibility Test

The modified Lottman test (AASHTO T283) is the most widely used to evaluate the moisture susceptibility of asphalt mixtures. This test measures the effect of moisture on the indirect tensile strength of the mixture. The LaDOTD adopts the same procedure with a slight modification (LaDOTD TR 322M/322-97). The test procedure
requires that six SGC compacted specimens (150-mm in diameter) have air voids in the range of six to eight percent. The six specimens are then divided into two sets of three specimens. One set is used as the control set, whereas the other set is used for moisture-conditioning. Moisture-conditioning starts by inducing between 55 to 80 percent saturation in the specimens which are then placed in a freezer for a minimum of 15 hours at \(-18 \pm 5\) °C. The specimens are then placed in a hot water bath at \(60 \pm 0.5\) °C for \(24 \pm 0.5\) hours. The moisture-conditioned specimens are ready for testing after they are removed from the hot water bath and are kept in a \(60 \pm 0.5\) °C water bath for \(40 \pm 5\) minutes. In this study, however, achieving the six to eight percent air voids for the open-graded LSAM and the Type 508 drainable mix considered impractical. Therefore, these mixtures were conditioned at much higher air void levels.

An anti-stripping additive, Permatac-99® was added to the asphalt cement at a dosage of 0.5 percent by weight of asphalt according to the current LADOTD’s specifications. According to Gopalakrishnan (1999), Permatac-99® is the most effective anti-stripping agent (when compared with Pavebond T-Lite® and Adhere HP Plus®).

The indirect tensile strength of both the control set and conditioned set of specimens is determined at \(25\) °C. The moisture susceptibility is indicated by the Tensile Strength Ratio (TSR) expressed as:

\[
TSR = \frac{S_{im}}{S_{tc}}
\]  

(3.12)

Where
TSR – Tensile Strength Ratio,

$S_{tm}$ – Average tensile strength of the moisture-conditioned set, and

$S_{tc}$ – Average tensile strength of the control set.

3.5.10 Permeability Test

Permeability tests of asphalt mixtures normally followed the test protocols developed for soils/granular materials (ASTM D 5084 90, AASHTO T-215 90). Basically there are two types of tests: constant head and falling head methods. All the current test specifications assume that Darcy’s law is valid for water flows through the porous media. In other words, the flows must be controlled as laminar during the test.

When characterizing the permeability of drainable paving materials, confusion often arise for the measured values of coefficient of permeability. The difference of reported valued of coefficient of permeability for the similar material can be as high as 100 times (Huang, et al, 1999). For this reason, a sub-study of permeability in asphalt mixtures has been conducted during this study. A dual mode flexible wall permeameter has been developed for the purpose of measuring the water permeability of asphalt mixtures. This device works on both constant head and falling head principles. It is also capable of determining the materials’ water permeability when the common Darcy’s law is no longer valid, a situation when testing the open-graded LSAM and the Type 508 drainable base mixes. The details of the test procedure with this new dual mode flexible wall permeameter are presented in section 3.6, “Fundamentals of Permeability in Asphalt Mixtures.”
3.5.11 Draindown Test

The draindown test was used to evaluate the runoff of asphalt cement in loose mixtures of OG_LSAM and Type 508. Loose mixture (1200 gram) was placed in the sieve and put into the oven at the prescribed temperature for 60 minutes. A paper plate was placed under the sieve. The calculation of asphalt draindown is performed by subtracting the initial paper plate mass from the final paper plate mass and dividing this by the initial total specimen mass expressed as a percentage. A percent loss of greater than 0.3 indicates that draindown may be a problem for the mix.

3.6 FUNDAMENTALS OF PERMEABILITY IN ASPHALT MIXTURES

Permeability or hydraulic conductivity is an important characteristic of pavement materials. A dense graded asphalt mix will prevent water from passing through the layer so that the pavement structure will not be saturated. On the other hand, an open-graded asphalt that enters will not stay in the pavement structure will quickly flow through the drainage system.

The common design procedures require drainability characteristics of the paving materials in terms of hydraulic conductivity and effective porosity (AASHTO, 1993). Hydraulic conductivity is generally considered the same as the coefficient of permeability as defined in the famous Darcy’s Law, in which fluid’s discharge velocity is directly proportional to hydraulic gradient (Bowles, 1992). The validity of Darcy’s Law depends on the flow condition. It is only valid when the fluid travels at a very low speed in the porous media and no turbulence occurs. Such a flow is called a laminar flow.

Unfortunately, pavement engineers often forget to check for this important criterion when applying Darcy’s Law to characterize flow through porous paving materials.
When characterizing the permeability of drainable paving materials, confusion often arises for the measured values of coefficient of permeability. According to Zhou, et al. (1992), the reported coefficient of permeability for untreated permeable base from different state DOTs varies from 0.7 mm/sec (200 ft/day) to 70 mm/sec (20,000 ft/day). One of the important factors for this variation is the different test condition under which the coefficient of permeability is being calculated. Tan et al. (1997) reported that for the open graded coarse mixtures in their study, Darcy's law was no longer valid.

This section presents the results of a drainability study of several asphalt mixtures ranging from dense-graded conventional mixture to open-graded LSAM mixtures. The fundamentals of hydraulic conductivity have been reviewed and the validity of Darcy's Law has been discussed. A dual mode permeability test device has been developed, and a statistical model to predict the hydraulic conductivity has been developed for the drainable asphalt mixtures included in this study.

3.6.1 Fundamentals of Hydraulic Conductivity

3.6.1.1 Darcy's Law

In 1856, Henry Darcy investigated the flow of water in vertical homogenous sand filters in connection with the fountains of the city of Dijon, France, Figure 3.39 He concluded that the rate of flow, Q, is (a) proportional to the cross-sectional area A, (b) proportional to water head loss, (h1 - h2), and (c) inversely proportional to the length L. When combined, these conclusions give the famous Darcy's Law

\[ Q = K A (h_1 - h_2) / L \]  \hspace{1cm} (3.13)

or

\[ v = - K i \]  \hspace{1cm} (3.14)
where $K$ is the proportional factor called hydraulic conductivity (or coefficient of permeability), $v = Q / A$ is the discharge velocity and, $i = \partial h / \partial L$ is the hydraulic gradient.

Later researchers, having further developed Darcy’s basic ideas, determined the dependence of conductivity on the parameters of the transported fluid (Kovacs, 1981). They found that hydraulic conductivity is proportional to the ratio of specific weight ($\gamma$) and dynamic viscosity ($\mu$) of the fluid, which is the acceleration due to gravity ($g$) divided by the kinematic viscosity ($v$) of the fluid. Thus, the hydraulic conductivity as defined by Darcy’s Law can further be defined as:

$$K = k (\gamma/\mu) = k (g/v)$$

(3.15)

where $k$ is a factor that depends only on the properties of the solid matrix of the porous medium, and is called intrinsic permeability, matrix permeability or sometimes only permeability. The dimension for $K$ is $[LT^{-1}]$ and $k$, $[L^2]$.

Figure 3.39 Darcy’s Experiment

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3.6.1.2 Theoretical Determination of Darcy's Hydraulic Conductivity

Having understood the basic equation of Darcy's Law as well as the definition of hydraulic conductivity and intrinsic permeability, it is not difficult to relate the hydraulic conductivity with geometric characteristics of porous media. The following derivations are excerpted from the translation of the original work of G. Kovacs (1981).

Assuming that the irregularly connected channels formed by the pores of porous medium can be simplified into a bundle of small straight pipes and assuming only two main forces influence the laminar movement (i.e. gravity and friction), their equilibrium can be expressed in a mathematical form from a model pipe with a diameter of $2r_0$ (Figure 3.40).

Poiseuille's equation can be derived in this way. The equilibrium of a cylinder concentric about the axis of the pipe having a radius $r$ and a length of $l$, gives the following equation:

![Figure 3.40 Symbols used for Deriving Poiseulle's Equation](image-url)
\[ i \gamma \cdot r^2 \pi \cdot l + 2r \pi \cdot l \eta \frac{dv}{dr} = 0 \]  \hspace{1cm} (3.16)

where \( \eta \) is the viscosity of the fluid (Pa sec).

After solving this differential equation with a boundary condition, where the velocity at the wall of the pipe is zero (\( v = 0 \) at \( r = r_0 \)), the velocity at a point at a distance of \( r \) from the axis can be determined by:

\[ v = \frac{i \gamma}{4 \eta} (r_0^2 - r^2) \]  \hspace{1cm} (3.17)

Integrating the product of the velocity and an elementary area (\( dA \)) along the total surface of the cross section, the flow-rate through one pipe with a radius of \( r_0 \) can be obtained by:

\[ Q_0 = \int_{(A)} v dA = \frac{i \gamma}{4 \eta} \int_0^{r_0} (r_0^2 - r^2) 2r \pi dr = \frac{\pi}{8 \eta} \frac{i \gamma}{4 \eta} r_0^4 \]  \hspace{1cm} (3.18)

Dividing Equation (3.18) by the total area, produces the mean velocity,

\[ v = \frac{Q_0}{A} = \frac{i \gamma}{8 \eta} r_0^2 \]  \hspace{1cm} (3.19)

The number of pipes in the model system crossing the unit area of the sample is known, and thus, the total discharge and the virtual seepage velocity can be calculated as follows:

\[ v_s = \frac{Q}{A_s} = Q_0 N = \frac{i \gamma}{32 \eta} n r_0^2 \]  \hspace{1cm} (3.20)

where \( A_s \) is the total cross-sectional area of the sample, \( n \) is the porosity of the medium and \( N \) is the total number of pipes which is given by:

\[ N = \frac{4n}{d_0^2 \pi} \]  \hspace{1cm} (3.21)
where \(d_0\) is the average diameter of the model pipe \((d_0 = 2r_0)\) and it is related to the effective particle diameter \(D_h\) through the following equation:

\[
d_0 = \frac{4n}{1-n} \cdot \frac{D_h}{\alpha}
\]  

(3.22)

where \(\alpha\) is the coefficient of shape factor.

Substitute Equation (3.22) into Equation (3.20), the following relationship can be determined:

\[
\nu_s = \frac{1}{2} \frac{\gamma}{\eta} \frac{n^3}{(1-n)^2} \left( \frac{D_h}{\alpha} \right)^2 \cdot i
\]  

(3.23)

Hydraulic conductivity of the model pipes with constant diameter calculated from Equation (3.23) is greater than the actual value determined by experiment since the actual pipe diameter is not a constant. Kovacs suggested multiplying right side of Equation (3.23) by a factor of 0.4. The theoretical value of hydraulic conductivity can therefore be determined, which agrees with the dynamic analysis and includes all the effects of the influencing factors.

\[
K = k \frac{\gamma}{\eta} = \frac{1}{5} \frac{\gamma}{\eta} \frac{n^3}{(1-n)^2} \left( \frac{D_h}{\alpha} \right)^2
\]  

(3.24)

From Equation (3.24), it can be concluded that hydraulic conductivity is determined by three factors:

- The fluid characteristics (\(\gamma/\eta\), or using the kinematic viscosity \(v=\eta/\rho\), the equivalent ratio is \(g/v\));

- Effective particle diameter, \(D_h\), shape and distribution;

- The effect of porosity, \(n^3/(1-n)^2\).
3.6.1.3 Range of Validity of Darcy's Law

As stated before, Darcy's Law is valid for the laminar flow condition. The fact is, Darcy's Law neglects variations in interstitial pressure associated with the inertia the liquid as it moves around the grains or along the convoluted pathways. If, at some point, its trajectory has a radius of curvature, \( r \), the fluid inertia sets up an additional pressure gradient \( \rho v^2/r \), where \( v \) is the pore velocity -- this provides the centripetal acceleration associated with the curved trajectory. Darcy's Law is accurate then, only when these inertial pressure gradients are small compared to the viscous stress gradients \( \mu v/d^2 \).

Generally, \( r \) is approximately equal to the pore diameter \( d \). Thus, it follows that:

\[
\frac{\rho \cdot v^2}{d} \ll \frac{\mu \cdot v}{d^2} \tag{3.25}
\]

or

\[
R = \frac{v \cdot d}{v} < 1. \tag{3.26}
\]

where again \( v = \mu/\rho \) is the kinematic viscosity and \( d \) is some representative length of the porous matrix.

The term R in Equation (3.26) is the pore Reynolds number, a dimensionless grouping of the pore velocity, pore width, and kinematic viscosity. For the validity of Darcy's Law, the R-value must be small. Generally, when \( R << 1 \) a flow is called a creeping flow.

Although by analogy to the Reynolds number for pipes, \( d \) should be a length of the cross section of an elementary channel of the porous medium, it is customary to select \( d \) equal to the representative length of the aggregate particles. Thus the numerical values differ when different particle sizes are chosen. Most literatures suggest using \( d_{10} \), the diameter of the particles corresponding to 10% passing at the gradation curve. Bear
(1979) suggested that for the validity of Darcy’s Law, the Reynolds number should not exceed some value between 1 and 10 (Fig. 3.41).

When the Reynolds number $R > 1 - 10$, there are mainly two types of equations to approximate the relationship between hydraulic gradient and flow velocity (Kovacs, 1981):

- **Binomial form:** $i = av + bv^2$;
- **Potential form:** $i = Cv^n$; or $v = K' i^{1/n}$.

Although neither of the above forms can be applied with unified material parameters, the second potential form seems to be more accepted in the literature (Bowles, 1992, Tan, et al, 1997, Kovacs, 1981, and Bear, 1979) when a validity zone is attached to a given value of the power.
3.6.2 Laboratory Test to Measure Hydraulic Conductivity

3.6.2.1 Test Methods

Laboratory tests to measure hydraulic conductivity for asphalt mixture normally adapt the test protocols of the soils/granular materials (ASTM D 5084 90, AASHTO T-215 90). Basically there are two types of tests: constant head and falling head method. So the current test specifications assume Darcy’s Law to be valid. In other words, the flows must be controlled to remain laminar during the test.

3.6.2.2 Testing Concerns

- **Short-Circuiting Through Side Walls**

  In most laboratory permeability tests, a cylindrical specimen is tested with water flow through its vertical direction. The specimen is placed in a cell either wrapped by a flexible membrane or adjacent to a rigid wall. It is very critical to prevent the short-circuiting of flow around the side of the specimen, a situation that greatly increases the measured hydraulic conductivity. For asphalt mixture specimens, it is advisable to use flexible wall permeameters can apply a level of confining pressure to the outside of the membrane to minimize the possibility of short-circuiting.

- **Air Blockage**

  Air bubbles in the specimens tend to block the flow of water, reducing the measured hydraulic conductivity. Unfortunately, it is sometimes nearly impossible to achieve full saturation for certain mixtures. Common ways to saturate specimens are submersion in water for a certain period of time and initial vacuum saturation.
• Non-laminar Flows

Non-laminar flows are generally caused by excessive hydraulic gradients during the test. One way to prevent this from happening is to reduce the hydraulic gradient. ASSHTO and ASTM standards limit the upper gradient for rigid wall cell to 0.2 – 0.5, and 1 – 5 for flexible membrane wall systems. However, for drainable paving materials such as the open graded large stone asphalt mixtures considered in this study, a very small hydraulic gradient may cause turbulence due to the large air cavities present in these mixtures. In this case, it becomes impractical to simply reduce the hydraulic gradient to some very small values (like 0.01) in order to satisfy the laminar flow condition. Tan et al. (1997) suggested the use of a pseudo-coefficient of permeability, the rate of specific discharge when the hydraulic gradient equals 1, as a benchmark to compare hydraulic conductivity of different materials. They modified a traditional falling head permeameter and tested three asphalt mixtures under the non-laminar flow conditions (Tan et al, 1997).

3.6.3 Laboratory Study of Hydraulic Conductivity for Asphalt Mixtures

3.6.3.1 Objectives

Realizing the problems in determining hydraulic conductivity of asphalt mixtures, a sub-study was initiated to investigate the water permeability characteristics of different asphalt mixtures. The main objectives of the sub-study were to:

• Develop a test apparatus/procedure capable of measuring hydraulic conductivity of different asphalt mixtures;

• Provide typical values of hydraulic conductivity of different mixes used in Louisiana;
• Establish empirical relations between hydraulic conductivity and other physical indexes such as mix gradation and effective porosity.

3.6.3.2 Dual Mode Permeameter

Figure 3.42 is a diagram of the dual mode permeameter developed and used in this study. The initial device, purchased from Virginia LAB Supply Co., and modified in this study is capable of measuring hydraulic conductivity of different materials from dense graded low permeable mixtures to open-graded drainable mixes under both constant and falling head modes. Two pressure transducers installed at the top and bottom of the specimen give accurate readings of the hydraulic head difference during the test. Data acquisition makes it possible to have continuous readings during a falling head test so that the test can be conducted even at very high flow rates (for drainable mixes). The specimen is placed in an aluminum cell with a retractable anti-scratch rubber membrane that is clamped tightly at both end of the cylindrical cell. A vacuum is applied between the membrane and the cell to facilitate the installation of the specimen. During the test, a confining pressure of up to 103.5 kPa (15 psi) is applied on the membrane to prevent short-circuiting around the side of the. Two different top reservoir tubes have been designed for testing different materials. One with a diameter of 25 mm (1 inch) is used for dense graded or less permeable materials and the other with 75 mm (3 inch) diameter is used for highly permeable materials. Both reservoir tubes are 90 mm (3 feet) long. A vacuum is applied on the top of the reservoir tube before the test to saturate the specimen.

3.6.3.3 Materials

Five types of asphalt mixtures have been tested for their hydraulic conductivity and effective porosity characteristics. Figure 3.43 shows the gradations of these mixes. Mix
Figure 3.42 Dual Mode Flexible Wall Permeameter

Figure 3.43 Gradations of the Mixtures for the Permeability Study

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LSAM is an open-graded large stone asphalt mix, D_508 is Louisiana Type 508 asphalt treated drainable base mix, Su_WC is a dense-graded 19mm Superpave wearing course, C_10 and C_12 are core specimens of a dense-graded mixes taken from interstates I-10 and I-12 near Baton Rouge, Louisiana. AC content and other gradation related parameters are presented in Table 3.11.

Table 3.11. Mix Asphalt Content and Other Gradation Parameters

<table>
<thead>
<tr>
<th>Mix Symbols</th>
<th>LSAM</th>
<th>D_508</th>
<th>Su_WC</th>
<th>C_10</th>
<th>C_12</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix Type</td>
<td>Open Graded Large Stone #57 ATBC</td>
<td>Open Graded Dense Graded 19mm Superpave Mix</td>
<td>SMA</td>
<td>Dense Graded</td>
<td></td>
</tr>
<tr>
<td>AC %</td>
<td>2 to 3</td>
<td>2.2</td>
<td>4.6</td>
<td>5.0</td>
<td>5.0</td>
</tr>
<tr>
<td>d_{10} (mm)</td>
<td>5</td>
<td>0.4</td>
<td>0.32</td>
<td>0.14</td>
<td>0.1</td>
</tr>
<tr>
<td>d_{50} (mm)</td>
<td>25</td>
<td>14</td>
<td>6.5</td>
<td>7.2</td>
<td>3.1</td>
</tr>
<tr>
<td>C_u=d_{60}/d_{10}</td>
<td>6</td>
<td>37.5</td>
<td>1.76</td>
<td>1.64</td>
<td>1.16</td>
</tr>
<tr>
<td>P_{3/8}</td>
<td>18.3</td>
<td>14</td>
<td>66</td>
<td>66</td>
<td>73</td>
</tr>
</tbody>
</table>

Note: d_{10} - Aggregate diameter of the 10% passing; d_{50} - Aggregate diameter of the 50% passing; C_u - Coefficient of non-uniformity; P_{3/8} - Percent passing 9.5mm (3/8") sieve.

ATBC - Asphalt treated base course

3.6.3.4 Effective Porosity (n_e)

As described earlier, porosity is one of the three main factors that influence the hydraulic conductivity of porous media. But in asphalt mixes, a portion of the air voids is trapped by asphalt and mineral fillers and is therefore, water impermeable. So instead of the air voids, the index of effective porosity relates more directly to the hydraulic conductivity. By definition, effective porosity is the ratio of the volume of voids that can be drained under gravity to the total volume of mixture. The effective porosity is calculated as following:
• First calculate the total air void through regular mixture bulk specific gravity $G_{mb}$ test method using air and water, SSD weight for most mixes and glass beads method for open graded, LSAM (TTI, 1997);

• Similar to Rice specific gravity test, place the cylindrical specimen into the Rice specific gravity container and conduct the vacuum saturated specific gravity test of the briquette, $G_{vs}$;

• Based on the difference between maximum theoretical specific gravity $G_{mm}$ and the vacuum saturated specific gravity of the briquette $G_{vs}$, calculate the air voids that are undrainable;

• The effective porosity is the difference between the total air void and the undrainable air void.

3.6.3.5 Test Data Processing

Both constant head and falling head tests indicate that for dense graded mixtures, Darcy’s Law is a good approximation of flow, however, for open-graded drainable mixes, a linear relation between hydraulic gradient and the fluid discharge velocity no longer exists. This can be well illustrated by the following experimental curves of two very different mixes, LSAM, an open-graded large stone asphalt mixture, and, C_10, a 19mm Superpave dense-graded mixture.

Figure 3.44 shows hydraulic head difference vs. time curve obtained from the two pressure transducers for these two mixes. A second order polynomial regression was fitted to these data with an $R^2$ exceeding 0.999.

$$h = a_0 + a_1t + a_2t^2$$ (3.27)
where \( a_0 \), \( a_1 \) and \( a_2 \) are regression coefficients. Differentiating Equation (4.15), yields:

\[
\frac{dh}{dt} = a_1 + a_2 t
\]

(3.28)

Therefore, the discharge velocity is expressed as:

\[
v = \frac{dQ}{dt} = \frac{A_1}{A_2} \frac{dh}{dt} = \frac{r_1^2}{r_2^2} \frac{dh}{dt}
\]

(3.29)

where \( A_1, A_2, r_1, r_2 \) are the cross section areas and radii of upper cylindrical reservoir and the specimen.

![Figure 3.44 Hydraulic Head vs. Time in Falling Head Test](image)

Figure 3.44 Hydraulic Head vs. Time in Falling Head Test

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Plotting the discharge velocity against the corresponding hydraulic gradient and applying curve fitting of the potential form \( v = K'^m \), one obtains two curve fitting parameters \( K' \) and \( m \) (Fig. 3.45). \( K' \) is defined as the pseudo-coefficient of permeability which equals the average discharge velocity when the hydraulic gradient equals 1. The factor \( m \) is a shape parameter. It is well known that laminar seepage is described with a power of \( m = 1 \). The power gradually decreases as the effect of inertia becomes stronger, achieving an \( m = 0.5 \) value in the case of turbulent flow.

![Discharge Velocity vs. Hydraulic Gradient](image)

Figure 3.45 Discharge Velocity vs. Hydraulic Gradient
3.6.3.6 Analysis of Test Results

All test data are plotted and processed similar to the analysis presented in Figures 3.44 and 3.45 to obtain the pseudo-coefficient of permeability $K'$ and the shape factor $m$. The test results shown in Table 3.12 indicate that hydraulic conductivity varies greatly from different mixes. The pseudo-coefficient of permeability ($K'$) gives a good benchmark to compare the hydraulic conductivity of different mixes regardless of their conformity with Darcy's Law. The shape factor of power $m$ indicates that for dense, impermeable mixes, the values of $m$ close to 1, an indication of laminar flow. On the other hand, the values of $m$ for the drainable mixes are all much less than 1, a clear sign of turbulence.

Table 3.12 Hydraulic Conductivity Test Results

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>AC %</th>
<th>$n_e$</th>
<th>$K'$ (mm/s)</th>
<th>$K'$ (ft/day)</th>
<th>$m$</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>LSAM 2</td>
<td>3.0</td>
<td>18.5</td>
<td>14.76022</td>
<td>4184</td>
<td>0.4866</td>
<td>0.9996</td>
</tr>
<tr>
<td>LSAM 6</td>
<td>3.0</td>
<td>10.7</td>
<td>0.585611</td>
<td>166</td>
<td>0.5153</td>
<td>0.9774</td>
</tr>
<tr>
<td>LSAM 8</td>
<td>2.5</td>
<td>22.1</td>
<td>14.37922</td>
<td>4076</td>
<td>0.5356</td>
<td>0.9938</td>
</tr>
<tr>
<td>LSAM 10</td>
<td>2.5</td>
<td>22.6</td>
<td>12.65767</td>
<td>3588</td>
<td>0.3458</td>
<td>0.9998</td>
</tr>
<tr>
<td>LSAM 12</td>
<td>2.5</td>
<td>13.7</td>
<td>2.667</td>
<td>756</td>
<td>0.2856</td>
<td>0.9833</td>
</tr>
<tr>
<td>LSAM 13</td>
<td>2.0</td>
<td>18.0</td>
<td>10.02242</td>
<td>2841</td>
<td>0.5226</td>
<td>0.9968</td>
</tr>
<tr>
<td>LSAM 14</td>
<td>2.0</td>
<td>17.4</td>
<td>7.729361</td>
<td>2191</td>
<td>0.5403</td>
<td>0.9982</td>
</tr>
<tr>
<td>LSAM 16</td>
<td>2.0</td>
<td>23.2</td>
<td>10.50572</td>
<td>2978</td>
<td>0.4183</td>
<td>0.9916</td>
</tr>
<tr>
<td>LSAM 17</td>
<td>2.0</td>
<td>16.8</td>
<td>7.775222</td>
<td>2204</td>
<td>0.5253</td>
<td>0.9998</td>
</tr>
<tr>
<td>LSAM 20</td>
<td>2.5</td>
<td>20.0</td>
<td>8.011583</td>
<td>2271</td>
<td>0.3191</td>
<td>0.9978</td>
</tr>
<tr>
<td>LSAM 24</td>
<td>2.5</td>
<td>19.5</td>
<td>5.9548899</td>
<td>1688</td>
<td>0.5381</td>
<td>0.9927</td>
</tr>
<tr>
<td>D_508 16</td>
<td>2.3</td>
<td>30.5</td>
<td>24.70503</td>
<td>7003</td>
<td>0.5447</td>
<td>0.9865</td>
</tr>
<tr>
<td>D_508 18</td>
<td>2.3</td>
<td>29.8</td>
<td>36.09269</td>
<td>10231</td>
<td>0.3187</td>
<td>0.9824</td>
</tr>
<tr>
<td>C_10 #15</td>
<td>5.0</td>
<td>4.7</td>
<td>0.017639</td>
<td>5</td>
<td>0.9988</td>
<td>0.9984</td>
</tr>
<tr>
<td>C_10 #16</td>
<td>5.0</td>
<td>5.0</td>
<td>0.052917</td>
<td>15</td>
<td>1.0339</td>
<td>0.9852</td>
</tr>
<tr>
<td>Su_WC #2</td>
<td>4.6</td>
<td>6.0</td>
<td>0.116417</td>
<td>33</td>
<td>0.9734</td>
<td>0.8983</td>
</tr>
<tr>
<td>Su_WC #12</td>
<td>4.6</td>
<td>6.1</td>
<td>0.102306</td>
<td>29</td>
<td>0.8433</td>
<td>0.9780</td>
</tr>
<tr>
<td>C_12 #1</td>
<td>5.0</td>
<td>4.1</td>
<td>0.003528</td>
<td>1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.13 shows the values of the Reynolds number and $d_{10}$ for the different mixtures at the hydraulic gradient of 1. Here $d_{10}$ is used to calculate the Reynolds number.
<table>
<thead>
<tr>
<th>Mix Type</th>
<th>LSAM</th>
<th>D_508</th>
<th>Su_WC</th>
<th>C_10</th>
<th>C_12</th>
</tr>
</thead>
<tbody>
<tr>
<td>Re</td>
<td>3  73</td>
<td>10  14</td>
<td>0.03</td>
<td>0.005</td>
<td>0.0004</td>
</tr>
<tr>
<td>d_{10} (mm)</td>
<td>5</td>
<td>0.4</td>
<td>0.32</td>
<td>0.14</td>
<td>0.1</td>
</tr>
</tbody>
</table>

Table 3.13 Reynolds Number for Different Mixes at i = 1

Figure 3.46 shows the relationship between the hydraulic conductivity of pseudo-coefficient of permeability (K') and effective porosity (n_e). Mixes with similar gradation exhibit an increase in K' with the increase in n_e.

![Graph showing K' vs. Effective Porosity](image)

Figure 3.46 K' vs. Effective Porosity

It should be pointed out that if we disregard the fact that Darcy's Law is no longer valid is ignored and the standard procedure is used to analyze the test data, very erroneous results occur. Figure 3.47 shows the curve of the ratio of discharge velocity and hydraulic gradient (w/i), which supposedly being the coefficient of permeability (a

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material constant) under the Darcy’s Law, versus the hydraulic gradient. The figure clearly shows that the ratio of $v/i$ varies greatly with $i$. Therefore, it is meaningless to compare this parameter from different sources. A standard value at $i = 1$ is more reasonable for comparisons.

![Graph showing $v/i$ varies greatly with hydraulic gradient](image)

Figure 3.47 $v/i$ Varies Greatly with Hydraulic Gradient (from Specimen D_16)

3.6.3.7 Estimation of Hydraulic Conductivity

It is more convenient to estimate hydraulic conductivity from volumetric indexes of asphalt mixtures rather than having to perform the hydraulic conductivity testing. The FHWA has published a widely used algorithm based on data from the literature. Most
recently, Richardson (1997) published four predictive formulae based on the data from his own research and the literature. Most of these empirical relations are for granular unbound materials. Hardly any of the existing predictive formulae are for asphalt mixtures. Furthermore, most of the previous test data secured from rigid wall, low head tests, using either constant head or falling head procedures. There was no provision for prevention of water short-circuiting along the permeameter walls. Additionally, manometer ports were not used in many of the permeameters considered (Richardson, 1997).

Theoretical formula (Eq. 3.24) indicates that fluid characteristics, effective porosity, effective grain size, shape and distribution determine hydraulic conductivity. Based on that concept, two regression formulae have been developed from the hydraulic conductivity test results considered in this study. It should be noted that the relationship presented in this study is based on a very set of limited test data. For more generalized empirical relations, more test data will be needed. Figure 3.48 shows the estimated $K'$ values compared to the experimental test results. This figure also includes the estimation of Tan's (1997) test results in which the effective porosity is assumed to be 90% of the air void since the actual test results is not available. Two linear regression equations for $K'$ can be expressed as:

For open-graded mixtures,

$$K'(\text{mm/sec}) = 0.917n_e - 52.41d_{10} + 15.45d_{50} - 1.75C_w + 1.17P_{95} - 143.3 \quad (3.30)$$

\[ (r^2 = 0.8831) \]

For dense-graded mixtures,
\[ K'_{(\text{mm/sec})} = 0.917n - 0.45d_{10} - 0.0273d_{50} + 0.216C_v + 0.00155P_{3/8} - 0.607 \]  
\[ (r^2 = 0.9699) \]

Again it should be emphasized that these regression equations are limited to the hydraulic conductivity test data in this study. More test data will be needed from different types of mixes in order to obtain predictive formulae of practical use.

### 3.6.4 Conclusions of Permeability

Hydraulic conductivity is a fundamental material characteristic that is determined by the properties of the fluid, effective porosity, effective aggregate diameter, particle shape and gradations. Darcy's Law is only valid for dense graded, low permeability asphalt mixtures under the normal test hydraulic gradients. For mixtures with high effective
porosity such as the drainable asphalt mixtures used in this study, Darcy’s Law is no longer valid even for very small hydraulic gradients. A potential form of $v = K' i^n$ can be used for an approximation when the laminar flow condition is not satisfied. A pseudo-coefficient of permeability $K'$ can be used to compare the relative hydraulic conductivity of different materials.

A flexible wall, dual mode permeameter was developed in this study through the modification of Virginia LAB Supply Co.’s flexible wall permeameter cell for the hydraulic conductivity test of asphalt mixtures. The device has been validated through the hydraulic conductivity tests of five different asphalt mixtures used or proposed by the LaDOTD.

The typical values of pseudo-coefficient of permeability $K'$ for the mixtures in this study are: for open-graded large stone asphalt mixture, 2.7 mm/sec (765 ft/day) to 14.8 mm/sec (4190 ft/day), for LA Type 508 open graded drainable base, 24.7 mm/sec (7000 ft/day) to 36.1 mm/sec (10200 ft/day). The coefficient of permeability for dense mixtures varies from 0.003 mm/sec (1 ft/day) to 0.116 mm/sec (33 ft/day).

Statistical models to predict the hydraulic conductivity have been developed for the asphalt mixtures in the range of materials of this study.
CHAPTER 4.
ANALYSIS OF MIXTURE TEST RESULTS

This chapter presents the results of mixture test results. The first part of the chapter (Section 4.1) summarizes the volumetric properties of mixtures. The rest of chapter describes the analysis of the engineering performance test results. A standard statistical procedure, One Way ANOVA has been used to test if the mean values of the fundamental engineering properties are significantly different among the mixtures. A 95-percent confidence level has been utilized to analyze the test results. The ANOVA analysis places sample averages into groups by determining which averages are statistically equal. Groups are designated by letters “A”, “B”, “C”, “AB”, “BC”, etc. Group “A” has a mean that is statistically higher than group “B” and so forth. A designation of “AB” shows that the average can be placed into either its corresponding statistical ranking group “A” or “B”.

4.1 VOLUMETRIC PROPERTIES

A total of eight mixtures were designed and characterized. For the convenience of comparison, these mixtures were coded alphabetically as shown in Table 4.1. The volumetric properties of eight mixtures are presented in Table 4.2.

Table 4.1 Mixtures Evaluated

<table>
<thead>
<tr>
<th>No.</th>
<th>Mixtures</th>
<th>Asphalt</th>
<th>Abbreviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Dense-graded Conventional Base Mix</td>
<td>PG 70-22M</td>
<td>A-P</td>
</tr>
<tr>
<td>II</td>
<td>Dense-graded Superpave LSAM</td>
<td>PG 70-22M</td>
<td>L-P</td>
</tr>
<tr>
<td>III</td>
<td>Dense-graded Superpave LSAM</td>
<td>PG 70-22MAlt</td>
<td>L-MG</td>
</tr>
<tr>
<td>IV</td>
<td>Dense-graded Superpave LSAM</td>
<td>PG 64-22</td>
<td>L-A</td>
</tr>
<tr>
<td>V</td>
<td>Conventional Open-graded Drainable Mix</td>
<td>PG 70-22M</td>
<td>DT-P</td>
</tr>
<tr>
<td>VI</td>
<td>Open-graded LSAM</td>
<td>PG 70-22M</td>
<td>OG-P</td>
</tr>
<tr>
<td>VII</td>
<td>Open-graded LSAM</td>
<td>PG 70-22MAlt</td>
<td>OG-MG</td>
</tr>
<tr>
<td>VIII</td>
<td>Open-graded LSAM</td>
<td>PG 64-22</td>
<td>OG-A</td>
</tr>
</tbody>
</table>
Table 4.2 Volumetric Properties of Mixtures

<table>
<thead>
<tr>
<th></th>
<th>A-P</th>
<th>L-P</th>
<th>L-MG</th>
<th>L-A</th>
<th>DT-P</th>
<th>OG-P</th>
<th>OG-MG</th>
<th>OG-A</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC (%)</td>
<td>3.5</td>
<td>3.8</td>
<td>3.8</td>
<td>3.8</td>
<td>2.3</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>NDo (SGC)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>25</td>
<td>25</td>
<td>25</td>
<td>25</td>
</tr>
<tr>
<td>Conn</td>
<td>2.547</td>
<td>2.546</td>
<td>2.546</td>
<td>2.546</td>
<td>2.584</td>
<td>2.592</td>
<td>2.592</td>
<td>2.592</td>
</tr>
<tr>
<td>Gm b (average)</td>
<td>2.459</td>
<td>2.440</td>
<td>2.399</td>
<td>2.442</td>
<td>1.760</td>
<td>1.915</td>
<td>1.932</td>
<td>1.899</td>
</tr>
<tr>
<td>Air voids (%)</td>
<td>4.3</td>
<td>3.9</td>
<td>4.1</td>
<td>4.0</td>
<td>34.3</td>
<td>26.2</td>
<td>25.3</td>
<td>27.0</td>
</tr>
<tr>
<td>VMA (%)</td>
<td>12.1</td>
<td>12.1</td>
<td>13.0</td>
<td>12.6</td>
<td>36.0</td>
<td>30.7</td>
<td>29.8</td>
<td>31.7</td>
</tr>
<tr>
<td>VFA (%)</td>
<td>67.1</td>
<td>69.5</td>
<td>68.5</td>
<td>68.8</td>
<td>10.8</td>
<td>14.9</td>
<td>14.5</td>
<td>13.5</td>
</tr>
<tr>
<td>Film Thickness (micron)</td>
<td>7.07</td>
<td>7.3</td>
<td>7.3</td>
<td>7.3</td>
<td>38.9</td>
<td>30.2</td>
<td>30.2</td>
<td>30.2</td>
</tr>
<tr>
<td>VCA (%)</td>
<td>41.2</td>
<td>41.2</td>
<td>41.2</td>
<td>41.2</td>
<td>44.9</td>
<td>44.9</td>
<td>44.9</td>
<td>44.9</td>
</tr>
<tr>
<td>SSC (%)</td>
<td>52</td>
<td>85</td>
<td>85</td>
<td>85</td>
<td>50</td>
<td>92</td>
<td>92</td>
<td>92</td>
</tr>
</tbody>
</table>

The volumetric properties of the mixtures indicated that the most significant difference between the LSAM and conventional mixtures is their degree of stone-on-stone contact (SSC%). The conventional Type 5A base course mixture (A-P) had a degree of stone-on-stone contact of 52 percent, while its LSAM counter parts had the SSC of 85 percent. For the open-graded mixtures, the conventional Type 508 drainable base mixture (DT-P) had a degree of stone-on-stone contact of 50 percent while the open-graded large stone asphalt mixtures had the SSC of 92 percent.

4.2 ELASTIC PROPERTIES

Elastic Properties of asphalt mixtures were measured using results from the indirect tensile resilient modulus ($M_R$) test. The $M_R$ tests were conducted at three temperatures (4, 25 and 40 °C). In the current AASHTO design procedure, the structural number of the asphalt pavement layer was correlated to the value of resilient modulus ($M_R$). The higher the resilient modulus, the stronger support the asphalt concrete layer can provide (AASHTO, 1987). The average values of the resilient modulus for eight mixtures are presented in Figure 4.1 and their statistical groupings are presented in Tables 4.3 and 4.4.
The resilient modulus values of all eight mixtures decreased as the testing temperatures increased, as expected, since HMACs are known to be stiffer at lower temperature.

![Figure 4.1 Average Values of Resilient Modulus](image)

4.2.1 Comparison Between LSAM and Conventional Mixtures

Table 4.3 presents the statistical comparison of the resilient modulus ($M_R$) of the LSAM and conventional mixtures. It appeared that there was no significant differences of $M_R$ values between the open-graded and dense-graded LSAM and their conventional counterpart mixtures at all three temperatures, except at 40 °C, where the conventional dense-graded Type 5A base course mixture showed significant higher $M_R$ value than the dense-graded LSAM mixture.
Table 4.3 Comparisons of $M_R$ Between the LSAM and Conventional Mixes

<table>
<thead>
<tr>
<th>Temperature</th>
<th>Engineering Property</th>
<th>Mixture Type</th>
<th>Dense-graded</th>
<th>Open-graded</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>A-P</td>
<td>L-P</td>
</tr>
<tr>
<td>4°C (40°F)</td>
<td>$M_R$ (GPa)</td>
<td>17.68</td>
<td>17.97</td>
<td>6.13</td>
</tr>
<tr>
<td></td>
<td>Ranking</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>25°C (77°F)</td>
<td>$M_R$ (GPa)</td>
<td>7.42</td>
<td>5.29</td>
<td>1.17</td>
</tr>
<tr>
<td></td>
<td>Ranking</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>40°C (104°F)</td>
<td>$M_R$ (GPa)</td>
<td>2.12</td>
<td>1.24</td>
<td>0.77</td>
</tr>
<tr>
<td></td>
<td>Ranking</td>
<td>A</td>
<td>B</td>
<td>A</td>
</tr>
</tbody>
</table>

Columns (for each mix type) with similar letter indicate no significant difference.

4.2.2 Effect of AC Types to the Mixtures

Table 4.4 presents the statistical grouping of the resilient modulus for the dense and open-graded mixtures with three different binders. At 4 °C and 25 °C temperatures, asphalt types showed no significant influence on the resilient modulus of the dense and open-graded LSAM mixtures. At 40 °C, the dense-graded LSAM with PG 60-22 and fiber exhibited significantly higher $M_R$ values than the LSAM with SB polymer modified asphalt PG 70-22M. The indirect tensile resilient modulus of the dense-graded LSAM with gelled asphalt PG 70-22MAlt was not significantly different from either the one with PG 64-22 and fiber, or the one with polymer modified asphalt PG 70-22M. At 40 °C, asphalt types showed no effect on the $M_R$ values of the open-graded LSAM mixtures.

Table 4.4 Comparisons of $M_R$ for LSAMs with Different Binder Types

<table>
<thead>
<tr>
<th>Temperature</th>
<th>Engineering Property</th>
<th>Mixture Type</th>
<th>Dense-graded</th>
<th>Open-graded</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>L-P</td>
<td>L-MG</td>
</tr>
<tr>
<td>4°C (40°F)</td>
<td>$M_R$ (GPa)</td>
<td>17.97</td>
<td>17.14</td>
<td>22.62</td>
</tr>
<tr>
<td></td>
<td>Ranking</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>25°C (77°F)</td>
<td>$M_R$ (GPa)</td>
<td>5.29</td>
<td>5.97</td>
<td>9.72</td>
</tr>
<tr>
<td></td>
<td>Ranking</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>40°C (104°F)</td>
<td>$M_R$ (GPa)</td>
<td>1.24</td>
<td>1.92</td>
<td>2.34</td>
</tr>
<tr>
<td></td>
<td>Ranking</td>
<td>B</td>
<td>A/B</td>
<td>A</td>
</tr>
</tbody>
</table>

Columns (for each mix type) with similar letter indicate no significant difference.
4.3 PERMANENT DEFORMATION PROPERTIES

Permanent deformation properties of asphalt mixtures were characterized through the axial creep test at 40 °C, the indirect tensile creep test at 40 °C, the frequency sweep at constant height (FSCH) test at 60 °C, the repetitive shear at constant height (RSCH) test at 60 °C, and the APA rut test at 60 °C.

4.3.1 Axial Creep Test

Table 4.5 and 4.6 presents the results of axial creep test. In this test, lower slope value, higher stiffness and lower permanent strain are desired for rut-resistant mixtures.

- Comparisons Between LSAM and Conventional Mixtures

Based on axial creep test results (Table 4.5), it is evident that open-graded LSAM mixtures exhibited higher rut-resistance than the conventional Type 508 open-graded drainable base mixture. The dense-graded LSAM had higher average values of stiffness, as well as lower creep slope and permanent strain than the conventional Type 5A mix. However, statistical analysis showed no significant differences between the LSAM and conventional dense-graded mixtures.

Table 4.5 Axial Creep Test for Dense-graded and Open-graded Mixtures

<table>
<thead>
<tr>
<th>Engineering Property</th>
<th>Mixture Type</th>
<th>Dense-graded</th>
<th></th>
<th></th>
<th>Open-graded</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LSAM</td>
<td>Conventional</td>
<td>LSAM</td>
<td>Conventional</td>
<td>Sample</td>
<td>Failed</td>
<td>Failed</td>
</tr>
<tr>
<td>Stiffness (MPa)</td>
<td>58.2</td>
<td>53.6</td>
<td>30.4</td>
<td></td>
<td>Sample</td>
<td>Failed</td>
<td>Failed</td>
</tr>
<tr>
<td>Ranking</td>
<td>A</td>
<td>A</td>
<td>-</td>
<td></td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
</tr>
<tr>
<td>Slope (x10^5 sec^-1)</td>
<td>6.70</td>
<td>12.2</td>
<td>32.0</td>
<td></td>
<td>Sample</td>
<td>Failed</td>
<td>Failed</td>
</tr>
<tr>
<td>Ranking</td>
<td>A</td>
<td>A</td>
<td>-</td>
<td></td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
</tr>
<tr>
<td>Permanent Strain (x10^-4)</td>
<td>12.1</td>
<td>13.6</td>
<td>22.9</td>
<td></td>
<td>Sample</td>
<td>Failed</td>
<td>Failed</td>
</tr>
<tr>
<td>Ranking</td>
<td>A</td>
<td>A</td>
<td>-</td>
<td></td>
<td>Failed</td>
<td>Failed</td>
<td>Failed</td>
</tr>
</tbody>
</table>

Columns (for each mixture type) with similar letter indicate no significant difference.
- **Effect of AC Types to the LSAM Mixtures**

Table 4.6 presents the effect of asphalt binders on the axial creep properties of open-graded and dense-graded LSAM mixtures. Asphalt types showed no significant influence on the axial creep test results of the open-graded large stone asphalt mixtures (OG-P, OG-MG and OG-A). For the dense-graded 37.5-mm Superpave LSAM (L-P, L-MG and L-A), the mixture containing SB polymer-modified asphalt cement (L-P) exhibited significant higher rut resistance than the one containing gelled asphalt (L-MG). The axial creep properties of the mixture containing PG 64-22 (L-A) was between the other two mixtures and showed no significant difference from either L-P or L-MG.

<table>
<thead>
<tr>
<th>Table 4.6 Axial Creep Test for LSAMs with Different Binder Types</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Engineering Property</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Stiffness (MPa)</td>
</tr>
<tr>
<td>Ranking</td>
</tr>
<tr>
<td>Slope (x10^3 sec^-1)</td>
</tr>
<tr>
<td>Ranking</td>
</tr>
<tr>
<td>Permanent Strain (x10^-4)</td>
</tr>
<tr>
<td>Ranking</td>
</tr>
</tbody>
</table>

Note: Columns (for each mixture type) with similar letter indicate no significant difference.

**4.3.2 Indirect Tensile Creep Test**

The indirect tensile creep test results of the eight mixes at 40 °C (104 °F) are presented in Tables 4.7 and 4.8. In this test, flat slopes and longer failure time are indicative of rut-resistant mixtures. The conventional Type 508 open-graded drainable base mixture failed at the start of the test. Among the open-graded LSAM mixtures, the mixture containing SB polymer-modified AC (OG-P) showed lower creep slope and longer time to failure.
than the mixture containing PG 64-22 (OG-A), whereas, the mixture containing gelled asphalt (OG-MG) showed no significant difference from either OG-P or OG-A. Among the dense-graded mixtures (Table 4.8), all the mixtures exhibited similar test results.

Table 4.7 Indirect Tensile Creep Test Results of Open and Dense-graded Mixes

<table>
<thead>
<tr>
<th>Engineering Property</th>
<th>Mixture Type</th>
<th>Dense-graded</th>
<th>Open-graded</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>L-P (LSAM)</td>
<td>A-P (Conventional)</td>
</tr>
<tr>
<td>Creep Slope (log(psi)/log(sec))</td>
<td>0.24</td>
<td>0.28</td>
<td>0.51</td>
</tr>
<tr>
<td>Ranking</td>
<td>A</td>
<td>A</td>
<td>-</td>
</tr>
<tr>
<td>Time to Failure (sec)</td>
<td>&gt;3600</td>
<td>&gt;3600</td>
<td>206.5</td>
</tr>
<tr>
<td>Ranking</td>
<td>A</td>
<td>A</td>
<td>-</td>
</tr>
</tbody>
</table>

Columns (for each mix type) with similar letter indicate no significant difference.

Table 4.8 Indirect Tensile Creep Test Results for LSAMs with Different Binder Types

<table>
<thead>
<tr>
<th>Engineering Property</th>
<th>Mixture Type</th>
<th>Dense-graded LSAM</th>
<th>Open-graded LSAM</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>L-P</td>
<td>L-MG</td>
</tr>
<tr>
<td>Creep Slope (log(psi)/log(sec))</td>
<td>0.24</td>
<td>0.29</td>
<td>0.30</td>
</tr>
<tr>
<td>Ranking</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>Time to Failure (sec)</td>
<td>&gt;3600</td>
<td>&gt;3600</td>
<td>&gt;3600</td>
</tr>
<tr>
<td>Ranking</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
</tbody>
</table>

Columns (for each mix type) with similar letter indicate no significant difference.

- **Comparisons Between LSAM and Conventional Mixtures**

Indirect tensile creep test results (Table 4.7) indicate that open-graded LSAM mixtures exhibited higher rut-resistance than the conventional Type 508 open-graded drainable base mixture, which failed prematurely during the test.
For the dense-graded mixtures, however, there was no significant difference between the 37.5-mm Superpave LSAM and the conventional dense-graded Type 5A based mixture.

- **Effect of AC Types to the LSAM Mixtures**

  Asphalt types showed no significant influence to the indirect tensile creep test results of the 37.5-mm Superpave large stone asphalt mixtures (L-P, L-MG and L-A). For the open-graded LSAM, mixture containing SB polymer-modified asphalt cement (OG-P) exhibited significant higher rut resistance than the mixture containing PG 64-22 asphalt cement (OG-A). The mixture containing gelled asphalt cement (OG-MG) showed significantly shorter time to failure than OG-P, but no significant difference in creep slope from either OG-P or OG-A.

4.3.3 **Superpave Simple Shear Frequency Sweep at Constant Height (FSCH)**

The Superpave simple shear frequency sweep at constant height (FSCH) test evaluates the viscoelastic characteristics of the mixtures. Materials properties obtained from this test are dynamic shear modulus ($G^*$) and shear phase angles ($\delta$) as shown in Figures 4.2 though 4.5.

Dynamic shear modulus ($G^*$) is defined as the ratio of the peak stress amplitude to the peak strain amplitude. It is a measure of total stiffness of asphalt mixtures and is composed of elastic and viscous components of asphalt mixture stiffness. Thus far, the correlation between dynamic shear modulus and pavement rutting has not been well established although it is well known that, for a stiff mixture, the strain generated in asphalt pavement under traffic loading is relatively small and, therefore, the pavement
rutting performance is enhanced. Phase angle is defined as the time lag between the application of a stress and the resulting strain.

Dynamic shear modulus increased with the increase in frequency. It appears that for the open-graded mixes (Figure 4.2), OG-P had the highest dynamic complex shear modulus at 10 Hz, followed by the OG-MG, OG-A and DT-P. Whereas for the dense-graded mixes (Figure 4.3), L-MG exhibited the highest dynamic complex shear modulus at 10 Hz, followed by L-A, A-P and L-P. At low frequency (0.01 Hz), Mix OG-P showed the highest dynamic shear modulus in the open-graded mix group, followed by DT-P, OG-A and OG-MG. Whereas for the dense-graded mix group, conventional Type 5A base mix (A-P) exhibited the highest dynamic shear modulus, followed by L-MG, L-A and L-P.

![Dynamic Shear Modulus (G*) of Open-Graded Mixes](image)

*Figure 4.2 FSCH Dynamic Shear Modulus (G*) of Open-graded Mixtures*
Figure 4.3 FSCH Dynamic Shear Modulus (G*) of Dense-graded Mixtures

The shear phase angles for all the asphalt mixtures increased with increasing frequency (Figures 4.4 and 4.5), which is different from asphalt binder in that the shear phase angle for asphalt binder generally decreases with increasing frequency. The explanation is as follows:

If the frequency sweep test for the asphalt mixture were performed at different temperatures and the master curve were created, shear phase angle would increase with increasing frequency, reach a peak, and then decrease (Alavi, et al, 1994, Fonseca, 1995, Mohammad, et al, 1999, Monismith, et al, 1994, Sousa and Weismann, 1994). This is because at high frequency (low temperature), the phase angle of asphalt mixtures is primarily affected by the asphalt binder. Hence, the shear phase angle of the asphalt binder and asphalt mixture follows similar trend. However, at low frequency (high
temperature), it is predominantly affected by the aggregate, and therefore, the shear phase angle for asphalt mixtures decreases with decreasing frequency or increasing temperature because of the aggregate influence.

It appears that the shear phase angle for all the asphalt mixtures at 60 °C would only represent the left side portion of the master curve in which the aggregate influence becomes more important.

![FSCH Phase Angle of Open-graded Mixes](image)

Figure 4.4 FSCH Phase Angle (δ) of Open-graded Mixtures

- **Comparisons Between LSAM and Conventional Mixtures**

The open-graded large stone asphalt mixtures with SB polymer modified asphalt (OG-P) exhibited significantly higher dynamic shear modulus (G*) at both 10 Hz and 0.01 Hz than the conventional Type 508 drainable base mixture (DT-P). There was no significant
difference in the FSCH phase angle (δ) between Mix DT-P and OG-P at frequencies of either 10 or 0.01 Hz.

For the dense-graded mixtures, there was no significant difference in the dynamic shear modulus (G*) and the phase angle (δ) between the 37.5-mm Superpave LSAM (L-P) and the conventional dense-graded Type 5A base mixture (A-P) at frequencies of either 10 or 0.01 Hz.

Figure 4.5 FSCH Phase Angle (δ) of Dense-graded Mixtures

- **Effect of AC Types to the LSAM Mixtures**

  Asphalt types showed no significant influence to both dynamic shear modulus (G*) and phase angle (δ) of the 37.5-mm Superpave large stone asphalt mixtures (L-P, L-MG and L-A) at the frequency of 0.01 Hz. At frequency of 10 Hz, Mix L-MG exhibited significantly higher G* and δ values than Mix L-P, whereas, Mix L-A showed no
significant difference in both G* values to either L-P or L-MG. Mix L-P had a significantly lower value in δ than L-MG and L-A at the frequency of 10 Hz.

Table 4.9 FSCH Results for Open-graded and Dense-graded Mixtures

<table>
<thead>
<tr>
<th>Engineering Properties</th>
<th>Mixtures</th>
<th>Dense-graded</th>
<th>Open-graded</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>L-P (LSAM)</td>
<td>A-P (Conventional)</td>
</tr>
<tr>
<td>@ 10 Hz Frequency</td>
<td></td>
<td>69.2</td>
<td>74.2</td>
</tr>
<tr>
<td>G* (MPa)</td>
<td></td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>δ (°)</td>
<td></td>
<td>41.4</td>
<td>40.0</td>
</tr>
<tr>
<td>@ 0.01 Hz Frequency</td>
<td></td>
<td>20.6</td>
<td>22.4</td>
</tr>
<tr>
<td>G* (MPa)</td>
<td></td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>δ (°)</td>
<td></td>
<td>21.3</td>
<td>21.1</td>
</tr>
<tr>
<td>Ranking</td>
<td></td>
<td>A</td>
<td>A</td>
</tr>
</tbody>
</table>

Columns (for each mix type) with similar letter indicate no significant difference.

Table 4.10 FSCH Results for LSAMs with Different Binder Types

<table>
<thead>
<tr>
<th>Engineering Property</th>
<th>Mixtures</th>
<th>Dense-graded LSAM</th>
<th>Open-graded LSAM</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>L-P</td>
<td>L-MG</td>
</tr>
<tr>
<td>@ 10 Hz Frequency</td>
<td></td>
<td>69.2</td>
<td>103.2</td>
</tr>
<tr>
<td>G* (MPa)</td>
<td></td>
<td>B</td>
<td>A</td>
</tr>
<tr>
<td>δ (°)</td>
<td></td>
<td>41.4</td>
<td>46.8</td>
</tr>
<tr>
<td>Ranking</td>
<td></td>
<td>B</td>
<td>A</td>
</tr>
<tr>
<td>@ 0.01 Hz Frequency</td>
<td></td>
<td>20.6</td>
<td>21.4</td>
</tr>
<tr>
<td>G* (MPa)</td>
<td></td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>δ (°)</td>
<td></td>
<td>21.3</td>
<td>21.2</td>
</tr>
<tr>
<td>Ranking</td>
<td></td>
<td>B</td>
<td>B</td>
</tr>
</tbody>
</table>

Columns (for each mix type) with similar letter indicate no significant difference.

For the open-graded large stone asphalt mixtures, Mix OG-P exhibited significantly higher dynamic shear modulus (G*) than both OG-MG and OG-A at frequencies of 10 and 0.01 Hz. There was no significant difference in the G* values.
between mixes OG-MG and OG-A at either 10 or 0.01 Hz. Mix OG-A showed significantly higher values of phase angle (δ) than OG-P at frequencies of both 10 and 0.01 Hz. Mix OG-MG exhibited no significant difference in the phase angle (δ) to either OG-P or OG-A at either frequencies of 10 and 0.01 Hz.

4.3.4 Superpave Simple Shear Repetitive Shear at Constant Height (RSCH)

A pavement rutting performance prediction model was developed during SHRP – A003A project (Monismith et al, 1994). Permanent shear strain obtained from the repeated shear test at constant height (RSCH) can be input into this performance prediction model to predict rut depth as a function of equivalent single axle loads (ESALs). This model was based on a relationship between rut depth and maximum shear strain that was developed from a non-linear elastic, visco-plastic constitutive equation to describe the behavior of the asphalt concrete incorporated into a finite element program (Sousa, Solaimanian and Weissman, 1994).

It was found that all the open-graded large stone asphalt mixtures (OG-L, OG-MG and OG-A) as well as the conventional Type 508 drainable base mix (DT-P) failed within the first few cycles of the test, therefore, can not be characterized by RSCH at 60 °C. Among the dense-graded mixes, it appeared that Mix L-MG had the highest permanent shear strain, followed by Mixes A-P, L-A and L-P.

• Comparisons Between LSAM and Conventional Mixtures

Figure 4.6 presents the permanent shear strain as a function of load repetitions. The open-graded LSAMs as well as the conventional Type 508 drainable base mix failed within the first few cycles of the test, therefore, can not be characterized by the RSCH at 60 °C.
The dense-graded LSAM had lower permanent shear strain at 5000 cycles than the conventional Type 5A base mixture, although the difference was not statistically significant (Table 4.11).

Table 4.11 Permanent Shear Strain at 5000 Cycles of RSCH Test

<table>
<thead>
<tr>
<th>Engineering Property</th>
<th>Mixture Type</th>
<th>Dense-graded</th>
<th>Open-graded</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>L-P (LSAM)</td>
<td>A-P (Conventional)</td>
</tr>
<tr>
<td>Shear Strain @5000 Cycles (%)</td>
<td>1.29</td>
<td>1.04</td>
<td>Sample Failed</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>0.1997</td>
<td>0.2107</td>
<td>-</td>
</tr>
<tr>
<td>Coefficient of Variation (%)</td>
<td>15.5</td>
<td>20.3</td>
<td>-</td>
</tr>
<tr>
<td>Ranking</td>
<td>A</td>
<td>A</td>
<td>-</td>
</tr>
</tbody>
</table>

Columns (for each mix type) with similar letter indicate no significant difference.
• **Effect of AC Types to the LSAM Mixtures**

Among the dense-graded LSAM mixtures, the one with gelled asphalt PG70-22Malt (L-MG) had the highest permanent shear strain, followed by mixes containing PG64-22 (Mix L-A) and PG70-22M (Mix L-P). Statistical analysis, however, indicated that there was no significant difference of the permanent strains at 5000 cycles among the dense-graded mixtures, Table 4.12.

### Table 4.12 Permanent Shear Strain at 5000 Cycles of RSCH Test for LSAMs with Different AC Binders

<table>
<thead>
<tr>
<th>Engineering Property</th>
<th>Dense-graded Superpave LSAM</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>L-P (PG 70-22M)</td>
</tr>
<tr>
<td>Shear Strain @5000 Cycles (%)</td>
<td>1.04</td>
</tr>
<tr>
<td>Ranking</td>
<td>A</td>
</tr>
</tbody>
</table>

Columns (for each mixture type) with similar letter indicate no significant difference.

### 4.3.5 APA Rut Test

Asphalt Pavement Analyzer (APA) is the new generation of the Georgia Loaded Wheel Tester (GLWT). The APA can test three beam specimens (320 x 127 x 76 mm) or six cylindrical specimens (150 mm x 76 mm) simultaneously. The concave shaped wheels travel back and forth over a stiff, pressurized rubber hose, which rests directly on the specimen. Typical test conditions as set by the Georgia DOT specification are only for beam specimens. The Georgia specification sets a test temperature of 40 °C, vertical load of 444.4 N, and 0.7 MPa hose pressure with a criterion of no more than 7.6 mm rut depth after 8,000 cycles (16,000 passes) under dry conditions. The wheel speed is approximately 60 cm/sec.
In this study, cylindrical specimens of the eight mixtures were tested at a temperature of 60 °C. Vertical load, hose pressure and wheel speed were the same as specified in Georgia specification. An automated system that continuously measures the rut depth was adopted.

Figures 4.7 and 4.8 present the APA rut depth versus load repetitions of the open-graded and dense-graded mixtures. Figures 4.9 and 4.10 present the slope of rut depth versus load repetitions of the open-graded and dense-graded mixtures. It appears that in the group of open-graded mixes (Figures 4.7, 4.9), Type 508 drainable base mix (DT-P) had the highest rut depth (in fact, it failed prematurely within the first 1000 cycles), whereas, in the group of dense-graded mixtures (Figures 4.8, 4.10), Mix L-A exhibited higher rutting than the other three mixes (A-P, L-P and L-MG).

Figure 4.7 APA Rut Depth Vs. Number of Cycles for Open-graded Mixtures

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Figure 4.8 APA Rut Depth Vs. Number of Cycles for Dense-graded Mixtures

Figure 4.9 APA Slope Vs. Number of Cycles for Open-graded Mixtures
• Comparisons Between LSAM and Conventional Mixtures

Table 4.13 presents a comparison of the results of APA rut test for the two mixtures considered. The open-graded conventional Type 508 drainable base mixture exhibited much higher values of rut depth than the open-graded LSAM. However, for the dense-graded mixtures, there was no significant difference in the final rut depth between the LSAM and the conventional Type 5A base mixture.

Table 4.13 APA Rut Depth at 8,000 Cycles for Dense and Open-graded Mixtures

<table>
<thead>
<tr>
<th>Engineering Property</th>
<th>Mixture Type</th>
<th>Dense-graded</th>
<th>Open-graded</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LSAM (L-P)</td>
<td>Conventional (A-P)</td>
<td>LSAM (OG-P)</td>
</tr>
<tr>
<td>Rut Depth at 5000 Cycles (mm)</td>
<td>3.20</td>
<td>3.23</td>
<td>5.01</td>
</tr>
<tr>
<td>Ranking</td>
<td>A</td>
<td>A</td>
<td>B</td>
</tr>
</tbody>
</table>

Columns (for each mixture type) with similar letter indicate no significant difference.
The effect of the binder type on the rut depth of APA is presented in Table 5.14. The types of the asphalt cement considered in this study showed no significant influence on the APA rut test results for the open-graded LSAM mixtures (OG-P, OG-MG and OG-A). However, for the dense-graded LSAMs, the mix (L-P) with PG 70-22M (SB polymer modified) and L-MG with PG 70-22MAlt (gelled) exhibited significantly lower rut depth than did the mix with PG 64-22 binder (L-A). There was no significant difference in the APA rut test results between mixes with PG 70-22M (L-P) and PG 70-22MAlt (L-MG).

### Table 4.14 APA Rut Depth at 8,000 Cycles for LSAMs with Different Binder Types

<table>
<thead>
<tr>
<th>Engineering Property</th>
<th>Mixture Type</th>
<th>Dense-graded LSAM</th>
<th>Open-graded LSAM</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>PG 70-22M</td>
<td>PG 70-22M Alt.</td>
</tr>
<tr>
<td></td>
<td>L-P</td>
<td>L-MG</td>
<td>L-A</td>
</tr>
<tr>
<td>Rut Depth (mm)</td>
<td>3.20</td>
<td>3.40</td>
<td>4.51</td>
</tr>
<tr>
<td>Ranking</td>
<td>B</td>
<td>B</td>
<td>A</td>
</tr>
</tbody>
</table>

Columns (for each mixture type) with similar letter indicate no significant difference.

### 4.3.6 Summary of Permanent Deformation Properties

To summarize the permanent deformation characterization of mixtures, the following method is applied to rank the rut resistance of each individual mixture.

When ranking a specific test, the overall points for the test will be based on the statistical rankings from Tables 4.6 through 4.14. The point for a specific test ranged from 0 to 2, depending on the statistical grouping of “A”, “B”, or “C”. In an “A” means more rut-resistant for that test, it will be assigned as 2 points, “B” would be 1 point and “C”, 0 point (since we didn’t have more than three mixes to compare). This assignment
could be reversed if the group “C” meant to be more rut-resistant than “B” and “A” (such as the APA rut depth), in which case “C” would be 2, “B” 1 and “A”, 0. If a test has more than one parameter, each parameter will count for a fraction of the points to make the total points the same. For example, an “A” in the stiffness of the axial creep test would count as 2/3 points since the test has three parameters.

- **Comparisons Between LSAM and Conventional Mixtures**

Table 4.15 and 4.16 presents the comparisons of overall ranking of rut-susceptibility for the dense and open-graded mixtures between the LSAM and conventional mixtures. It was evident (from Table 4.15) that open-graded large stone asphalt mixture (OG-P) exhibited better rut-resistance than the conventional Louisiana Type 508 drainable base mixture (DT-P). The former had an overall ranking of 2 while the latter had a ranking of 1. The dense-graded LSAM (Superpave LSAM, L-P) showed very similar rut-resistance when compared to the conventional Louisiana Type 5A base mixture (A-P). Both of them had an overall ranking point of 2 for rut-resistance (see Table 4.15).

### Table 4.15 Rut Susceptibility of Dense-graded Mixtures

<table>
<thead>
<tr>
<th>Fundamental Engineering Tests</th>
<th>Engineering Properties</th>
<th>LSAM (L-P)</th>
<th>Conventional (A-P)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Creep</td>
<td>Stiffness</td>
<td>A 2/3</td>
<td>A 2/3</td>
</tr>
<tr>
<td></td>
<td>Slope</td>
<td>A 2/3</td>
<td>A 2/3</td>
</tr>
<tr>
<td></td>
<td>Permanent Strain</td>
<td>A 2/3</td>
<td>A 2/3</td>
</tr>
<tr>
<td>Indirect Tensile Creep</td>
<td>Creep Slope</td>
<td>A 2/2</td>
<td>A 2/2</td>
</tr>
<tr>
<td></td>
<td>Time To Failure</td>
<td>A 2/2</td>
<td>A 2/2</td>
</tr>
<tr>
<td>FSCH</td>
<td>G* @0.01Hz</td>
<td>A 2/2</td>
<td>A 2/2</td>
</tr>
<tr>
<td></td>
<td>G* @10Hz</td>
<td>A 2/2</td>
<td>A 2/2</td>
</tr>
<tr>
<td>RSCH</td>
<td>Strain @5000</td>
<td>A 2</td>
<td>A 2</td>
</tr>
<tr>
<td>APA</td>
<td>Rut @8000</td>
<td>A 2</td>
<td>A 2</td>
</tr>
<tr>
<td><strong>OVERALL RANKING</strong></td>
<td></td>
<td><strong>2</strong></td>
<td><strong>2</strong></td>
</tr>
</tbody>
</table>
• Comparisons Among the LSAMs with Different AC Types

Tables 4.17 and 4.18 present the comparisons of overall rut-susceptibility ranking of the open and dense-graded LSAMs with three different asphalt binders. For the open-graded LSAM mixtures (Table 5.17), the one with SB polymer modified PG 70-22M (Mix OG-P) showed better overall rut-resistance than the other two mixtures (OG-MG and OG-A). The mixture with PG 64-22 and fiber (OG-A) showed slightly better overall ranking in rut-resistance than the one with gelled asphalt, PG 70-22MAlt (Mix OG-MG).

Table 4.16 Rut Susceptibility of Open-graded Mixtures

<table>
<thead>
<tr>
<th>Fundamental Engineering Tests</th>
<th>Engineering Properties</th>
<th>LSAM (OG-P)</th>
<th>Conventional (DT-P)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Creep</td>
<td>Stiffness</td>
<td>A 2/3</td>
<td>B 1/3</td>
</tr>
<tr>
<td></td>
<td>Slope</td>
<td>A 2/3</td>
<td>B 1/3</td>
</tr>
<tr>
<td></td>
<td>Permanent Strain</td>
<td>A 2/3</td>
<td>B 1/3</td>
</tr>
<tr>
<td>Indirect Tensile Creep</td>
<td>Creep Slope</td>
<td>A 2/2</td>
<td>B 1/2</td>
</tr>
<tr>
<td></td>
<td>Time To Failure</td>
<td>A 2/2</td>
<td>B 1/2</td>
</tr>
<tr>
<td>FSCH</td>
<td>(G^* @0.01)Hz</td>
<td>A 2/2</td>
<td>B 1/2</td>
</tr>
<tr>
<td></td>
<td>(G^* @10)Hz</td>
<td>A 2/2</td>
<td>B 1/2</td>
</tr>
<tr>
<td>RSCH</td>
<td>Strain @5000</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>APA</td>
<td>Rut @8000</td>
<td>B 2</td>
<td>A 1</td>
</tr>
<tr>
<td>OVERALL RANKING</td>
<td></td>
<td>2</td>
<td>1</td>
</tr>
</tbody>
</table>

The dense-graded LSAMs (Table 4.18) exhibited a pattern similar to that of the open-graded LSAM. The dense-graded LSAM with SB polymer modified PG 70-22M (Mix L-P) exhibited the best rut-resistance among the three mixtures (the other to mixes, L-MG and L-A). The mixture with PG 64-22 (L-A) exhibited slightly better overall rut-resistance than the one with gelled asphalt, PG 70-22MAlt (Mix L-MG).
Table 4.17 Rut Susceptibility of Open-graded LSAM with Different Asphalt Cements

<table>
<thead>
<tr>
<th>Fundamental Engineering Tests</th>
<th>Engineering Properties</th>
<th>PG 70-22M (OG-P)</th>
<th>PG 70-22MAlt (OG-MG)</th>
<th>PG 64-22 (OG-A)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Rank</td>
<td>Points</td>
<td>Rank</td>
</tr>
<tr>
<td>Axial Creep</td>
<td></td>
<td></td>
<td>2/3</td>
<td></td>
</tr>
<tr>
<td>Stiffness</td>
<td>A</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slope</td>
<td>A</td>
<td></td>
<td>2/3</td>
<td></td>
</tr>
<tr>
<td>Permanent Strain</td>
<td>A</td>
<td></td>
<td>2/3</td>
<td></td>
</tr>
<tr>
<td>Indirect Tensile Creep</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Creep Slope</td>
<td>A</td>
<td></td>
<td>2/2</td>
<td></td>
</tr>
<tr>
<td>Time To Failure</td>
<td>A</td>
<td></td>
<td>2/2</td>
<td></td>
</tr>
<tr>
<td>FSCH</td>
<td>G* @0.01Hz</td>
<td>A</td>
<td>2/2</td>
<td></td>
</tr>
<tr>
<td>G* @10Hz</td>
<td>A</td>
<td></td>
<td>2/2</td>
<td></td>
</tr>
<tr>
<td>RSCH</td>
<td>Strain @5000</td>
<td></td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>APA</td>
<td>Rut @8000</td>
<td>A</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>OVERALL RANKING</td>
<td>2.00</td>
<td></td>
<td>1.56</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.18 Rut Susceptibility of Dense-graded LSAM with Different Asphalt Cements

<table>
<thead>
<tr>
<th>Fundamental Engineering Tests</th>
<th>Engineering Properties</th>
<th>PG 70-22M (L-P)</th>
<th>PG 70-22MAlt (L-MG)</th>
<th>PG 64-22 (L-A)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Rank</td>
<td>Points</td>
<td>Rank</td>
</tr>
<tr>
<td>Axial Creep</td>
<td></td>
<td></td>
<td>2/3</td>
<td></td>
</tr>
<tr>
<td>Stiffness</td>
<td>A</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slope</td>
<td>B</td>
<td></td>
<td>2/3</td>
<td></td>
</tr>
<tr>
<td>Permanent Strain</td>
<td>B</td>
<td></td>
<td>2/3</td>
<td></td>
</tr>
<tr>
<td>Indirect Tensile Creep</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Creep Slope</td>
<td>A</td>
<td></td>
<td>2/2</td>
<td></td>
</tr>
<tr>
<td>Time To Failure</td>
<td>A</td>
<td></td>
<td>2/2</td>
<td></td>
</tr>
<tr>
<td>FSCH</td>
<td>G* @0.01Hz</td>
<td>A</td>
<td>2/2</td>
<td></td>
</tr>
<tr>
<td>G* @10Hz</td>
<td>B</td>
<td></td>
<td>1/2</td>
<td></td>
</tr>
<tr>
<td>RSCH</td>
<td>Strain @5000</td>
<td>A</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>APA</td>
<td>Rut @8000</td>
<td>B</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>OVERALL RANKING</td>
<td>2.00</td>
<td></td>
<td>1.65</td>
<td></td>
</tr>
</tbody>
</table>

4.4 MOISTURE SUSCEPTIBILITY PROPERTIES

Moisture susceptibility properties of asphalt mixtures were characterized through the modified Lottman test and the permeability test as developed in this study. The modified Lottman test characterizes mixtures through the tensile strength ratio (TSR) while the permeability test yields the coefficient of (pseudo) permeability.
4.4.1 Moisture Susceptibility (Modified Lottman) Test

Table 4.19 presents the results of modified Lottman testing. The air voids for the dense-graded mixtures (A-P, L-P, L-MG and L-A) were between 6 and 8 percent as specified in the Louisiana specification (LADOTD, 1992), whereas the air voids of the open-graded mixtures were much higher and varied between 25 to 34 percent.

During the test, the conditioned mixture specimens for three mixtures (Mixes DT-P, OG-MG and OG-A) disintegrated when taken out the hot water bath after freezing. Two of the three OG-P specimens were intact after conditioning. All the specimens of the dense-graded mixtures (A-P, L-P, L-MG and L-A) finished conditioning and remained intact.

Table 4.19 Modified Lottman Test Results

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cond'd (kPa)</th>
<th>Control (kPa)</th>
<th>TSR</th>
<th>Mix</th>
<th>Cond'd (kPa)</th>
<th>Control (kPa)</th>
<th>TSR</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-P</td>
<td>269.1</td>
<td>593.4</td>
<td>0.45</td>
<td>Broken</td>
<td>132.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>L-P</td>
<td>289.8</td>
<td>414.0</td>
<td>0.70</td>
<td>DT-P</td>
<td>170.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>L-MG</td>
<td>476.1</td>
<td></td>
<td></td>
<td>Broken</td>
<td>149.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>L-A</td>
<td>314.6</td>
<td>453.3</td>
<td>0.58</td>
<td>Average</td>
<td>NA</td>
<td>150.9</td>
<td>NA</td>
</tr>
<tr>
<td>Average</td>
<td>314.6</td>
<td>453.3</td>
<td>0.58</td>
<td>Average</td>
<td>NA</td>
<td>227.7</td>
<td>NA</td>
</tr>
<tr>
<td>OG-P</td>
<td>Broken</td>
<td>200.1</td>
<td></td>
<td>Broken</td>
<td>282.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>OG-MG</td>
<td>Broken</td>
<td>289.8</td>
<td></td>
<td>OG-A</td>
<td>193.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>OG-A</td>
<td>Broken</td>
<td>193.2</td>
<td></td>
<td>Broken</td>
<td>255.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>NA</td>
<td>227.7</td>
<td></td>
<td>Average</td>
<td>NA</td>
<td>243.8</td>
<td>NA</td>
</tr>
<tr>
<td>L-P</td>
<td>1297.2</td>
<td>1649.1</td>
<td>0.79</td>
<td>A-P</td>
<td>1255.8</td>
<td>1524.9</td>
<td>0.82</td>
</tr>
<tr>
<td>L-MG</td>
<td>1311.0</td>
<td>1649.1</td>
<td>0.79</td>
<td>1255.8</td>
<td>1138.5</td>
<td>1352.4</td>
<td>0.84</td>
</tr>
<tr>
<td>L-A</td>
<td>1304.1</td>
<td>1373.1</td>
<td>0.95</td>
<td>1104.0</td>
<td>1138.6</td>
<td>1338.6</td>
<td>0.82</td>
</tr>
<tr>
<td>Average</td>
<td>1304.1</td>
<td>1559.4</td>
<td>0.84</td>
<td>Average</td>
<td>1200.6</td>
<td>1407.6</td>
<td>0.83</td>
</tr>
<tr>
<td>OG-P</td>
<td>648.6</td>
<td>1186.8</td>
<td>0.55</td>
<td>L-MG</td>
<td>945.3</td>
<td>1117.8</td>
<td>0.85</td>
</tr>
<tr>
<td>OG-MG</td>
<td>669.3</td>
<td>1207.5</td>
<td>0.55</td>
<td>L-A</td>
<td>862.5</td>
<td>1186.8</td>
<td>0.73</td>
</tr>
<tr>
<td>OG-A</td>
<td>786.6</td>
<td>959.1</td>
<td>0.82</td>
<td>Average</td>
<td>855.6</td>
<td>979.8</td>
<td>0.87</td>
</tr>
<tr>
<td>Average</td>
<td>703.8</td>
<td>1117.8</td>
<td>0.64</td>
<td>Average</td>
<td>903.9</td>
<td>1097.1</td>
<td>0.82</td>
</tr>
</tbody>
</table>
• **Comparisons Between LSAM and Conventional Mixtures**

It is evident that open-graded LSAM with SB polymer modified AC (Mix OG-P) exhibited much better performance in moisture susceptibility than the conventional Type 508 drainable base mixture (Mix DT-P). The latter (DT-P) completely disintegrated after freeze-thaw conditioning.

For the dense-graded asphalt mixtures, there was no significant difference (Table 4.20) in the tensile strength ratio (TSR) between the Mixes A-P and L-P.

<table>
<thead>
<tr>
<th>Engineering Property</th>
<th>LSAM (A-P)</th>
<th>Conventional (L-P)</th>
<th>LSAM (OG-P)</th>
<th>Conventional (DT-P)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TSR (%)</td>
<td>82.6</td>
<td>84.3</td>
<td>57.5</td>
<td>0</td>
</tr>
<tr>
<td>Ranking</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>B</td>
</tr>
</tbody>
</table>

Columns (for each mixture type) with similar letter indicate no significant difference.

• **Effect of AC Types to the LSAM Mixtures**

For the open-graded large stone asphalt mixtures, mixture containing SB polymer modified asphalt cement (Mix OG-P) behaved best in moisture susceptibility test, whereas, the other two (Mixes OG-MG and OG-A) disintegrated during the freeze-thaw conditioning.

For the dense-graded large stone asphalt mixtures, mixtures containing SB polymer modified asphalt (Mix L-P) and PG 64-22 (Mix L-A) behaved significantly better in moisture susceptibility than the mixture containing gelled asphalt (Mix L-MG).
Dense-graded LSAM with PG 64-22 (L-A) exhibited no significant difference in moisture susceptibility to the mix with PG 70-22M (L-P).

Table 4.21 TSR for LSAMs with Different Binder Types

<table>
<thead>
<tr>
<th>Mixture Type</th>
<th>Dense-graded LSAM</th>
<th>Open-graded LSAM</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PG 70-22M</td>
<td>PG 70-22M Alt.</td>
</tr>
<tr>
<td>PG 70-22M</td>
<td>L-P</td>
<td>L-MG</td>
</tr>
<tr>
<td>PG 64-22</td>
<td>L-A</td>
<td>OG-P</td>
</tr>
<tr>
<td></td>
<td></td>
<td>OG-MG</td>
</tr>
<tr>
<td>TSR (%)</td>
<td>84.3</td>
<td>64.0</td>
</tr>
<tr>
<td>Ranking</td>
<td>A</td>
<td>B</td>
</tr>
</tbody>
</table>

Columns (for each mixture type) with similar letter indicate no significant difference.

4.4.2 Permeability Test

Four mixes (DT-P, OG-P, A-P and L-P) were tested for their permeability. The results of permeability test are presented in Table 4.22. It is apparent that Mixes A-P and L-P were very impermeable, whereas, both OG-P and DT-P were very permeable. While the conventional Type 508 drainable base mixture (DT-P) exhibited relatively higher value in the pseudo-coefficient of permeability (K'), OG-P still had a K' value that was in the same order of magnitude. This indicates that the open-graded large stone asphalt mixture would still function as a good drainable mixture.

Table 4.22 Permeability Test Results

<table>
<thead>
<tr>
<th>Mixtures</th>
<th>K or K'</th>
<th>Ranking</th>
<th>m</th>
</tr>
</thead>
<tbody>
<tr>
<td>DT-P</td>
<td>3.04</td>
<td>A</td>
<td>0.4317</td>
</tr>
<tr>
<td>OG-P</td>
<td>1.27</td>
<td>A</td>
<td>0.3458</td>
</tr>
<tr>
<td>A-P</td>
<td>7.06x10^{-4}</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>L-P</td>
<td>17.6x10^{-4}</td>
<td>B</td>
<td></td>
</tr>
</tbody>
</table>
4.5 MIXTURE DURABILITY PROPERTY

Mixture durability property in this study was characterized through indirect tensile strength and strain test.

4.5.1 Indirect Tensile Strength and Strain Test

Tables 4.23 and 4.24 present the results of indirect tensile strength and strain test. Higher indirect strength (ITS) normally means more durable. In addition to the ITS values, the toughness index (TI) represents the mixture’s capability to absorb energy in the indirect tensile mode. The higher the toughness index, the more ductile the mixture is. Therefore, an ideal mixture should have both high ITS and TI values.

- Comparisons Between LSAM and Conventional Mixtures

Indirect tensile creep test results (Table 4.23) indicated that open-graded LSAM mixtures exhibited higher indirect strength than the conventional Type 508 open-graded drainable base mixture. It was noticeable that although the Conventional Type 508 drainable base mix (DT-P) had a higher value of strain at failure than the open-graded LSAM (OG-P), the toughness index (TI) of these two mixtures indicated that LSAM (OG-P) was more ductile than the Type 508 drainable base mix (DT-P).

For the dense-graded mixtures, however, there was no significant difference in the ITS and the toughness index (TI) between the 37.5-mm Superpave LSAM and the conventional dense-graded Type 5A base mixture.

- Effect of AC Types to the LSAM Mixtures

Among the open-graded LSAM mixtures, mixture with SB polymer modified asphalt (OG-P) had the highest ITS as shown in Table 4.24. There was no significant difference in ITS between the mixtures containing conventional PG 64-22 (OG-A) and
gelled asphalt (OG-MG). There was no significant difference in strain at failure and the toughness index among the open-graded LSAM mixtures.

Table 4.23 Indirect Tensile Strength (ITS) Test Results of Dense and Open-graded Mixtures

<table>
<thead>
<tr>
<th>Engineering Property</th>
<th>Mixture Type</th>
<th>Dense-graded</th>
<th>Open-graded</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LSAM (L-P)</td>
<td>Conventional (A-P)</td>
<td>LSAM (OG-P)</td>
</tr>
<tr>
<td>ITS (kPa)</td>
<td>1560</td>
<td>1408</td>
<td>453</td>
</tr>
<tr>
<td>Strain at Failure (%)</td>
<td>0.88</td>
<td>0.91</td>
<td>1.3</td>
</tr>
<tr>
<td>Toughness Index</td>
<td>0.81</td>
<td>0.86</td>
<td>0.91</td>
</tr>
</tbody>
</table>

Columns (for each mixture type) with similar letter indicate no significant difference.

For the dense-graded large stone asphalt mixtures, those containing SB polymer modified asphalt (Mix L-P) showed significantly higher indirect tensile strength than mixtures with conventional PG 64-22 (Mix L-A). Mix L-MG showed no significant difference in ITS from either L-P or L-A. Mix L-A showed significantly higher toughness index than Mix L-MG. Mix L-P showed no significant difference in toughness index from either L-MG or L-A. There was no significant difference in strain at failure among the 37.5-mm Superpave LSAM mixtures with different asphalt binders.

4.6 DRAIN-DOWN SUSCEPTIBILITY

Both Type 508 drainable mixture and open-graded LSAM had draindown values of less than 0.3 percent. Therefore, draindown should not be a problem for these mixtures during construction.
Table 4.24 ITS Test for LSAMs with Different Binder Types

<table>
<thead>
<tr>
<th>Engineering Property</th>
<th>Mixture Type</th>
<th>Superpave (Dense-graded)</th>
<th>Open-graded LSAM</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LSAM</td>
<td>PG70-22M L-P</td>
<td>PG70-22Mal L-MG</td>
</tr>
<tr>
<td>ITS (kPa)</td>
<td></td>
<td>1560 1118</td>
<td>1097 453</td>
</tr>
<tr>
<td>Ranking</td>
<td>A</td>
<td>AB B B A B</td>
<td>A B B</td>
</tr>
<tr>
<td>Strain at Failure (%)</td>
<td>0.88 0.92 0.83</td>
<td>1.3 1.5 1.6</td>
<td></td>
</tr>
<tr>
<td>Ranking</td>
<td>A</td>
<td>A A A A A</td>
<td>A A A</td>
</tr>
<tr>
<td>Toughness Index</td>
<td>0.81 0.68 0.89</td>
<td>0.91 0.88 0.85</td>
<td></td>
</tr>
<tr>
<td>Ranking</td>
<td>AB B A A A</td>
<td>A A A</td>
<td></td>
</tr>
</tbody>
</table>

Columns (for each mixture type) with similar letter indicate no significant difference.

4.7 Summary of Mixture Characterization

Eight mixtures were characterized through a series of asphalt mixture performance tests. These eight mixtures were divided into two comparing groups: one with the open-graded LSAM and the conventional Type 508 drainable base mixture, the other one with 37.5-mm Superpave LSAM and the conventional Type 5A base mixture. Three different types of asphalt binders were employed to study the effects of asphalt binders on the performance of LSAM mixtures. These three asphalt binders include: an SB polymer modified asphalt binder meeting Louisiana Superpave performance grading specification of PG 70-22M, a conventional asphalt cement meeting Louisiana specification of PG 64-22, and a gelled asphalt cement meeting Louisiana Superpave performance grading specification of PG 70-22Mal. The results of laboratory mixture characterization can be summarized as followings.

- The most significant difference in volumetric properties between the LSAM and conventional mixtures is the degree of stone-on-stone contact. The LSAM mixtures have a higher degree of stone-on-stone contact than the conventional mixtures.
mixtures in this study had the degree of stone-on-stone contact between 85 to 92 percent, while their conventional mix counter parts had only 50 and 52 percent;

- The elastic property test of the indirect tensile resilient modulus test did not show significant difference between the open-graded LSAM and conventional open-graded drainable mixture;

- The $M_r$ result of the conventional Type 5A base mix was higher in 40 °C than the Superpave LSAM, while there was no significant difference in $M_r$ values between the two at 4 °C and 25 °C;

- The overall mixture properties of permanent deformation indicated that the open-graded LSAM had significantly higher rut-resistance when compared with the conventional Type 508 drainable base mixture;

- The Superpave LSAM and the conventional Type 5A base mixture showed very similar overall rut-resistance based on the performance tests conducted in this study;

- For both open-graded and dense-graded LSAM mixtures, the ones with SB polymer modified asphalt binder (PG 70-22M) showed the best rut-resistance, while the ones with PG 64-22 exhibited slightly better rut-resistance than the ones with gelled asphalt, PG 70-22MAI;

- Moisture susceptibility tests conducted in this study indicated that the open-graded LSAM was much less susceptible to moisture damage when compared to the conventional Type 508 drainable base mix;

- There was no significant difference in moisture susceptibility between the Superpave LSAM and the conventional Type 5A base mix;
• For both open-graded and dense-graded LSAM, the mixes with SB polymer modified asphalt cement showed lower susceptibility to moisture damage than the mixes with gelled asphalt and PG 64-22;

• A permeability study was conducted to study the fundamental properties of hydraulic conductivity in the asphalt mixtures and a dual-mode permeameter was developed in the study;

• The permeability test results indicated that both open-graded and dense-graded LSAM had (pseudo) coefficients of permeability in the same order of magnitude as their conventional counterparts;

• Indirect tensile strength (ITS) of the open-graded LSAM was significantly higher than that of Type 508 drainable base mixture;

• There was no significant difference in the ITS between the Superpave LSAM and the conventional Type 5A base mix;

• For both open-graded and dense-graded LSAM, the ones with SB polymer modified asphalt cement, PG 70-22M, had higher ITS values than the ones with gelled asphalt, PG 70-22MAlt and the PG 64-22;

• The overall performance of the open-graded LSAM was better than the conventional Type 508 drainable base mixture;

• The overall performance of the Superpave LSAM was similar to the conventional Type 5A base mixture;

• The overall performance of open-graded and dense-graded LSAM with the SB polymer modified asphalt cement, PG 70-22M was better than the ones with gelled asphalt, PG 70-22MAlt, and PG 64-22.
4.8 APPLICATIONS OF MIX CHARACTERISTICS TO PAVEMENT PERFORMANCE PREDICTION

The overall purpose of asphalt mix design and laboratory mix characterization is to produce HMA mixtures that make pavement perform better. Therefore, it is necessary to correlate pavement field performance to the fundamental engineering properties. One way to correlate mix properties to field performance is to establish empirical relationship based on a standard pavement structure. The other approach is to apply the engineering properties of mixes into certain material models and predict the pavement performance structural analysis.

Each fundamental mix property test has an underlying constitutive model that can be used to predict mix performance under the test condition. For example, the indirect tensile resilient modulus reflects the elastic modulus and poisons ratio more the mixture under the test temperature and loading level. Indirect tensile strength (ITS) provides the damage parameters for plasticity material models. Frequency sweep at constant height (FSCH) test reflect the rate-dependency characteristics of HMA mixtures and can be modeled through viscoelastic or viscoplastic material models. Axial and indirect creep characteristics can be modeled through viscoelastic or viscoplastic creep models.

The latter part of this research involved the development of 3-D dynamic finite element analysis, in which viscoplasticity material models were applied for the asphalt mixtures. After a model validation through the analysis of a test lane from the accelerated loading facility (ALF), the 3-D finite element procedure was used to predict the performance of two groups of pavements, one with large stone asphalt mixtures and the other one with conventional mixes. During the finite element analysis of pavement
performance prediction, the following results from the material characterization were used:

- Indirect tensile resilient modulus for the elastic properties;
- Indirect tensile strength (ITS) for the damage criteria;
- Axial creep for creep model (rutting prediction).
CHAPTER 5.
DEVELOPMENT OF 3-D DYNAMIC FINITE ELEMENT PROCEDURE

A true mechanistic pavement design procedure should be able to correctly predict pavement response and the development of pavement distress (such as rutting and fatigue cracking) under various traffic and environmental conditions. This will require the incorporation of realistic constitutive models for the paving materials and reasonable geometrical models for pavement structures into the pavement design system. A 3-D numerical simulation procedure with realistic material models would be ideal to achieve such a goal.

Unfortunately, the technology of 3-D numerical simulation has not yet been adapted to the normal pavement design. Full-size pavement load testing facilities are still widely used by various agencies as a benchmark to correlate pavement performance of different pavement designs. Although full-size load test provides excellent relationship between pavement performance and designs, they are costly and time-consuming. It would save substantial amount of time and money should 3-D numerical simulation procedures be calibrated from the existing full-size pavement load tests and then applied to the future pavement designs. This chapter presents a research effort to achieve such a goal through a 3-D finite element simulation of test sections being tested with the Louisiana Accelerated Loading Facility (ALF).

5.1 PREVIOUS STUDIES

Only a very limited number of engineering problems can be solved by closed form stress or deformation analyses, therefore, numerical procedures (such as the finite element method) have been used extensively to solve complicated engineering problems.
The early computer programs such as the ELSYM5 and BISAR use the linear elastic constitutive equations to calculate the stresses and strains in the pavement structure. Later more sophisticated finite element programs were developed for pavement analyses. Among these programs, VESYS, a linear viscoelastic program (Meyer, 1977) and the FLEXPASS that uses non-linear elastoplastic constitutive models have been widely used (Monismith, et al, 1994). These programs are able to calculate the stress and strain distributions in the pavement structure based on the constitutive models used, but they all assume the traffic load as a static load, and the pavement geometry as an axisymmetrical system to the center of a circular area of evenly distributed load. These gross simplifying assumptions inevitably introduce a lot of errors to their solutions and thus limit the further applications of these programs.

Zaghloul (1993) applied three dimensional dynamic finite element procedures, through ABAQUS, the commercial finite element software, to analyze the flexible and rigid pavements under the traffic loads. He applied linear viscoelastic models for the asphalt concrete layers, Drucker-Prager model for the aggregate base layer, and the Cam Clay model for the subgrade soils. Zaghloul uses 3-D brick elements for the pavement structure, thus makes it possible to study the various boundary effects, such as shoulders. White (1998) and his co-workers (Huang, 1995, Pan, 1997, Hua, 2000) apply the 3-D FEM procedure they have developed in a number of projects to study the response of both asphalt and concrete pavements.

Uddin (1998) applies 3-D finite element dynamic analysis for the pavements under the impact load of the falling weight deflectometer (FWD) and back-calculates the elastic modulus of the pavement layers.
Seibi (1993) develops an elastic visco-plastic constitutive relation for the asphalt concrete under high rates of loading. The model adds the rate dependent characteristics to the traditional Drucker-Prager plastic model. He conducts some parametric studies for the pavement samples from the Federal Highway Administration’s (FHWA) existing ALF (Accelerated Loading Facilities) sections. By incorporating the model into ABAQUS, he compares the analysis against the FHWA ALF test results.

5.2 OBJECTIVES AND SCOPE

The objective of this part of the study was to develop a 3-D finite element procedure using fundamental engineering properties acquired from mix characterization to simulate the dynamic traffic load. One pavement test lane from the Louisiana Accelerated Loading Facilities (ALF) was used to calibrate the numerical simulation procedure. In order to obtain more realistic pavement responses under the traffic loads, rate dependent viscoplastic model was applied for the asphaltic concrete. Extended Drucker-Prager elastoplastic model was used to describe the aggregate base and subgrade. Four finite element (FE) analyses were performed for numerical comparisons. Table 5.1 presents the nature of these four FE analyses. The commercial finite element software, ABAQUS was selected for the numerical simulation.

Table 5.1 Scope of FE Analysis

<table>
<thead>
<tr>
<th>No.</th>
<th>FE Analyses</th>
<th>Material Models</th>
<th>Load Models</th>
<th>Geometric Models</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2-D Static Analysis</td>
<td>Linear Elastic</td>
<td>Static</td>
<td>2-D</td>
</tr>
<tr>
<td>2</td>
<td>3-D Dynamic Analysis</td>
<td>Linear Elastic</td>
<td>Dynamic</td>
<td>3-D</td>
</tr>
<tr>
<td>3</td>
<td>3-D Dynamic Analysis</td>
<td>Viscoplastic, Elastoplastic</td>
<td>Dynamic</td>
<td>3-D</td>
</tr>
<tr>
<td>4</td>
<td>3-D Rutting Analysis</td>
<td>Viscoplastic (Creep), Elastoplastic</td>
<td>Dynamic</td>
<td>3-D</td>
</tr>
</tbody>
</table>
5.3 GEOMETRIC MODELS FOR THE FINITE ELEMENT ANALYSES

The ALF test lane being studied had the geometrical structure as shown in Figure 5.1. The test section was 12 m long by 1.2 m wide. The pavement had 38.1 mm Louisiana Type 8F HMA wearing course, 50.8 mm Type 8 HMA binder course, 88.9 mm Type 5A HMA base course with crumb rubber modifier, 215.9 mm crushed limestone sub-base course sitting on the top of compacted silty clay embankment which had the top 254 mm stabilized by 8% cement.

In finite element analysis, the elements used for 2-D analyses include triangular and quadrilateral elements as shown in Figure 5.2. The 3-node triangular and 4-node quadrilateral elements offer linear interpolation along the element surfaces (edges), and provide solutions that are difficult to smooth between elements. The 6-node triangular and 8-node quadrilateral elements use quadratic interpolation and are called second order elements. The second order elements provide smoother solutions than the linear elements.

Similar to the 2-D analyses, in 3-D finite element analyses, the continuum elements include the tetrahedral, wedge (triangular prism) and hexahedron (brick) elements. There are also have the linear and quadratic forms for all these elements as shown in Figure 5.3.
Figure 5.1 Layouts of the Pavement Layers and Instrumentation of the Test Lane
Figure 5.2. 2-D Continuum Elements (HKS, 1998)
Figure 5.3. 3-D Continuum Elements (HKS, 1998)
According to the literature (HKS, 1998), both triangular and tetrahedral elements are “notoriously poor elements (extremely fine meshes are needed to obtain results of reasonable accuracy)”, therefore, these elements should be avoided whenever possible. Both 8-node quadrilateral and 20-node hexahedron (brick) elements offer satisfactory accuracies for the 2-D and 3-D stress and strain analyses.

Finite element analysis uses numerical techniques to integrate various quantities over the volume of each element. Using Gaussian quadrature for most elements, material response at each integration point in each element is evaluated. Figure 5.4 illustrates the 2-D continuum elements with integration points. When using continuum elements, one may choose between full or reduced integration. Full integration offers solutions at more (integration) points, but it tends to make the element “too stiff” by introducing too many constraints within the elements. Therefore, reduced integration elements are normally recommended when the number of reduced integration points is more than one. In this study, the 8-node quadrilateral element with reduced integration points (CAX8R in ABAQUS) was used for 2-D finite element analysis, and the 20-node hexahedron (brick) element with reduced integration points (C3D20R in ABAQUS) was used for 3-D finite element analysis.

Figure 5.5 presents the geometric mesh for the 3-D finite element analyses. Due to symmetry, only half of the pavement structure was included. Twenty-node hexahedron (brick) elements with reduced integration points were used to form the finite element mesh. A brief sensitivity analysis suggested that a mesh of 6760 (52x13x10) would provide reasonable continuity for the stress and strain details of pavement response under the ALF loads.
Figure 5.6 presents the finite element mesh for the 2-D numerical analysis. Eight-node axisymmetric isoparametric reduced integration elements were used to form the finite element mesh. A total of 825 (33x25) elements were included in the mesh.

Figure 5.4. 2-D Continuum Elements with Integration Points (HKS, 1998)
Figure 5.5. 3-D Finite Element Mesh of ALF Test Lane
Figure 5.6. 2-D Finite Element Mesh of ALF Test Lane
5.4 MATERIAL MODELS FOR THE FINITE ELEMENT ANALYSES

5.4.1 Rate-Dependent Viscoplastic Model

Asphalt concrete exhibits elastic behavior at low stress levels and a strain rate dependent plastic behavior. Seibi (1993) suggested the linear strain hardening relationships for the inelastic behavior of asphalt concretes. In the theory of viscoplasticity, the total (deviatoric) strain rate can be divided into elastic and inelastic components as shown in Equation (5.1):

\[ \dot{\varepsilon}_y = \dot{\varepsilon}_y^e + \dot{\varepsilon}_y^\text{vp} \]  \hspace{1cm} (5.1)

where the dot represents the derivative of strain with respect to time and the superscripts "e" and "vp" represent elastic and viscoplastic, respectively. The elastic strain rate is given by the time derivative of the isotropic elastic theory as shown in Equation (5.2):

\[ \dot{\varepsilon}_y^e = \frac{1}{2G} \dot{S}_y, \quad \dot{\sigma}_{kk} = \frac{1}{3K} \ddot{\sigma}_{kk} \]  \hspace{1cm} (5.2)

where \( G \) and \( K \) are the shear and bulk modulus, respectively, and \( e \) and \( S_y \) are the deviatoric strain and stress.

The inelastic strain rate in Equation (5.1) can be expressed as the result of combined viscous and plastic effects as shown in Figure 5.7. The plastic element is active only when the applied stress exceeds the yield stress of the material. Based on Perzyna (1966) postulate, the total strain rate for one-dimensional stress state can be expressed as:

\[ \dot{\varepsilon} = \frac{\dot{\sigma}}{E} + \gamma' \Phi \left[ \frac{\sigma}{\phi(e^p)} - 1 \right] > \]  \hspace{1cm} (5.3)
\[ \sigma = \phi(e^p) \cdot \left(1 + \Phi^{-1}\left(\frac{\dot{e}^{vp}}{\gamma^*}\right)\right) \]  \hspace{1cm} (5.4)

where \(\sigma = \phi(e^p)\) represents the static stress-strain relation. The function \(\Phi\) can be obtained from the experimental data to represent the results of the dynamic loading tests of the material.

\begin{figure}[h]
\centering
\includegraphics[width=0.5\textwidth]{elastic-viscoplastic-model.pdf}
\caption{Elastic Viscoplastic Model}
\end{figure}

In the case of multi-axial state of stress, Equation (5.3) can be generalized by introducing the concept of yield function. This can be achieved by extending Malvern’s (1951) relation to more general constitutive relations for isotropic work hardening and strain rate sensitive materials as shown in the following equation:
\[ \dot{\epsilon}_y = \frac{1}{2G} \dot{S}_{yy} + 1 - 2\nu \frac{\dot{S}_{yy}}{E} + \gamma \Phi(F) \frac{\partial F}{\partial \sigma_{yy}} \]  

(5.5)

where \( \dot{\epsilon}_y \) denotes the rate of strain, \( \sigma_{yy} \) is the normal stress, \( \nu \) is Poisson's ratio, \( G \) is the shear modulus, \( k \) is the strain hardening coefficient, and \( \Phi(F) \) is a function defined as:

\[
\Phi(F) = \begin{cases} 
\Phi(F) & \text{for } F > 0 \\
0 & \text{for } F \leq 0 
\end{cases}
\]  

(5.6)

The argument \( F \) denotes plastic yield condition. The initial yield condition is the same as the static yield criterion and can be expressed as:

\[ F(\sigma_{yy}, e^p_{yy}) = \frac{f(\sigma_{yy}, e^p_{yy})}{\kappa} - 1 \]  

(5.7)

where \( \kappa \) is the strain hardening coefficient and is related to the plastic work.

\[ \kappa = \kappa(W_p) = \kappa \left( \int_0^1 \sigma_{yy} de^p_{yy} \right) \]  

(5.8)

The last term on the right hand side of Equation (5.5) represents the instantaneous plastic strain rate as a function of the applied stress taking the following form:

\[ \dot{\epsilon}^p_{yy} = \gamma \Phi(F) \frac{\partial F}{\partial \sigma_{yy}} \]  

(5.9)

Take a square on both sides of Equation (5.9) and replace second strain invariant to the left hand side, one obtains:

\[ (I_2^p)^{1/2} = \gamma \Phi(F) \left( \frac{1}{2} \frac{\partial f}{\partial \sigma_{yy}} \frac{\partial f}{\partial \sigma_{uu}} \right)^{1/2} \]  

(5.10)
Combining Equation (5.7) and (5.10), we get the dynamic yield condition for an elastic viscoplastic isotropic work hardening and strain rate dependent materials as shown in the follow:

\[ f(\sigma_y, e_y) = \kappa(W_p) \left\{ 1 + \Phi^{-1} \left[ \frac{(f_x)^{1/2}}{\gamma} \left( \frac{1}{2} \frac{\partial f}{\partial \sigma_{mn}} \frac{\partial f}{\partial \sigma_{mn}} \right)^{-1/2} \right] \right\} \]  \tag{5.11}

Equation (5.11) determines the change of the current yield surface during a dynamic loading/unloading process that involves inelastic straining. Two factors, isotropic hardening and rate dependency, contribute to the change of yield surface. In the commercial finite element software, ABAQUS, this relation is simplified as follows (HKS, 1998):

\[ f(\sigma) = \overline{\sigma}(\overline{\varepsilon}^{pl}, \dot{\varepsilon}^{pl}, \theta, f^\alpha) \]  \tag{5.12}

\[ \overline{\sigma} = \left[ 1 + \left( \frac{\dot{\varepsilon}^{pl}}{D} \right)^n \right] \sigma^0 \]  \tag{5.13}

where \( \overline{\sigma} \) is the equivalent yield stress; \( \overline{\varepsilon}^{pl} \) is the equivalent plastic strain; \( \dot{\varepsilon}^{pl} \) is the equivalent plastic strain rate; \( \theta \) is the temperature; \( f^\alpha, \alpha = 1, 2, \ldots \) are other predefined field variables; \( \sigma^0 \) is the static equivalent yield stress; and \( D \) and \( n \) are material parameters that determine the overstress ratio \( R \).

\[ R = 1 + \left( \frac{\dot{\varepsilon}^{pl}}{D} \right)^n \]  \tag{5.14}

Evidently the static equivalent yield stress \( \sigma^0 \) includes plastic strain hardening and the overstress ratio represents the rate dependency.

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Equation (5.13) is recommended for high-speed dynamic process, such as traffic movement. For low speed process, a creep model in ABAQUS can be used to characterize the permanent deformation properties of the asphalt mixtures.

\[
\dot{\varepsilon}^{cr} = A \left( \bar{\sigma}^{cr} \right)^m \cdot t^n
\]

(5.15)

where \( \dot{\varepsilon}^{cr} \) is equivalent creep rate; \( \bar{\sigma}^{cr} \) is the equivalent Mises stress; \( t \) is the total creep time; and \( A, m, \) and \( n \) are material parameters.

5.4.2 Elastoplastic Model (Drucker-Prager Model)

The linear Drucker Prager model was used to define the yield criteria for the paving materials in this study. The Drucker Prager model can be expressed as:

\[
F = t - p \cdot \tan \beta - d(w_p) = 0
\]

(5.16)

\[
t = g \cdot \frac{q}{2} \left[ 1 + \frac{1}{K} \left( 1 - \frac{1}{K} \right) \left( \frac{r}{q} \right)^3 \right]
\]

(5.17)

Where \( p \) is the equivalent pressure stress; \( d(w_p) \) is the material parameter that includes plastic work hardening; \( q \) is the Mises equivalent stress; \( r \) is the third invariant of deviatoric stress; and \( K \) is the ratio of the yield stress in triaxial tension to the yield stress in triaxial compression. Figure 5.8 presents the linear Drucker Prager yield surface in meridian and deviatoric planes.

5.4.3 Material Parameters

Table 5.2 presents the material parameters used in finite element simulation. The elastic parameters were obtained from FWD back-calculation. Rate-dependent viscoplastic constitutive models were used for the asphalt concrete layers. The rate-dependent parameters for the asphaltic concrete were obtained from uniaxial compressive test at different strain rate (Seibi, 1993). Linear Drucker-Prager model was used for the crushed
Linear Drucker-Prager: \[ F = t - p \tan \beta - d = 0 \]

\[ t = \frac{1}{2} q \left[ 1 + \frac{1}{K} \left( 1 - \frac{1}{K} \right) \left( \frac{E'}{q} \right) \right] \]

<table>
<thead>
<tr>
<th>Curve</th>
<th>K</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>1.0</td>
</tr>
<tr>
<td>b</td>
<td>0.8</td>
</tr>
</tbody>
</table>

Typical yield surfaces for the linear model in the deviatoric plane.

Figure 5.8. Linear Drucker Prager Model (HKS, 1998)
stone base, compacted embankment soil and subgrade soil. The predicted responses of asphaltic concrete and crushed limestone under a triaxial test with a confining pressure equal to the in-situ stress of the corresponding layer are provided in Figures 5.9 and 5.10.

It is noticeable that for the asphaltic concrete, rate dependent viscoplastic model exhibited a series of dynamic yield surfaces under different strain rates.

Table 5.2. Material Parameters Used for Finite Element Analyses

<table>
<thead>
<tr>
<th>Layer #</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
<td>AC Wearing Course</td>
<td>AC Binder Course</td>
<td>CRM AC Base Course</td>
<td>Crushed Limestone</td>
<td>Compacted Soil</td>
<td>Subgrade Soil</td>
</tr>
<tr>
<td>Thickness, mm (inch)</td>
<td>38.1 (1.5)</td>
<td>50.8 (2.0)</td>
<td>88.9 (3.5)</td>
<td>215.9 (8.5)</td>
<td>254.0 (10.0)</td>
<td>&gt;254.0 (&gt;10.0)</td>
</tr>
<tr>
<td>Material Model</td>
<td>Visco Plastic</td>
<td>Visco Plastic</td>
<td>Visco Plastic</td>
<td>Drucker Prager</td>
<td>Drucker Prager</td>
<td>Drucker Prager</td>
</tr>
<tr>
<td>Elastic Modulus, E kPa (ksi)</td>
<td>5.43x10^6 (787)</td>
<td>4.41x10^6 (640)</td>
<td>5.93x10^6 (860)</td>
<td>5.0x10^5 (72.5)</td>
<td>2.6x10^5 (37.7)</td>
<td>1.5x10^5 (21.7)</td>
</tr>
<tr>
<td>Poisson's Ratio v</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.30</td>
<td>0.45</td>
</tr>
<tr>
<td>Drucker Prager K,kPa (psi)</td>
<td>470 (68)</td>
<td>470 (68)</td>
<td>400 (58)</td>
<td>15 (2.17)</td>
<td>80 (11.6)</td>
<td>50 (7.2)</td>
</tr>
<tr>
<td>β (°)</td>
<td>40</td>
<td>40</td>
<td>40</td>
<td>50</td>
<td>30</td>
<td>20</td>
</tr>
<tr>
<td>Visco-plastic A</td>
<td>1.8x10^-2</td>
<td>1.8x10^-3</td>
<td>1.8x10^-3</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>m</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>-</td>
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</tr>
<tr>
<td>n</td>
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<td>-0.5</td>
<td>-0.5</td>
<td>-</td>
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<td>-</td>
</tr>
</tbody>
</table>

- Not applicable

5.5 LOAD MODELS

The test lane for numerical analysis in this study was 60 m long and 3.6 m wide. The ALF loading area was 12 m long. The ALF load was applied through a trolley that
Figure 5.9. Predicted Stress-Strain Behavior of Crushed Limestone
Figure 5.10. Predicted Stress-Strain Behavior of Asphalt Concrete Wearing Course
traveled in one direction at a speed of 16 km/hour (10 mph). Figure 5.11 presents a picture of the ALF equipment. The applied load was 44.5 kN for the first 400,000 cycles. The load was then increased to 54.7 kN after the 400,000th cycles, 65.0 kN after the 500,000th cycles, and 75 kN after the 650,000th cycles. Figure 5.12 presents the ALF loading history for the test lane.

Figure 5.11. Louisiana Accelerated Loading Facility (ALF)
Figure 5.12. ALF Loading History
In order to simulate the moving load in 3-D dynamic finite element analysis, a trapezoid shaped load amplitude function was applied to each element. As presented in Figure 5.13, the segment AB represented the approaching of the wheel, the segment BC represented the full wheel load, and the segment CD represented the departure of the wheel. In the 3-D finite element analysis in this paper, the segment AB and CD occupied \( \frac{1}{4} \) of the total duration of the wheel loading time. The element length along the traffic direction was divided in such a way that it equaled the \( \frac{1}{4} \) of the footprint of the wheel.

![Figure 5.13. Load Amplitude Function](image)

In the 3-D rutting analysis, instead of applying dynamic moving loads on the pavement surface at the traffic speed over and over for hundreds of thousands times.
(which is unrealistic under the current computer technology and the material models), a step load was applied on the pavement surface over the equivalent amount of time to the number of passes of ALF loads. This concept was originally proposed by Huang (1995) and recently further developed by Hua (2000). Hua (2000) recognizes the effect of wheel wandering and applies the static equivalent load based on the statistical distributions of wheel over the extent of wandering distance.

As presented in Figure 5.14, the frequency of the ALF wheels wandering along the transverse direction was similar to a normal distribution. Based on Hua's method, the total loading time over point A due to one single wheel can be calculated as follows:

\[ t(x) = \left[ F(x + \frac{W}{2}) - F(x - \frac{W}{2}) \right] \cdot T \]

where \( x \) is the offset distance from the centerline of the wheel, \( W \) is the tire width, \( F(x) \) is the normal cumulative distribution of the specified mean, and \( T \) is the total cumulative loading time of the tire load applied on the pavement during the entire loading level.

Superposition was applied to the area where the two wheels overlapped during their wandering.

5.6 NUMERICAL SIMULATIONS OF ALF

Numerical simulations were carried out on one of the test lanes of the Louisiana Accelerated Loading Facilities. The results from three sets of numerical modeling, 1) 2-D static analysis, 2) 3-D dynamic analyses with linear elastic models for all the pavement layers, and 3) 3-D dynamic analysis with different constitutive models, were presented for discussion. In addition, 3-D finite element rutting analysis was performed based on viscoplastic creep model.
Figure 5.14. Loading Model for 3-D Rutting Analysis
5.6.1 Pavement Surface Deflections

Figure 5.15 presents the results of pavement surface deflections along with transverse and longitudinal directions. The 2-D static analysis exhibited significantly higher deflection than either the 3-D dynamic analyses with linear elastic or viscoplastic material models. The 3-D dynamic analyses were able to capture the differences in surface deflection between the transverse and longitudinal directions. There was no significant difference of surface deflection in either transverse or longitudinal directions between the 3-D dynamic analyses of linear elastic or viscoplastic models.

5.6.2 Stresses and Strains

Figures 5.16 through 5.18 present the results of stresses at bottom of the asphalt base layer from the finite element analyses. The 3-D dynamic analyses were able to reflect the dynamic natures of the responses of stresses and strains in the pavement layers, whereas, the 2-D static analysis was only able to obtain the stress and strain distributions under the static load. For example, it has been known that the shear stress along the traffic direction changes its direction due to the moving of wheel loads. Both 3-D analyses showed this phenomenon, whereas, in the 2-D static analysis, only positive shear stress was reported. The value of vertical stress (S-ZZ) in the 2-D static analysis was significantly higher than the 3-D dynamic analyses.

It was noticeable that the longitudinal stress (S-YY) of the 3-D viscoplastic analysis was significantly different than that of 3-D elastic analysis. In the viscoplastic analysis, S-YY reversed its direction from tension to compress as the wheel passed, whereas, in the 3-D linear elastic analysis, S-YY only reduced to zero.
It was also noticeable that the amplitudes of longitudinal and transverse tensile stress (S-YY and S-XX) were significantly smaller than those of 3-D linear elastic analysis.

![Surface Deflection Along Transverse Direction](image1)

![Deflection Along Longitudinal Direction](image2)

Figure 5.15. Surface Deflections Along Transverse and Longitudinal Directions
Stresses at Bottom of AC Layer, 3-D, Linear Elastic

Designation of Directions:

- X – Transverse Direction;
- Y – Traffic Direction;
- Z – Vertical (Depth) Direction;
- R – Transverse and Traffic (in 2-D) Directions.

Figure 5.16. Stresses at Bottom of the Asphaltic Concrete, 3-D Linear Elastic Analysis
Figure 5.17. Stresses at Bottom of the Asphaltic Concrete, 3-D Viscoelastic Analysis

Stresses at Bottom of AC Layer, 3-D, Viscoelastic

Designation of Directions:
- X - Transverse Direction;
- Y - Traffic Direction;
- Z - Vertical (Depth) Direction;
- R - Transverse and Traffic (in 2-D) Directions.
Figure 5.18. Stresses at Bottom of the Asphaltic Concrete, 2-D Static Analysis
Figure 5.19 presents the longitudinal strains at the bottom of the surface asphaltic concrete (D=88.9 mm) and the asphalt base course (D=177.8 mm). Figure 5.20 presents a typical measured response curve of the longitudinal strain at the bottom of the asphalt base course. Again, the 2-D static analysis failed to reflect the dynamic nature of the strains. At the bottom of the surface asphaltic concrete, the 2-D static analysis showed a static strain of 67.2x10^{-6}. The field measured values of the longitudinal strain had peak values around 15x10^{-6} at the bottom of the surface asphaltic concrete and 45x10^{-6} at the bottom of the asphalt base course. Both 3-D dynamic analyses obtained values of strains that were to the field measurement.

Noticeably, the 3-D viscoplastic analysis showed viscosity characteristics of asphaltic concrete – the strain lagged behind the instantaneous elastic response. The 3-D viscoplastic analysis also exhibited permanent strains when compared to the linear elastic analysis.

5.6.3 Permanent Deformation (Rutting)

Permanent deformation (rutting) of the test lane was calculated through the rutting model (Equation 5.15) based on the load functions (Figure 5.13) which incorporated the wheel wander (Figure 5.14). Figure 5.21 presents the results of rutting prediction based on the creep model in the 3-D finite element analysis. The model predicted the rut depth development with the reasonable accuracy.

5.7 SUMMARY OF DEVELOPMENT OF 3-D FEM PROCEDURE

A three dimensional dynamic finite element procedure was developed to simulate the traffic load from the Louisiana accelerated loading facility (ALF). Viscoplastic models were successfully used in the commercial finite element software, ABAQUS, to simulate
Figure 5.19. Longitudinal Strain at Bottom of Surface AC and Asphalt Base

Figure 5.20. A Typical Measured Longitudinal Strain Response Curve
Figure 5.21. Rutting Transverse Profile and Rut Depth Vs. Load Cycles
the dynamic responses and predict permanent deformation (rutting) of a flexible pavement. A comparative study of numerical analyses of an ALF test lane was conducted through four finite element analyses: the 2-D static analysis with linear elastic material models, the 3-D dynamic analysis with linear elastic material models, the 3-D dynamic analysis with viscoplastic model for asphaltic concrete and elastoplastic model for other paving materials, and 3-D rutting analysis with viscoplastic creep model for the asphaltic concrete. The following observations can be made.

- The traditional two dimensional static finite element analysis was unable to simulate the dynamic nature of the traffic load and the correspondent pavement responses;
- Three dimensional dynamic finite element analysis can be achieved through the application of load functions using a commercial finite element software, ABAQUS, to simulate the traffic loads;
- 3-D dynamic finite element analyses were able to predict dynamic stress and strain responses of the asphaltic pavement that were close to field measurements;
- Rate-dependent viscoplastic models incorporated into the 3-D dynamic finite element procedure were able to predict the viscous and permanent strain characteristics of the asphaltic concrete material under the traffic loads;
- Permanent deformation (rutting) could be predicted through the application of a creep model and a load function that incorporated distributions of the actual wheel wander into the 3-D dynamic finite element procedure.
CHAPTER 6.
FINITE ELEMENT COMPARISONS OF PAVEMENTS CONTAINING LSAM AND CONVENTIONAL ASPHALT MIXTURES

This chapter presents the results of structural comparisons of two groups of pavements. Each group consists of two pavements: one with conventional mixtures and one with large stone asphalt mixtures designed in this study. A total of four pavement sections were analyzed. The 3-D dynamic finite element procedure described in Chapter 5 was used for the pavement analyses. Material parameters were obtained primarily from the mixture characterization tests described in Chapters 3 and 4. The results of the comparison showed that a pavement containing an open-graded LSAM exhibited stronger structural support than a pavement containing a conventional Louisiana Type 508 drainable base mixture, whereas, a pavement containing a 37.5-mm Superpave (dense-graded) LSAM exhibited similar structural support capability as a pavement containing a conventional Louisiana Type 5A base course mixture.

6.1 PAVEMENT STRUCTURES FOR COMPARISON
In order to quantify the improved structural capacity of the LSAM in a pavement structure, it is necessary to perform some structural analyses and compare the predicted performance of pavements that contain either conventional mixtures or the large stone asphalt mixtures developed in this study. Four typical pavements were designed for finite element analyses for this purpose. Figures 6.1 and 6.2 illustrate the cross-sections of these four pavements. Pavement 1 consisted of 50 mm conventional wearing course, 50 mm conventional binder course, 100 mm conventional asphalt base course, 100 mm conventional Type 508 drainable base mixture, and 250 mm cement soil base sitting on the compacted embankment soil. Pavement 2 consisted of 50 mm conventional wearing
course, 50 mm conventional binder course, 100 mm conventional asphalt base course, 100 mm open-graded large stone asphalt mixture, and 250 mm cement soil base sitting on the compacted embankment soil. Pavement 3 had 50 mm conventional wearing course, 50 mm conventional binder course, 100 mm conventional Type 5A base mixture, 100 mm crushed stone, and 250 mm cement soil base sitting on the compacted embankment soil. Pavement 4 contained 50 mm conventional wearing course, 50 mm conventional binder course, 100 mm dense-graded 37.5-mm Superpave LSAM, 100 mm conventional Type 508 drainable base mixture, 100 mm crushed stone, and 250 mm cement soil base sitting on the compacted embankment soil. Performance comparisons involved comparing pavement 1 with the performance Pavement 2, and Pavement 3 with Pavement 4.

Pavement sections with the same geometric dimension as the ALF test lanes were used for the numerical simulations. The detailed description of the geometry is included in Chapter 5, “Development of 3-D Dynamic Finite Element Procedure.” The load and traffic speed were also based on the ALF test loading with an applied load of 44.5 kN at a traffic speed of 16 km/hr (10 mph.)

6.2 FINITE ELEMENT GEOMETRIC MESH

Since all of the four pavements had the same geometric dimensions, only one set of geometric mesh was established as shown in Figure 6.3. By changing the material parameters, the numerical responses from different pavements were obtained. In order to reduce computer time, eight-node brick elements with reduced integration points were used to form the finite element mesh. The finite element mesh contains 4704 elements (12x28x14).
<table>
<thead>
<tr>
<th></th>
<th>Pavement 1</th>
<th>Pavement 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>50 mm</td>
<td>Conventional Wearing Course</td>
<td>Conventional Wearing Course</td>
</tr>
<tr>
<td>50 mm</td>
<td>Conventional Binder Course</td>
<td>Conventional Binder Course</td>
</tr>
<tr>
<td>100 mm</td>
<td>Conventional Type 5A Base Mixture</td>
<td>Conventional Type 5A Base Mixture</td>
</tr>
<tr>
<td>100 mm</td>
<td><strong>Conventional Type 508 Drainable Mixture</strong></td>
<td><strong>Open-graded LSAM</strong></td>
</tr>
<tr>
<td>250 mm</td>
<td>Cement Treated Soil Base</td>
<td>Cement Treated Soil Base</td>
</tr>
<tr>
<td></td>
<td>Compacted Embankment Soil</td>
<td>Compacted Embankment Soil</td>
</tr>
</tbody>
</table>

Figure 6.1. Comparisons Between Type 508 Drainable Base and Open-graded LSAM
<table>
<thead>
<tr>
<th>Pavement 3</th>
<th>Pavement 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>50 mm</td>
<td>Conventional Wearing Course</td>
</tr>
<tr>
<td>50 mm</td>
<td>Conventional Binder Course</td>
</tr>
<tr>
<td>100 mm</td>
<td>Conventional Type 5A Base Mixture</td>
</tr>
<tr>
<td>100 mm</td>
<td>Crushed Stone</td>
</tr>
<tr>
<td>250 mm</td>
<td>Cement Treated Soil Base</td>
</tr>
<tr>
<td></td>
<td>Compacted Embankment Soil</td>
</tr>
</tbody>
</table>

Figure 6.2. Comparisons Between Type 5A Base Mix and Dense-graded 37.5-mm Superpave LSAM
Figure 6.3. 3-D Finite Element Mesh for the Pavements
6.3 MATERIAL PARAMETERS

The material models used in the finite element analyses were as follows. The asphalt mixtures were modeled by the elastic viscoplastic model described in Chapter 6. The elastoplastic Drucker-Prager model was used to describe the crushed limestone, cement treated soil base, compacted embankment soil and subgrade soil. The material parameters for the conventional wearing course mixture, binder mixtures, crushed limestone, cement treated soil base, compacted embankment soil and subgrade soil were obtained from another independent ALF study carried out at the Louisiana Transportation Research Center (Mohammad, et al, 2000). These parameters also agreed with those presented in Chapter 5, “Development of 3-D Dynamic Finite Element Procedure.” The material parameters of the Open-graded LSAM, Type 508 drainable base mixture, 37.5-mm Superpave LSAM, and Type 5A base mixture were obtained primarily from results of the material characterization tests as described in Chapters 3 and 4 (for the mixtures containing SB polymer modified asphalt cement only, i.e. Mixes OG-P, DT-P, D-P, and A-P.) Tables 6.1 through 6.4 present the summary of the material parameters used in the structural comparisons of the 3-D dynamic finite element analyses.

6.4 COMPARISONS OF PREDICTED PAVEMENT RESPONSES

The differences of structural responses of the four pavements were compared using their primary responses to the dynamic load (44.5 kN at 16 km/hr). The primary responses used for comparison were pavement deflections, pavement stresses and strains in the longitudinal and vertical directions, and shear stresses and strains.

6.4.1 Deflections

Figure 6.4 presents the pavement surface deflections along the transverse direction.
### Table 6.1 Material Parameters Used in the Structural Comparisons for Pavements 1

<table>
<thead>
<tr>
<th>Layer #</th>
<th>Material</th>
<th>Wearing Course</th>
<th>Binder Course</th>
<th>Type 5A Base</th>
<th>Type 508 Base</th>
<th>CTB Soil</th>
<th>Embankment Soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness, mm</td>
<td>50</td>
<td>50</td>
<td>100</td>
<td>100</td>
<td>250</td>
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<td></td>
</tr>
<tr>
<td>Material Model</td>
<td>Visco Plastic</td>
<td>Visco Plastic</td>
<td>Visco Plastic</td>
<td>Drucker Prager</td>
<td>Drucker Prager</td>
<td>Drucker Prager</td>
<td></td>
</tr>
<tr>
<td>Elastic Modulus E, kPa</td>
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<td>1.6x10^5</td>
<td></td>
</tr>
<tr>
<td>Poisson's Ratio</td>
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<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.30</td>
<td>0.45</td>
<td></td>
</tr>
<tr>
<td>Drucker Prager</td>
<td>470</td>
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<td>100</td>
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<td>50</td>
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</tr>
<tr>
<td>Visco-plastic</td>
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<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td></td>
</tr>
</tbody>
</table>

Note: Type 5A Base Mix - Mix A-P in the previous chapters; CTB Soil - Cement treated base soil; Embankment Soil - Compacted Embankment Soil.

### Table 6.2 Material Parameters Used in the Structural Comparisons for Pavements 2

<table>
<thead>
<tr>
<th>Layer #</th>
<th>Material</th>
<th>Wearing Course</th>
<th>Binder Course</th>
<th>Type 5A Base</th>
<th>OG-LSAM</th>
<th>CTB Soil</th>
<th>Embankment Soil</th>
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</thead>
<tbody>
<tr>
<td>Thickness, mm</td>
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<td>50</td>
<td>100</td>
<td>100</td>
<td>250</td>
<td>250</td>
<td></td>
</tr>
<tr>
<td>Material Model</td>
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<td>Visco Plastic</td>
<td>Visco Plastic</td>
<td>Drucker Prager</td>
<td>Drucker Prager</td>
<td>Drucker Prager</td>
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</tr>
<tr>
<td>Elastic Modulus E, kPa</td>
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<td>2.26x10^6</td>
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</tr>
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<tr>
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<td>N/A</td>
<td>N/A</td>
<td></td>
</tr>
</tbody>
</table>

Note: Type 5A Base Mix - Mix A-P in the previous chapters; CTB Soil - Cement treated base soil; Embankment Soil - Compacted Embankment Soil; OG-LSAM - Open-graded Large Stone Asphalt Mixture.
Table 6.3 Material Parameters Used in the Structural Comparisons for Pavements 3

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<th>6</th>
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<tbody>
<tr>
<td>Material</td>
<td>Wearing Course</td>
<td>Binder Course</td>
<td>Type 5A Base Mix</td>
<td>Crushed Limestone</td>
<td>CTB Soil</td>
<td>Embankment Soil</td>
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<tr>
<td>Thickness, mm</td>
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<td>50</td>
<td>100</td>
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<tr>
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<td>Drucker Prager</td>
<td>Drucker Prager</td>
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<tr>
<td>Elastic Modulus E, kPa</td>
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<td>1.6x10^5</td>
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<td>Poisson's Ratio</td>
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<td>0.35</td>
<td>0.35</td>
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<td>0.45</td>
</tr>
<tr>
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<td>470</td>
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<td>15</td>
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<td>Drucker Prager (\beta)</td>
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<td>40</td>
<td>40</td>
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<td>20</td>
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<tr>
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<td>1.3</td>
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</tbody>
</table>

Note: Type 5A Base Mix – Mix A-P in the previous chapters;  
CTB Soil – Cement treated base soil;  
Embankment Soil – Compacted Embankment Soil.

Table 6.4 Material Parameters Used in the Structural Comparisons for Pavements 4

<table>
<thead>
<tr>
<th>Layer #</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
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<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
<td>Wearing Course</td>
<td>Binder Course</td>
<td>Superpave LSAM</td>
<td>Crushed Limestone</td>
<td>CTB Soil</td>
<td>Embankment Soil</td>
</tr>
<tr>
<td>Thickness, mm</td>
<td>50</td>
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<td>&gt;250</td>
</tr>
<tr>
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<td>Visco Plastic</td>
<td>Visco Plastic</td>
<td>Drucker Prager</td>
<td>Drucker Prager</td>
<td>Drucker Prager</td>
</tr>
<tr>
<td>Elastic Modulus E, kPa</td>
<td>5.43x10^6</td>
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<td>5.0x10^5</td>
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<td>1.6x10^5</td>
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<td>Poisson's Ratio</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.30</td>
<td>0.45</td>
</tr>
<tr>
<td>Drucker Prager K, kPa</td>
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<td>470</td>
<td>400</td>
<td>15</td>
<td>75</td>
<td>50</td>
</tr>
<tr>
<td>Drucker Prager (\beta)</td>
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<td>40</td>
<td>40</td>
<td>40</td>
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<td>20</td>
</tr>
<tr>
<td>Visco-plastic D</td>
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<td>0.5</td>
<td>1.0</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Visco-plastic n</td>
<td>1.5</td>
<td>1.4</td>
<td>1.3</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Note: Type 5A Base Mix – Mix A-P in the previous chapters;  
CTB Soil – Cement treated base soil;  
Embankment Soil – Compacted Embankment Soil.
Figure 6.5 presents the pavement deflections underneath the wheel along the pavement depth.

- **Comparisons Between Pavement 1 and Pavement 2**

  Pavement 1 (with conventional Type 508 drainable base mix) showed higher values of deflections in the pavement structure than Pavement 2 (with open-graded large stone asphalt mixture,OG-P). Higher deflection values are normally associated with weaker pavement structure; therefore, the open-graded large stone asphalt mixture improved the structural support of the pavement when compared with the conventional Type 508 drainable mixture.

**Comparison Between Pavement 3 and Pavement 4**

There was no significant difference in pavement deflections between the Pavement 3 (with conventional Type 5A base mixture) and Pavement 4 (with 37.5-mm Superpave large stone asphalt mixture, L-P). Therefore, there was no significant improvement of pavement support by using the 37.5-mm Superpave LSAM to replace the conventional Type 5A base mixture.

### 6.4.2 Strains

Figures 6.6 through 6.9 present the dynamic responses of strains of pavements within one load cycle. The responses of Pavement 1 were compared with those of Pavement 2, and Pavement 3 was compared with Pavement 4. The longitudinal and shear strains at the bottom of the wearing course and the binder course were used for comparison. Figures 6.10 through 6.13 presents the various peak values of pavement strains underneath the wheel.
Figure 6.4. Pavement Surface Deflections Along the Transverse Direction

Figure 6.5. Pavement Deflections Along the Depth Underneath the Wheel
Figure 6.6. Longitudinal Strain $\varepsilon_{yy}$ at Bottom of Wearing Course
Figure 6.7. Longitudinal Strain $\varepsilon_{yy}$ at Bottom of Binder Course

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Figure 6.8. Shear Strain $\varepsilon_{yz}$ at Bottom of Wearing Course
Figure 6.9. Shear Strain $\varepsilon_{yz}$ at Bottom of Binder Course

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Figure 6.10. Longitudinal Strain $\varepsilon_{yy}$ Along the Depth
Figure 6.11. Vertical Strain $\varepsilon_{zz}$ Along the Depth
Figure 6.12. Shear Strain $\varepsilon_{yz}$ Along the Depth
Figure 6.13. Shear Strain $\varepsilon_{xx}$ Along the Depth

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• **Comparisons Between Pavement 1 and Pavement 2**

Within one cycle of dynamic loads (Figures 6.6 through 6.9), Pavement 1 (with conventional Type 508 drainable base mix) showed higher longitudinal positive strains and shear strains at the bottom of the wearing course and binder course than Pavement 2 (with open-graded large stone asphalt mixture, OG-P). Along the pavement depth underneath the wheel (Figures 6.10 through 6.13), Pavement 2 (with open-graded LSAM, OG-P) exhibited lower longitudinal ($\varepsilon_{yy}$), vertical ($\varepsilon_{zz}$) and shear ($\varepsilon_{yz}$) strains than Pavement 1 (with conventional Type 508 drainable base mixture). Higher strains especially higher horizontal positive strain and shear strains are normally associated with pavement distresses such as fatigue cracking and rutting. Therefore, the open-graded large stone asphalt mixture improved the structural support of the pavement when compared with the conventional Type 508 drainable mixture.

• **Comparison Between Pavement 3 and Pavement 4**

Within one cycle of dynamic loads (Figures 6.6 through 6.9), there was no difference in longitudinal strain ($\varepsilon_{yy}$) at the bottom of the wearing and binder courses between Pavement 3 (with conventional Type 5A base mixture) and Pavement 4 (with 37.5-mm Superpave large stone asphalt mixture, L-P). Pavement 4 showed less peak positive and greater negative longitudinal strain ($\varepsilon_{yy}$) than Pavement 3 at the bottom of binder course. There was no evidence of significant improvement of pavement support by using the 37.5-mm Superpave LSAM to replace the conventional Type 5A base mixture.
6.4.3 Stresses

Figures 6.14 through 6.17 present the dynamic response stresses of pavements within one load cycle. The responses of Pavement 1 were compared with those of Pavement 2, and Pavement 3 with Pavement 4. The longitudinal and shear stresses at the bottom of the wearing course and the binder course were used in comparisons. Figures 6.18 through 6.21 presents the various peak values of pavement stress underneath the wheel.

- **Comparisons Between Pavement 1 and Pavement 2**

  Within one cycle of dynamic loads (Figures 6.14 through 6.17), Pavement 1 (with conventional Type 508 drainable base mix) showed higher longitudinal compressive stress at the bottom of the wearing course and lower longitudinal compressive stress at the bottom of the binder course than Pavement 2 (with open-graded large stone asphalt mixture, OG-P). Pavement 1 showed higher shear stresses at both bottom of the wearing and binder courses than Pavement 2. Along the pavement depth underneath the wheel (Figures 6.18 through 6.21), Pavement 2 (with open-graded LSAM, OG-P) exhibited less longitudinal ($\sigma_{yy}$), and shear ($\sigma_{yz}$) stresses than Pavement 1 (with conventional Type 508 drainable base mixture) at locations close to the pavement surface. Higher longitudinal and shear stresses at the top asphalt concrete layers will develop higher longitudinal and shear strains at these layers, which in turn will accelerate the development of pavement distress such as rutting and fatigue cracking. Therefore, the open-graded large stone asphalt mixture improved the structural support of the pavement when compared with the conventional Type 508 drainable mixture.
Comparison Between Pavement 3 and Pavement 4

Within one cycle of dynamic loads (Figures 6.14 through 6.17), Pavement 4 (with 37.5-mm Superpave large stone asphalt mixture, L-P) exhibited slightly lower longitudinal stress ($\sigma_{yy}$) at the bottom of the wearing course and lower longitudinal stress ($\sigma_{yy}$) at the bottom of the binder course than Pavement 3 (with conventional Type 5A base course). There was no appreciable difference in shear stresses ($\sigma_{yz}$) at the bottom of the wearing and binder courses between Pavement 3 (with conventional Type 5A base mixture) and Pavement 4 (with 37.5-mm Superpave large stone asphalt mixture, L-P). Along the pavement depth underneath the wheel (Figures 6.18 through 6.21), there was no appreciable difference in longitudinal stress ($\sigma_{yy}$), vertical stress ($\sigma_{zz}$) and shear stress ($\sigma_{yz}$) between Pavement 3 and Pavement 4. There was no evidence of any appreciable improvement of pavement support by using the 37.5-mm Superpave LSAM to replace the conventional Type 5A base mixture.

6.5 SUMMARY OF NUMERICAL STRUCTURAL COMPARISONS

Numerical simulations were conducted for two groups of pavements. Each group had two pavements: one with conventional mixtures and one with large stone asphalt mixtures developed in this study. A total of four pavement sections were analyzed in this chapter. The 3-D dynamic finite element procedure developed in this study was used for the pavement structural analyses. Material parameters were obtained primarily from the mixture characterization tests described in Chapters 3 and 5. The following conclusions and observations can be drawn from the numerical analyses of the pavement structures.
• Observations:
  o Pavement 2 (with open-graded large stone asphalt mixture, OG-P) predicted lower longitudinal and shear strains than did pavement 1 (with conventional Type 508 drainable mixture);
  o Pavement 2 predicted lower longitudinal and shear stresses than did pavement 1 in the wearing and binder courses;
  o Pavement 2 predicted lower deflections under the wheel along the transverse cross section as well as with pavement depth than did pavement 1;
  o Pavement 4 (with dense-graded, 37.5-mm Superpave large stone asphalt mixture, L-P) predicted values of stress, strain and deflection that were similar to those of pavement 3 (with conventional Type 5A base mixture).

• Conclusions:
  o It was evident, through the numerical simulations, that additional structural support was predicted by replacing the conventional Type 508 drainable mixture with the open-graded large stone asphalt mixture developed in this study;
  o It was not evident, through the finite element analyses, that the use of the dense-graded, 37.5-mm Superpave large stone asphalt mixture would improve the pavement structural support when compared with the conventional Type 5A base mixture.
Figure 6.14. Longitudinal Stress $\sigma_{yy}$ at Bottom of Wearing Course
Figure 6.15. Longitudinal Stress $\sigma_{yy}$ at Bottom of Binder Course
Figure 6.16. Shear Stress $\sigma_{yz}$ at Bottom of Wearing Course
Figure 6.17. Shear Stress $\sigma_{yz}$ at Bottom of Binder Course
Figure 6.18. Longitudinal Stress $\sigma_{yy}$ Along the Depth
Figure 6.19. Vertical Stress $\sigma_{zz}$ Along the Depth
Figure 6.20. Shear Strain $\sigma_{yz}$ Along the Depth
Figure 6.21. Shear Stress $\sigma_{xz}$ Along the Depth
CHAPTER 7.
SUMMARY AND CONCLUSIONS

A study has been conducted to develop fundamental material characterization and numerical simulations of large stone asphalt mixtures (LSAM) in flexible pavement structures. Two types of large stone asphalt mixture, an open-graded LSAM and a dense-graded LSAM were designed for this study. The dense-graded LSAM also satisfied the Superpave volumetric specifications for Level II traffic volume. These two LSAMs were compared against two Louisiana DOTD conventional mixtures, the Type 508 open-graded drainable base mixture and the Type 5A base mixture. Three types of asphalt binders, an SB polymer modified PG 70-22M, a gelled asphalt, PG 70-22MAlt and a conventional PG 64-22 were used to study the effect of asphalt binders on the characteristics of the large stone asphalt mixtures. Fundamental engineering property tests were used to characterize the laboratory rut susceptibility, durability and moisture susceptibility of these mixtures. A 3-D dynamic finite element procedure was developed during this study. Advanced material models of viscoplasticity and elastoplasticity were incorporated into the 3-D dynamic finite element procedure. This procedure was used to estimate the structural performance of two groups of pavements, each with two pavements, a conventional mixtures and a the LSAM developed in this study. The specific conclusions were given in the individual chapters. The following general observations and conclusions could be made through this study.

- The mixture design and volumetric test procedures such as the glass beads method and the degree of stone-on-stone contact, were effective in developing the designs for the open-graded and dense-graded LSAM;
• The dual mode permeameter and the corresponding permeability test procedures developed in this study were more effective for the evaluation of permeability characteristics of the mixtures in this study than the traditional procedures;

• The overall laboratory rut-resistance of the open-graded large stone asphalt mixture was significantly higher than that of the conventional Type 508 drainable base mixture;

• The laboratory moisture susceptibility of open-graded large stone asphalt mixture was significantly better (less susceptible) than that of the conventional Type 508 drainable base mixture;

• The laboratory indirect tensile strength (ITS) of the open-graded large stone asphalt mixture was significantly higher than that of the conventional Type 508 drainable base mixture;

• There was no significant difference in the rut-resistance, indirect tensile strength and moisture susceptibility between the dense-graded, 37.5 Superpave large stone asphalt mixture and the conventional Type 5A base mixture;

• Among the open-graded and dense-graded LSAM, the mixtures with SB polymer modified asphalt, PG 70-22M showed better performance than mixtures including other asphalt binders;

• The 3-D dynamic finite element procedure was able to predict the dynamic stress and strain responses of the asphalt pavement that were close to field measurements;
Rate-dependent viscoplastic models incorporated into the 3-D dynamic finite element procedure were able to simulate the viscous and permanent strain characteristics of the asphaltic concrete material under the traffic loads;

The numerical simulation indicated that the pavement containing open-graded large stone asphalt mixture developed in this study had superior structural support when compared with the pavement containing the conventional Type 508 open-graded drainable base mixture;

It was not evident through the numerical simulation that the pavement containing dense-graded, 37.5-mm Superpave large stone asphalt mixture developed in this study produced appreciable improvement in structural support when compared with the one containing conventional Type 5A base mixture.

Based on the results of this study, it is recommended that the following researches be considered:

• Build pavement test sections, such as the ALF sections to compare the field performance of pavements containing large stone asphalt mixtures with pavements constructed with conventional mixtures;

• Conduct a complete sensitivity analysis using the viscoplastic models developed in this study;

• Conduct sophisticated laboratory tests to evaluate the rate-dependency of asphalt mixtures and calibrate the material parameters;

• Develop a temperature dependent, thermo-visco-plastic model since the model used in this study does not have temperature as a model parameter.
REFERENCES


VITA

Baoshan Huang was born on August 27, 1962, in Shanghai, China. He received his bachelor of engineering degree from Tongji University in 1984. He worked as an assistant engineer at the Shanghai Institute of Geotechnical Investigation and Design during 1984 and 1985. He received his master of science degree from the Tongji University in 1988. He worked as a research and consulting engineering at the Shanghai Geotechnical Investigation and Design from 1988 through 1994. Since 1994, he has studied for his doctoral degree in Civil Engineering at Louisiana State University.

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