NCHRP Report 386

Design and Evaluation of Large-Stone Asphalt Mixes

Transportation Research Board
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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Report 386

Design and Evaluation of Large-Stone Asphalt Mixes

Texas Transportation Institute
Texas A & M University System
College Station, TX

Subject Areas

Pavement Design, Management and Performance
Materials and Construction

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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FOREWORD

By Staff
Transportation Research Board

This report presents a practice for the design and analysis of dense- and open-graded, large-stone mixes (LSM) and guidelines for the construction of hot-mix asphalt (HMA) pavements incorporating LSM. Though typically used as base layers, LSM can serve as surface layers where rutting (permanent deformation) is a concern. For the purposes of this study, LSM are defined as HMA paving mixes containing maximum aggregate sizes between 25 and 63 mm (1 and 2.5 in.). The report also contains the detailed research results that support the mix design and analysis practice, including an evaluation of the performance of LSM pavements included in the Strategic Highway Research Program (SHRP) Long-Term Pavement Performance (LTPP) general pavement sections (GPS) inventory and a study of the effectiveness of LSM designs using accelerated pavement testing (APT). The report should be of interest to the personnel of state highway agencies (SHAs), paving contractors, and others responsible for the design and construction of asphalt pavements with LSM.

Large increases in truck traffic on the nation's highways in the past three decades have contributed to rutting of asphalt pavements subjected to high traffic volume, heavy wheel loads, and high tire-contact pressures. SHAs have adopted several approaches to mitigate this problem. One that shows considerable potential is the use, in surface courses, of HMA prepared from LSM. There is considerable evidence that properly designed LSM provide increased support for these heavy traffic loads.

Under NCHRP Project 4–18 "Design and Evaluation of Large Stone Mixtures," the Texas Transportation Institute's team (including Brent Rauhut Engineering, Inc.; the Indiana Department of Transportation; the University of California–Berkeley; and Mr. James A. Scherocman) was assigned the tasks of evaluating the effectiveness of LSM in resisting rutting in asphalt pavements, developing an LSM mix design and analysis method, and preparing construction guidelines for LSM pavements.

The research team reviewed relevant domestic and foreign literature, surveyed the SHAs on the specifications for and effectiveness of LSM, conducted laboratory testing on field cores and laboratory-compacted specimens, and carried out full-scale APT of LSM.

The published report presents several products expected to facilitate the wider use of LSM: an LSM design and analysis method in the form of an AASHTO standard practice (provided in Chapter 2); a method for estimating the degree of stone-on-stone contact in compacted LSM specimens (provided in Chapter 3); a method for estimating the draindown characteristics of open-graded LSM (provided in Chapter 3); a method for accurately measuring the bulk specific gravity of LSM specimens with water-permeable voids (provided in Chapter 3); and LSM field construction guidelines, *Guidelines for HMA Pavement Construction with LSM* (provided as Chapter 4).

The LSM design and analysis method is a two-level (low and high traffic-volume) system for dense- and open-graded LSM. The Level 1 LSM design method is appropriate for low-volume roads and requires only minimal materials testing. It uses a spreadsheet-based computer program to estimate an optimum design on the basis of measured or reported grading of available aggregate stockpiles; the designer selects the desired air voids content (VTM) of the compacted LSM.

The Level 2 dense-graded LSM design is intended for higher volume roads. It is an iterative process that begins with a Level 1 design and ensures that the LSM provides required resistance to permanent deformation (rutting) through development of a coarse aggregate skeleton with good stone-on-stone contact capable of carrying the intended traffic load. A performance-related mix test (the Superpave SST procedures [AASHTO TP 7] or a uniaxial [static] compressive creep test) is used to determine that the LSM meets minimum rutting resistance criteria.

A Level 2 open-graded LSM design incorporates the same volumetric design principles used for dense-graded LSM to ensure that the mixture has an adequate permeability, but it does not require mix testing to evaluate rutting resistance. A draindown test is used to verify or make final adjustments to the optimum asphalt content selected with the computer program.

The companion to this published report is the Level 1 computer program written to run as a spreadsheet in Microsoft Excel, Version 5.0 or higher. It is available for downloading from the Transportation Research Board's Internet World Wide Web site at *http://www2.nas.edu/trbcrp/229a_35a.html.*

CONTENTS

CHAPTER 1

INTRODUCTION

This report presents a practice for the design and analysis of dense- and open-graded, large-stone mixes (LSM) and guidelines for the construction of hot-mix asphalt (HMA) pavements incorporating LSM. Work was accomplished under NCHRP Project 4-18, "Design and Evaluation of Large Stone Mixtures." For the purposes of this study, LSM are defined as HMA paving mixes containing maximum aggregate sizes between 25 and 63 mm (1 and 2.5 in.). Though typically used as base layers, LSM can serve as surface layers where rutting (permanent deformation) is a problem or concern.

In keeping with current directions in asphalt paving mix design technology (typified by the Superpave system for dense-graded mixes), the design practice provides the option of two levels of volumetric design complexity, including the use of performance-related mixture analysis tests.

The Level 1 LSM design method is appropriate for low-volume roads and requires only minimal materials testing. With this method, personnel use a spreadsheet-based computer program to estimate an optimum design on the basis of measured or reported grading of available aggregate stockpiles; the designer can select the desired air voids content (VTM) of the compacted LSM.

The Level 2 dense-graded LSM design is intended for higher volume roads. It is an iterative process that begins with a Level 1 design and ensures that the mixture provides a required resistance to permanent deformation (rutting). The objective is to design a mix with a coarse aggregate skeleton with good stone-on-stone contact capable of carrying the intended traffic load. Use of this method requires specific knowledge of the measured gradations of all aggregate stockpiles; the computer program is used to build a coarse stone skeleton and fill its voids with the smaller aggregates from succeeding stockpiles. Optimum asphalt binder content is then estimated on the basis of desired levels of voids in mineral aggregate (VMA), VTM, and other factors. Finally, a performance-related mixture test (the Superpave simple shear test [SST] procedures [AASHTO TP 7] or a uniaxial [static] compressive creep test) is used to ensure that the LSM meets minimum rutting resistance criteria.

The Level 2 LSM design method employs the Superpave gyratory compactor for specimen preparation. Depending on the maximum aggregate size in the LSM and other specific aggregate and volumetric properties, the angle of gyration may increase substantially from the 1.25 deg specified in AASHTO TP 4 for conventional, dense-graded mixes. Other gyratory compactors may be used if their operating characteristics can be adjusted to meet the requirements of AASHTO TP 4.

A Level 2 open-graded LSM design incorporates the same volumetric design principles used for dense-graded LSM to ensure that the mixture has an adequate permeability, but it does not require mix testing to evaluate rutting resistance. A draindown test is used to verify or make final adjustments to the optimum asphalt content selected by the computer program.

In general, the construction of LSM pavements demands application of the same sound principles of production, placement, and compaction used for conventional (maximum aggregate size of 25 mm [1 in.] or less) paving mixes. Therefore, the LSM pavement construction guidelines emphasize procedures to prevent or remedy the most frequent problems characteristic of LSM, which have slowed its adoption and use in some states—segregation, aggregate fracture, and equipment wear.

The report has two parts. The first part, Chapters 2 through 5, is intended principally for the practitioner responsible for LSM design and pavement construction and includes the following:

- A practice, intended for laboratory engineers and technicians, for design and analysis of LSM, presented in the format of an AASHTO provisional standard (Chapter 2);
- Three test methods that support the mix design practice, presented in the format of AASHTO provisional standards (Chapter 3);
- A manual, intended for field personnel including inspectors responsible for quality control and quality acceptance (QC/QA), for the construction of large-stone HMA pavements (Chapter 4); and
- A brief, introductory summary of the research results of NCHRP Project 4-18 and the conclusions drawn from them that form the basis for the LSM design practice and pavement construction guidelines (Chapter 5).

The appendixes form the second part of the report; they are intended principally for specialists interested in the experimental results upon which the practical products in Chapters 2 through 4 are based. The appendixes include the following:

- The detailed research approach; experimental findings; and their interpretation, appraisal, and application to practice (Appendix A);
- A summary of the performance, structural, and material properties of LSM pavements included in the SHRP Long-Term Pavement Performance (LTPP) general pavement sections (GPS) inventory (Appendix B);
- Detailed results of material, volumetric, and performance tests obtained on laboratory-compacted specimens and field pavement cores in the course of the research (Appendix C); and
- The results of accelerated performance testing (APT) conducted on LSM specimens with the full-scale accelerated loading facility operated by the Indiana DOT's Division of Research at West Lafayette, Indiana (Appendix D).

The Level 1 computer program is written to run as a spreadsheet in Microsoft Excel, Version 5.0 or higher. It is available for downloading from the Transportation Research Board's Internet World Wide Web site at *www2.nas.edu/trbcrp/229a_35a.html* (go to the icon at the bottom of the *Status* block). Instructions for using the computer program are in Annex X1 of the mix design practice in Chapter 2.

CHAPTER 2

LSM DESIGN AND ANALYSIS PROCEDURES: FEATURES AND STANDARD PRACTICE

This chapter presents the mix design and analysis method developed in this project for dense-, open-, and gap-graded LSM.

Section 2.1 briefly describes the key features in the development of the LSM design and analysis method. The method is similar to that of the Superpave design method for conventional, dense-graded paving mixes in that it employs gyratory compaction, and the level of complexity of the volumetric design and the need for performance-related mix analysis tests are guided by the traffic loads expected on the pavement. Detailed research results supporting the design and analysis method are included in Appendix A.

Section 2.2 presents the mix design and analysis method in the format of an AASHTO standard practice.

2.1 KEY DEVELOPMENT FEATURES

2.1.1 LSM Design

A new philosophy was adopted for the LSM design procedure. With the trend to place most or all of the burden for design and QC on the HMA paving contractor, it becomes increasingly important for the contractor to be able to use existing material stockpiles in as many types of mix designs as practical. For example, the good stone-on-stone contact required in LSM might favor combination of small railroad ballast with some other existing stockpiles. Two HMA plants operated by the same contractor might have stockpiles with quite different gradations that could be blended in different ways to prepare a well-performing LSM. Allowing contractors flexibility to achieve mixes with the required performance characteristics can save the owner agency money in terms of both the initial investment and life-cycle costs.

This design philosophy has two key implications as follows:

- Contractors should find it relatively easy to produce a good LSM design using existing stockpiles of the requisite quality, and
- The time-honored concepts of gradation bands and restricted zones no longer play a particularly useful role.

The LSM design method produces mixes that are relatively coarse—there is no "restricted zone" in the gradation curve. It uses stockpile gradations available to the contractor so there will be cases where the gradation has a "hump" in it. However, the methodology in the design procedure balances these normally undesirable features by positively ensuring good stone-on-stone contact in the larger stone fraction. This permits the method to accommodate even cold-milled RAP and some lower quality materials in the LSM without a performance penalty.

In its original concept, the LSM design methodology emphasized particle shape, volume, and film thickness calculations. This resulted in mixes considered too harsh for practical use, and the methodology evolved to one that produces somewhat less harsh mixes and that allows the user to specify a desired dust-to-asphalt ratio.

The original concept also highlighted "stone-filled" and "open-graded" designs. The distinction between stone-filled and dense-graded mixes can be unclear. Although a stone-filled gradation may be close to that found typically for dense-graded mixes, stone-filled mixes are not limited to gradations near the 0.45 power line and often have gradations like stone-mastic asphalt (SMA) or gap-graded mixes.

Open-graded mix designs aim at the high permeability needed for drainage courses; this requires an air voids content near 15 percent. The open-graded LSM design may also be suitable for highly textured, open-graded friction courses, but its applicability for this purpose was not studied in this project.

The LSM design method allows the user to design a stone-filled or open-graded LSM simply by specifying an appropriate air voids content (e.g., 4 percent for stone-filled, 15 percent for open-graded, or any other air voids content desired).

The LSM design method, whether applied to dense-graded, open-graded, or stone-filled mixes, has three specific objectives as follows:

- Design the LSM so that the traffic load is carried by the stone (coarse aggregate) skeleton—Except for conservation of asphalt binder, there is little reason to use large-size aggregates in a paving mix if the large stone does not participate in carrying the load;

- Emphasize the characteristics of stockpiles at the HMA plant—This facilitates the transfer of the laboratory mix design to field production, ensures that the aggregate in the largest stockpile selected for use actually participates in carrying the traffic load, and makes certain that the aggregate contributed by the other stockpiles is used to the best advantage in the LSM; and
- Provide adequate asphalt binder for the intended use of the LSM.

The new LSM design process is made up of seven basic steps as follows:

- Select asphalt binder grade and modifiers (if any).
- Select the candidate aggregate stockpiles.
- For the largest stone stockpile only, obtain the aggregate unit weight,[1] specific gravity, and gradation.
- For all other stockpiles, obtain the aggregate-specific gravity and gradation.
- Eliminate, through computation, any stockpiles that will not fit within the void structure of the largest stone stockpile.
- Estimate the effective asphalt binder content[2] on the basis of the maximum aggregate size and the required air voids content.
- Proportion the remaining stockpiles such that the sum of the aggregate proportions is 1.0 and the dust-to-asphalt ratio falls between the limits specified by the owner agency.[3]

2.1.2 LSM Analysis

The LSM design and analysis method was developed to ensure as much compatibility with the Superpave system as possible. Toward this goal, two mix analysis procedures employing the SST device (tests and equipment are described in AASHTO TP 7) and one using a uniaxial test machine are provided. Agencies having access to an SST device will find that the repeated shear at constant height (RSCH) test is the more rapid of the two Superpave alternatives and the test for which significant results were obtained in this project. Agencies without access to an SST device will find the uniaxial procedure useful.

Open-graded mixes, SMA, and LSM, which clearly outperform conventional mixes in certain field conditions, often yield poor results in unconfined laboratory tests because confinement, Poisson's ratio, and dilation enhance field performance.

Analysis procedures were chosen that include some measure related to Poisson's ratio and dilation. The load required to maintain the specimen at constant height in the RSCH test is an indirect indicator of the desired parameter (but see the caveat in Paragraph 3.4 of the standard practice in Section 2.2 of this chapter). In the uniaxial test, the dilation can be estimated if, as is strongly recommended, the radial displacements are measured in addition to the axial displacements.

The application of confining pressure in the uniaxial test is one way to make the material's laboratory response approach that in the field. Another way is to use the full Superpave analysis procedures, which include the application of confining pressure during the conduct of the volumetric and uniaxial strain tests (AASHTO TP 7). However, the use of confining pressure increases the complexity of the analysis testing, and the standard practice opts for the simpler, unconfined procedures for most routine LSM designs.

The pay factor bands included in the standard practice in Section 2.2 of this chapter are quite approximate and based on a very limited data set. Owner agencies may wish to alter the proposed bands to suit local needs.

2.1.3 LSM Specimen Preparation

Specimens used in this study were either cored from field pavements or compacted in the laboratory using a Texas DOT gyratory compactor operating with an angle of gyration of 5 deg. Specimens of the height and diameter required for the LSM design and analysis method could not be successfully compacted using the Superpave gyratory compactor set at an angle of gyration of 1.25 deg.

The Superpave mix design and analysis system provides two nominal maximum size aggregate gradations (25 and 37.5 mm) that will fall within the definition of LSM used in this project. Superpave mixes designed with these nominal aggregate sizes *to meet the Superpave aggregate control point and restricted zone requirements* should achieve adequate compaction at 1.25 deg. However, it does not follow that LSM based on the same nominal maximum sizes of aggregate will compact at this angle to the desired air voids content within a reasonable number of gyrations because the LSM method produces harsher mixes. Results discussed in Appendix A indicate that a higher angle of gyration is probably necessary for LSM designs with largest stone sizes in the range between 37.5 mm (1.5 in.) and 63.5 mm (2.5 in.) when tall, wide-diameter specimens are needed.

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[1]The design method uses centimeters as the dimension of length throughout because the unit weight of water is expressed as 1 g/cm^3.

[2]This method expresses the asphalt binder content as a percent of weight of the total mix and its volume as a percent of the total mixture volume.

[3]The dust-to-asphalt ratio, or dust proportion, is defined as the ratio of the weight percent of aggregate passing the #200 (0.0075 cm) sieve to the effective asphalt binder content expressed as a percent by weight of the total mix; a ratio between 0.6 and 1.2 is suggested by the FHWA.

Therefore, the standard practice for LSM mix design and analysis provides for the use of three alternative compaction methods as follows:

- The Superpave gyratory compactor (or another gyratory compactor meeting the AASHTO TP4 requirements) operating at an angle of gyration sufficient to provide satisfactory densification without requiring excessive ram pressure or numbers of gyrations,
- Rolling wheel compaction (AASHTO PP3), and
- Marshall compaction (AASHTO T 245) if no other means are available.

2.1.4 Basic Properties of LSM

The laboratory program, described in Appendix A, revealed the difficulty in obtaining precise, accurate measurements of bulk specific gravity for LSM specimens with method AASHTO T 166, especially for open-graded designs. For this reason, the glass bead method presented in Section 3.2 of Chapter 3 was developed; it has the additional benefit of providing a dry specimen that can be used immediately in the testing program.

2.1.5 Relationship to the Superpave System

The draft standard practice presented in Section 2.2 differs from the Superpave system in terms of the technique used to establish the design aggregate structure and the design asphalt content. Both the LSM and Superpave methods are based on similar volumetric design principles, but the lack of control points and restricted zone in establishing the LSM gradation and the flexibility permitted the design engineer to specify whatever air voids content and dust-to-asphalt ratio are appropriate are unique to the LSM method.

On the other hand, the LSM method was made as compatible as possible to the Superpave method in terms of specimen preparation, compaction, and mix analysis. Other similarities (and differences) will become clear to practitioners as both methods come into more routine, widespread use.

2.2 STANDARD PRACTICE FOR DESIGN AND ANALYSIS OF LARGE-STONE MIXES (LSM) FOR HOT-MIX ASPHALT (HMA)—AASHTO DESIGNATION: PPAA-96

The recommended standard practice for design and analysis of LSM for HMA is provided starting on page 7.

Standard Practice
for

Design and Analysis of Large-Stone Mixes (LSM) for Hot-Mix Asphalt (HMA)
AASHTO DESIGNATION: PPaa-96

1. SCOPE

1.1 This standard provides laboratory test procedures for the design and analysis of large-stone mixes (LSM).

1.2 This standard is applicable to specimens prepared by gyratory compaction or cored from a laboratory-scale slab compacted with a rolling wheel compactor or from a full-scale pavement. The minimum specimen diameter is 150 mm. The height required varies with the analysis procedure selected (75 mm for shear testing, 150 mm for axial testing). The standard is applicable to materials having maximum aggregate sizes in the range of 25 to 63.5 mm.

1.3 Two general categories of mixes are addressed in the mix design procedure: low-permeability standard LSM and open-graded, permeable LSM intended for use as a drainable base material.

1.4 The standard provides three methodologies for analyzing LSM. Although the procedures may be used for LSM of various degrees of permeability, these procedures are optional for open-graded, drainable base LSM and will not normally be conducted on such drainable base materials.

Procedure A—Superpave Performance Testing
Procedure B—Repeated Shear at Constant Height (RSCH) Test for Rut Estimation
Procedure C—Uniaxial Test

1.5 Complete analysis of the test results from Procedure A requires the use of the Superpave Software, Version 2.1 or higher.

1.6 *This practice may involve hazardous materials, operations, and equipment. It does not purport to address all the safety problems associated with its use. It is the responsibility of whoever uses this practice to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to its use.*

2. REFERENCE DOCUMENTS

2.1 AASHTO Standards

PP2	Practice for Short and Long Term Aging of Hot Mix Asphalt (HMA)
PP3	Practice for Preparing Hot Mix Asphalt (HMA) Specimens by Means of the Rolling Wheel Compactor
Ppbb	Practice for Determining the Bulk Specific Gravity of Compacted Bituminous Large Stone Mixes Using Glass Beads
Ppcc	Practice for Estimating Degree of Stone-on-Stone Contact in Compacted Large Stone Mixes (LSM)
Ppdd	Practice for Determining Draindown Characteristics of Open-Graded Compacted Large Stone Mixes (LSM)
T 2	Sampling of Aggregates
T 19	Unit Weight and Voids in Aggregate
T 27	Sieve Analysis of Fine and Coarse Aggregates
T 40	Sampling Bituminous Materials
T 84	Specific Gravity and Absorption of Fine Aggregate
T 85	Specific Gravity and Absorption of Coarse Aggregate
T 96	Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
T 168	Sampling Bituminous Paving Mixtures
T 245	Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus
T 283	Resistance of Compacted Bituminous Mixture to Moisture Induced Damage
TP 4	Method for Preparing and Determining the Density of Hot Mix Asphalt (HMA) Specimens by Means of the SHRP Gyratory Compactor
TP 7	Method for Determining the Permanent Deformation and Fatigue Cracking Characteristics of Hot Mix Asphalt (HMA) Using the Simple Shear Test (SST) Device

2.2 *ASTM Standards*

D3202	Practice for Preparation of Bituminous Mixture Beam Specimens by Means of the California Kneading Compactor
D3387	Method for Compaction and Shear Properties of Bituminous Mixtures by Means of the U.S. Corps of Engineers Gyratory Testing Machine (GTM)
D5361	Practice for Sampling Compacted Bituminous Mixtures for Laboratory Testing

2.3 Other Documents

The *Superpave Mix Design Manual for New Construction and Overlays,* SHRP Report SHRP-A-407, 1994.

Superpave Asphalt Mixture Design & Analysis, National Asphalt Training Center Demonstration Project 101, 1994.

3. SIGNIFICANCE AND USE

3.1 Care must be taken in interpreting mix analysis results from LSM tested using 150-mm-diameter specimens and maximum aggregate sizes larger than 37.5 mm. Although the volume of the maximum-sized rock is often relatively small, it can contribute to increased variability in the results and significant disturbances in the stress field. The 150-mm size was specified primarily to maintain compatibility with existing Superpave compaction and testing equipment.

3.2 For LSM with significant portions of the volume taken up by stones 37.5 mm and larger, users should expect difficulties in handling compacted specimens in the unconfined state at 7-day maximum pavement temperatures. If significant handling and testing are contemplated at these temperatures, users should consider using a membrane stretcher and placing a membrane on the specimen (a 0.6-mm thickness or greater nitrile, neoprene, or latex membrane should suffice to help protect the specimen from damage when reasonable care and speed are exercised). Asphalt binders at these high temperatures often simply do not have the binding properties necessary to counteract the body forces (or "self-weight") present in specimens containing large aggregate particles. See Note 3 for further discussion of the temperature issues.

3.3 Unconfined uniaxial testing conducted without radial measurements is known to be of limited utility when trying to relate laboratory performance to field performance with LSM and SMA in particular. The two key elements of this limitation are as follows:

- If the test is both unconfined and conducted without radial measurements, there is (a) virtually no possibility of mobilizing the internal friction of the aggregate without the benefit of confinement similar to that which is present in a full-scale pavement because the binder acts more as a lubricant than a binder at the relatively high temperatures used in tests related to rutting; and (b) if there is no radial measurement as well, there is very little possibility of quantifying the potential for mobilizing friction.
- If an attempt is made to relate laboratory and field performance using only one parameter out of a full, time-dependent, elastic-plastic model (e.g., complex compliance or slope of the creep response curve only), the relationship will likely be quite crude. The minimum slope of the creep compliance can sometimes be successfully used to rank materials tested in the laboratory. The ratio of plastic strain to total strain at a selected time, coupled with Poisson's ratio/dilation measurements, appears to be reasonably well suited for ranking materials in the laboratory with some possibility of relating results to field conditions as well.

The acceptance guidelines presented in paragraph 14.2 of this standard are based on a very limited dataset. It is recommended that owner agencies set up a continuous improvement program to continuously review and adjust these guidelines to reflect local experience.

3.4 Poisson's ratio is expected to equal or exceed 0.5 for LSM with good stone-on-stone contact. Materials with good stone-on-stone contact will often exhibit higher axial loads while maintaining constant height in the RSCH test, although recent evidence from the University of California—Berkeley indicates that other factors are at work here as well, so no guidance is given in this standard for how much higher these loads might be. The previous two statements should only be interpreted as identifying superior mixtures if other portions of the analysis indicate that long-term performance is acceptable (i.e., impending failure and nonuniform strain fields can also be accompanied by high Poisson's ratios in the uniaxial test and high axial loads in the RSCH test under certain circumstances).

3.5 If the analysis procedure indicates that the mixture will not perform adequately, the following suggestions are offered for correcting the problem:

- Remove any stockpile that has a D_{50} size equal to *VSI* (see paragraph 9.3.4) and perform a new mix design.
- Change the compaction effort to increase the unit weight of the mixture if that is the proper direction to proceed (within the limits of what available compaction equipment will realistically be able to accomplish at the site, which will also be affected by characteristics of the platform against which the material will be compacted) and/or modify the asphalt binder with materials such as fillers or polymer modifiers (which may not cost-effectively enhance resistance to rutting).
- If a differently graded material is desired, obtain it by dropping an intermediate-sized aggregate from the design and altering the desired air voids content input to fill the void with an increased amount of the next smaller stockpile, or open the voids by decreasing that fraction or increasing a larger-sized stockpile.
- If the problem is the result of the breakdown of a weak aggregate during mixing and/or compaction, a different parent material source for the aggregate should be selected.
- The analysis procedure herein does not evaluate moisture effects. This can be done through surface-chemistry-related tests, water permeability tests with leachate analysis, and/or measurements of the degradation of material properties upon exposure to moisture (e.g., AASHTO T 283). If moisture effects are not measured, select the highest asphalt content that gives the desired indication of

long-term performance (minus the expected variability of asphalt content at the plant). If moisture effects are measured, select the lowest asphalt content that meets both moisture and performance requirements (plus the plant variability) to maximize resistance to rutting.

4. APPARATUS

4.1 Mix Design—The equipment needed for the basic tests (e.g., abrasion, sieve analysis, unit weight, and specific gravity) are given in the referenced standards. In addition, a personal computer (PC) with the Microsoft Excel for Windows computer software (Version 5.0 or higher) is required to produce the mix design (other spreadsheets that can read Excel files and can perform circular reference and optimization may be used).

4.2 Mix Analysis—SST Device for Procedures A and B

4.2.1 An SST device with the capabilities and specifications described in TP7 is required.

4.3 Mix Analysis—Uniaxial Apparatus for Procedure C

4.3.1 Axial Loading Device—The loading device shall be capable of providing a constant load of 12 kN with a resolution of 5 N or better. The device shall be closed-loop and feedback-controlled and shall be capable of applying a ramp load from 0.02 kN to 12 kN in compression within 0.75 s when the platens are loaded against each other (i.e., with no specimen in the machine).

4.3.2 Load Measuring Device—The load measuring device shall consist of an electronic load cell designed for placement between the loading platen and the actuator rod or the reaction frame, with a sensitivity of 5 N or better, and a minimum capacity of 14 kN.

4.3.3 Loading Platens—Steel loading platens shall be at least 25 mm thick and 160 mm in diameter. The face of the platens contacting the asphalt specimens shall be surfaced with a replaceable teflon sheet as illustrated in Figure 1.

4.3.4 Specimen Deformation Measurement Devices—Recommended con-

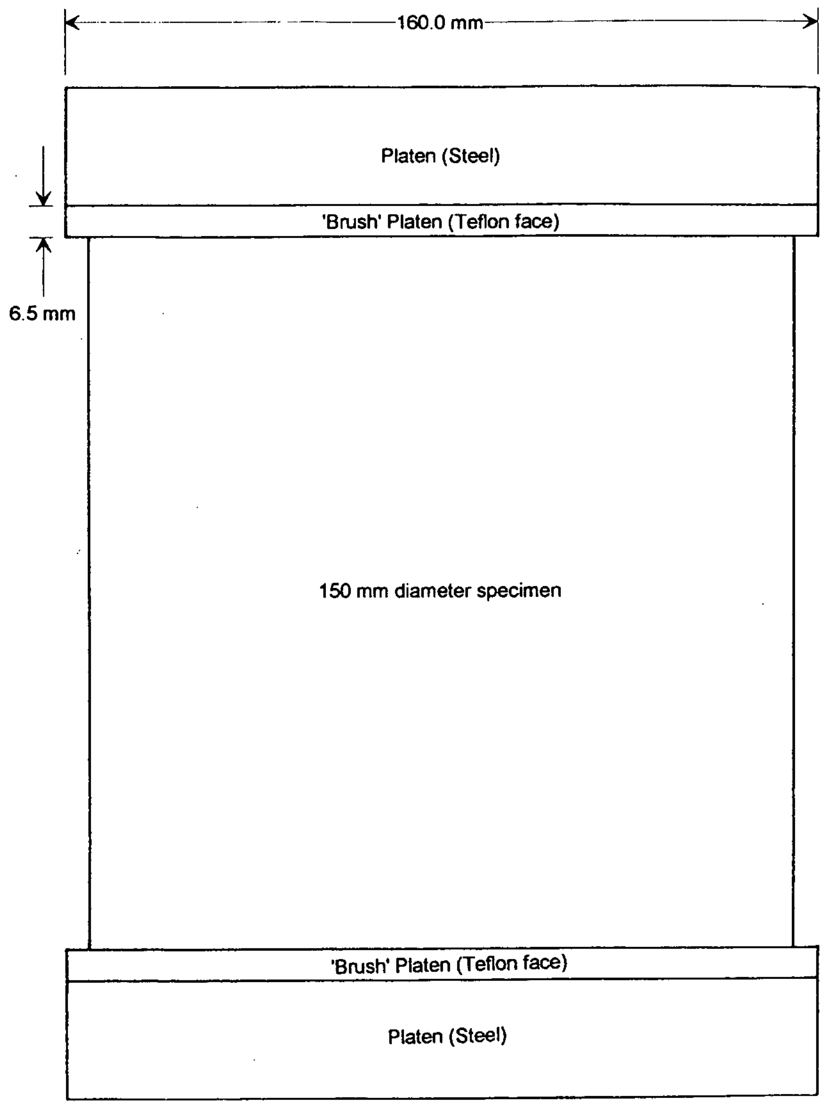

Figure 1. Procedure C assembly and platen schematic.

figurations for specimen deformation measurement are given in Figure 2. Axial measurement devices shall have a range sufficient to measure the strain that occurs during the test. The required range may be different for different temperatures and materials. A range of ± 1.5 mm with a gage length of 75 mm should be sufficient for most applications. Radial measurement devices shall have a range sufficient to measure the strain that occurs during the test. A range of ± 1.0 mm with a gage length of 75 mm should be sufficient for most applications. A resolution of 0.125 μm or better is recommended.

4.3.5 Environmental Chamber—The environmental chamber shall

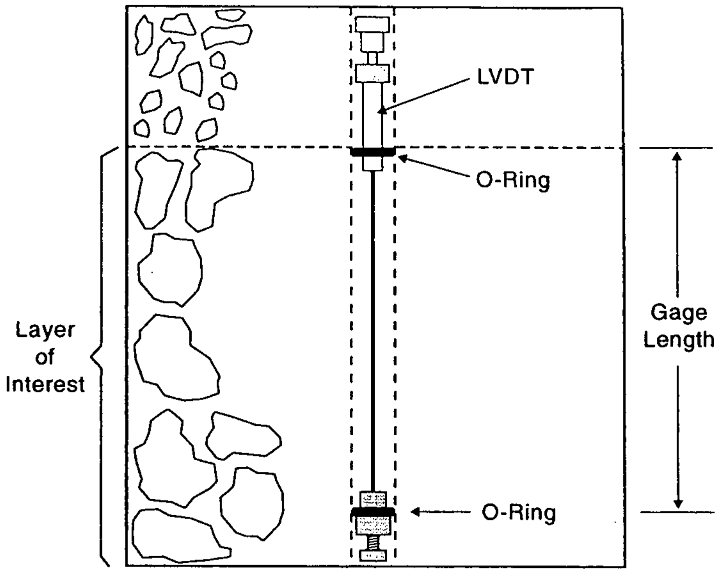

Figure 2a. Procedure C—preferred axial measurement system.

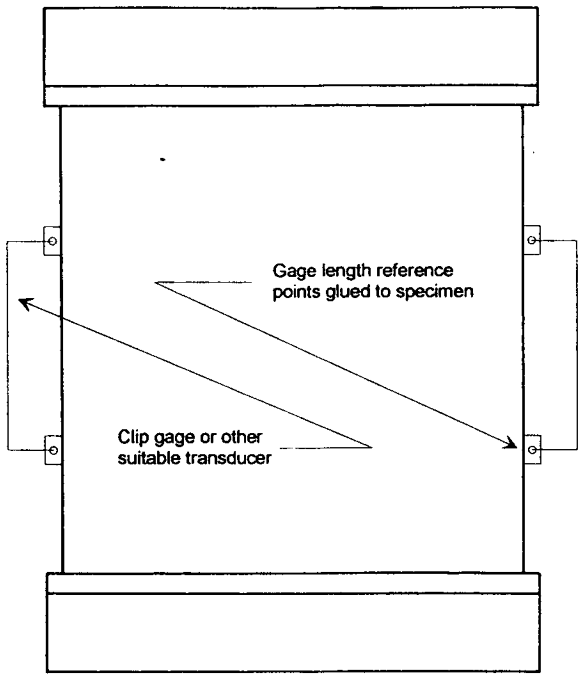

Figure 2b. Procedure C—alternate axial measurement system (surface mount).

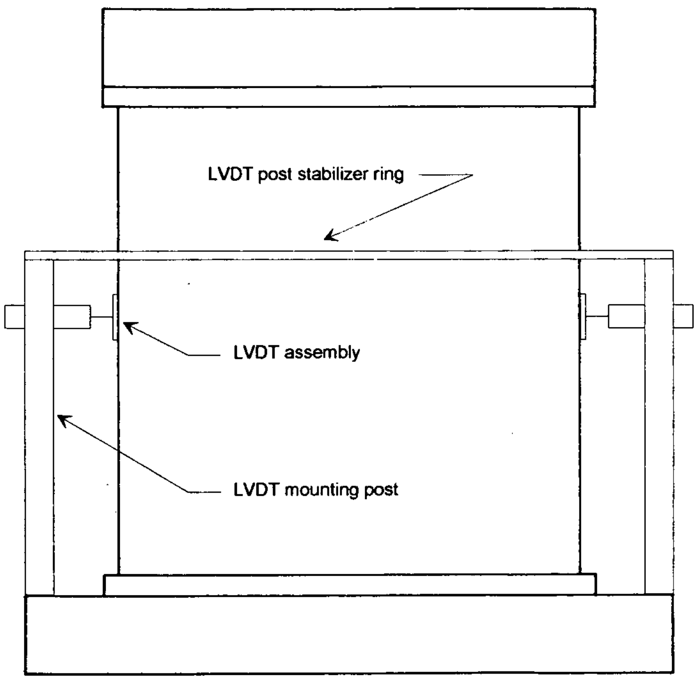

Figure 2c. Procedure C—base-plate-mounted radial measurement system (alternate).

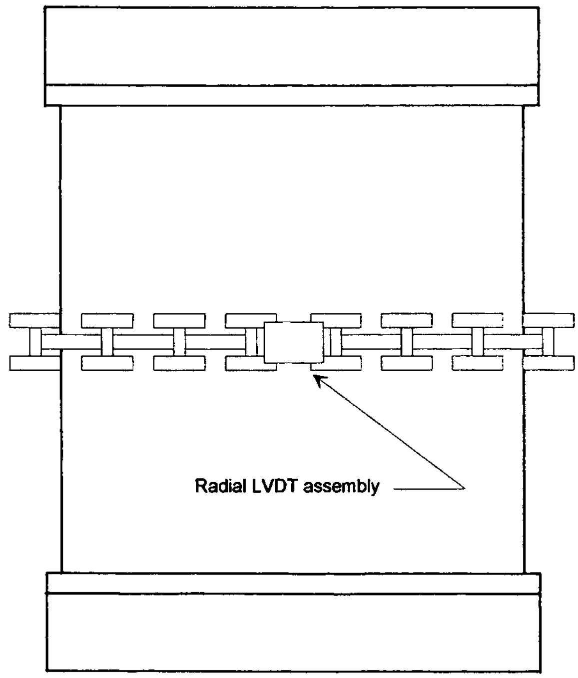

Figure 2d. Procedure C—circumferential measurement system (preferred).

maintain the desired test temperature to within ±0.5°C over the range 30 to 60°C.

4.3.6 Data Acquisition—The data acquisition system shall be capable of simultaneously sampling at least 6 channels at a minimum rate of 10 Hz with a minimum of 12-bit analog-to-digital resolution.

5. HAZARDS

5.1 Observe all safety precautions recommended by the manufacturer, as well as standard laboratory safety precautions, when operating testing equipment and preparing, testing, and disposing of HMA test specimens.

6. CALIBRATION

6.1 Testing systems shall be calibrated prior to initial use and at least once every year thereafter.

6.1.1 Verify the calibration of all measurement components and verify the capability of the environmental and specimen conditioning systems to maintain temperatures and pressures within specified limits.

6.1.2 If any of the verifications yield data that do not comply with the accuracy specifications, correct the problem prior to testing. Appropriate action may include correction of menu entries, maintenance of system components, calibration of system components (using an independent calibration agency, service by the manufacturer, or in-house resources), or replacement of system components.

7. TEST REQUIREMENTS

7.1 Procedures A, B, and C (paragraph 1.4) are optional for LSM having air voids contents of 15 percent or more that are intended for use as open-graded drainage layers. For other mixtures, the user may select any one of the three procedures, depending on time constraints and availability of equipment.

8. SAMPLING, SPECIMEN PREPARATION, AND PRELIMINARY DETERMINATIONS

8.1 Laboratory-Mixed and -Compacted Specimens—Select the binder in accordance with established performance grading procedures. Determine the temperature for mixing at 0.170 Pa·s and the compaction temperature at 0.280 Pa·s. Consideration of the use of modified binders in open-graded drainage layers to enable the use of thick films without draindown problems is recommended. Sample asphalt binder in accordance with T 40 and aggregate in accordance with T 2. Compact specimens in accordance with PP3, TP4, or paragraph 8.1.1. Specimens shall have a diameter of at least 150 mm and a height-to-diameter ratio of at least 0.97:1. The ratio of the smallest dimension of the specimen (i.e., the smaller of height or diameter) to the nominal maximum aggregate size should not be less than 2.5:1. If equipment is unavailable to produce specimens with the dimensions recommended herein, the dimensions and actual ratios shall be entered in the report and the sentence ***"Specimen dimensions do not meet the recommendations of the AASHTO standard"*** shall be prominently entered in boldface type on the report.

8.1.1 Gyratory Compaction—If desired air voids are not attainable with the procedure given in TP4, either PP3, ASTM D3387, or the following procedure may be used. Using a suitable gyratory compactor with the gyratory angle set at 5 deg, compact for 120 gryations at 30 rpm and 275 kPa. Reduce the angle to zero and apply a leveling load of 190 kPa. Adjust the number of gyrations, the vertical pressure, and/or the gyratory angle, if necessary, to obtain the desired air voids content.

Note 1: The large gyratory compactor that has been manufactured by the Texas DOT only needs slight modification to meet the intent of this standard. A gyratory compactor that has an electropneumatically adjustable angle has been manufactured by Industrial Process Controls Ltd. of Melbourne, Australia. The Corps of Engineers GTM has been manufactured by Engineering Developments Co. of Vicksburg, Mississippi. Although 5 deg is specified on the basis of the results of one research study, smaller angles, on the order of 3 to 4 deg, may be adequate, but this remains to be verified by future research efforts. If the agency has no suitable compactor, mixtures may be compacted with a modified version of the T 245 or ASTM D3202 apparatus until such equipment is acquired.

8.2 Plant-Mixed, Laboratory-Compacted Specimens—Obtain HMA sample(s) in accordance with T 168. Follow the guidance given in paragraphs 8.1 and 8.1.1 for compacting specimens.

8.3 Roadway Specimens—Obtain specimens of the necessary diameter and height from the pavement in accordance with ASTM D5361.

9. MIX DESIGN PROCEDURE

9.1 Select the binder in accordance with the Superpave binder selection process in report SHRP-A-407.

9.2 Select the stockpiles of material that will be candidates for use in the LSM.

9.2.1 Evaluate the consensus and source properties of each stockpile of material in accordance with the Superpave mix design manual (SHRP-A-407).

9.3 Sieve Analyses (material 0.75 µm and larger)

9.3.1 Perform a sieve analysis of the selected stockpiles in accordance with T 27.

9.3.2 (Optional) Take 10,000-g samples of each stockpile having a moisture content approximately equal to that expected at the outlet of the mix plant drum and run 5 min in the LA abrasion machine (T 96) *without* the steel balls (the drum must be sealed to prevent loss of fines). Dry-sieve the abraded material. This portion of the procedure is intended to simulate the action of plant-mixing and pavement construction. The change in gradation from the original to the abraded material will affect optimum asphalt content. If this portion of the procedure is performed, use this sieve analysis in the spreadsheet for the mixture design (Annex X1). If the modified LA abrasion is not performed, the user may substitute the values obtained by multiplying the percent passing figures obtained in paragraph 9.3.1 by a ratio determined as follows. The equation is valid for $S \leq 50.8$ mm (use a ratio of 1.0 for larger particles unless measurements indicate a larger value):

$$R_{lp} = 1.242(5.08 - S_{\max} + S)^{-0.133}$$

where S is the square sieve opening size in cm, $S_{\max}$ is the size of the largest sieve that has less than 100 percent passing (all sieves having 100 percent passing are arbitrarily assigned a value of $R_{lp} = 1.0$), and R_{lp} is the laboratory-to-plant adjustment ratio. If the material in the plant is not completely dried, the increased asphalt demand from degradation by the mixing plant, handling, and compaction may be offset by a symptomatic decrease in demand resulting from incomplete drying. The engineer must weigh these factors during the mix design process. If neither method in this paragraph is desired, use the sieve analysis from paragraph 9.3.1.

9.3.3 Compute the size at 50 percent passing (i.e., the D_{50} size), by linear interpolation if necessary, for each stockpile.

9.3.4 For the largest stockpile, compute the void size index (VSI) as follows:

$$VSI = (\alpha)(D_{50}^{LS})$$

where D_{50}^{LS} indicates the largest stone stockpile data and α is 0.15 for uncrushed river gravel type aggregates, 0.29 for cubical (ratio of all lengths approximately 1:1) aggregate, and 0.40 for elongated particles (length-to-width ratios of 2:1 or more). For this standard, it is adequate to determine the prevailing shape in the stockpile of largest aggregate subjectively and to use the factor corresponding to that shape (interpolation permissible) for calculations involving all other stockpiles.

9.3.5 Eliminate from further consideration any stockpiles having a D_{50} size that is larger than *VSI*. The remaining paragraphs of the design procedure are applicable only to the largest stone stockpile and the smaller stone stockpiles that are not eliminated in this paragraph.

9.4 Measure the bulk specific gravity and absorption of each stockpile using T 84 and T 85.

9.5 Measure the unit weight of the largest stone stockpile material using T 19 or paragraph 9.5.1.

9.5.1 Alternate Procedure for Unit Weight

9.5.1.1 Obtain a suitably sized container (e.g., a unit weight bucket) and a surcharge weight having sufficient mass to apply a static vertical pressure of approximately 2 kPa and follow the aggregate particles as they rearrange.

9.5.1.2 Obtain a minimum 4,500-g sample of aggregate from the largest stone stockpile.

9.5.1.3 Weigh the empty container.

9.5.1.4 Weigh the container plus the aggregate.

9.5.1.5 Vibrate the dry aggregate in the container using the surcharge to follow the material as it rearranges its particle orientation to a denser state. If a vibratory table is not available, use a tamping or jiggling procedure, or a high-capacity sieve shaker may suffice. At the end of the densification process, measure and record the height of the lid at three points 120 deg apart. Remove the lid and loosen the aggregate. Repeat the vibration and measurements two more times. Compute the volume occupied by the aggregate plus air in the densified state using the previously measured bulk specific gravity of the aggregate and the average of the nine height measurements corrected for the thickness of the lid. Compute the unit weight of the material from these measurements.

$$\mathit{Unit\ Weight} = \frac{\mathit{Aggregate\ Weight}}{\mathit{Average\ Compacted\ Volume}}$$

9.6 Select the desired air voids content and the desired range of dust-to-asphalt ratio.

9.7 Enter the data from this section of the standard on the spreadsheet, and perform the calculations as described in Annex X1.

9.8 NCHRP Level 2 LSM Design

9.8.1 Prepare specimens not intended for use as open-graded mixtures in accordance with Section 8 of this standard and in the quantities required by the selected mix analysis procedure (A, B, or C) of this standard.

9.8.2 For open-graded mixtures, perform the draindown test (PPdd).

9.8.3 Perform an initial check on the effective utilization of the largest stockpile of material and the compaction effort by computing the voids in the coarse aggregate stockpile material in the compacted asphalt mix. This computation is simply the bulk specific gravity, G_{mb}, of the compacted mix times the weight fraction of the aggregate in the total mix times the weight fraction of the large stone material. The result of this calculation is the unit weight of the material from the large stone in the total mix, and this number should ideally be greater than or equal to the measured unit weight of this material.

9.8.4 Conduct the test(s) required in Procedure A, B, or C (paragraph 1.4), in accordance with paragraphs 10, 11, or 12.

10. MIX ANALYSIS PROCEDURE A—SUPERPAVE PERFORMANCE TESTING

10.1 Prepare specimens for testing in accordance with Section 8 of this standard.

10.2 Conduct tests required for Superpave abbreviated or full mix analysis as desired using TP 7 as modified herein.

10.2.1 Test Requirements—As discussed in paragraph 10.2.1.1 below, the air voids contents specified in TP 7 are optional in this procedure. The Superpave abbreviated mix analysis tests conducted at $T_{eff}(FC)$ are also optional in this procedure.

10.2.1.1 Lab-Mixed and -Compacted Specimens—For laboratory mixes (paragraph 8.1), prepare a minimum of 23 specimens for the Superpave abbreviated mix analysis, seven each at (1) the design binder content found in Section 9 of this standard, (2) 0.8 percent above the design binder content, and (3) 0.8 percent below the design binder content, and two additional specimens at the high binder content. The total number of Superpave abbreviated mix analysis specimens may be reduced by six if the tests at T_{eff} *(FC)* will not be conducted (tests at T_{eff} *(FC)* are optional for this procedure). For a Superpave full mix analysis, prepare a minimum of 59 specimens, 19 each at (1) the design binder content found in Section 9 of this standard, (2) 0.8 percent above the design binder content, and (3) 0.8 percent below the design binder content, and two additional specimens at the high binder content. Conduct the Superpave accelerated performance tests required in TP 7 for the desired level of analysis on these specimens (the air voids content of these specimens will not necessarily be 3 percent and 7 percent as given in TP 7).

Note 2: Some LSM have very low binder contents because of the large portion of the total volume that is occupied by large stones. In some cases, the minus 0.8 percent binder content may be impractical. In these cases, the engineer should select an intermediate binder content on the low side that is closer to the design binder content or make binder content number (3) at a higher level such as 1.6 percent above the design.

10.2.1.2 Plant-Mixed or Field Cores—Sample and prepare a minimum of 9 cylinders for Superpave abbreviated mix analysis and 21 cylinders for Superpave full mix analysis. Perform the tests required in TP 7 for the desired level of analysis. Because all specimens are tested at only one condition (i.e., the in-place pavement air voids and asphalt content), use the sequence of testing given in TP 7 for the high binder content, because this will include the TP 7 Procedure C test (Repeated Shear Test at Constant Stress Ratio).

11. MIX ANALYSIS PROCEDURE B—REPEATED SHEAR AT CONSTANT HEIGHT (RSCH) TEST FOR RUT ESTIMATION

11.1 Prepare specimens for testing in accordance with Section 8 of this standard.

11.1.1 For plant-mixed and roadway specimens (paragraphs 8.2 and 8.3), prepare a minimum of three specimens.

11.1.2 For laboratory mixes (paragraph 8.1), prepare a minimum of nine specimens, three each at (1) the design binder content found in Section 9 of this standard, (2) 0.8 percent above the design binder content, and (3) 0.8 percent below the design binder content (see Note 2).

11.2 Perform tests on the specimens prepared in paragraph 11.1 in accordance with AASHTO TP 7, Procedure F.

Note 3: Difficulty handling and testing LSM at the maximum 7-day temperature for shallow depths should be expected. For LSM having nominal maximum sizes of 37.5 mm and larger, it is recommended that the maximum 7-day pavement temperature at a 20-mm depth obtained by using the mean maximum 7-day air temperature be replaced by the pavement temperature calculated in the same manner, but using the mean maximum air temperature for the hottest month of the year. If this temperature is still too high, the hottest 3 months of the year may be used. If 20 mm is not the depth of interest (e.g., for lower layers/lifts and base courses), the temperature should be computed on the basis of the appropriate depth as well. These computations are automated in the SHRP Superpave Binder Selection computer program available through the FHWA. The report shall reflect the basis on which the pavement temperature was determined for testing.

12. MIX ANALYSIS PROCEDURE C—UNIAXIAL TEST

Note 4: A test using the same specimen dimensions as used in this procedure is being evaluated as one of the candidate procedures for QC/QA in NCHRP Project 9-7. That procedure is a uniaxial frequency sweep test with a slightly different displacement measuring system. The 9-7 test should take less time to complete because it is intended for QC/QA. If the test selected in the 9-7 project is applicable to LSM, the user may substitute the 9-7 procedure for this procedure. If the 9-7 procedure is used, the report for this standard will include the statement ***"The NCHRP 9-7 procedure was used in lieu of Procedure C."***

12.1 Prepare specimens for testing in accordance with Section 8 of this standard.

12.1.1 For plant mix and roadway specimens (paragraphs 8.2 and 8.3), prepare a minimum of three specimens.

12.1.2 For laboratory mixes (paragraph 8.1), prepare a minimum of nine specimens, three each at (1) the design binder content found in Section 9 of this standard, (2) 0.8 percent above the design binder content, and (3) 0.8 percent below the design binder content (see Note 2).

12.2 Compute test temperature for the depth and period of interest as described in Note 3.

12.3 Program the loading system to apply a constant compressive load of approximately 345 ± 5 kPa to the specimen for 1 hr (3600 sec), followed by an unload period of 1 hr (3600 sec). The minimum data acquisition rate during the loaded time and the unloaded time is 1 Hz. The maximum ramp time to apply the load and to remove the load is 0.1 s.

Note 5: The applied stress should normally be between approximately 25 and 60 percent of the compressive strength at the test temperature. The 345 kPa stress has been found to be applicable to a reasonably wide range of materials and test conditions. However, the user may find it necessary to increase this stress to obtain better signal-to-noise ratios for the displacement measurement system if the temperature of the test is relatively low (e.g., in the case of materials being placed in lower pavement layers). Conversely, it may be necessary to decrease the stress for any extremely high temperature work. Simulation of tire contact stresses is permitted under this standard at the discretion of the engineer. Such simulations require determination of appropriate confining pressures, as well as axial stress. If the engineer elects to deviate from the 345-kPa stress recommended in the standard, the applied stress shall be noted in the report, and, in the case of confinement, the magnitude of the confinement and the method of estimating the major and minor principal stresses shall be documented.

12.4 Confirm that the environmental chamber and specimen are maintaining the specified test temperature ±0.5° C.

12.5 Perform the test on the specimens prepared in paragraph 12.1 in accordance with paragraph 12.3.

13. CALCULATIONS

13.1 Mix Analysis Procedures A and B

13.1.1 Perform the applicable calculations given in TP 7 and the Superpave mix design manual (SHRP-A- 407) using the currently available version of the Superpave software.

13.2 Mix Analysis Procedure C

13.2.1 Plot the axial strain versus time as shown in Figure 3. If recorded, also plot the radial strain versus time. Recall that engineering strain is equal to the change in displacement divided by the gage length.

13.2.2 Extrapolate the strain response curve(s) out to 7,200 as shown in Figure 3.

13.2.3 At 7,000 s, determine the plastic strain, $\varepsilon^p = \varepsilon_p + \varepsilon_{vp}$, and the elastic strain, $\varepsilon^e = \varepsilon_e + \varepsilon_{ve}$ as shown in Figure 3.

13.2.3.1 If radial measurements were taken, compute the ratio of the radial elastic strain, $\varepsilon^e_{\text{radial}}$, to the axial elastic strain, $\varepsilon^e_{\text{axial}}$

$$\nu = \left| \frac{\varepsilon^e_{\text{radial}}}{\varepsilon^e_{\text{axial}}} \right|$$

13.2.3.2 Compute the following ratio for the axial measurements only:

$$RR = \frac{\epsilon^e}{\epsilon^e + \epsilon^p}$$

14. REPORT

14.1 Mix Analysis Procedures A and B

14.1.1 Report applicable test results as described in the Superpave mix design manual (SHRP-A-407) as supplemented by the mandatory reporting requirements of this standard.

14.2 Mix Analysis Procedure C

14.2.1 Report the results of the calculations given in paragraph 13.2.3 and its subparagraphs, and comply with the mandatory reporting requirements of this standard. Use Table 1 to assess the acceptability of the mix.

15. PRECISION AND BIAS

15.1 Precision—The research required to develop precision estimates for the procedures described in this standard has not been conducted.

15.2 Bias—The research required to establish the bias of this standard has not been conducted.

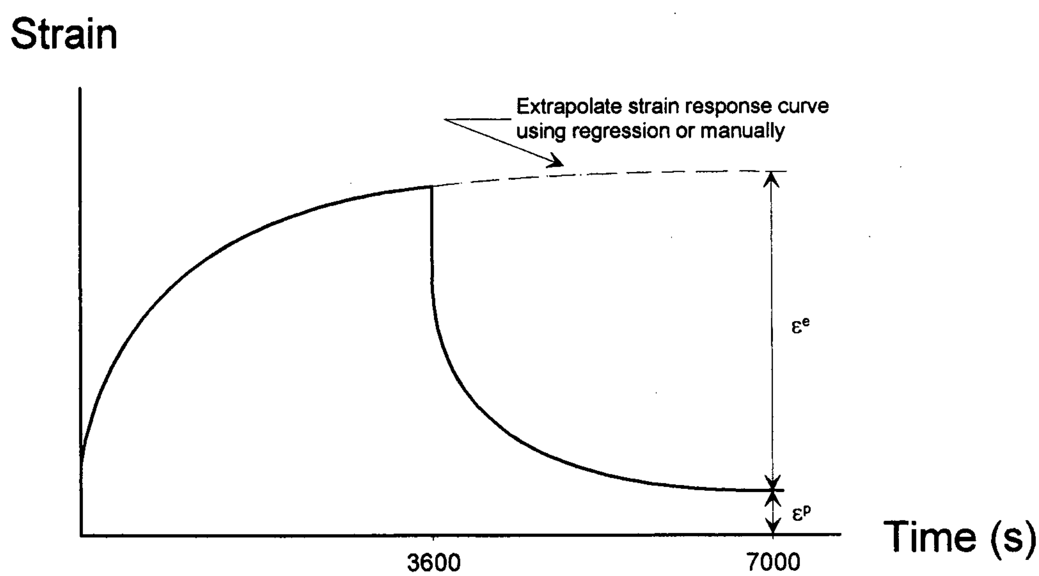

Figure 3. Illustration of curve construction for computation of RR (Procedure C).

TABLE 1 Recommended Acceptable Ranges

Parameter	Range	Acceptability
ν	>0.9	Pay < 1.0 or No Pay
ν	0.5-0.9	Pay > 1.0
ν	0.3-0.5	Pay = 1.0
ν	0-0.3	Pay < 1.0
RR	>0.8	Pay > 1.0
RR	0.5-0.8	Pay = 1.0
RR	<0.5	Pay < 1.0 or No Pay

16. KEY WORDS

Asphalt mixes, asphalt pavements, bituminous materials, large stone, shear, uniaxial creep, permanent deformation, rutting, dilation, Poisson's ratio

ANNEX X1

MIX DESIGN SOFTWARE QUICK REFERENCE

X1. COMPUTATIONAL BACKGROUND

The volume proportion of the large stone material is computed from its unit weight and specific gravity measurements (0.516 in the spreadsheet at cell *D39*). This number remains fixed during the mix design solution, thus assuming that the design is based on a unit volume defined by the unit weight of the large stone. Then, $V_{agg} - V_{aggls}$ (where V_{aggls} is the volume proportion of the large stone solids, while V_{agg} is the total volume proportion of aggregate) is the volume available to be occupied by the smaller stockpile materials.

The fundamental equations governing the computation process are shown below. In this procedure, an equation was developed from the chart illustrating the effect of maximum particle size on minimum VMA given in the Asphalt Institute's MS-2 (*1,2*), assuming that the upper bound of the tolerance band would suffice for 5 percent air voids (VTM) and adding a shift factor to adjust VMA for other air void contents. By using the formula

$$VEA = VMA - VTM$$

where *VEA* is the effective asphalt content on a volume basis, the following empirical relationship was established for estimating *VEA* (spreadsheet cell *L35*).

$$VEA = 13.261\left(\frac{S}{2.54}\right)^{-0.187} + 0.57(VTM - 5) - VTM$$

where *S* is the maximum aggregate size in centimeters, 2.54 is the conversion factor to inches, and 0.57 is a shift factor to move the curve relative to the *VTM* = 5 percent line. (Alternatively, the SHRP formula could be used (*3*), but it does not include the VTM component necessary for this design process and would need to be modified to address that issue.) During the solution process, a conversion between effective asphalt content on volume and weight bases must be made. An approximate relationship between bulk specific gravity of the mixture (G_{mb}) and asphalt content on a weight basis (P_{be}) was developed from NAPA data (*4*) for this purpose.

$$G_{mb} = -0.001P_{be}^3 + 0.0096P_{be}^2 - 0.0116P_{be} + 2.4613$$

This equation is refined by using the iterative circular reference feature of Excel to determine the effective asphalt content by weight during the solution process (cell *L25* in the spreadsheet). Excel's solver (optimization) is used to enforce the dust-to-asphalt ratio requirement basically as follows. Set the objective function (target cell *J39* on the spreadsheet) to the value 1.0

$$\sum_{agg} \beta_{agg} = 1$$

where

β_{agg} = decision variables (changing cells *E39–I39*) = stockpile proportions subject to the constraints

$$\beta_L V_{EA} \le \sum_{-200} \beta_{agg} V_{agg} \le V_{EA}$$

where

β_L and β_H are the low and high limits for the dust-to-asphalt ratio and are used to compute the constraints at each extreme (cells *K39–L39*)

and

$$\beta_{agg} \ge 0$$

for all aggregate stockpiles (cells *E39–I39*).

A final addition to the computed asphalt content is made by correcting for water absorption observed during aggregate-specific gravity measurements (cell *L26*). The film thickness has been indirectly considered in this methodology in two ways: (1) through consideration of the effect of maximum particle size on VMA and (2) by constraining the solution with a dust-to-asphalt ratio specification.

X2 EXCEL SPREADSHEET

X2.1 Two important features of the Excel software led to its selection as the design tool platform: (1) the ability to iterate to solve a "circular reference" and (2) the built-in "Solver" or "constrained optimization" capability.

X3 SPREADSHEET INPUT (FIGURE X1)

X3.1 Input on the spreadsheet is straightforward using data obtained in paragraph 9 of this standard. The maximum aggregate size is the smallest sieve size in the largest stone stockpile having 100 percent passing. The range

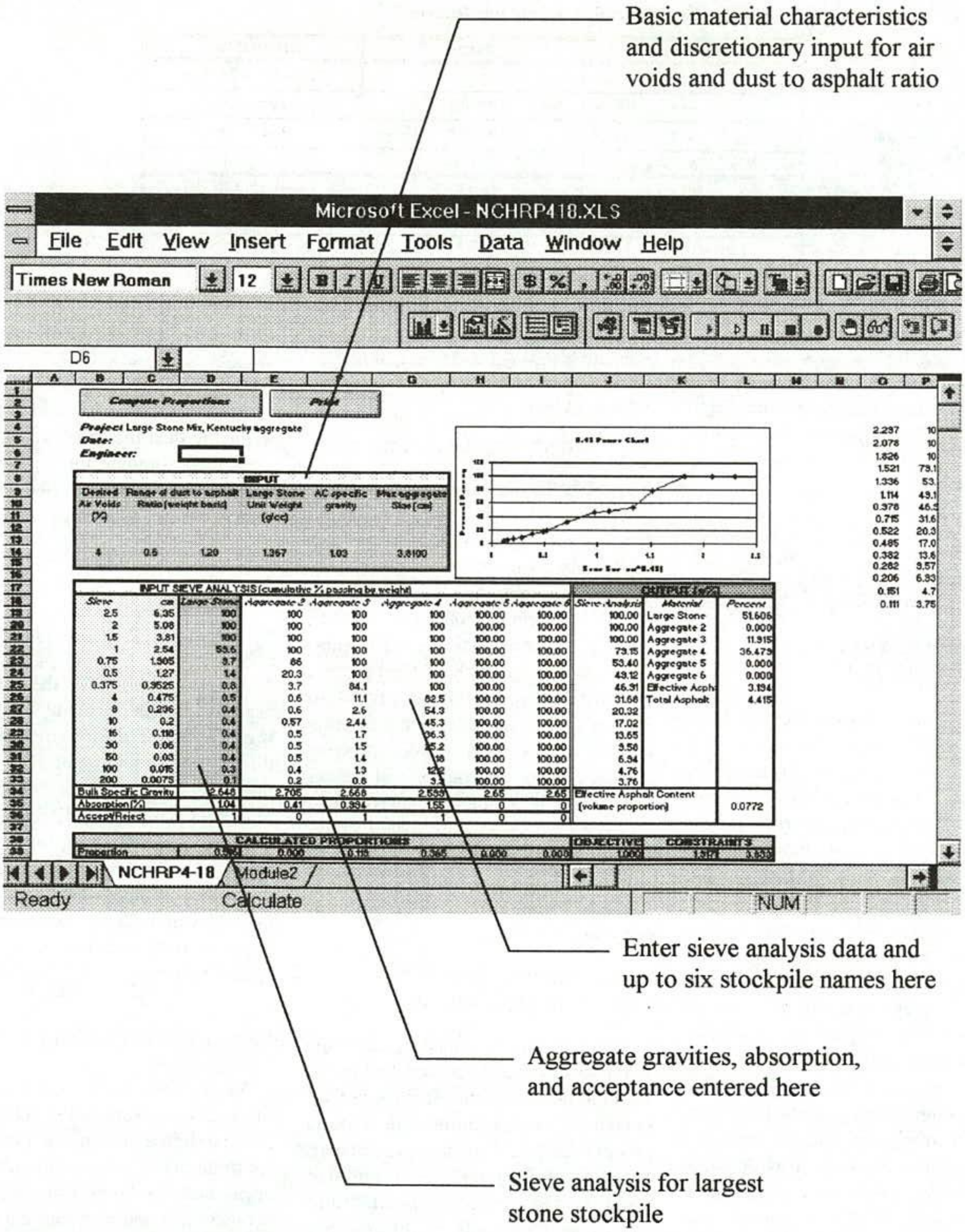

Figure X1. Spreadsheet input.

of dust-to-asphalt ratio on a weight basis recommended by the FHWA is 0.6 to 1.2. Desired air voids are normally set at 4 percent for a dense mix and 15 percent for a drainable layer or open-graded friction course. The sieve analysis data, bulk specific gravities, and absorptions are entered in the locations indicated.

X3.2 Users may change the names of the stockpiles at the top of the sieve analysis columns. If this is done, the "Output" column for the materials will automatically reflect the change.

X3.3 The "Accept/Reject" row of entries must have either a 1 (accept) or a 0 (reject). Normally, rejecting a stockpile is based either on the D_{50} size analysis that is done by hand as illustrated in paragraphs 9.3.4 and 9.3.5 of this standard or on some fac-

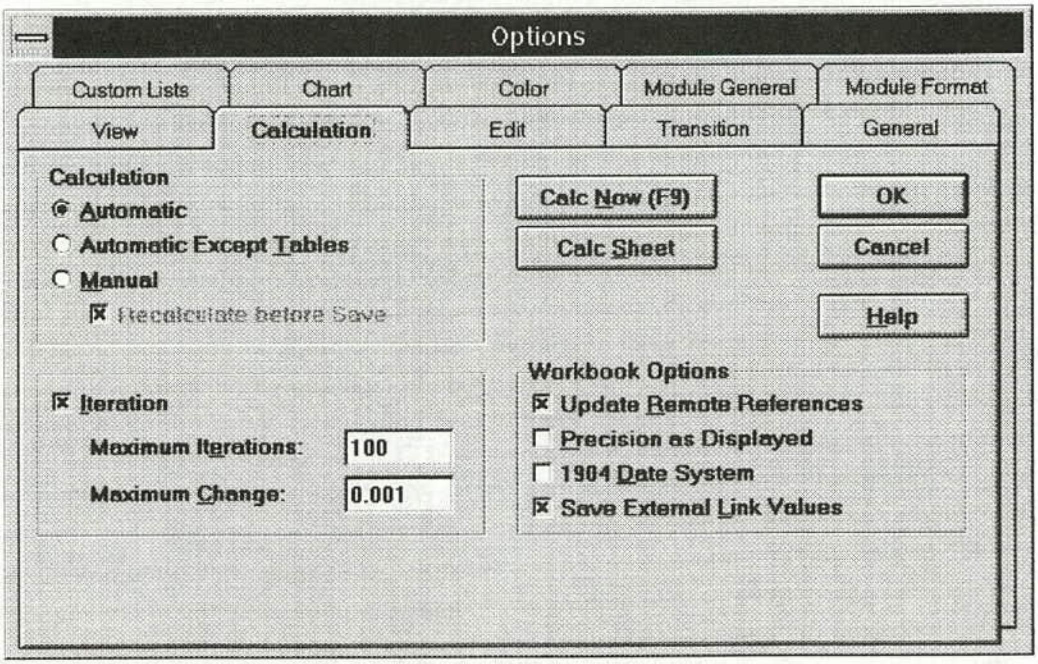

Figure X2. Ensuring that the circular reference feature is enabled.

tor such as cost or unacceptable material quality.

X4 SPREADSHEET PROCESSING

X4.1 Do not alter any of the formulas in the output or calculation portions of the spreadsheet. There are two columns of data off to the right of the main part of the sheet (not shown in the figures here) that are required for the graphic. These formulas also should not be altered.

X4.2 The Visual Basic computer code attached to the buttons (*Compute Proportions* and *Print*) on the *NCHRP 4-18* spreadsheet can be found by selecting the *Module2* spreadsheet tab. Users should not change this code unless they are familiar with Visual Basic and fully understand what they are doing.

X4.3 Manual Error Handling

X4.3.1 Two errors have been encountered on some machines. The first is a message box to the effect that a circular reference cannot be resolved. Users who encounter this error should simply select the "Tools," "Options," and "Calculation" sequence and then make the entries shown in Figure X2.

X4.3.2 The second error is a "Solver.x1a" file not found error. In this case, press the Esc key multiple times until the program releases control back to the keyboard and then go into the "Tools" and "Solver" sequence (Figure X3).

X4.4 When Solver has completed the solution process, the pop-up window shown in Figure X4 will be displayed.

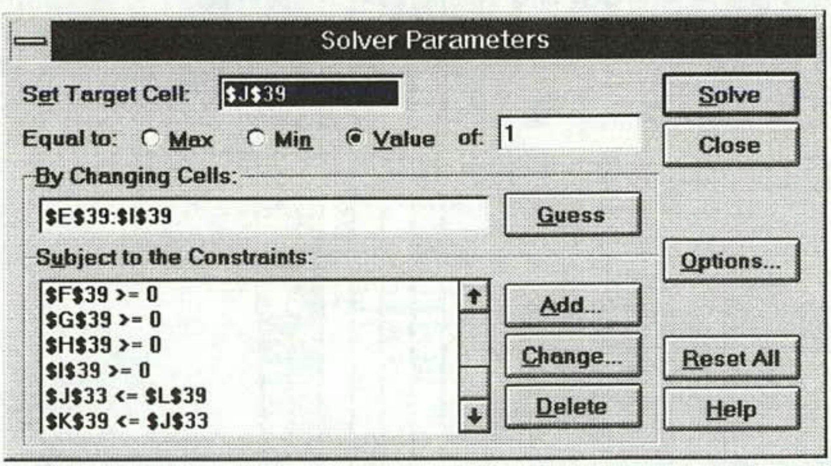

Figure X3. Pulling in the solver add-in through the tools menu item instead of through the compute proportions button.

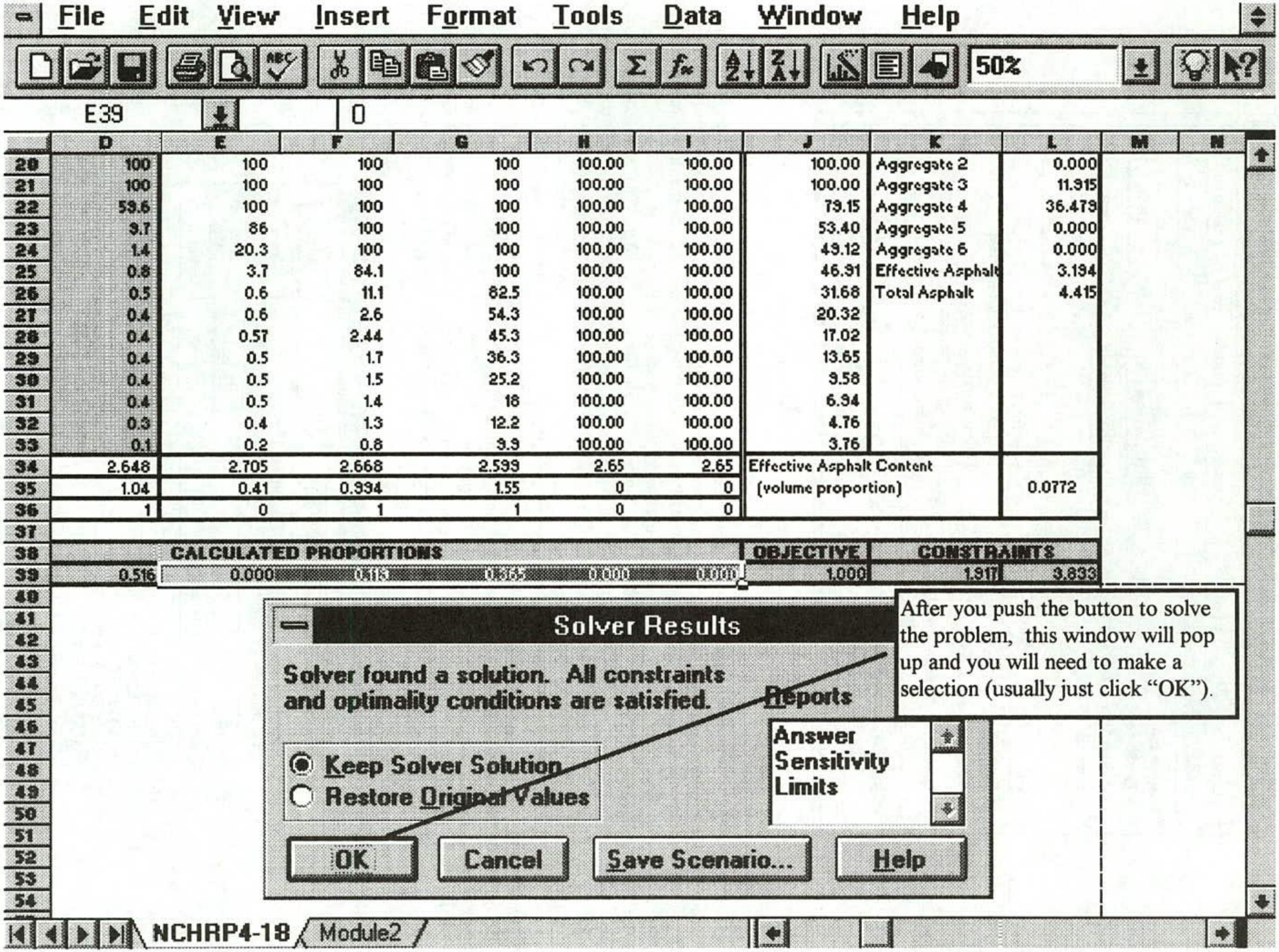

Figure X4. Solver pop-up item.

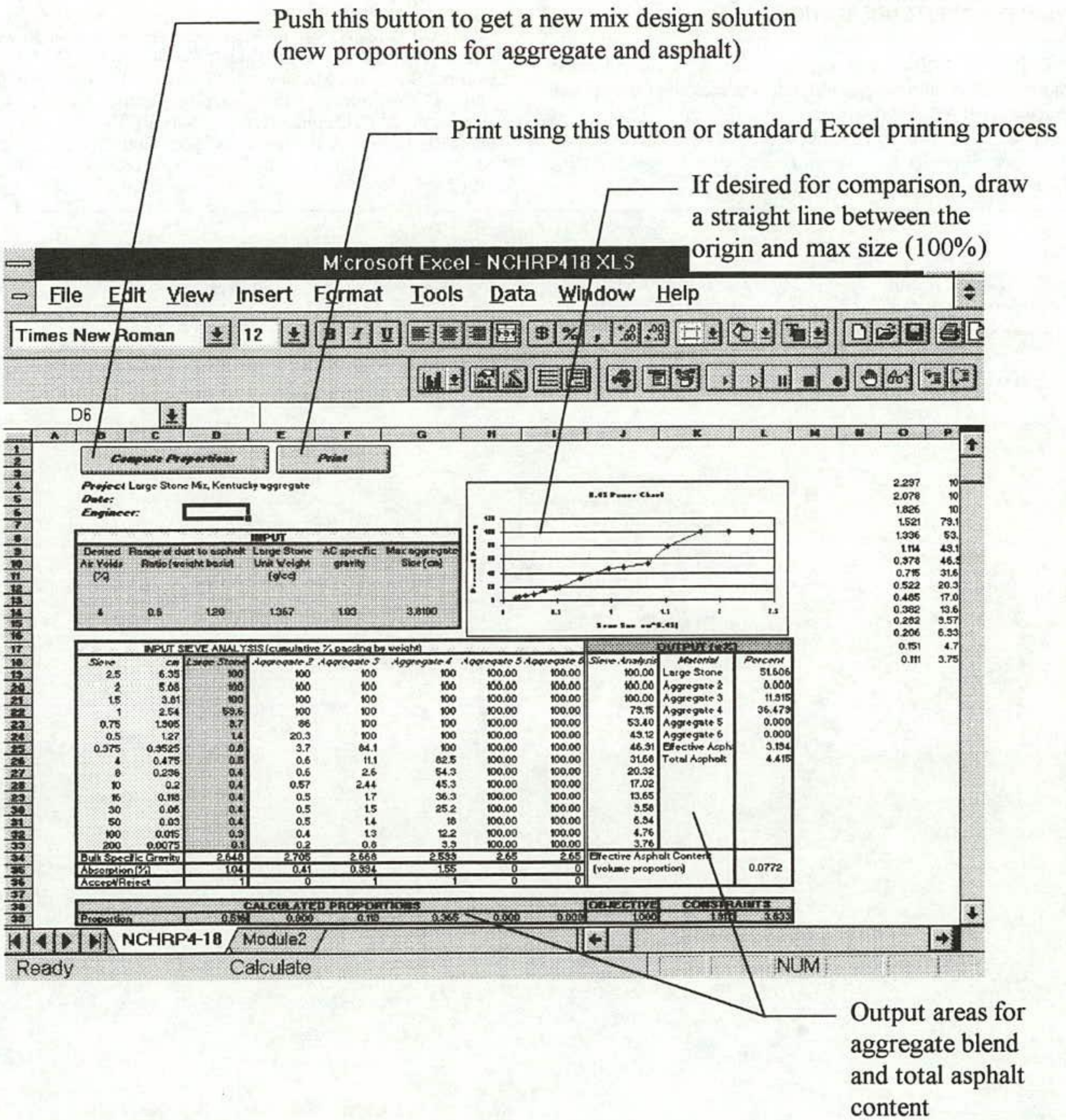

Figure X5. Output.

Click on "OK" and the solution will be displayed as shown in Figure X5.

X4.5 The spreadsheet may now be printed using either the Print button or the normal Excel toolbar or menu selections as desired.

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CHAPTER 3

LSM DESIGN AND ANALYSIS PROCEDURES: SUPPORTING TEST METHODS

Three new test methods were developed in this project to support the LSM design and analysis method in Chapter 2. This chapter presents these methods in standard AASHTO format.

3.1 STANDARD METHOD FOR DETERMINING THE DRAINDOWN CHARACTERISTICS OF LARGE-STONE MIXES (LSM) FOR HOT-MIX ASPHALT (HMA)—AASHTO DESIGNATION PPDD-96

A reommended test method to determine the draindown characteristics of open-graded LSM designs is provided on pages 23 and 24. The method was adapted from a method developed by the National Center for Asphalt Technology (NCAT) to measure the draindown characteristics of SMA mix designs. Although practitioners agree that some draindown is desirable for any open-graded mix, excessive draindown is costly and usually detrimental to pavement performance. Substantial draindown during storage or hauling of the HMA can result in fat and lean spots in the pavement as well as subsequent bleeding, ravelling, or rutting. Continued draindown in the compacted mat will lead to loss of strength and pavement integrity.

3.2 STANDARD METHOD FOR MEASURING AIR VOIDS CONTENT OF WATER-PERMEABLE, COMPACTED ASPHALT MIXES USING GLASS BEADS—AASHTO DESIGNATION PPBB-96

The typical size of the air voids in dense and open-graded LSMs is larger than those in conventional paving mixes with the same air voids content (VTM). Voids of this size are readily permeable to water. As a result, application of standard methods for measuring the bulk specific gravity of conventional paving mixes (AASHTO T 166) required for the calculation of the air voids content may give erroneously high values and, in turn, erroneously low values of VTM when applied to LSMs. An improved method of measuring the bulk specific gravity of LSMs developed in this project is provided starting on page 25. The method is based on AASHTO T 166, but uses 8 mm-diameter glass beads instead of water to determine the mass displacement of the LSM specimen.

3.3 STANDARD METHOD FOR DETERMINING DEGREE OF STONE-ON-STONE CONTACT OF LARGE-STONE MIXES (LSM) FOR HOT-MIX ASPHALT (HMA)—AASHTO DESIGNATION PPCC-96

An analysis of LSM field sections that failed to provide better rutting resistance than conventional paving mixes suggested that a key, common shortcoming was poor *stone-on-stone contact* in the coarse aggregate skeleton. Several existing test methods to quantify this characteristic during the LSM mix design process were evaluated; these had been developed to aid in SMA mix design where good stone-on-stone contact in the coarse aggregate skeleton is also crucial. The method developed in this project is provided starting on page 29. The method is based on a procedure originally developed by NCAT for SMA mix design. Adequate stone-on-stone contact is defined as the point at which the density of the coarse aggregate[4] in a compacted LSM is equal to or greater than 80 percent of the dry rodded density of the coarse aggregate according to AASHTO T 19.

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[4]For LSMs with a maximum aggregate size from 25 to 38 mm (1.0 to 1.5 in.), the coarse aggregate is that retained on the 12.5-mm (0.5-in.) sieve; for LSMs with a maximum aggregate size from 38 to 64 mm (1.5 to 2.5 in.), the coarse aggregate is that retained on the 19-mm (.75-in.) sieve.

Standard Method
for

Determining the Draindown Characteristics of Large-Stone Mix (LSM) for Hot-Mix Asphalt (HMA)

AASHTO DESIGNATION: PPdd-96

1. SCOPE

1.1 This test method covers the determination of the amount of draindown in a LSM sample when it is held at elevated temperatures comparable to those encountered during the production, storage, transport, and placement of the mix. The test is particularly applicable to LSM used in drainable base layers.

1.2 The values stated in gram-centimeter units are to be regarded as the standard.

1.3 This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

2. REFERENCED DOCUMENTS

2.1 *ASTM Standards:*

D 1559 Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus

E 11 Specifications for Wire-Cloth Sieves for Testing Purposes

3. DEFINITIONS

Draindown—For the purpose of this test method, draindown is considered to be that portion of material that separates itself from the bulk sample and is deposited outside the wire basket during the test. The material that drains may be either asphalt binder or a combination of asphalt cement and fine aggregate.

4. SUMMARY OF METHOD

A sample of the LSM to be tested is prepared in the laboratory or obtained from field production. The sample is placed in a wire basket positioned on a pre-weighed aluminum or paper plate. The sample, basket, and plate are placed in a forced-air oven for 1 hr at a pre-selected temperature. At the end of 1 hr, the basket containing the sample is removed from the oven, along with the aluminum or paper plate, and the plate is weighed to determine the amount of draindown that occurred.

5. SIGNIFICANCE AND USE

This test method can be used to estimate if the amount of draindown expected for a given LSM is within acceptable levels.

6. APPARATUS

6.1 *Oven*—An oven capable of maintaining the temperature in a range from 120 to 175°C (250 to 350°F). The oven should maintain the set temperature to within ±2°C (±3.6°F).

6.2 *Plates*—Aluminum or paper plates of appropriate size.

6.3 *Wire Basket*—Cylindrical wire mesh basket 8 in. (203 mm) in diameter and about 10 in. (254 mm) in height. The basket should be supported by legs or the wire mesh should extend below the bottom about 1 to 2 in. (25 to 51 mm). The basket should be constructed using standard 0.635 cm (0.25 in.) wire mesh.

6.4 *Sample Mixing Apparatus*—Suitable equipment is required for mixing the aggregate and the asphalt binder. Hand mixing is permissible, but mechanical mixing is recommended.

6.5 *Ancillary Equipment*—Thermometers, spatulas, trowels, and bowls as needed.

7. SAMPLE PREPARATION

7.1 *Number of Samples*—For each mixture tested, the draindown characteristics should be estimated at three different temperatures. The three temperatures should be the anticipated production temperature and 15°C above and 15° below that temperature. For each temperature, duplicate samples shall be tested. Thus, for one LSM, a minimum of six samples shall be tested.

7.2 Laboratory-Prepared Samples.

7.2.1 Dry the aggregate in an oven at a temperature of 110°C ±5°C (230°F ±9°F) until dry.

7.2.2 Sieve the aggregate into appropriate size fractions as indicated in ASTM D 1559, Section 4.2.

7.2.3 Determine the temperature for mixing at an asphalt binder viscosity of 0.170 Pa·s in accordance with ASTM D 1559, Section 4.3.1. For modified asphalt binders, consult the manufacturer or supplier for the recommended mixing temperature or viscosity.

7.2.4 Weigh into separate pans, for each test sample, the amount of each size fraction required to produce completed mix samples weighing 1200 grams. The aggregate fractions shall be combined such that the resulting aggregate blend has the same gradations as the job-mix-formula. Place the samples in an oven and heat to a temperature not to exceed the mixing temperature established in 7.2.3 by more than approximately 28°C (50°F).

Note 1: Some modifiers, such as fibers and some polymers, must be added directly to the aggregate prior to mixing with the asphalt cement. Other modifiers must be added directly to the asphalt cement prior to blending with the aggregate.

7.2.5 Heat the asphalt binder to the temperature established in 7.2.3.

7.2.6 Place the heated aggregate in the mixing bowl. Add any modifiers (Note 1) and mix the dry components thoroughly. Form a crater in the aggregate blend and weigh in the required amount of asphalt binder. The amount of asphalt binder shall be such that the final sample has the same asphalt content as the job-mix-formula. At this point, the temperature of the aggregate and asphalt binder shall be within the limits of the mixing temperature established in 7.2.3. Using a spatula (if mixing by hand) or a mixer, mix the aggregate (and modifier if any) and asphalt binder quickly until the aggregate is thoroughly coated.

7.3 Plant-Produced Samples.

7.3.1 Samples may be obtained during plant production by sampling the mix at any appropriate location, such as the trucks prior to the mix leaving the plant or at the paver. Samples obtained during actual production shall be reduced to the proper test sample size by the quartering method.

8. PROCEDURE

8.1 Transfer the laboratory-produced or plant-produced samples to the wire basket described in 6.3.

8.2 Pre-weigh an aluminum or paper plate and record the weight. Place the basket on the plate and place the assembly into the oven at the required temperature for 1 hr.

8.3 After the samples have been in the oven for 1 hr, remove the basket and plate. Weigh the aluminum or paper plate and record this weight.

9. CALCULATIONS

9.1 Calculate the percent of mix that drained by subtracting the initial weight of the plate from the final weight of the plate and divide this value by the initial total sample weight. Multiply the result by 100 to obtain a percentage.

10. REPORT

10.1 Report the average percent drainage at each of the test temperatures.

Standard Method
for

Measuring Air Voids Content of Water-Permeable, Compacted HMA Mixes Using Glass Beads

AASHTO DESIGNATION: PPbb-96

1. SCOPE

1.1 This method of test covers determination of bulk specific gravity and air voids content of compacted asphalt mixes containing water-permeable air voids using glass beads in place of water.

1.2 The bulk specific gravity of the compacted HMA mixes may be used in calculating the unit weight and air voids content of the mixture.

1.3 *This practice may involve hazardous materials, operations, and equipment. It does not purport to address all the safety problems associated with its use. It is the responsibility of whoever uses this practice to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to its use.*

2. REFERENCED DOCUMENTS

2.1 AASHTO T 19 (ASTM C 29), Unit Weight and Voids in Aggregate

2.2 AASHTO T 166, Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface-Dry Specimens

3. SIGNIFICANCE AND USE

This method should be used with specimens that contain open or interconnecting voids, absorb more than 2 percent water by volume, or both, as determined by AASHTO T 166. These include open-graded, large-stone, and drainable-base mixes.

4. TEST SPECIMENS

4.1 Test specimens may be either laboratory-molded asphalt paving mixes or cores from asphalt pavements. The mixes may be surface or wearing course, binder or leveling course, or hot mix base.

4.2 Size of Specimens—It is recommended (1) that the diameter of cylindrically molded or cored specimens or the length of the sides of sawed specimens be at least 4 times the maximum size of the largest aggregate and (2) that the thickness of specimens be at least 1.5 times the maximum size of the largest aggregate.

5. APPARATUS

5.1 *Unit Weight Measure*—A cylindrical metal measure, preferably provided with handles. It shall be watertight, with the top and bottom true and even, and sufficiently rigid to retain its form under rough usage. The top rim shall be smooth and plane within 0.01 in. (0.25 mm) and shall be parallel to the bottom within 0.5° (Note 1). For cylindrical specimens up to 6 inches (152 mm) in diameter and 6 inches in height, a ½ cubic foot (0.014 m^3) measure should be used; for larger specimens, a 1 cubic foot (0.028 m^3)measure should be used.

Note 1: The top rim is satisfactorily plane if a 0.01 inch (0.25 mm) feeler gage cannot be inserted between the rim and a piece of ¼ inch (6 mm) or thicker plate glass laid over the measure. The top and bottom are satisfactorily parallel if the difference in slopes between pieces of plate glass in contact with the top and bottom does not exceed 0.87 percent in any direction.

5.2 *Cone*—A metal cone 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. A prototype measure and cone is shown in Figure 4.

5.3 *Balance*—A balance of at least 50 kg capacity shall conform to the requirements of AASHTO M 231 for the class of general purpose balance required for the principal sample weight of the sample being tested.

5.4 *Glass Beads*—Round, 8 mm-diameter glass beads shall be used.

5.5 *Pan*—A large shallow pan in which the measure can be set and which can catch any overflow of glass beads.

5.6 *Rubber Mallet*—A 16-oz. (0.45-kg) rubber mallet for tapping the sides of the measure to consolidate the glass beads.

5.7 *Scoop*—A scoop of convenient size for filling the unit weight measure with glass beads.

5.8 *Calibration Equipment*—A piece of plate glass, preferably at least ¼-in. (6-mm) thick and at least 1 in. (25 mm) larger than the diameter of the measure to be calibrated. A supply of water pump or chassis grease that can be placed on the rim of the container to prevent leakage.

6. CALIBRATION OF MEASURE

6.1 Determine the volume of the unit weight measure with cone sealed in place using the following steps.

6.1.1 Seal cone inverted onto the top of the unit weight measure.

6.1.2 Record weight of unit weight measure plus cone.

Figure 4. Unit Weight Measure, Prototype Cone, and Glass Beads.

6.1.3 Fill apparatus with water to level with upper (small) end of cone.

6.1.4 Record weight of unit weight measure plus cone plus water.

6.1.5 Compute volume of measure/cone apparatus: $C = (F-B)/D$

where

- B = Weight of measure + cone, lb (kg),
- F = Weight of measure + cone + water, lb (kg),
- D = Density of water, lb/ft^3 (kg/m^3), and
- C = Volume of measure + cone, ft^3 (m^3).

6.1.6 Remove water from measure/cone apparatus and dry thoroughly.

6.2 Determine weight of unit weight measure plus cone plus beads:

6.2.1 Place the measure/cone apparatus in the large shallow pan. Fill measure about ⅓ full of beads and tap measure with rubber mallet at four locations equally spaced around the circumference with five blows per location.

6.2.2 Fill apparatus about ⅔ full with beads and tap measure with rubber mallet at four locations equally spaced around the circumference with five blows per location.

6.2.3 Fill apparatus to overflowing with beads and tap measure with rubber mallet at four locations equally spaced around the circumference with five blows per location.

6.2.4 Add beads necessary to fill apparatus level full with top (small end) of cone. Tap cone lightly around circumference and again fill with beads.

6.2.5 Use a straight edge to level beads with top of measure so that any portion of the beads above the level of the measure balances the voids on the surface below the top of the measure.

6.2.6 Record the weight of the measure plus the cone plus the beads.

6.3 Determine the specific gravity of the glass beads:

Use the following equation to determine specific gravity of the glass beads.

$$E = [(A - B)/C]/D$$

where

A = Weight of measure + cone + beads, lb (kg),

E = Specific gravity of beads.

7. PROCEDURE

7.1 Determine bulk specific gravity of compacted asphalt specimen using the following steps.

7.1.1 Record weight of asphalt specimen in air.

7.1.2 Place 2 to 3 in. (50–76 mm) of beads in the bottom of the measure.

7.1.3 Place specimen in center measure and resting on the beads; twist specimen to seat in beads.

7.1.4 Fill the measure with beads to the top of the specimen. Then tap the measure with the rubber mallet at four locations equally spaced around the circumference with five blows per location.

7.1.5 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. Use the procedure in paragraphs 5.2.2 through 5.2.5 to level the glass beads with the top of the measure.

7.1.6 Record the weight of the measure plus cone plus beads plus specimen.

7.1.7 Use the following equation to calculate bulk specific gravity of compacted asphalt specimen:

$$F = (G * E)/(G + A - H)$$

where

A = Weight of measure plus beads,

E = Specific gravity of beads,

F = Bulk specific gravity,

G = Weight of specimen in air, and

H = Weight of specimen + beads + measure + cone.

7.2 Calculate percent air voids in the compacted specimen using the following equation:

$$J = \left[1 - \frac{F}{K}\right] \times 100$$

where

F = Bulk specific gravity of compacted asphalt specimen

J = Percent air voids in the specimen

K = Maximum (Rice) specific gravity of specimen

Note 2: Glass beads should be maintained reasonably free of contamination from asphalt specimens and dust that might significantly alter their specific gravity. The specific gravity of the beads should be measured often as degradation of beads and unavoidable contamination will increase their specific gravity.

Note 3: A foam rubber pad placed in the bottom of the large pan in which the measure is set will help reduce degradation of beads by crushing due to the heavy 0.5 ft^3 (0.014 m^3) measure.

Standard Method
for

Determining Degree of Stone-on-Stone Contact of Large Stone Mixtures (LSM) for Hot-Mix Asphalt (HMA)

AASHTO DESIGNATION: PPcc-96

1. SCOPE

1.1 This method of test covers the determination of degree of stone-on-stone contact of compacted large stone mixes (LSM) for hot mix asphalt (HMA) pavements.

1.2 LSM are normally defined as those containing aggregate with a nominal maximum aggregate size of 1 to 2.5 in. (25 to 64 mm). For LSM with a maximum aggregate size from 1.0 to 1.5 in. (25 mm to 38 mm), the coarse portion is defined herein as that retained on the 0.5-in. (12.5-mm) sieve. For LSM with a maximum aggregate size from 1.5 to 2.5 in. (38 mm to 64 mm), the coarse portion is defined herein as that retained on the 0.75 in. (19-mm) sieve.

1.3 *This practice may involve hazardous materials, operations, and equipment. It does not purport to address all the safety problems associated with its use. It is the responsibility of whoever uses this practice to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to its use.*

2. REFERENCED DOCUMENTS

AASHTO T 19 (ASTM C 29), Unit Weight and Voids in Aggregate

3. SIGNIFICANCE AND USE

3.1 A higher degree of stone-on-stone contact is usually associated with greater resistance to permanent deformation in compacted mixes which translates to reduced rutting and shoving in HMA pavements.

3.2 Stone-on-stone contact is defined as adequate when as the calculated density of the coarse stones (as defined in paragraph 1.2) in a compacted LSM is equal to or greater than 80% of the measured density of the coarse stones as determined from the dry rodded density test (AASHTO T 19 or ASTM D 29).

4. APPARATUS

4.1 *Measure*—A cylindrical metal measure, preferably provided with handles. It shall be watertight, with the top and bottom true and even, and sufficiently rigid to retain its form under rough usage. The measure should have a height approximately equal to the diameter, but in no case shall the height be less than 80 percent nor more than 150 percent of the diameter. The capacity of the measure should be 1 cubic foot (0.028 m^3). The thickness of the metal in the measure shall be as described in Table 2. The top rim shall be smooth and plane within 0.01 inch (0.25 mm) and parallel to the bottom within 0.5° (Note 1).

Note 1: The top rim is satisfactorily plane if a 0.01 in. (0.25 mm) feeler gage cannot be inserted between the rim and a piece of 25-in. (6 mm) or thicker plate glass laid over the measure. The top and bottom are satisfactorily parallel if the difference in slope between pieces of plate glass in contact with the top and bottom does not exceed 0.87 percent in any direction.

4.2 *Tamping Rod*—A round, straight steel rod, ⅝ in. (16 mm) in diameter and approximately 24 in. (600 mm) in length, having one end rounded to a hemispherical tip of the same diameter as the rod.

4.3 *Shovel or Scoop*—A shovel or scoop of convenient size for filling the measure with aggregate.

4.4 *Calibration Equipment*—A piece of plate glass, preferably at least .25 in. (6 mm) thick and at least 1 in. (25 mm) larger than the diameter of the measure to be calibrated. A supply of water pump or chassis grease that can be placed on the rim of the container to prevent leakage.

5. CALIBRATION OF MEASURE

Calibrate measure in accordance with AASHTO T 19 (ASTM C 29)

6. PROCEDURE

6.1 Prepare a sample of the LSM coarse aggregate (as defined in paragraph 1.2) to be tested that is 1.25 to 2 times the quantity required to fill a 1-cubic-foot (0.028-m^3) standard measure.

6.2 Dry the aggregate sample to a constant mass in an oven at 230°F ± 9°F (110°C ± 5°C).

6.3 Perform dry-rodded density test in accordance with AASHTO T 19 (ASTM C 29). Fill the measure one-third full and level the surface with fingers. Rod the layer of aggregate with 25 strokes of the tamping rod evenly distributed over the surface. Fill the measure two-thirds full and again level the surface and rod as above. Finally, fill the measure to overflowing and rod again in the manner previously mentioned. Level the surface of the aggregate with fingers or a straight edge so that any projections of the larger pieces of the coarse aggregate approximately balance the larger voids in the surface below the top edge of the measure.

Note 2: In rodding the larger sizes of coarse aggregates, it may not be possible to fully penetrate the layer being consolidated, especially when angular aggregates are used. However, the intent of the procedure should be accomplished if vigorous effort is used.

Note 3: Alternatively, the jigging procedure described in paragraph 11 of AASHTO T 19 (ASTM C 29) can be used to consolidate the stones, or a vibrating table, if available, may be used.

6.4 In rodding the first layer, do not allow the rod to strike the bottom of the measure forcibly. In rodding the second and third layers, use vigorous effort, but no more force than necessary to cause the tamping rod to penetrate to the previous layer of aggregate.

6.5 Determine the mass of the measure plus its contents, and the mass of the measure alone, and record the values.

6.6 Calculate the density of the coarse aggregate by the following equation:

$$D_{ca} = \frac{(A - B)}{C}$$

where

- D_{ca} is the density of the coarse aggregate g/l(lb/ft^3);
- A is the mass of the coarse aggregate plus the measure;
- B is the mass of the measure g(lb); and
- C is the volume of the measure l(ft^3).

6.7 Calculate the voids content of the coarse aggregate by the following equation:

$$VCA = \left[\frac{(G_{ca} * d_w) - D_{ca}}{(G_{ca} * d_w)}\right] * 100$$

where

- VCA is the voids content of the coarse aggregate;
- G_{ca} is the specific gravity of the coarse aggregate;
- d_w is the density of water g/l (lb/ft^3); and
- D_{ca} is the density of the coarse aggregate g/l (lb/ft^3).

6.8 To ensure adequate stone-on-stone contact in the LSM, the density of the coarse aggregate in the compacted LSM (or equivalently, the voids in the coarse aggregate [VCA] in the compacted LSM) should be equal to or greater than 80 percent of that determined for the coarse aggregate by the dry-rodded density test in paragraph 6.3. The density of the coarse aggregate in the compacted LSM is calculated by the following equation:

$$D_{cm} = (G_{mb} * d_w) * (1 - AC) * R$$

where

- D_{cm} is the density of the coarse aggregate in the compacted LSM g/l (lb/ft^3);
- G_{mb} is the bulk specific gravity of the compacted LSM;
- d_w is the density of water g/l (lb/ft^3);
- AC is the asphalt binder content as a weight percent of the total LSM, expressed as its decimal equivalent (e.g., 0.035 for a binder content of 3.5% by weight of the total mix); and
- R is the percent of coarse aggregate, expressed as its decimal equivalent, in the LSM gradation retained on the .5-in. (12.5-mm) sieve (maximum aggregate size from 1 to 1.5 in. [25 to 38 mm]) or on the 0.75-in. (19-mm) sieve (maximum aggregate size from 1.5 to 2.5 in. [38 to 64 mm]).

The degree of stone-on-stone contact is then calculated by the following equation:

$$SSC = \left(\frac{D_{cm}}{D_{ca}}\right) * 100$$

where

- SSC is the degree of stone-on-stone contact in the compacted LSM expressed as a percent;
- D_{cm} is the density of the coarse aggregate in the compacted LSM g/l (lb/ft^3); and
- D_{ca} is the density of the coarse aggregate g/l (lb/ft^3).

Note 4: The calculations shown in paragraph 6.8 can be carried out in exactly the same way using the VCA in place of the densities.

Note 5: The following sample calculation demonstrates the application of the equations in paragraph 6.8:

- Nominal maximum size of aggregate = 2 in.
- Density of coarse aggregate from AASHTO T 19 = 1700 g/l
- Asphalt content of LSM = 3.5%
- % aggregate retained on the ¾-in. sieve (for maximum aggregate size = 2 in.) = 60%
- Bulk specific gravity of compacted LSM = 2.500

Then the density of the coarse aggregate in the compacted LSM is obtained as

$$\begin{aligned} D_{cm} &= (2.5 * 1000g/l) * \\ &\quad (1 - 0.035) * (.6) \\ &= 1448g/l \end{aligned}$$

and the degree of stone-on-stone contact is

$$SSC = \left(\frac{1448}{1700}\right) * 100 = 85\%$$

Therefore, the LSM in this example demonstrates adequate stone-on-stone contact.

CHAPTER 4

GUIDELINES FOR HMA PAVEMENT CONSTRUCTION WITH LSM

4.1 INTRODUCTION

The purpose of this manual is to identify and discuss potential problems in the production, placement, and compaction of LSM for HMA pavements and to provide practical solutions to those problems. It is intended for use by personnel with responsibility for construction and inspection of LSM pavements, including the development of QC and QA plans.

LSM are defined as HMA paving mixes containing maximum aggregate sizes between 25 and 63 mm (1.0 and 2.5 in.). LSM can include dense-graded, open-graded, or gap-graded aggregate blends.

At a minimum, LSM construction must adhere to the same principles employed for the proper production, placement, and compaction of conventional HMA paving mixes. In addition, construction with LSM can increase the likelihood or severity of problems found with the use of conventional mixes or introduce new ones. The guidelines in this manual deal with the prevention and remediation of three principal LSM construction problems: segregation, aggregate fracture, and equipment wear.

Pavement performance problems can also arise from improper LSM design or failure to implement a satisfactory laboratory mix design in field production. The use of the LSM mix design and analysis method in Chapter 2 and careful attention to proper materials specification and quality control will minimize these problems.

4.2 PROBLEM AREAS

4.2.1 Segregation

Segregation, defined as the separation of the coarsest aggregate particles from the rest of the asphalt concrete mixture, is a common problem with dense-graded asphalt concrete mix, but less often with open-graded mixes. Historically, segregation is the single most consistent problem in LSM production, placement, and compaction. Segregation of the large aggregate particles can occur at several points during the manufacture, storage, hauling, and placing of LSM, and LSM are more prone to segregation because particle sizes vary greatly in the mix.

Segregation in LSM has probably been the greatest deterrent to more widespread use of this mix type. Segregated areas which appear on the surface of the HMA pavement are unsightly; the surface texture of the segregated area is significantly more open than that of the surrounding pavement surface. Segregated pavement areas lack the load-spreading capabilities of a more uniform LSM and tend to ravel under applied traffic loads. Segregation also reduces the service life of the LSM since the LSM in the open areas exhibits high air voids content and ages more rapidly.

4.2.2 Aggregate Fracture

Another problem often associated with LSM is fracture of the larger aggregates during the HMA production process. Depending on the quality and hardness of the coarse aggregate particles, the corners of the large aggregate pieces may break off inside the batch plant dryer or inside the drum mixer during the heating and drying operation. In addition, the large stones falling onto smaller particles may cause their fracture. This changes the gradation of the HMA to an unknown degree during production and may affect the amount of interlock obtained between the various aggregate particles and, thus, the strength and performance of LSM.

During compaction, aggregate fracture can occur under the rollers. More fracture typically is found when the mixture contains many larger aggregate particles or relatively soft coarse aggregate. Normally, more fracture occurs when a vibratory roller is used in the breakdown position, directly behind the paver, than when a pneumatic tire roller or a static steel-wheel roller is used to compact LSM initially. Aggregate fracture may occur in some LSM, even if the vibratory roller is used in the intermediate position behind a pneumatic tire or static steel-wheel breakdown roller.

4.2.3 Equipment Wear

LSM may increase the wear of various components in the HMA plant and the paver. In a batch plant production operation, the larger-sized coarse aggregate may create additional wear on the flights inside the dryer and the dryer shell, on the screen cloth at the top of the tower, and on the liner plate and paddle tips in the pug mill. In a drum mix plant operation, incorporation of larger-sized coarse aggregate may increase the wear on the flights inside the drum as well as on the drum shell. Some increased wear can be expected on the flights and liner plate on the drag slat conveyor, on the liner inside the silo, and on the liner on the discharge cone of the silo. This increased wear results, in part, from the smaller proportion of fine aggregate in the mix and the lack of cushioning of the coarsest aggregate particles by the sand-sized particles during the processing of the aggregate through the plant.

At the paver, increased wear can be expected on the flights of the drag slat conveyor, which carries LSM from the hopper to the rear of the paver. Slight additional wear may be found on the augers that distribute the mix across the width of the screed.

The amount of increased wear on the various plant and paver components is difficult to quantify because few contractors have produced LSM recently in long production runs to require replacement of the dryer or drum mixer flights. Further, most contractors produce dense-graded and open-graded mixes along with LSM during a paving season and have no way of estimating the amount of wear caused by one particular mix. It is believed, however, that continued production and placement of LSM will disproportionately increase the wear on certain components of the plant and paver and increase the cost of maintaining that equipment.

4.3 SEGREGATION

Three principal types of segregation are found in asphalt concrete pavement layers: random or rock pockets, longitudinal or side to side, and truckload to truckload. Each type has a different pattern on the roadway and a different cause. More importantly, because the source of each type of segregation is different, their solution must be directed at the specific causes. As stated previously, segregation is defined as the separation of the coarse aggregate particles in the mix from the remainder of the HMA; typically, the asphalt content also varies in inverse proportion to the coarse aggregate content within the segregated pavement. Segregation, except for rock-pocket type, can be significantly reduced or prevented by reducing the distance that the coarse aggregate particles can roll during the various phases of the construction process.

4.3.1 Random Segregation

Random segregation, sometimes called rock-pocket segregation, can occur in any lift of HMA at variable locations, both transversely and longitudinally, along a roadway. The segregated areas may occur fairly regularly or only intermittently in the pavement mat. Rock pockets are generally caused by improper handling of the coarse aggregates at the asphalt plant—both at the aggregate stockpiles and at the cold feed bins.

Segregation can occur in the aggregate stockpiles if those piles are improperly constructed. The coarsest aggregate particles tend to roll down the side of the pile and collect at the bottom. If this occurs, the front-end loader operator must reblend the aggregate together before the material is picked up for transfer to the cold feed bins. If the segregated aggregate is not reblended, the loader operation will eventually place a bucketful of the coarser aggregate into a particular cold feed bin, followed by a bucketful or two of finer coarse aggregate material. This can result in significant variation in the gradation of the paving mix produced, depending on the type of plant used.

In a *batch* plant operation, a variation in the gradation of the coarsest aggregates in the cold feed bins will result in a change in the amount of each size of aggregate in the hot bins at the top of the mixing tower. The aggregate will pass through the dryer in about the same gradation as it comes out of the cold feed bins. As that aggregate passes over the screen deck at the top of the plant, however, it is divided into sizes and placed in the appropriate hot bin. As long as the plant operator does not run any individual hot bin empty or charge additional coarse aggregate into the mixture if the largest size hot bin is overflowing, the segregation that had occurred at the stockpile and cold feed bins will be eliminated at the hot bins. Further, the pug mill on the batch plant is very efficient in reblending any segregated aggregate during the mixing process. Random segregation, even for LSM, is generally not a problem in a batch plant HMA manufacturing process.

In a *drum mix* plant operation (either parallel flow or counter flow), however, segregation that occurs in the coarsest aggregate at the stockpile and/or at the cold feed bins will typically show up on the roadway behind the paver. A drum mix plant operates on a first-in, first-out principle. Because the drum mix plant operates on a continuous basis, any material delivered from the cold feed bins to the plant will pass through the plant relatively unchanged in gradation. Coarser-than-expected aggregate discharged from the cold feed bins will be discharged from the drum mixer with only minimal changes in aggregate size and gradation.

Random segregation also may occur during the truck loading operation. If a batch of HMA is delivered from the pug mill, random segregation is rarely a problem because the mix is discharged in a mass from the pug mill into the truck bed. If the mix is delivered from a silo into the haul truck, random segregation may occur, depending on how the truck is loaded. Rock pockets or random segregation may readily occur if the plant operator continually opens and closes the discharge gate on the silo to deliver small quantities of mix into the truck to "top off" the load.

4.3.2 Longitudinal Segregation

Segregation that occurs continuously on only one side of the paver is usually caused by improper loading of the haul trucks from the pug mill or silo. If the mix is not delivered into the center of the width of the truck bed, the coarsest aggregate particles in the mix can roll to one side of the truck bed and collect along that side. When the mix is delivered into the paver hopper, the segregated mix will be placed on the roadway along the same side, and the segregation will appear as an area of coarser texture in the longitudinal direction on one side of the paver only. This type of longitudinal segregation will generally be intermittent because most haul trucks tend to load into the middle of the width of the truck bed, and, if caused by improper truck loading, side-to-side segregation will also change sides at the paver depending on whether the truck was off center to the left or the right under the silo.

Longitudinal segregation normally originates at the top of the silo. It is caused by the method used to deliver the HMA, either LSM or conventional, into the silo from the conveying device—drag slat conveyor, bucket elevator, or conveyor belt. The mix should be introduced into the center of the silo, either into the batcher or directly into the silo itself. If the mix leaving the slat or belt conveyor or bucket elevator is thrown to the far side of the silo, it will travel down that side of the silo and eventually be discharged into the same side of the haul truck. If the mix is deflected to the same side of the silo as the conveying device, the coarsest aggregate particles will roll and collect on that side of the silo, travel down that side of the silo, and be delivered into the same side of the haul truck.

Longitudinal segregation, if caused by the way the mix is charged into the silo, will always be on the same side of the paver. In addition, this type of segregation will be continuous. Therefore, if the haul trucks are brought under the silo from the opposite direction and loaded, the segregation should switch sides at the paver. This test can help isolate the cause of segregation.

4.3.3 Truckload-to-Truckload Segregation

Truckload-to-truckload segregation may occur at every location where a truck transfers mix to the paver, or it may occur only intermittently at transfer points down the roadway. The frequency of this type of segregation depends on the method used to load the haul trucks at a batch plant pug mill or silo. Further, truckload-to-truckload segregation depends on the specific method used to transfer the mix from the truck bed into the paver hopper and the condition of the hopper between truckloads of mix. Mixes containing larger coarse aggregate particles, such as LSM, will tend to segregate more than will conventional, dense-graded HMAs.

In general, the truck loading process from the silo at a batch or drum mix plant is the point at which this type of segregation occurs. To prevent truckload-to-truckload segregation, it is necessary to place some of the LSM against the front bulkhead of the truck bed. It is also necessary to deposit some mix against its tailgate. Thus, the plant operator will have to pay particular attention to how and where the mix is discharged from the silo and placed in the haul truck bed.

Truckload-to-truckload segregation has been incorrectly described as "end-of-load" segregation. This type of segregation is really a combination of the last coarse aggregate particles from one truck bed and the first from the next truck bed. If a haul truck is loaded with mix in one or two drops in the center of the length of the truck bed, the coarsest aggregate particles will tend to roll down the mound of mix toward the front and the rear of the bed.

If the paver operator completely empties the hopper of the paver between truckloads of mix, any coarse aggregate particles that have collected at the tailgate of the truck bed will be delivered directly into the bottom of the paver hopper and onto its drag slat conveyors. Those coarse, segregated particles will pass through the paver to the augers and then onto the pavement surface under the screed. As the haul truck is emptied, any coarse aggregate particles that have rolled to the front of the bed during loading will be delivered last into the paver hopper. If the hopper is nearly empty when this occurs, the segregated material will quickly appear on the surface of the roadway behind the paver. Thus, the process of delivering the mix to the paver and the condition of the paver hopper between truckloads of mix can either increase or decrease the magnitude of segregation.

4.4 CONSTRUCTION GUIDELINES

4.4.1 Stockpiling and Aggregate Delivery

Random rock-pocket problems can be reduced by building the stockpile, particularly that of the coarser aggregate, in layers. Use the lowest stockpile height that space will permit. If those stockpiles are not built up in layers, the coarsest aggregate particles tend to roll down the pile and collect around the bottom. Stockpiles that are built by conveyor belt in a conical shape are the most susceptible to this type of segregation. In addition, as the size of the largest aggregate particle increases, segregation tends to increase.

If coarse aggregate particles do accumulate around the bottom of the stockpile, the front-end loader operator must reblend the material before it is placed in the appropriate cold feed bin. This may require significant manipulation of the pile to eliminate the segregation. The loader operator must deliver a consistent gradation of aggregate into the cold feed bins. Stockpile management, both in terms of adding aggregate to the stockpile and its subsequent removal, is the key to eliminating the rock-pocket problem on the roadway behind the paver. In general, the cold feed system on a batch or drum mix plant does not have to be modified in order to produce LSM. Depending on the number and gradation of the different-sized aggregates used to meet the mix gradation, however, additional cold feed bins may be needed. Most asphalt plants have four cold feed bins. If five aggregates are needed to produce the required mix gradation, it will be necessary to add another cold feed bin to the system to accommodate the greater number of aggregate sizes.

The front-end loader operator should exercise considerable care when placing the largest aggregate size into the cold feed bin. This material, because of its particle size, may cause some additional wear on the sides of the bin if it is delivered quickly into the bin in one bucketful. It is good practice to load this cold feed bin more slowly than normal from the lowest possible height to reduce the velocity of the aggregates as they hit the sides of the bin. In addition, the coarse aggregate should be delivered into the center of the cold feed bin, instead of against the sides. Finally, the loader operator should try to keep the bin relatively full, adding new aggregate more often, in order to use the aggregate already in the bin to cushion the impact of the new aggregate being delivered.

4.4.2 HMA Production

4.4.2.1 Batch Plant Operations

If the coarsest aggregate particles are segregated in the cold feed bins, the aggregate will pass through the dryer without any significant blending of this material with the other aggregate because the heating and drying process is a continuous flow operation. After the aggregate is discharged from the dryer and travels up the hot elevator and across the screen deck, it is divided into sizes and deposited in the appropriate hot bin. As long as the operator regulates the plant consistently, does not empty a hot bin of aggregate or overflow a hot bin with too much aggregate, and maintains constant bin pulls from each hot bin, the gradation of the HMA will be consistent. No random segregation problems will occur on the roadway because of the condition of the aggregate stockpiles.

Often, however, a plant will become "out of balance" if the front-end loader operator feeds a few bucketfuls of coarser aggregates and then a bucketful or two of less coarse aggregate into a particular cold feed bin. The coarse aggregate hot bins in the plant (bins number 3 and 4 at the top of the tower beneath the screen deck) may run out of material or may overflow depending on the rate of delivery of the aggregate into the cold feed bins and through the dryer. If this occurs, the plant operator should shut the plant down, work with the loader operator to eliminate the cold feed delivery problem, and then restart the plant.

This rarely happens in actual plant operations. In many cases, if one hot bin is short of material, the plant operator will "make up" for the lack of one aggregate size by temporarily increasing the amount of aggregate pulled from the other hot bins. Similarly, if one hot bin is overflowing with a particular size aggregate, chances are that the next few batches of HMA produced will have some additional material of that size included. However, this type of undesirable operation does not usually result in random rock pockets on the roadway behind the paver. Instead, it typically appears as a change in the texture of the surface of the pavement mat because of the variable amounts of coarse aggregate in the mix.

The operation of a batch plant is not normally a contributing factor to the occurrence of random segregation, longitudinal segregation, or truckload-to-truckload segregation. Differences in the texture of the finished mat may result, however, if proper stockpile management techniques are not used.

4.4.2.2 Drum Mix Plant Operations

If segregated aggregates are deposited into the cold feed bins by the front-end loader operator, that segregated material will pass through the drum mix plant (either parallel flow or counter flow) without any significant blending with the other aggregates because of the continuous flow process employed in drum mix plant operation. Indeed, the segregated mix will be discharged from the drum, travel up the slat conveyor and through the silo, be transported to the paver in the haul truck, and pass through the paver to the surface of the pavement layer being constructed. Random segregation can and does occur in drum mix plant operations.

As with batch plant operations, random segregation can be solved by proper stockpile management. Care must be taken in how the coarsest aggregates are delivered into and removed from the stockpile and how the coarsest aggregate particles are placed in the cold feed bins, as discussed above. There is generally nothing in the operation of either a parallel flow or a counter flow drum mix plant (in the mixing drum itself) that contributes to either longitudinal or truckload-to-truckload segregation.

The mix should be observed as it is discharged from the drum onto the slat elevator to detect inadequate mixing or material buildup on the slats.

4.4.3 Silo Operations

Longitudinal segregation can readily occur when HMA is improperly delivered into the silo from the conveying device—drag slat conveyor, bucket elevator, or belt conveyor. If side-to-side or longitudinal segregation occurs continuously on one side of the paver, the mix is being thrown to one side of the silo as it leaves the conveyor or elevator. In most cases, the coarsest aggregate particles in the mix will be flung to the far side of the silo and travel down that side. Depending on the configuration at the top of the silo, the mixture may hit a splitter or deflection plate at the top of the silo where the coarsest aggregate particles are directed back to the same side of the silo as the conveying device.

Continuous side-to-side or longitudinal segregation on one side of the paver signifies a problem at the top of the silo. The problem must be corrected by redirecting the LSM into the center of the batcher at the top of the silo (if a batcher is used) or into the center of the silo. Baffle plates or other deflection devices may be needed at the top of the silo to help solve the problem, particularly if the silo is not equipped with a batcher.

Side-to-side segregation that occurs intermittently and on both sides of the paver at different times is typically related to the position of the haul truck under the silo or under the pug mill of a batch plant. If the truck is off center while being loaded, the coarsest aggregate particles in the mix, particularly in LSM, may roll to one side of the truck bed. These coarse aggregate particles will be delivered into one side of the paver hopper and come out directly behind the screed on the same side of the paver.

The roadway should be inspected to determine if longitudinal segregation is continuous or intermittent and if it always occurs on one side of the lay down machine or on both sides. If the longitudinal segregation is intermittent and changes from side to side, the loading of the haul trucks at the plant should be investigated. Each truck should be loaded in the center of its bed from the center of the width of the batch plant pug mill or from the center of the silo discharge gate or gates.

If the longitudinal segregation always occurs only on one side of the paver, the direction that the haul trucks are facing when they are being loaded under the silo should be reversed. For example, if the trucks normally load while facing in a northerly direction, some trucks should be loaded while facing in a southerly direction. When the latter trucks arrive at the paver, the segregation typically found on one side of the lay down machine should switch to the opposite side of the paver. If this occurs, it is confirmation that the longitudinal segregation is occurring at the top of the silo.

Prevention of longitudinal (side-to-side) segregation on the roadway begins at the top of the silo. Mix delivered to the top of the silo by slat conveyor, belt conveyor, or bucket elevator will be discharged to the far side of the silo by the natural centrifugal force of the conveying device, unless some means is used to redirect the flow into the center of the silo. On some silos, a series of baffles are used to control the direction of the material. Other silos are equipped with a splitter system that divides the mix as it delivered, causing a portion to be placed in each part of the silo. Use of the baffle and splitter systems can reduce the tendency for longitudinal segregation on the roadway but does not always eliminate it. Use of a batcher system at the top of the silo is a better means to reduce this type of segregation.

4.4.4 Truck Loading

The objective of the truck-loading operation is to fill the haul truck with HMA and transport it to the paver as quickly as possible. This objective must be balanced, however, with the need to minimize segregation of the mix that occurs during loading. The primary cause of truckload-to-truckload segregation is improper loading of the haul truck with mix from the silo.

If coarse aggregate particles are permitted to roll to the tailgate of the haul truck bed, the first material out of the bed when the mix is delivered to the paver hopper will be the coarse aggregate particles that have collected at the tailgate. If the coarse aggregate particles are permitted to roll to the front of the truck bed, the last material out of the bed when the mix is delivered to the paver hopper will be the coarse aggregate particles that have collected at the front of the bed. In either case, truckload-to-truckload segregation will result. The degree of segregation will be even greater if both the front and rear of the haul truck beds are loaded incorrectly and the coarse aggregate that has rolled to the front of one haul truck is combined with the coarse aggregate that has collected at the rear (tailgate) of the next haul truck.

Such segregation can occur at every truck exchange point if all of the haul trucks are loaded incorrectly at the plant. This type of segregation will occur intermittently if only some of the trucks are loaded improperly. Some truck drivers will load some of the mixture against the tailgate as well as against the front bulkhead of the bed. Other drivers insist on loading only in center of the length of the truck bed. How the trucks are loaded will determine the frequency of the truckload-to-truckload segregation on the roadway.

If all the mix is placed in the haul vehicle in one or two drops from the silo, segregation of the larger aggregate particles will occur. If the mix is deposited into the center of the truck bed, the material will build into a conical-shaped pile. Because the growth of the pile will be restricted by the sides of the truck, the larger aggregate particles will roll toward the front and rear of the truck bed. These larger particles accumulate at both ends of the load and then are delivered into the hopper on the paver from the truck bed. The pockets of coarse material then appear in the mat behind the lay down machine at the end of the truckload of mix. In reality, some of the larger aggregates come from the front of one truckload and the rear of the next truckload of mix.

Proper loading procedures dictate multiple (more than two) drops of mix into the truck instead of only one or two drops. This is necessary to minimize the distance that the coarse aggregate particles can roll and to keep the mix consistent in gradation throughout the entire load. Using multiple drops of mix under the surge silo means that the truck should not be loaded by discharging the mix in only one or two drops into the center of the length of the truck bed and that the truck cannot be loaded while moving slowly forward under the silo during loading. If multiple drops are not used, the coarsest aggregate particles in the mix will tend to roll back to the tailgate of the truck bed or to its bulkhead.

It is important to deposit the HMA in a mass into the haul truck. The gates on the bottom of the cone should be opened and closed quickly. The gates should also open completely so that the flow of mix is unrestricted. There is only one reason to cut off the flow of mix into the vehicle once delivery has started—in order to divide the delivery of the mix among different segments of the truck bed.

If a tandem axle or a triaxle end dump truck is used to haul the mix, one drop of the material must be placed as close to the bulkhead of the bed of the haul truck as possible. In addition, another drop should be placed as close to the tailgate of the haul truck bed as possible. Both of these drops will minimize the distance that the coarse aggregate particles can roll to the front and rear of the truck bed. For either of these two types of trucks, a third drop of mix should be placed into the truck bed between the first two drops. Further, to ensure that the proper amount of mix is placed against the tailgate of the truck, it is good practice to place the first drop of mixture at the rear of the truck bed, the second drop at the front of the truck bed, and the third drop between the first two drops of mix.

If a semi-truck-and-trailer-type haul unit is used by the contractor, the loading sequence should be as follows: the first drop should be made into the rear of the truck bed as close to the tailgate as possible. The truck should then back up, and the second drop should be made into the truck bed as close to the front of the truck bed as possible. Additional drops should be placed between the first and second drops. The number of additional drops depends on the length of the semi-trailer truck bed. In general, at least three additional drops should be made, for a total of five drops. In no case should the bed of a semi-trailer be loaded while the truck is moving slowly forward under the silo. This action causes a preponderance of the coarse aggregate particles to roll toward the tailgate area of the truck bed. Because truckload-to-truckload-type segregation is a combination of both the end of one load of mix (at the front of the truck bed of the first truck) and the beginning of the next truckload of mix (at the tailgate of the truck bed of the second truck), segregation will be eliminated by depositing the mix into the truck bed as close as possible to both the bulkhead and tailgate.

In many states, weight distribution laws do not permit a contractor to place the same amount of mix into the truck bed at each drop of mix from the silo. In most cases, it is necessary to deposit less mix into the rear of the truck bed than in the rest of the truck bed. The laws of the particular state in question need to be checked to determine how much mix can be placed over the rear and front axles of the truck. For example, if a tandem-axle or triaxle dump truck is being used, about 20 percent of the total weight of mix to be hauled should be loaded into the middle of the rear half of the truck bed. The truck should then be backed up so that the next 40 percent or so of the total load can be deposited into the middle of the front half of the bed, near the front wall. The vehicle should then be moved forward again so that the remaining 40 percent of the mix can be dropped into the center of the bed, between the first two drops. The actual amount of mix deposited into the truck on each drop will depend on the length of the truck, the number and spacing of the axles, and on the weight distribution requirements in each particular state.

One practice that should not be permitted, especially for LSM, is to "top off" a truckload of HMA to attain the maximum legal weight on the haul truck. In many cases, the haul truck is sitting on a scale under the silo as it is being loaded. The plant operator wants to maximize the amount of mix that the truck hauls to the paver. If the total weight of the truck is not at the maximum, the plant operator may open the silo gates briefly to add a little extra HMA to the load. If the small drop is not enough, the silo gates might be opened one or more additional times to fill the truck to the legal weight limit.

The primary problem with this type of loading operation is that the small drops of mix fall on top of the mounds of mix already in the bed. The large aggregate particles in the mix roll down the slope to the front of the haul truck bed and to the tailgate. This can significantly increase the amount of segregation that occurs with each truckload of HMA.

Truckload-to-truckload segregation can also occur when live-bottom (flow-boy) trucks are used to haul the mix from the plant to the lay down machine. For this type of truck (those with a conveyor belt or slat conveyor in the bottom of the truck bed), it is often believed that the bed can be loaded with the truck moving slowly forward under the silo. This operation, however, results in the coarsest aggregate particles in the mix rolling continuously toward the rear of the truck bed during loading. When loading is completed, a preponderance of the coarse aggregate particles have collected near the tailgate of the truck and are delivered first into the paver hopper when the conveyor starts up. Although there is usually little segregation at the end of the load delivered from a live-bottom truck, there may be considerable segregation at the start of the load delivery.

When a live-bottom truck is being loaded, it is necessary to place the first drop of HMA as close to the rear of the truck bed as possible. This will reduce the distance that the coarse aggregate will roll. The truck should then be moved backward under the silo and the front of the truck bed loaded. The remainder of the bed length should be filled in with additional drops of HMA in a manner similar to the loading procedure used for end-dump-type semi-truck trailers. For some dense-graded HMA, after the first drop of mix has been made at the rear of the truck bed, it may be acceptable to move the truck backward and then load the truck from front to rear with the truck moving slowly forward; however, this should never be performed with LSM. When mix delivery reaches the rear of the truck bed, it will contact the mix already placed during the first drop. Normally, any coarse aggregate particles that have rolled toward the rear of the bed will be mixed in with the remainder of the mix as the conveyor in the bottom of the truck pushes the mix out the back of the truck. However, because they tend to segregate more, this loading procedure is not recommended for LSM. Distinct, multiple drops of mix into the live-bottom truck bed should be used for LSM.

For many dense-graded HMA, bottom- or belly-dump-type trucks can be loaded directly over the discharge gates at the bottom of the bed, and segregation will not normally be a problem because the discharge gate is the lowest point in the truck bed. With LSM, however, the coarsest aggregate particles tend to roll to the front and rear of the bed, at the top of the load, as the mix is delivered from a silo into the center of a belly-dump truck. The coarsest aggregate particles, in the top four corners of the load, are discharged last from the belly-dump truck. Segregation, in this case, occurs at the end rather than the beginning of each truck load delivery.

Therefore, for LSM, a bottom- or belly-dump truck should also be loaded in multiple drops. If the truck bed has only one discharge gate, the first drop of mix should be in the center of the truck bed, directly over the gate. Depending on the size of the truck bed, up to 70 percent of the total weight of the load should be delivered on the first drop. Before the truck is fully loaded, however, the truck should be moved backward and part of the load placed at the front of the bed. Then the truck should be pulled forward and the remainder of the load should be placed on the rear.

If the belly-dump truck has two discharge gates, the first drop of mix should be placed directly over the front gate. The truck should then be moved forward and the second drop of mix should be deposited directly over the rear gate. Drops three and four should be made on the front of the bed and on the rear of the bed. This procedure will greatly reduce the distance that the coarsest aggregate particles can roll and will significantly decrease the probability of segregation.

The truck loading procedures recommended here, using multiple drops of mix into the truck bed regardless of the type of truck, will increase the time needed to fully load the truck. However, this will not normally increase the cost of mix delivery because plant production capacity normally controls the overall rate of the construction process. For example, assume that the plant capacity is 400 tons per hour and the triaxle haul trucks can legally carry 20 tons of HMA per load. Twenty trucks per hour will then be needed to deliver the HMA produced to the paver. Therefore, approximately 3 min is available to load each truck. Because only about 20 sec are needed to place a drop of mix into the truck bed, plenty of time is available to load the trucks with three drops of material per truck and move the truck between drops.

Truckload-to-truckload segregation is eliminated by loading the haul truck correctly at the asphalt plant. The extra cost associated with loading the truck, if any, must be balanced against the contractor's cost to correct severe segregation or the agency's cost of reduced pavement life resulting from segregated mix on the surface of the roadway.

4.4.5 Truck Unloading

Unloading procedures used to deposit HMA into the paver hopper from the haul trucks are also important to minimize segregation. If an end-dump truck is used and if the mix being delivered to the paver tends to segregate, the truck driver should raise the truck bed, with the tailgate closed, to the point where the mix shifts toward the tailgate of the truck. The bed should remain partly raised while the truck driver is waiting to deliver mix to the paver (while another truck is in front of the paver) and also while the truck is backed into position at the paving machine. Once the truck and the paver are in contact, the tailgate should be opened and the mix discharged into the paver hopper. This procedure will deliver the mix from the truck in a mass and "flood" the hopper of the paver, reducing the probability of segregation behind the paver screed. When the mix is moved as a mass, the coarsest aggregate particles will not separate or roll away from the remainder of the mix.

For end-dump truck operation, the driver normally waits until the truck bed is empty before raising the bed to its highest position. This action causes all of the coarsest aggregate particles collected in the front corners of the bed to tumble into the paver hopper as individual particles, instead of moving into the hopper as part of the mass of HMA. It is much better to raise the truck bed to its highest position when 20 to 30 percent of the load is still in the bed. This permits incorporation of the coarse aggregate particles in the front corners of the bed into the remaining mass of mix and will, in turn, significantly reduce the segregation that occurs at the end of each truckload.

When a live-bottom truck is used to transport the mix, the belt or slat conveyor should be started for a few seconds before the end gate on the truck is opened. This will create a mass of material that can be delivered to the hopper, instead of allowing any coarse aggregate particles that have rolled to the rear of the truck bed or end gate to be discharged into the hopper first.

For bottom- or belly-dump trucks, a windrow-sizing box should be used to control the dimensions of the windrow. With the box in place, the gates on the bottom of the truck bed can be opened wide to discharge a mass of mix rather than a trickle. If truck discharge is controlled manually, the gates should still be opened wide so that the mix is deposited in a mass onto the roadway. The size of the windrow should be controlled by the forward speed of the haul truck. If coarse aggregate particles are visible on the top of the windrow at the end of the discharge of the mix from the belly-dump truck, this material should be distributed down the roadway and not left in a pile at the end of the load. This can be done by almost completely closing the discharge gates on the truck just before the bed is empty and keeping the truck moving forward until the bed is empty. This procedure is unnecessary, however, if the truck is loaded properly at the plant.

Another method used to deliver mix to the paver is with a material transfer vehicle. This piece of equipment is basically a surge bin on wheels. HMA is deposited into the hopper on the front of the vehicle. The device is equipped with a remixing auger. The mix is transported from the receiving hopper by a conveyor, through the auger, and to another conveyor, which delivers the remixed material to an extended hopper on the paver. The auger system reblends the coarse and fine particles of the mix and corrects segregation that might occur at the plant silo and during loading.

The material transfer vehicle also allows almost continuous paver operation (without stopping between truckloads of mix) if a continuous supply of mix is available from the asphalt plant. This provides for a smoother mat behind the paver screed because the paver operator can keep the head of material in front of the screed constant by supplying a continuous flow of mix back to the screed. The equipment also prevents the haul trucks from bumping the paver and truck drivers from applying their brakes when the truck is being pushed by the paver. Material transfer vehicles are expensive, however. The same ends can be achieved, at lower cost, by loading the haul trucks properly at the asphalt batch or drum mix plant and preventing segregation.

4.4.6 Paver Operations

Truckload-to-truckload segregation will be affected by the condition of the paver hopper between truckloads of mix. If the paver operator empties the hopper between truckloads, the degree of segregation that occurs on the pavement surface may be increased. If the paver operator dumps the wings on the sides of the paver hopper between truckloads of mix, the amount of segregation will be further increased. If, however, the paver operator keeps the paver hopper at least half full between truckloads of mix, the coarse aggregate particles delivered into the hopper from the end of one truckload and the beginning of the next will be deposited into the mass of mix already in the hopper. Thus, the amount of segregation that occurs on the road surface will be significantly reduced.

After the haul truck has deposited all its mix into the paver hopper, the truck driver should be directed to quickly lower the truck bed and drive away. The paver operator should also stop the paver quickly—the paver hopper should not be emptied of mix. The paver operator should not dump the wings of the paver. The next truck should be backed into the paver hopper and the mix delivered into the half full hopper.

The wings at the sides of the paver hopper should not be emptied between truckloads of mix. All of the coarse aggregate that accumulates in the front corners of the truck bed typically slides down the sides of the bed last—and into the wings on the paver. If the paver hopper is kept full between truckloads, any attempt to dump the wings will result in mix being forced out the front of the hopper and onto the pavement in a pile in front of the machine. This process results in a bump in the pavement surface. The paver operator quickly learns that the paver hopper must be essentially empty in order to dump the wings and not create a mess on the pavement surface. The problem is that when the wings are dumped into an empty hopper, all the coarsest aggregate particles that have collected in the wings are deposited in the bottom of the hopper on top of the slat conveyors. When the conveyors are started, all of the segregated material is carried back through the paver and delivered to the augers. A segregated pavement surface results.

One possible solution is to allow mix to accumulate in the corners of the paver hopper through the day. At the completion of paving, the cold material in the hopper wings is wasted or returned to the plant for recycling. Another solution is to slightly reduce the capacity of the hopper by placing a fillet or cutoff plate in each back corner of hopper. This will prevent mix from collecting in this area and make the dumping of the wings unnecessary. Segregation of LSM can be greatly reduced by not dumping the wings.

During paving, the flow gates at the rear of the paver hopper must be set so that the slat conveyors at the bottom of the hopper operate as close to 100 percent of the time as possible. This will supply a relatively constant head of material on the augers in front of the paver screed and allow the paver screed to ski at a constant angle of attack. If the paver operator empties the hopper between truckloads, the head of material in front of the screed will decrease as the augers are emptied of HMA and the thickness of the mat being placed will decrease. Further, if segregated material is deposited into the hopper at the end of one truckload as well as at the beginning of the following truckload, and if the hopper wings are dumped into the empty hopper, these coarse aggregate particles will be carried back through the paver on the slat conveyors and dumped on nearly empty augers. Severe truckload-to-truckload segregation on the pavement surface will result.

The minimum layer thickness should be at least twice the maximum size aggregate in the LSM. For example, if the LSM contains aggregate that has a maximum size of 2 in. (50 mm), the minimum layer thickness should be 4 in. (102 mm). Compaction may be inadequate and aggregate breakage may be unacceptable if lifts are placed too thin. Depending on the distribution of large aggregate in the mix (overall gradation of the LSM), it may be necessary to increase the minimum layer thickness to 2.5 times the maximum aggregate size in order to reduce the tearing of the mat under the paver screed and obtain a more uniform surface texture.

Yield is often difficult to check when placing LSM. Many paver screed operators use a probe to periodically check the mat thickness of conventional HMA being placed by a paver. The angle of attack of the screed is then adjusted to increase or decrease the thickness of the mat placed on the basis of the reading obtained. This is very poor construction practice. A better procedure is to periodically check the yield by comparing the amount of mix actually placed over a particular length and width of pavement to the quantity of material planned for placement over that area. If the values are significantly different, a small adjustment should be made in the angle of attack of the screed.

In a properly designed LSM, it is impossible to push any type of rod or probe through the layer being constructed because of the amount of coarse aggregate in the mix and the thickness of the course. In addition, an LSM is usually "fluffier" than a conventional mix, that is, it will not be compacted as much by the action of the paver screed as a conventional mix. Therefore, using a probe stuck into the mat behind the screed often conveys misleading measurements and, thus, results in improper adjustment to the angle of attack of the screed.

4.4.7 Handwork and Joint Construction

For most HMA paving projects, some handwork is necessary around catch basins, manholes, curbs, and driveways and in the corners of the pavement at intersections. In these cases, the paver operator usually feeds extra mix back through the paver and the laborers on the paving crew manually shovel the mix into the proper location. Once the mix has been moved into its approximate final position, it is further spread with a rake or lute to provide a uniform pavement surface ready for compaction.

Handwork is difficult, at best, with LSM. Because of the size of the aggregate in the LSM and because of the relatively thicker layer typically being constructed, it is not realistic to expect the laborers to move the mix by hand. For LSM, the paver operator must use the machine to place the mix as close to its final position as possible. This means more maneuvering of the paver and perhaps a slightly slower paving operation depending on the layout of the project. In some locations, such as intersections, consideration should be given to using a conventional dense-graded base course mix in place of LSM in the radius of the corners and in other areas where handwork is required. These areas usually do not receive much direct traffic action.

It is impossible to properly level LSM with a rake or lute. In addition, LSM will not be as dense as a conventional mix when moved by hand. This means that raking will leave LSM higher than the elevation of the surrounding mix and even higher than with a hand-placed conventional mix in order to achieve the proper density after final compaction. For good construction, LSM should be placed by the paver, instead of by hand, wherever possible.

For the same reasons, broadcasting of LSM back over a mat already placed by the paver is undesirable. In most cases, the added mix will sit on top of the previously placed mat and will not blend well with the original mix. After compaction, the broadcasted mixture will cause the "repaired" area to have a different surface texture and a different density than the mat adjacent to that area. If it is necessary to place more LSM in a location that lacks mixture for some reason, care must be taken to place the new mix only in the area that needs to be filled or repaired and not to spread mix all over the surrounding pavement surface. This means, once again, that any handwork with LSM is more difficult and time consuming than handwork with a conventional, dense-graded mix.

On many conventional HMA projects, the paver operator permits the end plate on the screed to overlap the top of the mat in the previously placed adjacent lane by an excessive amount. The raker then has to push extra mix back on top of the new mat with a rake or lute. If, however, the paver operator overlaps the top of the mat in the adjacent lane by only 1 to 1.5 in. (25 mm to 63 mm) or less, no raking of the longitudinal joint will be necessary because the paver will place the correct amount of mix in the proper location.

The same is true for longitudinal joint construction with LSM. If the paver operator places LSM with the proper amount of overlap over the previously placed lane, no raking of the mixture along the joint will be necessary. Because of the difficulty of moving LSM by hand (shovel or rake or lute), the mix should be placed in the correct position by the paver instead of by the raker. Attempting to rake a longitudinal joint constructed of LSM is difficult and tiring. In addition, the large aggregate particles that are pushed back across the new mat will not roll into the mat properly and will create variations in density and mat texture. Thus, the best longitudinal joint that can be constructed with LSM is one that is placed by the paver screed and not raked at all.

4.4.8 Mixture Compaction

Compaction is the single most important factor in the ultimate performance of a properly designed and mixed HMA pavement. Compaction is the process through which the HMA is compressed and reduced in volume. As a result of compaction, the asphalt-coated aggregates in the mix are forced together, which increases aggregate interlock and interparticle friction and reduces the air voids content of the mix. Adequate compaction of the mix increases the fatigue life, decreases permanent deformation (rutting), reduces oxidation or age hardening, decreases moisture sensitivity, increases strength and stability, and decreases low-temperature cracking. A paving mix that has all the desirable mix design characteristics will perform poorly under traffic if it is not compacted to the proper density.

LSM may require levels of compactive effort and thus rolling patterns or procedures that are considerably different from those used for conventional mixes. The rollers, however, used for LSM are no different than those used for conventional mixes. The actual rolling pattern used to compact the mix on a paving project should be determined at the inception of the project through the construction of a roller test strip. This strip should be located at a convenient point where the test layer will remain in place as part of the final pavement structure. The condition of the underlying layers at the test strip location should be representative of those on the remainder of the project. The mix should also be representative of the material to be produced for the project, and the thickness and width of the layer placed should be the same as that shown on the plans for the LSM course.

The selection of rollers that will densify the LSM pavement layer should receive careful consideration. The combination of rollers normally used on projects with conventional mixes might not be the most cost-effective or efficient for the variables involved in the LSM project. For many conventional HMA projects, breakdown or initial rolling is accomplished with a vibratory roller. Intermediate rolling is usually performed with a pneumatic tire roller and finish rolling with a static steel-wheel roller. To compact LSM properly, a different rolling pattern may be necessary.

When a vibratory roller is used in the breakdown position to compact LSM, the roller should be operated at the highest possible frequency setting and with an amplitude setting that is related to the thickness of the layer being compacted. For LSM courses more than 4 in. (102 mm) in compacted thickness, the amplitude setting on the vibratory roller should be "high." For LSM courses between 2 and 4 in. (50 mm and 102 mm) in compacted thickness, the amplitude setting should be set on "medium" (if the roller has a medium amplitude setting). If the roller does not have a "medium" amplitude setting, the roller test pattern should be conducted twice, once with the amplitude setting on "low," and again with the amplitude setting on "high," to determine the most efficient setting to obtain the required density level.

One of the primary problems with using a vibratory roller in the high amplitude setting in the breakdown rolling position is fracture of the larger aggregate in LSM. The amount of fracture possible depends on several factors, including gradation of the mix, hardness of the coarse aggregate, thickness of the layer being compacted, and speed of the roller. If the amount of fracture experienced becomes excessive, the compactive force of the vibratory roller should be reduced from the high amplitude setting to a medium or low amplitude setting. This change in compactive effort, however, may significantly reduce the effectiveness of the vibratory roller and more roller passes may be needed to achieve the same air voids content as at the higher amplitude setting.

For most LSM, placing a pneumatic tire roller in the intermediate position, behind a vibratory breakdown roller, is not as efficient as reversing the two rollers and putting the pneumatic tire roller in front of the vibratory roller (i.e., in the breakdown position). When the pneumatic tire roller is in the second position, this roller, which typically builds density in the pavement layer from the bottom of the layer up, must undo part of the compaction accomplished by the vibratory roller. If the pneumatic tire roller is employed in the breakdown position, however, the roller is most efficient in obtaining density in the previously uncompacted mix. The vibratory roller, in the intermediate position, behind the pneumatic tire breakdown roller, tends to compact the LSM from the top down and is more effective as the second roller instead of as the breakdown roller. In other words, the desired level of density in LSM is generally more easily obtained (with fewer total roller passes) when a pneumatic tire roller is used in the breakdown position and a vibratory roller is employed in the intermediate position, behind the pneumatic tire roller.

If the pneumatic tire roller is used in the breakdown position, the tires on the roller must be heated to the same temperature as the LSM to prevent its pickup on the tires. This means that early in the morning, before paving begins, the pneumatic tire roller should be operated on the old pavement for 5 to 15 min (depending on environmental conditions) to build up heat in the tires before the roller is placed on the mix. It may be necessary for the pneumatic tire roller to operate on the mat behind the vibratory roller for 5 to 10 min until the temperature of the surface of the tires approaches the temperature of the mat and pickup of the mix ceases. In this regard, using the pneumatic tire roller in the breakdown position on LSM is no different than using the same roller in the breakdown position on a conventional, dense-graded HMA. Because of the size of the aggregate and the thickness of the LSM layer, however, consideration should be given to using the largest pneumatic tire roller available—certainly not one normally used for surface treatment or seal coat construction.

When a pneumatic tire roller is employed in the breakdown position on LSM, using a nuclear density gauge to measure density level during compaction will be difficult because of the rough texture left on the pavement surface by the rubber tires. However, this same problem occurs when the pneumatic roller is used for initial compaction of a conventional, dense-graded HMA. Nuclear gauge density measurements need to be made after the vibratory roller in the intermediate position has made at least two passes over the mix. Cores cut from the compacted pavement in the test section should be used to determine the actual level of density achieved for each roller pattern tested.

If a vibratory roller is used in the intermediate position behind a pneumatic tire roller, it should be operated in the low amplitude mode. When operated at a high amplitude setting in the second rolling position, the vibratory roller will often cause a significant amount of fracture of the coarse aggregate in the LSM. Finish rolling should be completed using a static steel-wheel roller in conventional fashion.

Desired density levels are easier to obtain when the LSM is hot. Because the internal stability of LSM is generally greater than that of a conventional HMA (because of the increased degree of aggregate interlock in the mix), all rollers can typically operate closer to the paver. Instead of using the traditional roller "train" concept, consideration should be given to using two intermediate vibratory rollers in tandem (side by side), following the pneumatic tire breakdown roller. This compaction procedure should ensure that the desired level of density is obtained in the LSM with a minimum of roller passes.

CHAPTER 5

SUMMARY OF THE RESEARCH PROJECT

This chapter presents a summary of NCHRP Project 4-18, including its objectives, organization, principal tasks, and findings and conclusions. Detailed results are contained in the appendixes.

5.1 OBJECTIVES AND ORGANIZATION OF THE RESEARCH

The objectives of NCHRP Project 4-18 were to do the following:

(1) Evaluate the effectiveness of currently used LSM in resisting plastic deformation in asphalt concrete pavements;
(2) Develop a mixture design procedure for LSM; and
(3) Prepare guidelines on constructibility and quality assurance testing of LSM.

The research was organized into six major tasks as follows:

(1) Compilation of pertinent laboratory and field data on LSM to identify successful design procedures (if any), quality assurance test procedures and criteria, materials specifications, and construction guidelines;
(2) Development of a mixture design procedure for determining the optimum aggregate gradation and binder content for dense- and open-graded LSM;
(3) Development of mixture analysis procedures to ensure that the LSM design will provide the necessary level of performance under expected traffic and environmental conditions;
(4) Conduct of full-scale APT of LSM designs to confirm their rutting resistance compared to conventional control mixes;
(5) Development of construction guidelines to aid in production, placement, and compaction of LSM; and
(6) Preparation of the research products in readily implementable form.

5.2 CONDUCT OF THE RESEARCH

The research team surveyed all fifty SHAs and conducted follow-up interviews with those having experience with LSM. Project personnel then conducted on-site evaluation of LSM pavements (during and after construction) to identify successful LSM design procedures, and criteria, materials specifications, and construction procedures.

In the field project phase of the study, 191- mm (7.5-in.)-diameter pavement cores were obtained in five states from nine LSM projects with an additional three conventional base pavement projects in three states providing controls. A total of 200 LSM and control specimens were cored. These projects were selected from the inventory of LSM field pavements identified in the LTPP GPS experiments; data on all the LTPP LSM pavements are available in Appendix B.

The following tests were conducted on these core specimens:

- Viscosity of the recovered asphalt binder,
- IDT at 5°C (41°F) and 25°C (77°F) at a loading rate of 2 in. per min. (51 mm/min),
- Resilient modulus at 5°C (41°F) and 25°C (77°F),
- Unconfined compression at the Superpave 7-day MMAT,
- Monotonic compression to failure, and
- Superpave (AASHTO TP7) RSCH.

Viscosity of the recovered asphalt binders was used to permit creep testing of the core specimens at an equiviscous temperature, minimizing the effect of the binder on the test results and isolating the aggregate effects.

The IDT and resilient modulus test results were used to estimate the relative fatigue cracking resistance of LSM and conventional mixes in accordance with the NCHRP AAMAS procedures.

Unconfined compressive creep and recovery tests provided data to compare relative resistance to rutting (plastic deformation). Because the thickness of some of the LSM pavement layers was less than 2 times the diameter of the largest aggregate particles, the monotonic compression testing was carried out to determine the effect of specimen height on compressive strength, a key factor in the analysis of the creep data.

The RSCH test is designed to subject specimens to the primary distress mechanism responsible for rutting. A few

RSCH tests were conducted on core specimens; the test procedures and equipment were adapted to accommodate the large-sized specimens.

The results of the tests of field core specimens are presented in Appendix C and analyzed and discussed in Appendix A.

In the laboratory phase of the project, a total of 125 150-mm (6-in.) and 191-mm (7.5-in.)-diameter laboratory specimens were prepared at heights ranging from 50 to 279 mm (2 to 11 in.) from asphalt binder and aggregate supplied by several SHAs. As a first step, a gyratory method to compact specimens that simulated field compaction was developed. Because no Superpave gyratory compactors were available when the research began, a large-sized Texas gyratory compactor was modified to closely match the Superpave specifications in AASHTO TP4. Initial results from one project indicated that a 5-deg angle of gyration was required to simulate field compaction of the LSM in contrast to the 1.25-deg angle specified by the Superpave method for conventional, dense-graded mixes. An angle of 5 deg was used for consistency in all subsequent specimen preparation, and available resources did not permit any further study of the compaction process.

The following tests were conducted on the laboratory-compacted specimens:

- Superpave RSCH at 40°C (104°F);
- Unconfined axial creep and recovery at 40°C (104°F); and
- Full-scale, loaded-wheel rutting tests.

The limited RSCH and full-scale, loaded-wheel rutting tests provided means to estimate the relative resistance of LSM and conventional mixes to rutting. The creep testing was used to determine the effect of varying specimen dimensions on the measured mixture properties. Laboratory test results are presented in Appendix B; the results of the full-scale, loaded-wheel testing are summarized in Appendix D.

The findings from the laboratory study were used in conjunction with current published information to develop the LSM design and analysis method and verify its ability to produce mix designs with the requisite resistance to pavement rutting. Several ancillary LSM test methods, for draindown, bulk specific gravity, and stone-on-stone contact, were also developed to satisfy specific needs identified during the research.

Finally, LSM construction guidelines were developed on the basis of the on-site project evaluations, survey results, and published literature.

5.3 FINDINGS AND CONCLUSIONS

The results of NCHRP Project 4-18 strongly indicate that paving engineers should consider the use of LSM in surface and underlying layers in asphalt pavements to minimize permanent deformation associated with high-volume, heavy-vehicle, or slow-moving traffic facilities. LSM must be properly designed and constructed to achieve this desired result.

This key finding is supported by the following specific findings and conclusions:

- When properly designed and constructed, LSM have provided excellent resistance to heavy, concentrated, high shear loads without permanent deformation and cracking. Asphalt contents of LSMs may be reduced 30 percent or more from conventional mixtures. Production of coarser aggregates requires less crushing energy, which may result in lower costs for aggregates.
- For some SHAs, LSM are considered a standard design. Although 20 SHAs reported having constructed six or more LSM projects in the last 10 years, only 6 (i.e., Arkansas, California, Indiana, Kentucky, Tennessee, and Texas) have used aggregate with a top size greater than 38 mm (1.5 in.). Dense-graded LSM are by far the most common. Some states use open-graded (or gap-graded) LSM, but none report use of stone-filled gradations.
- Although interest in LSM is growing in the SHAs, there is insufficient evidence to establish whether LSM placed in the last 10 years are consistently yielding less rutting than conventional mixtures. Some LSM have resisted rutting; others have exhibited premature rutting. Mixture design appears to be the key factor in determining good or poor performance.
- Some SHAs reported that inadequate methods and equipment for designing LSM have inhibited performance for some projects. Bad experiences during construction or concern about problems, such as segregation, have contributed to a reluctance to specify LSM.
- Problems associated with LSM include segregation, incomplete coating of coarse aggregates, increased mixing time requirements, noise during drum mixing or drying; inadequate paddle clearance inside the pug mill; placement of the coarse-textured mixture; resistance to compaction; fracture of the larger stones; permeable voids in the compacted mat; more equipment wear, and water susceptibility.
- Several SHAs reported construction difficulties with LSM, particularly segregation. Compaction of LSM can be difficult because of insufficient knowledge and experience in constructing the thick lifts required for LSM, poor mixture designs, faulty materials-handling procedures, and improper compaction practices and equipment. These problems, however, do not appear to be insurmountable.
- A survey of SHAs revealed that many dense-graded LSM designs have not provided stone-to-stone contact of the largest aggregates. In most mixture designs cur-

rently used, a few large stones are merely "floating" in a matrix of smaller aggregate and asphalt with no interlock of the larger stones. Such LSM exhibit rutting resistance similar to that of conventional mixtures.

- A comparative analysis of LSM and conventional mixtures contained in the SHRP LTPP database indicates LSM on average, exhibit slightly less rutting, even though they were not necessarily designed with good stone-on-stone contact.
- A two-level LSM design procedure that strives for stone-on-stone contact was developed. Full-scale, loaded-wheel rutting tests revealed dense-graded mixtures designed by this method resist rutting better than an LSM with the same maximum size aggregate but without adequate stone-on-stone contact. The rutting resistance of a dense LSM with good stone-on-stone contact and an open-graded LSM was equivalent.
- A method was developed to quantify stone-on-stone contact of the coarse aggregate in LSM. LSM received from SHAs and tested in this study achieved stone-on-stone contact of only 50 to 70 percent, according to the method developed. The mixture design procedure developed in this study consistently produces a mixture with about 90 percent stone-on-stone contact.
- Open-graded LSM typically attain stone-on-stone contact in excess of 90 percent; these mixtures are also known to resist rutting. Data analyzed in this study indicate that, to obtain the benefit of the large stones in resisting rutting in dense-graded mixtures, stone-on-stone contact, as measured herein, should exceed 80 percent.
- A procedure for measuring mastic draindown in LSM was developed. For LSM containing aggregates up to 63 mm (2.5 in.), a 203-mm (8-in.)-diameter wire basket yielded a more sensitive measure of asphalt draindown than a 150-mm (6-in.)-diameter basket. The mesh size of both baskets was 6.4 mm (0.25 in.). The same mass of mixture was used in each basket; it appears that the additional sample height, or length of drain path, in the 150-mm (6-in.)-diameter basket slowed the quantity and rate of draindown of the asphalt mastic.
- Compressive creep testing of LSMs revealed that the strength and energy required to produce specimen failure increased significantly with an increase in maximum aggregate size.
- Data from compressive creep and recovery tests suggest that LSMs are best suited to pavements where load durations are longer than those associated with normal highway speed traffic, for example, intersections, urban streets, truck terminals, and airport taxiways and aprons where the load carrying capacity of the aggregate skeleton is more fully mobilized.
- Successful techniques were developed for testing large LSM specimens using the Superpave RSCH Method. The transfer function developed by SHRP researchers appears to be applicable to LSMs, although it was developed from a GPS database that did not include LSMs. The transfer function is applied to the permanent shear strain measured in the RSCH test to predict rut depth in situ caused by defined traffic loadings.
- Limited data from RSCH tests on laboratory-molded LSM suggest that an optimum level of dilation exists at which a given LSM would be the most resistant to permanent deformation.
- At equivalent air voids, LSM pavement cores exhibited a mean tensile strength at 5°C (41°F) about 30 psi greater than the mean tensile strength for the few control cores (from conventional asphalt bases) that were available. However, because of the data scatter, this difference cannot be considered statistically significant ($\alpha = 0.05$). This difference may result from the relatively higher surface area of the failure zone because of the larger-sized stones.
- When the Texas gyratory compactor was used to compact 191-mm (7.5-in.)-diameter by 191-mm (7.5-in.)-height specimens from a 38-mm (1.5-in.)-maximum-size mixture, the angle of gyration had to be greater than 1.25 deg to provide the mechanical energy necessary to achieve terminal density (i.e., that density of a pavement expected after 2 to 4 yr of traffic). At the 1.25 deg angle, such high pressures were used that unacceptable aggregate fracture occurred and adequate compaction was still not achieved.
- To ensure actual mixture properties are measured in the laboratory, the smallest specimen dimension (height or diameter) should be at least 4 times larger than the nominal maximum aggregate size. When the smallest dimension is less than 2.5 times the largest aggregate size, aggregate strength masks mixture strength; therefore, specimens with a dimension smaller than this should never be tested. If a specimen dimension between 2.5 and 4 times the largest aggregate size is used, a correction should be applied to the property measured.
- Testing of LSM specimens (or conventional asphalt mixtures) having heights less than 4 times the nominal maximum aggregate size may be acceptable for the SHRP RSST-CH because the height typically does not decrease during the test.
- Laboratory tests on LSM specimens indicate that strength and toughness during monotonic axial compression tests and resistance to permanent deformation during creep tests increase as maximum stone size increases.
- The Superpave 7-day maximum annual pavement temperature appears too high for testing of LSMs. Some of the taller, dense-graded specimens (i.e., height greater than about 3 aggregate diameters) were significantly deformed after 12 hr of conditioning at the specified 7-day maximum temperature.

- LSMs have larger air voids than conventional asphalt mixtures. As a result, when measuring bulk specific gravity according to AASHTO T 166, water may enter these larger voids and yield low, inaccurately calculated air voids content (VTM). One reasonably satisfactory method for measuring bulk specific gravity of large, compacted LSM specimens is to use glass beads in place of water.

5.4 IMPLEMENTABLE PRODUCTS

NCHRP Project 4-18 produced the following five major products recommended for implementation in the future construction of LSM pavements with enhanced rutting resistance:

- A two-level LSM design and analysis method in the form of an AASHTO standard practice (Chapter 2),
- A method for estimating the degree of stone-on-stone contact in compacted LSM specimens (Chapter 3),
- A method for estimating the draindown characteristics of open-graded LSM (Chapter 3),
- A method for accurately measuring the bulk specific gravity of LSM specimens with water-permeable voids (Chapter 3), and
- LSM field construction guidelines (Chapter 4).

APPENDIX A

RESEARCH FINDINGS

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

BACKGROUND

Along with the convenience of the interstate highway system came an unforeseen increase in use of trucks (over other transport methods) to move products and materials across the United States. Advances in tire technology to improve tire service life and fuel efficiency have resulted in increased tire inflation pressures and, more important, higher tire-pavement contact pressures. This "progress" in transportation technology has generated serious problems for asphalt pavements.

In 1984, the Western Association of State Highway and Transportation Officials (WASHTO) (*1*) reported that, in many states, rutting in asphalt pavements "is the most pressing issue presently facing the highway agencies." In a follow-up report in 1988, WASHTO (*2*) listed the main causes of rutting as plastic flow and consolidation under loading. There is considerable evidence that properly designed large-stone, asphalt paving mixes (LSM) will provide better support for these heavy loads than conventional mixes.

R. L. Davis has repeatedly reported that the types of problems described by WASHTO would have been minimized had properly designed LSM pavements been in routine use (*3* through *6*). More and more paving engineers and asphalt paving technologists are beginning to use LSM (*7* through *18*). Monroe (*11*), an engineer with the Iowa DOT, stated the following in 1988:

> I am confident that in the near future, when laws will again be changed to allow still heavier trucks and necessarily higher tire pressures, we will be required to go to still coarser mixes. We should be designing with these coarser mixes now, so we do not get caught with miles and miles of rutting pavement in the near future. We should not just be trying to catch up, but be ahead of the situation. . . . I am also confident that, in the future, agencies will be adopting gradations that have been coined "stone-filled" and contrary to the non-experienced opinion, very little segregation is encountered when actually placing these mixes.

Properly designed LSM resist harsh punching and shear loads well in loading yards where heavy equipment with very high tire-pavement contact pressures is operated (*19*).

LSM are defined herein as asphalt paving mixes having maximum aggregate sizes between 1 and 2.5 in. (25 and 63 mm) and may be dense graded, stone filled, or open graded. These types of mixes probably will be used in the base course of a flexible pavement system. Using LSM changes the basic concept of the paving mix; the traffic is now supported by direct stone-on-stone contact of the larger stones in order to minimize plastic deformation under load. Therefore, the fine-graded riding surface must be relatively thin so that the LSM can be near the surface in a pavement structure.

Because of their inherently lower voids in mineral aggregate (VMA), LSM typically have lower asphalt contents than conventional mixes. The larger aggregates require less crushing energy; therefore, the mixes are less expensive to produce (*20*). Construction operations can be performed faster, thereby reducing the time required to complete a contract and minimizing disruption of traffic (*21*). Conventional asphalt mixes develop their strength from aggregate interlock and viscosity of the binder. Performance of a properly designed LSM depends less on binder quantity and quality than does the performance of conventional paving mixes (*18*).

Stone-matrix asphalt (SMA) paving mixes use a stone-filled mix gradation, but the maximum aggregate size is usually limited to 1.25 in. (32 mm). Specifications and evaluation techniques for SMA are well developed in much of Europe; these mixes resist permanent deformation well. The concept of SMA mixes is based on the maximization of interaction and contact between the coarse aggregate fraction in the asphalt concrete mix. The coarse aggregate or stone fraction provides stability and shear strength to the mix. The large aggregate is held together by the mastic portion of the mix. To perform successfully, an SMA mix must contain the following:

- Large proportions of high-quality, angular coarse aggregate;
- A high content of high-viscosity asphalt cement;
- Relatively large portions of coarse aggregate filler; and
- A fiber, polymer, or fiber-polymer additive to prevent asphalt draindown during transport and placement.

The structural matrix skeleton (formed by the coarse aggregate fraction of the compacted mix) is the basic difference between SMA and traditional, dense-graded mixes. This difference can be seen by comparing gradation charts of the two types of mixes. Traditional, dense-graded mixes contain from 40 to 70 percent coarse aggregate. Internal friction within this type of mix is, consequently, developed by an interaction among the particles of various sizes within the graded mix. This often means that the large grains essentially float in a matrix composed of smaller particles, filler, and asphalt cement. The stability of the mix is primarily controlled by the stability (cohesion and internal friction) of the matrix, which supports the coarse aggregate.

As the aggregate gradation becomes very dense (i.e., low VMA), the resulting mix becomes very sensitive to asphalt binder content. An unstable condition may be reached at binder contents of less than 5 percent. The stability drops abruptly once the binder content fills the available voids. The level and degree of abruptness of stability loss depends on the gradation, shape, and texture of the aggregate and, to some degree, on the rheological properties of the asphalt cement.

The structural skeleton (the continuous phase) in SMA is provided by the coarse aggregate, which constitutes from 70 to 80 percent of the mix (more typically near 80 percent). Such a skeletal matrix can carry the load, even with high binder contents (6 to 7 percent or more), without significant loss of stability. The benefits of the higher binder contents are improved resistance to aging, moisture, and fatigue-cracking and, perhaps, less susceptibility to low-temperature cracking.

LSM, as designed and installed in the United States, do not always perform as desired. A gradation that ensures adequate stone-on-stone contact is important in resisting rutting. Researchers in Kentucky (*22,23*) and other states have demonstrated on heavily trafficked pavements that consolidation and shear failures cannot be prevented by simply putting large stones in an asphalt mix. An LSM composed of a few large stones "floating" in a matrix of conventional aggregate lubricated by warm asphalt is liable to exhibit permanent deformation properties similar to those of the corresponding conventional aggregate mix. Design of heavy-duty LSM should include a gradation that provides stone-on-stone contact of the larger sized aggregates, a maximum allowable quantity of rounded particles (particularly in the sand-sized particle range), and use of the complete aggregate gradation (not scalped) in mix design and quality assurance testing.

LSM materials specifications are needed that address abrasion resistance of aggregate, particle surface roughness, quantity and quality of sand-sized particles, asphalt absorption by aggregates, quantity of minus No. 100 or 200 (150 μm or 75 μm) sieve-sized material, VMA, and voids filled with asphalt. Compaction and testing of laboratory specimens should simulate field compaction and stress conditions as closely as possible. Although the 6-in. (150-mm)-diameter Marshall mix design method for LSM developed by Kandhal (*14*) was an improvement, it lacked these latter attributes. Construction guidelines must address the unique characteristics of LSM. Evidence provided by open-graded mixes, which resist permanent deformation well, indicate that relatively thick asphalt films help reduce segregation and moisture damage and facilitate compaction. These mix characteristics should be considered when designing an LSM, whether it be dense graded, stone filled, or open graded.

STATEMENT OF THE PROBLEM

The preceding concepts are not new (*8,24*) Around the turn of the century, wagons with steel-rimmed wheels were used to transport heavy loads of agricultural goods, building materials, and manufactured products. To help accommodate these high contact pressures, Warren Brothers obtained patents in 1901 and 1903 that specified a top size aggregate of 3 in. (76 mm) and a gradation that maximized density and stability. The high density and the large-sized stone reduced the optimum asphalt content of the mix, thus reducing the cost compared with finer grained mixes. The high stability made it possible to use a soft asphalt and to compact the pavement to less than 2 percent air voids without ensuing deformation of the pavement under the heaviest traffic (*3*). The thick films also provided for excellent durability and resistance to water damage and, together with the soft asphalt, promoted resistance to cracking. With the advent of the high-speed automobile and pneumatic tires (and thus much lower contact pressures than wagon wheels), the requirement for a smooth ride dominated over load-carrying capacity of pavement surfaces. Many paving companies began using small stone sizes to avoid infringement of the Warren Brothers patent (*3*); nevertheless, large stone penetration macadam, and later, plant mix macadam mixes, were fairly popular from the turn of the century to the 1950s. As the industry became more mechanized and production-oriented, however, it found that the finer (0.5-in. [12.7-mm]-maximum) stone sizes were easier to handle. They did not wear the flights in the mixing facility as much, and they produced a uniform, smooth pavement.

Further, contractors resisted the use of coarser, larger stone mixes, and SHAs stopped specifying them because benefits could not be demonstrated under the traffic conditions at that time. Standard mix design procedures (Marshall and Hveem) use 4-in. (102-mm)-diameter molds that cannot handle aggregates larger than 1 in. (25 mm) because of edge effects. This has probably limited the industry to 1-in. (25-mm) materials; therefore, engineers may be designing the mix to fit the mold and not the pavement structural requirements (*8*).

LSM are not without inherent problems and disadvantages such as incomplete coating of coarse aggregates, aggregate segregation, compaction difficulties, aggregate breakage during compaction, and permeable voids in the compacted mat. These problems, however, do not appear to be insurmountable. Guidelines and specifications are needed to delineate mixing and handling procedures that address aggregate coating and segregation (*25,26,27*). Gradation specifications and construction guidelines are needed to aid the contractor in achieving uniform, smooth pavements with adequate densification and minimum permeable voids in dense-graded LSM. Minimum layer thicknesses, aggregate quality specifications, and guidelines on the use of vibratory compactors are needed to eliminate unacceptable aggregate breakage. Additionally, definitive mix design procedures and a suitable mix analysis system need to be developed for LSM to maximize the probability of success. Resistance to permanent deformation is the most important parameter to consider, but resistance to moisture damage and cracking must not be overlooked.

OBJECTIVES

The three primary objectives of this study are as follows:

1. To evaluate the effectiveness of currently used LSM in resisting plastic deformation in asphalt pavements,
2. To develop a design procedure for the optimum proportioning of aggregate and binder for LSM, and
3. To provide guidance on quality assurance testing and constructibility of LSM.

These objectives were addressed by a team of research agencies, including the Texas Transportation Institute (TTI) at Texas A&M University; Brent Rauhut Engineers in Austin, Texas; the University of California at Berkeley (U.C. Berkeley); and the Indiana DOT. This appendix summarizes the results of a research project composed of the following seven formal tasks:

- Task 1—Review and Analyze State-of-the-Art of LSM,
- Task 2—Evaluate Effectiveness of LSM Based on Task 1 Results,
- Task 3—Prepare an Interim Report,
- Task 4—Develop a Mix Design Procedure for LSM,
- Task 5—Develop a Mix Analysis Procedure,
- Task 6—Develop Guidelines for Constructibility of LSM, and
- Task 7—Prepare a Final Report.

RESEARCH APPROACH

These tasks were accomplished through the following actions:

1. Compiling pertinent laboratory and field data on LSM to identify successful design procedures (if any), quality assurance test procedures and criteria, materials specifications, and construction practices;
2. Developing mix design procedures for determining the optimum aggregate gradations and binder contents for dense-graded and open-graded LSM;
3. Developing mix analysis systems to ensure that the LSM will provide the desired performance under the given conditions of traffic and environment;
4. Conducting full-scale, accelerated performance testing (APT) of LSM to determine their rutting behavior;
5. Developing construction guidelines to aid in production, placement, and compaction of LSM; and
6. Documenting the findings and reporting the results to the highway industry in a readily implementable form.

Tasks were achieved, in part, through the testing of pavement cores and laboratory-compacted specimens and by full-scale rutting testing. Two hundred (200) 7.5-in. (191-mm)-diameter pavement cores were obtained from nine LSM pavements in five states, including cores from three conventional control pavements in three states. Tests on the field cores included the following:

- Viscosity of extracted asphalt binders,
- Indirect tension at 5°C (41°F) and 25°C (77°F) at a loading rate of 2 in./min (51 mm/min),
- Resilient modulus at 5°C (41°F) and 25°C (77°F),
- Unconfined axial compression at the Superpave 7-day mean maximum annual temperature (MMAT),
- Unconfined axial compression at an equiviscous temperature,
- Monotonic axial compression to failure, and
- The Superpave repeated shear at constant height (RSCH) test.

The viscosity of the binders was determined to permit creep testing of the cores at temperatures that provided an equal binder viscosity from specimen to specimen. This was done in an attempt to isolate the effects of the aggregates and minimize the effects of the binder. Indirect tension (IDT) and resilient modulus tests were performed to permit a comparison of relative fatigue cracking resistance of LSM and conventional mixes in accordance with the NCHRP asphalt-aggregate mix analysis system (AAMAS). Unconfined compressive creep and recovery testing was performed to provide a comparison of relative resistance to rutting. Because the thickness of some of the LSM pavement layers were less than 2 times the diameter of the largest aggregate particles, monotonic compression testing was conducted to determine the effect of specimen height on compressive strength as an aid to analyzing the creep data. A few RSCH tests were conducted because this procedure simulates the primary distress mechanism responsible for pavement rutting. Procedures and equipment were modified to accommodate the large specimens.

An additional 125 6-in. (150-mm)- and 7.5-in. (191-mm)-diameter laboratory specimens were prepared at heights ranging from 2 to 11 in. (50 to 279 mm) using LSM materials from different state DOTs. As a first step, a laboratory compaction method that reasonably simulated LSM field compaction was developed for these large-sized specimens. Tests performed on the laboratory-compacted specimens included the following:

- Superpave RSCH at 40°C (104°F);
- Unconfined, axial, compressive creep and recovery at 40°C (104°F); and
- Full-scale, loaded wheel rutting tests.

Limited RSCH tests and full-scale, loaded wheel rutting tests were used to estimate the relative resistance to rutting of LSM and conventional asphalt mixes and to confirm that the proposed LSM design and analysis method yielded mix designs with adequate performance characteristics. Creep testing was used to determine the effect of specimen dimensions on measured mix properties. Research team personnel also developed procedures for measuring draindown of open-graded LSM, air voids content of LSM specimens with relatively large voids, and degree of stone-to-stone contact of the coarse aggregate in the compacted mix.

CHAPTER 2

FINDINGS

REVIEW OF CURRENT PRACTICE AND THE EFFECTIVENESS OF LSM

Extent of Use of LSM

Although LSM are not new to the asphalt paving industry, they are unfamiliar to most of the paving contractors in business today. A review of the literature and current practice revealed that the use of LSM (stones larger than 1 in. [25 mm] in diameter) has not been common practice in the last 50 years. In fact, relatively few LSM projects have been built since the early 1900s.

Table A-1 presents the questions and summarizes the responses from a survey of the DOTs of 50 states and Washington, D.C., and the Port Authority of New York and New Jersey conducted in 1992. Table A-2 lists responses from the individual agencies. Table A-1 shows that almost half of the agencies queried have not built any LSM in the last 10 years. More than 90 percent of the LSM built in recent years consisted of dense-graded mixes; most of the remaining LSM were open graded. No stone-filled LSM were located. To address the increase in traffic volume, vehicle loads, and tire pressures, more state DOTs are considering the use of LSM.

Figure A-1 shows the states indicating significant experience with LSM, states with LSM sections in the FHWA Long-Term Pavement Performance (LTPP) database, additional LSM projects visited by the research team, and LSM pavements with adjacent control sections that were visited.

Benefits of LSM

Van der Merwe et al. (*28*) stated that the benefits of LSM are improved structural capacity and improved economy. Other researchers (*19*) have found that pavement cores containing 1-in. (25-mm)-maximum-size aggregates deformed less when subjected to shear loads and were denser and stronger when compared to similar cores containing 0.75-in. (19-mm)-maximum-size aggregate. LSM have a relatively lower VMA, that is, a higher relative volume of aggregate than conventional mixes (*3,29*). This is a key to increasing rutting resistance (*3,19*). A higher relative volume of aggregate yields a relatively higher density and a lower optimum asphalt content of the paving mix; this results in lower materials costs compared to conventional mixes (*7,30*). Asphalt contents of LSM may be more than 30 percent lower than conventional mixes. Production of coarser aggregates means less crushing energy is expended, which, in some cases, may lead to lower aggregate cost (*31,32*). Increased VMA and reduction in aggregate surface area normally results in thicker asphalt films than in conventional mixes, which should provide better resistance to age hardening (*33*) and water damage. However, the most significant cost savings from using LSM should result from greater pavement service life under heavy-duty traffic (*34,35*).

To exploit the full potential of the larger stones, LSM must be properly designed (*22*). The survey of state DOTs revealed that many LSM designs have not provided stone-on-stone contact of the largest aggregates and, thus, have exhibited rutting responses similar to those of conventional mixes. As in any asphalt concrete mix, when stone-on-stone contact of the largest particles is achieved, maximum resistance to permanent deformation is realized and the performance of the paving mix depends less on the quantity and consistency of the asphalt binder (*5,36*); thus, the mix is more forgiving of variations in asphalt content that routinely occur during construction. Further, its high stability permits the use of soft (low viscosity or stiffness) asphalt binders and compaction to less than 2 percent air voids, which minimize cracking without promoting rutting (*3,6,37*). This is particularly beneficial for thick-asphalt, stabilized pavement layers or in severe climates.

Davis (*5*) found that, by changing the top size of the aggregate from 0.75 to 1.5 in. (19 to 37 mm), the bearing capacity of a particular mix could be increased by more than a factor of four (4). Abdulshafi et al. (*38*) saw a two- to threefold increase in unconfined compressive strength and significantly lower creep when LSM were compared with conventional mixes. They also reported much higher resilient moduli and fatigue resistance compared with conventional mixes.

Correctly designed LSM are reported to resist sustained, high-shear, punching loads (e.g., the steel dolly wheels of a loaded semi-trailer) (*29*). This indicates that such a pave-

TABLE A-1 Summary of Responses from Survey of 52 Highway Specifying Agencies

1. How many LSM projects have been conducted in your state in the past 10 years?

   0: 22 agencies 1-5: 10 agencies 6-10: 5 agencies Greater than 10: 15 agencies

   (42%) (19%) (10%) (29%)

2. If used, what has been your agencies experience with the performance of these large stone mixtures versus your "standard" mixes? (Percentages given below represent only those having experience with LSMs.)

   Poor: 0 Same: 6 agencies Good: 14 agencies Unsure: 10 agencies Not Applicable: 22

   (0%) (20%) (47%) (33%) --

3. Is your agency considering the use of LSMs in the future?

   Yes: 41 agencies No: 11 agencies No Response: 1 agencies

   (79%) (19%) (2%)

4. Are you interested in knowing more about LSM?

   Yes: 50 agencies No: 1 agencies No Response: 1 agencies

   (96%) (2%) (2%)

ment would also resist high stresses imparted by trucks with super single tires and even by high-performance aircraft (such as military fighter aircraft), which use extremely high tire pressures.

Claesson (*39*) reported that an LSM built in Sweden contained 2-in. (50-mm)-nominal-maximum-size stone and multigrade asphalt. Falling weight deflectometer measurements indicated the LSM had a stiffness modulus 2.3 to 4.2 times that of their standard paving mix, which contains 1-in. (25-mm)-maximum-size stone and 180 penetration asphalt. The multigrade asphalt was made from a 220 penetration base asphalt gelled to provide a 140 penetration.

Although no supporting research has been reported, when stone-on-stone contact of coarse angular aggregate is achieved, in some instances, it may be possible to use lower quality local materials to fill the voids. For example, 2-in. (50-mm)-maximum-size angular stones could be used as the coarse aggregate. Locally available or inexpensive materials, such as marginal-quality or variable-quality reclaimed asphalt pavement (RAP), aggregate fines (a by-product of the aggregates industry), substandard rounded materials, or combinations thereof could be used to fill the interstices between the larger stones. Research is needed to evaluate this concept.

Problems Sometimes Associated with LSM

LSM are substantially different from conventional, fine-grained asphalt mixes—some DOTs and contractors have had problems during design and construction of LSM pavements. Fudaly et al. (*7*) reported that the two primary concerns expressed by state DOTs contacted were: (1) there is no established AASHTO or ASTM design procedure for asphalt paving mixes containing aggregates larger than 1 in. (25 mm) in diameter and (2) segregation problems often occur when producing and placing LSM on a paving project. However, they concluded that using more and larger stones for improved long-term performance in hot-mix asphalt (HMA) appears to be a sound concept, and further experimentation was recommended.

Individual reports of problems with LSM include equipment wear, increased mixing time requirements (*23*), noise during mixing (drum plant) or drying (batch plant) (*40*), inadequate paddle clearance inside the pug mill (*40*), placement of the coarse mix (*40*), segregation (*30,41,42,43*), resistance to compaction (*7*), aggregate fracture (*7,23*), and water susceptibility (*44*). Although no formal studies have been conducted, agencies did not attribute unusual plant wear to LSM (*43*).

Although LSM are more difficult to compact, heavier rollers (*5*) or a different roller sequence can be used to

TABLE A-2 Written Survey Responses on LSM

Agency	Questions/Answers*				Surveys Sent To	Responses From
	1	2	3	4		
Alabama	D	S	Y	Y	Larry Lockett	Floyd Strickland
Alaska	A	NA	Y	Y	Ray Meketa	Ray Meketa
Arizona	A	NA	Y	Y	Doug Forstie	Don Corum
Arkansas	C	U	Y	Y	Roger Almond	Alan Meadors
California	D	U	Y	Y	Ron Reese	Jack Van Kirk
Colorado	C	G	Y	Y	Dennis Donnaly	R. F. LaForce
Connecticut	A	NA	Y	Y	C. E. Dougan	C. E. Dougan
Delaware	A	NA	N	Y	David R. Mills	Wayne Kling
Florida	A	NA	Y	Y	Larry Smith	Gale C. Page
Georgia	B	S	N	N	Ron Collins	Donald Watson
Hawaii	A	NA	N	Y	Frank K. Uyehara	Frank K. Uyehara
Idaho	A	NA	Y	Y	Bob Smith	Robert M. Smith
Illinois	A	NA	Y	Y	James G. Gehler	James G. Gehler
Indiana	B	G	Y	Y	Ron Walker	Ron Walker
Iowa	A	NA	N	Y	Rod Monroe	Douglas Heins
Kansas	D	G	Y	Y	Rodney Maag	Rodney Maag
Kentucky	D	S	Y	Y	Dwight Walker	Dwight Walker
Louisiana	A	NA	Y	Y	Harold Paul	Harold Paul
Maine	A	NA	N	Y	T.H. Karasopoulos	T.H. Karasopoulos
Maryland	B	G	Y	Y	Harleem Tahir	Harleem Tahir
Massachusetts	A	NA	N	Y	Leo C. Stevens	Leo C. Stevens
Michigan	D	G	Y	Y	Doug Coleman	Doug Coleman
Minnesota	B	G	Y	Y	Roger Olson	Roger Olson
Mississippi	A	NA	Y	Y	Al Crawley	Al Crawley
Missouri	B	U	Y	Y	William L. Trimm	W. L. Trimm
Montana	B	U	Y	Y	Jim Walther	Jim Walther
Nebraska	A	NA	Y	Y	Laird Weishahn	Laird Weishahn
Nevada	A	NA	N	Y	Jack Montrose	Ledo Quilici
N. Hampshire	D	S	Y	Y	Paul Matthews	Paul Matthews
New Jersey	A	NA	Y	Y	Henry Justice	Eileen Connolly
New Mexico	D	G	Y	Y	Douglas I. Hanson	Bob Bass
New York	A	NA	Y	Y	Peter Melas	Wayne J. Brule
North Carolina	D	G	Y	Y	Larry Sams	J. E. Grady, Jr.
North Dakota	B	U	Y	Y	Robert Peterson	Robert Peterson
Ohio	C	U	Y	Y	Bill Christianson	B. Christiansen
Oklahoma	D	U	N	Y	Jack Telford	Jack Telford
Oregon	C	G	Y	Y	Bill Quinn	Jeff Gower
Pennsylvania	D	U	Y	Y	Charles Kline	Charles Kline
Puerto Rico					Luis Castro	--
Rhode Island	A	NA	Y	Y	Francis Manning	Colin A. Franco
South Carolina	A	NA	Y	Y	R. L. Stewart	R. L. Stewart
South Dakota	B	G	Y	Y	Jim Jenssen	L. E. Engbrecht
Tennessee	D	G	Y	Y	Floyd Petty	Floyd Petty

(continued on next page)

TABLE A-2 Written Survey Responses on LSM *(Continued)*

Agency	Questions/Answers*				Surveys Sent To	Responses From
	1	2	3	4		
Texas	C	U	Y	Y	Billy R. Neeley	Paul E. Krugler
Utah	B	G	Y	Y	Wade B. Betenson	H. J. Anderson
Vermont	D	S	Y	Y	W. K. Wheatley	Charles Jerd
Virginia	D	S	Y	Y	W. E. Winfrey	Robert D. Horan
Washington	A	NA	Y	Y	Jim Walter	Jim Walter
Wash., D.C.	A	NA	Y	Y	Horace G. Jones	H. G. Jones
West Virginia	D	G	Y		Robert E. Titus	Gary L. Robson
Wisconsin	A	NA	N	Y	Steve Schober	Jokn Volker
Wyoming	D	G	Y	Y	Tom Atkinson	Tom Atkinson
Port Authority of New York/ New Jersey	C	G	Y	Y	Harry Schmerl	Harry Schmerl

*Question 1 A = 0 Sections
B = 1-5 Sections
C = 6-10 Sections
D = >10 Sections

2 P = Poorer Performance
S = Same Performance
G = Better Performance
U = Unsure
NA = Not Applicable

3&4 Y = Yes
N = No

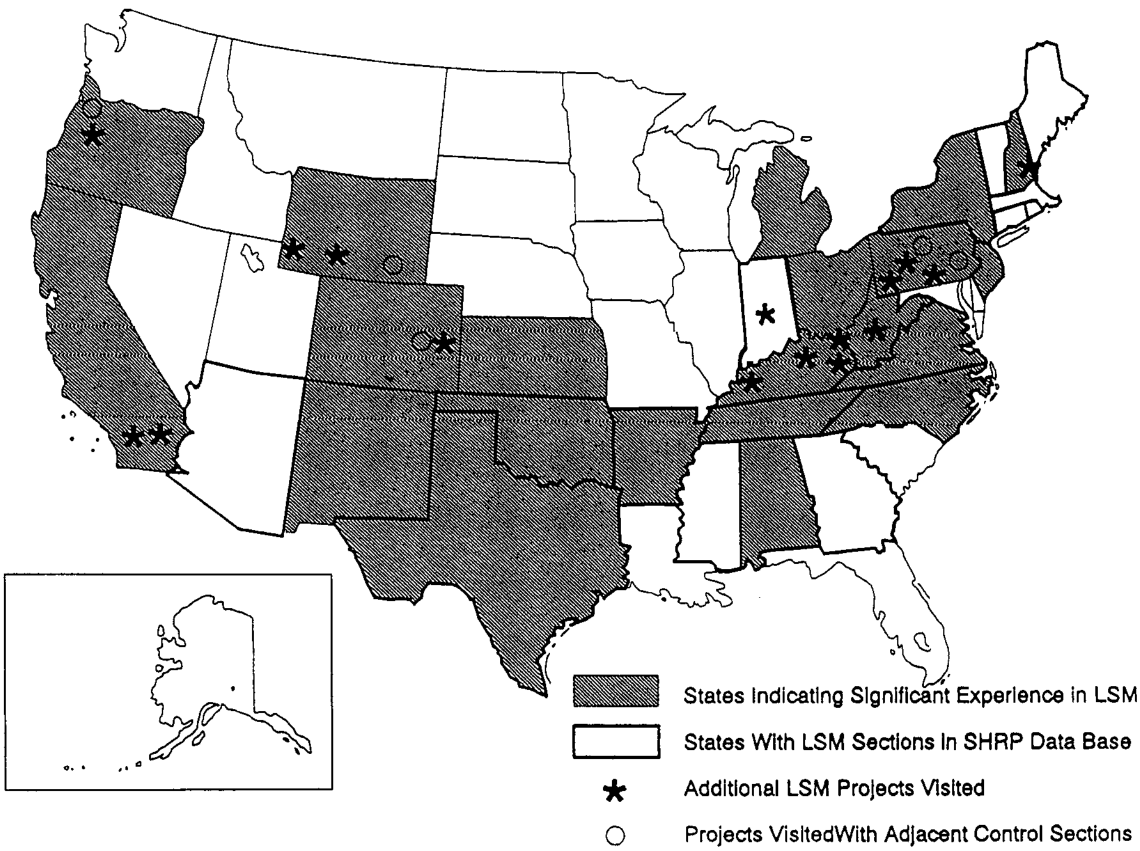

Figure A-1. Map showing LSM usage in the United States.

achieve the desired density. Several agencies report no problems compacting dense LSM (*40,41,45*). Open-graded LSM need only one or two passes to seat the aggregates because density is not normally a specification or performance consideration (*7*).

Aggregate fracture has been associated with the use of vibratory rollers (*7*). A special compactor, the AMIR, designed to apply less localized stress to the mat, was used to compact several open-graded LSM and achieved greater density with less stone fracture than a vibratory roller (*46*).

Acott et al. (*29*) reported the use of a technique that may reduce equipment wear and noise when preparing LSM in a drum mix plant. The center feed system for recycled asphalt pavement (RAP) was used to add the coarse rock to the smaller size aggregate before mixing with the asphalt binder. Ensuring adequate heat transfer to the coarser particles is a possible problem.

Stripping in LSM has been attributed to low asphalt contents (*44*) and inadequate coating of the larger stones. Typically, low asphalt contents in LSM do not result from lower asphalt film thickness but from lower surface area per unit weight. Because of the bridging effect of the large stones in properly designed LSM, they can accommodate thicker asphalt films than conventional dense-graded mixes (*47*).

Some agencies have reported rutting in LSM under heavy loads and high traffic volumes; however, most have explained that these mixes were designed improperly; that is, there was inadequate stone-on-stone contact of the larger aggregate particles. Thus, the LSM performed in a manner similar to a conventional mix (*22,48*). Some agencies have even reported higher cost when placing LSM (*49*). This probably resulted from higher bid prices to cover the contractor's lack of experience with LSM.

Mix Design Procedures for LSM

In some instances, on low-volume roads, formal mix design of LSM for base layers is not performed. Straight pit run or crusher run aggregate is used and asphalt binder is added until the mix "looks about right."

In recent years, it appears that most LSM applied in the United States that have been subjected to mix design were designed using the 6-in. (150-mm) Marshall procedure developed by Kandhal (*14,20*). This method produces a 6-in. (150-mm)-diameter by 3.4-in. (86-mm)-high specimen and is recommended for mixes containing aggregates with a nominal maximum size of 1.5 in. (38 mm). Kandahl increased the Marshall hammer mass and number of blows to produce about the same compaction energy per unit volume as in the usual 4-in. (102-mm)-diameter by 2.5-in. (63-mm)-high Marshall specimen. Although the method is reasonably adequate for determining optimum asphalt content (*23*), Marshall stability is not a good indicator of rutting potential (*50*). Because the procedure does not ensure that the resulting mix will resist rutting, an LSM design method is needed that will produce an aggregate grading that ensures adequate inter-particle contact of the larger stones and, thus, provides the required shear strength for high-volume, heavily loaded, highway pavements (*38*).

Kandhal and Brown (*51*) explained that the coefficient of variation of test data decreased when 6-in. (150-mm)-diameter Marshall specimens were compared with 4-in. (102-mm)-diameter specimens. The data included results from stability, flow, and creep tests.

Kandhal (*14*) reported that when compacting the 6-in. (150-mm) Marshall specimens, 75 to 112 blows per face of the specimen often resulted in fracture of the larger aggregates. Tests have shown that impact-type compaction does not simulate field densification methods. Von Quintus et al. (*52*) and Button et al. (*53*) have demonstrated for conventional mixes that gyratory compaction more closely simulates field compaction than either standard Marshall or rotating base Marshall compaction. Gyratory compaction was selected by the Strategic Highway Research Program (SHRP) for use in the Superpave mix design system (*54*).

The Southern African Bitumen and Tar Association (SABITA) conducted a comprehensive laboratory and field research program over several years (*28,40,47*, and *55* through *65*), which resulted in an informative LSM design manual (*66*). The manual points out that strength and resistance to permanent deformation are achieved from aggregate interlock. Durability is enhanced by thicker asphalt films, which are obtained by using large aggregate. To minimize stripping during wet conditions, the compacted paving mix must either be impermeable or open-graded with interconnected voids, to prevent a build-up of excessive pore water pressure resulting from tires passing over the pavement. Mix designs that trap water should be avoided.

Although workability is not a structural design parameter, it must not be overlooked and should be taken into account during design. Workability generally improves with the use of higher binder and filler contents and finer grading curves. Because LSM are typically used in the base layer, factors such as skid resistance, raveling, noise generation, and flushing do not have to be considered (*66*).

For laboratory mix preparation, SABITA (*66*) recommends mechanical mixing in conjunction with short-term aging. For compaction, its method uses a 6-in. (150-mm)-diameter, rotating-base Marshall hammer with six depressions in the face that provide some kneading action. For mix design and analysis, three criteria are used: (1) volumetric (i.e., air voids content and VMA), (2) IDT testing (i.e., tensile stiffness, strength, and strain at failure), and (3) dynamic creep modulus testing.

In a compaction study conducted during the SHRP asphalt research program, Shuler and Huber (*67*) pointed out that densification is more difficult for mixes with larger aggregate, which is encouraging because rutting should also be more difficult.

Traditionally, mix designers want to incorporate as much asphalt binder as possible without creating a rut-susceptible mix. Cooper et al. (*68* through *70*) and Brown et al. (*71*) developed a new asphalt mix design procedure. It produces as lean a mix as possible, without compromising durability or resistance to cracking, and emphasizes aggregate quality and grading. Its objective is to optimize aggregate gradation and asphalt content to produce a mix that resists permanent deformation. The procedure, which examines mix properties at a range of densities typical of those achieved in the field, enables the engineer to determine the best gradation and binder content for optimum mechanical properties at a target level of compaction. Compaction of the 6-in. (150-mm)-diameter specimens is achieved by using a percentage refusal density test, which is widely used in the United Kingdom (*69*). After extruding, the specimens are trimmed to 2.75 in. (70 mm) in height. Cooper et al. and Brown et al. indicate the procedure will accommodate stone sizes up to 1.57 in. (40 mm) (*72*). Because the method is comprehensive and provides extensive information about the designed mix, it is suitable for high-volume highways; however, it is probably not appropriate for lower volume roads. Its main disadvantage is that it requires the compaction and testing of at least 27 different, fairly large specimens.

Jimenez (*44*) suggested that two tests should be conducted to ensure adequate strength and durability of LSM, particularly for those designed for high-volume, heavily trafficked roads. The strength test should evaluate triaxial shear strength—not just plan or biaxial strength. The durability test should be a torture test to examine resistance to moisture, repetitive loads, or both.

Using data produced in this study, Harvey et al. (*73*) used the Superpave RSCH method to test field cores containing aggregates up to 1.5-in. (38-mm)-nominal-maximum size. Laboratory results along with measured in situ rutting depths and equivalent single-axle load (ESAL) data were used to evaluate the transfer function developed by Sousa and Solaimanian (*74*). The transfer function relates permanent shear strain in the RSCH test to vertical rut depth in situ. The transfer function appears applicable to LSM, despite having been developed from an LTPP general pavement section (GPS) database that did not include LSM. This indicates that the RSCH test may be suitable for specifying the performance of LSM.

Constructibility

LSM have exhibited problems in terms of constructibility in four primary areas as follows:

- Aggregate storage—Segregation during handling of larger aggregate particles in the stockpile at the asphalt mixing plant;
- Mixing plant—Segregation of the aggregate during mix production, particularly during discharge from the surge silo to the haul vehicles, and complete coating of the larger aggregate with asphalt;
- Paving machine—Mix segregation during its delivery to the paver hopper and placement of a smooth mat; and
- Asphalt mat—Fracturing of the coarse aggregate particles under the rolling equipment.

Problems related to constructibility can be avoided by taking appropriate measures (*40,66*). Constructibility generally improves with the use of higher binder contents, stiffer or modified binders, higher filler contents, and finer aggregate grading. Semi-gap-graded mixes tend to segregate more easily when the binder content drops below a certain level (*66*). Fudaly et al. (*7*) recommend that LSM be placed in thicker lifts than conventional mixes and that pneumatic rollers, especially for breakdown rolling, be specified. Tennessee DOT engineers have developed rolling patterns that they claim are critical to proper construction such that, if these rolling patterns are not maintained, fracture of aggregate and, thus, breakage of asphalt films will occur (*7*). Handwork of LSM is difficult and should be minimized; back-spreading should be avoided (*40*).

Because of the increased opportunity for segregation of LSM and the lack of experience with LSM of many U.S. paving contractors, construction guidelines are needed to aid the state DOTs and the contractors in preparing and placing LSM successfully.

Performance of LSM

When Davis (*3*) located asphalt pavements that had given good service for over 50 years, one type was particularly impressive. This type was characterized by soft, lively asphalt binder; a high-volume concentration of aggregate; low air voids; and comparatively large top size stone. When properly designed and constructed, LSM resist heavy, concentrated, high-shear loads without permanent deformation (*29,75*) and also resist cracking (*3,6,37*).

Dense-Graded LSM

Good performance of dense-graded LSM compared with conventional paving mixes has been reported in Colorado (*7*), Kentucky (*23,50*), Minnesota (*29*), Nevada (*7*), Ohio (*38*), Tennessee (*7*), Texas (*7*), Wyoming (*7*), South Africa (*47,56*), the former Soviet Union (*37*), and other areas (*43*). Good performance is typically defined in these reports as resistance to rutting and possibly resistance to cracking, moisture damage, and age hardening of the binder. Fudaly et al. (*7*) stated that because LSM are intended to provide more resistance to rutting, they should be placed as close to the surface of the pavement as possible. The thickness of subsequent finer grained surface mixes should be limited to the minimum necessary for smooth placement and proper compaction.

Researchers in Kentucky (*23,76*) reported significantly less rutting with LSM than with conventional mixes on heavily trafficked roads, even though mix designs were not optimized to obtain the bridging effect between the larger stones. They also found that substituting angular sand for rounded sand significantly reduced subsequent rut depth. Grobler and Rust (*47*) claim that most of the strength of this type of paving mix (without good stone-on-stone contact) depends on the interparticle friction provided by the sand. They further assert that "sharp" (angular) sand improves not only the rutting resistance but the fatigue characteristics.

During field tests of LSM base layers using the Heavy Vehicle Simulator (HVS), Grobler and Rust (*47*) found that rutting was 2 to 20 times greater in conventional pavements than in LSM pavements. When the LSM sections were heated to 40° and 50°C, they still outperformed all the conventional mixes tested in earlier HVS experiments, even though these conventional sections were tested at lower temperatures. This indicates that LSM have a lower temperature sensitivity than conventional mixes and that properly designed LSM depend less on asphalt viscosity for shear strength. Measured stiffnesses of the LSM pavement layers were significantly lower than anticipated, but compared well with subsequent laboratory measurements. Although pavement deflections were sometimes very high (up to 1 mm), no signs of cracking were observed. Indications are, therefore, that these bases are more resistant to fatigue damage than predicted by the computer models (*47*).

Dense bitumen macadam (DBM) mixes, with stone sizes up to 1.5 in. (38 mm) in diameter have been used successfully in the United Kingdom (*47*). They are recommended for pavements carrying traffic in excess of 200 million E80 trucks.

Open-Graded LSM

The strength of open-graded LSM results from stone-on-stone contact. Good performance of open-graded LSM has been reported in Arkansas (*7*), Indiana (*8,9,32*), Tennessee (*8,9,32*) and Wyoming (*7*). No significant rutting has occurred, and there is no evidence of stripping or other materials-related problems. Confined, open-graded LSM resist rutting exceptionally well; however, the purpose of the open-graded LSM placed in Arkansas was to reduce reflection cracking in overlays over portland cement concrete pavements (*7*). Wyoming has experience with 3-in. (75-mm)-maximum-size stones and reports that pneumatic rolling improves aggregate orientation (*7*). Tennessee does not perform a laboratory mix design for its open-graded LSM (*7*).

Fehsenfeld (*9*) reported that, even though the LSM base layers were highly permeable, no stripping was evident and asphalt aging was minimal after 8 to 18 years of service. Grobler and Rust (*47*) attribute these results to the relatively thick asphalt binder films, even though the asphalt content was low (typically less than 2 percent) compared with conventional mixes.

Summary

Published literature reports mostly good results with LSM; however, interviews with highway officials indicate the performance of LSM, particularly in dense-graded mixes, has been mixed. Some LSM have performed extremely well, as shown by no rutting after the application of very heavy traffic loads. Other pavement structures incorporating LSM have experienced permanent deformation. In these instances, the bridging effect of the large aggregate was not achieved and, thus, the large stones were essentially floating in a conventional mix. The performance of such mixes often was similar to that of conventional mixes.

Summary of State DOT Specifications for LSM

To further document the status of LSM applications across the country, a review of state DOT specifications for mixes containing aggregate larger than 1 in. (25 mm) in diameter was conducted in 1992. Of the states surveyed by telephone, only 10 (i.e., Arkansas, Indiana, Mississippi, New Hampshire, New Mexico, Oregon, Pennsylvania, Texas, West Virginia, and Wyoming) incorporated LSM gradations in their standard specifications. A Marshall-based mix design procedure was most common, with stabilities specified at 3,000 to 4,000 lb (1,360 to 1,810 kg) at 112 blows per face; 1,500 to 2,000 lb (680 to 907 kg) at 75 blows per face; or both. Specimen diameter (4 in. or 6 in. [102 mm or 150 mm]) was not noted in the specifications. Only a few states (i.e., Arkansas, Indiana, New Hampshire, and Wyoming) have experience with open-graded or gap-graded LSM. Similarly, only a few states (i.e., Arkansas, Indiana, Kentucky, Ohio, Texas, and Virginia) have experience with a top size aggregate in excess of 1.5 in. (38 mm). Most states specified a low asphalt binder content on the order of 4.5 percent (plus or minus 1 percent), an air voids content of 5 percent (plus or minus 1.5 percent), and a minimum VMA of approximately 15 percent. These specification data are summarized in Table A-3, along with the data specific to each of the test sections reviewed in the field.

Although many states made no specific reference to the control of segregation, several stated that efforts should be made to control segregation and that the engineer had the authority to stop work if the degree of segregation was unacceptable. Arkansas and Colorado, however, provided detailed guidelines on the control of segregation.

TABLE A-3 Summary of Telephone Survey on LSM

STATE	HWY	LOCATION	DATE PLACED	TRAFFIC AADT	% TRUCKS	20 YR KESALS	LEVEL	NEW/ REHAB	INCLUDES CONTROL	RAIN	CONDITION	MARSHALL DESIGN	SPEC	# BLOWS	MIN STAB	FLOW	VMA	% AC		% AIR	
Arkansas	IH 40	LITTLE ROCK	UC				HIGH	REHAB	YES	HIGH		NCAT	R60009	112	2000	0.16" min.	13(MIN)	2.5	7	3	6
													408	75	1800	0.08–0.14"	13(MIN)	3.7	5.7	3	5
California	Rt. 60	RIVERSIDE	1990				HIGH	REHAB	NO	LOW	FAIR	HVEEM	10–1.17(TYP A)					5.7			
	Rt. 60	RIVERSIDE	1990				HIGH	REHAB	NO	LOW			10–1.15(TYP A)					5.7		7	4
	Rt 40	SAN BERNADINO	UC				HIGH	REHAB	NO	LOW			10–1.15(TYP A)								
	Rt 40	SAN BERNADINO	1992				HIGH	REHAB	NO	LOW			10–1.17(TYP A)					5.0		7	4
	Rt 5	SACRAMENTO	1992				HIGH	REHAB	NO	MEDIUM			10–1.14(TYP A)					4.9		7	4
	Rt 5	COLUSA	1992				HIGH	REHAB	NO	LOW			10–1.13(TYP A)					4.8		7	4
Colorado	SH 14	BRIGGSDALE	1990				HIGH	REHAB	NO	LOW	GOOD										
	US 287	LOVELAND	1991				HIGH	REHAB	NO	LOW											
	US 34	YUMA	1992				HIGH	REHAB	NO	LOW											
	SH 14	PAWNEE HILL	1992				HIGH	REHAB	NO	LOW											
	SH 14	AOLT	1990				HIGH	REHAB	NO	LOW	GOOD										
	SH 30	DENVER	1989				HIGH	REHAB	NO	LOW	GOOD										
	IH 70	CEDAR POINT	1989	6800	34.6	11779	HIGH	REHAB	YES	LOW	GOOD		403(TYP G)				16.2	4.7		4.9	
	IH 70	FLAGGLER	1991	5700	32.3	8319	HIGH	REHAB	NO	LOW	FAIR		403(TYP G)				15.2	4.5		4.3	
	SH 59	SIEBERT	1991				LOW	NEW	NO	LOW	GOOD										
Indiana	IH 65		1992				HIGH	NEW	NO	HIGH	EXCELLENT										
	US 31		1991				HIGH	NEW	NO	HIGH	EXCELLENT										
Kentucky												Marshall	Class K Base	112	4000	0.24 max	11.5(min)			5.5	3.5
													Class K Base		2000	0.28 max	11.5(min)			12	6
	US 23	LAWRENCE	1989				HIGH	NEW	NO	HIGH	FAIR		Class K Base		5225	0.16	11.6	3.7		3.5	
	MAD.BYP.	HOPKINS	1990				HIGH	REHAB	NO	HIGH	SATISFACTORY		Class K Base		3410	0.18	13.1	3		6.5	
	US 23	PIKE	1991				HIGH	NEW	NO	HIGH	GOOD		Class K Base		5175	0.2	12.6	3.7		4.5	
	MNT.PKY.	POWELL	1990				HIGH	REHAB	NO	HIGH	SATISFACTORY		Class K Base		5950	0.19	13.1	3.5		4.2	
	IH 75	SCOTT	UC				HIGH	REHAB	NO	HIGH			Class K Base		6170	0.22	12	3.3		4.6	
Michigan																					
	US 127	JACKSON	1991	13300			HIGH	REHAB	NO	HIGH		Marshall			2479	0.1	12.39	3.5		4	
	M 44	GRAND RAPIDS	1991	24800			HIGH	NEW	NO	HIGH					2753	0.11	13.6	4		4	
	M 44	KENT	1991	17100			HIGH	NEW	NO	HIGH					2006	0.11	15	4.7		4	
	CLYDE PARK AV		1991				HIGH	REHAB	NO	HIGH					1868	0.15	15.4	5		4	
	IH 75	GENESEE	1989	42000			HIGH		NO	HIGH					2871	0.11	13.95	4.7		3	
	US 131		1989	72500			HIGH	REHAB	NO	HIGH					1857	0.12	13.49	4.4		3	
New Hampshire													403.11		1290	0.11	15.9	4.7	3.9		
	IH 93	SALEM	1987				HIGH		NO	HIGH								4.7			
	N.H.RT. 101	HAMPTON	UC				HIGH		NO	HIGH											
	N.H.RT. 9	NELSON	UC				HIGH	NEW	NO	HIGH											
New Mexico												Marshall	401.2(GR D)		1800	0.16–0.08				4.5	3.5
	IH 40	MCKINLEY/CIBOLA	1991				HIGH	REHAB	NO	LOW	GOOD				4281	0.15	12.9	4		4.5	
	IH 40	GUADALUPE	1991				HIGH	REHAB	NO	LOW	GOOD				4140	0.13	12.2	4.3		4.5	
	IH 40	CIBOLA/BERNALILLC	1991				HIGH	REHAB	NO	LOW	GOOD				2540	0.11	12.9	3.9		4.5	
	IH 40	THOREAU	1991				HIGH	REHAB	NO	LOW	GOOD				4281	0.15	12.8	4		4.5	
	IH 40	SANTE FE	1991				HIGH	REHAB	NO	LOW	GOOD				3550	0.12	12.9	4.1		4.5	
	IH 40	PREWITT	1992				HIGH	REHAB	NO	LOW					4492	0.13	13.6	4		4.5	
	IH 40	TUCUMCARI	1992				HIGH	REHAB	NO	LOW					3500	0.14	13	4.1		4.5	
	IH 40						HIGH	REHAB	NO	LOW					2605	0.11	12.1	3.6		4.5	
	IH 40	GALLUP	UC				HIGH	REHAB	NO	LOW					4492	0.16	13.2	4.1		4.5	
	US 54	QUAY	1992				HIGH	REHAB	NO	LOW					3500	0.14	13	4.1		4.5	
	US 70	ORGAN	UC				HIGH	REHAB	NO	LOW											
	IH 40	GALLUP	UC				HIGH	REHAB	NO	LOW											
	IH 10	DONA ANA	UC				HIGH	REHAB	NO	LOW											
	IH 40	MCKINLEY	UC				HIGH	REHAB	NO	LOW											

Sections Planned For Inclusion in the Field Studies & Features of Particular Interest

(Continued on next page)

TABLE A-3 Summary of Telephone Survey on LSM *(Continued)*

STATE	HWY	LOCATION	DATE PLACED	TRAFFIC AADT	% TRUCKS	20 YR KESALS	LEVEL	NEW/ REHAB	INCLUDES CONTROL	RAIN	CONDITION	DESIGN	SPEC	MARSHALL # BLOWS	MIN STAB	FLOW	VMA	% AC	% AIR
NY/NJ	JFK AIRPORT		1992									Marshall	FAA I-02564		1800	0.16-0.08	12	5.7	4.3
Port Authority	NEWARK AIRPORT		1992																
	JFK AIRPORT		1992																
	NEWARK		1992																
	NEWARK		UC																
Ohio	IH 71	FRANKLIN	1991				HIGH	REHAB	NO	HIGH									
	IH 71	FAYETTE/MADISON	1992				HIGH	REHAB	NO	HIGH									
Oregon													403					8	4
	McNary Hwy	UMATILLA	1991				HIGH	NEW	NO	LOW									
	OR 58	LANE					HIGH	NEW	NO	HIGH									
	IH 5	LINN/MARION	1991	21100	16.6	57000	HIGH	REHAB	YES	HIGH									
	IH 84	BAKER					HIGH	REHAB	NO	LOW									
	US 97	KLAMATH					HIGH	REHAB	NO	LOW									
	IH 5	LINN		13500	23.3		HIGH	REHAB		HIGH								6.1	
	IH84	MALHEUR					HIGH	REHAB	NO	LOW									
Pennsylvania	SR 30	WESTMORELAND	1986				HIGH	REHAB	NO	HIGH	FAIR								
	IH 70	WASHINGTON	1986				HIGH	REHAB	NO	HIGH	POOR								
	IH 76	MONTGOMERY	1987				HIGH	REHAB	NO	HIGH	EXCELLENT								
	IH 83	YORK	1987				HIGH	REHAB	NO	HIGH	EXCELLENT								
	IH 70	FULTON	1987				HIGH	REHAB	NO	HIGH	GOOD								
	IH 80	JEFFERSON	1987				HIGH	REHAB	NO	HIGH	EXCELLENT								
	IH 76	MONTGOMERY	1985				HIGH	REHAB	YES	HIGH	FAIR								
	IH 80	CLEARFIELD	1989				HIGH	REHAB	YES	HIGH	EXCELLENT								
Tennessee												Marshall		75	1500	0.16-0.08	14		5.5 3
	IH 65	WILLIAM./DAVID	UC	51917			HIGH	REHAB	NO	LOW			307-A					3.8	
	IH 40	DAVIDSON		38410			HIGH	REHAB	NO	LOW			307-A					3.8	
	ST 177	SHELBY																	
	SR 65	ROBERTSON																	
	SH 15	HARDIN																	
	SH 364	CARROLL																	
	SH 53	CLAY																	
	SH 29	SCOTT																	
	SH 386	SUMNER																	
	SH 1	HUMPHREYS																	
	SH 7	MAURY																	
	IH 24	MARION																	
	SH 68	MONROE																	
Texas	IH 40	CARSON	1971	8400	29		HIGH	NEW	NO	LOW	FAIR	Hveem	292 GR 4					3.9	3.4
	US 175	KAUFMAN	1977	15000	13		HIGH	NEW	NO	HIGH	FAIR		292 GR 4					4.3	2.3
	IH 37	LIVE OAK	1979	8100	19		HIGH	NEW	NO	LOW	FAIR		292 GR 4					8.1	12
	US 77	CAMERON	1983	20000	10		HIGH	NEW	NO	LOW	FAIR		292 GR 4					4.7	6.3
West Virginia	RT 85	CLINTON	1991				HIGH	REHAB	NO	LOW									
	RT 85	CLINTON	1990				HIGH	REHAB	NO	LOW									
	RT 85	ROCKLICK	1992				HIGH	REHAB	NO	LOW									
	RT 52	TOSIA	1990				HIGH	REHAB	NO	LOW									
	RT 52	FORT GAY	1991				HIGH	REHAB	NO	LOW									
Wyoming	IH 90	SUNDANCE	1988				HIGH	REHAB	NO	LOW		Marshall							
	COLNY. RD.		1990				HIGH	NEW	NO	LOW									
	IH 80	LARAMIE	1991	6050	37.2		HIGH	REHAB	YES	LOW			703.09 HPMBP					4.3	
	IH 80	RK.SPRINGS	1992				HIGH	REHAB	NO	LOW							16.3	4.8	6.7
	IH 80	WAMSUTTER	1989				HIGH	REHAB	NO	LOW								4.5	

Sections Planned For Inclusion In the Field Studies & Features of Particular Interest

STATE	HWY	LOCATION	DATE PLACED	2		1.5		1.3		1		0.8		0.5		0.4		4		8		10		16		30		40		50		100		200		GRADE
Arkansas	IH 40	LITTLE ROCK	UC	100		92	78					62	48			47	33	34	26			24	16			16	8					10	2	5	1	DENSE
										100	92	97	75	85	55			60	35			45	20					30	10						
California	Rt. 60	RIVERSIDE	1990			100				95	88	85	75			70	55	50	32							24	10							7	2	DENSE
	Rt. 60	RIVERSIDE	1990			100				97	85	88	72			73	52	53	29							27	7							9	0	DENSE
	Rt 40	SAN BERNADINO	UC			100				97	88	88	72			73	52	53	29							27	7							9	0	DENSE
	Rt 40	SAN BERNADINO	1992			100				97	85	88	72			73	52	53	29							27	7							9	0	DENSE
	Rt 5	SACRAMENTO	1992	100		100	88			93	72	85	64			67	45	51	33	41	23					27	9							10	2	DENSE
	Rt 5	COLUSA	1992	100		100	88			93	72	85	64			67	45	51	33	41	23					27	9							10	2	DENSE
Colorado	SH 14	BRIGGSDALE	1990																																	DENSE
	US 287	LOVELAND	1991																																	DENSE
	US 34	YUMA	1992																																	DENSE
	SH 14	PAWNEE HILL	1992																																	DENSE
	SH 14	AOLT	1990																																	DENSE
	SH 30	DENVER	1989																																	DENSE
	IH 70	CEDAR POINT	1989			100				92		78		61		50		40		28				18		12				9		6		4		DENSE
	IH 70	FLAGGLER	1991			100				92		74		64		58		48		34				25		19				14		10		7.2		DENSE
	SH 59	SIEBERT	1991																																	DENSE
Indiana	IH 65		1992																																	OPEN
	US 31		1991																																	OPEN
Kentucky				100		98	80			90	67	80	56	72	43	60	37	45	22	35	14			25	8	18	6			13	4	9	3	6	2	
				100		100	80			85	65	70	50	60	40	50	30	30	15	18	8			15	5	12	4			10	3	6	2	4	0	
	US 23	LAWRENCE	1989	100		99				86		75		58		50		29		21				15		10				8		5.4		3.5		DENSE
	MAD.BYP.	HOPKINS	1990	100		96				74		59		46		40		23		14				10		7				5		4		2.5		DENSE
	US 23	PIKE	1991	100		94				83		74		59		50		32		19				12		10				6		5		3.5		DENSE
	MNT.PKY.	POWELL	1990	100		97				80		69		52		45		40		24				15		11				8		6		4.5		DENSE
	IH 75	SCOTT	UC	100		98				85		74		60		54		33		23				16		12				9		7		5.5		DENSE
Michigan						100				87		77		68		62		45	30	37	22			28	18	20	8			15	5	15	4	6	3	
						100				90	80	85	70	70	55	65	50	50	35	40	25					25	10							8	4	
						100				95	80	95	70			80	55			55	30					35	15							8	3	
						100				85	75	75	65			55	45			40	30					30	20							7	3	
	US 127	JACKSON	1991			100				85		68		60		60		42		32				26		18				10		6.2		3.7		DENSE
	M 44	GRAND RAPIDS	1991			100				89		75		65		55		43		34						20								5.5		DENSE
	M 44	KENT	1991			100				88		74		56		51		38		27						15								5.3		DENSE
	CLYDE PARK AV		1991			100				86		74		66		55		41		33						17								5.2		DENSE
	IH 75	GENESEE	1989			100				88		79		72		67		55		46						25								5		DENSE
	US 131		1989			100				82		66		55		53		45		38						25								3		DENSE
New Hampshire								100	95	95	81	80	67	62	48	54	40	40	26			27	19					12	4					4	0	
	IH 93	SALEM	1987					100		94		83		68		62		43				28						9						2		GAP
	N.H.RT. 101	HAMPTON	UC			100		93		89		78		65		53		36				26						9						1.4		GAP
	N.H.RT. 9	NELSON	UC																																	GAP
New Mexico								100		98	86	90	70	80	60	70	50	54	34			42	22					22	8					7	3	
	IH 40	MCKINLEY/CIBOLA	1991							98		88		70		58		43				26						12						6.5		DENSE
	IH 40	GUADALUPE	1991					100		96		80		62		54		39				30						19						7.5		DENSE
	IH 40	CIBOLA/BERNALILLC	1991							95		82		64		53		38				26						13						5.4		DENSE
	IH 40	THOREAU	1991					100		98		88		70		58		43				26						12						6.5		DENSE
	IH 40	SANTE FE	1991					100		97		83		66		56		39				26						14						6.9		DENSE
	IH 40	PREWITT	1992					100		94		79		65		54		45				32						18						6		DENSE
	IH 40	TUCUMCARI	1992					100		95		88		76		62		42				31						17						5.7		DENSE
	IH 40						100		94		72		65		58		40				31						15						5.1		DENSE
	IH 40	GALLUP	UC					100		93		75		63		51		42				32						19						7		DENSE
	US 54	QUAY	1992					100		96		90		80		65		44				32						17						5.9		DENSE
	US 70	ORGAN	UC																																	DENSE
	IH 40	GALLUP	UC																																	DENSE
	IH 10	DONA ANA	UC																																	DENSE
	IH 40	MCKINLEY	UC																																	DENSE

Sections Planned For Inclusion in the Field Studies & Features of Particular Interest

(Continued on next page)

TABLE A-3 Summary of Telephone Survey on LSM *(Continued)*

STATE	HWY	LOCATION	DATE PLACED		1.5	1.3	1	0.8	0.5	0.4	4	8	10	16	30	40	50	100	200	GRADE
NY/NJ	JFK AIRPORT		1992			100	95 90	84 70		60 46	44 34	31 23			14 10			12 4	7 3	
Port Authority	NEWARK AIRPORT		1992																	
	JFK AIRPORT		1992																	
	NEWARK		1992																	
	NEWARK		UC																	
Ohio	IH 71	FRANKLIN	1991																	DENSE
	IH 71	FAYETTE/MADISON	1992																	DENSE
Oregon					100 99	100 95		87 74					30 22			18 8			6 2	
	McNary Hwy	UMATILLA	1991																	DENSE
	OR 58	LANE																		DENSE
	IH 5	LINN/MARION	1991		100	97	91	81	67	59			26			10			3.7	DENSE
	IH 84	BAKER																		DENSE
	US 97	KLAMATH																		DENSE
	IH 5	LINN			142	132	117	101	82	70	49		28			12			4.2	DENSE
	IH84	MALHEUR																		DENSE
Pennsylvania	SR 30	WESTMORELAND	1986																	DENSE
	IH 70	WASHINGTON	1986																	DENSE
	IH 76	MONTGOMERY	1987																	DENSE
	IH 83	YORK	1987																	DENSE
	IH 70	FULTON	1987																	DENSE
	IH 80	JEFFERSON	1987																	DENSE
	IH 76	MONTGOMERY	1985																	DENSE
	IH 80	CLEARFIELD	1989																	DENSE
Tennessee				100	100 75			70 45		55 30	40 20	30 10			20 5			12 0	6 0	
	IH 65	WILLIAM./DAVID	UC																	
	IH 40	DAVIDSON		100	99			58		48	27	18			9.7			6.1	4.2	DENSE
	ST 177	SHELBY		100	96			63		41	27	18			9.2			5.2	4	DENSE
	SR 65	ROBERTSON																		
	SH 15	HARDIN																		
	SH 364	CARROLL																		
	SH 53	CLAY																		
	SH 29	SCOTT																		
	SH 386	SUMNER																		
	SH 1	HUMPHREYS																		
	SH 7	MAURY																		
	IH 24	MARION																		
	SH 68	MONROE																		
Texas	IH 40	CARSON	1971	100	98		86	73	58	51	39		32			20			5.5	DENSE
	US 175	KAUFMAN	1977		100		84	76	69	66	57		38			23			6.5	DENSE
	IH 37	LIVE OAK	1979	100	97		80	69	59	54	46		42			38			8.5	DENSE
	US 77	CAMERON	1983	100	96		86	78	66	57	42		33			26			6.5	DENSE
West Virginia	RT 85	CLINTON	1991		100															DENSE
	RT 85	CLINTON	1990		100															DENSE
	RT 85	ROCKLICK	1992		100															DENSE
	RT 52	TOSIA	1990		100															DENSE
	RT 52	FORT GAY	1991		100															DENSE
Wyoming	IH 90	SUNDANCE	1988																	DENSE
	COLNY. RD.		1990																	DENSE
	IH 80	LARAMIE	1991			100	100 90	90 70	80 50		55 35	40 20			25 10				7 2	OPEN
	IH 80	RK.SPRINGS	1992		100		95	76	58	44	24	14			7				2.2	OPEN
	IH 80	WAMSUTTER	1989			100	96	78	56	48	40	23			7.5				2.7	OPEN

Sections Planned For Inclusion In the Field Studies & Features of Particular Interest

STATE	HWY	LOCATION	DATE PLACED	2	1.5		1.3		1		0.8		0.5		0.4		4		8		10		16		30		40		50		100		200		GRADE
Arkansas	IH 40	LITTLE ROCK	UC	100	92	78					62	48			47	33	34	26			24	16			16	8					10	2	5	1	DENSE
									100	92	97	75	85	55			60	35			45	20					30	10							
California	Rt. 60	RIVERSIDE	1990		100				95	88	85	75			70	55	50	32							24	10							7	2	DENSE
	Rt. 60	RIVERSIDE	1990		100				97	85	88	72			73	52	53	29							27	7							9	0	DENSE
	Rt 40	SAN BERNADINO	UC		100				97	88	88	72			73	52	53	29							27	7							9	0	DENSE
	Rt 40	SAN BERNADINO	1992		100				97	85	88	72			73	52	53	29							27	7							9	0	DENSE
	Rt 5	SACRAMENTO	1992	100	100	88			93	72	85	64			67	45	51	33	41	23					27	9							10	2	DENSE
	Rt 5	COLUSA	1992	100	100	88			93	72	85	64			67	45	51	33	41	23					27	9							10	2	DENSE
Colorado	SH 14	BRIGGSDALE	1990																																DENSE
	US 287	LOVELAND	1991																																DENSE
	US 34	YUMA	1992																																DENSE
	SH 14	PAWNEE HILL	1992																																DENSE
	SH 14	AOLT	1990																																DENSE
	SH 30	DENVER	1989																																DENSE
	IH 70	CEDAR POINT	1989		100				92		78		61		50		40		28				18		12				9		6		4		DENSE
	IH 70	FLAGGLER	1991		100				92		74		64		58		48		34				25		19				14		10		7.2		DENSE
	SH 59	SIEBERT	1991																																DENSE
Indiana	IH 65		1992																																OPEN
	US 31		1991																																OPEN
Kentucky				100	98	80			90	67	80	56	72	43	60	37	45	22	35	14			25	8	18	6			13	4	9	3	6	2	
				100	100	80			85	65	70	50	60	40	50	30	30	15	18	8			15	5	12	4			10	3	6	2	4	0	
	US 23	LAWRENCE	1989	100	99				86		75		58		50		29		21				15		10				8		5.4		3.5		DENSE
	MAD.BYP.	HOPKINS	1990	100	96				74		59		46		40		23		14				10		7				5		4		2.5		DENSE
	US 23	PIKE	1991	100	94				83		74		59		50		32		19				12		10				6		5		3.5		DENSE
	MNT.PKY.	POWELL	1990	100	97				80		69		52		45		40		24				15		11				8		6		4.5		DENSE
	IH 75	SCOTT	UC	100	98				85		74		60		54		33		23				16		12				9		7		5.5		DENSE
Michigan					100				87		77		68		62		45	30	37	22			28	18	20	8			15	5	15	4	6	3	
					100				90	80	85	70	70	55	65	50	50	35	40	25					25	10							8	4	
					100				95	80	95	70			80	55			55	30					35	15							8	3	
					100				85	75	75	65			55	45			40	30					30	20							7	3	
	US 127	JACKSON	1991		100				85		68		60		60		42		32				26		18				10		6.2		3.7		DENSE
	M 44	GRAND RAPIDS	1991		100				89		75		65		55		43		34						20								5.5		DENSE
	M 44	KENT	1991		100				88		74		56		51		38		27						15								5.3		DENSE
	CLYDE PARK AV		1991		100				86		74		66		55		41		33						17								5.2		DENSE
	IH 75	GENESEE	1989		100				88		79		72		67		55		46						25								5		DENSE
	US 131		1989		100				82		66		55		53		45		38						25								3		DENSE
New Hampshire							100	95	95	81	80	67	62	48	54	40	40	26			27	19					12	4					4	0	
	IH 93	SALEM	1987				100		94		83		68		62		43				28						9						2		GAP
	N.H.RT.101	HAMPTON	UC		100		93		89		78		65		53		36				26						9						1.4		GAP
	N.H.RT. 9	NELSON	UC																																GAP
New Mexico							100		98	86	90	70	80	60	70	50	54	34			42	22					22	8					7	3	
	IH 40	MCKINLEY/CIBOLA	1991						98		88		70		58		43				26						12						6.5		DENSE
	IH 40	GUADALUPE	1991				100		96		80		62		54		39				30						19						7.5		DENSE
	IH 40	CIBOLA/BERNALILLC	1991						95		82		64		53		38				26						13						5.4		DENSE
	IH 40	THOREAU	1991				100		98		88		70		58		43				26						12						6.5		DENSE
	IH 40	SANTE FE	1991				100		97		83		66		56		39				26						14						6.9		DENSE
	IH 40	PREWITT	1992				100		94		79		65		54		45				32						18						6		DENSE
	IH 40	TUCUMCARI	1992				100		95		88		76		62		42				31						17						5.7		DENSE
	IH 40						100		94		72		65		58		40				31						15						5.1		DENSE
	IH 40	GALLUP	UC				100		93		75		63		51		42				32						19						7		DENSE
	US 54	QUAY	1992				100		96		90		80		65		44				32						17						5.9		DENSE
	US 70	ORGAN	UC																																DENSE
	IH 40	GALLUP	UC																																DENSE
	IH 10	DONA ANA	UC																																DENSE
	IH 40	MCKINLEY	UC																																DENSE

Sections Planned For Inclusion In the Field Studies & Features of Particular Interest

(Continued on next page)

TABLE A-3 Summary of Telephone Survey on LSM *(Continued)*

STATE	HWY	LOCATION	DATE PLACED		1.5	1.3	1	0.8	0.5	0.4	4	8	10	16	30	40	50	100	200	GRADE
NY/NJ Port Authority	JFK AIRPORT		1992			100	95 90	84 70		60 46	44 34	31 23			14 10			12 4	7 3	
	NEWARK AIRPORT		1992																	
	JFK AIRPORT		1992																	
	NEWARK		1992																	
	NEWARK		UC																	
Ohio	IH 71	FRANKLIN	1991																	DENSE
	IH 71	FAYETTE/MADISON	1992																	DENSE
Oregon					100 99	100 95		87 74					30 22			18 8			6 2	
	McNary Hwy	UMATILLA	1991																	DENSE
	OR 58	LANE																		DENSE
	IH 5	LINN/MARION	1991		100	97	91	81	67	59			26			10			3.7	DENSE
	IH 84	BAKER																		DENSE
	US 97	KLAMATH																		DENSE
	IH 5	LINN			142	132	117	101	82	70	49		28			12			4.2	DENSE
	IH84	MALHEUR																		DENSE
Pennsylvania	SR 30	WESTMORELAND	1986																	DENSE
	IH 70	WASHINGTON	1986																	DENSE
	IH 76	MONTGOMERY	1987																	DENSE
	IH 83	YORK	1987																	DENSE
	IH 70	FULTON	1987																	DENSE
	IH 80	JEFFERSON	1987																	DENSE
	IH 76	MONTGOMERY	1985																	DENSE
	IH 80	CLEARFIELD	1989																	DENSE
Tennessee				100	100 75			70 45		55 30	40 20	30 10			20 5			12 0	6 0	
	IH 65	WILLIAM./DAVID	UC																	
	IH 40	DAVIDSON		100	99			58		48	27	18			9.7			6.1	4.2	DENSE
	ST 177	SHELBY		100	96			63		41	27	18			9.2			5.2	4	DENSE
	SR 65	ROBERTSON																		
	SH 15	HARDIN																		
	SH 364	CARROLL																		
	SH 53	CLAY																		
	SH 29	SCOTT																		
	SH 386	SUMNER																		
	SH 1	HUMPHREYS																		
	SH 7	MAURY																		
	IH 24	MARION																		
	SH 68	MONROE																		
Texas	IH 40	CARSON	1971	100	98		86	73	58	51	39		32			20			5.5	DENSE
	US 175	KAUFMAN	1977		100		84	76	69	66	57		38			23			6.5	DENSE
	IH 37	LIVE OAK	1979	100	97		80	69	59	54	46		42			38			8.5	DENSE
	US 77	CAMERON	1983	100	96		86	78	66	57	42		33			26			6.5	DENSE
West Virginia	RT 85	CLINTON	1991		100															DENSE
	RT 85	CLINTON	1990		100															DENSE
	RT 85	ROCKLICK	1992		100															DENSE
	RT 52	TOSIA	1990		100															DENSE
	RT 52	FORT GAY	1991		100															DENSE
Wyoming	IH 90	SUNDANCE	1988																	DENSE
	COLNY. RD.		1990																	DENSE
	IH 80	LARAMIE	1991			100	100 90	90 70	80 50		55 35	40 20			25 10				7 2	OPEN
	IH 80	RK.SPRINGS	1992		100		95	76	58	44	24	14			7				2.2	OPEN
	IH 80	WAMSUTTER	1989			100	96	78	56	48	40	23			7.5				2.7	OPEN

Sections Planned For Inclusion in the Field Studies & Features of Particular Interest

In the Arkansas specification under "paver operations," the following specifications are provided:

> A level of mix, 18–24 in. (457–610 mm), shall be maintained over the slat conveyor in the hopper to prevent coarse material in wings from reentering the mix. Wings shall not be dumped until the end of the day's production and that material shall be wasted. The paver shall be operated at the slowest speed possible that will accommodate production. Stop and go paving operations shall not be permitted. The paving equipment shall place the mix without segregation or tearing within the specified tolerance and true to the line grade, and crown indicated on the plans. In order to achieve a continuous spreading operation, the speed of the paver should be coordinated with the production of the mixing plant and the haul trucks.

The spirit of this specification is clear; however, some "stop and go" operations must occur. If rapid stop and rapid start procedures are followed, problems at these points should be minimized.

The Colorado specifications state the following:

> . . . the Contractor should prepare a method statement outlining the steps to be taken to minimize segregation of aggregates in hot bituminous large stone pavements. The statement should be submitted to the Engineer prior to beginning the pavement operation. When the Engineer determines there are large amounts of segregation, the paving operations shall stop and the cause of the segregation corrected before paving operations will be allowed to resume.

Summary of Interviews with State DOT Personnel—1992

In 1992, very few sections of LSM could be found that had been in place longer than 5 years. The use of LSM is typically limited to those areas previously identified as high-volume truck routes. In addition, because very few sections have been built next to a standard hot-mix pavement, direct comparisons of LSM with conventional mixes were even more limited. Most of the sites reviewed were rehabilitation jobs using dense-graded LSM. Few state DOTs use a gap-graded or open-graded mix design, and none use a stone-filled mix design. Very few state DOTs have current experience (within the past 10 years) with LSM using top size aggregate of greater than 1.5 in. (38 mm).

Compaction of LSM is often difficult because of insufficient experience or knowledge in constructing thick lifts (greater than 2 in. [50 mm]) as required by LSM. The proper rolling patterns and roller sequences must be used to ensure adequate compaction. Inspection of cores obtained as part of this study indicate that stone-on-stone contact was seldom obtained. Segregation is the most common construction-related problem associated with LSM. In every state visit and almost every report reviewed, segregation was identified as one of the main problems associated with LSM. The state DOTs identified deficiencies in design and construction practices for LSM as the primary obstacles in realizing the superior performance of LSM.

To address the substantial increases in traffic volume, vehicle loads, and tire pressures, more state DOTs are considering designing and constructing pavement structures that contain one or more LSM layers. Although interest in LSM is growing, current field experience is insufficient to determine whether LSM are outperforming conventional mixes. Further, a simple mix design procedure and guidelines on how to prevent segregation of LSM during construction will be necessary before the use of LSM spreads. Pavement-specifying agencies need a standard mix design procedure and construction guidelines focused specifically on LSM.

Analysis of the LSM Sections in the LTPP Database

The researchers reviewed the LTPP database to identify LSM test sections already being monitored as part of that program. A review of the gradations for the asphalt layers revealed 51 test sections containing material retained on the 1-in. (25-mm) sieve (Table B-1 and Figure B-1, Appendix B, summarize the data for these test sections). Most LSM in the LTPP database are dense-graded, and the maximum size aggregate is 1.5 in. (38 mm). This is comparable to the findings of the surveys of state DOTs conducted as part of this study.

In reviewing the performance of these test sections, caution should be exercised. Although there are 51 sections containing a layer with materials greater than 1 in. (25 mm) in diameter, making direct comparisons of these sections with each other or with sections that do not contain LSM, can be misleading. When the variations in subgrade type, base thickness and stiffness, traffic, and properties of the other HMA concrete (HMAC) layers were taken into account, it was clear that even these 51 test sections were not sufficient to enable the researchers to discern how an LSM affects performance. For example, comparing rutting depths of test sections in the LTPP database (the 51 test sections containing an LSM versus the roughly 200 conventional mix test sections) and ignoring all other factors, Figure B-1c seems to indicate that LSM promote rutting; however, placement of these LSM sections is typically limited to those areas where higher traffic is anticipated. To minimize this bias, when the traffic levels are considered, the LSM test sections appear to have a slight edge (see Figure B-1d).

This example highlights just one of the factors (i.e., traffic) that can confound any conclusions that can be drawn from these LSM sections. There are not sufficient LSM sections with corresponding control sections (in the LTPP database or from other current field experience) to establish all of the factors affecting the permanent deformation and to what extent they are affected by the use of LSM.

DESCRIPTION OF TESTS AND RESULTS ON PAVEMENT CORES

Selected LSM pavements were cored using an 8-in. (203-mm)-outside-diameter-core barrel. A brief description of the pavement layers and conditions at the surface is given in Table A-4. During the survey of the state DOTs, only five LSM projects with control sections were identified; cores were collected from three of these projects (see Table A-4).

These LSM and control pavement cores were subjected to a series of laboratory tests in order to estimate the relative ability of currently used LSM to resist plastic deformation. The tests consisted of the following: (1) unconfined compressive creep at the 7-day MMAT for pavement; (2) unconfined compressive creep at temperatures giving an equal viscosity for the asphalts in the cores; (3) confined compressive creep at the 7-day MMAT; (4) IDT at 5°C (41°F) and 25°C (77°F); and (5) resilient modulus at 5°C (41°F) and 25°C (77°F). The 7-day MMATs were calculated for the respective locations from which the cores were obtained. Results of these tests are described in the following sections.

Viscosity of Recovered Binders

Asphalt was extracted and recovered from one or more of the 7.5-in. (191-mm)-diameter pavement core specimens from each location. Viscosities were measured using the SHRP dynamic shear rheometer, and master curves were produced. Viscosities at 77°F and 140°F (25°C and 60°C) are reported in Table A-5.

As a precaution, gel permeation chromatography tests were conducted on the recovered binders to ensure that no solvent was retained from the extraction procedure. Retention of solvent in recovered asphalts can give misleading results. None of the extracted binders were found to contain solvent.

Viscosity was plotted as a function of temperature to determine the appropriate temperature for creep testing of cores at a common (equal) asphalt viscosity. For this element of the work, a viscosity of 1 by 10^3 Pa • s (4.1 by 10^4 poise) was selected for creep testing. To obtain this viscosity during creep testing, specimen temperatures from 45°C (113°F) to 60°C (140°F) were used. The goal was to remove or at least minimize the relative effect of the asphalt binder on creep and recovery test results in order to better assess the relative influence of the aggregates. This is not a perfect solution because it is still impossible to assess the relative effects of steric hardening of the binders or filler quantity and quality in the mastic of the mixes.

IDT and Resilient Modulus Tests

A section of the pavement layer of interest was sawn from selected 7.5-in. (191-mm)-diameter cores. IDT tests at 5°C (41°F) and 25°C (77°F) and a loading rate of 2 in. per min (51 mm per min) were conducted on LSM and control pavement cores. IDT tests were conducted using a servo-hydraulic, closed-loop MTS testing machine. The IDT data are summarized in Table A-6. These tests were performed primarily to determine the proper stresses for subsequent creep tests on corresponding core specimens. Creep tests are typically performed at about 10 to 25 percent of the specimen's tensile strength.

Figure A-2 shows that, at equivalent air voids, the LSM exhibit a mean tensile strength at 5°C (41°F) about 30 psi (207 kPa) greater than the mean tensile strength for the limited number of corresponding control cores that were available. However, because of data scatter, this difference cannot be considered statistically significant (α = 0.05). At 25°C (77°F), there is no statistical difference in tensile strength between the LSM and control cores (see Figure A-3).

These tensile strength values are fairly low compared with those in tensile strength databases for conventional mixes tested at these temperatures. This difference could partially result from sample configuration, because all conventional mix specimens in the database used for comparison were 4 in. (102 mm) in diameter. The IDT test is not a pure tension test, and a smaller diameter probably will yield higher tensile strengths and moduli.

Figure A-4 shows only a 10 percent increase in tensile strength with approximately an order of magnitude increase in binder viscosity for the LSM. Tensile strength of the limited number of control mixes depends much more on binder viscosity (see Figure A-4). Again, data scatter means this difference is not statistically significant (α = 0.05).

Tensile strength as a function of temperature is plotted in Figure A-5 for the control cores and their corresponding LSM cores. Although the data are very limited, the slopes of these curves are statistically different (α = 0.05).

Resilient modulus was measured using a servo-hydraulic, closed-loop testing machine. The IDT data were used with total resilient modulus data (see Table A-7) to predict the relative resistance of LSM to fatigue cracking following the procedures set forth by the NCHRP AAMAS (*52*) (see Figure A-6). Only LSM data are shown in Figure A-6. The plot shows considerable data scatter. This is not surprising because the pavement cores vary considerably in composition and age. In addition, both IDT and resilient modulus tests produce biaxial stress fields and are, therefore, sensitive to binder viscosity, filler-asphalt ratio, and air voids content—all of which also varied considerably in the cores tested.

The Oregon.LSM cores contain the softest asphalt and have the highest asphalt content (see Table A-6)—this probably contributed to the highest resistance to fatigue cracking according to AAMAS. The Pa.Fulton specimens have the

TABLE A-4 **Description of Pavements Cored and 191-mm (7.5-in.) Cores Received**

Location	Nominal Maximum Aggregate Size, in.	Type of Grading	Layer Thickness, inch	Layer No. (from top) & Depth of LSM, inch	Distress in Pavement Surface	Rut Depth, inch	No. Cores Received	Date Placed
Cedar Point, CO, IH 70 - MP 348	1.50	Dense	3.25	1/0	Extensive Fatigue Cracking	0.1-0.8	16	1989
Cedar Point, CO, IH70 - MP 347 - **Control**	0.75	Dense	1.5	1/0	Extensive Fatigue Cracking	<0.1	16	1989
Flaggler, CO, IH70 - 395	1.50	Dense	3.75	2/2	Segregation w/ Minor Popouts	0	16	1989
Flaggler, CO, IH70 - 374	1.50	Dense	3.0	1/0	Some Longitudinal Cracking + Minor Patching	0-0.2	16	1989
Lawrence, KY, US23	1.50	Dense	12	2/1	None	0.1-0.5	16	1989
Linn/Marion, OR, IH5	1.25	Dense	3.0	2/1.5	None	0.1	16	1991
Linn/Marion, IH5 - **Control**	0.75	Dense	3.0	2/1.5	None	0.1	8	1991
Fulton, PA, IH70	1.50	Dense	2.5	2/1.5	Minor Flushing	0.1-0.2	16	1987
Clearfield, PA, IH80	1.50	Dense	5.0	2/1.5	None	0-0.2	32	1989
Laramie, WY, IH80	1.25	Open	3.0	2/1	None	0-0.2	16	1990
Laramie, WY, IH80 - **Control**	0.75	Dense	3.0	2/0.75	None	0-0.2	16	1990
Rock Springs, WY, IH80	1.50	Open	2.5	2/0.75	None	0	16	1992

TABLE A-5 Results from Viscosity Tests on Asphalts Extracted from Pavement Cores

Location	Viscosity at 25°C, Pa·s x 10^6	Viscosity at 60°C, Pa·s	Temperature for Viscosity of 4.1 x 10^3 Pa·s, °C (°F)
Cedar Point, CO, IH 70 - 348 [1]	1.570	1377	53 (127)
Cedar Point, CO, IH70-347 - Control	1.620	971	51 (124)
Flaggler, CO, IH70 - 395	2.370	1939	54 (129)
Flaggler, CO, IH70 - 374	9.370	3657	60 (140)
Lawrence, KY, US23	2.370	2538	56 (133)
Linn/Marion, OR, IH5	0.420	368	45 (113)
Linn/Marion, IH5 - Control	0.452	391	45 (113)
Fulton, PA, IH70	0.469	639	48 (118)
Clearfield, PA, IH80	0.445	570	47 (117)
Laramie, WY, IH80	1.830	911	52 (125)
Laramie, WY - Control	6.670	2844	58 (136)
Rock Springs, WY, IH80	0.762	490	49 (120)

[1] The gradation of the aggregates in these materials is given in Table 3.

next softest asphalt and show relatively good fatigue cracking resistance at the lowest test temperature.

Although the Co.CP.348 specimens showed good resistance to fatigue cracking (see Figure A-6), the LSM pavement, which was the surface course, exhibited extensive alligator cracking in the outside wheelpath within 4 years of its construction. However, the source of this cracking is unknown and could result from a structurally soft underlying layer. The Wy.RkSprings cores contain relatively soft, polymer-modified asphalt and a fairly high asphalt content for an open-graded LSM. The resulting thick asphalt films probably contributed to the good resistance to fatigue cracking (see Figure A-6). The Co.Flag.374 cores contained the hardest asphalt, and according to Figure A-6, have a high probability of fatigue cracking; in fact, 3 years after construction, this pavement was exhibiting random cracking.

Figure A-7 shows that resilient moduli of most of the LSM cores fall within the values recommended by the NCHRP AAMAS (*52*). At 25°C (77°F), resilient moduli of three cores (i.e., Pa.Fulton, Co.Flag.374, and Oregon.LSM) fall within the area of Figure A-7 where AAMAS indicates the total resilient modulus is too high.

Unconfined Compressive Creep at 7-Day MMAT

In preparation for the unconfined compressive creep tests, strain transducers (LVDTs) were mounted on the sides of the 7.5-in. (191-mm)-diameter core specimens in the usual fashion. Epoxy was used to fasten the gage points at 120-deg increments around the specimen. Two gage points at the extremes of the gage length were mounted at each location. During the first ten axial compression tests, whether using rigid or flexible loading heads, one or two of the three LVDTs inexplicably indicated extension instead of compression. This, of course, is unrealistic and unacceptable. These phenomena were attributed to the lateral movement of large stones (dilation) near the circumference of these composite specimens, to radial expansion near the center of the relatively large radius specimens that rendered the LVDTs no longer parallel, or to both. Therefore, a new method for installing LVDTs was explored. A 0.5-in. (12.7-mm)-diameter hole was drilled at the center of the specimen along its axis, and two gage points were affixed at each end of this hole to accommodate a single LVDT. These gage points consisted of a small O-ring pressed against the sides of the hole in the specimen using specially designed, spring-loaded, metal attachments. On one end of the specimen, a small channel was cut radially from the hole to the edge of the specimen, through which associated wiring was routed. Measurements of deformation using this method appeared to be reasonably accurate, precise, and in accordance with accepted theory. Because curing of glue was eliminated, mounting of gages was much quicker. The contact point of the O-rings is much smaller than glued-on gage points, which enhanced accuracy. In addition, this internal mounting does not interfere with the membrane around the specimen during confined axial compression testing. Details, including a schematic of this apparatus, are provided in Chapter 2 of the report.

TABLE A-6 IDT Results on Cores at 5°C (41°F) and 25°C (77°F)

Sample Number	Location	Core Diameter, in	Core Height, in	Air Voids, %	Max Aggr Size, mm (in)	Asphalt Content, %	Viscosity AC @ 25C Pa·s x 10^6	Tensile Strain, mm/mm	Tensile Strength, psi	Tensile Modulus, psi
LSM Cores at 41°F (5°C)										
348LSM1	Co.CP.348	7.62	2.264	4.2	1.50	5.16	1.57	0.005110	154	62353
348LSM8	Co.CP.348	7.62	1.732	4.0	1.50	5.16	1.57	0.003794	162	88318
LSM37414	Co.Flag.374	7.70	4.166	6.4	1.50	5.25	9.37	0.001591	187	246068
LSM3745	Co.Flag.374	7.69	2.467	3.7	1.50	5.25	9.37	0.002221	320	300661
LSM39516	Co.Flag.395	7.70	2.046	2.2	1.50	4.65	2.37	0.002856	185	135636
KC1M1	Kentucky	7.63	2.634	1.7	1.50	3.5	2.37	0.002128	203	198142
KC1M2	Kentucky	7.64	2.686	4.0	1.50	3.5	2.37	0.002259	184	168869
OC10SB	Oregon.LSM	7.65	2.766	3.6	1.50	6.28	0.42	0.005217	151	60265
OCOC3SB	Oregon.LSM	7.74	2.500	4.4	1.50	6.28	0.42	0.006804	134	41399
I70C3	Pa.Fulton	7.50	1.870	2.4	1.50	3.9	0.469	0.005180	122	47826
I70C4601	Pa.Fulton	7.50	2.170	2.4	1.50	3.9	0.469	0.001059	248	476405
I80C13	Pa.Clearfield	7.50	3.385	3.9	1.50	3.1	0.445	0.001731	210	246945
I80C7	Pa.Clearfield	7.50	3.388	3.9	1.50	3.1	0.445	0.002179	165	154458
B11LSM	Wy.Lar.LSM	7.62	3.002	6.1	1.25	4.47	1.83	0.001965	142	149127
B12LSM	Wy.Lar.LSM	7.62	3.079	5.3	1.25	4.47	1.83	0.001307	140	222184
RSA15B	Wy.RkSpring	7.69	3.080	3.9	1.50	5.3	0.762	0.003229	152	98644
RSA9B	Wy.RkSpring	7.73	2.767	4.6	1.50	5.3	0.762	0.002511	155	129241
Control Cores at 41°F (5°C)										
C6347B	Co.CP.347	7.65	2.527	4.2	0.75	5.78	1.62	0.001302	116	184776
C16347B	Co.CP.347	7.64	2.975	3.6	0.75	5.78	1.62	0.004517	145	66682
OC4BB	Oregon.Control	7.66	3.435	5.8	1.00	5.83	0.452	0.006216	124	41654
LCA3B	Wy.Lar.Control	7.67	2.537	6.5	0.75	4.2	6.61	0.001512	123	169051

(Continued on next page)

TABLE A-6 **IDT Results on Cores at 5°C (41°F) and 25°C (77°F)** *(Continued)*

Sample Number	Location	Core Diameter, in	Core Height, in	Air Voids, %	Max Aggr Size, in.	Asphalt Content, %	Viscosity AC @ 25C, Pa·s x 10^6	Tensile Strain, mm/mm	Tensile Strength, psi	Tensile Modulus, psi
LSM Cores at 77°F (25°C)										
348LSM6	Co.CP.348	7.62	2.575	6.5	1.50	5.16	1.57	0.007033	95	28007
348LSM2	Co.CP.348	7.62	2.613	5.7	1.50	5.16	1.57	0.013683	113	17154
LSM37410	Co.Flag.374	7.66	3.626	5.7	1.50	5.25	9.37	0.002590	114	91886
LSM37412	Co.Flag.374	7.69	3.882	6.1	1.50	5.25	9.37	0.002753	106	80314
L395T11	Co.Flag.395	7.72	2.009	6.9	1.50	4.65	2.37	0.005124	103	42051
L395B7	Co.Flag.395	7.69	2.213	1.1	1.50	4.65	2.37	0.004396	109	51729
KC16M2	Kentucky	7.66	3.022	4.7	1.50	3.5	2.37	0.005096	118	47993
KC16M1	Kentucky	7.65	3.134	1.5	1.50	3.5	2.37	0.004060	132	67598
OC12SB	Oregon.LSM	7.85	3.169	4.6	1.50	6.28	0.42	0.022204	81	7744
OC15SB	Oregon.LSM	7.69	3.333	4.4	1.50	6.28	0.42	0.020533	88	8982
I70C2	Pa.Fulton	7.50	2.402	2.4	1.50	3.9	0.469	0.002674	107	81461
I70C5	Pa.Fulton	7.50	2.222	2.4	1.50	3.9	0.469	0.004200	100	48616
I80C2	Pa.Clearfield	7.50	3.865	3.9	1.50	3.1	0.445	0.004270	109	51810
I80C1	Pa.Clearfield	7.50	4.000	3.9	1.50	3.1	0.445	0.004937	105	43139
B9LSM	Wy.Lar.LSM	7.62	3.418	5.3	1.25	4.47	1.83	0.005026	88	36339
B15LSM	Wy.Lar.LSM	7.62	3.317	5.1	1.25	4.47	1.83	0.000485	49	209099
RSA12B	Wy.RkSpring	7.74	2.831	4.7	1.50	5.3	0.762	0.009777	108	23226
RSA13B	Wy.RkSpring	7.77	3.148	4.6	1.50	5.3	0.762	0.011811	120	21452
Control Cores at 77°F (25°C)										
C347B9	Co.CP.347	7.64	3.375	4.2	0.75	5.78	1.62	0.010677	105	20401
C347B12	Co.CP.347	7.63	3.156	3.9	0.75	5.78	1.62	0.009865	89	18676
OC3B	Oregon.Control	7.69	3.008	4.9	1.00	5.83	0.452	0.019236	86	9378
LCA6B	Wy.Lar.Control	7.66	2.607	6.3	0.75	4.2	6.61	0.003019	112	77520
LCA4B	Wy.Lar.Control	7.66	2.607	5.8	0.75	4.2	6.61	0.003379	134	82185

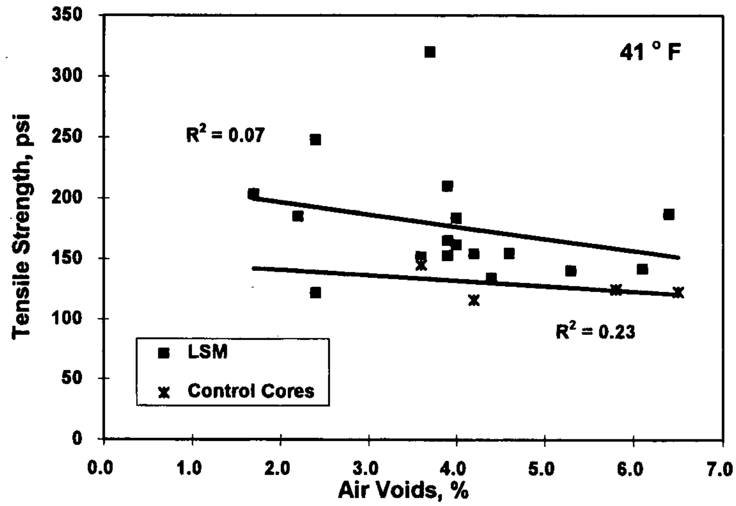

Figure A-2. Tensile strength versus percent air voids at 5°C (41°F).

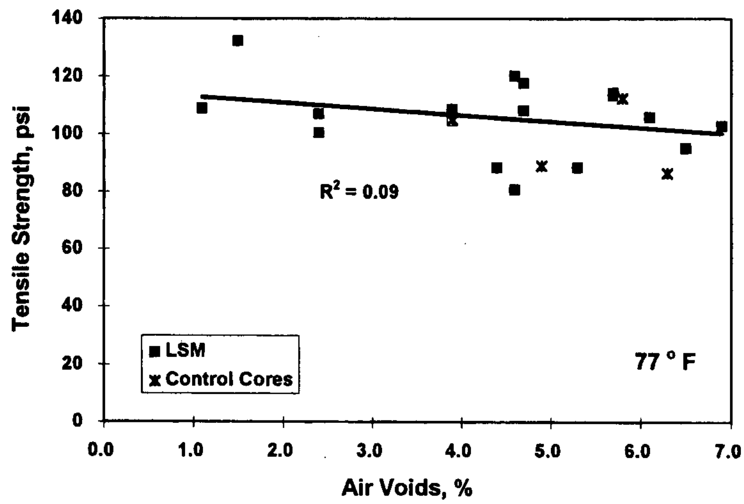

Figure A-3. Tensile strength versus percent air voids at 25°C (77°F).

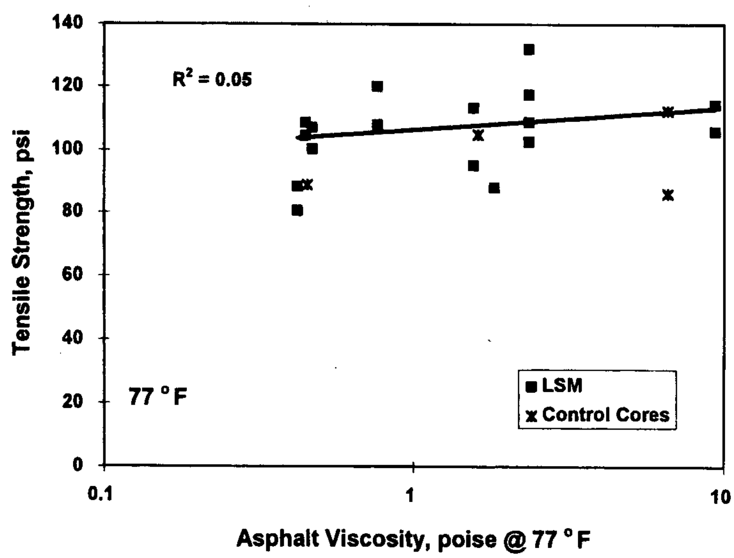

Figure A-4. Tensile strength versus asphalt viscosity at 25°C (77°F).

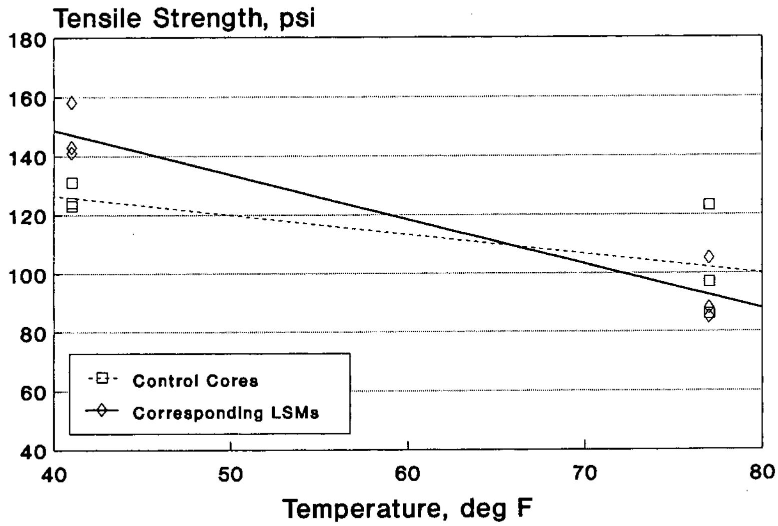

Figure A-5. Tensile strength versus temperature for LSM cores and corresponding control cores.

Using the solution for pressurization and axial loading of a thick-walled pressure vessel, it was determined that the difference between the axial stresses in the drilled cylinder and those in a solid cylinder, assuming an unconfined test, is approximately 0.25 percent. If the internal and external volumes are pressurized to the same pressure, the difference is still less than 1 percent. The consistency and accuracy of the new measurement system more than compensated for this small difference in the stress fields. The solution also revealed that there is a more significant difference between the solid and drilled cylinders if the external confining pressure does not equal the pressure inside the drilled cavity, but the differences are still quite small.

Monotonic Axial Compression to Failure

Upon completion of the creep tests, selected specimens were loaded to failure in axial compression at a constant strain rate of 0.05 in/in/min (0.05 mm/mm/min) and at the same temperature at which the creep tests were performed. Many of these core test specimens were fairly thin because the layer of interest in the pavement was thin. As reported earlier, the thickness of all the pavement layers except two (one 12-in. [305-mm] and one 5-in. [127-mm]) were between 1.5 in. (38 mm) and 3.75 in. (95 mm). Results of these tests demonstrated that measured compressive strength depends heavily on specimen height. This fact should be kept in mind during the discussion of these test

TABLE A-7 Total Resilient Modulus at 5°C (41°F) and 25°C (77°F) of LSM Cores

Location	Specimen Height, inches	Resilient Modulus at 41°F (5°F), psi x 10^3	Resilient Modulus at 77°F (25°C), psi x 10^3
Co.CP.348	2.60	2,080	763
Co.Flag.374	3.62	2,587	1,029
Co.Flag.395	2.35	2,084	364
Kentucky.LSM	2.69	2,086	641
Oregon.LSM	3.06	2,270	850
Pa.Fulton	2.37	2,990	1,479
Pa.Clearfield	4.05	2,024	472
Wy.Lar.O.G.LSM	2.50	1,650	382
Wy.RkSpring	2.95	2,295	550

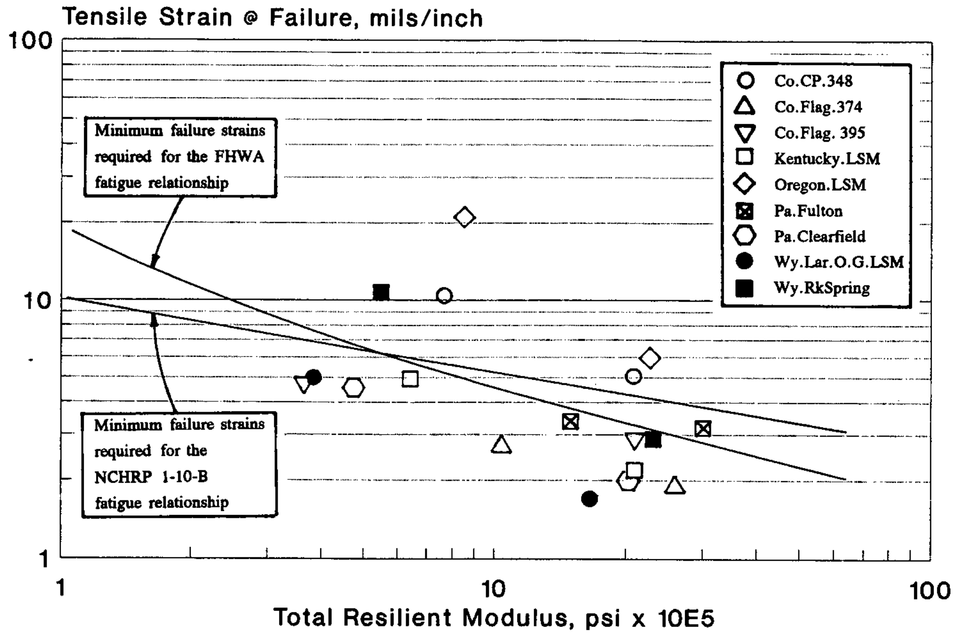

Figure A-6. LSM pavement core data plotted on NCHRP AAMAS chart to examine relative resistance to fatigue-cracking.

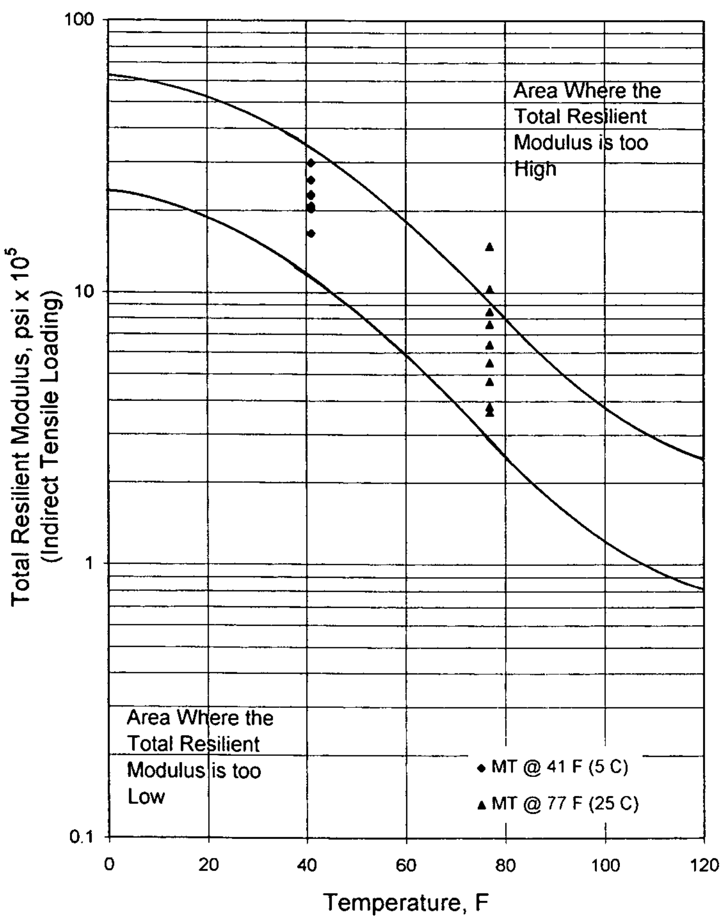

Figure A-7. Total resilient modulus versus temperature for LSM cores.

results. Whenever possible, cores were tested without removing layers on either side of the layer of interest to minimize end effects.

Compressive strengths for only those LSM cores with corresponding control cores (from adjacent control pavements) are shown in Table A-8. Results are shown for cores tested at MMAT and at constant viscosity. At the high strain levels required to produce failure, the strain transducers often slipped near the end of this monotonic test; therefore, the energy and modulus data exhibit considerable scatter. Two sample statistical analyses (α = 0.10) indicated that there are no significant differences between compressive strength of any of the LSM cores and their corresponding control cores, whether tested at MMAT or constant viscosity. The control cores came from base layers and contained aggregate of 0.75-in. (19-mm) maximum size.

Asphalt contents and air voids content of these core specimens were estimated on the basis of measurements made on other cores from the same pavement. Whenever possible, these core test specimens were sawn from the whole core, leaving a small amount of the adjacent layers on each end to reduce end effects during compressive testing and to increase the allowable gage length. Presence of these nonuniform layers precluded accurate measurements of air voids content in the layer of interest. Teflon-coated end caps were also used to minimize end effects because of radial friction during loading.

Compressive Creep and Recovery Tests on Pavement Cores

The comparative rutting susceptibility of LSM was evaluated from compressive creep and recovery tests on pavement cores of LSM and conventional base mixes. Results from these tests are mixed. Some LSM exhibit improved rutting resistance over conventional mixes but not all.

Findings

Figure A-8 illustrates the format of the test data from compressive creep and recovery tests performed on core samples obtained from different pavement sites around the United States. For a given specimen, the creep response is analyzed using a viscoelastoplastic model proposed by Sides et al. (*77*), in which the strain is divided into elastic, viscoelastic, viscoplastic, and plastic components. The instantaneous response upon loading (see Figure A-8) is time independent and includes elastic and plastic components. During constant loading, the time-dependent response includes viscoelastic and viscoplastic components. Upon unloading, there is an instantaneous elastic response followed by a time-dependent recovery that is viscoelastic. After a very long time, the sum of plastic and viscoplastic components which make up the permanent deformation in the material responds by forming an asymptote.

The creep and recovery response illustrated in Figure A-8 was modeled using the following equation:

$$\varepsilon_v(t) = D_e + D_{ve}\left(\frac{t^m}{1+at^m}\right) + D_p + D_{vp}\left(\frac{t^q}{1+bt^q}\right) \qquad \text{(Eq. 1)}$$

where,

$\varepsilon_v\,(t)$ is the measured vertical strain at time, t, and
D_e, D_{ve}, D_p, D_{vp}, m, q, a, and b are model coefficients.

A pattern search algorithm, formulated by Lytton et al. (*78*), was used to determine the coefficients of Equation 1, using the creep and recovery data collected for each specimen. To conduct a comparative evaluation of the rut susceptibility of the different mixes, the sum of the plastic and viscoplastic components for each specimen was determined using the backcalculated, creep-compliance coefficients. This sum (hereafter referred to as plastic strain) was used to rank the specimens tested in terms of rutting susceptibility. The larger the predicted plastic strain, the greater the potential for permanent deformation (rutting) under repeated traffic loading.

The predicted plastic strain depends on time of loading. The longer the load duration, the higher the predicted plastic strain. To establish appropriate values of this variable for the analysis, the zone of influence of a given wheel load was evaluated using elastic layered theory. Specifically, researchers investigated the variation in predicted pavement response with distance from a given wheel load. Because the susceptibility to load-associated permanent deformation is of interest in this study, vertical compressive stresses and shearing stresses were calculated at different depths within the asphalt layer and at different offsets from a given wheel load. (The horizontal stresses were not calculated because fatigue behavior was not being studied.) These calculations were made for two pavement structures, "weak" and "strong," and at low and high load levels of 5 and 10 kips, respectively. The weak and strong pavement structures used in the analysis are characterized in Table A-9.

Figures A-9 and A-10 illustrate predicted variations in pavement response with distance from a given wheel load. The plain, solid line in each figure shows the predicted stresses as percentages of the maximum stress calculated for a given wheel load and pavement structure. The other curve with the shaded triangles shows the predicted stress basin. Because of symmetry, it is necessary to illustrate only one-half of the stress basin in Figures A-9 and A-10. The predicted stresses diminish with distance from a given wheel

TABLE A-8 Unconfined Axial Compression Test to Failure of Pavement Cores at Temperatures Indicated

File Name	Specimen ID	Temp, °F	Height, inch	Gage Length, inch	Max Aggr Size, inch	Asphalt Content, %	Air Voids, %	Energy @ 2% strn, lb-in/ft^3	Compressive Strength, psi	Modulus, psi
Cores Tested at 7-Day Maximum Annual Pavement Temperature for the Pavement Site										
CC347-8	Co.C.P.Control	124	5.05	3.00	0.75	5.78	4.1	-	-	-
CC347-1	Co.C.P.Control	124	6.30	3.00	0.75	5.78	4.1	0.279	114.4	19458
CC348-6	Co.C.P.LSM	124	3.00	1.5/1.83	1.5	5.16	5.1	0.195	67.5	17739
CC348-11	Co.C.P.LSM	124	2.78	1.00	1.5	5.16	5.1	-	-	-
OLC-C25	Oregon.Control	130	3.60	2.31	0.75	5.83	5.9	1.75	286.2	6705
OLC-C6	Oregon.Control	130	4.80	2.01	0.75	5.83	5.9	1.92	213.0	6430
OLC-11S	Oregon.LSM	130	4.30	2.33	1.25	6.28	4.5	2.56	197.0	53000
OLC-6S	Oregon.LSM	130	4.05	1.90	1.25	6.28	4.5	0.69	197.1	3600
WLA-11	Wy.Lar.Control	117	3.91	2.36	0.75	4.20	6.3	3.52	329.6	85996
WLA-1	Wy.Lar.Control	117	3.85	2.29	0.75	4.20	6.3	4.14	359.8	31286
WLB-3	Wy.Lar.O.G.LSM	117	3.30	1.39	1.25	4.47	5.5	1.55	349.3	6256
WLB-51	Wy.Lar.O.G.LSM	117	3.55	1.79	1.25	4.47	5.5	1.66	214.3	6377

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TABLE A-8 **Unconfined Axial Compression Test to Failure of Pavement Cores at Temperatures Indicated** *(Continued)*

File Name	Specimen ID	Temp, °F	Height, inch	Gage Length, inch	Max Aggr Size, inch	Asphalt Content, %	Air Voids, %	Energy @ 2% strn, lb-in/ft^3	Compressive Strength, psi	Modulus, psi
Cores Tested at Temperature that Provided Constant Asphalt Viscosity										
CC347-1	Co.C.P.Control	124	6.30	3.00	0.75	5.78	4.1	0.279	114.4	19458
CC347-8	Co.C.P.Control	124	5.05	3.00	0.75	5.78	4.1	-	-	-
CC348-13	Co.C.P.LSM	127	3.05	1.94	1.5	5.16	5.1	-	414.1	-
CC348-9	Co.C.P.LSM	127	3.40	1.36	1.5	5.16	5.1	3.99	430.1	21312
OL-C1	Oregon.Control	113	3.75	1.76	0.75	5.83	5.9	5.00	422.8	-
OL-C7	Oregon.Control	113	4.25	1.98	0.75	5.83	5.9	1.96	251.8	9922
OL-C4S	Oregon.LSM	113	4.30	2.32	1.25	6.28	4.5	1.17	239.7	4818
OL-C8S	Oregon.LSM	113	4.05	2.55	1.25	6.28	4.5	2.61	311.4	13579
WL-A15	Wy.Lar.Control	136	3.80	2.14	0.75	4.20	6.3	1.66	266.2	6696
WL-A2	Wy.Lar.Control	136	3.15	1.74	0.75	4.20	6.3	2.56	419.2	11624
WL-B13	Wy.Lar.O.G.LSM	125	3.20	1.70	1.25	4.47	5.5	4.05	301.1	12682
WL-B14	Wy.Lar.O.G.LSM	125	3.40	1.68	1.25	4.47	5.5	2.36	410.5	11179

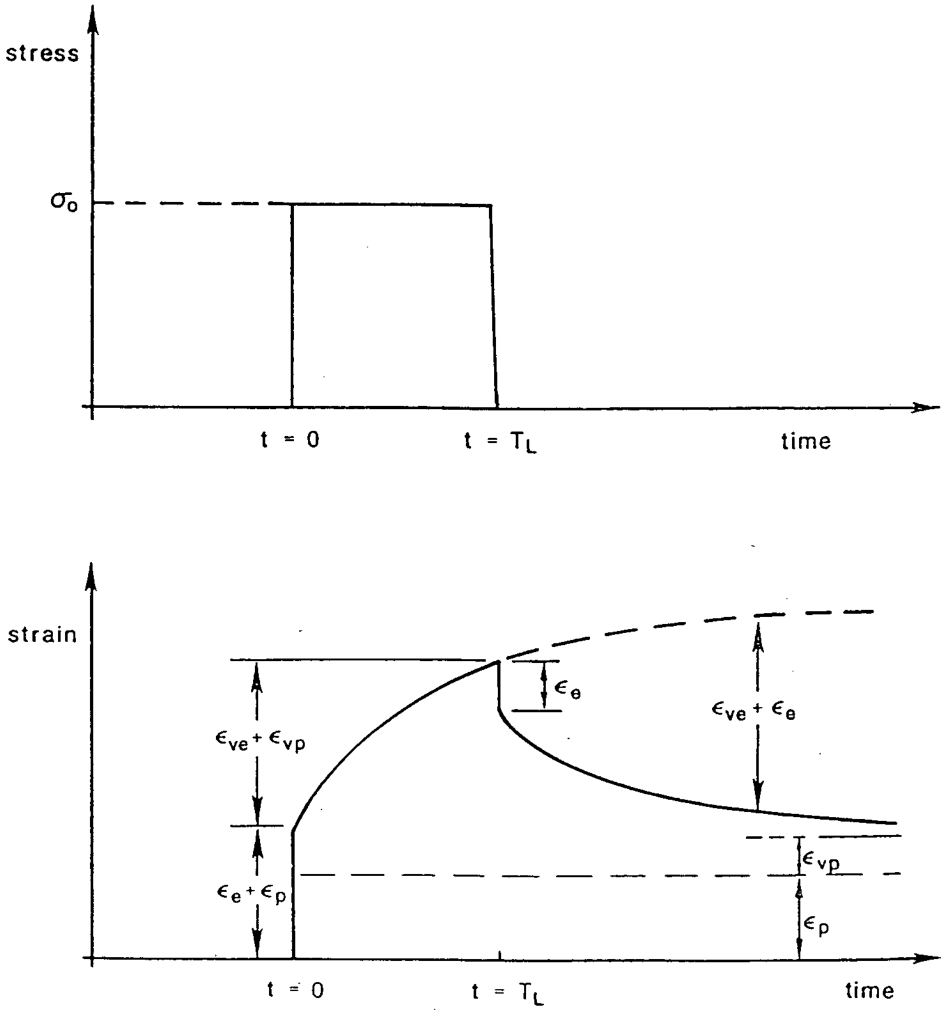

Figure A-8. Viscoelastoplastic model.

load as illustrated in the figures, and a point is reached beyond which the effect of the load is negligible. The location of this point defines the size of the influence zone of the given wheel load or the stress basin width.

The predicted stress basins shown in Figures A-9 and A-10 of the report indicate that the vertical compressive stress and the shear stress attenuate very rapidly with distance from the load. In Figure A-9, for example, the vertical compressive stress at a distance of 1 ft (305 mm) from the center of the load is only about 5 percent of the predicted vertical compressive stress directly underneath the load. In Figure A-10, the predicted shear stress is about 8 percent of the maximum at 2 ft (710 mm) from the load. These results indicate that rutting development will take place underneath the load or within the wheelpath. As wheelpath rutting develops, forces outside the wheelpath will induce lateral flow of material, which will appear as shoving beside the wheelpath.

TABLE A-9 Description of Pavement Structures Used in the Computer Analysis

Layer	Weak		Strong	
	Modulus, ksi	Thickness, inches	Modulus, ksi	Thickness, inches
Asphalt Surface	250	1	750	6
Base	15	4	60	10
Subgrade	5	semi-infinite	20	semi-infinite

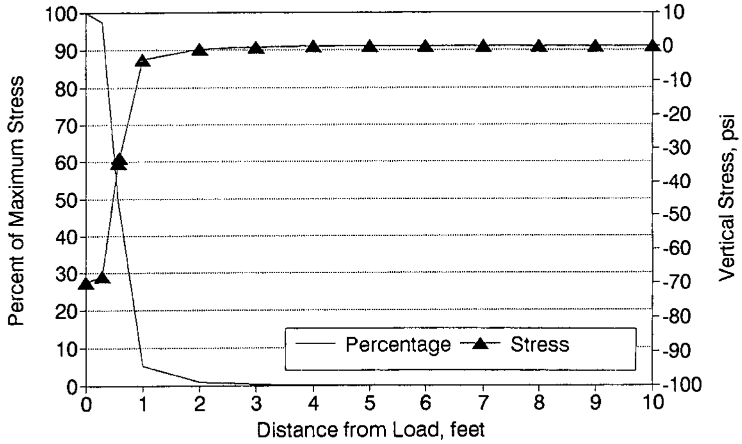

Figure A-9. Predicted variation in vertical stress with distance from a given wheel load (stresses calculated at mid-depth of a strong pavement under heavy load).

From the results, a range of loading times were calculated for different stress basin widths and vehicle speeds. Figure A-11 shows the variation of loading time with size of the load influence zone (basin width) and vehicle speed. For the asphalt layer, appropriate loading times are predicted to range from about 0.05 to 0.60 sec on the basis of the stress basin widths calculated for this layer. Deeper layers will experience longer loading times because of the effect of load spreading with depth into the pavement. On the basis of the results, loading times of 0.1 and 0.6 sec were selected for calculating plastic strains to conduct the comparative evaluation of LSM and conventional mixes.

The creep compliance coefficients backcalculated from the compressive creep and recovery data were used to predict plastic strains for load durations of 0.1 and 0.6 sec and for a stress level of 100 psi (690 kPa). Figures A-12 and A-13 show the calculated plastic strains for mixes tested under unconfined compressive creep and recovery at constant viscosity. In these tests, temperatures were selected so that the binders were approximately at constant viscosity. In this way, the effects of differences in the aggregate skeletons of the different mixes were isolated and accentuated.

The predicted plastic strains were used to rank the different mixes in terms of resistance to permanent deformation. The rankings established are summarized in Tables A-10 and A-11 for the two loading times considered. In these tables, higher rankings (rank 1 is highest) indicate smaller predicted plastic strains and, ostensibly, greater rutting resistance.

Figures A-12 and A-13 and Tables A-10 and A-11 show that the CC348 and WRA LSM are the most rutting resistant of the mixes evaluated on the basis of the calculated plastic strains at 0.1- and 0.6-sec loading time. At a 0.1-sec load dura-

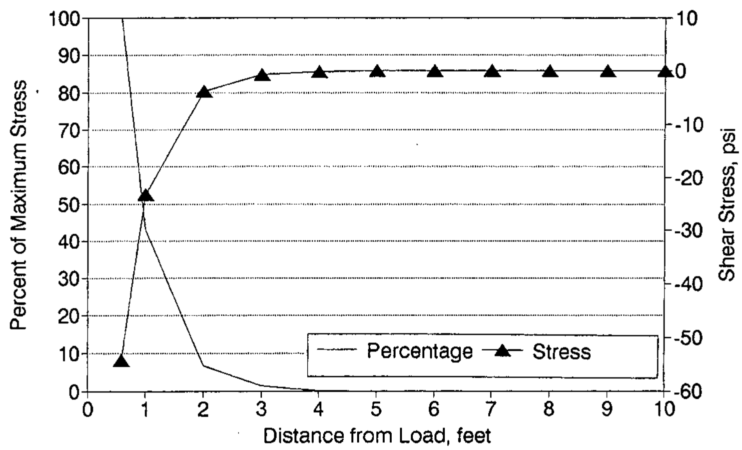

Figure A-10. Predicted variation in shear stress with distance from a given wheel load (stresses calculated at mid-depth of a strong pavement under heavy load).

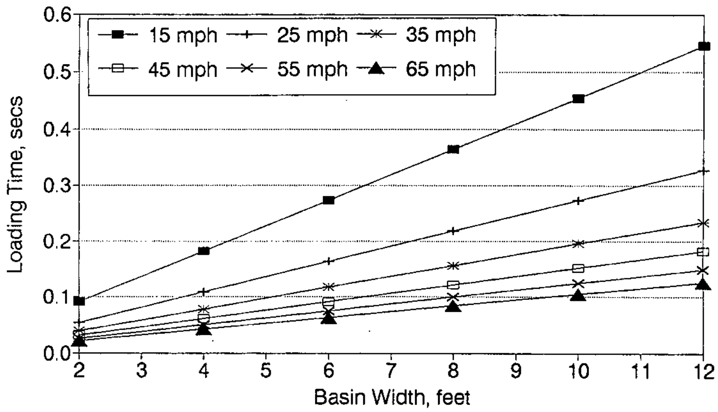

Figure A-11. Predicted variation of loading time with size of load influence zone and vehicle speed.

tion, the Oregon control and Oregon LSM are predicted to have comparable rutting resistance, having very similar plastic strains. Both mixes are ranked third in terms of rutting resistance, as shown in Table A-10. Relative to the LSM evaluated, the Pennsylvania mixes are predicted to be the least rutting resistant. The Oregon (OL_C7) and Wyoming (WLA) control mixes are predicted to have better rutting resistance than these mixes for the loading times considered. Overall, the Pennsylvania LSM are comparable to the CC347_8 control mix from Cedar Point, Colorado. The rankings of the LSM relative to their corresponding controls improve with higher loading times. By comparing the results at the loading times of 0.1 and 0.6 sec, one observes that the LSM (with the exception of PC and PF_C6) are predicted to have better rutting resistance than the control mixes at the longer loading time of 0.6 sec. This trend is also observed at even higher loading times as illustrated in Figures A-14 and A-15, which show calculated plastic strains at loading times of 1 and 2 sec, respectively. This finding suggests that LSM are more effectively used on pavements that experience loading times longer than those associated with normal highway speed traffic, for example, intersections, shoulders, urban streets, truck termi-

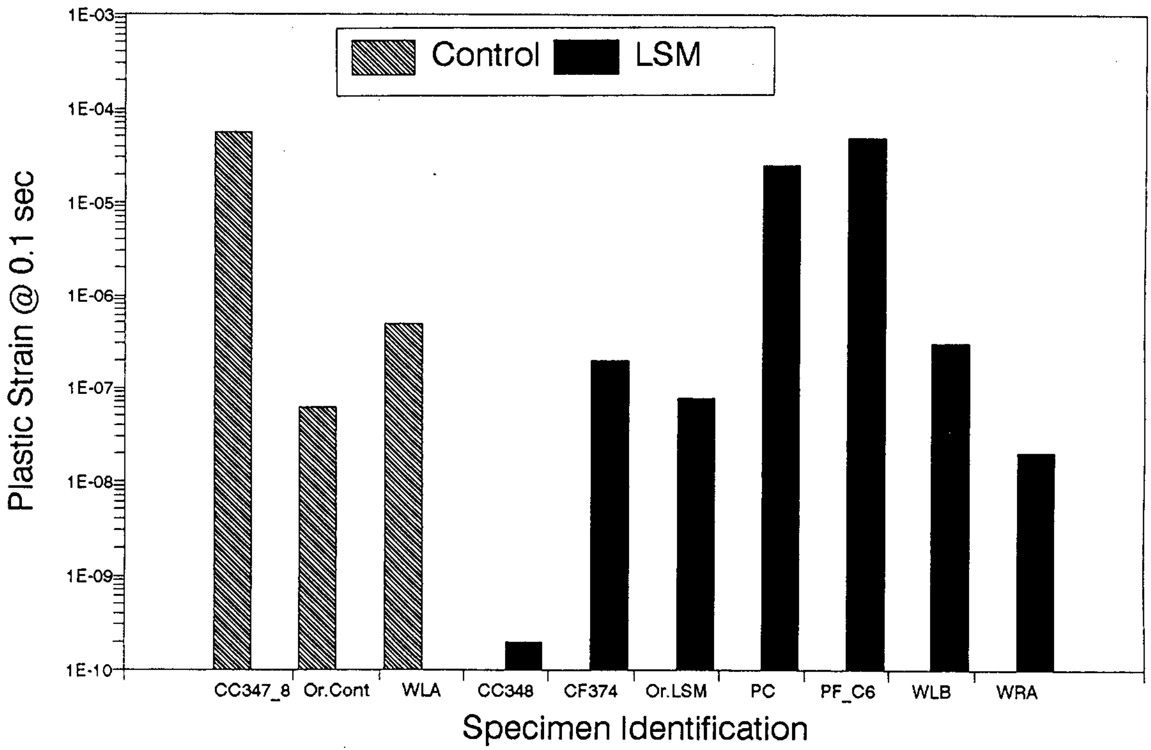

Figure A-12. Predicted plastic strains at 0.1-sec loading time.

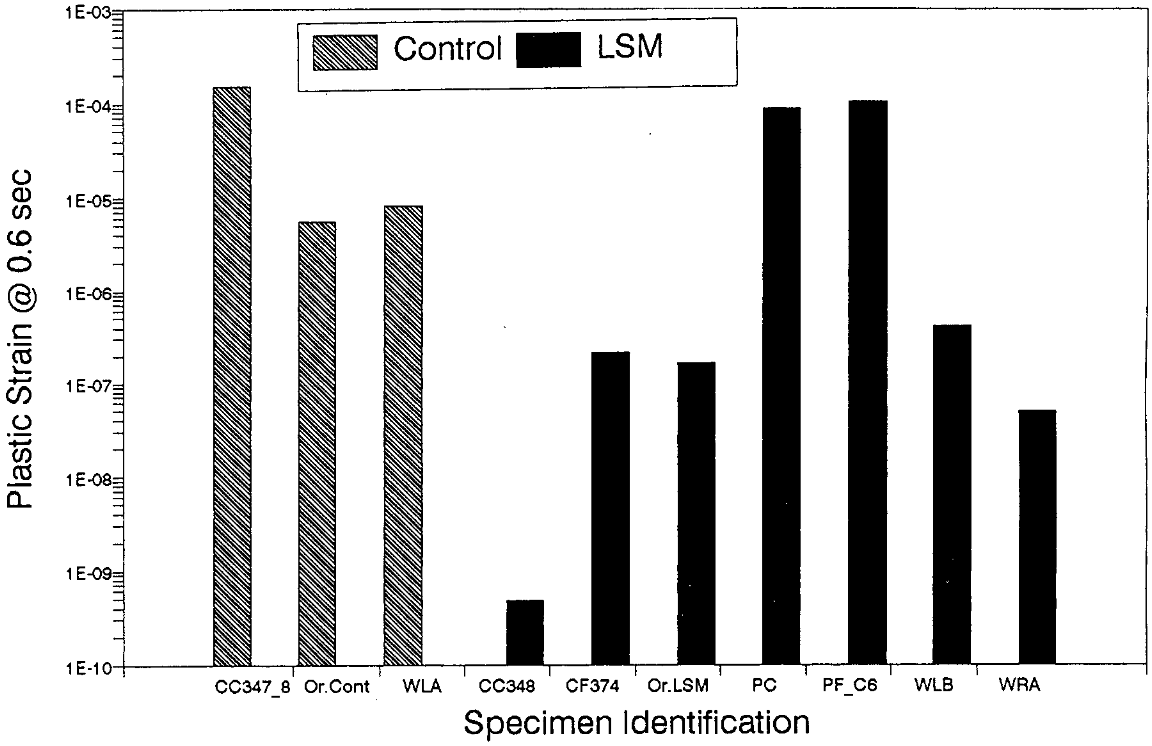

Figure A-13. Predicted plastic strains at 0.6-sec loading time.

TABLE A-10 Ranking of Rut Resistance Based on Average Predicted Plastic Strains at 0.1-Sec Loading Time

Core Specimen	Location Where Core(s) Was Taken	Type of Mix	Ranking of Rut Resistance
CC348	Cedar Point, [2] Colorado on IH-70, MP 348	Dense-Graded LSM	1
WRA	Rock Springs, Wyoming on IH-80	Open-Graded LSM	2
Or.Cont[1]	Linn/Marion, Oregon on IH-5	Dense-Graded Control Mix	3
Or.LSM	Linn/Marion, Oregon on IH-5	Dense-Graded LSM	3
CF374	Flagler, Colorado on IH-70, MP 374	Dense-Graded LSM	4
WLB	Laramie, Wyoming on IH-80	Open-Graded LSM	4
WLA	Laramie, Wyoming on IH-80	Dense-Graded Control	5
PC	Clearfield, Pennsylvania on IH-80	Dense-Graded LSM	6
PF_C6[1]	Fulton, Pennsylvania on IH-70	Dense-Graded LSM	7
CC347_8[1]	Cedar Point, Colorado on IH-70, MP 347	Dense-Graded Control	7

[1] Only one test data file was suitable for evaluation.
[2] The gradation of the aggregates in these materials is provided in Table 3.

TABLE A-11 Ranking of Rut Resistance Based on Average Predicted Plastic Strains at 0.6-Sec Loading Time

Core Specimen	Location Where Core(s) Was Taken	Type of Mix	Ranking of Rut Resistance
CC348	Cedar Point, Colorado on IH-70, MP 348	Dense-Graded LSM	1
WRA	Rock Springs, Wyoming on IH-80	Open-Graded LSM	2
Or.LSM	Linn/Marion, Oregon on IH-5	Dense-Graded LSM	3
CF374	Flagler, Colorado on IH-70, MP 374	Dense-Graded LSM	3
WLB	Laramie, Wyoming on IH-80	Open-Graded LSM	4
Or.Cont[1]	Linn/Marion, Oregon on IH-5	Dense-Graded Control	5
WLA	Laramie, Wyoming on IH-80	Dense-Graded Control	5
PC	Clearfield, Pennsylvania on IH-80	Dense-Graded LSM	6
PF_C6[1]	Fulton, Pennsylvania on IH-70	Dense-Graded LSM	6
CC347_8[1]	Cedar Point, Colorado on IH-70, MP 347	Dense-Graded Control	7

[1] Only one test data file was suitable for evaluation.

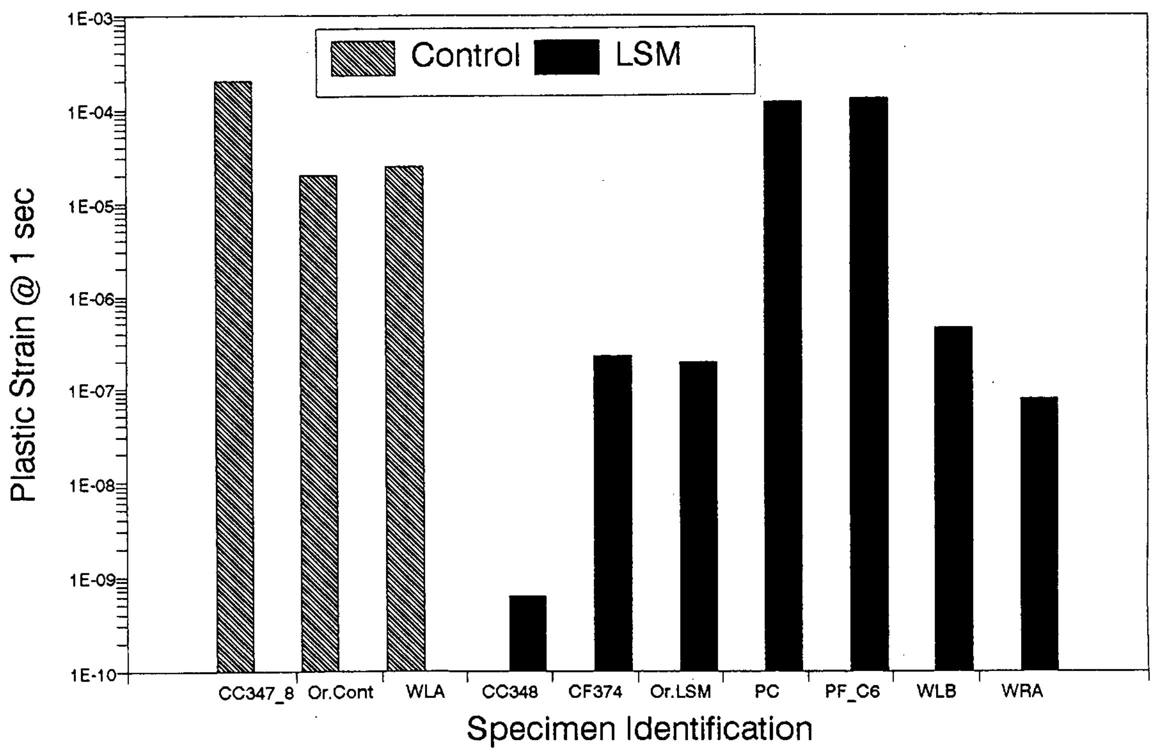

Figure A-14. Predicted plastic strains at 1-sec loading time.

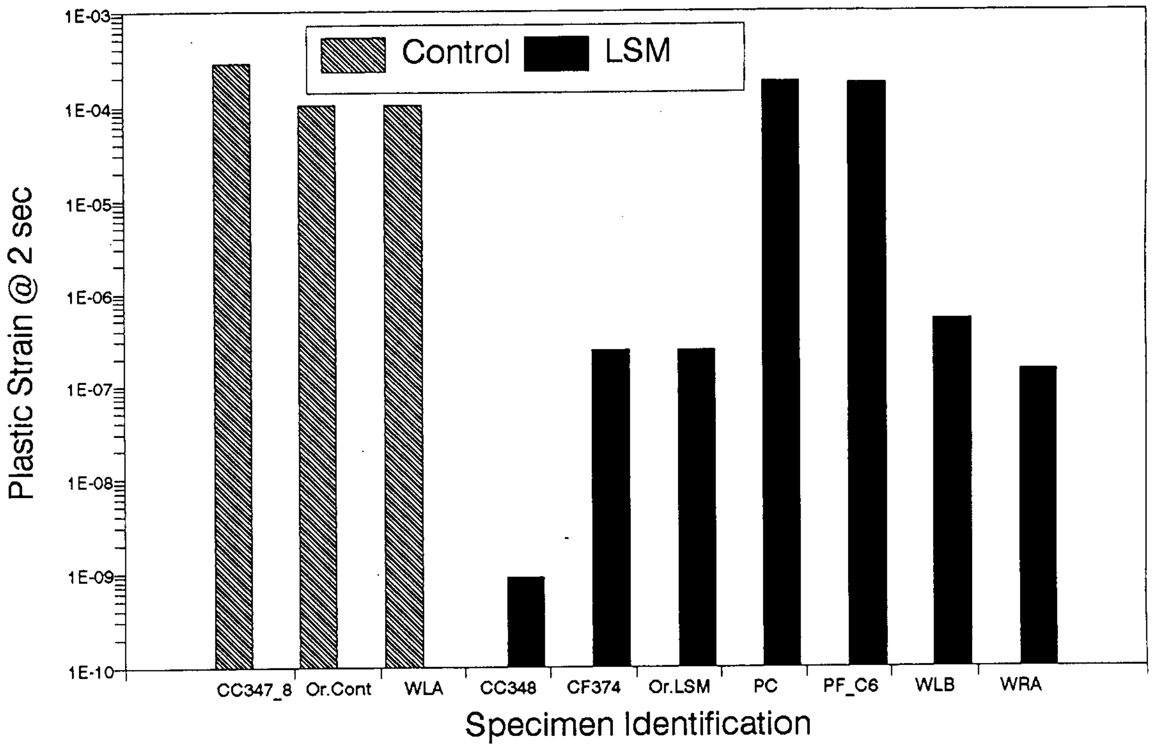

Figure A-15. Predicted plastic strains at 2-sec loading time.

nals, and airport taxiways and aprons, where the load carrying capacity of the aggregate skeleton is more fully mobilized.

Summary

Rutting susceptibility of LSM was evaluated from compressive creep and recovery tests on pavement cores from LSM and conventional base mixes. Although some LSM exhibit improved rutting resistance over conventional mixes, this was not consistent for all comparisons made—this may be the result of differences in the degree of stone-on-stone contact between the LSM cores tested.

These results support the following conclusions:

- Data from compressive creep and recovery tests suggest that LSM may be particularly effective at reducing rutting when used on pavements where load durations are longer than those associated with normal highway speed traffic.
- Although some LSM exhibit improved rutting resistance over conventional mixes, this was not consistent for all pavement core comparisons made, particularly at loading times associated with normal highway speed traffic (45 to 70 mph [72 to 113 km/h]). This may be the result of differences in the degree of stone-on-stone contact between the LSM cores tested. A mix design procedure that ensures good stone-on-stone contact would be useful in obtaining the rutting resistance expected from LSM.

RSCH on Pavement Cores—Tested at U.C. Berkeley

As part of the recently completed SHRP Project A-003A, test methods and analysis procedures were developed to predict the rutting performance of asphalt-aggregate mixes. For rapid prediction of rutting performance of typical mixes, the RSCH test was recommended by the SHRP A-003A contractor, because the test applies to the specimen the primary distress mechanism responsible for rutting (*79*).

The RSCH test is performed at a temperature at which most permanent deformation typically will occur at the project site. Transfer functions are applied to the output data from the test (permanent shear strain caused by simple shear load repetitions) to predict the vertical rutting depth in situ caused by traffic loading. The transfer function relating permanent shear strain in the RSCH test to vertical rutting depth in situ is based on finite element analysis using a nonlinear viscoelastic constitutive relationship (*79*).

A transfer function relating RSCH load repetitions to ESALs in situ was developed by Sousa and Solaimanian (*74*) from a statistical correlation of RSCH results and traffic and rutting depth measurements from LTPP general pavement sections (GPS). The mixes in the GPS sections used for developing the transfer function included only 0.75-in. (19-mm)-maximum-aggregate-size dense gradations and conventional asphalt binders. However, because the RSCH measures the response of the material to the primary distress mechanism responsible for rutting, in many cases, use of the relation developed from the LTPP GPS sections can be extrapolated to LSM and mixes with modified binders with reasonable accuracy.

Application of the transfer function to mix types outside the original GPS database requires that the nonlinearity of the mix response to simple shear testing in the laboratory and typical tire pressures and loads in situ are not significantly affected by binder modification or gradation change. The portability of the transfer function must be determined through experimentation. This type of experimentation has been executed with successful results on a limited basis for asphalt-rubber paving mixes with gap-grading (ARHM-GG), RAP, SMA with rubber and polyolefin modified binders (*80*), and dense-graded asphalt concrete with PBA-6 (*81*) and PG-70 (*82*) modified binders.

The purpose of this work element was to test LSM and control pavement cores using the RSCH test and compare predicted rutting performance using in situ traffic with actual measured rutting depth. New techniques for the RSCH test developed for LSM are also presented here.

Description of the RSCH Test

The RSCH test was performed using the prototype Universal Testing System (UTS) device built by James Cox and Sons and in operation at the University of California, Berkeley, since 1991. The UTS has two hydraulic actuators under closed-loop digital control (one horizontal and one vertical). For the RSCH test, the specimen is bonded to platens, which are, in turn, clamped to the actuators. The vertical actuator is used to maintain the specimen at a constant height, while the horizontal actuator applies a repetitive haversine shear stress. For this work, the RSCH test was performed using a 68.9 kPa (10 psi) shear stress, with a 0.1-sec loading time followed by a 0.6-sec rest period. Each specimen was subjected to approximately 10,000 load repetitions.

Special RSCH Procedures for LSM

The UTS (RSCH prototype) was designed to accommodate only 6-in. (150-mm)-diameter specimens, which allowed room on both sides to mount all necessary instrumentation. Part of this instrumentation is an LVDT mounted on the specimen, which measures the shear displacement between two horizontal planes in the specimen (see Figure A-16 [Part a]). Another LVDT is mounted to measure the relative distance between the top and bottom platens. The LVDT placement on the 7.75-in (197-mm)-diameter specimen is shown in Figure A-17.

Readings from the latter LVDT are used by the feedback, closed-loop, control algorithm in the equipment software to control the vertical actuator (*83*). This LVDT is mounted on the center axis of the specimen 90 deg from the direction of shear. This mounting arrangement prevents the LVDT from reading any possible angular movements between the platens caused by flex in the loading frame. With this system, at least the center distance of the platens is maintained constant. There is no guarantee that the platens at the front and back of the specimen maintain the same spacing. The parallelism of the platens during testing depends on the rigidity of the testing frame.

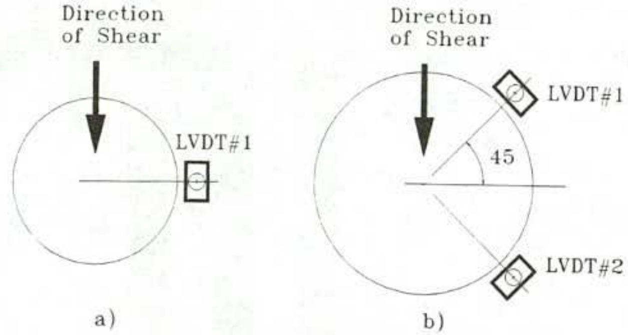

Figure A-16. Schematic drawing of LVDT mountings on a 150-mm (6-in.) specimen (a) and a 191-mm (7.75 in.) specimen (b).

Because of the large stones in the LSM, a large specimen size was selected. Instead of the usual 6-in. (150-mm) diameter by 2-in. (50-mm)-thick cylindrical specimens, 7.75-in. (197-mm)-diameter by 3-in. (76-mm)-thick specimens were used. Larger platens were manufactured by Cox and Sons to accommodate the larger specimens. With the larger diameter specimens, it was no longer possible to mount the LVDT on the side of the specimen because of lack of space in the load frame of the prototype UTS. Although mounting only one LVDT at the front or rear of the specimen, where there was space, was considered, with this configuration, the software would be maintaining the height constant in the front or rear of

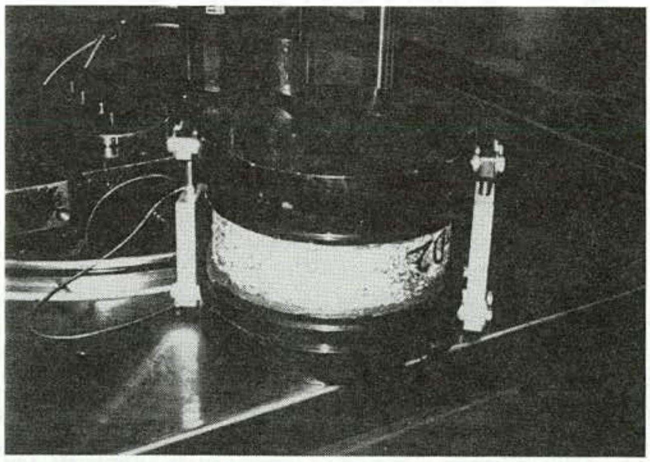

Figure A-17. LVDT mounting on a 191-mm (7.75-in.) specimen for height control (horizontal LVDT to measure shear deformation not shown) (73).

the specimens, not in the center as with the 6-in (150-mm)-diameter specimens typically used for the RSCH test. (If the testing frame was infinitely rigid, the location of the LVDT would be irrelevant.)

To overcome this problem, two LVDTs were mounted 45 deg apart (see Figure A-16 [part b]), fore and aft of a central plane perpendicular to the direction of motion. This LVDT placement ensured that the software could control the average readings from both LVDTs. With this arrangement, any change of angle caused by possible changes in parallelism would be canceled. To accomplish this, advantage was taken of the "virtual channels" feature in the software used with the UTS. This feature permits the creation of a virtual channel with the sum of, average of, or difference between any two real channels. After this assignment, the software uses the virtual channel as if it were any real measured channel. It can be used as a feedback control channel, as a limit detector, or to save data. During tests executed for this project, the feedback loop for the vertical actuator was closed using the virtual channel.

This mounting arrangement for the two LVDTs ensured that the center of the specimen was maintained at a constant height; however, during testing, the front and back of the specimen might have changed height. For this reason, the specifications used by the FHWA for the procurement of the Superpave shear test (SST) device state that the test platens must remain mutually parallel within ± 0.00025 in. (0.00635 mm) in 6 in. (150 mm) (*84*). With this specification in place, machines with different flexibility will not yield different results. The specification for maintaining constant height requires that the change in height during a pulse should not be greater than ± 5 by 10^{-5} in. (1.3 by 10^{-3} mm) (as stated in AASHTO TP7) (*84*).

Test Temperature

The temperature used for testing was the 7-day MMAT, $T_{7day\ max}$, which is calculated from weather station data and latitude (*85*). The calculated test temperatures are included in Table A-12.

Description of LSM Specimens

All pavement cores had approximately a 7.7-in. (196-mm) diameter instead of the typical 6-in. (150-mm) diameter in order to ensure an adequate aspect ratio between aggregate size and specimen size. Larger diameter specimens also ensure a higher percentage of uniform shear stress distribution in the RSCH test.

Air voids content was calculated using the bulk specific gravity determined using the parafilm method (*86*) and the maximum specific gravity found using the Rice method (ASTM D 2041). Air voids content appear in Table A-12.

The field cores tested in the RSCH test were taken from four sites constructed between 1989 and 1992, two in Colorado and two in Wyoming. Site descriptions and information from a condition survey performed in 1993 are summarized in Table A-4.

Test Results and Analysis

The method used to compare the performance predicted from laboratory testing with measured field performance was based on a statistical correlation between certain field measurements at GPS sites and laboratory measurement of permanent shear deformation on field cores from the same GPS sites, using the RSCH test at $T_{7day\ max}$ for each GPS section (*74*). The field measurements were rutting depth, ESAL, and 7-day MMAT ($T_{7day\ max}$).

To correlate these measurements, permanent shear strain from the RSCH was converted to vertical rutting depth on the basis of finite element analysis (*79*). During the test, the permanent shear strain increases with each load application. Using finite element analysis, permanent shear strain in the RSCH test has been estimated to correspond to vertical rut depth in situ as follows:

$$\textit{vertical rut depth} = A \times (\textit{permanent shear strain}) \qquad \text{(Eq. 2)}$$

where A = approximately 11 in. (276 mm) for thick lifts.

The analysis method assumes that nearly all permanent deformation will occur in situ at temperatures approximately 5°C (9°F) below and above $T_{7day\ max}$. ESALs to obtain the vertical rutting depth of interest are converted to load repetitions (from the RSCH test) using the following transfer function, developed from the GPS data by Sousa and Solaimanian (*74*):

$$\log(\textit{RSCH load repetitions}) = -4.36 + 1.240 \times \log(\textit{ESAL}) \qquad \text{(Eq. 3)}$$

Performance Prediction and Comparison with Field Data

The RSCH results were used with the model described above for prediction of ESALs to obtain a selected rutting depth. The selected rutting depths for this project were those measured during the condition surveys described previously. These results are shown in Table A-12, along with the accumulated ESALs measured in situ between the time of construction and the condition survey. Except for the Cedar Point sections, the predicted performance (the solid line) compares reasonably well with the actual measured results (plotted points). It bears repeating that the Cedar Point sections were badly cracked, and some of the rutting observed in the section may have occurred in the underlying layers because of moisture passing through the cracked surface.

The Sousa and Solaimanian (*74*) transfer function is plotted with the data presented in Figure A-18. Again, except for the Cedar Point LSM, the data presented here generally fit

TABLE A-12 Summary of Test Information, Core Location, In Situ Rut Depth and Traffic Data, RSST-CH Results, and Predicted ESAL to In Situ Rut Depth (*73*)

Specimen ID	Max Agg Size, mm	Air Voids, %	Depth of Layer Top, mm	Test Temp, °F	In Situ Rut Depth		ESALs to In Situ Rut Depth	RSST-CH Repetitions to Permanent Shear Strain		Log (RSST-CH) = 4.36 + 1.24 log (ESAL)	
					Min Rut Depth, mm	Max Rut Depth, mm				Equiv ESALs to Rut Depth	
Laramie								1.0%	2.0%	2.5 mm	5.0 mm
LAA13	19.0	5.2	38	117	0.0	5.0	1.86E + 06	2.46E + 03	1.28E + 04	1.78E + 06	6.73E + 06
LAA14	19.0	7.3	38	117	0.0	5.0	1.86E + 06	6.66E + 02	2.55E + 03	6.21E + 05	1.83E + 06
LAB2	31.8	4.7	38	117	2.5	5.0	1.86E + 06	1.49E + 04	4.95E + 04	7.63E + 05	2.00E + 07
LAB16	31.8	5.4	38	117	2.5	5.0	1.86E + 06	7.98E + 02	5.01E + 03	7.19E + 05	3.16E + 06
Cedar Point								1.0%	8.0%	2.5 mm	20.0 mm
CP34711	19.0	3.9	0	124	2.5	2.5	8.17E + 05	5.60E + 01	-	8.43E + 04	-
CP34767	19.0	1.9	0	124	2.5	2.5	8.17E + 05	8.74E + 03	-	4.93E + 06	-
CP348612	38.0	6.5	0	124	2.5	20.0	8.17E + 05	1.19E + 07	1.98E + 12	1.67E + 09	2.71E + 13
Flagler								1.0%	2.0%	2.5 mm	5.0 mm
FL3743	38.0	5.1	0	129	2.5	5.0	2.70E + 05	2.19E + 03	1.16E + 04	1.62E + 06	6.23E + 06
FL3746	38.0	6.3	0	129	2.5	5.0	2.70E + 05	5.57E + 03	4.93E + 04	3.44E + 06	2.00E + 07
FL3955	38.0	3.6	0	129	0.0	0.0	2.70E + 05	1.98E + 02	-	2.33E + 05	-
FL39514	38.0	4.1	50	129	0.0	0.0	2.70E + 05	8.70E + 01	-	1.20E + 05	-
Rock Springs								1.0%	-	2.5 mm	-
RSA7	38.0	3.7	38	120	0.0	0.0	1.25E + 06	2.30E + 01	-	4.11E + 04	-
RSA111	38.0	5.7	38	120	0.0	0.0	1.25E + 06	1.80E + 01	-	3.38E + 04	-
RSA112	38.0	6.4	38	120	0.0	0.0	1.25E + 06	1.05E + 02	-	1.40E + 05	-

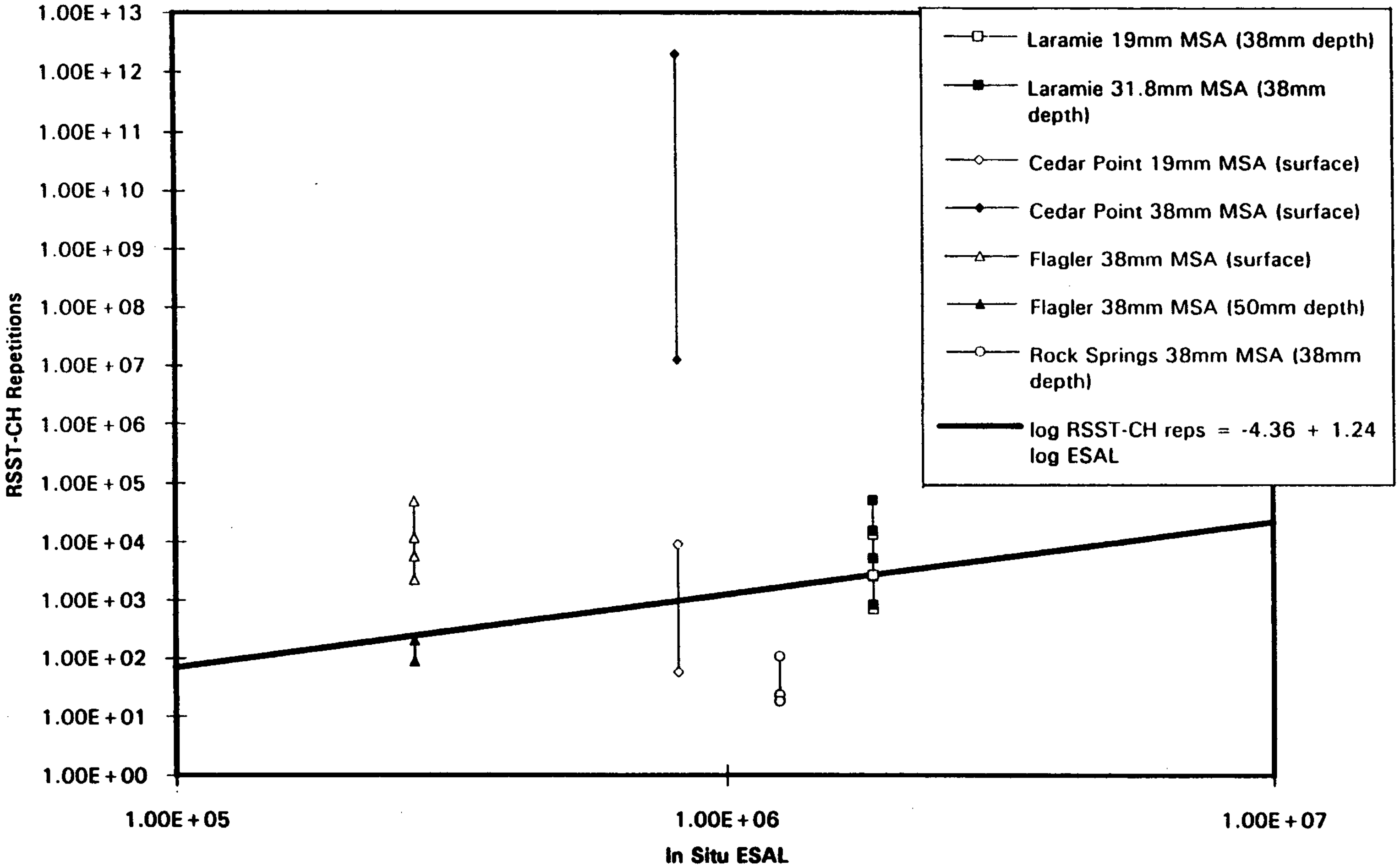

Figure A-18. Comparison of LSM data with Sousa and Solaimanian transfer function (74).

the relationship generated from the GPS site data. This indicates that the shift factor is portable to LSM, despite their lack of representation in the original GPS database.

Summary of Findings

RSCH results were compared with rutting depths measured in situ and ESAL data and used to evaluate the transfer function developed by Sousa and Solaimanian. The following was concluded:

- The new techniques developed for testing LSM with the RSCH test were effective and, in combination with a large specimen size, appear to result in satisfactory data for this type of paving mix.
- The Sousa and Solaimanian transfer function appears to be applicable to LSM, although it was developed from a GPS database that did not include LSM.

TESTS AND RESULTS ON LABORATORY-COMPACTED LSM SPECIMENS

Development of a Laboratory Compaction Procedure for LSM

Gyratory Compaction

Ideally, this project would have used a Superpave gyratory compactor; however, these compactors were not yet commercially available when the study began. Further, the maximum size specimen that the Superpave compactor produces is 6 in. in diameter by 6 in. high (150 mm by 150 mm). The large Texas gyratory compactor can routinely compact specimens 6 in. (150 mm) in diameter and about 10.8 in. (274 mm) high and uses a 5-deg gyratory angle. The rate of gyration is 30 rpm. TTI's Texas gyratory compactor has been modified to accommodate 7.5-in. (191-mm)-diameter specimens. Therefore, 6-in. (150-mm)-diameter and 7.5-in. (191-mm)-diameter LSM specimens were used in this study. Maximum height was limited to about 10.8 in. (274 mm). The large Texas gyratory compactor with 6-in. and 7.5-in. (150- and 191-mm)-diameter molds is shown in Figure A-19. To accurately measure engineering properties of mixes containing large stones (see Figure A-20), one needs large specimens (see Figure A-21).

Before the compaction study began, the gyration angle of the large Texas gyratory compactor was changed from 5 to 1.25 deg to simulate the Superpave gyratory compactor as closely as possible. In an effort to develop a suitable compaction procedure for LSM in this study, many LSM specimens were compacted using different compaction head pressures and gyration times (number of gyrations) and different aggregate gradations. LSM paving materials from Arizona DOT, Indiana DOT, Kentucky DOT, and New Mexico DOT were compacted in this work element. All

Figure A-19. Large Texas gyratory compactor with 150-mm (6-in.)- and 191-mm (7.5-in.)-diameter molds.

compaction development work used specimens with dimensions of 7.5 in. (191 mm) in diameter and approximately 7.5 in. (191 mm) in height. More than 3 tons (2,722 kg) of materials were compacted in the laboratory using the modified Texas gyratory compactor.

To achieve reasonable densification, compaction of several specimens was performed at a gyratory head pressure of 147 psi (1,014 kPa) for 4 min at a gyratory angle of 1.25 deg, followed by a leveling load of 1,000 pounds (450 kg) for 1 min. However, when air voids of gyratory compacted mixes using the Kentucky DOT mix design were compared with air voids content of similar mixes compacted in the field and subjected to about 2 years of traffic, it was found that the air voids content of the laboratory mixes (9 to 13 percent) were much greater than those of the field mixes (4 to 8 percent). Furthermore, the 6-in. (150-mm)-diameter Marshall hammer had been used by KentuckyDOT during mix design to produce specimens of this same material with 4.5 percent air voids. The gyratory head pressure was increased incrementally from 60 psi to 150 psi (414 to 1,034 kPa) and the gyration time was increased to 5 min and no appreciable decrease in air voids was observed in

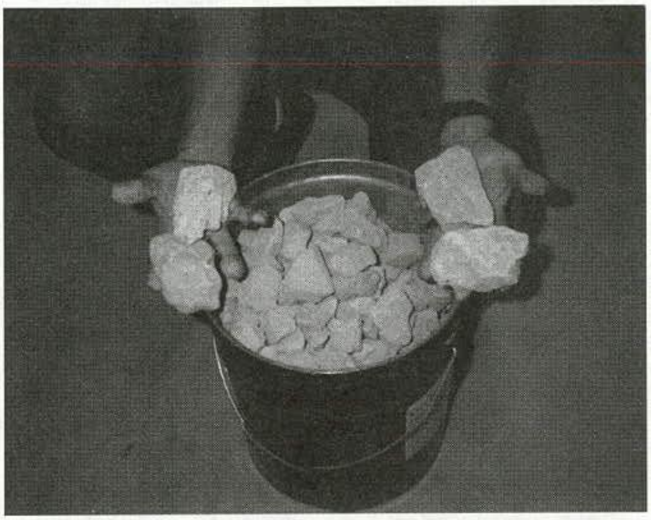

Figure A-20. Large stones 50 mm (2 in.) to 63 mm (2.5 in.) in diameter.

subsequent specimens. However, much aggregate fracture was experienced, particularly around the periphery of the top of the specimen. In addition, the roller bearings that produce the gyratory action of the mold were broken several times, demonstrating that the device was not designed for such high pressures.

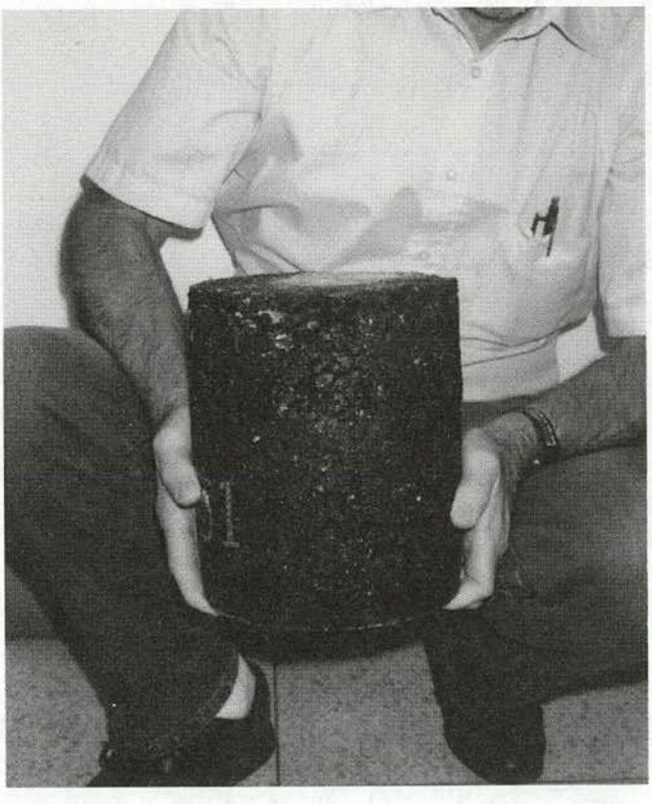

Figure A-21. Compacted asphalt specimen 191 mm (7.5 in.) in diameter and 216 mm (8.5 in.) in height.

Upon changing the gyratory angle back to the original 5 deg, the air voids content of the Kentucky DOT mix dropped to less than 2 percent with the 150 psi (1,034 kPa) head pressure. (Data on all the specimens prepared during this compaction study are not reported here.)

Using the 5-deg angle, the gyratory head pressure was reduced to 35 psi (241 kPa) and air voids were still lower than those obtained using 150 psi (1,014 kPa) with the 1.25-deg gyratory angle (see Table A-13). It was concluded, therefore, that the higher angle of gyration was necessary to provide adequate mechanical energy (kneading action) to simulate "terminal" compaction (i.e., that imparted during construction plus 2 to 4 years' worth of traffic) of these large LSM specimens. Further research is needed to definitively investigate compaction of LSM in the laboratory.

Consultation with the FHWA suggested that it is generally difficult to densify harsh mixes or specimens higher than 3 in. (76 mm) using the Superpave gyratory compactor at the specified 1.25-deg gyratory angle and 87 psi (600 kPa). The present model of the Superpave compactor can accommodate a maximum gyratory angle of about 2 deg and a maximum specimen size of 6-in. (150-mm) in diameter by 6-in. (150-mm) in height. The FHWA indicated that later versions of the compactor may provide greater gyratory angles and affirmed that the higher gyration angle is more important as specimen height increases.

After studying several variations of compaction techniques using the large Texas gyratory compactor, a method was selected for laboratory preparation of dense-graded 7.5- by 7.5-in. (191- by 191-mm) LSM specimens using the gyratory compactor. The following compaction procedure was used for all remaining specimens of various configurations: 40 psi (276 kPa) head pressure on the specimen, 5-deg angle, 4-min gyration time (or 120 gyrations), and a 27-psi (186-kPa) leveling load for 1 min. The compaction temperature was selected to provide an asphalt viscosity of 280 centistokes. Some aggregate breakage was still noticed, particularly at the outer edges of the top and bottom of the specimens. To control air voids content to some specified level, the compaction procedure would be different for specimens of different sizes. All specimens compacted in this study were higher than 7 in. (178 mm); they were sawn transversely to produce the shorter specimens needed.

Rolling wheel compaction may be a viable option for preparing large LSM specimens. To fully develop compaction procedures that simulate field processes, more work is needed than was possible in this study. The compaction device will significantly influence properties of the specimen.

Measurement Required for Air Voids Content Calculations

After measuring the air voids content of whole specimens, selected 7.5-in. (191-mm)-diameter specimens were cored using a 4-in. (102-mm)-diameter core drill, and then the smaller core was cut into three layers. The air voids content of each piece was determined. This was done to examine the axial and radial density distribution within the specimens. Generally, the differences in density of the various pieces were less than 2 percent and there were no consistent density gradients within the specimens, either radially or axially. This variation was considered adequate. Table A-13 shows some representative data regarding air voids content distribution as a function of compaction procedure.

Bulk specific gravity of these large specimens was measured using five different methods: standard saturated surface dry, covering the specimen with the parafilm, covering the specimen with masking tape, covering the specimen with a plastic bag, and immersing the specimen in a known volume of glass beads. Of course, covering the specimen with a membrane usually gave higher air voids content. Parafilm worked reasonably well, but it was almost impossible to apply to these comparatively large, rough specimens without the large stones punching holes in it or entrapping air between the parafilm layers. Some type of film is needed to accurately measure bulk specific gravity of LSM when air voids contents are above about 7 percent; however, the film often interferes with accurate measurements at lower air voids content. If parafilm is used, parafilm-coated specimens should be handled carefully on a foam-rubber-padded counter top to minimize punching holes in the film.

TABLE A-13 Air Voids of Kentucky DOT Mixture Using 1.25- and 5-Deg Angle with Large Texas Gyratory Compactor

Asphalt Content, %	Gyratory Press, psi	Gyratory Angle, degrees	Specimen Description	Gyration Time, min	Air Voids, % (SSD)	Air Voids, % (Parafilm)
3.3	150	1.25	Whole	4	9.9	13.3
			4-In Core	4	8.7	-
3.3	150	1.25	Whole	4	9.7	14.0
			4-In Core	4	8.8	-
3.3	150	1.25	Whole	4	9.2	14.7
			4-In Core	4	9.1	-
3.5	150	1.25	Whole	4	12.3	-
			4-In Core	4	11.2	-
4.3	150	1.25	Whole	4	9.0	-
			4-In Core	4	10.4	-
2.7	150	1.25	Whole	4	11.3	-
			4-In Core	4	11.3	-
3.3	40	5	Whole	4	6.4	-
			Annulus	4	6.4	-
			4-In Core	4	6.1	-
			Top	4	7.1	-
			Middle	4	6.1	-
			Bottom	4	5.8	-
3.3	60	5	Whole	4	6.1	-
			Annulus	4	6.1	-
			4-In Core	4	5.7	-
			Top	4	7.0	-
			Middle	4	4.7	-
			Bottom	4	6.5	-
3.3	60	5	Whole	4	5.1	-
			Annulus	4	5.2	-
			4-In Core	4	4.6	-
			Top	4	3.7	-
			Middle	4	3.8	-
			Bottom	4	6.3	-
3.3	75	5	Whole	2	7.5	-
			Annulus	2	5.6	-
			4-In Core	2	5.0	-
			Top	2	5.3	-
			Middle	2	3.9	-
			Bottom	2	6.1	-

(Continued on next page)

TABLE A-13 Air Voids of Kentucky DOT Mixture Using 1.25- and 5-Deg Angle with Large Texas Gyratory Compactor *(Continued)*

Asphalt Content, %	Gyratory Press, psi	Gyratory Angle, degrees	Specimen Description	Gyration Time, min	Air Voids, % (SSD)	Air Voids, % (Parafilm)
3.3	40	5	Whole	2	7.9	-
			Annulus	2	8.4	-
			4-In Core	2	7.5	-
			Top	2	7.4	-
			Middle	2	7.4	-
			Bottom	2	8.7	-
3.3	150	5	Whole	4	4.3	6.2
			Annulus	4	4.1	-
			4-In Core	4	4.3	-
			Top	4	3.9	-
			Middle	4	3.0	-
			Bottom	4	4.4	-
3.3	150	5	Whole	4	2.8	4.3
			Annulus	4	2.3	-
			4-In Core	4	2.2	-
			Top	4	2.4	-
			Middle	4	1.7	-
			Bottom	4	2.2	-

LSM, even dense-graded mixes, have air voids of larger volume than conventional mixes with equivalent air voids content. As a result, when measuring bulk specific gravity in accordance with AASHTO T 166, water may freely enter these larger voids, resulting in the calculation of incorrectly low air voids content. A practical, reasonably accurate method for measuring bulk specific gravity of compacted LSM specimens with permeable voids uses glass beads in place of water. The procedure is described in Chapter 3 of the report.

RSCH Test on Lab-Molded Samples—Tested at TTI

Rutting susceptibility of LSM was evaluated from repetitive simple shear tests using the SST device. Specimens were laboratory-compacted, dense-graded LSM designed using the Level 1 TTI mix design procedure and limestone aggregates from Indiana DOT. Asphalt content varied from optimum (2.8 percent) by about 0.8 percent above and below optimum. An additional aggregate grading (0.45 power or maximum density grading) was tested as a control mix. The goals of this limited study were to evaluate the Level 1 LSM mix design procedure, provide a link to the Superpave mix analysis, and provide a basis for establishing a Level 2 mix design procedure. The data demonstrated the potential for generating additional confining pressures because of the tendency of the mixes to dilate or expand under load.

The SST device was received (at TTI) just before this LSM study was due to end. Consequently, only a few RSCH tests could be conducted on laboratory-molded specimens of dense-graded and open-graded LSM using it.

Findings

A description of the LSM and their qualitative rankings are given in Table A-14. Higher rankings in the table denote smaller predicted rutting depths or greater rutting resistance. Figure A-22 shows the data on accumulated shear strains measured for the specimens tested and analyzed. The data were used to predict the progression of rutting depth with increasing 18-kip ESAL applications using the procedure developed by Sousa et al. (*74*) described above.

Figure A-23 shows the predicted performance curves on the basis of rut depths that were obtained with this procedure. The predicted rut depths were used to rank the LSM tested in terms of rutting resistance. As expected, increasing asphalt content yielded a corresponding decrease in rutting resistance (see Table A-14).

TABLE A-14 Description of Specimens and Ranking of Rut Resistance Based on Predicted Rut Depths for Molded LSM Specimens

Specimen ID	Grading	Asphalt Content, %	Ranking of Rut Resistance
M7_IND	TTI Design*	2.5	1
M8_IND	TTI Design	2.0	2
IND_45.2	0.45 power	4.8	3
M4_IND	TTI Design	2.8	3
M5_IND	TTI Design	3.0	4
M9_IND	TTI Design	3.5	5

* The TTI mixture design is described in detail in Tables D-1 and D-2 of Appendix D. It is termed "Texas coarse" in those tables.

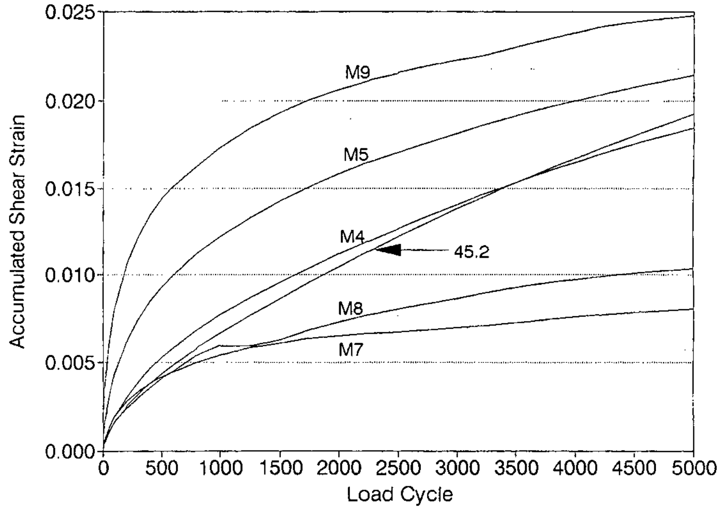

Figure A-22. Compacted shear strain data for LSM tested under repetitive shear.

The data from the RSCH tests also demonstrated the tendency of the LSM specimens to dilate under repeated shear loading. This behavior is indicated in Figure A-24, which shows that a compressive vertical load was required to maintain a given specimen at constant height during the test and that the magnitude of this load increased with the number of shearing cycles applied. If the total change in the compressive load is used as an indirect measure of dilation, a qualitative ranking of the specimens in terms of the tendency to dilate may be obtained as shown in Table A-15. A higher ranking in the table denotes a larger change in the compressive load required to maintain constant height or a greater tendency to dilate.

An evaluation was made to determine if there is any correlation between the tendency of the materials to dilate and the predicted resistance to rutting. The accumulated shear strains at the last load cycle of the repeated shear test were plotted against the corresponding measured changes in vertical compressive load required to maintain constant height (see Figure A-25). The data suggest that there may be an optimum level of dilatancy at which the rutting susceptibility is at a minimum. Figure A-25 shows that the IND_45.2 mix (which had less stone-on-stone contact) exhibited the least dilation among the mixes evaluated. However, the measured accumulated shear strain at the last load cycle for this specimen is more than twice the value for the M7_IND mix, which had the lowest accumulated shear strain.

The tendency to dilate is expected to generate additional confining pressures in pavements, which would have a stiff-

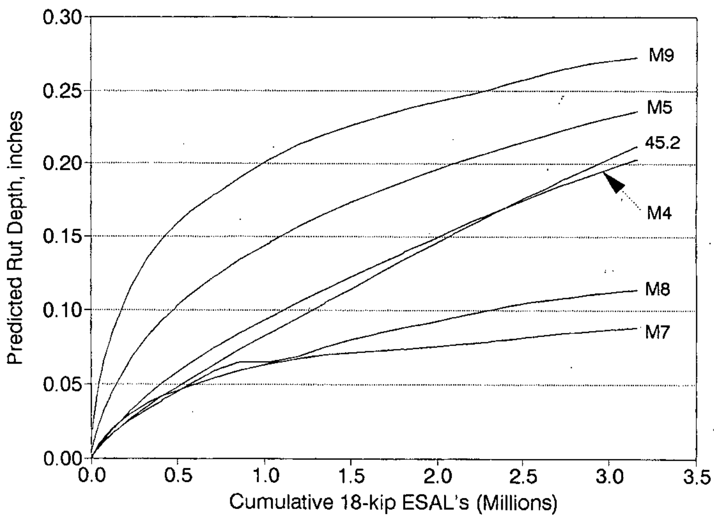

Figure A-23. Predicted development of rutting for LSM tested under repeat shear.

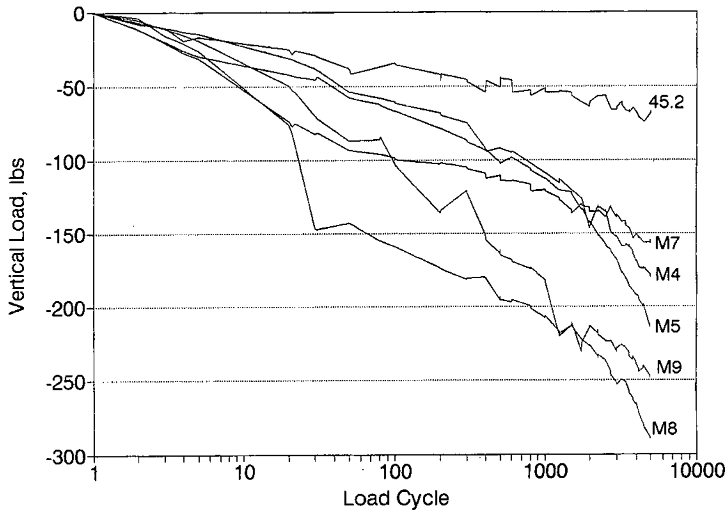

Figure A-24. Increase in compressive load required to maintain a constant height for a specimen tested under repeated shear.

ening effect and result in less permanent strains. Consequently, some level of dilation will be beneficial. However, the data in Figure A-25 also suggest that, at levels of dilation above that measured for the M7_IND specimen, the potential for permanent deformation increases, with the exception of the M8_IND specimen. The accumulated shear strain curve for this specimen (see Figure A-22) exhibits a dip, which may explain the relatively low permanent shear strain measured for the specimen at the last load cycle. Extrapolation of the trend in the data prior to the dip indicates that the permanent strain would have been more comparable to those measured for the M5_IND and M4_IND mixes than to that measured for the M7_IND mix. The dip in the data may be the result of slippage in the sensor used for measuring shear strain. This slippage may have produced a change of reference for the sensor during the test, thus explaining the low measurement of accumulated shear strain. Taking this into consideration, the data suggest that too much dilation may have the unwanted effect of destabilizing the aggregate skeleton of the mix, thereby reducing the degree of stone-on-stone contact and, thus, producing a greater susceptibility to permanent deformation under repeated loading.

Summary

Data from Superpave RSCH tests on laboratory-molded LSM suggest that an optimum level of dilation exists at which a given LSM will be the most rutting resistant. This finding reflects limited data. More research is needed to understand the effects of dilation and how this behavior is affected by mix characteristics. Although the additional confinement because of dilation would tend to stiffen a mix, test results indicate that allowing too much dilation will not increase rutting resistance. Further investigations into the dilatant behavior of LSM and conventional mixes are needed in order to develop guidelines that will optimize the amount of dilatancy mobilized under traffic loading, and thus, enhance the rutting resistance of paving mixes.

TABLE A-15 Ranking of Specimens on the Basis of Tendency to Dilate

Specimen ID	Total Change in Compressive Load to Maintain Constant Height, lbs	Ranking of Tendency to Dilate
M8_IND	290	1
M9_IND	249	2
M5_IND	214	3
M4_IND	180	4
M7_IND	154	5
IND_45.2	67	6

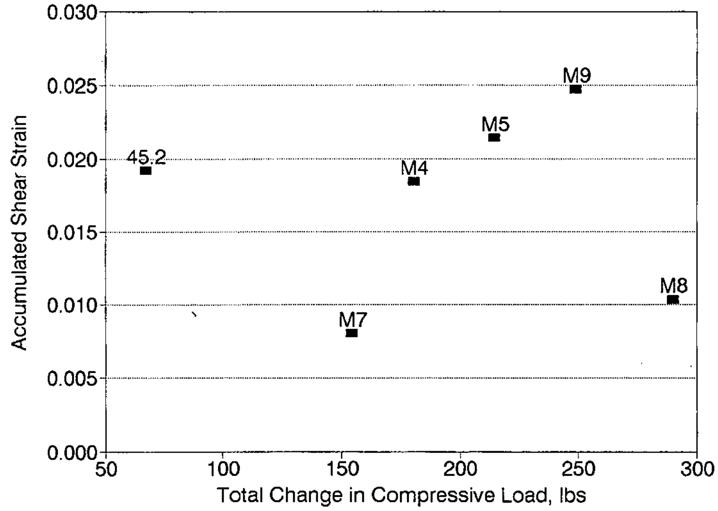

Figure A-25. Variation in accumulated shear strain at last load cycle with the total change in compressive load required to maintain constant height.

Effect of Specimen Size on Measured Mix Properties

Mix Design

To develop laboratory-prepared LSM specimens suitable for use in the study of the effect of specimen size on measured mix properties, researchers used asphalt mixes composed of 100 percent limestone aggregates. The aggregates were sieved, separated into bins, and recombined to produce the desired gradations. An AC-20 asphalt cement was used as binder. Materials were mixed at 153°C (308°F), where the asphalt viscosity is 0.17 Pa•s (170 cP) and compacted at 145°C (293°F), where the viscosity is 0.28 Pa•s (280 cP). For mix design, compaction of 7.5-in. (191-mm)-diameter specimens was performed using the large Texas DOT gyratory compactor with a head pressure of 150 psi (1,034 kPa) and a gyratory angle of 1.25 deg. Air voids contents were measured using parafilm-covered specimens (see Table A-16). This work was performed early in the study before the compaction procedure was modified (40 psi [276 kPa] at a 5-deg gyratory angle) as discussed earlier. Therefore, the air voids contents for these mix design specimens are higher than those prepared for the subsequent study of the effects of specimen size on measured mix properties.

TABLE A-16 Laboratory-Molded 191-mm (7.5-in.)-Diameter Specimens Using 50-mm (2-in.)-Maximum-Size Limestone Aggregates with Various Asphalt Contents Tested at 56°C (133°F)

File Name	Height, inch	Gage Length, inch	Asphalt Cont., %	Air Voids, %	Energy @ 2%, lb-in/ft^3	Strength, psi	Modulus, psi
K17-2	7.30	4.89	1.7	15	2.28	134	29,321
K17-1	7.55	5.50	1.7	15	1.59	97	24,431
K22-8	6.95	5.06	2.2	12	2.32	143	26,382
K22-4	7.30	5.04	2.2	14	2.04	132	14,527
K27-1	7.10	4.78	2.7	11	1.43	95	19,412
K27-2	7.05	4.33	2.7	11	1.78	110	15,648
K33-1	6.50	5.06	3.3	11	1.68	175	8,885
K35-2	6.90	4.92	3.5	11	-	-	-
K35-1	6.80	4.85	3.5	12	1.37	87	16,925
K43-2	7.00	4.64	4.3	10	0.29	39	3,898
K43-1	6.90	2.82	4.3	9	-	-	-

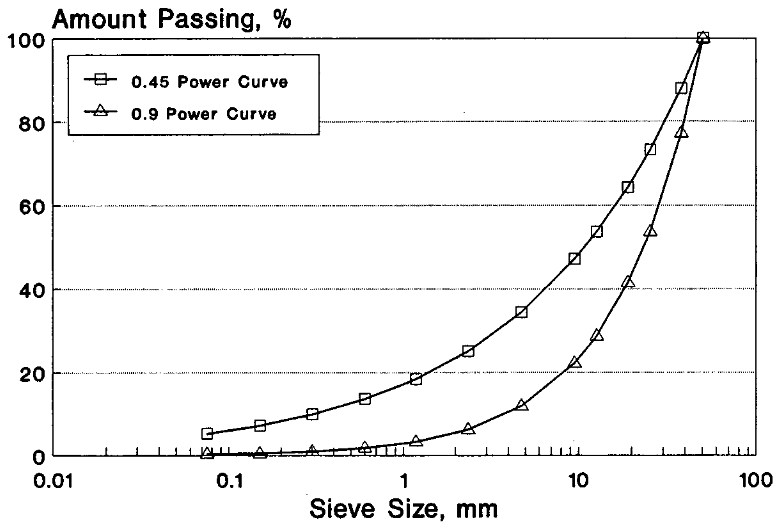

Figure A-26. Gradation of asphalt mixtures used in study of specimen size on measured mixture properties.

An aggregate blend with a gradation curve exponent of 0.45 (see Figure A-26) was selected to produce a dense-graded mix reasonably representative of many LSM used around the United States. This aggregate grading has a specific surface area of about 26 ft^2/lb (5.3 m^2/kg). Figure A-27 shows results from monotonic compressive strength tests on the dense-graded mix with various asphalt contents. Figure A-28 shows strain energy density (i.e., energy per unit volume required to produce 2 percent of the strain at failure) data during the same series of tests. In a uniaxial test, strain energy density (or strain per unit volume, *W*) is simply the area under the stress-strain (*F*-,) curve, up to the indicated strain level and is defined mathematically as

$$W = \int_0^{\varepsilon} \sigma d\varepsilon$$

Figure A-29 illustrates that modulus decreases linearly as asphalt content is increased over the range of asphalt contents tested. On the basis of these monotonic data alone, it appears that the optimum asphalt content should be about 2.2 percent. However, the mixes at 2.2 percent asphalt appeared too lean and the calculated film thickness was only about 4.2 μm. On the basis of these data and the general appearance of the compacted specimens, an optimum asphalt content of 2.9 percent by weight of mix was selected.

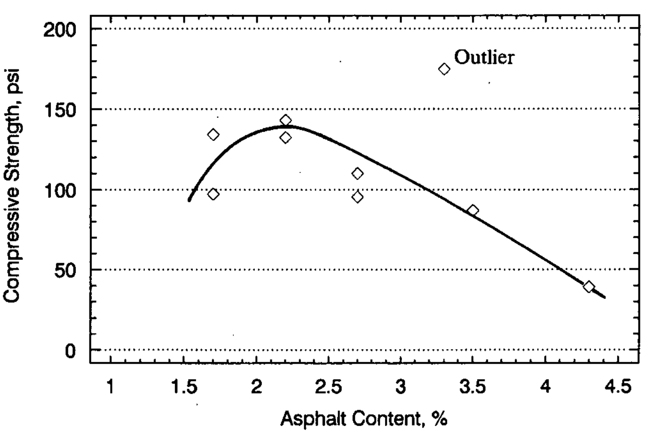

Figure A-27. Compressive strength versus asphalt content.

An open-graded mix, with a grading curve exponent of 0.9 and a maximum aggregate size of 1.5 in. (38 mm) (see Figure A-26), was designed using these same materials. Assuming a film thickness of about 20 μm would be suitable, an optimum asphalt content of 2.0 percent was selected for the open-graded mix. Subsequent drain-down tests revealed that a more suitable asphalt content for paving purposes would have been about 0.4 percent higher.

Because of the timing of the project, this mix design work had to be performed before the LSM design procedures were completely established. However, aggregate gradings and optimum asphalt contents selected for this laboratory experiment were not critical as long as they were reasonably representative of field mixes.

Experiment Design

A test plan was developed to determine the sensitivity of measured mix properties to specimen size and maximum aggregate size. Such sensitivity is particularly important when testing LSM in the laboratory. To optimize economics and convenience, the smallest specimen possible should be tested that does not adversely affect the mix properties being measured. Testing of these laboratory-prepared specimens

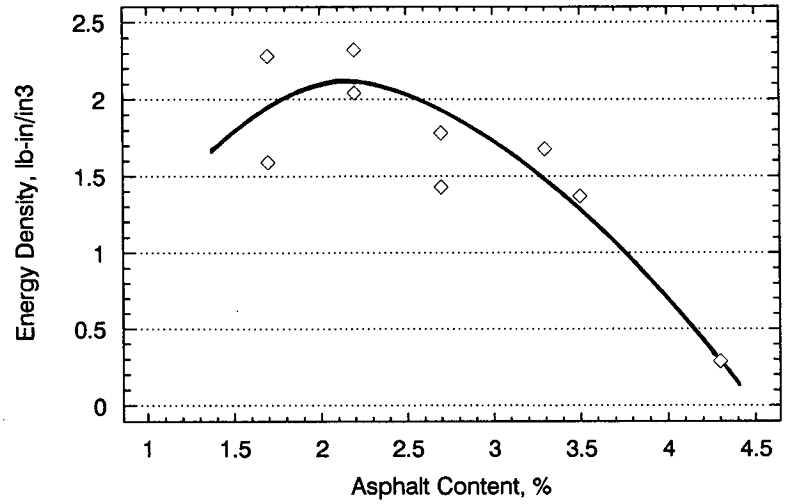

Figure A-28. Energy density of 2 percent strain versus asphalt content.

was performed as described earlier for the pavement cores. Crushed limestone aggregate blended with AC-20 asphalt binder was used in the large Texas gyratory compactor to prepare the specimens.

The experiment described in Table A-17 evaluates several issues: specimen height and diameter, specimen height-to-diameter ratio, maximum particle size-to-specimen size ratio for 0.45 power gradation, and the influence of a more open or coarser (0.9 power) gradation. The power of the gradation curve is n in the following equation:

$$P = 100\ (d/D)^n \qquad \text{(Eq. 4)}$$

where

P = weight percent of aggregate passing a given sieve
d = diameter of the sieve size in question
D = maximum size of the aggregate.

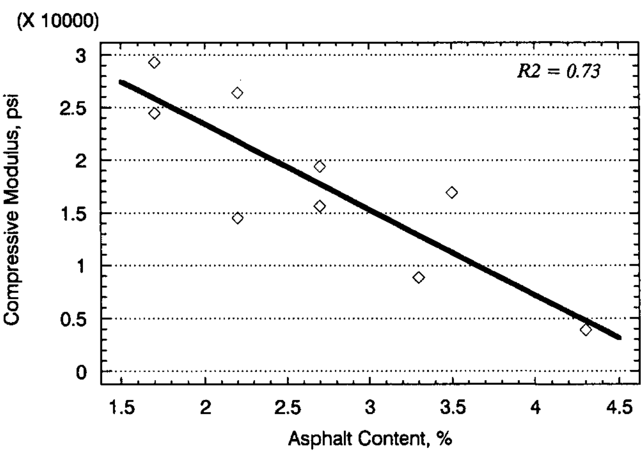

Figure A-29. Compressive modulus versus asphalt content.

In addition, an analysis of variance (ANOVA) was conducted to determine whether 2.5-in. (63-mm)-diameter aggregates produce better performing mixes (in terms of compressive strength, creep resistance, and plastic deformation resistance) than 1.5-in. (38-mm)- or 1.0-in. (25-mm)-diameter materials and if 7.5-in. (191-mm)-diameter specimens yield different results from 6-in. (150-mm)-diameter specimens. The raw data from this experiment are provided in Tables A-18 and A-19.

Comparisons from this experiment included (1) ranking of the three aggregate size treatments in the 0.45 power matrix, (2) evaluating whether results from 6-in. (150-mm)-diameter specimens are different from those of 7.5-in. (191-mm)-diameter specimens in the 0.45 power matrix, and (3) evaluating whether a 0.45 power gradation is different from a 0.9 power gradation for the 1.5-in. (38-mm)-diameter aggregate. The height-to-diameter ratio did not reach 2:1 as desired for this experiment because the maximum height of a 7.5-in. (191-mm)-diameter specimen prepared using the Texas gyratory compactor is about 9.5 in. (241 mm), thus reducing the maximum ratio to approximately 1.3:1. The maximum height for 6-in. (150-mm)-diameter specimens is 10.8 in. (274 mm), thus yielding a maximum height-to-diameter ratio of 1.8:1.

Results and Discussion

The analysis of the sensitivity of mix properties to specimen size yielded several very important findings. Figure A-30 is a plot of monotonic, unconfined, axial compressive strength of dense-graded LSM (0.45 gradation) as a function of the smallest specimen dimension divided by the maximum aggregate size (SD/AS). These specimens are

TABLE A-17 Experiment Design for Determining Sensitivity of Mixture Properties to Specimen Size

Approximate Height:Diameter Ratio	0.45 power Gradation			0.9 power Gradation
	1.0 inch Aggregate	1.5 inch Aggregate	2.5 inch Aggregate	1.5 inch Aggregate
1.8:1	3[1]	0	3[1]	0
1.3:1	7[2]	4[3]	7[2]	4[3]
1.0:1	4[3]	9[3]	4[3]	4[3]
0.375:1	4[3]	4[3]	7[2]	4[3]

No. Specimens Tested: 68
Mold Diameter: [1]6-inch only, [2]3 @ 6-inch and 4 @ 7.5-inch, [3] 7.5-inch only
Aggregate: crushed limestone stone
Test: unconfined creep & recovery followed by axial loading to failure
Environment: 104°F (40°C)
Instrumentation: axial LVDT

cylindrical, and the smallest specimen dimension may be either diameter or height. Although the data are rather limited, Figure A-30 is very revealing with regard to the required laboratory test specimen dimensions for LSM. The figure shows that (1) when the smallest specimen dimension is at least 4 times bigger than the largest aggregate size, the actual strength of the mix is measured; (2) when the smallest dimension is less than 2.5 times the largest aggregate size, aggregate strength masks the measurement of mix strength; therefore, specimens with a dimension smaller than this should never be tested; and (3) for specimens where the SD/AS falls between 2.5 and 4.0, a correction factor (see Figure A-31) needs to be applied to compute the actual mix strength. This correction factor will vary, of course, depending on the mix property being measured.

Figure A-32 exhibits a trend for energy density as a function of SD/AS similar to that in Figure A-30. Figure A-33 demonstrates that no such trend exists for modulus as a function of SD/AS.

Figure A-34 shows that the height-diameter ratio for dense-graded LSM (0.45 power gradation) should be at least 1.0 to avoid significant interference with compressive strength measurements from the larger aggregates in the mix. Figure A-35 exhibits a trend for energy density as a function of height-diameter ratio similar to that in Figure A-34. Figure A-36 demonstrates that no such trend exists for modulus as a function of height-diameter ratio.

Figures A-30 and A-34 show that for a maximum aggregate size of 2 in. (50 mm), a minimum specimen dimension larger than 5.0 in. (i.e., 2 in. multiplied by 2.5 = 5.0 in. [50 mm multiplied by 2.5 = 125 mm]) should be satisfactory. However, a correction factor will need to be applied to compressive strength data, unless the smallest sample dimension is larger than about 8.0 in. (i.e., 2 in. multiplied by 4 = 8.0 in. [50 mm multiplied by 4 = 200 mm]).

These data further indicate that a 6-in.-diameter-by-6-in.-high (150-mm-by-150-mm) specimen will accommodate aggregate sizes up to 1.5 in. (38 mm) without correction (6 divided by 4 = 1.5). Furthermore, a 6-in.-diameter-by-6-in.-high (150-mm-by-150-mm) specimen will accommodate aggregates up to 2.4 in. (i.e., 6 in. divided by 2.5 = 2.4 in.) [152 mm multiplied by 2.5 = 381]; however, for a maximum stone size of 2.4 in. (61 mm), measured unconfined compressive strength of a specimen with a minimum dimension of 6 in. (150 mm) would need to be multiplied by a correction factor of about 0.76 (from Figure A-31) to obtain the actual strength value.

Figures A-37 and A-38 show results from the 1-hr creep and recovery tests for the 0.45 power gradation laboratory-compacted specimens. Creep compliance and plastic strain measured at 0.6 sec are plotted as a function of smallest specimen dimension divided by the maximum aggregate size (SD/AS). Although there is considerable scatter in the data (which is fairly typical for creep tests), the trend line indicates that actual mix properties are measured when the SD/AS is greater than 4 and that only a small error in mix properties is encountered when the SD/AS is between 3 and 4. No logical trend was found between creep compliance or plastic strain and height-diameter ratio for the 0.45 power gradation mixes.

The LSM specimens tested in this experiment contained aggregates from 1 in. (25 mm) to 2.5 in. (63 mm). Specimen diameters were either 6 in. or 7.5 in. (150 mm or 191 mm) (see Tables A-18 and A-19). Specimen heights ranged from about 2 in. to 10.25 in. (50 mm to 260 mm). On the basis of compressive strengths, energy densities, creep compliance, and plastic strain from the monotonic tests, a cylindrical specimen size of 6 in. in diameter by 6

TABLE A-18 Summary of Specimens Prepared for Sensitivity Study and Results of Unconfined Axial Compression Tests

File Name	Grad-ation Power	Dia-meter, in.	Height, in.	Gage Length, in.	Max Aggr Size, in.	Asp. Cont, %	Air Void, %	Energy Density, lb-in/in³	Comp. Strength psi	Comp. Modulus psi	Smallest Dimen/ Max Aggr Size	Height/ Diameter Ratio
S2254	0.45	7.5	2.85	1.29	2.5	2.90	6.1	6.01	423.2	32772	1.14	0.38
S2151	0.45	7.5	2.90	1.51	1.5	2.90	6.5	5.02	412.3	43016	1.93	0.39
S213	0.45	7.5	3.05	1.68	1	2.90	6.5	6.19	410.9	80888	3.05	0.41
S212	0.45	7.5	3.05	1.74	1	2.90	5.7	5.54	414.7	33840	3.05	0.41
S211	0.45	7.5	3.05	1.82	1	2.90	5.7	2.25	293.7	9866	3.05	0.41
S2252	0.45	7.5	3.05	1.94	2.5	2.90	6.1	6.43	423.6	39761	1.22	0.41
S2253	0.45	7.5	3.10	1.60	2.5	2.90	6.1	-	412.5	70771	1.24	0.41
S214	0.45	7.5	3.10	1.70	1	2.90	6.5	2.37	344.2	11476	3.10	0.41
S2251	0.45	7.5	3.10	1.82	2.5	2.90	6.1	6.16	422.2	35743	1.24	0.41
S2154	0.45	7.5	3.10	1.76	1.5	2.90	6.9	4.55	405.5	23644	2.07	0.41
S2152	0.45	7.5	3.30	1.99	1.5	2.90	6.5	-	404.0	-	2.20	0.44
S7152	0.45	7.5	7.20	5.03	1.5	2.90	5.7	3.90	273.9	44010	4.80	0.96
S7153	0.45	7.5	7.20	5.35	1.5	2.90	5.7	3.32	256.0	46480	4.80	0.96
S7251	0.45	7.5	7.40	5.28	2.5	2.90	1.8	3.48	235.9	44115	2.96	0.99
S7157	0.45	7.5	7.40	5.71	1.5	2.90	6.2	2.41	181.7	26087	4.93	0.99
S711	0.45	7.5	7.50	5.54	1	2.90	4.6	2.66	213.7	20329	7.50	1.00
S712	0.45	7.5	7.50	3.47	1	2.90	4.7	2.48	228.9	22080	7.50	1.00
S7155	0.45	7.5	7.50	5.40	1.5	2.90	6.3	2.37	196.1	25855	5.00	1.00
S7156	0.45	7.5	7.55	4.51	1.5	2.90	6.8	3.17	210.1	38470	5.00	1.01
S7159	0.45	7.5	7.55	5.40	1.5	2.90	6.6	3.47	268.4	28035	5.00	1.01
S713	0.45	7.5	7.55	5.66	1	2.90	4.6	3.87	293.7	36317	7.50	1.01
S7151	0.45	7.5	7.55	5.56	1.5	2.90	6.1	2.36	189.1	27448	5.00	1.01
S714	0.45	7.5	7.55	5.49	1	2.90	4.6	3.79	288.1	36467	7.50	1.01
S7158	0.45	7.5	7.60	5.64	1.5	2.90	6.3	2.66	211.5	20964	5.00	1.01
4510252	0.45	7.5	9.10	7.59	2.5	2.90	4.3	2.93	218.4	18932	3.00	1.21
4510153	0.45	7.5	9.20	8.08	1.5	2.90	4	2.11	138.0	28900	5.00	1.23
4510152	0.45	7.5	9.25	7.68	1.5	2.90	5.6	1.85	119.9	27471	5.00	1.23
4510151	0.45	7.5	9.25	7.64	1.5	2.90	5.1	2.88	179.3	29408	5.00	1.23
4510251	0.45	7.5	9.30	7.70	2.5	2.90	4.2	2.57	156.1	37733	3.00	1.24
4510254	0.45	7.5	9.30	7.74	2.5	2.90	4.8	3.08	193.5	34525	3.00	1.24

in. in height (150 mm by 150 mm) should be acceptable for use with LSM containing stones up to 2 in. (50 mm) in diameter, provided a correction is made when the maximum aggregate size exceeds 1.3 in. (33 mm). It is significant that this is about the maximum specimen size that the Superpave gyratory compactor can produce. Further, on the basis of findings from the literature and the field, it is estimated that the nominal maximum size of more than 95 percent of the dense-graded LSM used in the United States is less than 2 in. (50 mm). This large experiment included only one aggregate type, one asphalt grade, and two gradations. Much more testing on a wide variety of LSM materials is needed before accurate statements can be made and correction factors can be generated for all mixes.

Figure A-39 shows an increase in unconfined compressive strength with an increase in maximum aggregate size for the specimens composed of the 0.45 power gradation mixes. Figure A-40 shows a similar increase in the energy density required to produce 2 percent strain during these

TABLE A-18 Summary of Specimens Prepared for Sensitivity Study and Results of Unconfined Axial Compression Tests *(Continued)*

File Name	Grad-ation Power	Dia-meter, in.	Height, in.	Gage Length, in.	Max Aggr Size, in.	Asp. Cont, %	Air Void, %	Energy Density, lb-in/in³	Comp. Strength psi	Comp. Modulus psi	Smallest Dimen/ Max Aggr Size	Height/ Diameter Ratio
4510253	0.45	7.5	9.35	7.72	2.5	2.90	4	1.89	126.4	27901	3.00	1.25
4510154	0.45	7.5	9.40	7.84	1.5	2.90	4.8	2.18	142.0	29447	5.00	1.25
451012	0.45	7.5	9.50	7.83	1	2.90	6	2.09 @ 1.9%	145.9	21253	7.50	1.27
451013	0.45	7.5	9.50	7.90	1	2.90	5.8	2.39	161.2	22164	7.50	1.27
451014	0.45	7.5	9.55	7.93	1	2.90	6.6	2.74	184.6	25628	7.50	1.27
S2153	0.45	7.5	2.80	1.82	1.5	2.90	6.9	4.52	427.6	21898	1.87	0.38
S2253B	0.45	6.0	2.15	1.48	2.5	2.90	6.1	5.34 @ 1.6%	620.4	31694	0.86	0.36
S2251B	0.45	6.0	2.25	1.19	2.5	2.90	6.1	2.78	616.0	13827	0.90	0.38
S2252B	0.45	6.0	2.33	1.29	2.5	2.90	6.1	8.54	630.6	75308	0.93	0.39
S8251	0.45	6.0	7.60	6.54	2.5	2.90	3.5	2.36	177.2	17135	2.40	1.27
S8252	0.45	6.0	8.00	6.70	2.5	2.90	3.7	3.42	213.5	52570	2.40	1.33
S8253	0.45	6.0	8.10	6.67	2.5	2.90	4	4.68	279.3	61553	2.40	1.35
S8101	0.45	6.0	8.15	6.85	1	2.90	6.7	2.66	167.9	35765	6.00	1.36
S6103	0.45	6.0	8.20	6.56	1	2.90	6.7	2.35	164.3	29462	6.00	1.37
S6102	0.45	6.0	8.60	6.82	1	2.90	7.1	2.35	144.4	32760	6.00	1.43
451011	0.45	6.0	9.40	7.75	1	2.90	6.8	1.78	115.5	20360	6.00	1.57
451213	0.45	6.0	10.60	9.13	1	2.90	4.1	-	249.8	24447	6.00	1.77
4512253	0.45	6.0	10.80	9.23	2.5	2.90	6.7	2.61	172.2	32815	2.40	1.80
451212	0.45	6.0	10.85	9.21	1	2.90	4.5	-	261.9	34416	6.00	1.81
451211	0.45	6.0	10.85	9.14	1	2.90	5.1	4.62	311.9	36367	6.00	1.81
4512252	0.45	6.0	10.95	9.34	2.5	2.90	7.1	3.19	194.5	38697	2.40	1.83
4512251	0.45	6.0	11.00	9.34	2.5	2.90	5.9	-	192.1	26680	2.40	1.83
S7152B	0.9	7.5	7.40	6.09	1.5	2.00	5.7	-	-	-	0.00	ERR
S2156	0.9	7.5	2.90	1.34	1.5	2.00	15.8	2.36	388.1	11682	0.00	ERR
S2155	0.9	7.5	3.00	1.53	1.5	2.00	15.8	1.86	382.4	7244	0.00	ERR
S3151	0.9	7.5	2.95	1.37	1.5	2.00	6.5	-	-	-	1.97	0.39
S3152	0.9	7.5	3.00	1.27	1.5	2.00	6.5	1.18	391.1	6642	2.00	0.40
S7158B	0.9	7.5	7.70	6.38	1.5	2.00	18	1.86	109.6	27651	5.00	1.03
S7151B	0.9	7.5	7.90	6.53	1.5	2.00	5.7	1.18	70.0	20111	5.00	1.05
97154	0.9	7.5	8.00	6.27	1.5	2.00	19	0.38 @ 1.7%	30.7	5462	5.00	1.07
910153	0.9	7.5	9.10	7.71	1.5	2.00	16.1	1.46 @ 1.9%	90.5	23075	5.00	1.21
910151	0.9	7.5	9.20	7.65	1.5	2.00	18.5	0.98	63.2	11512	5.00	1.23
910152	0.9	7.5	9.40	7.79	1.5	2.00	16.2	1.17	70.1	21399	5.00	1.25
910154	0.9	7.5	9.40	7.82	1.5	2.00	14.7	1.12	66.4	31651	5.00	1.25

TABLE A-19 Summary of Specimens Prepared for Sensitivity Study and Results of Unconfined Axial Compressive Creep Tests

Specimen No.	Gradation Power	Diameter, in.	Spec. Height, in.	Max Aggr Size, in.	Asp Cont, %	Air Void Cont, %	Creep Compliance @ 0.1 sec, psi^{-1}	Creep Compliance @ 0.6 sec, psi^{-1}	Plastic Strain @ 0.1, sec.	Plastic Strain @ 0.6, sec.	Smallest Dimen/ Max Aggr Size	Height/ Diameter Ratio
S2151	0.45	7.5	2.9	1.5	2.9	6.5	2.24E-07	4.25E-07	1.55E-09	2.69E-09	1.93	0.39
S213	0.45	7.5	3.05	1	2.9	6.5	3.60E-07	7.46E-07	4.98E-10	8.43E-10	3.05	0.41
S212	0.45	7.5	3.05	1	2.9	5.7	5.75E-06	6.29E-06	3.59E-05	6.58E-05	3.05	0.41
S2252	0.45	7.5	3.05	2.5	2.9	6.1	5.65E-07	7.19E-07	7.11E-07	1.00E-05	1.22	0.41
S211	0.45	7.5	3.05	1	2.9	5.7	5.55E-07	1.29E-06	7.75E-11	9.31E-11	3.05	0.41
S214	0.45	7.5	3.1	1	2.9	6.5	4.42E-07	1.07E-06	2.03E-10	2.92E-10	3.10	0.41
S2253	0.45	7.5	3.1	2.5	2.9	6.1	3.71E-07	4.20E-07	8.83E-08	2.89E-06	1.24	0.41
S7254	0.45	7.5	7.2	2.5	2.9	1.9	2.48E-06	3.70E-06	1.63E-10	2.09E-10	2.88	0.96
S7152	0.45	7.5	7.2	1.5	2.9	5.7	2.12E-06	2.32E-06	2.25E-06	4.11E-06	4.80	0.96
S7153	0.45	7.5	7.2	1.5	2.9	5.7	1.01E-06	2.02E-06	6.38E-06	6.28E-05	4.80	0.96
S7253	0.45	7.5	7.25	2.5	2.9	1.6	2.66E-06	4.58E-06	2.15E-10	2.15E-10	2.90	0.97
S7154	0.45	7.5	7.3	1.5	2.9	6.5	2.54E-06	3.08E-06	2.00E-06	2.73E-05	4.87	0.97
S7251	0.45	7.5	7.4	2.5	2.9	1.8	2.41E-06	2.85E-06	8.15E-07	2.68E-05	2.96	0.99
S7252	0.45	7.5	7.4	2.5	2.9	1.9	1.08E-06	2.48E-06	6.01E-11	6.89E-11	2.96	0.99
S7157	0.45	7.5	7.4	1.5	2.9	6.2	2.36E-07	1.12E-06	5.02E-06	1.10E-05	4.93	0.99
S711	0.45	7.5	7.5	1	2.9	4.6	3.41E-06	3.92E-06	2.98E-06	2.60E-05	7.50	1.00
S712	0.45	7.5	7.5	1	2.9	4.7	2.26E-06	2.60E-06	1.07E-06	1.27E-05	7.50	1.00
S7155	0.45	7.5	7.5	1.5	2.9	6.3	2.72E-06	3.26E-06	1.20E-05	3.08E-05	5.00	1.00
S7151	0.45	7.5	7.55	1.5	2.9	6.1	1.79E-06	2.26E-06	1.94E-06	2.65E-05	5.00	1.01
S7156	0.45	7.5	7.55	1.5	2.9	6.8	1.18E-06	1.47E-06	2.21E-06	1.95E-05	5.00	1.01
S713	0.45	7.5	7.55	1	2.9	4.6	1.66E-06	2.02E-06	5.79E-06	1.29E-05	7.50	1.01
S714	0.45	7.5	7.55	1	2.9	4.6	3.31E-06	3.59E-06	7.67E-06	1.66E-05	7.50	1.01
S7159	0.45	7.5	7.55	1.5	2.9	6.6	4.37E-06	4.66E-06	3.39E-07	1.25E-05	5.00	1.01
S7158	0.45	7.5	7.6	1.5	2.9	6.3	4.32E-06	4.74E-06	7.57E-06	1.91E-05	5.00	1.01
4510252	0.45	7.5	9.1	2.5	2.9	4.3	4.75E-06	5.06E-06	7.04E-06	1.80E-05	3.00	1.21

(Continued)

TABLE A-19 Summary of Specimens Prepared for Sensitivity Study and Results of Unconfined Axial Compressive Creep Tests *(Continued)*

Spec-men No.	Grad-ation Power	Dia-meter, in.	Spec. Height, in.	Max Aggr Size, in.	Asp Cont, %	Air Void Cont, %	Creep Compliance @ 0.1 sec, psi^{-1}	Creep Compliance @ 0.6 sec, psi^{-1}	Plastic Strain @ 0.1, sec.	Plastic Strain @ 0.6, sec.	Smallest Dimen/ Max Aggr Size	Height/ Diameter Ratio
4510151	0.45	7.5	9.25	1.5	2.9	5.1	2.48E-06	2.80E-06	1.88E-07	1.32E-05	5.00	1.23
4510254	0.45	7.5	9.3	2.5	2.9	4.8	1.97E-06	2.37E-06	7.41E-07	3.15E-06	3.00	1.24
4510253	0.45	7.5	9.35	2.5	2.9	4.0	3.43E-06	3.70E-06	3.17E-07	1.27E-05	3.00	1.25
451013	0.45	7.5	9.5	1	2.9	5.8	2.66E-06	3.17E-06	1.95E-06	2.42E-05	7.50	1.27
451012	0.45	7.5	9.5	1	2.9	6.0	4.00E-06	4.42E-06	9.04E-07	2.04E-05	7.50	1.27
451014	0.45	7.5	9.55	1	2.9	6.6	3.26E-06	3.71E-06	1.43E-06	2.53E-05	7.50	1.27
S2253B	0.45	6	2.15	2.5	2.9	6.1	1.01E-06	1.17E-06	2.24E-06	1.59E-05	0.86	0.36
S2251B	0.45	6	2.25	2.5	2.9	6.1	4.99E-06	5.36E-06	9.39E-06	2.87E-05	0.90	0.38
S2252B	0.45	6	2.33	2.5	2.9	6.1	7.39E-07	9.10E-07	1.14E-07	7.57E-06	0.93	0.39
S8101	0.45	6	8.15	1	2.9	6.7	3.90E-06	4.62E-06	8.16E-06	2.29E-05	6.00	1.36
S6103	0.45	6	8.2	1	2.9	6.7	7.13E-06	7.94E-06	1.33E-05	5.76E-05	6.00	1.37
451011	0.45	6	9.4	1	2.9	6.8	1.37E-06	1.70E-06	1.79E-06	1.68E-05	6.00	1.57
451213	0.45	6	10.6	1	2.9	4.1	2.72E-06	3.17E-06	2.36E-06	3.22E-05	6.00	1.77
4512253	0.45	6	10.8	2.5	2.9	6.7	1.51E-06	1.72E-06	1.83E-06	6.60E-06	2.40	1.80
451211	0.45	6	10.85	1	2.9	5.1	1.37E-06	1.70E-06	2.12E-06	1.78E-05	6.00	1.81
451212	0.45	6	10.85	1	2.9	4.5	2.81E-07	8.24E-07	1.19E-06	1.74E-05	6.00	1.81
4512252	0.45	6	10.95	2.5	2.9	7.1	3.58E-07	4.55E-07	1.45E-06	8.01E-06	2.40	1.83
4512251	0.45	6	11	2.5	2.9	5.9	2.73E-07	7.72E-07	7.33E-07	1.83E-05	2.40	1.83
S2156	0.9	7.5	2.9	1.5	2	15.8	2.82E-06	3.21E-06	1.71E-06	3.53E-05	1.93	0.39
S2155	0.9	7.5	3	1.5	2	15.8	1.69E-06	2.33E-06	4.80E-06	4.58E-05	2.00	0.40
S3152	0.9	7.5	3	1.5	2	6.5	2.58E-06	3.09E-06	7.48E-06	5.07E-05	2.00	0.40
S7151B	0.9	7.5	7.9	1.5	2	5.7	2.93E-06	4.15E-06	1.32E-05	7.27E-05	5.00	1.05
910153	0.9	7.5	9.1	1.5	2	16.1	2.04E-06	2.51E-06	5.31E-06	3.73E-05	5.00	1.21
910151	0.9	7.5	9.2	1.5	2	20.0	3.23E-06	3.84E-06	1.96E-05	5.29E-05	5.00	1.23
910152	0.9	7.5	9.4	1.5	2	16.2	4.22E-06	4.77E-06	1.93E-06	3.23E-05	5.00	1.25
910154	0.9	7.5	9.4	1.5	2	14.7	2.31E-06	2.73E-06	3.38E-06	1.75E-05	5.00	1.25

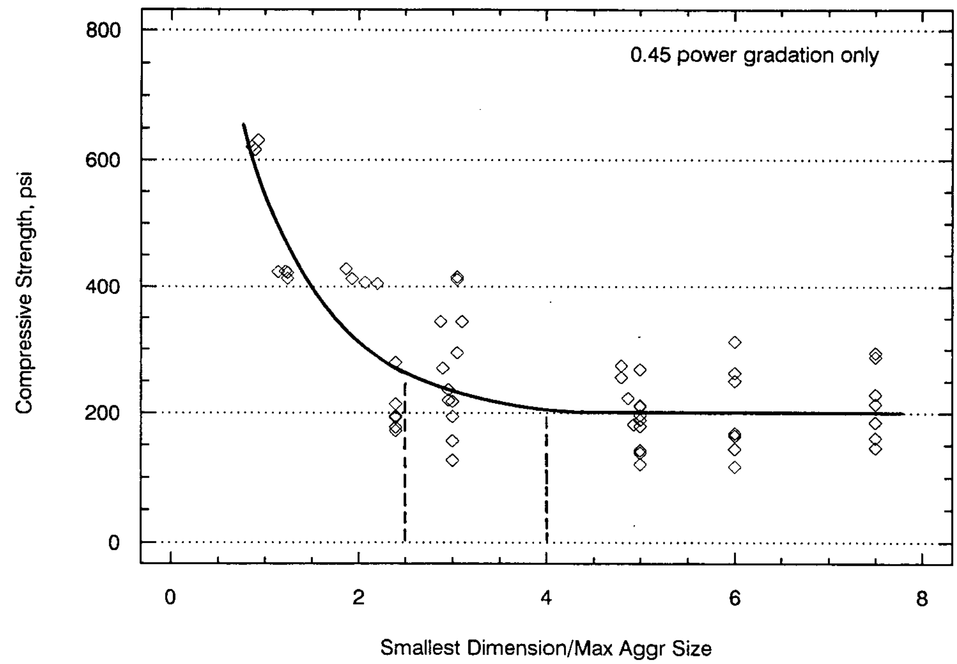

Figure A-30. Unconfined compressive strength versus smallest specimens dimension/maximum aggregate size for LSM with 0.45 power gradation.

tests. Analyses of variance ($\alpha = 0.10$, $n = 50$) indicated that both strength and energy for the mixes containing 2.5-in. (63-mm)-diameter stones are significantly greater than corresponding values for mixes containing 1-in. or 1.5-in. (25-mm or 38-mm)-diameter stones. Although these data show substantial scatter, the slopes of the regression curves indicate higher strength and toughness as the maximum stone size increases. The data scatter can be partially explained by the wide variations in specimen height and diameter. The influence of maximum aggregate size on mix strength is, of course, much more pronounced for the short specimens (i.e., as specimen height approaches the maximum aggregate size). To minimize the effect of specimen height and diameter, these regressions were repeated using only the data from the 7.5-in. (191-mm)-diameter specimens with heights greater than 7 in. (178 mm). In this instance, strength and energy density still exhibited slight increases with an increase in maximum aggregate size, thus illustrating the contribution of the larger aggregate. Nonetheless, analyses of variance revealed that the differences are not statistically significant.

An ANOVA ($\alpha = 0.05$, $n = 56$) of the 0.45 power gradation mixes revealed that compressive strength, energy density, and modulus of the 7.5-in. (191-mm)-diameter

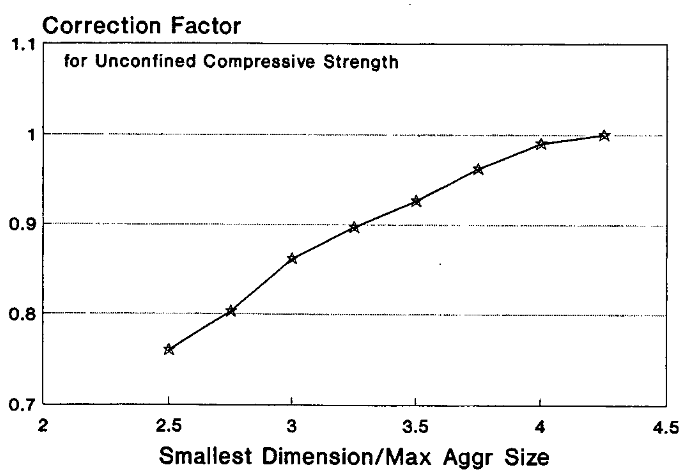

Figure A-31. Correction factor for unconfined compressive strength of LSM as a function of SD/AS.

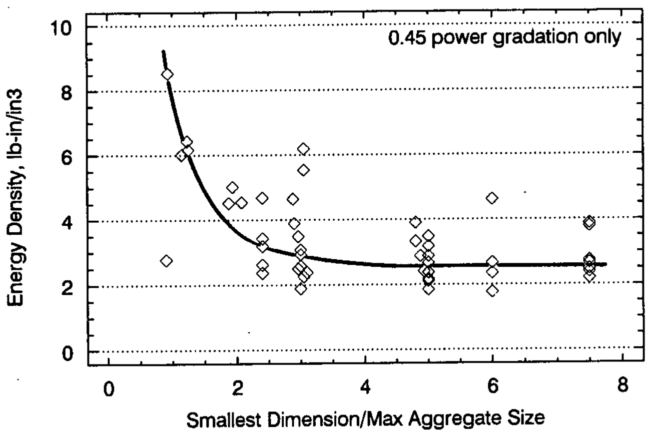

Figure A-32. Energy density versus SD/AS for LSM containing 0.45 power gradation.

specimens are not significantly different from corresponding values of the 6-in. (150-mm)-diameter specimens. Furthermore, when the 1-in. (25-mm) and 2.5-in. (63-mm) maximum aggregate size specimens were considered separately, the differences in compressive strength, energy, and modulus of the 6-in. (150-mm) and 7.5-in. (191-mm)-diameter specimens are not significant even at $\alpha = 0.1$. These findings alone indicate that 6-in. (150-mm)-diameter specimens can be used for axial compressive tests of LSM containing aggregates up to 2.5 in. (63 mm), without adversely affecting compressive strength properties; however, this result should not be applied indiscriminately to other types of tests or measurements.

For the unconfined 1-hr creep and recovery tests, analyses of variance ($\alpha = 0.05$, n = 50) revealed that the coarser 0.9 power gradation mixes exhibited significantly more plastic strain than the 0.45 power gradation mixes; however, the creep compliances were not significantly different. Confined tests, not included in this experiment, might have produced different results.

ANOVAs ($\alpha = 0.05$, $n = 43$) of the 0.45 power gradation mixes indicated there was no significant difference in creep compliance of the mixes containing either 1-, 1.5-, or 2.5-in. (25-, 38-, or 63-mm)-maximum-size aggregates. However, plastic deformation at 0.6 sec of the mixes containing 1-in. (25-mm) aggregates was significantly greater than that for the mixes containing the 2.5-in. (63-mm) aggregates; the mix containing the 1.5-in. (38-mm) aggregates was not different from either 1-in. or the 2.5-in. mix. Analyses of variance to compare the 6-in. (150-mm)-diameter specimens with the 7.5-in. (191-mm)-diameter specimens revealed no significant differences in any of the creep properties measured.

When only the 1.5-in. (38-mm)-maximum-size aggregate specimens were considered, analyses of variance ($\alpha = 0.05$) showed that unconfined compressive strength and energy density at 2 percent strain were significantly higher for the 0.45 power gradation specimens than for the 0.9 power specimens. However, modulus for the two different gradations was not significantly different even at $\alpha = 0.1$. Lower compressive strengths for the more open gradation are to be expected for an unconfined test. Compressive strength and resistance to permanent deformation in pavements composed of open-graded mixes are gained primarily from confinement by the surrounding pavement, which restricts dilation of the larger aggregate. In an unconfined test, dilation is essentially uninhibited and unrealistically low strengths are measured.

ANOVAs ($\alpha = 0.05$, $n = 19$) were also performed on the specimens containing only the 1.5-in. (38-mm)-maximum-size aggregates to determine if there were significant differences between the creep properties of the 0.45 power and the 0.9 power gradation mixes. The only difference

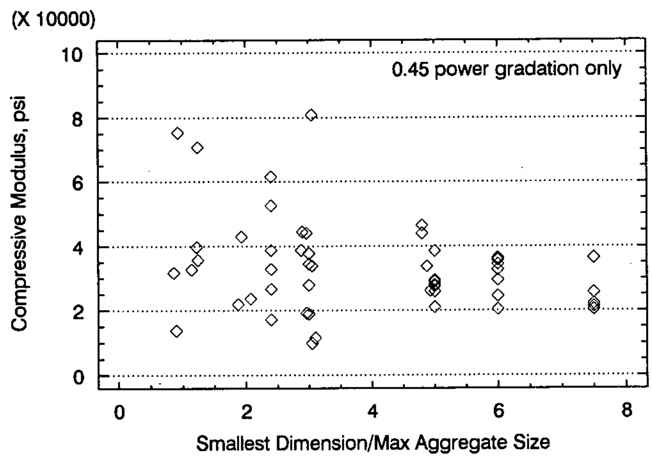

Figure A-33. Modulus versus SD/AS for LSM containing 0.45 power gradation.

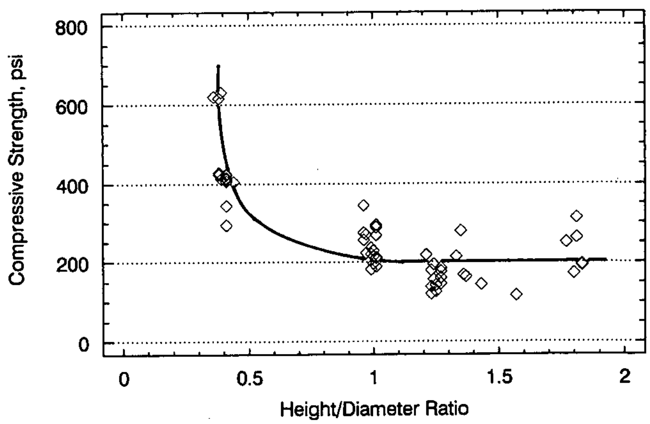

Figure A-34. Unconfined compressive strength versus specimen height/specimen diameter for dense-graded LSM.

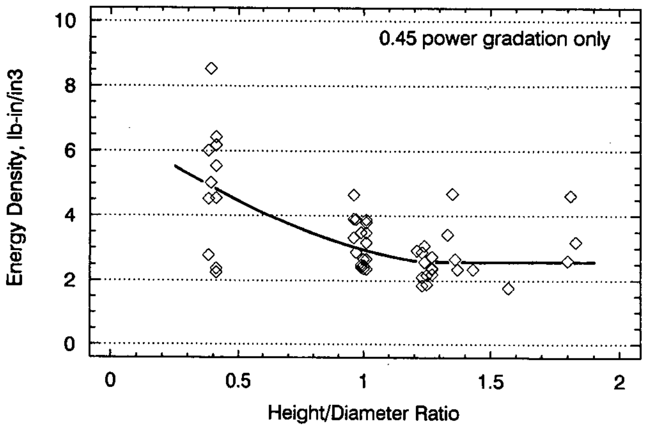

Figure A-35. Energy density versus height/diameter ratio for LSM containing 0.45 power gradation.

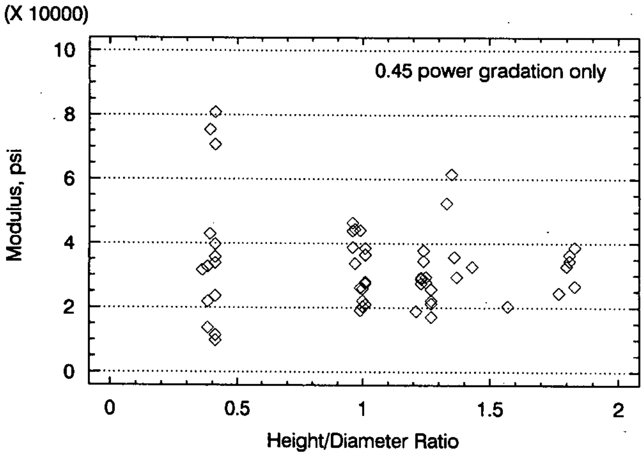

Figure A-36. Modulus versus height/diameter ratio for LSM containing 0.45 power gradation.

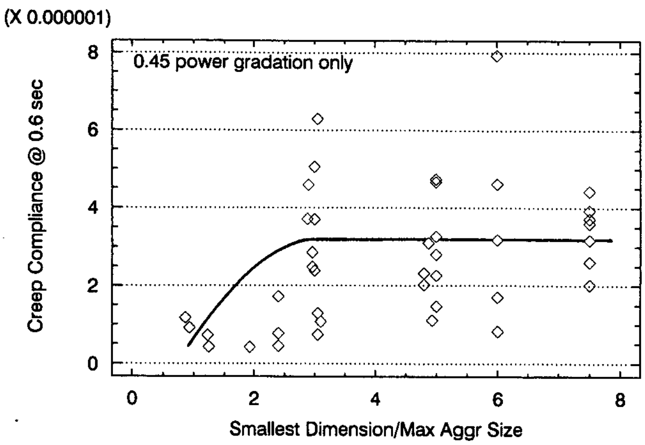

Figure A-37. Creep compliance at 0.6 sec versus SD/AS for unconfined axial creep tests.

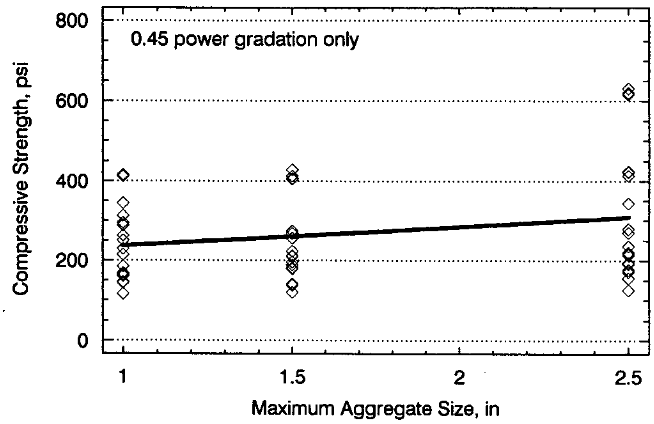

Figure A-38. Plastic strain at 0.6 sec versus SD/AS for unconfined axial creep tests.

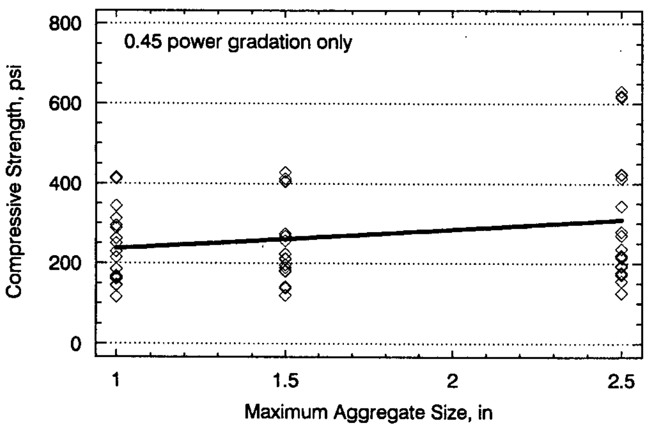

Figure A-39. Unconfined compressive strength versus maximum aggregate size for 150-mm (6-in.)- and 191-mm (7.5-in)-diameter specimens.

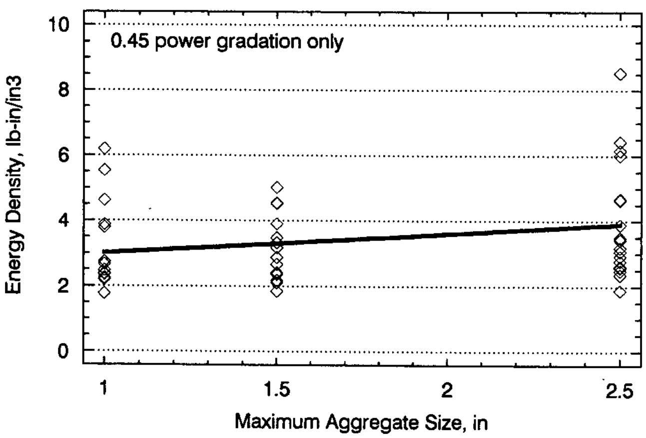

Figure A-40. Energy density required to produce 2 percent strain during unconfined axial compression.

in any of the measured properties was plastic strain at 0.6 sec.

Measurement of Stone-on-Stone Contact of LSM

Methods Evaluated

The field core study indicated that LSM often did not resist rutting any better than conventional asphalt mixes. It was surmised, on the basis of interviews with pavement engineers and review of mix designs, that LSM that performed poorly with regard to rutting did not achieve adequate stone-on-stone contact of the larger stones in the compacted mix. This suggests the need for a method to assess the degree of stone-on-stone contact in the mix design process. Five laboratory procedures for quantifying this characteristic were evaluated. Two were developed at the National Center for Asphalt Technology (NCAT) (*87*), two were from the Texas DOT, and one was from the Heritage Research Group in Indiana. Some of the methods are fairly similar.

All five procedures studied appeared promising. They are simple, practical, and readily implementable. All of these procedures were developed for use with SMA or mixes for fine-grained surface courses. Some require significant testing; others need little more than the data from routine mix design testing. The latter were, of course, more appealing. Only two of the methods for estimating the degree of stone-on-stone contact are described below.

The method developed by Heritage Research Group is empirical. It defines the large-stone contact index (LSCI) as the volume concentration of large aggregates (greater than No. 4 (4.75-mm) sieve) in an SMA compared with the volume concentration of compacted large aggregates. LSCI is essentially the packing efficiency of large aggregates in an SMA. For example, assume an SMA grading contains 75 percent aggregate retained on the No. 4 (4.75-mm) sieve. The bulk specific gravity for the "skeleton" can be measured by making specimens of coarse aggregate (plus No. 4 sieve only) mixed with a small but known amount of binder and filler. Upon preparation of such a mix, the specific gravity was 1.680 g/cc. Next, the entire SMA gradation was mixed and compacted at the optimum asphalt content. Bulk specific gravity of the compacted SMA specimen was 2.344 g/cc. By calculation (2.344 multiplied by 0.75), the concentration of large aggregate in the compacted SMA specimen is 1.643 g/cc. Hence, the LSCI is 1,643/1,680 = 98 percent. That is, the compacted SMA achieved 98 percent of the possible plus No. 4 (4.75-mm) sieve size stone skeleton. Criteria can be established for LSCI using performance-based tests and performance predictions. Although this procedure is very revealing, it requires some nonroutine testing.

The simpler of two procedures from NCAT was adopted for this project and modified to accommodate LSM; this procedure is presented below. A key item in this process is the definition of "coarse aggregate." Coarse aggregate for conventional mixes is normally defined as that retained on the No. 4 (4.75-mm) sieve. It was necessary to define coarse aggregate for LSM. Using a ratio of maximum aggregate size to "coarse" aggregate size similar to that for conventional mixes, coarse aggregate for gradations containing maximum stone sizes of 1.0 to 1.5 in. (25 to 38 mm) was defined as that retained on the 0.5-in. (12.7-mm) sieve, and coarse aggregate for gradations containing a maximum stone size of 1.5 to 2.5 in. (38 to 63 mm) was defined as that retained on the 0.75-in. (19-mm) sieve. Utility and practicality of this procedure was evaluated in the laboratory in a limited experiment using LSM.

Recommended Procedure

Adequate stone-on-stone contact is defined here as the point at which the density of the "coarse" stones in a compacted LSM is equal to or greater than 80 percent of the density of the coarse stones as determined from the dry-rodded density test (AASHTO T 19 or ASTM D 29).

The degree of stone-on-stone contact can be determined using the procedure described in Chapter 3 of the report.

Additional Comments

The "acceptable" value of 80 percent of the density from the dry-rodded test was arbitrarily selected after performing these calculations on several mixes made using aggregates from around the United States (see Table A-20). Even fairly open (i.e., 10 to 15 percent air voids), compacted LSM did not achieve 100 percent of the density obtained from the dry-rodded test. Density depends on the compaction procedure used. Only the modified large Texas DOT gyratory compactor was employed in this study. Although this process appears adequate for estimating stone-on-stone contact of LSM, actual pavement performance data are needed to determine a minimum acceptable value for degree of stone-on-stone contact. Further, the effect of different compaction methods and degrees of compaction need to be investigated.

With the aggregates used in this study, the LSM design procedure developed in this project (see Chapter 2 of the report) typically produced a compacted mix (see Figure A-41), using the modified Texas gyratory compactor, with greater than 90 percent stone-on-stone contact by this method.

DEVELOPMENT OF A TWO-LEVEL LSM MIX DESIGN PROCEDURE

Basis for the Design Procedure

A two-level, computer-based mix design procedure for LSM was developed, which provides good stone-on-stone contact of the aggregate from the coarsest stockpile. The rationale behind the computer program is described in detail in Chapter 2 of the report. The main objective of the design procedure (computer program) is to minimize rutting by ensuring that vehicular loads are borne by a coarse stone skeleton. Specific objectives of the design procedure are as follows:

- Design a mix that will ensure that the load is carried by the stone skeleton. Except for conservation of asphalt binder, there is little benefit in using large aggregates in a paving mix unless the large stones actively participate in the load-bearing function,
- Ensure that the aggregate in the largest stockpile used in the mix actually participates in carrying traffic loads, and
- Provide for adequate binder film thickness for the intended use of the pavement layer designed.

Two types of mixes at the ends of the spectrum covered in the design procedure are addressed: a low permeability, dense-graded mix and a permeable, open-graded mix intended for use primarily as a drainable base. An open-graded mix designed according to this procedure may be applicable also as a highly textured, open-graded friction course, but its suitability for such a use was not studied in this project. In either case, the design procedure generally yields open-graded mixes that appear relatively coarse and slightly open textured.

Level 1 of the procedure basically designs the stockpile blend and provides a starting asphalt content. Level 1 alone could be used for lower volume facilities (e.g., less than 3 million ESALs); whereas Level 2 is proposed for medium- to heavy-trafficked roadways (i.e., more than 3 million ESALs). However, the user has the flexibility to determine which level of design is appropriate for any given project. Very little testing (none of it specialized) is required to obtain the Level 1 optimized mix design. Required tests are as follows:

- For the largest stone stockpile only: gradation (AASHTO T27 or ASTM C 136), unit weight (AASHTO T19 or ASTM C29), and bulk specific gravity (AASHTO T84 and T85 or ASTM C127 and C128),
- For all other stockpiles: gradation and bulk specific gravity.

TABLE A-20 Degree of Stone-on-Stone Contact of Several LSM Calculated Using the Recommended Procedure

Source of Mix	Nominal Max Aggregate Size, in	Specimen Dimension, in x in	Air Void Content, %	Asphalt Content, %	Voids in Mineral Aggregate, %	Degree of S-O-S[1] Contact, %
TTI Mix Design	1	7.5 x 7.0	3	4.6	-	94
	1	7.5 x 7.3	10.9	3.1	-	92
	1	7.5 x 7.7	11.7	3.0	-	95
	1	7.5 x 7.8	19.3	2.7	-	92
Full-Scale Rutting Tests	2	7.5 x 7	5.1	2.5	-	93[2]
	2	7.5 x 7	5.9	4.8	-	30[3]
	2	7.5 x 7	4.2	3.0	-	92[4]
Arizona	1	-	5.5	3.6	13.5	53
Colo-C.P.	1.5	7.6 x 2.2	4.0	4.7	15.7	55
Colo-Flag	1.5	7.6 x 2.5	3.6	4.7	14.8	55
Kentucky	1.5	7.6 x 2.7	4.5	3.3	11.5	70
New Mexico	1	-	4.0	4.5	13.5	50
Oregon	1.5	-	4.5	5.3	13.0	45

[1] S-O-S = Stone-on-Stone
[2] Open-graded mixture designed by Indiana DOT
[3] Dense mixture without good stone-on-stone contact
[4] Dense mixture designed using TTI LSM design procedure

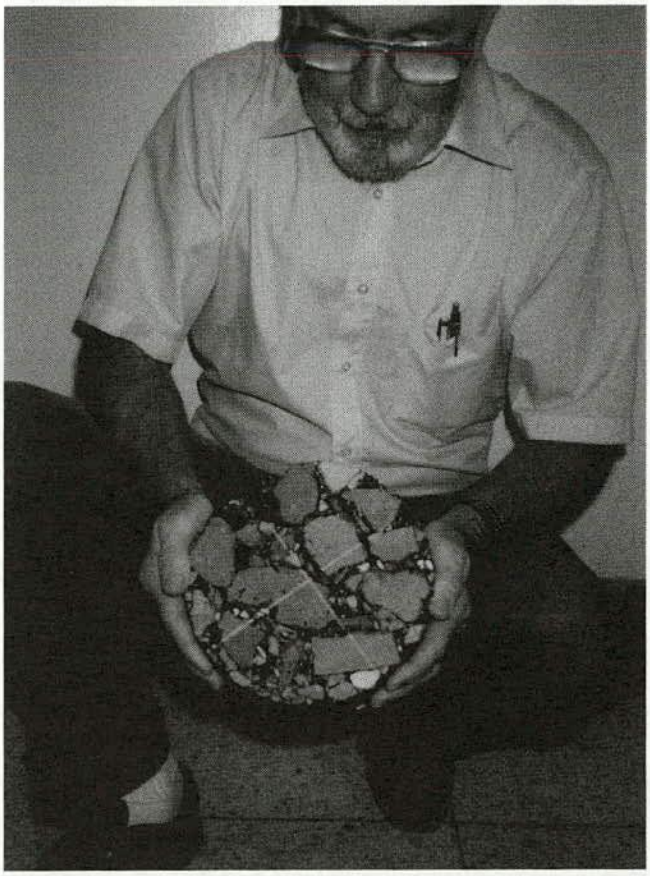

Figure A-41. Cross section of a laboratory-compacted specimen with 92 percent stone-on-stone contact.

The fundamental concept for the Level 1 procedure is as follows:

- Eliminate from the computations any stockpiles that do not fit in the void structure of the largest stone stockpile;
- Roughly estimate the binder film thickness from the air voids content required by the user;
- Compute the effective asphalt content as a function of the volume of solids using a cubic equation, which is solved to yield the maximum volume of aggregate to be used in the LSM; and
- Proportion the remaining stockpiles such that the volume of solids in these stockpiles does not exceed the air volume in the largest stockpile, minus the effective asphalt volume, and such that the weight percent passing the No. 200 (75μm) sieve lies between 0.6 and 1.2 times the effective asphalt content.

Should the user agency wish to go to a Level 2 design to verify the optimum asphalt content, adjust the gradation, quantify resistance to permanent deformation, or combinations thereof, mix analysis procedures are described in Chapter 2 of the report. In general, state-of-the-art NCHRP or Superpave (SHRP) procedures are recommended. For performance analysis of dense-graded mixes, either or both of the following tests are suggested:

- Axial compressive creep and
- RSCH test.

For open-graded LSM, a Level 2 design may be performed using a mastic draindown test as described below. Selecting an air voids content of about 15 percent generally provides permeability suitable for a drainage base. The ability of the new mix design procedure to produce open-graded LSM of a particular air voids content was tested using aggregates obtained from the Kentucky DOT. Two open-graded LSM were designed by this method to contain 15 percent air voids. The actual measured air voids content of these two LSM compacted using the large Texas gyratory compactor were 14 percent and 17 percent. This indicated that the procedure can do a reasonably good job of producing the desired air voids content.

The performance of dense-graded LSM designed by this method was verified by full-scale accelerated performance testing (APT), discussed in the next section.

Full-Scale Rutting Testing of Dense-Graded LSM Designed Using the New Method

Indiana DOT conducted repetitive, loaded-wheel APT on three LSM: (1) a dense mix designed using the method developed in this project, (2) a dense mix designed to exhibit poor stone-on-stone contact, and (3) an open-graded mix. All three LSM had 1.5-in. (38-mm)-nominal-maximum-size aggregate. Test pads that were 3-in. (76-mm) thick, 5-ft (1.5-m) wide, and 20-ft (6.1-m) long were constructed by a local contractor using a batch plant, a conventional paver, and a static-steel-wheel roller. No serious difficulties were experienced during placing or compacting of the LSM. The compacted surfaces were quite rough because of the large particle sizes. This rough texture apparently introduced significant variability in the density of the compacted test pads. The finer LSM designed to have poor stone-on-stone contact gave the highest density.

Five thousand repetitions of a 9,000-lb (4,082-kg) force were applied to each pavement section through dual truck tires inflated to 90 psi (620 kPa). Cross-section profiles were measured periodically throughout testing. Rut depths measured in the LSM designed using the new design method (Texas Coarse) were significantly less than those in the other two mixes. The mix made purposely with poor stone-on-stone contact (Texas Fine) exhibited the highest rut depths, which were similar to those measured in conventional binder courses in the INDOT database. This indicates that large stones floating in a matrix of conventional mix will have little effect on rutting resistance. The INDOT #2 open-graded LSM exhibited more rutting than the Texas Coarse mix but less than the Texas Fine mix. The use of this open-graded mix as a surface course may have allowed unnatural displacement of large aggregate near the surface at the sides of the applied loads, which contributed to greater-than-expected rutting depths. Details of these tests and the results are provided in Appendix D.

Development of Draindown Procedures for Open-Graded LSM

Level 1 mix design for open-graded LSM is accomplished using the mix design software. The software is intended to be a stand-alone program that will design an LSM on the basis of permeability by using aggregate gradation and computed air voids. As with the dense-graded design procedure, this approach designs the open-graded LSM on the basis of the gradation of the contractor's existing aggregate stockpiles, computes asphalt content on the basis of a specific surface area of the selected aggregate grading and a required binder film thickness, and predicts air voids content of the compacted mix. Air voids contents are based on compaction using the modified large Texas DOT gyratory compactor. The desired binder film thickness may vary depending on the grade of asphalt binder and its degree of modification. Film thickness can be specified in the program to meet requirements of a specific user agency or construction project.

For open-graded LSM, Level 2 of the mix design procedure is relatively simple and should always be performed. Level 2 requires a draindown test to verify or make final adjustments to the optimum asphalt content selected with the computer program.

Draindown Methods Studied

An investigation was conducted to develop or identify a suitable draindown procedure for LSM. Draindown procedures from seven different agencies were evaluated: Texas DOT, Indiana DOT, California DOT, FHWA, Institute for Materials Testing in Germany, SABITA, and NCAT. These agencies generally agree that a little draindown is desirable in an open-graded mix, but excessive draindown of asphalt binder is unnecessarily costly and detrimental to performance. Excessive draindown during storage or hauling of the HMA results in fat and lean spots in the pavement, which can later manifest themselves in the form of pavement distress. Excessive draindown in the compacted pavement may result in loss of strength and integrity at some point in its service life.

The SABITA draindown procedure (*88*) is interesting because it takes a different approach than the other procedures. In the SABITA open-graded mix design procedure, the aggregate is immersed in oil and drained to obtain the quantity of oil retained. This value is used to estimate an aggregate surface area constant. An optimum asphalt content is then calculated that provides a certain specified film thickness on the basis of the gradation (theoretical surface area per unit weight) of the aggregate. Then a series of draindown tests with asphalt binder are performed at different temperatures to determine the appropriate plant mixing temperature (for a binder of a given viscosity) that provides adequate aggregate coating without excessive draindown. The SABITA procedure states that a slight puddle at the points of contact between aggregate and glass plate is suitable and desirable.

The draindown test method recommended for use with LSM is a modification of the method developed by NCAT for SMA (*87*) and implemented by Texas DOT. It was selected because, although simple, it provides an objective measure of draindown rather than a subjective measure as do most of the other methods. In addition, the NCAT procedure had much more extensive documentation and laboratory validation than the other methods. Although the California method also provides objective measures of drainage, it requires unique equipment. In this study, the NCAT procedure, modified to accommodate LSM, was evaluated.

The NCAT draindown procedure is similar in many respects to the other draindown procedures. Briefly, the original concept of the test for SMA or conventional open-graded mixes involves mixing asphalt binder and aggregate in accordance with a proposed mix design, placing a sample of the mix in a 4-in. (102-mm)-diameter wire basket, placing the basket in an oven at the expected plant mix temperature for about 2 hr, and measuring the mass of material (asphalt and aggregate fines) that drains through the basket. The design asphalt content is essentially the maximum amount of asphalt the aggregate can retain for a specified period (about 1 hr) at the specified oven temperature. Although no published verification is available, most SHAs agree that 0.2 percent to 0.6 percent draindown is permissible and desirable.

Brown and Mallick (*87*) pointed out that size distribution and absorption capacity of the aggregate filler can significantly affect quantity and rate of draindown (e.g., SMA containing very fine, absorbent baghouse dust exhibited much less draindown than similar mixes containing coarser, less absorbent marble dust). In fact, their study demonstrated that as the amount of material passing the No. 4 (4.75-mm) sieve increased, draindown decreased. They concluded that the higher surface area per unit weight of the finer material reduced the flow of the asphalt binder. They further postulated that, with coarser mixes, the internal voids in the uncompacted mix are larger, resulting in a more freely draining mix. They also showed that incorporation of fibers and polymers into a mix will decrease the amount of draindown as well as the rate of draindown. Their statistical analyses indicated that filler type, fines content, asphalt content, and fiber type had significant effects on draindown ($\alpha = 0.05$) for mixes prepared with two different aggregate types (gravel and limestone).

Laboratory Testing

For LSM it was assumed that a 4-in. (102-mm)-diameter basket would be too small. Therefore, three 6-in. (152 mm)-diameter and three 8-in. (203-mm)-diameter wire baskets were fabricated using 0.25-in. (6.4-mm) wire mesh, and a test plan was devised to examine the significance of selected test parameters. The 0.25-in. (6.4-mm) wire mesh appears ideal. A larger mesh allows larger stones to fall through; whereas, capillarity of a smaller mesh interferes with free drainage of the viscous mastic. In addition, a smaller mesh may not be strong enough to support the weight of a 3,000-gm LSM sample without reinforcement.

A partial factorial experiment design was developed to study draindown of LSM using a modified NCAT procedure. Variables included wire basket (6-in. or 8-in. [150- or 203-mm] diameter), asphalt content (2.4, 3.0, or 3.6 percent by mass), aggregate gradation (power of grading curve = 0.8, 0.9, or 1.0), oven temperature (138, 149, or 160°C [280, 300, or 320°F]), and asphalt modification (AC-20 or MG 10–30). Replicate tests were performed for each condition tested. Crushed limestone from Indiana with a maximum size of 2.5 in. (63 mm) was used in this limited LSM draindown study. The purpose of this experiment was to build on the SMA work performed at NCAT (*87*) to develop a suitable draindown procedure for LSM. Percent draindown was computed by dividing the mass drained down by the total mass of the sample.

Table A-21 shows draindown in percent by weight of mix after 1 hr and 2 hr of drain time for duplicate tests at the conditions stated. The data exhibit substantial variation in the quantity of material drained down. The material draining down includes a considerable amount of fine aggregate and filler; therefore, the material draining is mastic. When draindown is small, the weight of a few small stones can significantly affect the total percent drained down and thus the variability. These large variations in the duplicate tests indicate that duplicate or, preferably, triplicate tests should be performed at each condition during design to obtain an average value.

Figures A-42, A-44, A-46, A-48, and A-50 show results for the 6-in. (150-mm)-diameter baskets while Figures A-43, A-45, A-47, A-49, and A-51 show results for the 8-in. (203-mm)-diameter baskets. When results for the 6-in. and 8-in.-diameter baskets are compared, there is always greater draindown for the 8-in. (203-mm)-diameter basket. The slope of the curves during the first 30 min are also much steeper for the 8-in. (203-mm)-diameter baskets, signifying a faster rate of draindown. Because the same sample size (3,000 gm) was used in each basket, the sample height was greater for the 6-in. (150-mm)-diameter basket. It appears that the additional sample height, or length of drain path, in the 6-in. (150-mm)-diameter wire basket consistently inhibited the quantity and rate of draindown of the asphalt mastic. It appears, therefore, that the more sensitive measure of draindown will result with the thinner layer in the 8-in. (203-mm)-diameter wire basket. It cannot be stated, however, which is more representative of the field because draindown may occur in the silo, haul unit, or the mat; that is, when the draining layer is both thick and thin.

When all the draindown data (see Table A-21) are considered collectively, ANOVA indicates that basket diameter does not significantly affect draindown of the unmodified asphalt at either 1 or 2 hr ($\alpha = 0.10$). However, basket diameter does significantly affect draindown of the modified asphalts ($\alpha = 0.05$), which were tested only at 149°C (300°F) for the 0.9 power gradation. When the data at 149°C (300°F) for the 0.9 power gradation for the unmodified asphalt are considered independently, basket diameter still does not significantly affect draindown at either 1 or 2 hr ($\alpha = 0.05$). When the data at 3 percent asphalt for the 0.9 power gradation are considered independently, basket diameter has no significant effect ($\alpha = 0.05$).

Figures A-42 and A-43 confirm that, as expected, total quantity and rate of draindown increase with asphalt content. These two figures also show that draindown can increase by a factor greater than 2 when the asphalt content is increased by only 0.6 percent. This finding is important because it verifies that this method is quite sensitive to asphalt content and is, therefore, suitable for use in optimizing asphalt content of open-graded LSM. ANOVAs ($\alpha = 0.05$) show that, when all the data are considered collectively or when the data at 149°C (300°F) and 0.9 power gradation are considered independently, asphalt content has a significant effect on draindown of the unmodified asphalt.

The gradation powers mentioned in Figures A-44 and A-45 indicate the exponent of sieve size in the equation for the gradation curve (see Equation 4) used to prepare the aggregate blend. As the gradation power increases from 0.8 to 1.0, the resulting aggregate blend becomes more open and coarser (i.e., a higher percentage of large stones). Figures A-44 and A-45 show that, at the same asphalt content and viscosity, the more open mixes consistently yield the higher draindown and rate of draindown. This finding indicates the procedure is sensitive to aggregate grading. Figures A-46 and A-47 show that draindown is not extremely sensitive to temperature. With all other variables held constant, one would expect that more draindown would occur at the higher temperature; however, with 11°C (20°F) temperature intervals, neither the 6-in. (150-mm) nor the 8-in. (203-mm)-diameter basket gave this anticipated result. This suggests that factors other than temperature had greater influence on draindown. Apparently, to differentiate between draindown characteristics at different temperatures, increments greater than 11°C (20°F) must be used. Because bulk quantities of asphalt mix in a silo or a haul unit lose heat rather slowly, one could presume that use

TABLE A-21 Results from Draindown Tests

Basket Diameter, inch	Asphalt Content, %	Power of Gradation Curve	Oven Temp., °F (°C)	Draindown @ 2 hours, percent by mass of mix		Draindown @ 1 hour, percent by mass of mix	
				AC-20	MG 10-30	AC-20	MG 10-30
6	2.4	0.9	300 (149)	0.07 0.48	0.06 0.12	0.02 0.24	0.03 0.06
6	3	0.8	300 (149)	0.27 0.19	-	0.16 0.11	-
6	3	0.9	280 (138)	1.1 0.26	-	0.72 0.15	-
6	3	0.9	300 (149)	0.35 0.18	0.01 0.08	0.23 0.11	0.01 0.04
6	3	0.9	320 (160)	0.10 0.08	-	0.10 0.03	-
6	3	1	300 (149)	0.73 0.30	-	0.63 0.20	-
6	3.6	0.9	300 (149)	0.78 1.40	0.12 0.04	0.64 0.96	0.06 0.01
8	2.4	0.9	300 (149)	0.29 0.43	0.09 0.20	0.16 0.24	0.06 0.17
8	3	0.8	300 (149)	0.62 0.25	-	0.52 0.16	-
8	3	0.9	280 (138)	0.64 0.45	-	0.51 0.32	-
8	3	0.9	300 (149)	0.79 0.56	0.40 0.17	0.62 0.39	0.38 0.09
8	3	0.9	320 (160)	0.64 0.46	-	0.52 0.24	-
8	3	1	300 (149)	1.22 0.46	-	1.09 0.44	-
8	3.6	0.9	300 (149)	1.30 0.65	0.32 0.23	1.13 0.45	0.26 0.18

* Duplicate values represent results of two runs of each experiment.

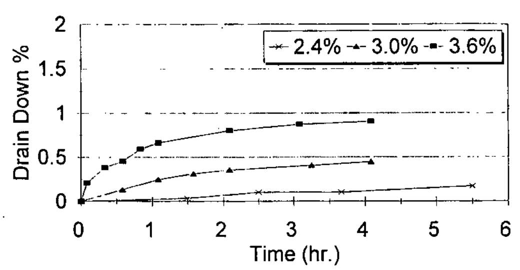

Figure A-42. Asphalt content effect, AC-20 (150-mm [6-in.]-diameter basket, 149°C [300°F], 0.9 power gradation).

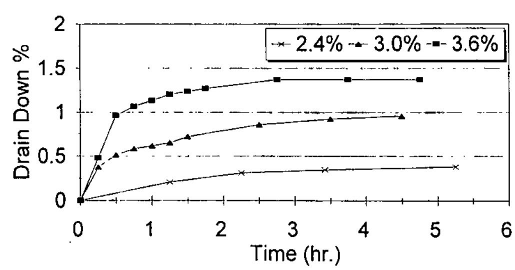

Figure A-43. Asphalt content effect, AC-20 (203-mm [8-in.]-diameter basket, 149°C [300°F], 0.9 power gradation).

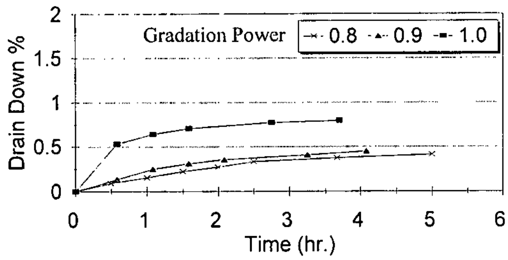

Figure A-44. Aggregate gradation effect, AC-20 (150-mm [6-in.]-diameter basket, 149°C [300°F], A.C. = 3 percent).

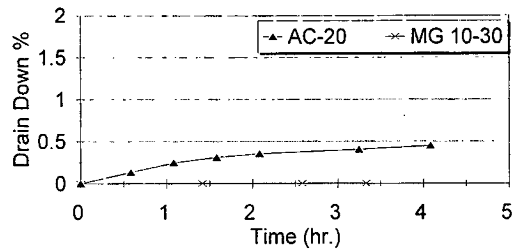

Figure A-48. Effect of additives (150-mm [6-in.]-diameter basket, 0.9 power gradation, A.C. = 3 percent).

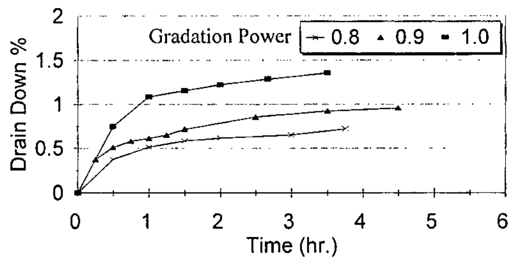

Figure A-45. Aggregate gradation effect, AC-20 (203-mm [8-in.]-diameter basket, 149°C [300°F], A.C. = 3 percent).

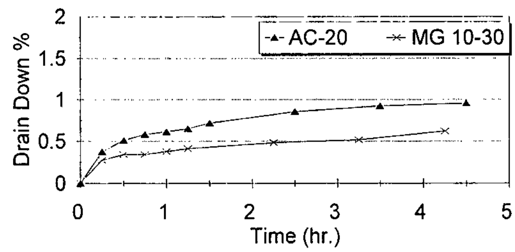

Figure A-49. Effect of additives (203-mm [8-in.]-diameter basket, 0.9 power gradation, A.C. = 3 percent).

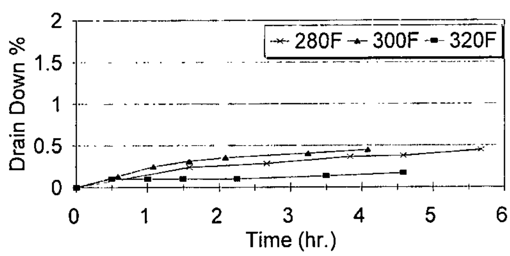

Figure A-46. Temperature effect, AC-20 (150-mm [6-in.]-diameter basket, 0.9 power gradation, A.C. = 3 percent).

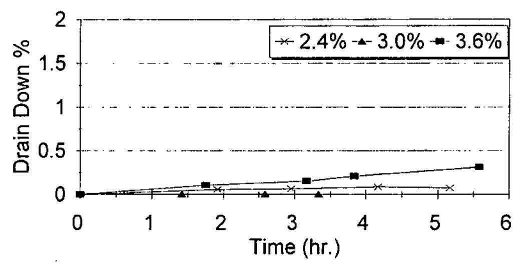

Figure A-50. Asphalt content effect, MG 10-30 (150-mm [6-in.]-diameter basket, 149°C [300°F], 0.9 power gradation).

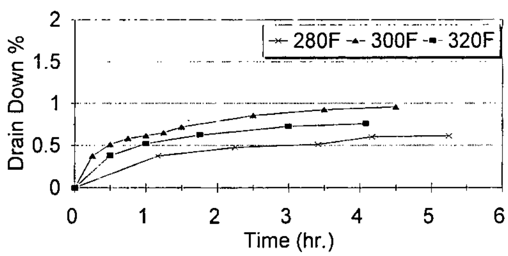

Figure A-47. Temperature effect, AC-20 (203-mm [8-in.]-diameter basket, 0.9 power gradation, A.C. = 3 percent).

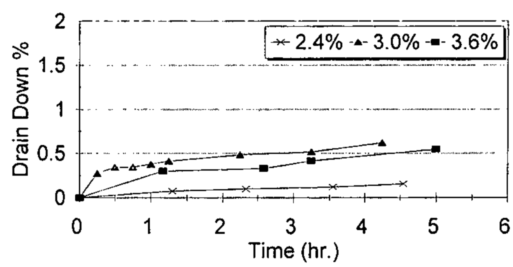

Figure A-51. Asphalt content effect, MG 10-30 (203-mm [8-in.]-diameter basket, 149°C [300°F], 0.9 power gradation).

of the proposed plant mixing temperature for 1 hr is appropriate for draindown studies.

Figures A-48 and A-49 reveal that asphalt modification significantly affects the quantity and rate of draindown. MG 10-30 is a multigrade asphalt containing additives that impart a gel structure that resists creep flow. These findings clearly demonstrate the ability of multigrade asphalt to resist draindown as well as the sensitivity of the test procedure to these binder properties.

Comparison of Figures A-50 and A-51 with Figures A-42 and A-43 shows that draindown of the modified binder is less sensitive to binder content than that of the unmodified AC-20. It also indicates that, when using high-viscosity modified binders or when draindown is low, one could expect more relative variability in draindown test results.

Figures A-42 through A-51 show that, for all cases, draindown is essentially complete after 2 hr at oven temperatures simulating realistic mixing plant temperatures. Because about 90 percent of the draindown at 2 hr occurs within the first hour, it appears that a 1-hr test period is satisfactory for an LSM draindown test. This is in agreement with the findings of Brown and Mallick (*87*) for SMA.

Recommended Draindown Procedure

The results of this portion of the study were used to modify the NCAT procedure for use with LSM. A detailed description of the procedure is provided in Chapter 3 of the report.

The concept of the open-graded LSM mix design method was described previously. The final mix design should meet an end-result specification that reflects the requirements of the owner agency. For example, a specification might require a minimum permeability of 0.35 cm/sec (302 m/day), a strength criterion, and minimal stripping. (At the request of NCHRP, stripping was not addressed in this study.)

CHAPTER 3

INTERPRETATION, APPRAISAL, APPLICATION

Using large stones to produce rutting-resistant, HMA mixes makes sense intuitively; research has confirmed this. Nevertheless, most highway agencies are reluctant to specify them. Because LSM have not been widely used for more than 50 years, the resources available to design, evaluate, and apply LSM are insufficient.

NCHRP Project 4-18 defined LSM as HMA paving mixes containing maximum aggregate sizes between 1 and 2.5 in. (25 to 63 mm). LSM may be termed dense graded, stone filled, or open graded.

A comprehensive research study of LSM was conducted to determine their capabilities and limitations and why they are not being more widely used. The main focus of this study was to develop practical procedures for designing and evaluating LSM in the laboratory and for producing, placing, and compacting them in the field. Implementable products developed as a part of this research project include the following:

- A two-level mix design procedure for dense-graded and open-graded LSM,
- Mix analysis strategies for dense-graded and open-graded LSM,
- A procedure for laboratory fabrication of dense-graded LSM specimens with gyratory compaction,
- A procedure for estimating stone-on-stone contact of compacted LSM specimens,
- A procedure for measuring draindown characteristics of open-graded LSM,
- A procedure for measuring bulk specific gravity of compacted LSM specimens with water-permeable voids, and
- Guidelines for construction of LSM pavements.

Details of the development and rationale behind these procedures were provided in Chapter 2 of this appendix. The utility of these products is discussed below.

MIX DESIGN AND ANALYSIS PROCEDURES

Selection of Mix Components

No work was performed in this project to evaluate the basic qualities of mix components for LSM. Aggregate characteristics (e.g., particle shape, surface texture, hardness, adsorptiveness, porosity, and resistance to polishing) were not investigated. Most pavement-specifying agencies already have specifications to control aggregate quality in conventional asphalt mixes. Existing aggregate specifications should be suitable for use with LSM.

The SHRP asphalt research program has produced comprehensive, state-of-the-art specifications for asphalt cement and aggregates, which are included in the Superpave system. On the basis of this study, these specifications also appear satisfactory for use with LSM.

To maintain an asphalt film of the desired thickness on aggregates in open-graded LSM, a modified asphalt or addition of fibers may be required. The draindown procedure developed for LSM can determine whether a modified asphalt is necessary.

Standard materials specifications for conventional asphalt concrete base courses should be used for selection of mix components for LSM until information can be generated specifically for LSM.

The Design Process

Rather than simply recommending one of the existing LSM design procedures, that is the 6-in. (150-mm) Marshall (*14*), the British (*54*), the South African (*66*), or the National Asphalt Pavement Association (*89*), a completely new approach was pursued. A two-level LSM design procedure was formulated. Level 1 uses a Microsoft Excel-based computer analysis system to develop an optimum design on the basis of measured or reported gradations of available stockpiles. The designer can select the desired air voids content for the compacted mix. Only minimal materials testing is required to start the mix design process. The software prompts the user for the information needed. The Level 1 mix design, possibly with minor field adjustments to the job mix formula, should be suitable for low-volume roads.

Level 2 is an iterative process for improving the Level 1 design to ensure that the mix provides the required resistance to permanent deformation (dense graded) or permeability (open graded). Specific requirements can be selected by the user. A Level 2 mix design process will normally be used for higher volume roadways. The main objective of this procedure is to design an LSM in which the load is carried by a coarse stone skeleton. Gradation of aggregates in the existing stockpiles must be measured. The software uses these gradations to build the coarse aggregate skeleton and successively fill any voids with the smaller aggregates from succeeding stockpiles. Once an aggregate gradation is computed, the software estimates the optimum asphalt content on the basis of desired VMA, air voids in the total mix (VTM), and other factors. The filler-asphalt weight ratio is maintained between 0.6 and 1.2, as recommended by the FHWA, to ensure adequate mastic consistency.

Details of the LSM design procedure and instructions on its use are presented in Chapter 2 of the report.

Preparation of Test Specimens

To go to Level 2 for dense-graded LSM, it is necessary to prepare test specimens. The method used to prepare compacted LSM specimens will influence their density, VMA, and aggregate orientation to the extent that engineering properties of the mix and, thus, the selection of the optimum design will be affected. Therefore, the Level 2 procedure developed for a given SHA should be designed around a chosen laboratory compaction device. Recommendations for specimen compaction are given in Chapter 2 of the report.

For design of open-graded LSM, compaction of laboratory specimens is not normally required. Loose LSM specimens used in a draindown test should be mixed at a temperature that represents anticipated field operations. However, compacted specimens may be needed to confirm that any permeability requirements are achieved.

Mix Analysis

For dense-graded LSM, mix analysis should consist of computation of the degree of stone-on-stone contact and either a simple axial creep test or the Superpave RSCH test. Production of a mix that meets all the criteria is an iterative process. If the analysis procedure indicates that the mix will not perform adequately, either aggregate grading or asphalt content must be adjusted and additional specimens should be prepared and tested. Subsequent modification in the mix components will depend on which test the LSM failed and by how much. Details for conducting these tests and suggested criteria are given in Chapter 2 of the report.

A draindown test for open-graded mixes is described in Chapter 3 of the report. This test helps optimize the mix design. The draindown test has been developed specifically for LSM. For open-graded LSM designed for use as drainable bases, test specimens should be compacted in the laboratory to verify that air voids content is adequate to provide the desired permeability. Bulk specific gravity testing of these high air void specimens will require use of a water-tight wrapping or the glass bead method described in Chapter 3 of the report.

LABORATORY FABRICATION OF DENSE-GRADED LSM

A gyratory or kneading action is recommended for compacting large LSM specimens. This project indicates that energy from the kneading action supplied by these types of compactors is necessary to orient the comparatively large aggregates in an LSM to simulate field compaction.

The smallest specimen dimension (either height or diameter) should be at least 4 times the nominal maximum size aggregate in the mix. Measured properties of specimens with a dimension smaller than this recommended value must have a correction applied. Suggested corrections for axial compressive strength are shown in Figure A-31; corrections for all other measured values must be determined by a controlled experiment. Therefore, the smallest dimension of a specimen containing 1.5-in. (38-mm)-diameter stones should be 6 in. (150 mm). At no time should an LSM specimen with a dimension less than 2.5 times the nominal maximum size aggregate be tested.

A word of caution is needed with regard to recent developments in compaction equipment for relatively large asphalt concrete specimens. The work conducted in this study used a large Texas gyratory compactor normally used for compacting unbound base materials. As explained earlier, a gyratory angle of 1.25 deg (specified for conventional, dense-graded mixes in the Superpave system) did not provide adequate mechanical energy to compact these harsh LSM. Head pressures high enough to severely fracture aggregates at the ends of a specimen did not produce the desired densities. Although the Superpave gyratory compactor will produce a comparatively large specimen (6-in. diameter by 6-in. height [150-mm by 150-mm]), the device, as it now exists, will probably not be suitable for compacting LSM without provision for substantially increasing the angle of gyration.

ESTIMATING STONE-ON-STONE CONTACT

A step-by-step procedure for estimating the degree of stone-on-stone contact in LSM was developed in this study and is described in detail in Chapter 3 of the report. Very little testing other than routine procedures is needed to obtain this value. On the basis of comparisons of very limited field and laboratory data, adequate stone-on-stone contact should be achieved in a compacted LSM when the density of the coarsest stones is equal to or greater than 80 percent of the density of similar coarse stones as determined from the dry-rodded density test (AASHTO T 19 or ASTM D 29). For this test, coarse stones are defined as those retained on the 0.5-in. (12.7-mm) sieve for LSM containing a maximum aggregate size from 1.0 to 1.5 in. (25 to 38 mm) or as those stones retained on the 0.75-in. (19-mm) sieve for LSM containing a maximum aggregate size from 1.5 to 2.5 in. (38 to 63 mm).

DRAINDOWN OF OPEN-GRADED LSM

Most open-graded mix design procedures involve a draindown test of some kind. The draindown test developed for LSM is simple and can be performed quickly without sophisticated equipment. Therefore, it is recommended that a draindown test be conducted on all Level 1 open-graded LSM designs (i.e., all open-graded mixes should involve Level 2 analyses).

The draindown test is described in Chapter 3 of the report.

BULK SPECIFIC GRAVITY OF LSM

LSM, whether dense-graded or open-graded, typically possess larger air voids than conventional asphalt paving mixes with comparable air voids content. These voids may be permeable to water. As a result, standard procedures for measuring bulk specific gravity (AASHTO T 166) of dense-graded mixes may give erroneously high values and, in turn, yield erroneously low values of air voids content.

An improved method for measuring bulk specific gravity of LSM was developed and used in this research project. The basic procedure is similar to AASHTO T 166 but uses glass beads instead of water. The step-by-step technique is provided in Chapter 3 of the report.

GUIDELINES FOR CONSTRUCTION OF LSM

Chapter 4 of the report, Guidelines for Construction of LSM Pavements, identifies and discusses construction problems often associated with LSM and suggests solutions to those problems by providing guidelines for the production, placement, and compaction of LSM.

The manual discusses the following areas of construction (with special attention given to LSM):

- Segregation,
- Aggregate fracture,
- Equipment wear,
- Stockpiling,
- Aggregate delivery,
- Mix production,
- Silo operations,
- Truck loading and unloading,
- Paver operations,
- Handwork and joint construction, and
- Mix compaction.

This manual should be useful to DOT construction inspectors as well as key paving contractor personnel.

CHAPTER **4**

CONCLUSIONS AND SUGGESTED RESEARCH

In this project, the literature and current state DOT specifications and practices were reviewed, and on-site evaluations of LSM pavements during and after construction were conducted. A laboratory phase included testing of pavement cores and laboratory-prepared LSM, development of a procedure for measuring degree of stone-on-stone contact of the larger stones, formulation of an LSM design and analysis method, and full-scale APT to estimate rutting resistance. Guidelines for avoiding inherent construction problems associated with LSM were prepared. For this study, NCHRP defined LSM as those mixes containing aggregates between 1.0 and 2.5 in. (25 and 63 mm) nominal maximum size.

On the basis of the results of this study, the following conclusions and recommendations are offered.

CONCLUSIONS

- When properly designed and constructed, LSM have provided excellent resistance to heavy, concentrated, high shear loads without permanent deformation (*29,75*) and cracking (*3,6,37*). Researchers (*19*) found that pavement cores containing 1-in. (25-mm) maximum size aggregates deformed less when subjected to shear loads and were denser and stronger compared to similar cores containing 0.75-in. (19-mm) maximum size aggregate. Asphalt contents of LSM may be more than 30 percent lower than those of conventional mixes. Production of coarser aggregates requires less crushing energy, which may result in lower costs for aggregates (*31,32*).
- For some SHAs, LSM are considered a standard design. Although 20 SHAs reported having constructed six or more LSM projects in the last 10 years, only six (i.e., Arkansas, California, Indiana, Kentucky, Tennessee, and Texas) have used aggregate with a top size greater than 1.5 in. (38 mm). Dense-graded LSM are by far the most common. Some states use open-graded (or gap-graded) LSM, but none report using stone-filled gradations.
- Although interest in LSM is growing in SHAs, evidence is not sufficient to establish whether LSM consistently yield less rutting than conventional mixes. Some LSM have resisted rutting; others have exhibited premature rutting. Mix design appears to be the key factor in determining relative performance.
- Some SHAs reported that inadequate methods and equipment for designing LSM have inhibited their use for some projects. Bad experiences during construction or concern about problems such as segregation have contributed to a reluctance to specify LSM.
- Problems associated with LSM include greater equipment wear, incomplete coating of coarse aggregates, increased mixing time requirements, noise during drum mixing or drying, inadequate paddle clearance inside the pug mill, placement of the coarse-textured mix, segregation, resistance to compaction, fracture of the larger stones, permeable voids in the compacted mat, and water susceptibility.
- Several SHAs reported having construction difficulties with LSM. The most consistent and significant problem reported was segregation. Compaction of LSM is sometimes difficult because of lack of knowledge and experience in constructing thick lifts (as required by LSM), poor mix designs, faulty material handling procedures, and improper compaction practices and equipment. These problems, however, do not appear to be insurmountable. Guidelines for construction of LSM were developed in this study and are presented in Chapter 4 of the report.
- A survey of state DOTs revealed that many dense-graded LSM designs have not provided stone-to-stone contact of the largest aggregates. In most mix designs used, a few large stones are merely "floating" in a matrix of smaller aggregate and asphalt (i.e., there is no interlock of the larger stones). Such LSM exhibit rutting performance similar to that of conventional mixes.
- Comparative analyses of LSM and conventional mixes contained in the LTPP database indicate that LSM, on the average, exhibit slightly less rutting, even though they were not necessarily designed to provide good stone-on-stone contact.
- A two-level mix design procedure for LSM was developed. Full-scale APT revealed that dense-graded LSM designed by this new method resist rutting better than LSM with the same maximum size aggregate but with poor stone-on-stone contact.

- A method was developed to quantify stone-on-stone contact of the coarse aggregate in LSM. LSM received from SHAs and tested in this study achieved stone-on-stone contact of only 45 percent to 70 percent, according to the method developed. The mix design procedure developed in this study produces mixes with about 90 percent stone-on-stone contact.
- Open-graded LSM typically attain stone-on-stone contact in excess of 90 percent; these mixes are also known to be very resistant to rutting. To obtain the benefit of the large stones in resisting rutting in dense-graded LSM, data analyzed in this study indicate that stone-on-stone contact should exceed 80 percent.
- A procedure for measuring draindown in LSM was developed. For LSM containing aggregates up to 2.5 in. (63 mm), an 8-in. (203-mm)-diameter wire basket with a mesh size of 0.25 in. (6.4 mm) yielded a more sensitive measure of asphalt draindown than a 6-in. (150-mm)-diameter basket. The same mass of mix was used in each basket; it appears that the additional sample height, or length of drain path, in the 6-in. (150-mm)-diameter basket inhibited quantity and rate of draindown of the asphalt mastic.
- Compressive creep testing of LSM revealed that strength and energy required to produce specimen failure increased significantly with an increase in maximum aggregate size.
- Data from compressive creep and recovery tests suggest that LSM will be particularly effective at reducing permanent deformation when used on pavements where load durations are longer than those associated with normal highway speed traffic (e.g., intersections, shoulders, urban streets, truck terminals, and airport taxiways and aprons, where the load carrying capacity of the aggregate skeleton is more fully mobilized). They should certainly be effective for typical highway applications.
- Successful techniques were developed for testing large LSM specimens using the Superpave RSCH test. The Sousa and Solaimanian (*74*) transfer function appears applicable to LSM, although it was developed from an LTPP GPS database that did not include LSM. The transfer function is applied to the output data from the RSCH test (permanent shear strain) to predict rut depth in situ caused by defined traffic loads.
- Limited data from RSCH testing on laboratory-molded LSM suggest that an optimum level of dilation exists at which a given LSM will be the most rut resistant.
- At equivalent air voids content, LSM pavement cores exhibited a mean tensile strength at 5°C (41°F) about 30 psi greater than the mean tensile strength for the limited number of control cores (from conventional asphalt binder courses) that were available. However, because of the data scatter, this difference cannot be considered statistically significant (α = 0.05). This difference may result from the relatively higher surface area of the failure zone, which is the result of the larger sized stones.
- For compaction of a 1.5-in. (38-mm)-maximum-size Kentucky mix, an angle of gyration higher than 1.25 deg (as specified by the Superpave system for conventional dense-graded mixes) was required when using the Texas gyratory compactor and 7.5-in. (191-mm)-diameter molds to provide adequate mechanical energy to achieve "terminal" density (i.e., that density of a pavement expected after 2 to 4 years' worth of traffic). At the 1.25-deg angle, such high pressures were required to achieve terminal density that unacceptable aggregate fracture occurred and adequate compaction was still not achieved.
- To ensure that accurate mix properties are measured in the laboratory, the smallest specimen dimension (height or diameter) should be at least 4 times larger than the nominal maximum aggregate size. When the smallest dimension is less than 2.5 times the largest aggregate size, aggregate strength masks the measured mix strength; therefore, specimens with a dimension smaller than this should never be tested. If a specimen dimension between 2.5 and 4 times the largest aggregate size is used, a correction should be applied to the property measured.
- Testing of LSM or conventional mix specimens having heights less than 4 times the nominal maximum aggregate size may be acceptable for the RSCH test because the height does not decrease during the test. A lower height limit for this test was not established.
- Laboratory tests on LSM indicate that strength and toughness during monotonic axial compression tests and resistance to permanent deformation during creep tests increase as the maximum stone size increases.
- The SHRP 7-day MMAT appears too high for testing of LSM. Some of the taller dense-graded specimens (height greater than about 3 aggregate diameters) were significantly deformed after 12 hr of conditioning at the specified temperature.
- LSM have physically larger air voids than those typically found in conventional asphalt mixes. As a result, when measuring bulk-specific gravity in accordance with AASHTO T 166, water may enter these larger voids and yield inaccurately low calculated air voids content. A method for measuring bulk-specific gravity of large compacted LSM specimens with glass beads in place of water was found to be satisfactory. The procedure is described in Chapter 3 of the report.

RECOMMENDATIONS

- Specify LSM as thick bases in asphalt pavement structures designed for carrying high volumes of heavy traffic loads.
- Be sure the LSM is designed to ensure stone-on-stone contact of the coarsest aggregates. This should, in turn,

ensure good resistance to rutting and static punching type loads such as those on the parking aprons of truck terminals and airfields.

- Measure stone-on-stone contact using the method described in Chapter 3 of the report and ensure that it is greater than 80 percent to produce dense-graded LSM that possess the desired characteristics.
- Use or specify use of Chapter 4 of the report to minimize problems often associated with LSM pavement construction.
- To ensure measurement of actual mix properties, make sure that laboratory-compacted LSM specimens have the smallest dimension (height or diameter) that is at least 4 times larger than the nominal maximum aggregate size. Under no circumstances should the smallest dimension be less than 2.5 times the largest aggregate size.

SUGGESTED RESEARCH

The interest expressed in LSM by the SHAs indicates that more resources should be devoted to this important topic. The following should be considered:

- The objectives of this project did not permit a detailed study of laboratory compaction of LSM; however, this project did reveal that more research is needed to investigate the effect of large stones on compaction of LSM and to develop a laboratory compaction procedure for LSM that simulates field compaction processes (i.e., construction and subsequent traffic). On the basis of these findings, the current Superpave gyratory compaction device with a gyratory angle at or near 1.25 deg may not develop sufficient energy density to densify some LSM to field-compacted levels when the LSM mix contains angular coarse stones with a high degree of stone-on-stone contact. Further study appears warranted if the use of LSM becomes widespread.
- To determine the relative benefits of LSM in resisting rutting and to provide irrefutable evidence thereof to the SHAs, a series of carefully controlled side-by-side test pavements should be constructed across the United States. LSM and control mixes should be prepared to achieve various degrees of stone-on-stone contact to determine how much stone-on-stone contact is required in order to gain the benefit of the large stones. Any construction difficulties experienced need to be addressed using the guidelines provided in Chapter 4 of the report. Performance of these pavements should be monitored for the life cycle of the pavements. For best results, the LSM and corresponding control layers should be near the pavement surface and traffic should be heavy. To obtain rapid results, an accelerated load simulator, such as used in the ALF or Westrack experiments, could be applied to selected sections.
- For the short term, LSM and control mixes with different maximum stone sizes and various degrees of stone-on-stone contact should be prepared and tested with accelerated laboratory tests such as dynamic creep, loaded wheel tracking devices, and the various SST procedures.
- Consider the use of lower quality local materials to fill the voids in a large stone skeleton. For example, use 2-in. + (50-mm+)-maximum-size angular stones to form the aggregate structure. Use locally available or inexpensive materials, such as marginal-quality RAP, aggregate fines, substandard rounded gravel, or combinations thereof to fill the interstices between the larger stones. Research is needed to evaluate this concept.
- More research is needed to understand the effects of dilation and how this behavior is affected by mix characteristics. Although the additional confining pressure because of dilation would tend to stiffen a mix in a pavement, test results indicate that allowing too much dilation will not improve rutting resistance. Further investigations into the dilatant behavior of asphalt mixes are generally warranted to develop guidelines to optimize the amount of dilatancy mobilized under traffic loads and, thus, enhance the rutting resistance of paving mixes.
- Layer coefficients for LSM (or some other method to estimate the relative strength of LSM) need to be determined so that the structural contribution of these mixes in pavements can be assessed.

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APPENDIX B

PERFORMANCE, STRUCTURAL, AND MATERIALS PROPERTIES OF LSM PAVEMENTS IN THE LTPP GPS INVENTORY

TABLE B-1 LTPP Sections with Large Stone Mixes

#	State	SHRP ID	COUNTY	ROUTE		EXP	ENV	(1) RAIN	(2) AVG32	DATE OPEN	PSI CALCULATIONS: PROFILE DATE	AVG SV	DATE RUTS MEASURED	AGE	AVG RD	(3) CALC. PSI	INIT PSI	(4) OBS PSI LOSS
	AL	11021	Elmore	SH	14	1	WNF	53.5	39	Jun−85	04/03/92	1.95	04/05/89	3.8	0.20	4.08	4.5	0.42
2	AL	14129	Coosa	US	280	1	WNF	54.5	59	Jun−89	04/06/92	2.15	04/03/90	0.8	0.17	4.04	4.4	0.35
	AZ	41025	Yavapai	IH	40	1	DNF	14.5	142	Oct−78	04/03/90	4.27	01/09/91	12.3	0.16	3.62	3.8	0.20
	AZ	41062	Mohave	IH	40	2	DNF	14.8	150	Jun−79	04/02/90	7.75	11/03/89	10.4	0.14	3.20	3.8	0.59
3	AZ	41065	Yavapai	IH	40	2	DNF	14.1	128	Jun−79	04/02/90	4.26	11/03/89	10.4	0.21	3.59	3.9	0.26
1	AR	53071	Benton	US	71	2	WNF	46.8	87	Feb−88	08/30/90	1.09	03/15/89	1.1	0.14	4.39	4.7	0.31
1	CT	91803	New London	SH	117	1	WF	49.6	103	Jul−85	07/26/91	5.86	07/31/89	4.1	0.11	3.42	4.2	0.78
	GA	134111	Oconee	US	78	2	WNF	46.3	46	Nov−80	08/07/90	1.18	03/20/89	8.3	0.18	4.34	4.2	−0.1
2	GA	134113	Camden	IH	95	2	WNF	48.3	15	Jun−77	06/18/90	2.16	05/04/89	11.9	0.13	4.05	4.0	−0.1
	IN	182008	Allen	US	27	2	WF	37.5	123	Jan−80	09/11/91	6.58	10/13/90	10.8	0.35	3.18	4.2	1.02
2	IN	182009	Hamilton	SH	37	2	WF	37.5	123	Jan−81	05/13/91	4.85				3.56	4.2	0.64
1	KY	211014	Pike	US	119	1	WF	45.5	99	Jan−84	01/11/91	6.46	10/17/89	5.8	0.18	3.32	3.5	0.18
	ME	231001	Penobscot	IH	95	1	WF	44.2	170	Nov−72	08/17/91	10.96	08/11/89	16.8	0.31	2.84	4.5	1.66
	ME	231026	Franklin	US	2	1	WF	45.5	172	Jul−73	08/16/91	6.66	08/18/89	16.1	0.40	3.12	4.5	1.38
3	ME	231028	Oxford	US	2	1	WF	44.8	171	Nov−72	08/16/91	8.03	08/18/89	16.8	0.31	3.07	4.5	1.43
1	MA	251004	Bristol	IH	195	1	WF	50.0	88	Nov−74	07/26/91	2.60	08/04/89	14.8	0.19	3.92	4.5	0.58
	MS	281001	Lee	US	45	1	WNF	52.1	48	Jan−87	02/12/92	1.34	01/10/90	3.0	0.21	4.26	4.3	0.04
2	MS	283081	Itawamba	US	78	1	WNF	52.1	48	Nov−84	02/12/92	1.08	06/02/89	4.5	0.28	4.31	4.3	−0.0
1	NH	331001	Merrimack	IH	393	1	WF			Jun−81	08/13/91	2.07	08/10/89	8.2	0.19	4.05	4.3	0.25
1	NJ	341030	Passaic	SH	23	1	WF			Jul−69	09/06/91	46.54	07/28/89	20.1	0.56	1.39	4	2.61
1	NY	361011	Onondaga	IH	481	1	WF	38.7	129	Jun−84	12/14/91	2.76	08/08/89	5.2	0.14	3.91	4.7	0.77
1	NC	371006	Wake	IH	40	1	WNF	45.1	54	Jul−82	03/14/91	2.08	10/13/89	7.3	0.05	4.09	4.2	0.11
1	OK	401015	Seminole	SH	3	1	WNF	38.1	65	Apr−78			01/08/90	11.7	0.23	4.96	4.2	−0.8
	TN	471023	Anderson	IH	75	1	WNF	54.0	88	Jun−72	06/13/91	2.54	10/27/89	17.4	0.17	3.94	4.7	0.76
	TN	471029	Marion	SH	28	2	WNF	60.8	73	Oct−82	05/08/90	1.45	01/10/90	7.3	0.10	4.27	4.9	0.63
	TN	472001	Dyer	US	51	2	WNF	53.5	78	Jul−89	05/24/90	1.40	11/13/89	0.4	0.31	4.17	4.7	0.53
	TN	472008	Gibson	US	45	2	WNF	55.7	78	Jun−73	05/24/90	9.62	01/11/89	15.6	0.07	3.06	4.8	1.74
	TN	473075	Dekalb	SH	56	1	WNF	56.8	90	Jun−71	06/17/91	5.95	11/04/89	18.4	0.17	3.38	4.4	1.02
	TN	473108	Anderson	IH	75	6B	WNF	54.1	89	Jul−72	06/11/91	1.17	01/11/89	16.5	0.10	4.37	4.7	0.33
	TN	473109	Maury	SH	50	6B	WNF	53.9	92	Nov−78	05/23/90	9.25	01/11/89	10.2	0.09	3.09	4.8	1.71
	TN	473110	McMinn	SH	68	6B	WNF	54.3	86	Aug−81	05/18/92	1.79	11/04/89	8.2	0.08	4.17	4.8	0.63
9	TN	479025	Cannon	SH	96	2	WNF	52.5	86	Jan−80	05/16/90	6.33	11/04/89	9.8	0.14	3.35	4.5	1.15
	TX	481047	Carson	IH	40	1	DNF	22.3	106	Jul−71	11/13/91	9.40	04/24/89	17.8	0.20	3.03	4.1	1.07
	TX	481048	Ector	US	385	1	DNF	15.4	58	Nov−74	11/24/91	6.27	12/06/89	15.1	0.20	3.33	4.2	0.87
	TX	481060	Refugio	US	77	1	DNF	33.1	9	Mar−86	04/22/91	3.53	06/18/90	4.3	0.18	3.73	4.2	0.47
	TX	481065	Oldham	US	385	1	DNF	18.2	120	May−70	10/25/90	14.10	01/24/90	19.7	0.24	2.70	4.2	1.50
	TX	481068	Lamar	SH	19	1	WNF	50.2	44	Jun−87	10/23/91	2.34	03/23/89	1.8	0.11	4.01	4.2	0.19
	TX	481069	Kaufman	US	175	1	WNF	37.3	39	Jun−77	03/13/90	2.75	01/30/90	12.7	0.34	3.77	4.2	0.43
	TX	481070	Kaufman	US	175	1	WNF	37.5	40	Jul−77	03/13/90	2.36	03/23/89	11.7	0.13	4.00	4.2	0.20
	TX	481076	Terry	US	62	1	DNF	18.7	84	Nov−77	10/23/90	4.41	12/06/89	12.1	0.19	3.58	4.2	0.62
	TX	481096	Medina	US	90	1	DNF	25.8	17	Apr−81	12/06/91	6.85	10/14/90	9.5	0.30	3.20	3.7	0.50
	TX	481174	Nueces	SH	44	1	DNF	31.1	7	May−75	03/23/92	2.50	10/17/90	15.5	0.23	3.92	4.2	0.28
	TX	481181	Live Oak	IH	37	1	DNF	25.8	12	May−80	02/28/92	5.45	04/14/89	9.0	0.27	3.38	3.8	0.42
	TX	481183	Garza	US	84	1	DNF	21.2	77	Feb−75	09/12/90	13.96	12/06/89	14.9	0.23	2.71	4.2	1.49
	TX	482172	Mitchell	IH	20	2	DNF	20.5	57	Aug−82	09/13/90	2.83	12/06/89	7.4	0.14	3.89	4.0	0.11
14	TX	483729	Cameron	US	83	1	DNF	26.5	4	Jun−83	03/19/92	3.20	06/22/90	7.1	0.39	3.63	3.7	0.07
	VT	501002	Addison	US	7	1	WF	41.1	157	Aug−84	05/09/91	18.43	08/09/89	5.0	0.26	2.48	4.2	1.72
2	VT	501004	Grand Isle	US	2	1	WF	30.6	133	Sep−84	05/08/91	18.12	08/09/89	4.9	0.15	2.55	4.2	1.65
	VA	511002	Floyd	SH	8	1	WF	42.4	126	Oct−79	03/27/91	23.47	10/15/89	10.0	0.31	2.25	4.2	1.95
2	VA	511023	Prince George	IH	95	1	WF	46.3	86	Dec−80	06/23/91	4.73	10/12/89	8.9	0.43	3.33	4.5	1.17
1	WV	541640	Kanawha	US	119	2	WF	43.5	95	Jun−83	11/13/91	9.48	09/28/89	6.3	0.18	3.04	4.4	1.36

(1) RAIN = AVERAGE ANNUAL RAINFALL
(2) AVG32 = AVERAGE ANNUAL NUMBER OF DAYS BELOW 32 F
(3) "CALC PSI" = 5.03−1.91*LOG(1+AVG SV)−1.38*(AVG RD ^ 2)
(4) "OBS PSI LOSS" = INIT. PSI. − "CALC. PSI"

(continued on next page)

TABLE B-1 LTPP Sections with Large Stone Mixes (*Continued*)

State	SHRP ID	COUNTY	ROUTE		TRAFFIC KESALS /YEAR	TRUCK FACTOR	CUMUL KESALS	LAYER THICKNESSES: SC	AC	BBB	NBBB	UBB	SUBB	SS	(5) SN	SUBGRADE: TYPE	PI	LL	%-200	(6) EST. Mr
AL	11021	Elmore	SH	14	91	0.83	621	0.0	7.6	0.0	0.0	17.4	0.0	0.0	5.78	SILT	7	28	69.0	53226
AL	14129	Coosa	US	280	163	0.88	163	0.0	4.5	0.0	0.0	18.2	0.0	0.0	4.53	SAND				37484
AZ	41025	Yavapai	IH	40	995	3.24	11456	0.8	6.8	0.0	0.0	0.0	7.2	0.0	3.50	GRAVEL	0	19	18.3	92295
AZ	41062	Mohave	IH	40	1001		10853	0.3	5.4	11.2	0.0	0.0	0.0	4.5	6.86	GRAVEL	6	18	12.4	59317
AZ	41065	Yavapai	IH	40	992	4.70	10755	0.4	4.8	13.7	0.0	0.0	0.0	5.5	7.60	GRAVEL	14	37	19.5	30785
AR	53071	Benton	US	71	464	1.62	1196	0.5	15.9	0.0	0.0	0.0	0.0	0.0	7.00	CLAY	18	35	92.1	58819
CT	91803	New London	SH	117	26	0.65	157	0.0	7.2	0.0	0.0	12.0	0.0	0.0	4.85	SILT	.	.		57546
GA	134111	Oconee	US	78				0.7	8.0	0.0	0.0	8.4	0.0	0.0	4.70	CLAY				50830
GA	134113	Camden	IH	95	1174	1.43	2347	0.1	3.6	11.5	0.0	0.0	0.0	0.0	5.49	SAND				35901
IN	182008	Allen	US	27	1211	3.59	14173	0.0	25.7	12.0	0.0	0.0	0.0	0.0	15.39	CLAY	10	32	67.5	28952
IN	182009	Hamilton	SH	37				0.0	5.7	10.3	0.0	9.5	0.0	0.0	7.34	CLAY				42820
KY	211014	Pike	US	119	378	5.20	2661	0.0	11.2	0.0	7.3	0.0	0.0	0.0	6.61	ROCK	.	.	.	63501
ME	231001	Penobscot	IH	95	158	1.14	2973	0.0	8.9	0.0	0.0	0.0	3.0	0.0	4.13	SAND	0	0	.	38018
ME	231026	Franklin	US	2	79	1.14	1431	0.0	6.4	0.0	0.0	0.0	18.3	0.0	4.10	SAND	0	0	11.9	41182
ME	231028	Oxford	US	2	133	1.14	2500	0.0	6.4	0.0	0.0	19.8	0.0	0.0	5.59	SAND	0	0	1.7	35245
MA	251004	Bristol	IH	195	185	1.00	3094	0.0	9.6	0.0	0.0	0.0	24.6	0.0	5.95	GRAVEL	0	0	20.5	45898
MS	281001	Lee	US	45	84	0.69	430	0.0	9.7	0.0	0.0	0.0	8.2	0.0	4.84	CLAY	14	32	73.3	17856
MS	283081	Itawamba	US	78	117	0.69	351	0.0	9.0	0.0	8.7	0.0	0.0	0.0	5.96	SAND				18520
NH	331001	Merrimack	IH	393	138	0.73	1405	0.0	8.4	0.0	0.0	0.0	32.2	0.0	5.95	SAND	0	0	11.4	44652
NJ	341030	Passaic	SH	23	60	0.42	1338	0.0	6.0	0.0	0.0	6.8	23.4	0.0	5.23	SAND	0	0	9.2	105631
NY	361011	Onondaga	IH	481	166	1.34	1252	0.0	9.9	0.0	0.0	15.6	0.0	0.0	6.54	GRAVEL	6	22	42.6	69548
NC	371006	Wake	IH	40	367	0.66	3370	0.0	9.3	0.0	0.0	9.4	0.0	0.0	5.41	SILT	0	0	50.6	19369
OK	401015	Seminole	SH	3	218	2	2617	0.0	9.5	0.0	0.0	0.0	0.0	0.0	4.18	SAND				35267
TN	471023	Anderson	IH	75	820	0.76	15612	0.0	5.4	6.1	0.0	6.0	0.0	0.0	5.29	CLAY	20	40	59.7	58671
TN	471029	Marion	SH	28	48	1.29	365	0.0	2.8	12.9	0.0	6.1	0.0	0.0	6.47	SAND	12	33	41.1	76633
TN	472001	Dyer	US	51	298	1.97	267	0.9	6.8	0.0	4.5	0.0	0.0	0.0	4.03	CLAY	12	32	87.9	26232
TN	472008	Gibson	US	45	130	1.17	2211	0.0	11.6	0.0	9.3	0.0	0.0	0.0	7.24	CLAY	6	28	92.9	39527
TN	473075	Dekalb	SH	56	30	1.06	604	0.0	5.0	0.0	0.0	9.2	0.0	0.0	3.49	GRAVEL	4	34	39.3	12961
TN	473108	Anderson	IH	75	820	0.76	14757	0.0	12.2	0.0	0.0	6.1	0.0	0.0	6.22	CLAY				45892
TN	473109	Maury	SH	50	53	1.22	632	0.0	5.2	4.3	0.0	4.5	0.0	0.0	4.38	CLAY				50670
TN	473110	McMinn	SH	68	50	1.19	452	0.9	8.3	0.0	0.0	4.8	0.0	0.0	4.32	CLAY				31117
TN	479025	Cannon	SH	96	24	1.26	245	0.8	3.7	2.3	0.0	12.0	0.0	0.0	4.09	ROCK	6	20	.	162048
TX	481047	Carson	IH	40	290	1.08	5903	0.0	10.0	0.0	0.0	15.3	0.0	14.4	8.70	CLAY	22	40	79.6	29732
TX	481048	Ector	US	385	49	0.60	839	0.0	11.0	0.0	0.0	0.0	0.0	0.0	4.84	SAND	0	0	19.9	26963
TX	481060	Refugio	US	77	167	0.96	858	0.0	7.5	0.0	0.0	12.3	0.0	6.0	5.92	SAND	4	20	34.0	20622
TX	481065	Oldham	US	385	98	1.24	2014	0.3	8.3	0.0	0.0	4.8	0.0	0.0	4.32	CLAY	22	40	93.2	24972
TX	481068	Lamar	SH	19	114	0.90	500	0.0	10.9	0.0	0.0	6.0	0.0	8.0	6.84	CLAY	20	38	74.0	29460
TX	481069	Kaufman	US	175	197	0.93	2519	0.0	9.5	0.0	0.0	0.0	0.0	6.5	5.16	CLAY	41	72	93.1	29945
TX	481070	Kaufman	US	175	197	0.93	2503	0.0	10.5	0.0	0.0	0.0	0.0	10.0	6.12	CLAY	41	66	89.9	33698
TX	481076	Terry	US	62	119	1.07	1543	0.0	5.4	0.0	0.0	8.4	0.0	0.0	3.55	SAND	0	0	17.7	38053
TX	481096	Medina	US	90	76	0.75	816	0.0	7.1	0.0	0.0	8.1	0.0	0.0	4.26	CLAY	22	50	78.5	23264
TX	481174	Nueces	SH	44	86	0.71	1462	0.0	4.7	0.0	0.0	13.2	0.0	0.0	3.92	CLAY	34	55	64.0	14323
TX	481181	Live Oak	IH	37	207	0.92	2452	0.6	6.3	0.0	0.0	9.6	0.0	5.9	5.00	CLAY	18	44	65.0	19679
TX	481183	Garza	US	84	147	1.14	2293	0.4	5.3	0.0	0.0	0.0	8.4	0.0	2.92	CLAY	12	27	63.3	29417
TX	482172	Mitchell	IH	20	404	1.06	3284	0.9	10.0	6.8	0.0	0.0	8.8	0.0	7.33	SAND	10	24	47.5	32026
TX	483729	Cameron	US	83	216	0.76	1899	0.0	10.0	0.0	0.0	10.5	0.0	5.4	6.68	CLAY	26	46	95.4	20074
VT	501002	Addison	US	7	59	0.56	401	0.0	8.3	0.0	0.0	25.8	0.0	0.0	7.26	GRAVEL	0	0	6.9	24379
VT	501004	Grand Isle	US	2	37	0.55	246	0.0	8.0	0.0	0.0	0.0	47.1	0.0	6.82	SILT	17	28	65.3	44037
VA	511002	Floyd	SH	8	115	0.77	1326	0.0	5.7	0.0	0.0	0.0	6.0	0.0	2.93	GRAVEL	0	0	46.8	40537
VA	511023	Prince George	IH	95	626	0.77	6618	0.0	10.1	0.0	0.0	6.0	0.0	8.4	6.54	SILT	5	16	97.2	37382
WV	541640	Kanawha	US	119	68	0.69	574	0.0	15.3	4.1	0.0	0.0	5.1	0.0	8.48	GRAVEL	8	24	34.9	37399

CODES FOR VARIOUS MATERIAL TYPES:
SEAL COATS (SC): 2,11,71–73
ASPHALT (AC): 1,319,700
BIT BOUND BASE (BBB): 3,9,10,320–330
NON BIT BOUND BASE (NBBB): 331,333,334,339,340,730
UNBOUND BASE (UBB): 302–305,308,337
SUBBASE (SUBB): 201–292,306,307
STAB SUBGRADE (SS): 181–183,333,338

(5) SN=0.44*AC+0.34*BBB+0.23*NBBB+0.14*UBB+0.07*SUBB+0.15*SS
(6) "BACK CALCULATED" Mr=((FWD Load)*0.2792)/((Defl. at 60")*60)

TABLE B-1 LTPP Sections with Large Stone Mixes (*Continued*)

										Average Gradations									
State	SHRP ID	COUNTY	ROUTE		LAY	BSG	MSG	%AIR	%AC	1 1/2	1	3/4	1/2	3/8	#4	#10	#40	#80	#200
AL	11021	Elmore	SH	14	3	2.28	2.45	6.94	4.75	100	99	95.5	87	74.5	54	41	16	5	2
AL	14129	Coosa	US	280	4	2.339	2.549	8.24	4.15	100	95	85	74	69	52.5	30.5	13	9.5	7.5
AZ	41025	Yavapai	IH	40	3	2.481	2.515	1.33	4.85	100	86.5	72.5	60.5	53.5	46	31	12.5	8	6
AZ	41062	Mohave	IH	40	4	2.221	2.393	7.21	5.3	100	93.5	80.5	65	57	48.5	29	14.5	9.5	7.5
AZ	41065	Yavapai	IH	40	4	2.237	2.357	5.11	5.55	100	91.5	77	61.5	54.5	47.5	29.5	14.5	10	7.5
AR	53071	Benton	US	71	3	2.362	2.469	4.31	4.15	100	97.5	84	64	57.5	45	35.5	21	11	7.5
CT	91803	New London	SH	117	3	2.495	2.527	1.26	3.55	100	92.5	78.5	63.5	54.5	43.5	32	19.5	11.5	5.5
GA	134111	Oconee	US	78	3	2.376	2.519	5.70	4.85	100	94.5	83.5	66	57	45	34.5	21	12.5	7.5
GA	134113	Camden	IH	95	2	2.297	2.451	6.29	4.3	100	93	73.5	52.5	46.5	39.5	35	22	11	4
IN	182008	Allen	US	27	4	2.365	2.531	6.55	4.75	100	97.5	86	66	54	41	30.5	12.5	4.5	2.5
IN	182009	Hamilton	SH	37	5	2.398	2.542	5.62	3.4	100	92.5	77	60.5	42	29.5	22	7.5	4.5	3.5
KY	211014	Pike	US	119	3	2.282	2.376	3.95	5.95	100	89	76.5	62.5	57.5	48.5	42.5	32.5	18	11
ME	231001	Penobscot	IH	95	4	2.401	2.484	3.34	5.5	100	90	79	58	52	39	33	17	9	4
ME	231026	Franklin	US	2	3	2.483	2.559	2.99	4.4	100	92.5	78	60.5	50.5	37.5	28.5	13.5	9.5	5.5
ME	231028	Oxford	US	2	3	2.445	2.514	2.74	4.2	100	92.5	74	57.5	49.5	38	30.5	18.5	10.5	4.5
MA	251004	Bristol	IH	195	3	2.519	2.645	4.78	4.55	100	84.5	70.5	63	50	33.5	26.5	14	9	5
MS	281001	Lee	US	45	4	2.198	2.311	4.91	5.05	100	91.5	85	73	64	48.5	41	34	14.5	8
MS	283081	Itawamba	US	78	3	2.126	2.345	9.36	4.4	100	92	81	70	62	50	43	37	16.5	6
NH	331001	Merrimack	IH	393	4	2.376	2.568	7.50	4.2	100	88	72.5	57.5	48	33	23.5	11	6	3
NJ	341030	Passaic	SH	23	4	2.487	2.579	3.59	4.95	100	96.5	81.5	61	53.5	41.5	35	19	9	4.5
NY	361011	Onondaga	IH	481	3	2.438	2.525	3.44	4.55	100	83	75.5	62	55	45	37	18.5	13	8.5
NC	371006	Wake	IH	40	3	2.339	2.462	4.98	4.05	100	93	82	61	50	39.5	32.5	16.5	8.5	4
OK	401015	Seminole	SH	3	3	2.428	2.512	3.34	3.35	100	90	80	65	56.5	40.5	28.5	19.5	11	6
TN	471023	Anderson	IH	75	4	2.448	2.527	3.12	4.15	100	86.5	74	63	60	51	34.5	14.5	10	7.5
TN	471029	Marion	SH	28	4	2.414	2.495	3.27	4	100	86	72	69.5	61.5	36	22.5	12	9	6.5
TN	472001	Dyer	US	51	4	2.434	2.459	1.03	4.2	100	92	80.5	66.5	57.5	40.5	29.5	13	5.5	4.5
TN	472008	Gibson	US	45	3	2.399	2.504	4.19	3.35	95	73.5	63.5	54.5	47	35	26	16.5	11.5	8.5
TN	473075	Dekalb	SH	56	3	2.282	2.527	9.72	2.55	92	58.5	52.5	47.5	38.5	17	8	4.5	3.5	2.5
TN	473108	Anderson	IH	75	3	2.396	2.584	7.29	3.2	100	86.5	69	51	45	30.5	18.5	7.5	5	4
TN	473109	Maury	SH	50	4	2.393	2.525	5.24	4.05	100	91.5	79.5	65	56.5	40.5	24.5	10.5	7.5	5.5
TN	473110	McMinn	SH	68	4	2.403	2.615	8.11	3.9	100	87.5	75	60	54.5	43.5	25	10	7	4.5
TN	479025	Cannon	SH	96	4	2.363	2.534	6.75	4.3	100	95	85	65	56	46	31	14	10	8
TX	481047	Carson	IH	40	4	2.383	2.466	3.39	3.85	98	86	73	57.5	50.5	39	32	19.5	8.5	5.5
TX	481048	Ector	US	385	3	2.259	2.349	3.81	5.55	100	96	88.5	67	59	43	32.5	23	17	12
TX	481060	Refugio	US	77	4	2.362	2.454	3.74	4.05	98.5	86.5	78.5	69.5	64	53	43.5	23.5	12	7
TX	481065	Oldham	US	385	4	2.327	2.453	5.13	4.4	100	94	82	70	63	50.5	41.5	25.5	11	6
TX	481068	Lamar	SH	19	4	2.291	2.412	5.00	4.05	100	90.5	80	67	57	45	34.5	23.5	17.5	9
TX	481069	Kaufman	US	175	4	2.353	2.408	2.31	4.25	100	84	76	69	65.5	56.5	37.5	22.5	11.5	6.5
TX	481070	Kaufman	US	175	4	2.306	2.422	4.79	4.6	99	83.5	77.5	71.5	67.5	50.5	40.5	30	11	4
TX	481076	Terry	US	62	3	2.132	2.294	7.05	6.65	100	93.5	81.5	66.5	58	37.5	28	22.5	18	9.5
TX	481096	Medina	US	90	5	2.332	2.452	4.90	4.7	99	92	86.5	81	73.5	48.5	36	26	15.5	10
TX	481174	Nueces	SH	44	5	1.957	2.213	*****	6.25	97	89.5	82.5	73.5	68.5	54.5	41.5	30.5	19.5	7.5
TX	481181	Live Oak	IH	37	5	1.892	2.142	*****	8.05	97	79.5	68.5	59	54	46	41.5	38	28.5	8.5
TX	481183	Garza	US	84	3	2.186	2.329	6.10	5.45	100	93	81	67.5	58.5	41.5	30	21	15.5	7
TX	482172	Mitchell	IH	20	4	2.3	2.427	5.21	4.25	100	94	87.5	79	71.5	53.5	33.5	21.5	16.5	14
TX	483729	Cameron	US	83	4	2.258	2.41	6.31	4.7	95.5	86	78	65.5	56.5	42	32.5	25.5	15	6.5
VT	501002	Addison	US	7	3	2.439	2.507	2.73	5.85	98	86.5	74.5	63	52.5	36.5	26	11	5.5	3
VT	501004	Grand Isle	US	2	4	2.418	2.474	2.28	4.65	100	87	65	53.5	46	30.5	24	14	5.5	2.5
VA	511002	Floyd	SH	8	3	2.463	2.597	5.18	4.35	100	93.5	76	60.5	55.5	46	28.5	11.5	8.5	6
VA	511023	Prince George	IH	95	4	2.41	2.459	1.99	4.3	100	97.5	81	62	49.5	37	29	15	11.5	4
WV	541640	Kanawha	US	119	4	2.317	2.462	5.87	3.85	100	94.5	79	64.5	60	35	21.5	10.5	4	3

(continued on next page)

TABLE B-1 LTPP Sections with Large Stone Mixes (*Continued*)

State	SHRP ID	COUNTY	ROUTE		LAY	BSG	MSG	%AIR	%AC	Average Gradations 1 1/2	1	3/4	1/2	3/8	#4	#10	#40	#80	#200
AL	11021	Elmore	SH	14	5	2.413	2.47	2.33	4.45	100	97	93	71	61.5	48	38	16.5	7	5
AL	14129	Coosa	US	280															
AZ	41025	Yavapai	IH	40															
AZ	41062	Mohave	IH	40															
AZ	41065	Yavapai	IH	40															
AR	53071	Benton	US	71															
CT	91803	New London	SH	117															
GA	134111	Oconee	US	78															
GA	134113	Camden	IH	95															
IN	182008	Allen	US	27															
IN	182009	Hamilton	SH	37															
KY	211014	Pike	US	119	4	2.314	2.395	3.38	5.9	100	91	81.5	68.5	62	48	38	30	19	10.5
ME	231001	Penobscot	IH	95															
ME	231026	Franklin	US	2															
ME	231028	Oxford	US	2															
MA	251004	Bristol	IH	195															
MS	281001	Lee	US	45															
MS	283081	Itawamba	US	78															
NH	331001	Merrimack	IH	393															
NJ	341030	Passaic	SH	23															
NY	361011	Onondaga	IH	481															
NC	371006	Wake	IH	40															
OK	401015	Seminole	SH	3															
TN	471023	Anderson	IH	75															
TN	471029	Marion	SH	28															
TN	472001	Dyer	US	51															
TN	472008	Gibson	US	45															
TN	473075	Dekalb	SH	56															
TN	473108	Anderson	IH	75	4	2.505	2.532	1.07	4.1	98.5	76	67	60.5	59	52	34	14	10	7.5
TN	473109	Maury	SH	50	3	2.295	2.578	*****	2.9	100	93	83	63	48	12	5	3	3	2.5
TN	473110	McMinn	SH	68															
TN	479025	Cannon	SH	96															
TX	481047	Carson	IH	40															
TX	481048	Ector	US	385															
TX	481060	Refugio	US	77															
TX	481065	Oldham	US	385															
TX	481068	Lamar	SH	19															
TX	481069	Kaufman	US	175															
TX	481070	Kaufman	US	175															
TX	481076	Terry	US	62															
TX	481090	Medina	US	90															
TX	481174	Nueces	SH	44															
TX	481181	Live Oak	IH	37															
TX	481183	Garza	US	84															
TX	482172	Mitchell	IH	20															
TX	483729	Cameron	US	83															
VT	501002	Addison	US	7															
VT	501004	Grand Isle	US	2															
VA	511002	Floyd	SH	8															
VA	511023	Prince George	IH	95															
WV	541640	Kanawha	US	119															

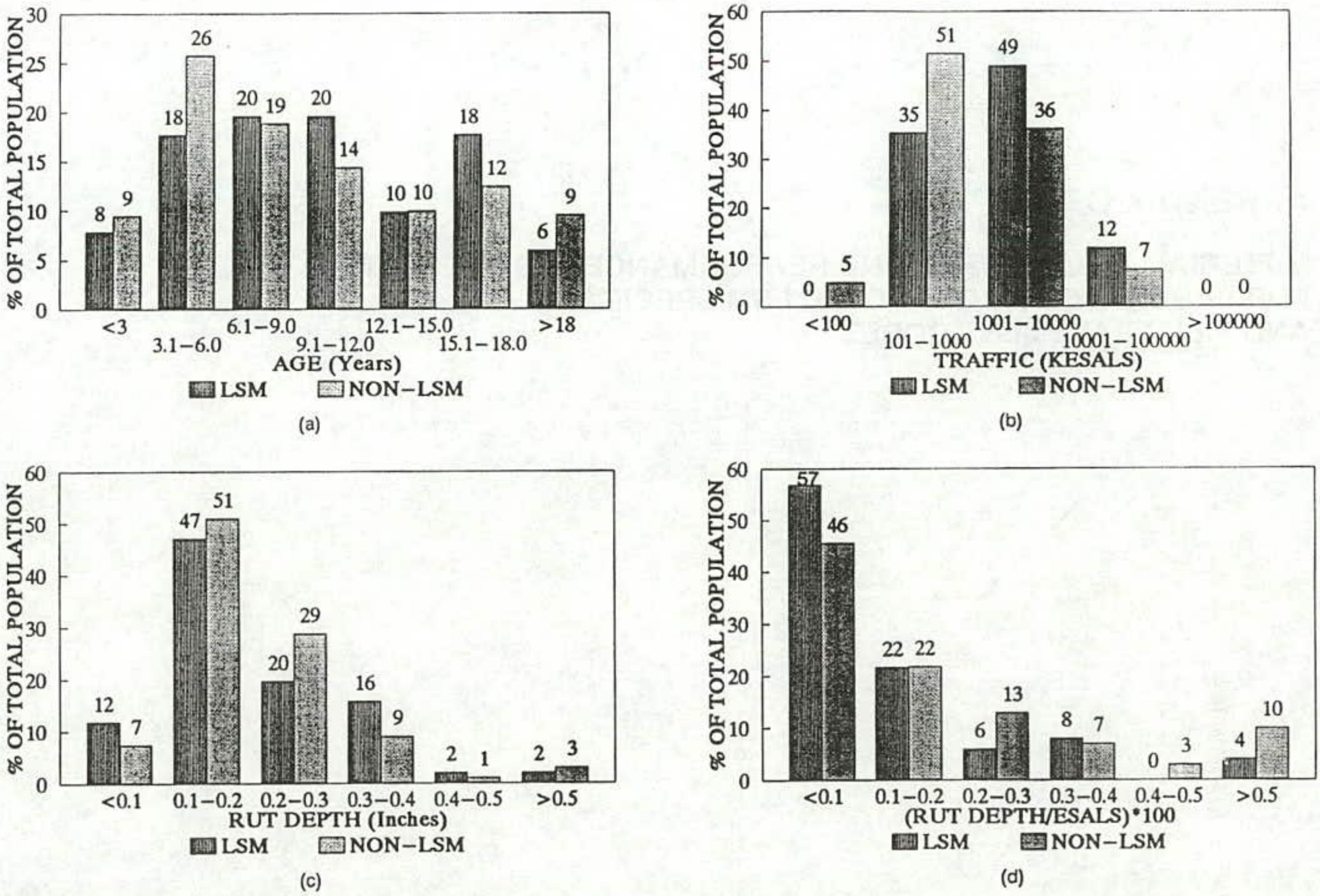

Figure B-1. Analysis of LSM vs. Non-LSM Test Sections in the LTPP Database.

APPENDIX C

MATERIAL, VOLUMETRIC, AND PERFORMANCE TEST RESULTS FOR LABORATORY-COMPACTED LSM SPECIMENS AND FIELD PAVEMENT CORES

TABLE C-1 Test Results on Pavement Cores and Laboratory Compacted Specimens

File Name	Specimen ID	Temp, F	Height, in	Dia, in	Gage Ln, in	Gradn	MxAg, in	AC%	AV%	Enrgy@2%, lb-in/ft3	Strength, psi	Modulus, psi
	Pavement Cores - Unconfined Compression at 7-Day Maximum Annual Pavement Temperatures											
'CC347_8'	'Co.C.P.Control'	124	5.05	7.6	3.00	NoData	0.75	5.78	4.1	NoData	NoData	NoData
'CC347_1'	'Co.C.P.Control'	124	6.30	7.5	3.00	NoData	0.75	5.78	4.1	0.279	114.4	19458
'CC348_6'	'Co.C.P.LSM'	124	3.00	7.6	1.83	NoData	1.5	5.16	5.1	0.195	67.5	17739
'CC348_11'	'Co.C.P.LSM'	124	2.78	7.7	1.00	NoData	1.5	5.16	5.1	NoData	NoData	NoData
'CF395_4'	'Co.Flag.Base'	129	4.47	7.7	1.18	NoData	1.5	4.65	3.7	NoData	NoData	NoData
'CF395_6'	'Co.Flag.Base'	129	4.15	7.6	1.06	NoData	1.5	4.65	3.7	NoData	NoData	NoData
'CF_395_2'	'Co.Flag.Base'	129	4.10	7.7	1.66	NoData	1.5	4.65	3.7	2.25	317.1	11208
'CF395_9'	'Co.Flag.Base'	129	4.20	7.7	1.42	NoData	1.5	4.65	3.7	2.50	310.0	5680
'CF374_5'	'Co.Flag.Surface'	129	3.50	7.7	2.06	NoData	1.5	5.25	5.5	3.49	412.4	19999
'CF374_1'	'Co.Flag.Surface'	129	3.52	7.7	2.09	NoData	1.5	5.25	5.5	4.70	424.2	19265
'KC10'	'Kentucky.Long'	133	11.70	7.6	9.50	NoData	1.5	3.50	3.8	NoData	NoData	NoData
'KLL_11'	'Kentucky.Long'	133	8.30	7.7	6.30	NoData	1.5	3.50	3.8	1.14	83.5	10626
'KLS_11'	'Kentucky.Short'	133	3.08	7.7	1.98	NoData	1.5	3.50	3.8	1.93	418.9	8034
'K11_B'	'Kentuky.Long'	133	11.50	7.6	9.50	NoData	1.5	3.50	3.8	NoData	NoData	NoData
'OLC_C25'	'Oregon.Control'	130	3.60	7.6	2.31	NoData	0.75	5.83	5.9	1.75	286.2	6705
'OLC_C6'	'Oregon.Control'	130	4.80	7.7	2.01	NoData	0.75	5.83	5.9	1.92	213.0	6430
'OLC_11S'	'Oregon.LSM'	130	4.30	7.6	2.33	NoData	1.25	5.70	4.5	2.56	197.0	53000
'OLC_6S'	'Oregon.LSM'	130	4.05	7.7	1.90	NoData	1.25	5.70	4.5	0.69	197.1	3600
'PC_C4'	'Pa.Clearfield'	122	5.90	7.7	3.94	NoData	1.5	3.10	3.9	1.23	133.8	12316
'PC_C3'	'Pa.Clearfield'	122	4.94	7.7	3.48	NoData	1.5	3.10	3.9	1.87	214.4	16344
'PF_C1'	'Pa.Fulton'	125	5.10	7.7	2.81	NoData	1.5	3.90	2.4	2.87	241.4	18619
'PF_C2'	'Pa.Fulton'	125	5.25	7.7	2.73	NoData	1.5	3.90	2.4	1.81	200.1	23334
'WLA_11'	'Wy.Lar.Control'	117	3.91	7.7	2.36	NoData	0.75	4.20	6.3	3.52	329.6	85996
'WLA_1'	'Wy.Lar.Control'	117	3.85	7.7	2.29	NoData	0.75	4.20	6.3	4.14	359.8	31286
'WLB_3'	'Wy.Lar.O.G.LSM'	117	3.30	7.7	1.39	NoData	1.25	4.47	5.5	1.55	349.3	6256
'WLB_51'	'Wy.Lar.O.G.LSM'	117	3.55	7.7	1.79	NoData	1.25	4.47	5.5	1.66	214.3	6377
'WR_A10'	'Wy.RkSpring.O.G.'	120	7.70	7.7	5.82	NoData	1.5	5.30	5.5	1.11 @ 0.016%	105.6	11803
'WRA_8'	'Wy.RkSpring.O.G.'	120	5.25	7.7	1.57	NoData	1.5	5.30	5.5	2.57	186.4	21757

(continued on next page)

TABLE C-1 Test Results on Pavement Cores and Laboratory Compacted Specimens *(Continued)*

File Name	Specimen ID	Temp, F	Height, in	Dia, in	Gage Ln, in	Gradn	MxAg, in	AC%	AV%	Enrgy@2%, lb-in/ft3	Strength, psi	Modulus, psi
	Pavement Cores - Unconfined Compression at Temperature Giving Constant Asphalt Viscosity											
'CC347_1'	'Co.C.P.Control'	124	6.30	7.5	3.00	NoData	0.75	5.78	4.1	0.279	114.4	19458
'CC347_8'	'CO.C.P.Control'	124	5.05	7.6	3.00	NoData	0.75	5.78	4.1	NoData	NoData	NoData
'CC348_13'	'Co.C.P.LSM'	127	3.05	7.7	1.94	NoData	1.5	5.16	5.1	NoData	414.1	NoData
'CC348_9'	'Co.C.P.LSM'	127	3.40	7.7	1.36	NoData	1.5	5.16	5.1	3.99	430.1	21312
'CF374_6'	'Co.Flag.Surface'	140	3.72	7.7	2.29	NoData	1.5	5.25	5.5	2.72	314.0	13450
'CF374_8'	'Co.Flag.Surface'	140	3.60	7.7	1.78	NoData	1.5	5.25	5.5	1.41	342.2	7862
'CF395_4'	'Co.Flag.Base'	129	4.47	7.7	1.18	NoData	1.5	4.65	3.7	NoData	NoData	NoData
'CF395_6'	'Co.Flag.Base'	129	4.15	7.6	1.06	NoData	1.5	4.65	3.7	NoData	NoData	NoData
'K11_B'	'Kentucky'	133	11.50	7.6	9.50	NoData	1.5	3.50	3.8	NoData	NoData	NoData
'KC10'	'Kentucky'	133	11.70	7.6	9.50	NoData	1.5	3.50	3.8	NoData	NoData	NoData
'OL_C1'	'Oregon.Control'	113	3.75	7.7	1.76	NoData	0.75	5.83	5.9	5.00	422.8	NoData
'OL_C7'	'Oregon.Control'	113	4.25	7.7	1.98	NoData	0.75	5.83	5.9	1.96	251.8	9922
'OL_C4S'	'Oregon.LSM'	113	4.30	7.7	2.32	NoData	1.25	6.28	4.5	1.17	239.7	4818
'OL_C8S'	'Oregon.LSM'	113	4.05	7.6	2.55	NoData	1.25	6.28	4.5	2.61	311.4	13579
'PC_C5'	'Pa.Clearfield'	117	4.60	7.6	2.54	NoData	1.5	3.10	3.9	2.36	165.8	13299
'PC_C8'	'Pa.Clearfield'	117	5.80	7.6	3.89	NoData	1.5	3.10	3.9	1.65	139.1	14077
'PF_C6'	'Pa.Fulton'	118	4.70	7.7	2.74	NoData	1.5	3.90	2.4	2.19	277.7	18621
'PF_C7'	'Pa.Fulton'	118	4.00	7.7	1.67	NoData	1.5	3.90	2.4	2.11	357.8	9744
'WL_A15'	'Wy.Lar.Control'	136	3.80	7.7	2.14	NoData	0.75	4.20	6.3	1.66	266.2	6696
'WL_A2'	'Wy.Lar.Control'	136	3.15	7.7	1.74	NoData	0.75	4.20	6.3	2.56	419.2	11624
'WL_B13'	'Wy.Lar.O.G.LSM'	125	3.20	7.8	1.70	NoData	1.25	4.47	5.5	4.05	301.1	12682
'WL_B14'	'Wy.Lar.O.G.LSM'	125	3.40	7.7	1.68	NoData	1.25	4.47	5.5	2.36	410.5	11179
'WRA_8'	'Wy.RkSpring.O.G.'	120	5.25	7.7	1.57	NoData	1.5	5.30	5.5	2.57	186.4	21757
'WR_A10'	'Wy.RkSpring.O.G.'	120	7.70	7.7	5.82	NoData	1.5	5.30	5.5	NoData	105.6	11803
'K17_2'	'Kentucky Molded'	133	7.30	7.5	4.89	NoData	2	1.70	14.9	2.28	134.0	29321
'K17_1'	'Kentucky Molded'	133	7.55	7.5	5.50	NoData	2	1.70	14.5	1.59	97.1	24431
'K22_8'	'Kentucky Molded'	133	6.95	7.5	5.06	NoData	2	2.20	12.3	2.32	143.0	26382
'K22_4'	'Kentucky Molded'	133	7.30	7.5	5.04	NoData	2	2.20	13.5	2.04	132.2	14527
'K27_1'	'Kentucky Molded'	133	7.10	7.5	4.78	NoData	2	2.70	11.3	1.43	95.4	19412
'K27_2'	'Kentucky Molded'	133	7.05	7.5	4.33	NoData	2	2.70	11.3	1.78	110.0	15648
'K33_1'	'Kentucky Molded'	133	6.50	7.5	5.06	NoData	2	3.30	11	1.68	175.0	8885
'K35_2'	'Kentucky Molded'	133	6.90	7.5	4.92	NoData	2	3.50	11.2	NoData	NoData	NoData
'K35_1'	'Kentucky Molded'	133	6.80	7.5	4.85	NoData	2	3.50	12.3	1.37	86.8	16925
K43_2'	'Kentucky Molded'	133	7.00	7.5	4.64	NoData	2	4.30	10.4	0.29	39.2	3898
K43_1'	'Kentucky Molded'	133	6.90	7.6	2.82	NoData	2	4.30	9	NoData	NoData	NoData

TABLE C-1 Test Results on Pavement Cores and Laboratory Compacted Specimens *(Continued)*

File Name	Specimen ID	Temp, F	Height, in	Dia, in	Gage Ln, in	Gradn	MxAg, in	AC%	AV%	Enrgy@2%, lb-in/ft3	Strength, psi	Modulus, psi
	Study of Sensitivity of Mixture Properties to Specimen Dimensions											
'S7156'	'Kentucky Molded'	104	7.55	7.5	4.51	0.45	1.5	2.90	6.8	3.17	210.1	38470
'S7158'	'Kentucky Molded'	104	7.60	7.5	5.64	0.45	1.5	2.90	6.3	2.66	211.5	20964
'S711'	'Kentucky Molded'	104	7.50	7.5	5.54	0.45	1	2.90	4.6	2.66	213.7	20329
'S712'	'Kentucky Molded'	104	7.50	7.5	3.47	0.45	1	2.90	4.7	2.48	228.9	22080
'S714'	'Kentucky Molded'	104	7.55	7.5	5.49	0.45	1	2.90	4.6	3.79	288.1	36467
'S7159'	'Kentucky Molded'	104	7.55	7.5	5.40	0.45	1.5	2.90	6.6	3.47	268.4	28035
'S7157'	'Kentucky Molded'	104	7.40	7.5	5.71	0.45	1.5	2.90	6.2	2.41	181.7	26087
'S7154'	'Kentucky Molded'	104	7.30	7.5	5.25	0.45	1.5	2.90	6.5	2.87	223.3	33693
'S7155'	'Kentucky Molded'	104	7.50	7.5	5.40	0.45	1.5	2.90	6.3	2.37	196.1	25855
'S7153'	'Kentucky Molded'	104	7.20	7.5	5.35	0.45	1.5	2.90	5.7	3.32	256.0	46480
'S7252'	'Kentucky Molded'	104	7.40	7.5	5.91	0.45	2.5	2.90	1.9	2.49	219.7	19135
'S7253'	'Kentucky Molded'	104	7.25	7.5	5.77	0.45	2.5	2.90	1.6	3.88	269.9	44492
'S713'	'Kentucky Molded'	104	7.55	7.5	5.66	0.45	1	2.90	4.6	3.87	293.7	36317
'S7254'	'Kentucky Molded'	104	7.20	7.5	4.59	0.45	2.5	2.90	1.9	4.65	344.2	38747
'S7251'	'Kentucky Molded'	104	7.40	7.5	5.28	0.45	2.5	2.90	1.8	3.48	235.9	44115
'S7152'	'Kentucky Molded'	104	7.20	7.5	5.03	0.45	1.5	2.90	5.7	3.90	273.9	44010
'S214'	'Kentucky Molded'	104	3.10	7.5	1.70	0.45	1	2.90	6.5	2.37	344.2	11476
'S2253'	'Kentucky Molded'	104	3.10	7.5	1.60	0.45	2.5	2.90	6.1	NoData	412.5	70771
'S212'	'Kentucky Molded'	104	3.05	7.5	1.74	0.45	1	2.90	5.7	5.54	414.7	33840
'S213'	'Kentucky Molded'	104	3.05	7.5	1.68	0.45	1	2.90	6.5	6.19	410.9	80888
'S2254'	'Kentucky Molded'	104	2.85	7.5	1.29	0.45	2.5	2.90	6.1	6.01	423.2	32772
'S2151'	'Kentucky Molded'	104	2.90	7.5	1.51	0.45	1.5	2.90	6.5	5.02	412.3	43016
'S211'	'Kentucky Molded'	104	3.05	7.5	1.82	0.45	1	2.90	5.7	2.25	293.7	9866
'S2252'	'Kentucky Molded'	104	3.05	7.5	1.94	0.45	2.5	2.90	6.1	6.43	423.6	39761
'S2154'	'Kentucky Molded'	104	3.10	7.5	1.76	0.45	1.5	2.90	6.9	4.55	405.5	23644
'S2152'	'Kentucky Molded'	104	3.30	7.5	1.99	0.45	1.5	2.90	6.5	NoData	404.0	NoData
'S2251'	'Kentucky Molded'	104	3.10	7.5	1.82	0.45	2.5	2.90	6.1	6.16	422.2	35743
'S2153'	'Kentucky Molded'	104	2.80	7.5	1.82	0.45	1.5	2.90	6.9	4.52	427.6	21898
'S7151'	'Kentucky Molded'	104	7.55	7.5	5.56	0.45	1.5	2.90	6.1	2.36	189.1	27448
'S7152B'	'Kentucky Molded'	104	7.40	7.6	6.09	0.9	1.5	2.00	5.7	NoData	NoData	NoData
'S7151B'	'Kentucky Molded'	104	7.90	7.5	6.53	0.9	1.5	2.00	5.7	1.18	70.0	20111
'S2251B'	'Kentucky Molded'	104	2.25	6.0	1.19	0.45	2.5	2.90	6.1	2.78	616.0	13827
'S2252B'	'Kentucky Molded'	104	2.33	6.0	1.29	0.45	2.5	2.90	6.1	8.54	630.6	75308
'S2253B'	'Kentucky Molded'	104	2.15	6.0	1.48	0.45	2.5	2.90	6.1	5.34 @ 1.6%	620.4	31694

TABLE C-1 Test Results on Pavement Cores and Laboratory Compacted Specimens *(Continued)*

File Name	Specimen ID	Temp, F	Height, in	Dia, in	Gage Ln, in	Gradn	MxAg, in	AC%	AV%	Enrgy@2%, lb-in/ft3	Strength, psi	Modulus, psi
	Study of Sensitivity of Mixture Properties to Specimen Dimensions											
'S6103'	'Kentucky Molded'	104	8.20	6.0	6.56	0.45	1	2.90	6.7	2.35	164.3	29462
'S3152'	'Kentucky Molded'	104	3.00	7.5	1.27	0.9	1.5	2.00	6.5	1.18	391.1	6642
'S3151'	'Kentucky Molded'	104	2.95	7.5	1.37	0.9	1.5	2.00	6.5	NoData	NoData	NoData
'S2155'	'Kentucky Molded'	104	3.00	7.6	1.53	0.9	1.5	2.00	15.8	1.86	382.4	7244
'S2156'	'Kentucky Molded'	104	2.90	7.6	1.34	0.9	1.5	2.00	15.8	2.36	388.1	11682
'S8101'	'Kentucky Molded'	104	8.15	6.0	6.85	0.45	1	2.90	6.7	2.66	167.9	35765
'S8252'	'Kentucky Molded'	104	8.00	6.0	6.70	0.45	2.5	2.90	3.7	3.42	213.5	52570
'S8251'	'Kentucky Molded'	104	7.60	6.0	6.54	0.45	2.5	2.90	3.5	2.36	177.2	17135
'S8253'	'Kentucky Molded'	104	8.10	6.0	6.67	0.45	2.5	2.90	4	4.68	279.3	61553
'S6102'	'Kentucky Molded'	104	8.60	6.0	6.82	0.45	1	2.90	7.1	2.35	144.4	32760
'S7158B'	'Kentucky Molded'	104	7.70	7.5	6.38	0.9	1.5	2.00	18	1.86	109.6	27651
'451011'	'Kentucky Molded'	104	9.40	6.0	7.75	0.45	1	2.90	6.8	1.78	115.5	20360
'451012'	'Kentucky Molded'	104	9.50	7.5	7.83	0.45	1	2.90	6	2.09 @ 1.9%	145.9	21253
'4510152'	'Kentucky Molded'	104	9.25	7.5	7.68	0.45	1.5	2.90	5.6	1.85	119.9	27471
'451013'	'Kentucky Molded'	104	9.50	7.5	7.90	0.45	1	2.90	5.8	2.39	161.2	22164
'4510151'	'Kentucky Molded'	104	9.25	7.5	7.64	0.45	1.5	2.90	5.1	2.88	179.3	29408
'451014'	'Kentucky Molded'	104	9.55	7.5	7.93	0.45	1	2.90	6.6	2.74	184.6	25628
'97154'	'Kentucky Molded'	104	8.00	7.5	6.27	0.9	1.5	2.00	19	0.38 @ 1.7%	30.7	5462
'4510153'	'Kentucky Molded'	104	9.20	7.5	8.08	0.45	1.5	2.90	4	2.11	138.0	28900
'4510154'	'Kentucky Molded'	104	9.40	7.5	7.84	0.45	1.5	2.90	4.8	2.18	142.0	29447
'4510251'	'Kentucky Molded'	104	9.30	7.5	7.70	0.45	2.5	2.90	4.2	2.57	156.1	37733
'4510252'	'Kentucky Molded'	104	9.10	7.5	7.59	0.45	2.5	2.90	4.3	2.93	218.4	18932
'4510254'	'Kentucky Molded'	104	9.30	7.5	7.74	0.45	2.5	2.90	4.8	3.08	193.5	34525
'4510253'	'Kentucky Molded'	104	9.35	7.5	7.72	0.45	2.5	2.90	4	1.89	126.4	27901
'910152'	'Kentucky Molded'	104	9.40	7.5	7.79	0.9	1.5	2.00	16.2	1.17	70.1	21399
'910153'	'Kentucky Molded'	104	9.10	7.5	7.71	0.9	1.5	2.00	16.1	1.46 @ 1.9%	90.5	23075
'910154'	'Kentucky Molded'	104	9.40	7.5	7.82	0.9	1.5	2.00	14.7	1.12	66.4	31651
'910151'	'Kentucky Molded'	104	9.20	7.5	7.65	0.9	1.5	2.00	18.5	0.98	63.2	11512
'451212'	'Kentucky Molded'	104	10.85	6.0	9.21	0.45	1	2.90	4.5	NoData	261.9	34416
'451211'	'Kentucky Molded'	104	10.85	6.0	9.14	0.45	1	2.90	5.1	4.62	311.9	36367
'4512251'	'Kentucky Molded'	104	11.00	6.0	9.34	0.45	2.5	2.90	5.9	NoData	192.1	26680
'4512253'	'Kentucky Molded'	104	10.80	6.0	9.23	0.45	2.5	2.90	6.7	2.61	172.2	32815
'4512252'	'Kentucky Molded'	104	10.95	6.0	9.34	0.45	2.5	2.90	7.1	3.19	194.5	38697
'451213'	'Kentucky Molded'	104	10.60	6.0	9.13	0.45	1	2.90	4.1	NoData	249.8	24447

TABLE C-1 Test Results on Pavement Cores and Laboratory Compacted Specimens *(Continued)*

File Name	Specimen ID	Temp, F	Height, in	Dia, in	Gage Ln, in	Gradn	MxAg, in	AC%	AV%	Enrgy@2%, lb-in/ft3	Strength, psi	Modulus, psi
	Laboratory Molded Specimens Designed using TTI LSM Mixture Design Procedure on Aggregates from Az, Ky, N Mex.											
'TAZ271'	'Arizona'	104	7.88	7.5	6.35	0.92	1	2.70	17.8	0.81	58.5	7354
'TAZ272'	'Arizona'	104	7.80	7.5	6.21	0.92	1	2.70	18.1	0.61	40.7	13436
'TAZ321'	'Arizona'	104	7.73	7.5	6.03	0.92	1	3.20	17.6	1.38	89.1	16595
'TAZ322'	'Arizona'	104	7.65	7.5	6.06	0.92	1	3.20	16.9	1.09	71.8	13436
'TKY201'	'Kentucky'	104	7.85	7.5	6.17	0.71	1.5	2.00	17.3	1.4	85	19178
'TKY202'	'Kentucky'	104	7.80	7.5	6.04	0.71	1.5	2.00	16.4	1.68	100.4	27845
'TKY281'	'Kentucky'	104	7.40	7.5	5.72	0.71	1.5	2.80	13.3	1.65	113	26842
'TKY282'	'Kentucky'	104	7.50	7.5	5.82	0.71	1.5	2.80	13.3	1.47	97.5	15397
'TKY362'	'Kentucky'	104	7.60	7.5	5.92	0.71	1.5	3.60	15.5	1.68	100.8	31058
'TKY361'	'Kentucky'	104	7.50	7.5	6.09	0.71	1.5	3.60	15.1	1.44	72.3	23297
'TNM272'	'New Mexico'	104	7.65	7.5	5.91	0.6	1.5	2.70	12.4	1.48	112.9	14253
'TNM271'	'New Mexico'	104	7.725	7.5	6.375	0.6	1.5	2.70	12.4	0.81	58.8	7354
'TNM352'	'New Mexico'	104	7.55	7.5	5.82	0.6	1.5	3.50	12.1	1.71	116	20984
'TNM351'	'New Mexico'	104	7.45	7.5	5.68	0.6	1.5	3.50	12.9	0.98	75.5	13328
	Laboratory Molded Specimens for SHRP Constant Height Shear Tests Using Indiana DOT Aggregates											
'M8_INDF'	'Indiana Shear'	104	3.75	7.5		NoData	2	2.00	7.1	NoData	NoData	NoData
'M7_INDF'	'Indiana Shear'	104	3.45	7.5		NoData	2	2.50	5.1	NoData	NoData	NoData
'M4_INDF'	'Indiana Shear'	104	3.19	7.5		NoData	2	2.77	6.7	NoData	NoData	NoData
'M1_INDF'	'Indiana Shear'	104	3	7.5		NoData	2	2.80	6.1	NoData	NoData	NoData
'M3_INDF'	'Indiana Shear'	104	3.25	7.5		NoData	2	2.80	6.8	NoData	NoData	NoData
'M5_INDF'	'Indiana Shear'	104	3.19	7.5		NoData	2	3.00	4.2	NoData	NoData	NoData
'M6_INDF'	'Indiana Shear'	104	3.3	7.5		NoData	2	3.25	7.1	NoData	NoData	NoData
'M9_INDF'	'Indiana Shear'	104	3.8	7.5		NoData	2	3.50	6.6	NoData	NoData	NoData
'IND_45.1'	'Indiana Shear'	104	3.25	7.5		0.45	2	4.77	4.2	NoData	NoData	NoData
'IND_45.2'	'Indiana Shear'	104	3.25	7.5		0.45	2	4.77	7.6	NoData	NoData	NoData

TABLE C-1 Test Results on Pavement Cores and Laboratory Compacted Specimens *(Continued)*

File Name	Specimen ID	Temp, F	Height, in	Dia, in	Gage Ln, in	Gradn	MxAg, in	AC%	AV%	Enrgy@2%, lb-in/ft3	Strength, psi	Modulus, psi
	Confined Compression at 7-Day Maximum Annual Pavement Temperature											
'CC347_14'	'Co.CP.Control'	124	5.55	7.7	5.55	NoData	0.75	5.78	4.1	NoData	NoData	NoData
'CC347_10'	'Co.CP.Control'	124	5.8	7.7	5.8	NoData	0.75	5.78	4.1	0.165	NoData	10015
'CC347_13'	'Co.CP.Control'	124	4.35	7.6	4.35	NoData	0.75	5.78	4.1	.279	114.4	19458
'CC348_6'	'Co.CP.LSM'	124	2.6	7.65	2.6	NoData	1.5	5.16	5.1	0.195	67.5	17739
'CC348_14'	'Co.CP.LSM'	124	2.4	7.7	2.4	NoData	1.5	5.16	5.1	0.208	111.6	17005
'CF395_7'	'Co.Flag.Base'	129	3.55	7.7	3.55	NoData	1.5	4.65	3.7	0.201	NoData	11824
'CF395_1'	'Co.Flag.Base'	129	2.94	7.7	2.94	NoData	1.5	4.65	3.7	0.231	NoData	14106
'CF374_4'	'Co.Flag.Surface'	129	3.41	7.7	3.41	NoData	1.5	5.25	5.5	NoData	NoData	46698
'CF374_14'	'Co.Flag.Surface'	129	3.31	7.7	3.31	NoData	1.5	5.25	5.5	0.159	NoData	8995
'KY_11'	'Kentucky'	133	3.8	7.7	3.8	NoData	1.5	3.50	3.8	0.200	111.1	9855
'KY_8'	'Kentucky'	133	4.13	7.7	4.13	NoData	1.5	3.50	3.8	NoData	111.2	13932
'OLCS_13'	'Oregon.LSM'	130	4	7.7	4	NoData	1.25	6.28	4.5	0.108	NoData	5584
'OLCS_7'	'Oregon.LSM'	130	3.63	7.7	3.63	NoData	1.25	6.28	4.5	0.250	NoData	16775
'WLA_10'	'Wy.Lar.Control'	117	3.85	7.7	3.85	NoData	0.75	4.20	6.3	0.183	NoData	12684
'WL_A16B'	'Wy.Lar.Control'	117	3.45	7.7	3.45	NoData	0.75	4.20	6.3	NoData	NoData	NoData
'WLA_16'	'Wy.Lar.Control'	117	3.5	7.7	3.5	NoData	0.75	4.20	6.3	NoData	NoData	13627
'WL_B10B'	'Wy.Lar.O.G.LSM'	117	3.15	7.7	3.15	NoData	1.25	4.47	5.5	NoData	NoData	NoData
'WL_B6B'	'Wy.Lar.O.G.LSM'	117	3.4	7.65	3.4	NoData	1.25	4.47	5.5	NoData	NoData	NoData
'WL_B6'	'Wy.Lar.O.G.LSM'	117	3.45	7.7	3.45	NoData	1.25	4.47	5.5	NoData	NoData	20653
'WL_B10'	'Wy.Lar.O.G.LSM'	117	3.2	7.7	3.2	NoData	1.25	4.47	5.5	0.120	NoData	11956
'WRA_1'	'Wy.RkSpring.O.G.'	120	7.05	7.7	7.05	NoData	1.5	5.30	5.5	NoData	NoData	NoData
'WR_A4'	'Wy.RkSpring.O.G.'	120	6.82	7.7	6.82	NoData	1.5	5.30	5.5	0.135	NoData	9221

APPENDIX D

ACCELERATED PERFORMANCE TESTING OF LSM WITH THE INDIANA DOT ACCELERATED LOADING FACILITY

INTRODUCTION

The National Cooperative Highway Research Program (NCHRP) Project 4-18, "Design and Evaluation of Large Stone Asphalt Mixtures," was conducted by the Texas Transportation Institute (TTI). The study developed a two-level mix design and analysis method for hot-mix asphalt (HMA) containing aggregates from 1 inch (25 mm) to 2.5 inches (63 mm), including laboratory procedures to evaluate the relative ability of LSM to resist rutting. The new design procedure guarantees stone-on-stone contact of the largest aggregate. The objective, of course, is resistance to rutting under heavy, concentrated repeated loads.

As a part of this study, TTI contracted with the Division of Research of the Indiana Department of Transportation (INDOT) to conduct accelerated, full-scale rutting testing on a series of LSM. The primary objective of this portion of the study was to determine whether the new mix design procedure would, in fact, produce an LSM that is more resistant to rutting than conventional mixes and LSM designed with other procedures.

MATERIALS AND EQUIPMENT

Target gradations and target binder contents for the three LSM evaluated are shown in Tables D-1 and D-2, respectively. The LSM were manufactured using Indiana materials from the Fort Wayne district (France Stone Co.) and a typical AC-20 binder. A local contractor was engaged to manufacture the mixes. A batch plant was used to produce the mixes, which were delivered to the INDOT Division of Research Accelerated Loading Facility, placed using a conventional paver, and compacted using a steel wheel roller.

During construction, compaction was monitored using an INDOT practice of "peaking" the density using a Troxler nuclear gauge. A target, compacted layer thickness of 76 mm could not be maintained with these LSM due to the extreme particle size and gradations.

Each 5 ft × 20 ft (1.5 m × 6.1 m) lane contained a different LSM:

Lane 1: Texas Mix (Fine)
Lane 2: Texas Mix (Coarse)
Lane 3: INDOT #2 Base
Lane 4: INDOT #2 Base (Replicate)

Prior to trafficking, cross-sectional profiles were recorded at nine locations along each lane. These were used as the reference cross sections to measure rutting in each lane. Cross-section measurements were repeated at the same locations after 100, 300, 600, 1000, 1500, 2000, 2500, 3000, 3500, 4000, 4500, and 5000 repetitions of the loading mechanism.

The accelerated pavement testing (APT) equipment was used to traffic the lanes and applied a 9,000 lb (4082 kg) force through dual tires inflated to 90 psi (620 kPa). The loading frame applied the load in one direction at a speed of 5 mph (8 km/hr). The slabs were maintained at a temperature of 100° ± 2°F (64 ± 1°C) throughout the test.

PRE-PROCESSING OF THE RUTTING DATA

Cross-section profiles were recorded using a Rainhart Transverse Profilograph (#865). This equipment (used in its half-length configuration) has been modified by the INDOT Division of Research so that the measured cross section is directly recorded in digital form to a computer file, with the x and z motions of the recording carriage measured using linear transducers. In this way, the horizontal (x-coordinates) are recorded to an accuracy of 0.1 inch (2.54 mm) and the vertical (z-coordinates) to an accuracy of 0.01 inch (0.25 mm).

The recorded cross-section measurements were analyzed as follows:

- The cross section after N repetitions was superimposed (by computer) upon the original reference cross section.

TABLE D-1 Aggregate Target Gradations

Sieve	Mixture (% Passing)		
	Texas Fine	Texas Coarse	INDOT #2
2½ in	100	100	100
2 in	98.6	93.4	92.2
1½ in	93.6	69.1	63.6
1 in	89.3	50.8	43.0
3/4 in	83.6	38.4	33.2
1/2 in	80.2	31.5	28.0
3/8 in	73.6	25.4	23.5
#4	50.5	13.3	14.0
#8	35.3	7.6	9.3
#16	22.8	5.0	6.1
#30	11.6	2.8	3.3
#50	5.8	1.7	1.8
#100	2.5	1.0	1.0
#200	1.2	0.6	0.6

- The reference cross section was subtracted from the N-repetition cross-section, thereby removing from the plot any irregularities (roller marks, etc.) in the original surface.
- A template of the dual tire assembly was passed across the rectified cross section and the location of maximum vertical deformation (depression) was identified.
- At this location, the average vertical depression under the tire footprint was calculated.
- The cross section was smoothed using a low pass fast fourier transform technique to remove "spikes" from the data.
- The high points (maximum heave) outside and between the tires were located and a "string-line" constructed across them.
- The average height between the string-line and the base-line was computed under the tire template.
- The total rut depth was then computed as the summation of the average depression plus the average heave.

This analysis was repeated at each of the nine cross sections within each lane for each of the loading increments above. Thus, a total of 9 cross sections times 12 loading increments times 4 lanes or a total of 432 measurement points were analyzed.

TEST RESULTS

Compaction

Compaction densities were measured at nine locations in each lane prior to trafficking (Table D-3). The irregular surface of the LSM and the large degree of macrotexture introduced unusually large variabilities in the recorded densities. The initial densities have been subjected to statistical analysis (Table D-4). This indicates the Texas Fine LSM exhibits an overall higher density than all the other LSM. Depending on the statistical test involved (LSD [T] or Bonferroni), the Texas Coarse LSM and the two INDOT

TABLE D-2 Target Binder Contents

Mixture	Texas Fine	Texas Coarse	INDOT #2
Bit Content %	4.8%	2.5%	3.0%

TABLE D-3 Pre-Trafficking and Post-Trafficking Densities

X-section	Lane 1	Lane 2	Lane 3	Lane 4
1	135.4	110.9	124.3	122.7
2	131.0	123.7	122.4	125.7
3	138.2	118.3	129.7	128.6
4	135.6	126.3	128.0	132.7
5	136.0	129.1	126.1	133.8
6	132.2	125.0	122.5	121.4
7	138.4	114.1	121.6	121.6
8	136.5	125.0	120.9	121.7
9	134.1	121.4	119.7	128.6
Mean	135.3	121.5	123.9	126.3
St. Dev.	2.5	6.0	3.4	4.9

(a) Pre-Trafficking Densities (lb/cu.ft)

X-section	Lane 1	Lane 2	Lane 3	Lane 4
6/7	137.7	128.9	130.9	125.6
5/6	138.6	126.3	129.8	123.3
4/5	141.7	117.2	128.6	116.8
Mean	139.3	124.1	129.8	121.9
St. Dev.	2.1	6.1	1.2	4.6

(b) Post-Trafficking Densities

LSM may be classified together or in pairs (Texas Coarse + INDOT #2 [Lane 3], and INDOT #2 [Lane 3] + INDOT #2 [Lane 4]). The difference is pro-bably not of practical significance, but is reported for completeness.

As the project proceeded, it proved impossible to obtain densities in the developing ruts, because the rut width and nuclear gauge dimensions were such that a significant gap developed between the base of the gauge and the rutted surface.

At the conclusion of the testing, a few densities were obtained by manually removing some of the "heaved" material to widen the ruts in selected locations. There were still sufficient surface irregularities under the gauge that the results may not be entirely representative.

Rutting

The measured ruts are summarized for each LSM in Table D-5. Initial plots (Figures D-1 through D-4) of the results from each lane were inspected. The two Texas LSM were quite distinct from each other, and rutting in the Texas Coarse LSM (Lane 2) was only a fraction of that in the Texas Fine LSM (Lane 1). The two INDOT LSM (Lanes 3 and 4) appeared to be quite similar to each other and also to rut somewhat less than the Texas Fine LSM, but significantly more than the Texas Coarse LSM.

There was, however, significant variability observed throughout the testing. The coefficient of variation of ruts measured under the INDOT Minimum Crushed Aggregate Study demonstrated a stable value of about 12%. The values obtained under this study are significantly larger and may be attributable to the high degree of macrotexture of the surfaces due to the coarser gradations involved.

Statistical Analysis

Statistical analyses were performed on the rutting data to determine whether any statistical difference existed between the rutting response of the four LSM tested, and to

TABLE D-4 Statistical Analysis for Lane Density

ONE-WAY ANOVA
Coefficient of Density (Pre-traffic) by Lane

Source	DF	SS	MS	F	P
Between	3	973.694	324.565	16.85	0.0000
Within	32	616.433	19.2635		
Total	35	1590.13			

Bartlett's Test of Equal Variances	Chi-Sq	DF	P
	6.41	3	0.0935

Cochran's Q 0.4664
Largest var/Smallest var 5.8703

Lane	Mean	Sample Size	Group St Dev
1	135.28	9	2.4743
2	121.53	9	5.9948
3	123.91	9	3.3836
4	126.31	9	4.8524
TOTAL	126.76	36	4.3890

LSD (T) Pairwise Comparison of Means of COV by Lane:

Lane	Mean	Homogeneous Groups		
1	135.28	X		
2	121.53		X	
3	123.91		X	X
4	126.31			X

There are 3 groups in which the means are not significantly different from one another. [Critical t-value 2.037, rejection level 0.050]

Bonferroni Pairwise Comparison of Means of COV by Lane:

Lane	Mean	Homogeneous Groups		
1	135.28	X		
2	121.53			X
3	123.91			X
4	126.31			X

There are 3 groups in which the means are not significantly different from one another. [Critical t-value 2.812, rejection level 0.050]

determine whether there was any significant difference between the measured variabilities of the rutting response of each material.

The statistical analysis (Table D-6) showed, as anticipated, that the two INDOT LSM (Lanes 3 and 4) are not significantly different from each other. However, the INDOT LSM differ significantly from the Texas Fine LSM (Lane 1), although, in practical terms, the difference is very small. The Texas Coarse LSM (Lane 2) rutted significantly less than all the other LSM.

A further statistical analysis (Table D-7) was performed to determine whether the ruts in each lane were more or less variable. This analysis showed that the least variable

TABLE D-5 Summary of Measured Rut Depths (in.)

TEXAS LARGE STONE MIXES (Measured Rut Depths (in))

LANE 1:

	# of Loaded Repetitions											
X-Section	100	300	600	1000	1500	2000	2500	3000	3500	4000	4500	5000
1	0.076	0.111	0.145	0.180	0.225	0.222	0.284	0.296	0.273	0.293	0.325	0.325
2	0.076	0.116	0.164	0.188	0.239	0.252	0.289	0.304	0.323	0.330	0.349	0.348
3	0.058	0.099	0.127	0.171	0.198	0.227	0.248	0.269	0.296	0.324	0.317	0.330
4	0.078	0.124	0.166	0.215	0.257	0.271	0.299	0.338	0.340	0.359	0.391	0.361
5	0.084	0.120	0.168	0.186	0.243	0.292	0.284	0.335	0.313	0.346	0.364	0.359
6	0.080	0.106	0.162	0.200	0.231	0.247	0.273	0.291	0.327	0.347	0.366	0.372
7	0.108	0.148	0.188	0.246	0.292	0.316	0.347	0.374	0.393	0.406	0.409	0.466
8	0.103	0.152	0.207	0.238	0.292	0.333	0.356	0.394	0.406	0.440	0.445	0.454
9	0.117	0.140	0.185	0.199	0.253	0.278	0.301	0.325	0.330	0.357	0.342	0.406
Mean	0.087	0.124	0.168	0.203	0.248	0.271	0.298	0.325	0.334	0.356	0.368	0.380
St. Dev	0.019	0.019	0.024	0.026	0.030	0.038	0.034	0.040	0.043	0.044	0.041	0.051

LANE 2:

	# of Loaded Repetitions											
X-Section	100	300	600	1000	1500	2000	2500	3000	3500	4000	4500	5000
1	0.066	0.058	0.097	0.122	0.139	0.178	0.180	0.206	0.213	0.172	0.249	0.225
2	0.028	0.049	0.064	0.109	0.120	0.134	0.128	0.158	0.159	0.183	0.179	0.236
3	0.084	0.092	0.127	0.138	0.171	0.177	0.181	0.214	0.178	0.188	0.215	0.274
4	0.046	0.068	0.112	0.100	0.180	0.195	0.199	0.215	0.212	0.276	0.268	0.296
5	0.060	0.068	0.083	0.113	0.126	0.130	0.159	0.178	0.183	0.194	0.194	0.284
6	0.037	0.068	0.070	0.125	0.157	0.176	0.170	0.221	0.246	0.235	0.277	0.247
7	0.108	0.100	0.120	0.091	0.182	0.169	0.179	0.153	0.193	0.243	0.201	0.252
8	0.050	0.048	0.095	0.081	0.091	0.111	0.171	0.146	0.158	0.186	0.184	0.180
9	0.055	0.064	0.088	0.109	0.125	0.159	0.193	0.199	0.202	0.247	0.184	0.230
Mean	0.056	0.068	0.094	0.110	0.144	0.159	0.173	0.188	0.194	0.214	0.217	0.247
St. Dev	0.023	0.018	0.023	0.018	0.031	0.028	0.021	0.030	0.028	0.037	0.038	0.035

LANE 3:

	# of Loaded Repetitions											
X-Section	100	300	600	1000	1500	2000	2500	3000	3500	4000	4500	5000
1	0.038	0.109	0.168	0.229	0.251	0.211	0.314	0.302	0.348	0.358	0.397	0.371
2	0.106	0.183	0.187	0.225	0.266	0.288	0.320	0.338	0.351	0.326	0.358	0.369
3	0.144	0.227	0.146	0.183	0.166	0.206	0.209	0.194	0.225	0.233	0.237	0.242
4	0.109	0.165	0.157	0.215	0.229	0.237	0.268	0.280	0.269	0.303	0.313	0.299
5	0.050	0.088	0.081	0.125	0.110	0.158	0.175	0.198	0.196	0.208	0.206	0.207
6	0.074	0.106	0.108	0.115	0.140	0.152	0.156	0.173	0.172	0.204	0.177	0.183
7	0.121	0.162	0.200	0.234	0.246	0.251	0.277	0.272	0.320	0.318	0.319	0.336
8	0.045	0.127	0.124	0.181	0.196	0.254	0.256	0.264	0.264	0.307	0.311	0.325
9	0.071	0.149	0.206	0.234	0.259	0.322	0.359	0.379	0.399	0.439	0.456	0.464
Mean	0.084	0.146	0.153	0.193	0.207	0.231	0.260	0.267	0.283	0.300	0.308	0.311
St. Dev	0.038	0.044	0.043	0.046	0.057	0.056	0.068	0.069	0.077	0.076	0.090	0.089

LANE 4:

	# of Loaded Repetitions											
X-Section	100	300	600	1000	1500	2000	2500	3000	3500	4000	4500	5000
1	0.084	0.104	0.141	0.150	0.184	0.206	0.209	0.228	0.260	0.249	0.242	0.294
2	0.091	0.107	0.175	0.136	0.224	0.178	0.200	0.193	0.270	0.217	0.237	0.256
3	0.063	0.138	0.140	0.202	0.277	0.249	0.320	0.263	0.344	0.322	0.295	0.302
4	0.068	0.111	0.152	0.187	0.187	0.222	0.255	0.243	0.271	0.293	0.268	0.345
5	0.072	0.088	0.126	0.131	0.169	0.174	0.212	0.202	0.208	0.196	0.203	0.229
6	0.065	0.101	0.138	0.163	0.182	0.193	0.224	0.255	0.247	0.276	0.278	0.281
7	0.081	0.153	0.197	0.225	0.261	0.308	0.320	0.358	0.345	0.390	0.390	0.383
8	0.119	0.173	0.197	0.246	0.276	0.342	0.330	0.348	0.396	0.435	0.452	0.491
9	0.117	0.161	0.203	0.251	0.274	0.331	0.352	0.294	0.452	0.460	0.435	0.514
Mean	0.087	0.126	0.163	0.188	0.226	0.245	0.269	0.265	0.310	0.315	0.311	0.344
St. Dev	0.020	0.031	0.030	0.046	0.046	0.066	0.061	0.059	0.079	0.094	0.091	0.101

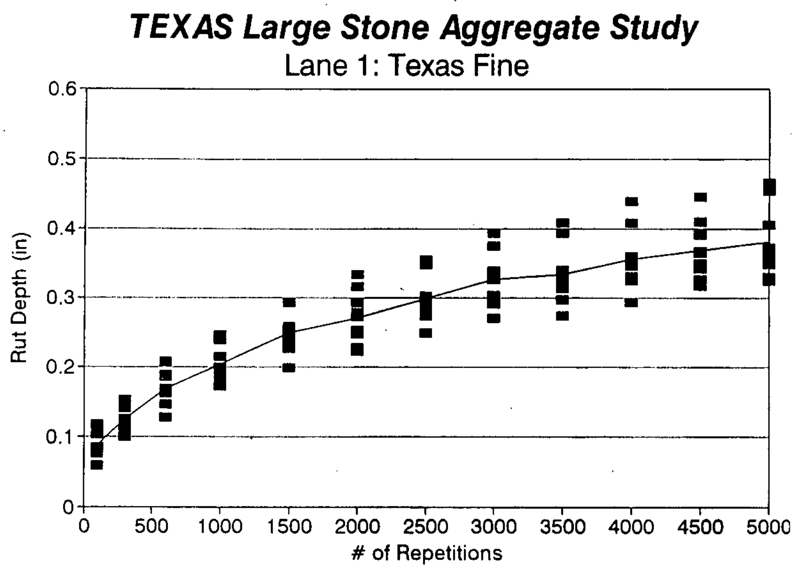

Figure D-1. Texas Study—Lane 1.

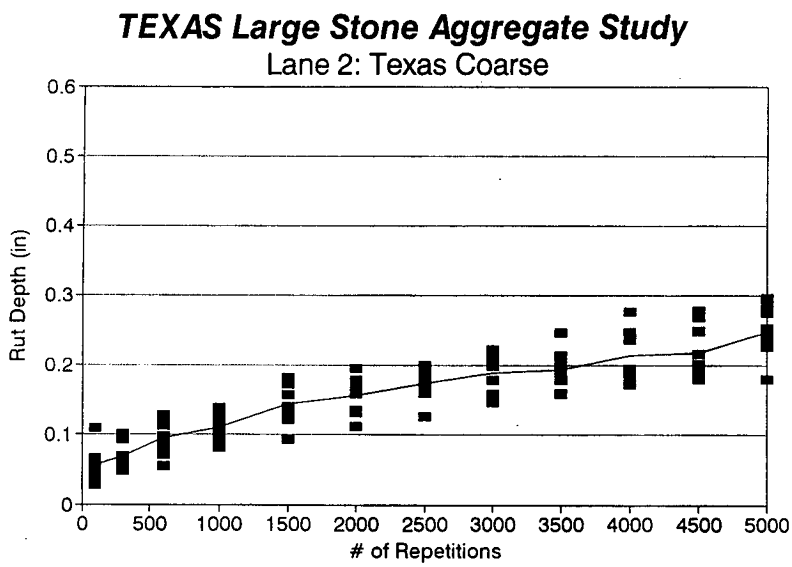

Figure D-2. Texas Study—Lane 2.

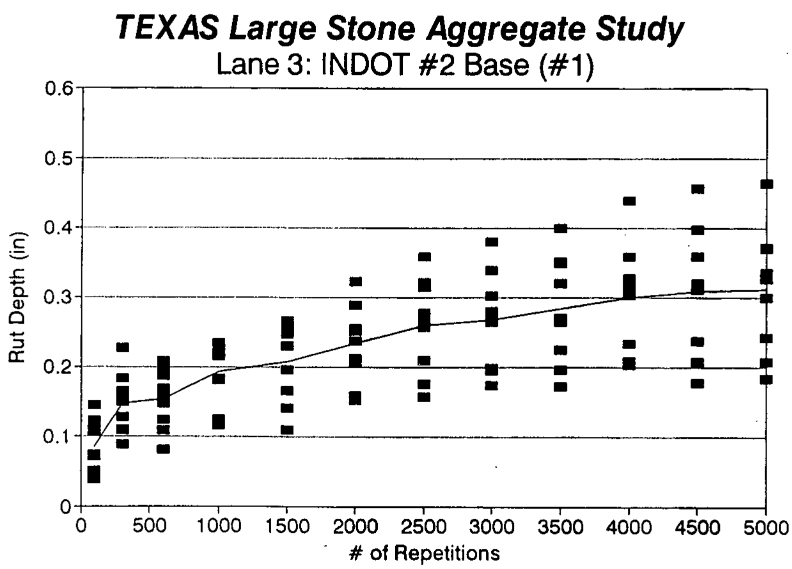

Figure D-3. Texas Study—Lane 3.

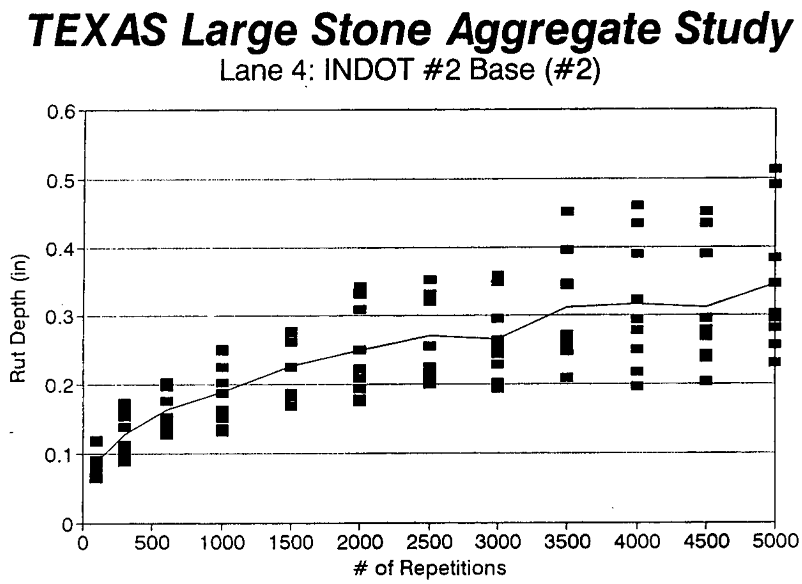

Figure D-4. Texas Study—Lane 4.

TABLE D-6 ANOVA Analysis of Rut Depth by Lane

ONE-WAY ANOVA

Rut Depth (RUT) by Lane

Source	DF	SS	MS	F	P
Between	3	0.69716	0.23239	28.11	0.0000
Within	428	3.53777	0.00826		
Total	431	4.23494			

Bartlett's Test of EqualVariances	Chi-Sq	DF	P
	23.78	3	0.0000

Cochran's Q 0.3058
Largest var/Smallest var 2.3847

Lane	Mean	Sample Size	Group St Dev
1	0.2634	108	0.1005
2	0.1552	108	0.0651
3	0.2285	108	0.0933
4	0.2375	108	0.1000
TOTAL	0.2211	432	0.0909

LSD (T) Pairwise Comparison of Means of RUT by Lane:

Lane	Mean	Homogeneous Groups		
1	0.2634	X		
2	0.1552		X	
3	0.2285			X
4	0.2375			x

There are 3 groups in which the means are not significantly different from one another. [Critical t-value 1.966, rejection level 0.050]

TABLE D-7 ANOVA Analysis of Rut Depth Variation by Lane

ONE-WAY ANOVA

Coefficient of Variation (COV) by Lane

Source	DF	SS	MS	F	P
Between	3	1464.20	488.065	17.42	0.0000
Within	44	1232.57	28.1030		
Total	47	2696.77			

Bartlett's Test of EqualVariances	Chi-Sq	DF	P
	12.75	3	0.0052

Cochran's Q 0.5498
Largest var/Smallest var 8.1868

Lane	Mean	Sample Size	Group St Dev
1	13.600	12	2.7433
2	19.900	12	7.8491
3	28.358	12	5.4139
4	24.675	12	3.6888
TOTAL	21.633	48	5.2927

LSD (T) Pairwise Comparison of Means of RUT by Lane:

Lane	Mean	Homogeneous Groups		
1	13.600	X		
2	19.900		X	
3	24.675			X
4	28.358			x

There are 3 groups in which the means are not significantly different from one another. [Critical t-value 2.015, rejection level 0.050]

rutting response (COV 13.6%) occurred in Lane 1 (Texas Fine). The two INDOT LSM (Lanes 3 and 4) demonstrate the greatest variability and are statistically identical (COV 24.68% and 28.36%), while the Texas Coarse LSM (Lane 2) was intermediate (COV 19.90%). These results agree with the in-place observations: the Texas Fine LSM (Lane 1) had the appearance of a binder or surface mix and its coefficient of variation (13.6%) is not dissimilar to that observed for other INDOT binder mixes (≈ 12.5%). The macrotexture of the materials in lanes 2 through 4 was significantly greater than has been observed on other mixes tested in the APT, which may account for the increased variability of the measured ruts. The two INDOT LSM (Lanes 3 and 4) do, however, exhibit significantly higher variability than the Texas Coarse LSM (COV 24.68% and 28.36% vs 19.9%).

Comparison of LSM to Conventional Mixes

Rutting histories of other mixes tested previously using the INDOT APT are shown in Figures D-5 through D-9. The following mixes are represented:

Figure D-5	MCA-5	Gravel (70% crush count) + 100% natural sand.
Figure D-6	MCA-6	Gravel (70% crush count) + 50% natural sand.
Figure D-7	MCA-7	Gravel (70% crush count) + 0% natural sand.
Figure D-8	MCA-8	Slag + 100% natural sand.
Figure D-9	MCA-9	Limestone (95%+ crush count) + 100% natural sand.

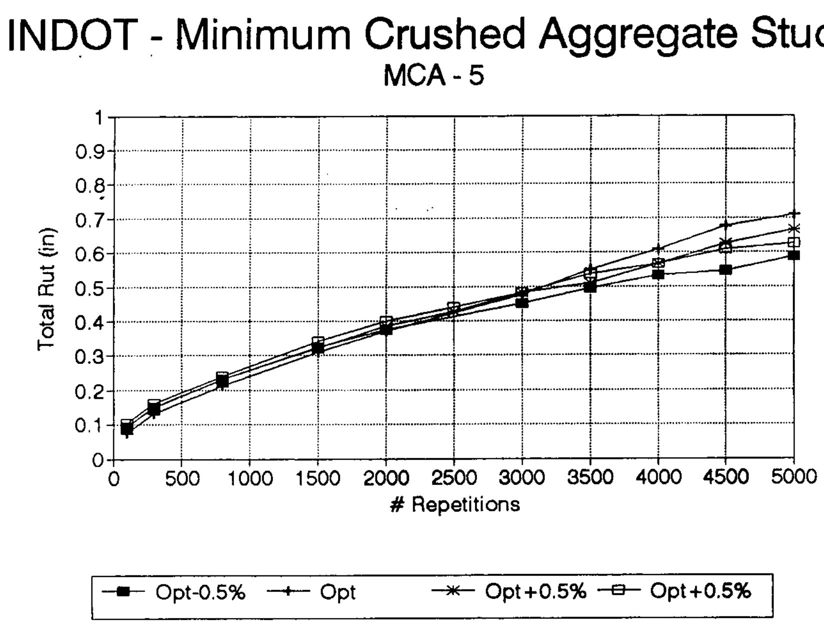

Figure D-5. INDOT Study MCA-5.

These mixes were blended, using different aggregates, to a common target gradation. An AC-20 asphalt binder was used in all mixes and three mixes placed for testing, at nominal binder contents of optimum AC%, optimum AC + 0.5%, and optimum AC – 0.5%.

The two aggregate blends represented in mixes MCA-8 and MCA-9 rutted significantly less than any of the mixes containing gravel and demonstrated magnitudes of rutting similar to those observed in the Texas Coarse LSM (Lane 2). The MCA-9 mix is typical of an INDOT binder course mix.

Figure D-10 summarizes the rutting histories of the four mixes tested under this project, bracketed by the similar histories of MCA-5 and MCA-9 for comparison purposes.

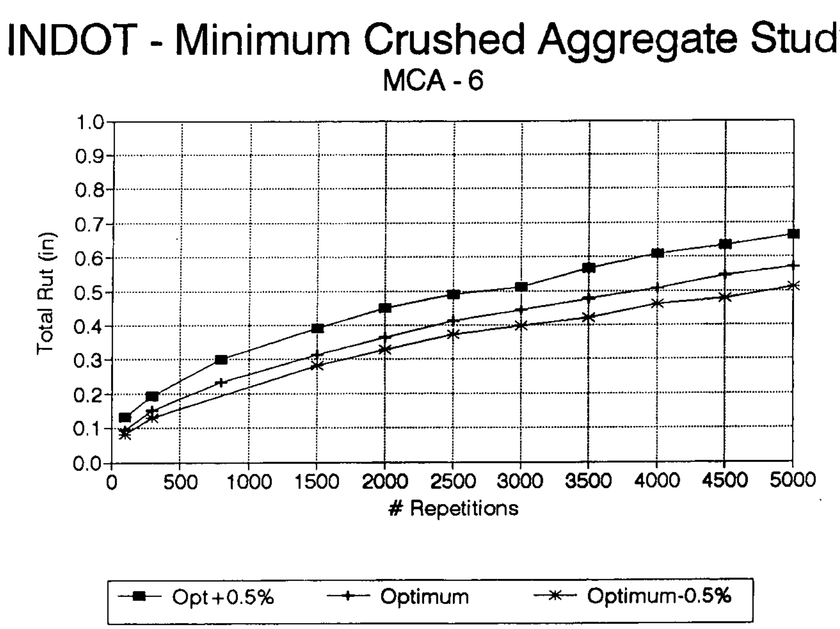

Figure D-6. INDOT Study MCA-6.

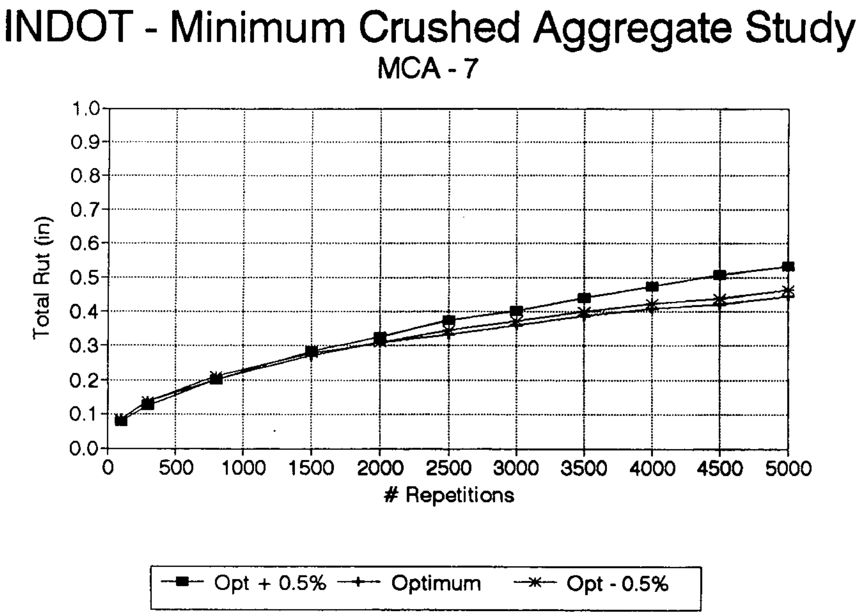

Figure D-7. INDOT Study MCA-7.

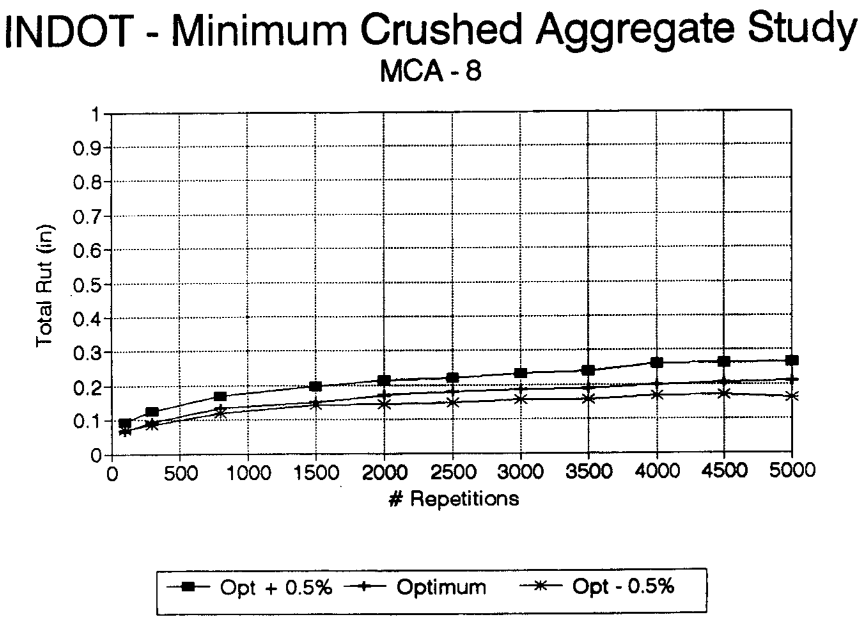

Figure D-8. INDOT Study MCA-8.

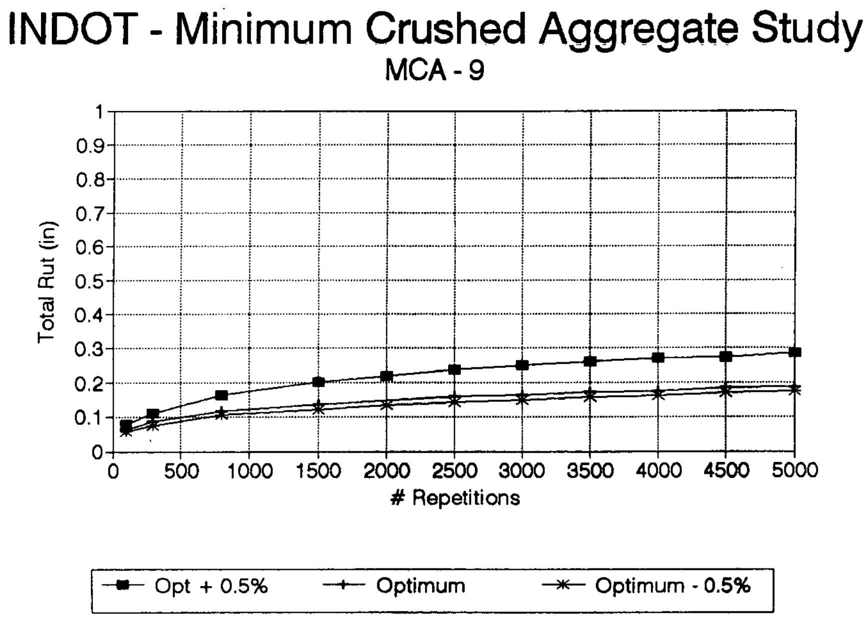

Figure D-9. INDOT Study MCA-9.

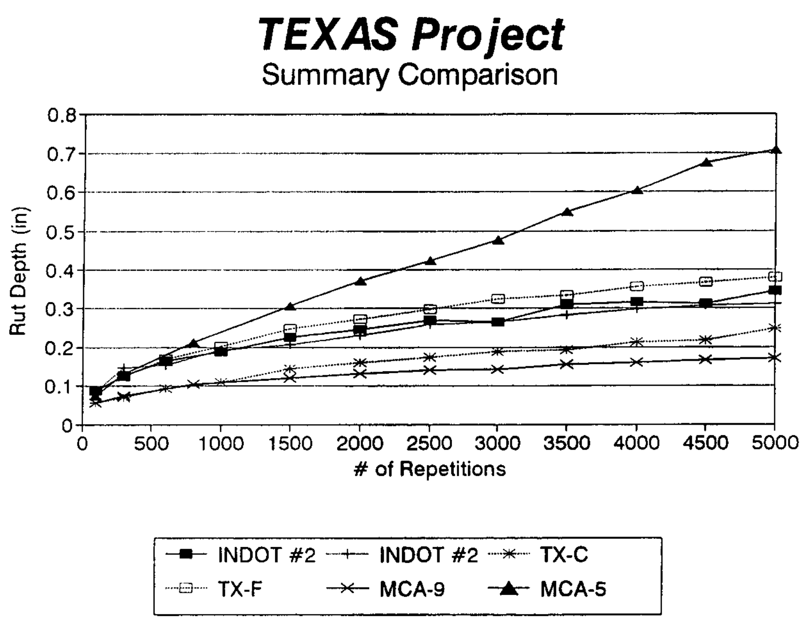

Figure D-10. Texas Study—Summary Comparison.

CONCLUSIONS

The test results support the following conclusions:

1. The Texas Coarse mix (Lane 2) rutted significantly less than all the other mixes.
2. The new LSM design procedure successfully designed a very rut-resistant mix.
3. The compacted surfaces of the LSM were quite rough due to the large particle sizes.
4. The LSM designed purposely with poor stone-on-stone contact (Texas Fine) exhibited the highest rut depths, which were similar to those measured in conventional binder courses. This indicates that large stones floating in a matrix of conventional mix will have little effect on rutting resistance.

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Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
SAE	Society of Automotive Engineers
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board
U.S. DOT	United States Department of Transportation

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