NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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NCHRP Report 425

Designing Stone Matrix Asphalt Mixtures for Rut-Resistant Pavements

Part 1: Summary of Research Results

Part 2: Mixture Design Method, Construction Guidelines, and Quality Control/Quality Assurance Procedures

Transportation Research Board
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Report 425

Designing Stone Matrix Asphalt Mixtures for Rut-Resistant Pavements

Part 1: Summary of Research Results

Part 2: Mixture Design Method, Construction Guidelines, and Quality Control/Quality Assurance Procedures

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

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The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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FOREWORD

By Staff
Transportation Research Board

This report presents a mix design method and construction guidelines for stone matrix asphalt (SMA) in the form of recommended AASHTO standard specifications and practices. It will be of particular interest to materials engineers in the state highway agencies as well as agency and contractor personnel responsible for design and construction of hot mix asphalt (HMA) paving projects. The report also contains a summary of research results that support the development of the method and guidelines, including experimental data obtained during SMA pavement construction to validate the laboratory findings.

Stone Matrix Asphalt (SMA) is defined as a hot mix asphalt (HMA) prepared with gap-graded aggregate in order to maximize the asphalt binder content and coarse aggregate fraction. This design provides a stable stone-on-stone skeleton held together by a rich mixture of asphalt cement, filler, and stabilizing agents such as fibers or asphalt modifiers.

SMA was developed in Europe as a mix design for exceptionally durable wearing courses; these are resistant to rutting (permanent deformation) and studded tire wear when applied over a satisfactory pavement structure. Interest in the use of SMA in the United States was sparked by the strong recommendation of the participants in the 1990 European Asphalt Study Tour sponsored by AASHTO, FHWA, the National Asphalt Pavement Association, the Asphalt Institute, and TRB.

In the United States, the original approach was to duplicate SMA mixtures developed in Europe, using domestic materials, equipment, and asphalt industry expertise. The FHWA has reported that in 1991 four states constructed SMA pavements totaling less than 50,000 tons of HMA; this number rose in 1994 to SMA projects in 23 states requiring 500,000 tons of HMA. A survey of the performance of 88 SMA projects constructed between 1991 and 1995 was completed in 1996; it found that over 90 percent of the pavements had rutting measurements less than 0.4 mm while approximately 25 percent had no measurable rutting.

Given this success, the state transportation agencies, FHWA, and industry identified the need to better define the currently specified components and properties of SMA mixtures and to codify methods and procedures for SMA design and pavement construction. This effort would deal with such issues as the type and amount of stabilizers, the required compactive effort, and mix volumetrics for design and placement. New tests might also be required to measure properties such as durability, rut resistance, draindown, and moisture susceptibility.

Under NCHRP Project 9-8, "Designing Stone Matrix Asphalt Mixtures," the National Center for Asphalt Technology at Auburn University was assigned the task of developing objective procedures for the use of SMA. This included definition of the

material and mix properties required to maximize the durability and rut resistance of SMA and the development and validation of a mix design procedure (including water sensitivity) and construction guidelines in AASHTO standard format.

The research team reviewed relevant domestic and foreign literature on SMA design and construction; carried out an extensive series of laboratory investigations to develop recommended materials specifications, a mix design method and supporting test methods, and construction guidelines; and validated these products with field data obtained during the construction of 11 SMA pavement projects in 1997.

This NCHRP report includes Volumes III and IV of the original five-volume final report for Project 9-8. The first part (the original Volume III) includes a general overview of the entire research effort and the significant findings in the areas of SMA material properties, aggregate skeleton testing, SMA mortar testing, aggregate breakdown analysis, mixing and compaction, permeability, field density requirements, and laboratory and field performance evaluation. The second part (the original Volume IV) provides the main deliverables of the project in the form of recommended AASHTO standards: a practice for designing SMA; a specification for designing SMA; a practice for construction of SMA; a method for determination of draindown characteristics in uncompacted asphalt mixtures; a practice for testing hot mix asphalt (HMA) mortars; and an example SMA mix design.

NOTICE TO READERS

The entire five-volume report for NCHRP Project 9-8 is available on the CD-ROM titled CRP-CD-1: *Bituminous Materials Research Series*, together with the final reports for NCHRP Projects 9-7, "Field Procedures and Equipment to Implement SHRP Asphalt Specifications" (also published as *NCHRP Report 409*, "Quality Control and Acceptance of Superpave-Designed Hot Mix Asphalt") and 9-9, "Refinement of the Superpave Gyratory Compaction Procedure." Copies of the CD-ROM are available, for $10.00, from TRB Publications Sales, PO Box 289, Washington, DC 20055.

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PART 2: Mixture Design Method, Construction Guidelines, and Quality Control/Quality Assurance Procedures

PART 1: Summary of Research Results

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DESIGNING STONE MATRIX ASPHALT MIXTURES FOR RUT-RESISTANT PAVEMENTS

SUMMARY

The National Center for Asphalt Technology (NCAT) has developed and validated a mix design procedure for stone matrix asphalt (SMA). Funded under NCHRP Project 9-8, the work was completed in 1998. The project involved 15 tasks that addressed the following topics: literature review, material and mixture properties, appropriate laboratory tests, development of a mix design procedure, field evaluation of the mix design procedure, development of quality control/quality assurance (QC/QA) procedures, development of construction guidelines, verification of mixing and compaction temperatures, verification of density requirements, and accuracy and precision of nuclear density gauges for SMA pavements.

The results of this project have been published in five volumes. Volume I, "Literature Review," summarizes available literature on SMA up to 1998. This review includes over 100 papers presented primarily during the time period 1989 to 1998.

Volume II(a), "Research Results," presents the results of the initial research performed to measure the critical material and mixture properties of SMA. This volume also describes work accomplished to evaluate the ability of different laboratory tests to characterize SMA mixtures. The work included finding a method of identifying when stone-on-stone contact is achieved within SMA mixtures. Volume II(a) also contains the first tentative SMA mixture design procedure.

Volume II(b), "Research Results," presents the results of work accomplished to finalize the tentative mixture design procedure for SMA. Additional work reported in this volume consisted of analyzing mixture properties that were designed with the finalized mixture design procedure. This testing included indirect tensile creep and wheel track testing.

Volume II(c), "Research Results," presents the results of work performed to verify that the mixtures designed by the final mixture design procedure could be produced and placed in the field. Additional work consisted of developing QC/QA procedures, evaluating SMA gradations with varying nominal maximum aggregate sizes, developing SMA construction guidelines, verifying laboratory mixing and compaction temperatures, verifying in-place density requirements, and evaluating the accuracy of nuclear density gauges for determining the in-place density of SMA pavements.

Volume III, "Summary of Research Results," (published herein) summarizes the research effort for NCHRP Project 9-8. This volume includes summaries of material properties, aggregate skeleton testing, mortar testing, mixture design testing, mixture design analysis, the effect of flat and elongated particles, aggregate breakdown analysis, mixing

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and compaction testing, permeability evaluation, evaluation of nominal maximum aggregate sizes, mixture performance evaluation, field evaluation of the mixture design procedure, field density requirements, and the accuracy of nuclear density gauges.

Volume IV, "Mixture Design Method, Construction Guidelines, and Quality Control/Quality Assurance Procedures," presents the deliverables of the study. Published herein, this Volume contains the final SMA mixture design procedure, guidelines for the construction of SMA, and QC/QA procedures to be used during construction. Volume IV also provides draft AASHTO standard test methods for tests developed during the course of the research, an example SMA mixture design using the final mixture design procedure, and an example of blending SMA gradations based on volumes.

Volume V, "Appendix for Phase II Work," provides detailed summaries of field validation projects and the data from Phase II experiments.

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

1.1 PROBLEM STATEMENT AND RESEARCH OBJECTIVE

Stone matrix asphalt (SMA) has been used in Europe for more than 20 years. Within the United States, SMA has been used successfully since 1991. Some states routinely use SMA, even though a standard mixture design procedure is not available. If a good mixture design procedure is adopted, it should be possible to improve the performance of SMA. A procedure that provides guidance on material properties, aggregate gradation, determination of optimum asphalt content, and mixture properties is needed.

As a result of this need, the NCHRP contracted with the National Center for Asphalt Technology (NCAT) in 1994 to develop a mixture design procedure for SMA. The emphasis of this study was to develop a straightforward procedure for SMA that is repeatable and that can be used by any agency or laboratory that is equipped to do hot mix asphalt (HMA) testing. This design procedure was envisioned as one method with two separate compaction options. The design can be accomplished with either a flat-faced, static base Marshall hammer or a Superpave gyratory compactor (SGC).

1.2 SCOPE OF STUDY

In order to achieve the overall goal of developing a quality mixture design procedure for SMA, two distinct phases of research were performed. Phase I-Part 1 began in April 1994 and had as its main goal the development of a tentative SMA mixture design method. This was done by evaluating critical material and mixture properties of SMA and choosing laboratory tests to evaluate the selected material and mixture properties. Phase I-Part 2 began in early 1995 on the basis of the results of Phase I-Part 1. The main objectives of this part were to evaluate and modify the tentative mixture design procedure and to analyze SMA mixtures with the revised method to make any necessary refinements.

Phase II of the research began in 1996 and was performed to field validate the proposed mixture design procedure established in Phase I research. Additionally, Phase II developed QC/QA and construction guidelines for the construction of SMA.

1.3 RESEARCH APPROACH

The research approach for this project involved 15 different tasks. Tasks 1 through 4 were accomplished under Phase I-Part 1; Tasks 5 through 7 were accomplished under Phase I-Part 2; and Tasks 8 through 15 were performed under Phase II. Following are brief overviews of the different tasks.

Task 1—State of the Art

Task 1 summarized the available information developed for SMA. Current mix design procedures, specifications, and construction procedures were obtained from all available sources, particularly published literature.

Task 2—Critical Material and Mixture Properties

This task evaluated aggregate, mortar, and mixture properties. The important aggregate properties such as Los Angeles (L.A.) abrasion, flat and elongated particles, absorption, soundness, and crushed content, and their specifications were studied. The effects of aggregate type, shape, and gradation on SMA mixtures also were studied.

Mortar testing under this task essentially looked at the applicability of the Superpave binder equipment and their testing procedures for evaluating SMA mortars. Both the total mortar (material passing 2.36-mm sieve, asphalt cement, and stabilizing additive) and fine mortars (material passing the 0.075-mm sieve, asphalt cement, and stabilizing additive) were evaluated. The total mortars were considered to be more like a mixture and were tested at low, intermediate, and high temperatures using the bending beam rheometer, resilient modulus, indirect tensile test, and the Brookfield viscometer. The fine mortars were considered to be binders and were tested with the Superpave binder equipment and test procedures.

Evaluation of mixture properties consisted of determining the volumetric properties, moisture susceptibility, creep properties, and draindown characteristics of SMA mixtures.

Task 3—Selection of Laboratory Tests

This task was performed to determine what laboratory tests could be used to evaluate SMA mixtures. Testing was

conducted for the aggregate skeleton, additional mortar testing, and SMA mixture testing. The aggregate skeleton testing consisted of finding a method for identifying when stone-on-stone contact is achieved within an SMA mixture.

Mortar testing under this task was again conducted on both the total and fine mortars. The primary purpose was to determine the potential of the tests listed under Task 2 to measure desirable properties of SMA mortars.

The SMA mixture testing evaluated the effect of changing volumetric properties on SMA. A dynamic, confined creep test was used for this evaluation.

Task 4—Interim Report

The requirement of Task 4 was to submit an interim report summarizing the work in Tasks 1 through 3. This report included a tentative mixture design procedure for SMA.

Task 5—Mixture Design Procedure

The main goal of Task 5 was to finalize the mixture design procedure proposed under Tasks 1 through 4. This was accomplished by adapting the Superpave volumetric design procedure for SMA, establishing a method for determining the proper laboratory mixing and compaction temperature for SMA, establishing fine mortar specification criteria, and further evaluating the design procedure.

Task 6—Mixture Analysis

Task 6 was completed to analyze the properties of mixtures produced using the design method established under Task 5. Under this task, testing included the Indirect Tensile Creep and wheel tracking.

Task 7—Final Report

The requirement of this task was to produce a final report documenting the research effort under Phase I of the project. An updated mixture design method was provided in this report.

Task 8—Field Evaluation of SMA Mix Design Procedures

This task collected actual field data from various SMA construction projects throughout the United States. For this phase, a mobile laboratory was used to develop much of the data. Within the mobile laboratory was equipment to compact specimens using both the Marshall hammer and the SGC, determine the maximum specific gravity of the SMA mixtures, and determine the draindown characteristics of the SMA mixtures.

Task 9—Develop QC/QA Procedures for SMA Mixtures

During this task, information from several states involved with SMA construction along with information developed during Task 8 were used to develop the QC/QA procedures. The QC/QA procedures also included a trouble-shooting guide to aid in determining the specific cause of potential problems.

Task 10—Laboratory Testing

Testing included in this task was designed to extend some of the results produced in Phase I of the research. Specific testing focused on various maximum aggregate sizes, determination of the effects of flat or elongated particles using the SGC, and direct tension testing of SMA fine mortars.

Task 11—Develop SMA Construction Guidelines

The purpose of this task was to develop SMA construction guidelines to assist producers during the production and placement of SMA mixtures. The guidelines included guidance on aggregate handling, fiber handling, plant operations, laydown operations, and compaction. These guidelines were developed on the basis of staff experiences from NCAT, state DOTs, the FHWA, and contractors, as well as information gathered during Task 8.

Task 12—Verify Laboratory Mixing and Compaction Temperatures

This task was accomplished to further the research effort of Task 5 with regards to establishing a procedure for determining the mixing and compaction temperatures for SMA mixtures. Testing was conducted on SMA fine mortars (minus fibers) in the Brookfield viscometer and on SMA mixtures using a workability device developed during the study.

Task 13—Verify Density Requirements

At the present time, most SMA projects are being compacted to in-place air void contents of 5 to 6 percent. This appears to be acceptable, but there is some evidence that indicates that SMA mixtures are permeable to water at a lower air void content than are dense-graded mixtures. Task 13

investigated this possibility using laboratory and field permeability tests.

Task 14—Accuracy and Precision of Nuclear Gauge for Determining Field Density

By nature, SMA mixtures have a rough surface texture. As a result, the use of nuclear density gauges to determine in-place density is questionable. Therefore, this task evaluated the accuracy of nuclear gauges for determining the density of SMA mixtures.

Task 15—Preparation of Reports

The requirement of this task was to produce a final report documenting the total research effort. Included within this report were the final mixture design procedure, the QC/QA procedures, and the guidelines for SMA construction.

CHAPTER 2

RESEARCH FINDINGS

2.1 LITERATURE REVIEW

Known by several similar names, such as Splittmastixasphalt in Europe where the material was first developed and Stone Matrix Asphalt when it was adopted in the United States, this durable, deformation-resisting mix has become known generically by its initials SMA. Stone Matrix Asphalt is, in fact, not a particularly good translation of the German word as it fails to convey the sense of crushed stone, which is an essential feature of the mix. However, the convenient SMA form of the name will be used in this review.

SMA is a simple idea: find a hard, durable, quality stone; fracture it into roughly cubical shape and of a size consistent with the proposed layer thickness; and then glue the stones together with a durable, moisture-resistant mortar of just the right quantity to give stone-on-stone contact among the coarse aggregate particles. For the asphalt technologist, the skill is getting the various parameters right. This is where the recipes come in. Most European specifications are reflections of mix ingredients and proportions that seem to work quite well. Many specifications are based on Germany's work (*10*).

It was not until about 1990 that interest in SMA was sparked in North America, following a study tour (*26*) of five European countries by a group of pavement specialists from the United States. The favorable reaction of the team to SMA soon caught the attention of the trade press and several articles on SMA followed, especially after SMA projects were constructed first in Canada in late 1990 (*24*) and then in several U.S. states in 1991. Technical reports and published papers of substance relating to North American experience began to appear in late 1991.

In 1990, news about SMA began to circulate through exchanges of technical information between local technologists and Europeans. During the European study tour (*26*), U.S. pavement specialists had the opportunity to see SMA in action and hear about it through presentations such as the one by Liljedhal (*8*). Armed with European experience and specifications (*1, 2, 4, 5, 6, 7, 9, 10, 11, 12*), decisionmakers in the United States deemed SMA worthy of trials. The first project was in Wisconsin and was reported in the trade press by Scherocman (*14*).

Gradations appear extensively throughout the papers reviewed. In the first project in the United States, Scherocman (*14*) noted that the gradation followed the "30-20-10 rule," which is a useful guideline for specifiers and designers of SMA who are lacking model procedures. The rule indicates that the aggregate gradation should have 30 percent passing the 4.75-mm (#4) sieve; 20 percent passing the 2.36-mm (#8) sieve; and 10 percent passing the 0.075-mm (#200) sieve.

Other projects followed the one in Wisconsin and were also reported in the trade press before more detailed reports were published by research bodies or state highway departments (*21, 22, 23, 25, 27, 28,* and *29*). To help potential users of SMA, Bukowski (*20*) produced the FHWA's work plan, which contained a summary of the first five SMA projects that had been built in the United States in 1991.

Several reports in 1991 and 1992 dealt with the U.S. SMA projects. Little (*16*), Brown (*40*), and Stuart (*35*) all performed comparative mix designs for Georgia DOT, which also produced a report (*31*) and supplemental specifications (*32*). Missouri's experience was reported by McDaniel (*34*). Scherocman (*46, 50*) and Brown (*40, 48*) reported on others, in addition to several items in the trade press. Brown et al. (*97*) reported an evaluation of more than 85 SMA projects. The practical problem of producing an SMA with cellulose fibers in a drum mixer without losing fibers in the exhaust gas dust collection system was addressed by Karnemaat, Vreibel, and Van Deusen (*54*) early in 1993 but this work was based on experience with the earlier Michigan projects.

Carpenter (*77*); Brown and Mallick (*84*); and Brown and Haddock (*99*) have addressed mixture volumetrics and achievement of the stone-on-stone contact that SMA is supposed to provide in service. Brown and Mallick (*84*) and Brown et al. (*98*) suggest a design procedure that is intended to answer this problem.

Brown et al. (*95*) performed a study in 1997 stating that the amount of filler finer than 20 μm did not matter. They based this conclusion on mortar characterizations using the Superpave binder equipment and procedures.

According to Bellin (*41*), type of filler is not named in specifications or requests for proposals in Germany. He indicates that ground limestone is used and that baghouse fines are not used much. Scherocman (*46*) noted that baghouse fines are not normally returned into the SMA production line in Europe.

One of the questions facing mix designers is how to deal with the design of the mortar. Milster (*53*), in a telling photograph, illustrates the difference in stiffening effects between the mortar fractions of an SMA and a dense-graded

mix. Bellin (*41*) reported that the Schellenberg draindown test is used in Germany to ensure that the mix does not lose its cementing mortar. Details of the test are reported by Stuart (*35*). In the United Kingdom, Walsh (*70*) referenced another test, while Brown and Mallick (*84*) proposed a test method, which has been accepted in the FHWA's Model Material and Construction Guidelines (*81*).

Brown et al. (*98*) looked at the effect of different stabilizing additives on draindown. They concluded that fibers (mineral or organic) did a better job of preventing draindown than did polymers.

Harders (*67*) used the Ring and Ball softening point test in evaluating the effect of stabilizers. Richter (*19*) also used this test among others that are less familiar. Serfass and Samanos (*33*), and Schröder and Kluge (*51*) have also used the softening point test. According to Schröder and Kluge, mortar compositions with softening points between 85° and 100°C (185°–212°F) provide the right viscosity and impact strength. Sometimes fibers and polymers are combined, as noted by Reinke (*59*) and Fujita (*86*); and a multigrade asphalt has also been used as reported in the trade press (*56*) and by Kriech and Haddock et al. (*57, 61*).

A review of the literature indicates that there are no clearly defined approaches to SMA mix design, nor are there performance-related mix tests that designers and practitioners can embrace with confidence. Despite these apparent drawbacks, SMAs in the United States appear to be living up to their reputation as a rut-resisting mix.

2.2 MATERIAL PROPERTIES

2.2.1 Aggregate Properties

Several different types of aggregates were used during the study and included traprock; granite (two different sources); limestone (two different sources); Florida limestone (sometimes called limerock); blast furnace slag; and siliceous gravel. These aggregates were selected because they provided a range of qualities from excellent to marginal.

The coarse aggregate for an SMA mixture must be sufficiently strong to carry the imposed loads because these mixtures are designed to have stone-on-stone contact. Presently, the aggregate test that is commonly used for determining the toughness of the aggregate particles is the L.A. abrasion test. On the basis of data accumulated during the study, coarse aggregates used within SMA should have L.A. abrasion values of less than 30 percent loss. SMA mixtures using aggregates with higher L.A. abrasion values have been used successfully but care must be used when using these softer aggregates. Higher abrasion values resulted in excessive aggregate breakdown during laboratory compaction.

Because the coarse aggregate must carry the load, fractured faces are required to provide a coarse aggregate structure with high internal friction. If the fractured face count is significantly less than 100 percent for the coarse aggregate, the SMA mixture is almost certainly going to be less resistant to shoving and rutting. This requirement for fractured faces eliminates the use of uncrushed gravels and will also eliminate many crushed gravels if the fractured face count is not high enough to meet the desired requirements.

The amount of aggregate absorption can affect the performance of SMA mixtures just as it does dense graded mixtures. Low absorptive aggregates are preferred, but some locations, due to availability of aggregates, may be required to use aggregates with higher absorption values. When high-absorptive aggregates are used, the optimum asphalt content should be selected on the high side but within the specification limits to allow for some additional absorption of the asphalt cement during placement and during the life of the pavement.

The angularity of fine aggregates was measured during the project using the National Aggregate Association (NAA) flow test (AASHTO TP33, Method A). The test results ranged from 44.6 to 49.9 percent. Based on the testing conducted, it was recommended that a value of 45 seemed reasonable for the fine aggregate requirement.

2.2.2 Mineral Filler Properties

Eleven different mineral fillers were used during the course of this project: limestone dust, marble dust, traprock dust, southeastern flyash, Georgia flyash, aglime, diabase, wimpey, Dankalk, Oyta, and Faxekalk. These mineral fillers were selected because they cover the range of mineral fillers that have been used within SMA. Characterization testing conducted to evaluate the different fillers was particle size distributions, the modified Rigden's voids test, and surface area measurements.

Fine mortar (mineral filler, stabilizing additives, and asphalt cement) testing was used to evaluate the influence of the different mineral fillers on SMA mixtures. During the fine mortar testing, the influence of the amount of mineral filler passing the 0.02-mm sieve, the modified Rigden's voids, and the surface area of the fillers were all evaluated. Based on this testing, it was determined that the amount of material passing the 0.02-mm sieve did not influence the mortar properties, the modified Rigden's voids correlated well with mortar testing results, and the surface area did not influence mortar properties.

It was decided that a good screening test for fillers is to ensure the percent voids do not exceed 50 when measured with the Rigden's voids. Fillers that exceed 50 cause the mortar to be excessively stiff and difficult to work.

2.2.3 Asphalt Binder Properties

The effect of different asphalt binders was not addressed in this study. However, three different binder types were

used: AC-20; AC-20M1 (SBS modified); and AC-20M2 (polyolefin modified). The three binder performance grades (PG) were 64-22 for the non-modified, 70-28 for the SBS modified, and 70-22 for the polyolefin modified.

On the basis of the information obtained during the study, the asphalt cement for SMA mixtures should be graded using the Superpave binder grading system (PG graded). The asphalt cement should be selected based on the climate in which it will be used. Guidance for adjusting the performance grade of asphalt binders for either traffic speed or traffic level is provided in AASHTO MP2, "Standard Specification for SUPERPAVE Volumetric Mix Design," Section 5.5.

2.2.3.1 Effects of High-Temperature Testing

Although the type of asphalt binder was not a major part of the study, a subtask of Task 11 was to evaluate the effect of high temperatures on asphalt binders. This subtask was included because high temperatures are sometimes needed for SMA, especially on occasions when polymer additives are used.

Testing to determine the effects of high temperatures on asphalt binders was done by aging six different asphalt binders (three modified and three unmodified) using six different temperatures in a thin film oven. For each set of aged asphalts, the Superpave binder tests were conducted to determine the properties after aging.

Based on this testing, it was shown that for each of the Superpave binder tests (except the direct tension), at a temperature between 180° and 195°C (356°–382°F), a significant change in the test results was exhibited. Therefore it was recommended that SMA mixtures not be heated to temperatures above 176°C (350°F).

2.2.4 Stabilizing Additives

Three major stabilizing additive types were used during the laboratory study of this project: cellulose fibers, mineral fibers (slag wool and rock wool), and polymers (SBS and polyolefin). Testing performed to evaluate the different stabilizing additives was the testing of the fine mortars using the Superpave binder equipment and test procedures. For the study, cellulose was added at 0.3 percent of total mixture mass and mineral fibers were added at 0.4 percent. Several different types and percentages of polymer were used. This testing indicated that at higher temperatures the mineral fibers stiffen the asphalt binders the least followed by cellulose fiber, SBS, and the polyolefin. At intermediate temperatures, both the cellulose and mineral fibers stiffen about the same or slightly more than no stabilizer. The polyolefin is less stiff than the fibers but slightly more stiff than the SBS stabilizer. Testing at low temperatures showed that the mineral fibers stiffened the asphalt binders the most. The cellulose and polyolefin stabilized mortars have about the same stiffness and the SBS stabilized mortars were the least stiff.

2.3 AGGREGATE SKELETON TESTING

Because one objective in the design of SMA is to produce a mixture with stone-on-stone contact, it was essential that a method be available to define this condition within the coarse aggregate fraction. Therefore, work was performed to evaluate the ability of five different methods to determine when stone-on-stone contact exists in the coarse aggregate fraction. The five methods used to determine the voids in coarse aggregate (VCA) included the Marshall hammer, dry-rodded test, Superpave gyratory compactor (SGC), vibrating table, and Kango (vibrating) hammer. A summary of VCA testing results is presented in Table 2.1.

TABLE 2.1 Void in coarse aggregate test results for the coarse aggregate function only

Method		Granite	FL Limestone	Gravel	Limestone	Traprock
Marshall Hammer	Avg	32.1	26.9	33.4	29.9	35.6
	SD	0.55	0.50	0.82	0.45	0.85
SGC	Avg	34.7	30.3	37.2	36.6	39.2
	SD	1.30	0.84	0.40	0.25	0.42
Dry-Rodded	Avg	39.2	38.9	37.4	42.5	40.8
	SD	0.12	0.9	0.17	0.30	0.10
Vibrating Table	Avg	40.0	39.8	37.8	41.6	40.8
	SD	0.21	0.80	0.15	0.06	0.21
Kango Hammer	Avg	47.5	44.1	45.4	47.5	48.6
	SD	0.61	0.35	0.4	0.26	0.60

Avg - Average SD - Standard Deviation

Based on the results of aggregate skeleton testing, it was decided that the dry-rodded condition was the easiest of the methods to use and provided reasonable results. It appeared to be the most appropriate method for determining the VCA at which stone-on-stone contact occurs.

2.4 MORTAR TESTING

Mortar testing was conducted in both main phases of the research project. During Phase I, specification criteria were recommended for SMA mortars. Mortar testing during Phase II dealt with evaluating the specification criteria.

To attempt to fully understand the mortar characteristics, SMA mortars were first tested in two fractions during Phase I: the total and fine mortars. The total mortar fraction contained that portion of aggregate passing the 2.36-mm sieve, stabilizing additive(s), and asphalt cement. Because of the relatively coarser aggregates in the total mortar, it was considered to be similar to a mixture. Therefore, it was tested at low, intermediate, and high temperatures using the bending beam rheometer (BBR), resilient modulus, indirect tensile test, and Brookfield viscometer (BV). The fine mortar fraction contained that portion of aggregate passing the 0.075-mm sieve (typically referred to as filler), stabilizing additive(s), and asphalt cement. Being of a finer nature, the fine mortar was considered a binder and was tested at low, intermediate, and high temperatures using the Superpave binder equipment before and after aging.

Based on the testing conducted on the two types of mortars, it was determined that the fine mortar properties were closely related to the total mortar properties (Figures 2.1 through 2.3). Based on this preliminary testing, it appeared that the fine mortar could be evaluated without the need for additional testing on the total mortar since these properties were related. During mix designs it would be much easier to prepare and test fine mortars than total mortars, so it was decided, therefore, to continue testing on the fine mortars.

In order to characterize the influences of the different constituents of a SMA fine mortar, the test plan called for varying the type of filler, percent of filler, and type of stabilizing additives. The testing of these different mortars was conducted at high, intermediate, and low temperatures. The results of the testing are summarized in Table 2.2. This table shows whether significant trends were found in the test results for the high-, intermediate-, and low-temperature testing.

Based on these test results, it was concluded that the dynamic shear rheometer (DSR), bending beam rheometer (BBR), direct tensile test (DTT), and Brookfield viscometer (BV) could be used to characterize fine mortars. After comparing the data, it appeared that the mortars were typically at least 5 times stiffer than the asphalt binders. Therefore, a good starting point for writing a fine mortar specification was to specify the following:

Test	Test Method	Property	Specification
Original Mortars			
Dynamic Shear	TP 5	G*/Sinδ	5.0 kPa min.
RTFO Aged Mortars			
Dynamic Shear	TP 5	G*/Sinδ	11.0 kPa min.
RTFO and PAV Aged Mortars			
Bending Beam	TP 1	S	1500 MPa max.

Testing during Phase II was conducted on fine mortars from the 11 field projects visited during Task 8 to determine the

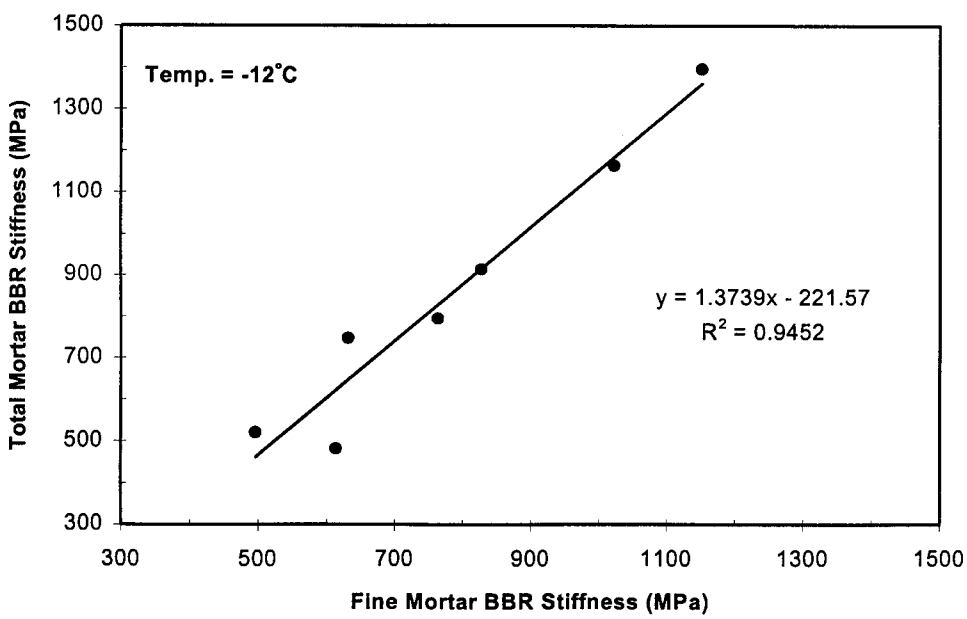

Figure 2.1. Relationship between total mortar stiffness and fine mortar stiffness at −12°C.

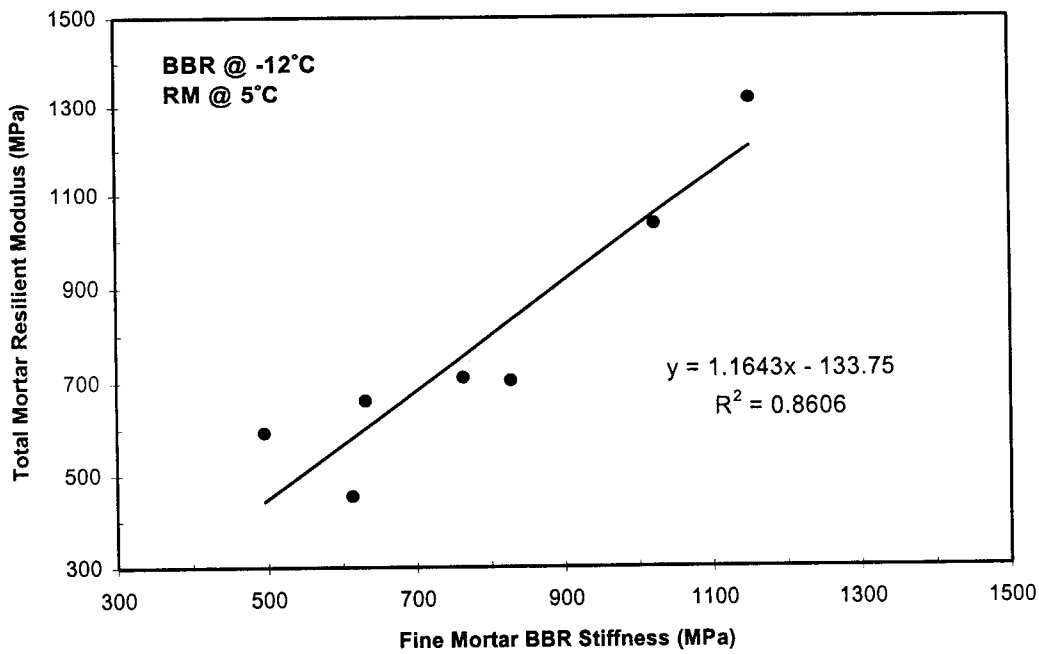

Figure 2.2. Relationship between total mortar resilient modulus at 5°C and fine mortar stiffness at −12°C.

applicability of these fine mortar specifications. For each of the 11 projects visited, fine mortars were fabricated at the NCAT laboratory to meet the job-mix-formula (JMF) percentages and tested using the Superpave binder equipment and testing protocols. Results of this testing indicated that the fine mortar specifications seem to be applicable. Figures 2.4 through 2.6 present the results of this testing in relation to the specifications.

These requirements provide a minimum stiffness of the fine mortar at high temperatures (original and RTFO DSR testing) to promote a stiff mortar to help in preventing permanent deformation. The requirements also address the cold temperature properties of the fine mortars (BBR testing) by limiting the creep stiffness (S) of the fine mortar.

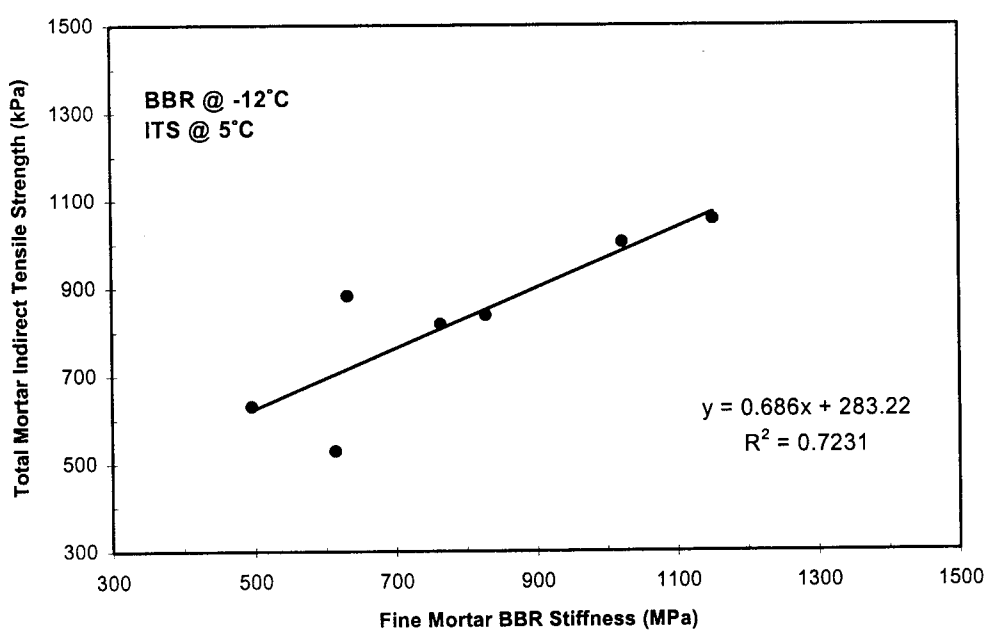

Figure 2.3. Relationship between total mortar tensile strength at 5°C and fine mortar stiffness at −12°C.

TABLE 2.2 Summary of fine mortar testing to characterize SMA mortars

Property	Significant Trend?		
	High-Temperature Testing	Intermediate-Temperature Testing	Low-Temperature Testing
Filler Type	No	Yes	No
Filler Percent	Yes	Yes	Yes
Stabilizer Type	Yes	Yes	Yes
Filler Particle Sizes	Yes	No	No
Filler Particle Shape	No	No	No
Filler Surface Area	Yes	Yes	No

2.5 MIXTURE DESIGN ANALYSIS

Early in Phase I, several aggregates were evaluated to determine the effect of aggregate type and gradation on SMA mixture properties. During this part of the study, mixture designs were performed with 50 blows of a flat-faced, static base Marshall hammer. The mix designs were conducted using several trial blends that varied the amount of material passing the 4.75-mm sieve until the VMA was 17 percent or higher and until stone-on-stone contact was achieved. The gradation for each mixture was then fixed and asphalt binder added to determine the optimum asphalt content.

The results of these mix designs were used to show that the concept of varying the percent passing the 4.75-mm sieve to obtain the minimum VMA requirement and the maximum VCA is viable. Figures 2.7 and 2.8 present a summary for the different mixtures when the percent passing the 4.75-mm sieve was varied. Figure 2.7 shows that as the percent passing the 4.75-mm sieve is decreased, VMA increases. Figure 2.8 illustrates that increases in the percent passing the 4.75-mm sieve likewise increased the VCA of the SMA mixtures.

The moisture susceptibility of each of the optimum mixtures as well as companion dense-graded mixtures were tested using AASHTO T283. This test procedure was modified slightly for the SMA mixtures in that they were compacted to an air void content of 6±1 percent. This air void content is more indicative of desired SMA pavement density

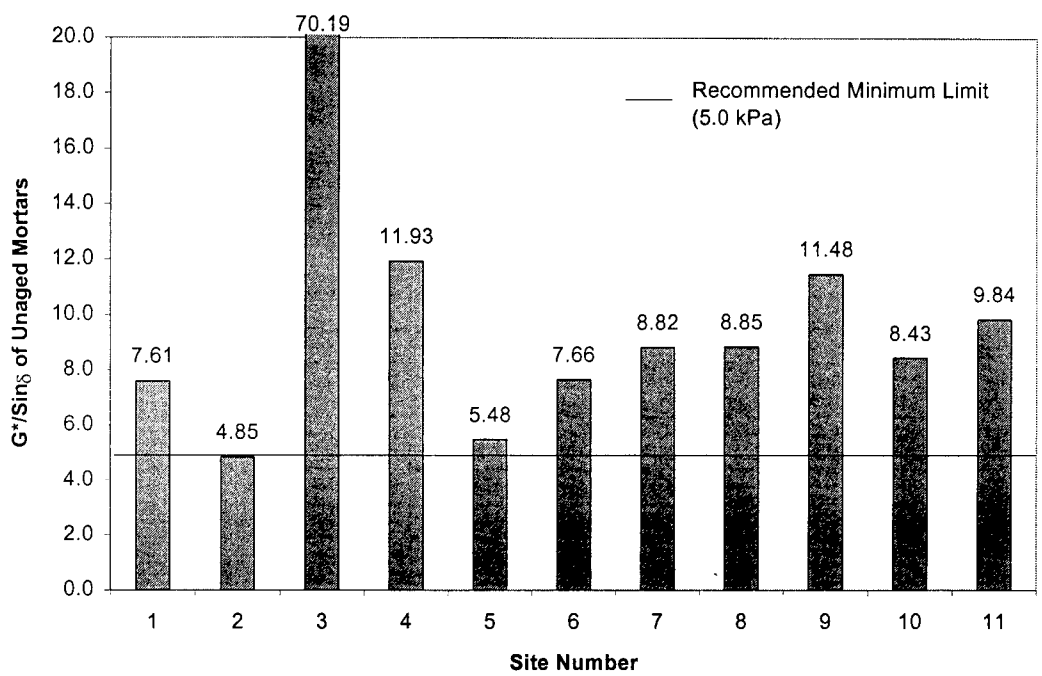

Figure 2.4. Results of project high-temperature fine mortar testing with the DSR on original fine mortars.

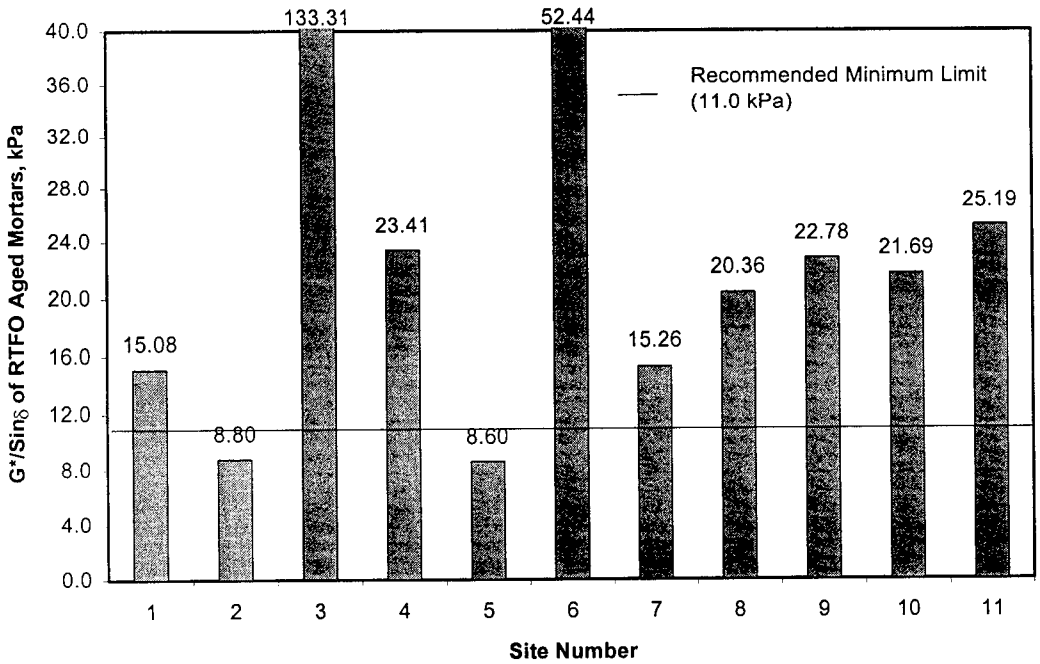

Figure 2.5. Results of project high-temperature fine mortar testing with the DSR on RTFO aged mortars.

at the time of construction. Also, no freeze-thaw cycle was used. Results of the moisture susceptibility testing indicated that for the testing conditions used, AASHTO T283 is applicable for SMA mixtures generally providing similar results as dense-graded mixtures.

Work during Phase I using a number of aggregates and mortars was conducted in the laboratory to validate the tentative mix design procedure developed during the first part of Phase I. Based on this work, some problems were encountered. There was a significant amount of aggregate breakdown that

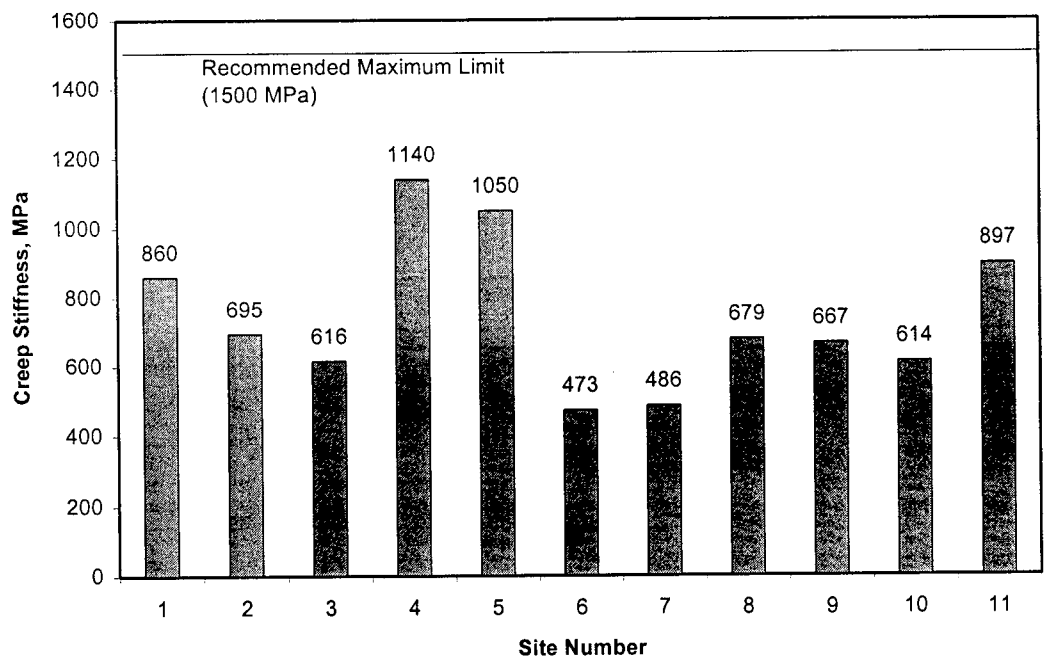

Figure 2.6. Results of project low-temperature testing with the BBR on PAV aged mortars (creep stiffness).

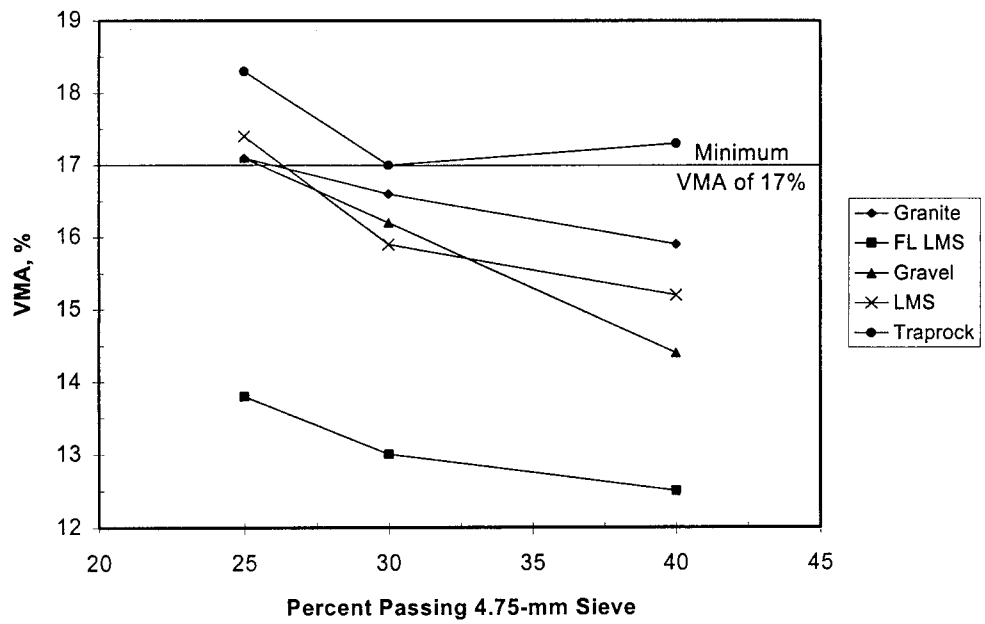

Figure 2.7. Summary of VMA versus percent passing 4.75-mm sieve.

occurred for some of the aggregates. As a result of this breakdown, VMA requirements were not always met. However, for the aggregates that did not experience excessive breakdown, VMA requirements were easily met. Even with sufficient VMA values, the optimum recommended asphalt content by weight was sometimes less than the recommended minimum of 6 percent. This was seen for aggregates that had high specific gravities. It was suggested that the best approach was to specify a minimum VMA (17 percent) and not to be concerned about the asphalt content by weight.

Also during Phase I of the study, the volumetrics for SMA were compared between 50 blows of the Marshall hammer and 100 gyrations of the SGC (Figure 2.9). On the basis of the data, a good correlation existed between the two compactive efforts. It appeared that in the range of lower optimum asphalt contents, the Marshall hammer tended to give a higher optimum asphalt content for a given aggregate type and gradation. Conversely, in the range of higher optimum asphalt contents, the SGC produced higher optimum asphalt contents for a given aggregate type and gradation. (Results

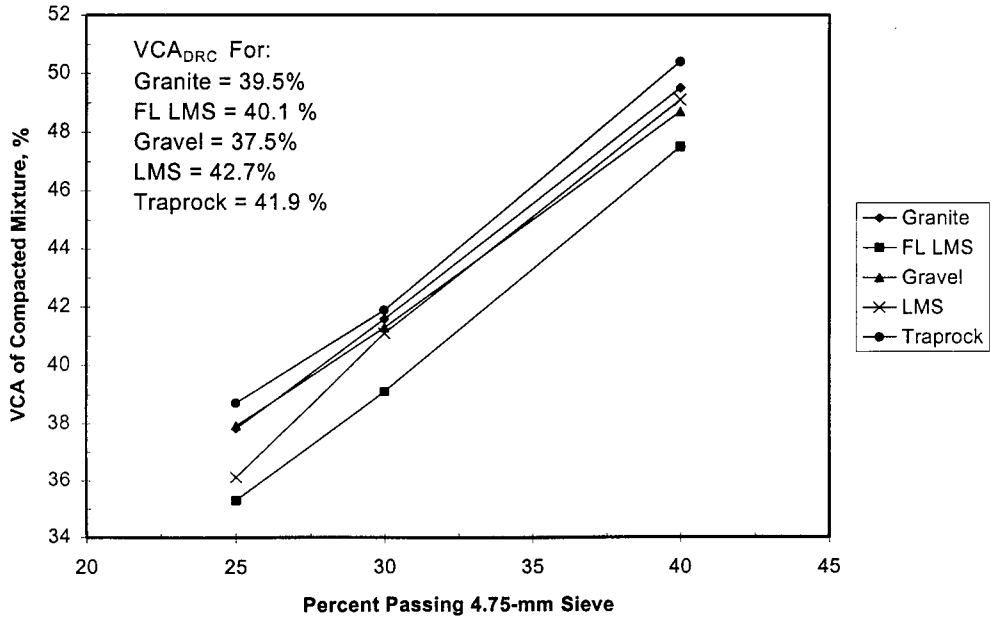

Figure 2.8. Summary of SMA mixture VCA versus percent passing 4.75-mm sieve.

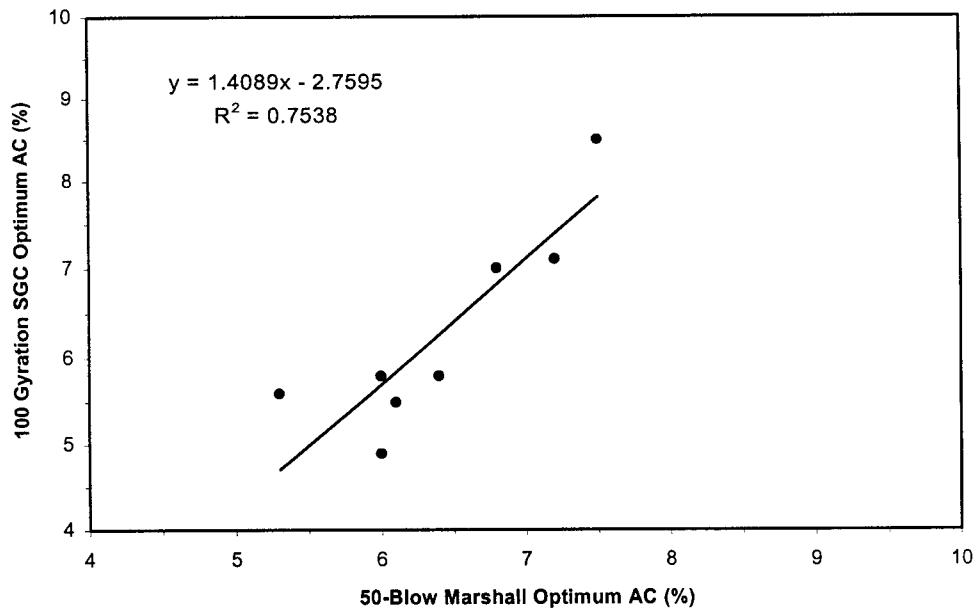

Figure 2.9. Correlation between the Marshall hammer and SGC optimum asphalt contents.

of later work during Phase II indicated that approximately 80 gyrations produced a similar density to 50 blows of the Marshall Hammer. See Section 2.12.2 of this volume.)

2.6 FLAT AND ELONGATED PARTICLES

The effect of flat and elongated particles within SMA was evaluated during both phases of the project. Work in both phases was similar with one major difference; Phase I looked at the influence of flat and elongated particles using 50 blows of the Marshall hammer while Phase II used the SGC.

For both phases of the study, a supply of aggregate was used that was quarried from the same ledge with one portion of the aggregate being crushed to produce a high percentage of flat and elongated particles and another portion crushed to produce a lower percentage. SMA mix designs using the proposed mix design procedure were accomplished with the Marshall hammer on gradations of varying percentages of the two aggregate types. Using the optimum asphalt content determined during the mix designs, specimens were compacted with both 50 blows of the Marshall hammer and 100 gyrations of the SGC to evaluate the effect of flat and elongated particles on volumetric properties. This evaluation showed that the VMA did increase as the amount of flat and elongated particles was increased (Figure 2.10).

Next, specimens were compacted with both compactors using a single gradation to evaluate the effect of flat and elongated particles on aggregate breakdown. For both compactors, as the percentage of flat and elongated particles increased, the amount of aggregate breakdown also increased (Figure 2.11).

It was thought that increasing percentages of flat and elongated particles would increase the moisture susceptibility of SMA mixes. Specimens were compacted with both compactors to evaluate the effect of flat and elongated particles on moisture susceptibility. Results of this analysis showed that flat and elongated particles had no effect on moisture susceptibility for either compactor.

On the basis of flat and elongated particles work in both phases, it was concluded that some amount of flat and elongated particles could be used without any detrimental effect on mixture properties. However, after some point was reached, the negative effect became more severe. It appears that the current criteria of no more than 20 percent 3 to 1 and no more than 5 percent 5 to 1 flat and elongated particles is satisfactory. Values in excess of these criteria were shown to lead to excessive aggregate breakdown in compacted samples.

2.7 AGGREGATE BREAKDOWN ANALYSIS

Aggregate breakdown occurs when SMA mixtures are compacted in the laboratory and field. The amount of breakdown is a function of hardness and the method of compaction. It is important that the amount of breakdown occurring in the laboratory be approximately equal to that which occurs in the field. If there is a significant difference in the breakdown in the laboratory and in the field, the mix design method is suspect because the volumetrics in the laboratory compacted samples may have no relationship to the volumetrics in the field.

Aggregate breakdown was evaluated during both phases of the study. During Phase I, laboratory breakdown was evaluated using eight different aggregates. This laboratory evaluation

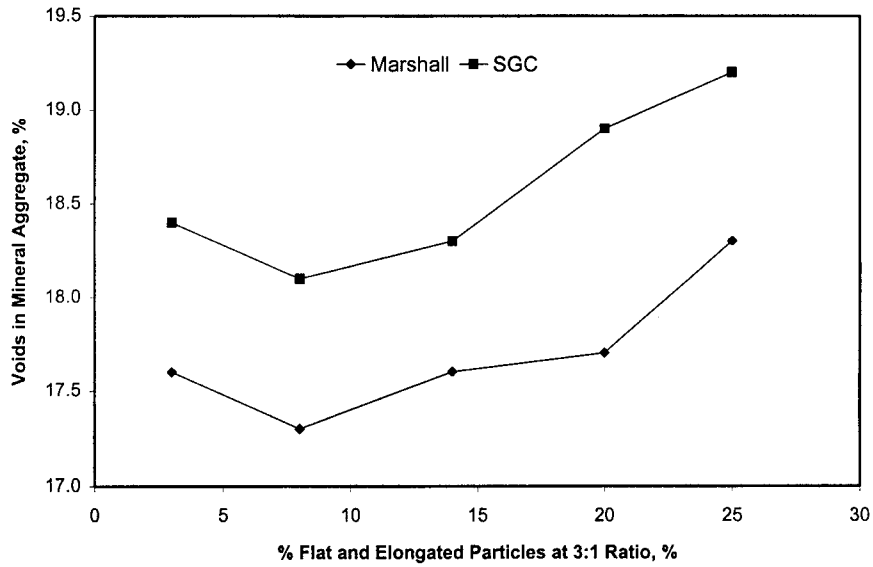

Figure 2.10. Effect of flat and elongated particles on VMA.

showed that for most of the aggregates used, there was very little difference in the breakdown with 50 blows of the Marshall hammer and 100 gyrations of the SGC. For some of the aggregates, however, the difference was significant. The Marshall hammer always gave as much or more breakdown than the SGC.

During this Phase I investigation, a good correlation existed between the amount of aggregate breakdown on the 4.75-mm sieve and the L.A. abrasion values for both the Marshall hammer and the SGC (Figures 2.12 and 2.13, respectively). For both compaction methods, higher abrasion values correlated to higher breakdown values. The data indicated that the expected breakdown on the 4.75-mm sieve at an L.A. abrasion value of 30 is approximately 10 percent.

During Phase II of the project, field-produced SMA mixtures from 11 projects were compacted with both 50 blows of the Marshall hammer and 100 gyrations of the SGC. In addition to these laboratory compacted specimens, field compacted (rollers) samples were obtained for eight of the 11 projects visited. Analysis of the data showed that there were significant differences in mixture gradation before and after compaction. Analysis of this data also showed that the Marshall hammer consistently resulted in more breakdown than the SGC. However, the data also showed that the breakdown underneath rollers was approximately equal to that observed with both laboratory compactors (Table 2.3).

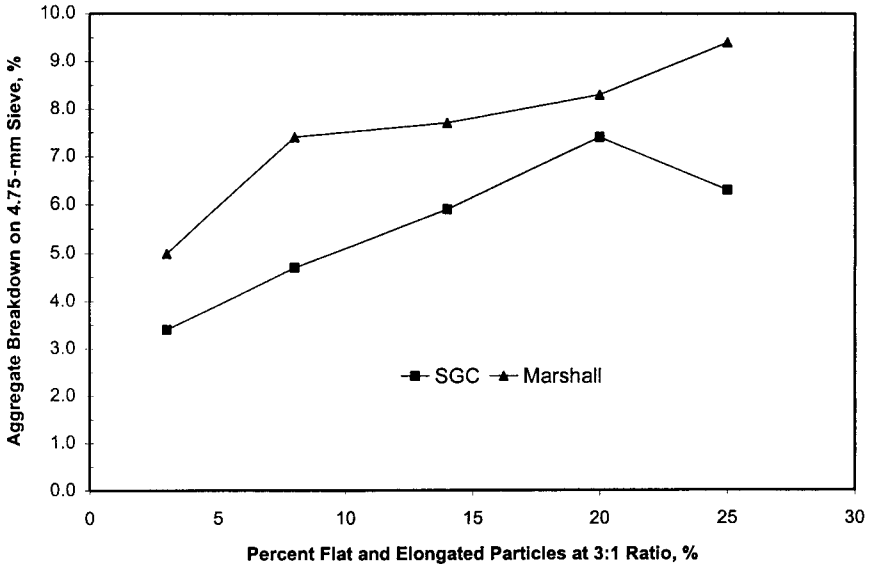

Figure 2.11. Effect of flat and elongated particles on aggregate breakdown.

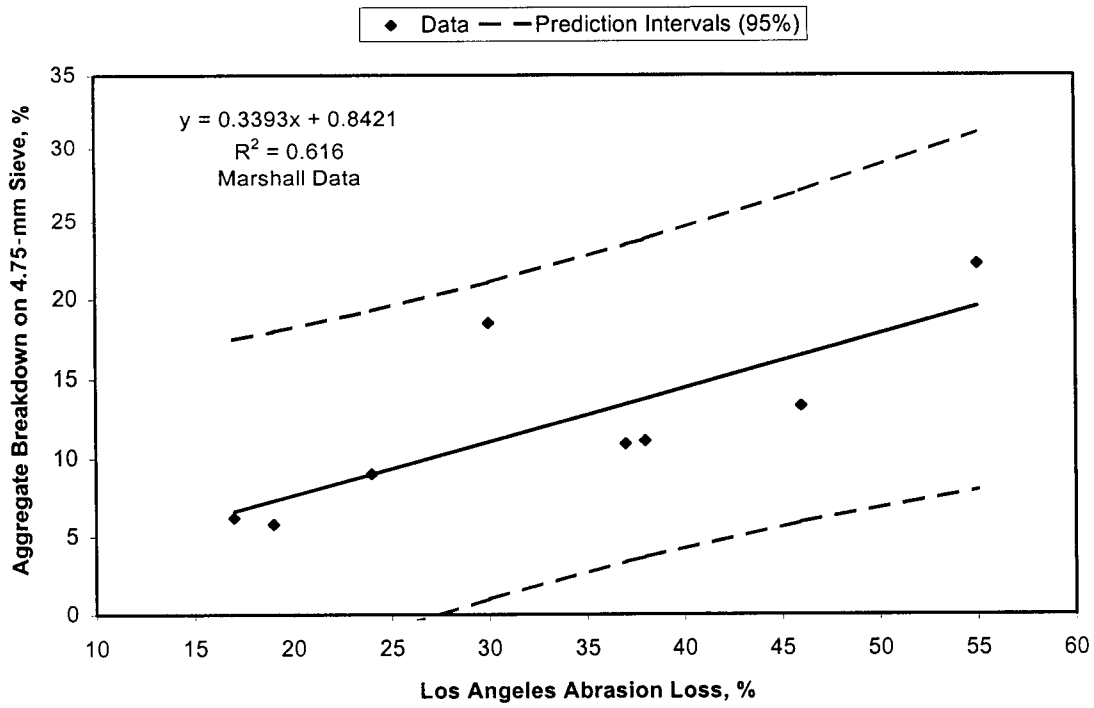

Figure 2.12. Aggregate breakdown caused by Marshall hammer as a function of L.A. abrasion loss.

2.8 MIXING AND COMPACTION TEMPERATURES

The mixture design procedure developed during Phase I specified that mixing and compaction temperatures be determined in accordance with AASHTO T245. This method is a temperature-viscosity relationship in which the mixing and compaction temperatures are selected on the basis of viscosity ranges. Because of the high filler contents and the frequent use of polymer modified asphalt cements, it was thought that this method of determining mixing and compaction may not be the best.

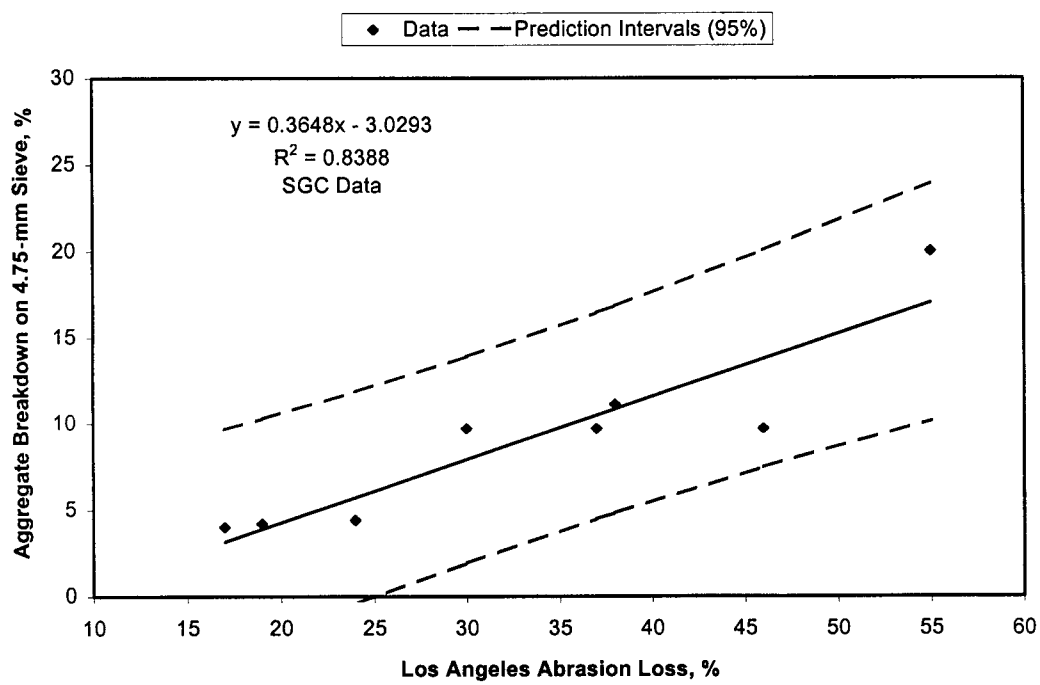

Figure 2.13. Aggregate breakdown caused by SGC as a function of L.A. abrasion loss.

TABLE 2.3 Results of ANOVA performed to compare aggregate beakdown

Site No.	Mean Percent Passing 4.75 - mm Sieve			F-stat	Probability > F	Significant Difference?
	SGC	Marshall	Field Cores			
2	32.5	35.4	36.0	4.57	0.030	Yes
3	35.5	36.3	40.6	20.98	0.000	Yes
4	33.9	36.2	33.0	2.87	0.084	No
5	37.3	37.9	36.5	1.35	0.286	No
6	31.6	33.5	33.9	3.96	0.051	No
9	34.2	38.4	33.5	16.19	0.000	Yes
10	31.5	34.0	33.4	1.19	0.328	No
11	32.7	35.1	33.4	7.97	0.004	Yes

Therefore, two different methods were evaluated to determine if they provided more reasonable results. The first method entailed combining the material finer than 0.075 mm from the design gradation and the asphalt binders (no fibers were included) for each of the eleven projects visited under Task 8. These mortars were then tested in the Brookfield viscometer. The second method was a workability device that was developed to measure the stiffness of SMA mixtures. Using this device at different temperatures allowed the stiffness of the mixture in relation to the temperature to be determined.

The data indicated that both tests do a good job of measuring the stiffness of the mortar (BV) and the mix (workability). However, neither test appeared to provide a method of determining the mixing and compaction temperature. Therefore, it was concluded that the best approach for determining the mixing and compaction temperatures was AASHTO T245.

2.9 PERMEABILITY EVALUATION

Evaluation of the permeability characteristics of SMA was undertaken in an effort to establish whether SMA mixtures are more or less permeable to water than are conventional dense-graded mixtures. Using four aggregates, specimens were prepared at optimum asphalt content by varying the compactive efforts (both Marshall and SGC). These specimens were then tested in a laboratory falling-head permeameter to determine how the permeability varies within SMA as a function of air voids.

These data generally suggested that the permeability began to increase rapidly as the air voids increased from 6 to 7 percent. Therefore, SMA mixtures ideally should be compacted to around 94 percent of theoretical maximum density. Field permeability work during Phase II was in agreement that 6 percent air voids are needed.

2.10 EVALUATION OF NOMINAL MAXIMUM AGGREGATE SIZE

Most SMA mixtures that have been placed in the United States have been 12.5- or 19.0-mm nominal maximum size gradations. There is often a need to use a smaller or larger nominal maximum aggregate size. This part of the study evaluated the effect of nominal maximum aggregate size.

SMA mixtures typically have a large percentage of aggregate down to a certain sieve size after which the percentage of aggregate is small. The sieve size that separates the sieve containing large percentages from the sieves with small percentages is considered the breakpoint sieve.

This part of the study looked at 11 SMA mixes ranging in nominal maximum size from 4.75 mm up to 25 mm. A number of breakpoint sieves were also evaluated. For comparison purposes, four Superpave mixes were designed and tested (two above the restricted zone and two below the restricted zone).

All of the 11 proposed SMA mixes were successfully designed using the proposed mix design method. Both of the 4.75-mm nominal maximum aggregate size mixtures evaluated had to be designed based on the limiting VCA ratio of 1.0 minimum. The remaining mixtures, which included two 9.5-mm, two 12.5-mm, two 19.0-mm, and three 25.0-mm nominal maximum aggregate size, were all designed successfully using the proposed mix design method. Stone-on-stone contact along with the desired air void level was achieved for all mixture designs.

Wheel tracking tests were conducted on the SMA mixes and the Superpave mixes. All mixes met the typical requirements. Generally speaking, the finer mixes had higher VMA, higher asphalt contents, and higher rutting.

As stated earlier, another concern that has been expressed about some SMAs is permeability. Coarse-graded mixes are more prone to being permeable than fine-graded mixes. The mixes with 4.75-mm nominal maximum aggregate size had

insignificant permeability until the air voids reached 8 to 10 percent. However, all the other SMA mixes had significant permeability even with less than 7 percent air voids. The mix permeability increased as the nominal maximum aggregate size of the mix increased. However, the coarse-graded Superpave mixes at the same nominal maximum size were less permeable than the SMA mixes for the same void level. The fine-graded Superpave mixes were even much less permeable.

2.11 MIXTURE PERFORMANCE EVALUATION TESTING

Dynamic creep tests, resilient modulus, indirect tensile strength, and Marshall stability were all conducted on SMA mixtures. Of these four tests, the dynamic creep test showed the most promise to predict the rutting performance of SMA.

A comparison of the results of the SMA mixtures and the dense-graded mixtures showed that the magnitude of the dynamic modulus at low voids is greater for the dense-graded mixtures than for the SMA mixtures. This was contrary to what was expected. Previous work performed at NCAT had shown that dense-graded mixtures were more sensitive to change in air voids than SMA mixtures (*69*). One intent of the dynamic creep testing was to show this sensitivity. The testing showed that SMA and dense-graded mixtures had about the same sensitivity to changes in voids but the dense-graded mixtures always had less permanent deformation. It should be noted though that the dense-graded mixtures were designed below the maximum density line and therefore were coarse and similar to SMA mixtures. The percent passing the 4.75-mm sieve for these dense-graded mixtures was 41.0 percent which is only about 10 percent finer than that for the SMA mixes. Even though the results of dynamic creep testing showed that the SMA did not perform (within the test) as well as dense-graded mixtures, the dynamic creep test was able to rank the SMA mixtures and thus showed the most promise. However, later work during Phase II indicated that the loaded wheel tester also had potential for evaluating SMA mixtures.

2.12 FIELD EVALUATION OF MIXTURE DESIGN PROCEDURE

It was important to verify that the proposed mix design procedure developed under Phase I work could be verified on a number of projects. For most of the 11 projects visited, the SMA mixtures were not designed using the recommended design procedure, specifically the existence of stone-on-stone contact was not evaluated. However, they were typically designed using an approach similar to the recommended procedure so the JMF was not that different from what would have been obtained using the proposed procedure. Most of the designs were performed using 50 blows of the Marshall hammer. Another goal of the field evaluation was to verify that the procedure developed for the SGC was applicable. Based on the following discussion, it was concluded that the proposed mix design method for SMA would produce acceptable SMA mixtures.

2.12.1 Comparison of Field Data to Mixture Design Procedure

The locations of the 11 projects were selected to include all regions of the country to provide a wide variation in materials and mixture design procedures. The mixtures encountered should be representative of mixtures currently utilized for SMA. Some projects used fibers (both mineral and cellulose), some used polymers, and some used both. Some projects used pelletized fibers while others used loose fibers. A wide range of mineral filler types was used and included limestone dust, marble dust, dolomite dust, fly ash, and agricultural lime. Eight of the projects used anti-stripping agents with some using lime and others using liquid anti-strip agents. Different asphalt binders encountered at these projects included PG 76-22, PG 76-28, PG 70-22, PG 64-28, PG 64-22, PG 58-28, AC-20, and 20-40 multigrade binders. Aggregate types encountered included crushed gravels, granite, limestone, traprock, and rhyolite.

Tables 2.4 through 2.8 present comparisons of the different mixtures encountered to the requirements in the proposed mixture design procedure. Table 2.4 shows how the different gradations compared to the gradation band recommended in the proposed mix design procedure. The sieve that had the most projects not meeting the criteria was the 4.75-mm sieve. Four projects did not meet the criteria. All of the mixtures that did not meet the criteria were above the recommended upper limit of 28 percent passing and ranged from 29 to 33 percent passing. Tables 2.5 and 2.6 show that most of the coarse and fine aggregate requirements were met. Based on these tables, it appears that the aggregate quality test that most often failed to meet the requirements was the flat and elongated requirement for the coarse aggregate (Table 2.5). All of the other aggregate quality requirements were met most of the time. Table 2.7 presents the comparison of mixture properties to the recommended criteria.

Two of the projects had asphalt contents below 6 percent but the combined bulk specific gravities of the aggregates was greater than 2.75; therefore, the minimum asphalt content could be lowered slightly. Five of the mixtures were constructed with laboratory air voids below 3.5 percent while three were constructed with laboratory voids above the 4.0 percent air void level. All of the mixtures met the recommended minimum VMA of 17 percent. Six of the mixtures did not meet the VCA requirements; however, by reducing the percentage passing 4.75-mm sieve the VCA requirements could have been met.

Table 2.8 presents the comparisons for the fine mortars test results for each of the projects. Based on this table, only one project did not meet the high-temperature requirements for

TABLE 2.4 Comparison of gradations from field projects to SMA gradation specification band

Sieve Size, mm	% Passing*	Meet Specification (Yes/No)?											
		Site 1		Site 2	Site 3	Site 4	Site 5	Site 6	Site 7	Site 8	Site 9	Site 10	Site 11
		JMF-01-1	JMF-01-2										
19.0	100	Yes	Yes	Yes	Yes	Yes	Yes	**	Yes	Yes	Yes	Yes	Yes
12.7	85-95	No	Yes	No	Yes	Yes	No	**	Yes	Yes	No	Yes	Yes
9.5	75 max.	Yes	Yes	No	Yes	Yes	No	Yes	Yes	Yes	No	Yes	Yes
4.75	20-28	Yes	No	No	Yes	No	No	Yes	Yes	Yes	Yes	Yes	Yes
2.36	16-24	Yes	Yes	Yes	Yes	Yes	Yes	**	Yes	Yes	Yes	Yes	Yes
0.60	12-16	Yes	Yes	Yes	Yes	Yes	Yes	**	Yes	Yes	Yes	Yes	Yes
0.30	12-15	Yes	Yes	No	Yes	Yes	Yes	**	Yes	Yes	Yes	**	No
0.075	8-10	Yes	Yes	Yes	Yes	Yes	Yes	No	Yes	Yes	Yes	Yes	No

* SMA Specification Band From Phase I Mixture Design Procedure.
** These sieves were not used on the JMF and therefore it is unclear whether the criteria would be met.

unaged mortars and two did not meet for RTFO aged mortars. All of the projects met the low-temperature stiffness requirements.

The field work accomplished during the design procedure evaluation did indicate that the recommended mix design does develop mixes that can be produced and placed in the field. The draindown test developed as part of the mix design procedure can be used as part of quality control testing during construction.

Based on the field projects, the draindown test was easy to conduct and appeared to be a good indicator of mix changes. When the filler content was low or the asphalt content was high, the amount of draindown observed in the laboratory tended to increase. This occurred for most of the projects.

2.12.2 Comparison of Marshall Hammer and Superpave Gyratory Compactor

Work in Phase I of this project indicated that 100 gyrations of the SGC would provide about the same density as 50 blows with the Marshall hammer. Since past experience with SMA has been good and since most SMA mixtures in the past were designed using 50 blows of the Marshall hammer, it was determined in this study that the compaction effort

TABLE 2.5 Comparison of coarse aggregate quality from field projects to SMA mixture design criteria

Test	Method	Criteria	Meet Specification (Yes/No)?											
			Site 1		Site 2	Site 3	Site 4	Site 5	Site 6	Site 7	Site 8	Site 9	Site 10	Site 11
			JMF-01-1	JMF-01-2										
LA Abrasion	AASHTO T 96	30 max.	Yes	Yes	Yes	Yes	No	Yes	Yes	Yes	Yes	Yes	Yes	No
Flat or Elongated 3:1 5:1	ASTM D 4791 Section 8.4	20 max. 5 max.	Yes **	** **	No Yes	Yes **	Yes **	** Yes	Yes	No No	No No	** **	No Yes	** **
Absorption	AASHTO T 85	2 max.	Yes	Yes	**	**	Yes	**	**	Yes	Yes	Yes	Yes	No
Soundness (5 cycles) Sodium Sulfate Magnesium Sulfate	AASHTO T 104	15 max. 20 max.	Yes	Yes	Yes	**	Yes	**	Yes	Yes	Yes[2]	Yes[2]	**	**
Crushed Content One Face Two Faces	[1]	100 min. 90 min.	Yes Yes	Yes Yes	Yes Yes	Yes Yes	Yes Yes	Yes Yes	Yes Yes	Yes Yes	Yes Yes	Yes Yes	Yes Yes	Yes Yes

[1] Most State DOTs have their own method for calculating crushed content. If no method currently exists for the agency, Pennsylvania DOT Test Method No. 621 can be used.
[2] This state used a modified version of AASHTO T 103 to determine soundness. Test results showed loss values below 6 percent. These results were interpreted as acceptable.
** Data Not Available.

TABLE 2.6 Comparison of fine aggregate quality from field projects to SMA mixture design criteria

Test	Method	Criteria	Meet Specification (Yes/No)?										
			Site 1		Site 2	Site 3	Site 4	Site 5	Site 7	Site 8	Site 9	Site 10	Site 11
			JMF-01-1	JMF-01-2									
Soundness (5 cycles) Sodium Sulfate Magnesium Sulfate	AASHTO T 104	15 max. 20 max.	**	**	Yes	**	**	**	Yes[2]	Yes[2]	**	**	**
Angularity	AASHTO TP 33	45 min.	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No	Yes	Yes	No
Liquid Limit	AASHTO T 89	25 max.	**	**	Yes	**	**	**	**	**	**	**	**
Plasticity Index	AASHTO T 90	NP[1]	**	**	Yes	**	**	**	Yes	Yes	**	**	**

[1] Non-Plastic
[2] This state used a modified version of AASHTO T 103 to determine soundness. Test results showed loss values below 1 percent. These results were interpreted as acceptable.
** Data Not Available.
NOTE: Site 6 did not use a fine aggregate.

with the SGC should be set to provide a density approximately equal to that with the Marshall hammer. Comparisons were made on the field projects to verify that 100 gyrations with the SGC was equal to 50 blows of the Marshall hammer (Figure 2.14). Based on Figure 2.14, there was a lot of scatter in the data when comparing the 50-blow Marshall density with 100-gyration SGC. Some projects needed more than 100 gyrations to produce density equal to that produced by 50 blows of the Marshall hammer. On other projects fewer than 60 gyrations of the SGC were needed to provide equal density. However, based on the data it appears that approximately 80 gyrations of the SGC are required to provide equal density to that of 50 blows with the Marshall hammer.

Part of the variation shown in Figure 2.14 could be caused by errors inherent in the back calculation of bulk specific gravities when using the SGC. Another possible source of variation in the data was the differences in aggregate L.A. abrasion loss values from the different field projects. SMA mixtures that contain softer aggregates will tend to compact faster than mixtures with harder aggregates. To determine if the hardness of the different aggregates affected the compaction characteristics of the different mixtures, a multiple linear regression analysis was performed. For this analysis, the G_{mb} ratio was the dependant variable while the gyration levels and L.A. abrasion loss values were designated the independent variables.

Based on the analysis, 47 percent of the variation in Figure 2.14 could be attributed to the two independent variables. Therefore a chart showing the effects of both gyration levels and L.A. abrasion loss values was developed (Figure 2.15). Instead of showing a regression line for all of the projects evaluated, this figure shows the results for L.A. abrasion loss values of 20, 30, and 40 percent with the same gyration levels as Figure 2.14.

Based on Figure 2.15, the number of gyrations needed to produce a similar density as 50 blows of the Marshall hammer ranges from 68 to 82 for aggregates with 20 to 40 percent loss. Therefore, it was recommended that for SMA mixtures using aggregates with L.A. abrasion loss values of more than 30 percent, a design number of gyrations of 70 be used. For harder aggregates (loss values less than 30 percent), a design number of gyrations of 100 should be used. The difference in these two gyration levels will result in a difference in optimum asphalt content of approximately 0.4 percent. The 70

TABLE 2.7 Comparison of volumetric data from field projects to SMA mixture specification

Property	Requirement	Meet Specification (Yes/No)?											
		Site 1		Site 2	Site 3	Site 4	Site 5	Site 6	Site 7	Site 8	Site 9	Site 10	Site 11
		JMF-01-1	JMF-01-2										
Asphalt Content	6 min	Yes	No[1]	Yes	Yes	No[1]	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Air Voids	3.0 - 4.0	Yes	No	Yes	No	No	No	Yes	No	No	Yes	No	No
VMA	17 min.	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
VCA	Less than VCA_{DRC}	No	No	No	Yes	No	No	Yes	Yes	Yes	Yes	No	Yes

[1] Using the stipulation that the asphalt content can be slightly lower if the combined bulk specific gravity exceeds 2.75, this property meets the requirement.

TABLE 2.8 Comparison of fine mortar properties from field projects to SMA mixture specification

Property	Requirement	Meet Specification (Yes/No)?										
		Site 1	Site 2	Site 3	Site 4	Site 5	Site 6	Site 7	Site 8	Site 9	Site 10	Site 11
Unaged DSR, G*/Sinδ (kPa)	5 min.	Yes	No	Yes[1]	Yes	Yes	Yes[1]	Yes	Yes	Yes	Yes	Yes
RTFO Aged DSR, G*/Sinδ (kPa)	11 min.	Yes	No	Yes[1]	Yes	No	Yes[1]	Yes	Yes	Yes	Yes	Yes
PAV BBR, Stiffness (MPa)	1500 max.	Yes	Yes	Yes[1]	Yes	Yes	Yes[1]	Yes	Yes	Yes	Yes	Yes

[1] Results of this test passed the criteria but the test results were suspect.

and 100 gyrations were selected to compare to the recommended gyration levels from NCHRP Project 9-9, "Refinement of the Superpave Gyratory Compaction Procedure."

2.13 FIELD DENSITY REQUIREMENTS

At the present time, most SMA projects are required to be compacted to an in-place air void content of 5 to 6 percent. This appears to be acceptable, but there is some evidence that indicates that SMA mixtures could be more permeable to water at a lower air void content than are dense-graded mixtures. Work in Phase I showed that SMA mixtures become permeable at approximately 6 percent air voids. This work was accomplished to determine at what density SMA becomes permeable based on field measurements.

In-situ permeability measurements were made at eight of the field projects visited under Task 8. The in-situ permeability was determined using a falling head method that consisted of sealing a plastic cylinder to the pavement. At each location that in-situ permeability measurements were made, cores were obtained. These cores were brought back to the NCAT laboratory to evaluate the laboratory permeability.

Based on these two methods of evaluating permeability, increasing air voids generally resulted in increasing permeability. Based on all of the data collected from the eight projects, it appeared that SMA mixtures become permeable to water at air void contents between 5.0 and 6.5 percent. This confirmed that SMA mixtures should be compacted in the field to air void contents of between 5 and 6 percent.

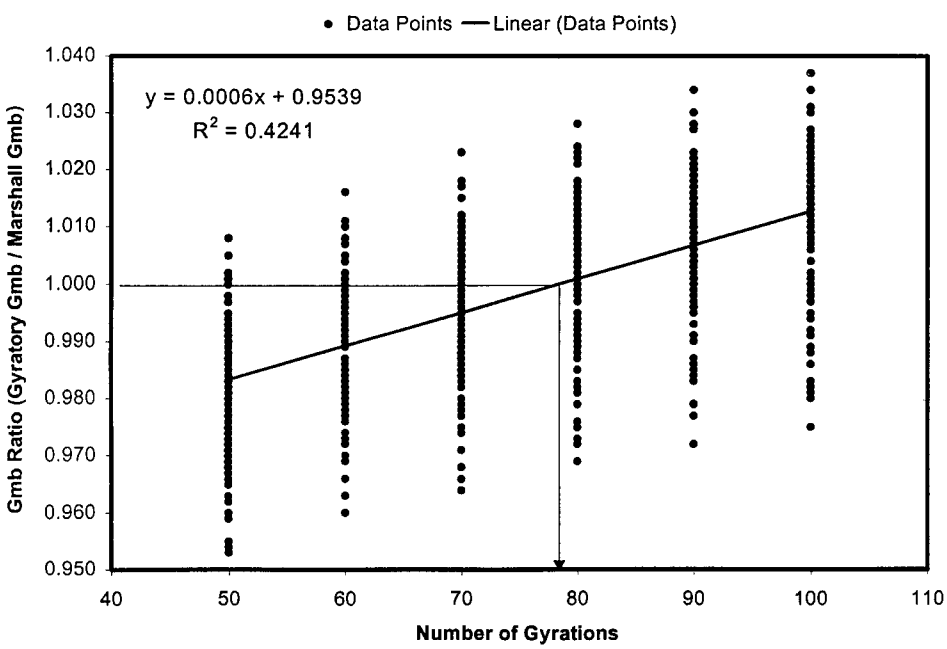

Figure 2.14. Comparison of densities of samples compacted with 50 blows of the Marshall hammer and 100 gyrations of the SGC.

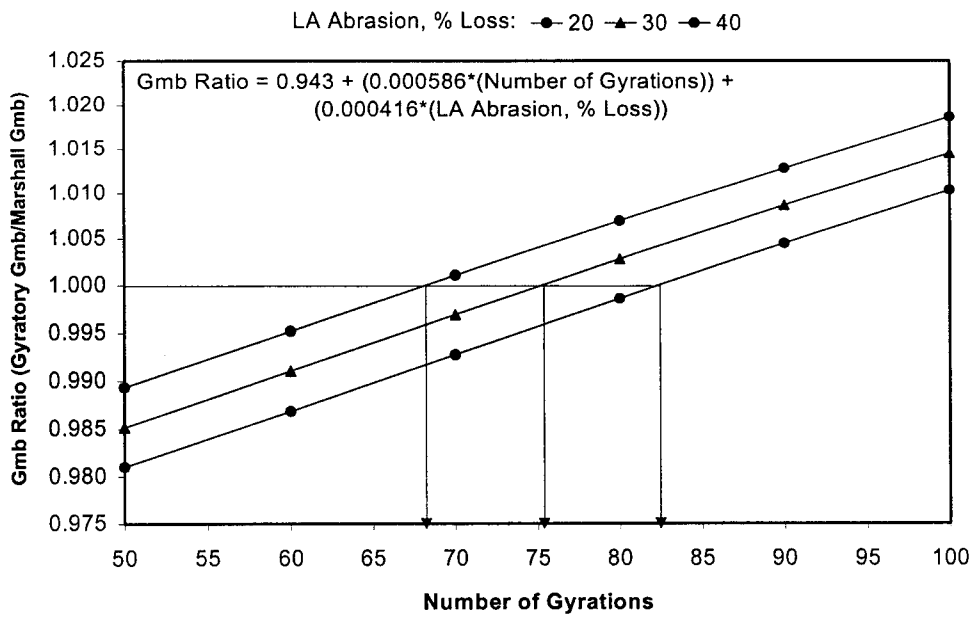

Figure 2.15. G_{mb} ratio as a function of gyration level and L.A. abrasion.

2.14 ACCURACY AND PRECISION OF NUCLEAR GAUGE

In order to evaluate the accuracy and precision of nuclear gauges for measuring the density of SMA pavements, cores were obtained from eight of the 11 projects visited under Task 8. At each location that cores were obtained, density measurements were made with a nuclear density gauge with and without Ottawa sand. Two different methods were used to analyze the data accumulated. Both methods consisted of determining a correction factor to correct the nuclear density gauge measurements, the primary difference being how the correction factors were determined. The two methods were investigated to determine which method was more accurate and precise.

The first method consisted of determining an average bias based on the average difference between the cores and the nuclear density measurements. The premise of this type of correction factor was that the final corrected nuclear gauge density would be equal to the core density.

The second method of evaluating the data consisted of determining a sliding (dynamic) correction factor based on measured nuclear density gauge readings. For a given project's data, the core densities were divided by the nuclear density gauge measurements to yield individual density ratios. This was done for both the "with" and "without" leveling sand data. Next, the individual density ratios were plotted as a function of nuclear density gauge measurements and a simple linear regression performed (separate regression analyses were performed for the with and without sand data). Based on the regression line, the correction factor was obtained for each individual nuclear gauge measurement. This correction factor was then multiplied by the nuclear density gauge reading for the corrected nuclear gauge density. Unlike the average bias correction factors, the premise of this analysis was that a strong relationship existed between the corrected nuclear density gauge readings and the core densities.

To compare which method of reducing the data provided a more accurate estimation of true density, the standard error of the estimate was used and was determined for both the with sand and without sand readings. Using the method of static correction factors, the standard error of the estimate was based on the line of equality because the premise of that analysis was that the corrected nuclear density gauge readings would be equal to the core density. For the dynamic correction factor measurements, the standard error of the estimate was based on the linear regression line between the corrected densities and the core densities.

Results of these analyses showed that the dynamic correction factors provided a better estimate of true densities (Table 2.9). Standard error of the estimates ranged from 0.61 to 2.75 lb/ft^3 for the dynamic correction factors. Using the static correction factors, the standard error of the estimates ranged from 1.02 to 5.19 lb/ft^3. Additionally, it appeared that the nuclear density gauge readings made with a leveling sand gave a better estimate of true densities than did the measurements without sand (Table 2.9). For the dynamic correction factors, the with sand readings had lower standard errors of the estimate for six of the eight sites.

TABLE 2.9 Standard error of the estimates for nuclear density gauge measurements

Site	Static Correction Factors		Dynamic Correction Factors	
	With Sand	Without Sand	With Sand	Without Sand
2	2.53	3.73	2.44	2.85
3	1.98	1.65	0.83	1.31
4	1.32	1.55	1.28	1.52
5	1.76	2.08	1.15	1.73
6	2.29	2.68	1.56	1.17
9	2.29	1.02	0.61	0.96
10	5.19	4.45	2.57	2.75
11	2.28	2.20	1.82	1.78
Avg.	2.46	2.42	1.53	1.76

Based on this discussion, it was concluded that the nuclear density gauges could be used to estimate the density of SMA pavements in the field. A dynamic correction factor should be employed to correct the nuclear density gauge measurements and Ottawa sand should be placed on the pavement surface when performing nuclear density tests for best accuracy. Tests can be conducted without sand but more readings would be necessary to develop the same accuracy as that obtained when sand is used.

CHAPTER 3

CONCLUSIONS AND RECOMMENDATIONS

The conclusions provided below are based on the research results obtained during both Phase I and Phase II along with available literature.

- The dry-rodded test for coarse aggregates is acceptable for measuring the voids in coarse aggregate (VCA) that can define when stone-on-stone contact is achieved in the mixture. (Phase I)
- Excessive aggregate breakdown in aggregates may make it difficult to meet VMA requirements. This often becomes a problem when the L.A. abrasion is over 30. (Phase I)
- Superpave binder tests can be used to evaluate the quality of SMA mortars. The DSR minimum requirement for $G^*/\sin\delta$ on unaged mortars should be set at 5.0 kPa. The minimum requirement for $G^*/\sin\delta$ for RTFO aged mortars should be set at 11.0 kPa. The maximum stiffness in the BBR test should be set at 1500 Mpa. These suggested requirements are optional and should be used with care until experience is obtained with these procedures. (Phase I and Phase II)
- No test was identified that clearly showed promise in predicting performance of the SMA mixture during Phase I; however, loaded wheel testing during Phase II showed promise. Tests during Phase I included dynamic creep tests, indirect tensile strength, resilient modulus, and Marshall stability. A major effort to develop a performance test is ongoing as part of Superpave. Results of this effort should be applicable to SMA. (Phase I and Phase II)
- The coarse and fine aggregates used in SMA mixtures should be 100 percent crushed. The fine aggregate angularity should equal or exceed 45. (Phase I)
- The L.A. abrasion of the coarse aggregate should be a maximum of 30, however, experience has shown that good SMA mixes have been constructed with L.A. abrasion values above 30. (Phase I and Phase II)
- Flat and elongated particles should be measured on a 3 to 1 and 5 to 1 ratio. The requirement for 3 to 1 should be set at 20 percent maximum. However, some locations may have problems meeting this requirement. The requirement for 5 to 1 should be set at 5 percent maximum. (Phase I)
- A good screening test for fillers is the modified Rigden voids. Fillers that exceed 50 cause the mortar to be excessively stiff and difficult to work. (Phase I)
- There should be no specification requirement for SMA that sets a limit on the percentage of material passing the 0.02-mm size. (Phase I)
- When SMA is used, it is normally better to increase the PG grade by one or two grades above that recommended based on the climate. AASHTO MP2, "Standard Specification for SUPERPAVE Volumetric Mix Design," should be consulted for guidance to change the performance grade of the asphalt binder for traffic speed or traffic level. (Phase I and Phase II)
- The minimum VMA requirement should be set at 17 percent. (Phase I)
- The 50-blow compactive effort with the Marshall hammer appears to be reasonable. The compactive effort with the SGC should be 100 gyrations for mixtures having aggregates with L.A. abrasion loss values of less than 30 percent. However, if the L.A. abrasion is above 30 then consideration should be given to lowering the compactive effort to 70 gyrations. These gyration levels are the design number of gyrations. For both gyration levels, VMA requirements should be met as well as in-place density requirements. (Phase I and Phase II)
- The tensile strength ratio (TSR) of SMA mixtures is typically lower than for dense-graded mixes. This does not indicate that SMAs are more susceptible to moisture but does indicate that the acceptance limit for TSR should be lower for SMA than for dense-graded mixtures when tested in accordance with AASHTO T283. The TSR requirement should be set at 70 percent. The TSR should be conducted at an average air void level of 6.0 percent. For SGC designed SMA, the samples should be compacted in accordance with AASHTO MP2, Section 7.6, which states the samples should be compacted to a height of 95 mm for 150-mm diameter specimens. (Phase I)
- The use of fibers tends to do a better job than polymers in reducing draindown. Polymers seem to do a good job of decreasing temperature susceptibility of a mixture. So there are advantages for both additives and in certain cases both may be used. (Phase I)

- Breakdown in the laboratory tends to be 5 to 10 percent on the 4.75-mm sieve for harder aggregates (L.A. abrasion loss value of 30 percent or less) and more for softer aggregates (L.A. abrasion loss value of more than 30 percent). The Marshall hammer tends to breakdown the aggregate more than the gyratory compactor (at 100 gyrations). Both laboratory compactors provide breakdown approximately equal to in-place compaction. (Phase I and Phase II)
- Mixes designed using the method provided within this report can be produced and placed in the field. There are no specific areas of difficulty in meeting specifications. (Phase II)
- SMA mixtures prepared in the laboratory perform well when tested under laboratory wheel tracking tests. (Phase II)
- Increasing the maximum aggregate size and/or the percentage of coarse aggregate produces a mix that is more susceptible to permeability. When a mix with a smaller nominal maximum aggregate size is used, such as 4.75 mm, the optimum asphalt content will tend to be very high. The in-place voids must be approximately 6 percent or lower to ensure that it is not permeable. Compaction requirements should be set at a minimum of 94 percent of theoretical maximum density for SMA mixtures. (Phase II)
- SMA mixes are more permeable than coarse-graded Superpave mixes and even more permeable than fine-graded Superpave mixes for similar void contents and similar nominal maximum size aggregates. Thus SMA should be compacted in-place to a void content of approximately 6 percent or less. (Phase I and Phase II)
- For the construction projects evaluated in this study, there was no significant difference in aggregate breakdown for static and vibratory rollers. The L.A. abrasion appeared to have little effect on the in-place breakdown of aggregate. (Phase II)
- Asphalt binders undergo significant physical changes when heated above 180°C (356°F). SMA mixes often use modified binders which require higher temperatures, but be cautioned when approaching temperatures of 180°C. (Phase II)
- The Brookfield viscometer and a workability test were used successfully to measure the mixture stiffness, however, these tests could not be successfully used to determine mixing and compaction temperatures. (Phase II)
- A laboratory test was used successfully to measure the permeability of SMA mixtures. However, a field test for permeability was not successful. (Phase II)
- A nuclear gauge can be used to measure the in-place density of SMA mixtures. However, the error can be significant. The correction factor for nuclear gauges should be based on cores and should be a function of measured in-place density. The use of a leveling sand, such as Ottowa sand, was shown to provide better accuracy than not using leveling sand. When measuring the density of SMA pavements, if leveling sand is not used, more nuclear gauge tests should be performed to provide similar accuracy. (Phase II)
- Too many projects are being constructed with incorrect laboratory voids and high in-place voids. For best performance, these properties must be controlled. SMA volumetrics are extremely sensitive to gradation changes, especially on the 4.75-mm sieve and on the 0.075-mm sieve. (Phase II)

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20.	1991	Stone Mastic Asphalt Test and Evaluation Project No 18	Bukowski, J.	Office of Technology Applications Federal Highway Administration, 1991
21.	1991	Stone Mastic Asphalt: A Potential Rutting Solution	Pryor, C.	Stone Review, Oct 1991
22.	1991	Asphalt Mix Technology Puts Emphasis on Aggregate	Drake, R.	Pit & Quarry, September 1991
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25.	1991	SMA Comes to the USA	Warren, J.M.	Hot Mix Asphalt Technology, Fall 1991, National Asphalt Pavement Association
26.	1991	Report on the 1990 Eurpean Asphalt Study Tour		American Association of State Highway and Transportation Officials, 1991
27.	1991	Split Mastic Asphalt-Next Overseas Import?	Kuennen, T.	Roads and Bridges, Vol 29, No 1, Jan 1991, p 48
28.	1991	Over 300 Gather for European SMA FHWA/ Michigan Demo Project	Eaton, M.	Roads and Bridges, Vol 29, No 10, Sep 1991, p 98
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31.	1992	Stone Mastic Asphalt		Georgia Department of Transportation, Sep1992/July1993
32.	1992	Supplemental Specifications		Georgia Department of Transportation, Sep 1992/July 1993
33.	1992	Stone Mastic Asphalt for Very Thin Wearing Courses	Serfass, J.P., and Samanos, J.	Transportation Research Board, 1992
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39.	1992	New Mixes, Modifiers Put to Test on I-94	Milo, A.C.	Centerline, Michigan Asphalt Pavement Association, Fall 1992
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46.	1992	Construction of Stone Mastic Asphalt Test Sections in the U.S.	Scherocman, J.A.	Association of Asphalt Paving Technologists, Vol 61, 1992, Symposium: Application of European Technology for Improved Pavement Performances, p 642
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50.	1992	The Design and construction of Stone Mastic Asphalt Pavements	Scherocman, J.A.	American Association of State Highway and Transportation Officials, Annual Meeting, 1992
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52.	1992	The Design, Construction and Performance of Stone Mastic Asphalt Pavement Layers	Scherocman, J.A.	Proceedings of the Canadian Technical Asphalt Association, Nov 1992, p 132
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55.	1993	SMA, Arizona Link No Exaggeration	Morris, G.R.	Roads and Bridges, Vol 31, No 6, Jun 1993
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57.	1993	Stone Matrix Pavements Require Special Technique	Kriech, A.J.	Asphalt Contractor, Feb 1993, p 8
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63.	1993	Evaluation of Stone Mastic Asphalt in Missouri Route I-70, Boon County	McDaniel, P.	Experimental Project No. Mo 92-07, Initial Report, Missouri Highway and Transportation Department, 1993
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67.	1993	Effect of Mortar Stabilizers in Split-Mastic-Asphalts	Harders, O.	5th Eurobitume Congress, Volume, 1B 1993, p 606
68.	1993	The Design, Construction and Performance of Stone Mastic Pavement Layers: the Continuing Story	Scherocman, J.A.	Proceedings of the Canadian Technical Asphalt Association, Nov 1993, p 333
69.	1993	Evaluation of Laboratory Properties of SMA Mixtures	Brown, E.R., and Manglorkar, H.	National Center for Asphalt Technology, Report No. 93-5, 1993
70.	1993	Stone Mastic Asphalt Wearing Course	Walsh, I.D.	Clause 995 AK, Kent County Council Specification, Nov 1993
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72.	1994	Dense Graded Mixtures Getting Industry Out of a Rut with Stone Matrix Asphalt Mix	Avera, L.T.	Asphalt Contractor, Jan 1994
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76.	1994	Evaluation of Using Different Stabilizers in the U.S. Route 15 (Maryland) Stone Matrix Asphalt (SMA)	Stuart, K.D., and Malmquist, P.	Transportation Research Board, 1994
77.	1994	Mix Design Considerations for SMA Mixes	Carpenter, S.H.	Transportation Research Board, 1994
78.	1994	Evaluation of Stone Matrix Asphalt Versus Dense-Graded Mixtures	Mogawer, W.S., and Stuart, K.D.	Transportation Research Board, 1994
79.	1994	Structrual Strength of Asphalt Rubber Concrete Developed Through Stone Mastic Asphalt Concept	Svec, O.J., and Veizer, R.	Transportation Research Board, 1994
80.	1994	Stone Mastic Asphalt, Route I-70, Callaway County, Construction Phase Report		Report No. Mo 93-01, Missouri Highway and Transportation Department, 1994
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90.	1996	An Investigation of Stone Matrix Asphalt Mortars	Brown, E.R., Haddock, J.E., and Crawford, C.	Transportation Research Board, 1996

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92.	1996	Summary of Georgia's Experience with Stone Matrix Asphalt Mixes	Watson, D. and Jared, D.	Transportation Research Board, 1996
93.	1996	Construction of SMA Section at Edwards AFB	Shoenberger, J.E.	Transportation Research Board, 1996
94.	1997	The Construction and Performance of Stone Matrix Asphalt Pavements In the United States	Scherocman, J.A.	Eighth International Conference on Asphalt Pavements, August 10-14, 1997, Seattle Washington, Vol. I Proceedings
95.	1997	Characterization of Stone Matrix Asphalt Mortars	Brown, E.R. and Haddock, J.E.	Eighth International Conference on Asphalt Pavements, August 10-14, 1997, Seattle Washington, Vol. I Proceedings
96.	1997	Development of a Stone Mastic Asphalt Design Methods for South African Conditions	Louw, L., Semmelink, C.J. and Verhaeghe, BMJA	Eighth International Conference on Asphalt Pavements, August 10-14, 1997, Seattle Washington, Vol. I Proceedings
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100.	1998	Stone Matrix Asphalt: The Wisconsin Experience	Schmiedlin, R.B.	Preprint prepared for Presentation at the 1998 Annual Meeting of the Transportation Research Board, National Research Council, Washington, D.C. 1998
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PART 2: Mixture Design Method, Construction Guidelines, and Quality Control/Quality Assurance Procedures

Volume IV
Mixture Design Method, Construction Guidelines, and Quality Control Procedures

Prepared for
National Cooperative Highway Research Program
Transportation Research Board
National Research Council

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INTRODUCTION

The National Center Asphalt Technology (NCAT) has developed and validated a mix design procedure for stone matrix asphalt (SMA). This work was funded by the National Cooperative Highway Research Program (NCHRP) under project number NCHRP 9-8 and was completed in 1998.

This project involved 15 tasks that included the following topics: literature review, material and mixture properties, appropriate laboratory tests, development of a mix design procedure, field evaluation of the mix design procedure, development of construction guidelines, development of quality control/quality assurance procedures, verification of density requirements, and the accuracy and precision of nuclear density gauges for SMA pavements.

The results of this study have been published in five volumes. This volume (IV) presents the finalized mix design procedure, construction guidelines, and quality control/quality assurance procedures for SMA mixtures as draft AASHTO standards. Also presented in AASHTO format are two standards for laboratory testing developed during the conduct of NCHRP 9-8 and include a standard test method for determining the draindown characteristics of uncompacted hot mix asphalt and a standard practice for the preparation and testing of hot mix asphalt mortars. Additionally, an example SMA mixture design and an example illustrating how to blend SMA gradations based upon percent aggregate passing by volume are provided.

Standard Practice for Designing Stone Matrix Asphalt (SMA)

AASHTO Format

Standard Practice for Designing
Stone Matrix Asphalt (SMA)

1. Scope

1.1 This standard practice covers the design of Stone Matrix Asphalt (SMA) using either the SHRP Gyratory Compactor (SGC) or a mechanical, static base, flat-faced Marshall hammer. The SMA design is based on the volumetric properties of the SMA in terms of air voids, the voids in mineral aggregate, and the presence of stone-on-stone contact.

1.2 The values stated in SI units are to be regarded as the standard. The English units in parentheses are for information only.

1.3 *This standard does not purport to address all of the safety concerns, if any, 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 AASHTO Standards:

T19 Unit Weight and Voids in Aggregate
T27 Sieve Analysis of Fine and Coarse Aggregates
T84 Specific Gravity and Absorption of Fine Aggregate
T85 Specific Gravity and Absorption of Coarse Aggregate
T100 Specific Gravity of Soils
T166 Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface-Dry Specimens
T209 Maximum Specific Gravity of Bituminous Paving Mixtures
T245 Resistance to Plastic Flow of Bituminous Materials Using Marshall Apparatus
T283 Resistance of Compacted Bituminous Mixtures to Moisture Induced Damage
MP1 Specification for Performance Graded Asphalt Binder
MP2 Specification for SUPERPAVE Volumetric Mix Design
PP2 Short and Long Term Aging of Bituminous Mixes
TP4 Preparing and Determining the Density of Hot-Mix Asphalt Specimens by Means of the SHRP Gyratory Compactor

2.2 Asphalt Institute

MS-2 Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types

3. Terminology

3.1 SMA - stone **m**atrix **a**sphalt - SMA is a hot mix asphalt consisting of two parts, a coarse aggregate skeleton and an asphalt binder rich mortar. The mixture must have an aggregate skeleton with coarse aggregate-on-coarse aggregate contact (generally referred to as stone-on-stone contact). The coarse aggregate is generally considered to be that fraction of the aggregate retained on the 4.75 mm sieve but may be designated as other sizes. Sufficient mortar of the desired consistency must be provided. Satisfactory mortar consistency and thus good SMA performance requires that a relatively high asphalt binder content be used. For this reason the voids in the mineral aggregate (VMA) must exceed some minimum requirement.

3.2 Air voids (V_a) - the total volume of the small pockets of air between the coated aggregate particles throughout a compacted paving mixture, expressed as percent of the bulk volume of the compacted mixture (MS-2).

3.3 Voids in mineral aggregate (VMA) - the volume of intergranular void space between the aggregate particles of a compacted paving mixture that includes the air voids and the effective asphalt content, expressed as a percent of the total volume of the specimen (MS-2).

3.4 Voids in Coarse Aggregate - the volume in between the coarse aggregate particles. This volume includes filler, fine aggregate, air voids, asphalt binder, and fiber if used.

3.5 Nominal Maximum Aggregate Size - one size larger than the first sieve that retains more than 10 percent.

3.6 Maximum Aggregate Size - one sieve size larger than the nominal maximum aggregate size.

3.7 SMA Mortar - mixture of asphalt binder, filler (material passing 0.075 mm sieve), and stabilizing additive

3.8 Stabilizing Additive - polymer and/or fiber

4. Summary of the Practice

4.1 Materials Selection - Asphalt binder, aggregate stockpiles, mineral fillers, and stabilizing additives are selected that meet specification.

4.2 Select Optimum Gradation - At least three trial aggregate gradations from the selected aggregate stockpiles are blended. Gradations for SMA are based on volumes. The dry-rodded unit weight for the coarse aggregate for each trial gradation is determined in accordance with T19. For each trial gradation, an initial trial asphalt binder content between 6 and 6.5 percent asphalt binder is selected and

at least two specimens are compacted in accordance with TP4 if the SHRP gyratory compactor is used and at least three specimens are compacted in accordance with T245 if the selected compaction method is the Marshall hammer. The gradation is selected to ensure that minimum VMA requirements and stone-on-stone contact are achieved.

4.3 Design Asphalt Binder Content Selection - Replicate specimens are compacted in accordance with TP4 or T245 at three asphalt binder contents. The design asphalt binder content is selected on the basis of satisfactory conformance with the requirements of Section 11.

4.4 Evaluation of SMA Mortar Properties - If mortar testing is required, the SMA mortar is prepared at the percentages (by mass) of the final designed mixture and evaluated in accordance with MP1. If the SMA mortar specification requirements are not met, the mortar fails and must be modified so that specification requirements are met.

4.5 Evaluating Moisture Susceptibility and Draindown - The moisture susceptibility of the mixture designed and compacted in accordance with TP4 or T245 to an air void content (V_a) of 6±1 percent, is evaluated in accordance with T283. The designed mixture is evaluated for sensitivity to asphalt binder draindown. If the mixture fails the moisture susceptibility or draindown tests, it must be modified so that specification requirements are met.

5. Significance and Use

5.1 The procedure described in this practice is used to design SMA mixtures which provide satisfactory SMA mixtures that will provide good performance when subjected to very high traffic volumes.

6. Test Specimens

6.1 Number of Samples - A total of twelve samples are initially required; four samples at each of the three trial gradations. Each sample is mixed with the trial asphalt binder content and three of the four samples for each trial gradation are compacted. The remaining sample of each trial gradation is used to determine the theoretical maximum density according to T209.

6.2 Preparation of Aggregates - Dry aggregates to a constant mass at 105 to 110°C (221 to 230°F) and separate the aggregates by dry-sieving into the desired size fractions. The following size fractions are recommended:

37.5 to 25.0 mm (1½ to 1 inch)
25.0 to 19.0 mm (1 to ¾ inch)
19.0 to 12.5 mm (¾ to ½ inch)
12.5 to 9.5 mm (½ to ⅜ inch)
9.5 to 4.75 mm (⅜ inch to No. 4)
4.75 to 2.36 mm (No. 4 to No. 8)
Passing 2.36 mm (No. 8)

6.3 Determination of Mixing and Compaction Temperatures:

6.3.1 The temperature to which the asphalt binder must be heated to produce a viscosity of 170±20 cSt shall be the mixing temperature.

6.3.2 The temperature to which the asphalt binder must be heated to produce a viscosity of 280±30 cSt shall be the compaction temperature.

6.3.3 However, while these temperatures work for neat asphalt binders, the selected temperatures may need to be changed for polymer modified asphalt binders. The polymer manufacturer's guidelines for mixing temperature should be used.

6.4 Preparation of Mixtures:

6.4.1 A mechanical mixing apparatus shall be used.

6.4.2 An initial batch shall be mixed for the purpose of "buttering" the mixture bowl and stirrers. This batch shall be emptied after mixing and the sides of the bowl and stirrers shall be cleaned of mixture residue by scraping with a small limber spatula but shall not be wiped with cloth or washed clean with solvent, except when a change is to be made in the asphalt binder or at the end of a design.

6.4.3 Weigh into separate pans for each test specimen the amount of each size fraction required to produce a batch of aggregate that will result in a compacted specimen of the correct size. For Marshall compaction this will be approximately 1200 grams. For SGC compaction, this will be approximately 4500 grams. Mix the aggregate in each pan and place in an oven and heat to a temperature not exceeding the mixing temperature established in Section 6.3 by more than approximately 28°C (50°F). Heat the asphalt binder to the established mixing temperature. The stabilizing additive, fiber and/or polymer, should be added to the heated aggregate prior to the introduction of the asphalt binder. The stabilizing additive should also be manually mixed with the heated aggregate. This procedure is needed to insure an even distribution of the stabilizing additive during the laboratory

mixing process. Slightly longer mixing times may be required due to the increased surface area added by the filler, fiber, and/or the stiffening effect of the polymer.

6.4.4 Form a crater in the dry blended aggregate and stabilizing additive and weigh the preheated required amount of asphalt binder into the crater formed in the aggregate blend. Care must be exercised to prevent loss of the mix during mixing and subsequent handling. At this point, the temperature of the aggregate and asphalt binder shall be within the limits of the mixing temperature established in Section 6.3. Mix the aggregate and asphalt binder rapidly until thoroughly coated.

6.5 Compaction of Specimens - The compaction temperature is determined in accordance with section 6.3. Laboratory samples of SMA are short-term aged in accordance with AASHTO PP2 and then compacted using either 50 blows per face of a flat-face, static base, mechanical Marshall hammer (T245) or 100 gyrations of the Superpave Gyratory Compactor (TP4). (NOTE 1)

NOTE 1 - When aggregates having a Los Angeles Abrasion loss value greater than 30 percent or the design ESALs are less than 1 million, the number of gyrations of the Superpave gyratory compactor should be 70 for design.

7. Selection of Trial Gradations

7.1 The trial gradations must be selected to be within the master specification range shown in Table 1. These gradations are based on percent volumes passing the respective sieves. The gradations are blended based on volumes and then converted to gradations based on mass for the mixture design process (NOTE 2). To design an SMA mixture it is recommended that at least three trial gradations be initially evaluated. It is suggested that the three trial blends fall along the coarse and fine limits of the gradation band along with one gradation falling in the middle. These trial gradations are obtained by adjusting the amount of fine and coarse aggregate in each blend. The percent passing the 0.075 mm (No. 200) sieve should have approximately 10 percent for each trial gradation with the exception of the 4.75 mm NMAS SMA mixtures, this mixture should have approximately 14 percent passing the 0.075 mm sieve.

NOTE 2 - An example problem illustrating how to blend gradations based on volume can be found later in this report.

7.1.1 With some aggregates it may be difficult to meet VMA requirements no matter how the aggregates used are blended. If VMA requirements are not met for any of the trial gradations, it is likely caused by excessive breakdown of the aggregate during laboratory compaction. If this is the case, the aggregate is probably not acceptable for SMA mixtures.

TABLE 1 Stone Matrix Asphalt Gradation Specification Bands (Percent Passing by Volume).

Sieve size, mm	25 mm NMAS[1]		19 mm NMAS		12.5 mm NMAS		9.5 mm NMAS		4.75 mm NMAS	
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
37.5	100	100								
25.0	90	100	100	100						
19.0	30	86	90	100	100	100				
12.5	26	63	50	74	90	100	100	100		
9.5	24	52	25	60	26	78	90	100	100	100
4.75	20	28	20	28	20	28	26	60	90	100
2.36	16	24	16	24	16	24	20	28	28	65
1.18	13	21	13	21	13	21	13	21	22	36
0.6	12	18	12	18	12	18	12	18	18	28
0.3	12	15	12	15	12	15	12	15	15	22
0.075	8	10	8	10	8	10	8	10	12	15

[1] NMAS - Nominal Maximum Aggregate Size - one size larger than the first sieve that retains more than 10 percent.

8. Selection of Trial Asphalt Binder Content

- As a starting point, for aggregates with bulk specific gravities approximately equal to 2.75, an asphalt content of approximately 6.0 percent or greater by mass should be selected. If the bulk specific gravity of the coarse aggregate exceeds 2.75, the trial asphalt binder content can be reduced by approximately 0.1 percent for each increment of 0.05 above 2.75. If the bulk specific gravity of the coarse aggregate is below 2.75, the trial asphalt binder content can be increased approximately 0.1 percent for each increment of 0.05 below 2.75.

9. Determination of VCA in the Coarse Aggregate (Stone) Fraction

9.1 For best performance, the SMA mixture must have a coarse aggregate skeleton with stone-on-stone contact. The stone fraction is that portion of the total aggregate blend retained on the 4.75 mm (No. 4) sieve for the 12.5 mm, 19 mm, and 25 mm NMAS SMA mixtures. For the 9.5 mm nominal maximum aggregate size SMA mixture, the stone is that portion of aggregate blend retained on the 2.36 (No. 8) sieve. For the 4.75 mm NMAS SMA mixture, the stone is that portion of the aggregate blend retained on the 1.18 mm (No. 16) sieve. The condition of stone-on-stone contact within a SMA mixture is defined as the point at which the voids in coarse aggregate (VCA) of the compacted mixture is less than the VCA of the coarse aggregate in the dry rodded test.

9.2 The VCA of the coarse aggregate only fraction (VCA_{DRC}) is determined by compacting the stone with the dry-rodded technique according to T19. When the dry-rodded density of the stone fraction has been determined, the VCA_{DRC} can be calculated using the following equation:

$$VCA_{DRC} = \frac{G_{CA}\gamma_w - \gamma_s}{G_{CA}\gamma_w} \times 100 \qquad (1)$$

where,

G_{CA} = bulk specific gravity of the coarse aggregate (T85)
γ_s = unit weight of the coarse aggregate fraction in the dry-rodded condition (kg/m³) (T19)
γ_w = unit weight of water (998 kg/m³)

10. Selection of Desired Gradation

10.1 After the trial samples have been compacted and allowed to cool, they are removed from the molds and tested to determine their bulk specific gravity in accordance with T166. The uncompacted samples are used to determine the theoretical maximum density in accordance with T209. Using the bulk specific gravity and theoretical maximum density, the percent air voids (V_a), VMA, and VCA of the compacted mixture (VCA_{MIX})can be calculated using the following equations:

$$VMA = 100 - \left(\frac{G_{mb}}{G_{sb}} \times P_s\right) \qquad (2)$$

$$V_a = 100 \times \left(1 - \frac{G_{mb}}{G_{mm}}\right) \qquad (3)$$

$$VCA_{MIX} = 100 - \left(\frac{G_{mb}}{G_{CA}} \times P_{CA}\right) \qquad (4)$$

where,

P_s = percent of aggregate in mixture
P_{CA} = percent coarse aggregate in the total mixture
G_{mb} = bulk specific gravity of the compacted mixture
G_{mm} = theoretical maximum density of mixture
G_{sb} = bulk specific gravity of the total aggregate
G_{CA} = bulk specific gravity of the coarse aggregate fraction

10.2 Of the three trial gradations evaluated, the one with the lowest percent of coarse aggregate that meets or exceeds the minimum VMA requirement, and has a VCA_{MIX} less than that determined by the dry-rodded technique (VCA_{DRC}) is selected as the desired gradation. Note that if possible, the selected gradation should have a VMA somewhat higher than 17 percent (typically at least 17.5 to 18.0 percent) to allow for some reduction in VMA during plant production. The trial gradation selected based on the above conditions is referred to as the optimum gradation.

11. Selection of Optimum Asphalt Binder Content

11.1 Once the optimum gradation of the mixture has been chosen, it may be necessary to raise or lower the asphalt binder content to obtain the proper amount of air voids in the mixture. In this case, additional samples are prepared using the selected gradation and varying the asphalt binder content. The optimum asphalt binder content is chosen to produce 3.0-4.0 percent air voids in the mixture. It is wise to use approximately 4 percent air voids as the design criteria in warmer U.S. climates while cooler climates could use a value closer to 3 percent. The VMA must be at least 17 percent.

11.2 The number of samples needed for this portion of the procedure is again twelve. This provides for three compacted and one uncompacted sample (used to determine the theoretical maximum density) at each of three asphalt binder contents. The mixture properties are again determined and the optimum asphalt content is selected to provide the desired void level. The SMA mixture selected should have properties meeting the criteria shown in Table 2 or Table 3 depending on the compaction method used. If these criteria are not met, the mixture must be modified so that requirements are met.

TABLE 2 SMA Mixture Specifications for Marshall Hammer Compacted Designs.

Property	Requirement
Air Voids, %	4.0 (NOTE 3)
VMA, %	17 min.
VCA_{MIX}, %	Less Than VCA_{DRC}
Stability, N[1]	6200 min.[1]
TSR, %	70 min.
Draindown @ Production Temperature, %	0.30 max.

[1] Successful SMA mixtures have been designed with Marshall Stability values below 6200 N, therefore this requirement can be waived based on experience.

TABLE 3 SMA Mixture Specifications for SGC Compacted Designs.

Property	Requirement
Air Voids, %	4.0 (NOTE 3)
VMA, %	17 min.
VCA_{MIX}, %	Less Than VCA_{DRC}
TSR, %	70 min.
Draindown @ Production Temperature, %	0.30 max.

NOTE 3 - For low traffic volume roadways or colder climates, air void contents less than 4.0 percent can be used, but should not be less than 3.0 percent.

11.3 To ensure that the SMA mixture has sufficient durability, a minimum asphalt content is required. This minimum asphalt binder content is based on volume of asphalt as a percentage of the aggregate volume and therefore changes as the combined bulk specific gravity of the aggregate changes. The minimum asphalt binder content for SMA mixtures should meet the requirements of Table 4.

TABLE 4 Minimum Asphalt Content Requirements for Aggregates With Varying Bulk Specific Gravities

Combined Aggregate Bulk Specific Gravity	Minimum Asphalt Content Based on Mass, %
2.40	6.8
2.45	6.7
2.50	6.6
2.55	6.5
2.60	6.3
2.65	6.2
2.70	6.1
2.75	6.0
2.80	5.9
2.85	5.8
2.90	5.7
2.95	5.6
3.00	5.5

12. Evaluation of SMA Mortars (Optional)

12.1 The SMA mortar is blended in accordance with the AASHTO draft test method, Laboratory Preparation and Testing of HMA Mortars.

12.2 The SMA mortar is tested at the high temperatures of the PG grade using the Dynamic Shear Rheometer (TP5)in accordance with MP1.

12.3 The SMA mortar is tested at the low temperature of the PG grade using the Bending Beam Rheometer (TP1) in accordance with MP1.

12.4 The results of SMA mortar testing should meet the criteria shown in Table 5. If acceptable mortar properties cannot be obtained the mixture must be modified so that requirements are met.

12.5 Mortar tests offer a method to better specify properties of the mortar. However, until more experience is available, mortar testing is optional.

TABLE 5 SMA Mortar Properties

Test	Requirement
Unaged DSR, G*/Sinδ (kPa)	5 min.
RTFO Aged DSR, G*/Sinδ (kPa)	11 min.
PAV Aged BBR, Stiffness (MPa)	1500 max.

13. **Moisture Susceptibility** - Moisture susceptibility of the selected mixture is determined using T283. The specimens for this test are compacted to an average air void content of 6±1 percent. The mixture should meet the applicable specification (70 percent minimum) indicated in Table 2 or Table 3.

14. **Draindown Sensitivity** - Draindown sensitivity of the selected mixture is determined using the AASHTO draft test method, Determination of Draindown Characteristics in Uncompacted Bituminous Mixtures. Draindown sensitivity is determined at the anticipated plant production temperature and should meet the applicable specification (0.30 percent maximum) indicated in Table 2 or Table 3.

15. **Adjusting Mixture To Meet Properties**

15.1 Air Void Content - The amount of air voids in the mixture can be controlled by the asphalt binder content. However, a problem occurs when low air void contents exist at asphalt binder contents below those specified in Table 4. Lowering the asphalt binder content below these values to achieve a proper amount of air voids violates the minimum asphalt content specification. Instead, the mixture gradation must be modified to increase the VMA.

15.2 Voids in Mineral Aggregate - The VMA may be raised by increasing the percentage of coarse aggregate. Changing the aggregate source may also be required to increase VMA.

15.3 Voids in the Coarse Aggregate - If the VCA_{MIX} in the mixture is higher than that in the dry-rodded coarse aggregate fraction (VCA_{DRC}) then the mixture gradation must be modified. This is typically accomplished by increasing the percentage of coarse aggregate.

15.4 Moisture Susceptibility - If the mixture fails to meet the moisture susceptibility requirements, hydrated lime or liquid anti-strip additives can be used. If these measures prove ineffective, the aggregate source and/or asphalt binder source can be changed to obtain better aggregate/asphalt binder compatibility.

15.5 Draindown Sensitivity - Problems with draindown sensitivity can be remedied by increasing the amount of stabilizing additive or by selecting a different stabilizing additive.

16. **Report**

16.1 The report shall include identification of the project name and project number.

16.2 The report shall include information on the materials used including: aggregate source, asphalt binder source and performance grade, type and amount of stabilizing additive, and material quality characteristics.

16.3 The report shall include results of the gradation optimization (results of trial gradations).

16.4 The report shall include the optimum gradation and optimum asphalt binder content.

16.5 The report shall include the volumetric properties for each trial blend and at optimum asphalt binder content.

16.6 The report shall include the moisture susceptibility results.

16.7 The report shall include the draindown sensitivity results.

Standard Specification for Designing Stone Matrix Asphalt (SMA)

AASHTO Format

Standard Specification for Designing Stone Matrix Asphalt (SMA)

AASHTO Format

1. Scope

1.1 This specification covers the design of Stone Matrix Asphalt (SMA) using either the Superpave Gyratory Compactor (SGC) or a mechanical, static base, flat-faced Marshall hammer. The SMA design is based on the volumetric properties of the SMA in terms of air voids, the voids in mineral aggregate, and the presence of stone-on-stone contact.

1.2 This standard specifies minimum quality requirements for asphalt binder, aggregate, mineral filler, and stabilizing additives for SMA mixture designs.

1.3 The values stated in SI units are to be regarded as the standard. The English units in parentheses are for information only.

1.4 *This standard does not purport to address all of the safety concerns, if any, 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 AASHTO Standards:

- T27 Sieve Analysis of Fine and Coarse Aggregates
- T84 Specific Gravity and Absorption of Fine Aggregate
- T85 Specific Gravity and Absorption of Coarse Aggregate
- T89 Determining the Liquid Limit of Soils
- T90 Determining the Plastic Limit and Plasticity Index of Soils
- T96 Resistance to Abrasion of Small Size Coarse Aggregate by Use of the Los Angeles Machine
- T100 Specific Gravity of Soils
- T104 Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate
- T245 Resistance to Plastic Flow of Bituminous Materials Using Marshall Apparatus
- T283 Resistance of Compacted Bituminous Mixtures to Moisture Induced Damage
- MP1 Specification for Performance Graded Asphalt Binder
- MP2 Specification for SUPERPAVE Volumetric Mix Design
- TP1 Test Method for Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR)
- TP4 Preparing and Determining the Density of Hot-Mix Asphalt Specimens by Means of the SHRP Gyratory Compactor
- TP5 Test Method for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)
- TP33 Uncompacted Void Content of Fine Aggregate as Influenced by Particle Shapes, Surface Texture, and Grading

2.2 ASTM Standards:
C612 Specification for Mineral Fiber Block and Board Thermal Insulation
D4791 Test for Flat or Elongated Particles in Coarse Aggregate
2.3 Asphalt Institute
MS-2 Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types
2.4 National Asphalt Pavement Association
IS101 "Guidelines on the Use of Baghouse Fines"

3. Terminology

3.1 SMA - stone matrix asphalt - SMA is a hot mix asphalt consisting of two parts, a coarse aggregate skeleton and a asphalt binder rich mortar. The mixture must have an aggregate skeleton with coarse aggregate-on-coarse aggregate contact (generally referred to as stone-on-stone contact). The coarse aggregate is generally considered to be that fraction of the aggregate retained on the 4.75 mm (No. 4) sieve but may be designated as other sizes. Sufficient mortar of the desired consistency must be provided. Satisfactory mortar consistency and thus good SMA performance requires that a relatively high asphalt binder content be used. For this reason the voids in the mineral aggregate (VMA).

3.2 Air voids (V_a) - the total volume of the small pockets of air between the coated aggregate particles throughout a compacted paving mixture, expressed as percent of the bulk volume of the compacted mixture (MS-2).

3.3 Voids in mineral aggregate (VMA) - the volume of intergranular void space between the aggregate particles of a compacted paving mixture that includes the air voids and the effective asphalt content, expressed as a percent of the total volume of the specimen (MS-2).

3.4 Voids in Coarse Aggregate (VCA) - the volume in between the coarse aggregate particles. This volume includes filler, fine aggregate, air voids, asphalt binder, and fiber (if used).

3.5 Nominal Maximum Aggregate Size - one size larger than the first sieve that retains more than 10 percent.

3.6 SMA Mortar - mixture of asphalt binder, filler, and stabilizing additive

3.7 Stabilizing Additive - polymer and/or fiber

4. Significance and Use - This standard may be used to evaluate material and mixture properties for SMA.

5. Asphalt Binder Requirements - The asphalt binder shall be a performance grade meeting the requirements of MP1 which is appropriate for the climate and traffic loading conditions at the site of the paving project. Guidance for the selection of the appropriate asphalt binder is provided in MP2.

6. Aggregate Requirements

6.1 Aggregates used within SMA shall conform to the requirements listed below.

6.1.1 *Coarse Aggregate* - Coarse aggregates shall be one-hundred percent crushed and conform to the quality requirements of Table 1.

TABLE 1 Coarse Aggregate Quality Requirements

Test	Method	Spec. Minimum	Spec. Maximum
Los Angeles Abrasion, % Loss	AASHTO T 96	-	[A]
Flat and Elongated, %			
3 to 1	ASTM D 4791	-	20
5 to 1	Section 8.4	-	5
Absorption, %	AASHTO T 85	-	2
Soundness (5 Cycles), %	AASHTO T 104		
Sodium Sulfate		-	15
Magnesium Sulfate		-	20
Crushed Content, %	ASTM D5821		
One Face		100	-
Two Face		90	-

[A] - Aggregates with L.A. Abrasion loss values up to 50 have been successfully used to produce SMA mixes. However, when the L.A. Abrasion exceeds approximately 30, excessive breakdown may occur in the laboratory compaction process or during in-place compaction.

6.1.2 Fine Aggregate - Fine aggregates shall be one-hundred percent crushed and conform to the quality requirements in Table 2.

TABLE 2 Fine Aggregate Quality Requirements

Test	Method	Spec. Maximum	Spec. Minimum
Soundness (5 Cycles), %	AASHTO T 104		
Sodium Sulfate		-	15
Magnesium Sulfate		-	20
Uncompacted Voids	AASHTO TP33, Method A	45	-
Liquid Limit, %	AASHTO T89	-	25
Plasticity Index, %	AASHTO T90	non-plastic	

7. **Mineral Filler** - Mineral filler shall consist of finely divided mineral matter such as crusher fines, lime, or fly ash. At the time of use, it should be sufficiently dry to flow freely and essentially free from agglomerations. Filler should be free from organic impurities and have a plasticity index not greater than 4. It is recommended that mineral fillers with modified Rigden voids (IS101) higher than 50 percent not be used in SMA. Experience has shown that fillers exceeding 50 percent excessively stiffen the SMA mortar.

8. **Stabilizing Additive** - A stabilizer such as cellulose fiber, mineral fiber, or polymer shall be added to the mixture. Dosage rate for cellulose shall be approximately 0.3 percent by total mixture mass and sufficient to prevent draindown. Cellulose used in SMA mixtures shall conform to the properties outlined in Table 3. For mineral fiber, the dosage rate shall be approximately 0.4 percent by total mixture mass and sufficient to prevent draindown. Mineral fibers used in SMA mixtures shall conform to the properties outlined in Table 4. The amount of polymer added will be that amount necessary to meet the required PG grade of asphalt binder.

TABLE 3 Cellulose Fiber Quality Requirements

Property	Requirement
Sieve Analysis	
Method A - Alpine Sieve[1] Analysis	
Fiber Length	6-mm (0.25 in.) Maximum
Passing 0.150-mm (No. 100 sieve)	70±10%
Method B - Mesh Screen[2] Analysis	
Fiber Length	6-mm (0.25 in.) Maximum
Passing 0.850 - mm (No. 20) sieve	85±10%
0.425 - mm (No. 40) sieve	65±10%
0.106 - mm (No. 140) sieve	30±10%
Ash Content[3]	18±5% non-volatiles
pH[4]	7.5±1.0
Oil Absorption[5]	5.0±1.0 (times fiber mass)
Moisture Content[6]	Less than 5% (by mass)

[1] Method A - Alpine Sieve Analysis. This test is performed using an Alpine Air Jet Sieve (Type 200 LS). A representative five gram sample of fiber is sieved for 14 minutes at a controlled vacuum of 75 kPa (11 psi) of water. The portion remaining on the screen is weighed.
[2] Method B - Mesh Screen Analysis. This test is performed using standard 0.850, 0.425, 0.250, 0.180, 0.150, and 0.106-mm sieves, nylon brushes, and a shaker. A representative 10 gram sample of fiber is sieved, using a shaker and two nylon brushes on each screen. The amount retained on each sieve is weighed and the percentage passing calculated. Repeatability of this method is suspect and needs to be verified.
[3] Ash Content. A representative 2-3 gram sample of fiber is placed in a tared crucible and heated between 595 and 650°C (1100 and 1200°F) for not less than two hours. The crucible and ash are cooled in a desiccator and weighed.
[4] Ph Test. Five grams of fiber is added to 100 ml of distilled water, stirred and let sit for 30 minutes. The Ph is determined with a probe calibrated with Ph 7.0 buffer.
[5] Oil Absorption Test. Five grams of fiber is accurately weighed and suspended in an excess of mineral spirits for not less than 5 minutes to ensure total saturation. It is then placed in a screen mesh strainer (approximately 0.5 mm^2 opening size) and shaken on a wrist action shaker for 10 minutes (approximately 32-mm (1 1/4 in) motion at 240 shakes per minute). The shaken mass is then transferred without touching to a tared container and weighed. Results are reported as the amount (number of times its own weight) the fibers are able to absorb.
[6] Moisture Content. Ten grams of fiber is weighed and placed in a 121°C (250°F) forced air oven for two hours. The sample is then re-weighed immediately upon removal from the oven.

TABLE 4 Mineral Fiber Quality Requirements

Property	Requirement
Size Analysis	
Fiber Length[1]	6-mm (0.25in) Maximum mean test value
Thickness[2]	0.005-mm (0.0002 in) Maximum mean test value
Shot Content[3]	
Passing 0.250-mm (No. 60) sieve	90±5%
Passing 0.005-mm (No. 230) sieve	70±10%

[1] The fiber length is determined according to the Bauer McNett fractionation.
[2] The fiber diameter is determined by measuring at least 200 fibers in a phase contrast microscope.
[3] Shot content is a measure of non-fibrous material. The shot content is determined on vibrating sieves. Two sieves, 0.250 and 0.063 are typically utilized. For additional information see ASTM C 612.

9. **SMA Mortar Properties (Optional Requirements)** - SMA mortars prepared using method for Laboratory Preparation and Testing of HMA Mortars, at design percentages (based on mass) shall conform to the quality requirements shown in Table 5.

TABLE 5 SMA Mortar Quality Requirements

Test	Test Method	Property	Specification
Original Mortar			
Dynamic Shear	TP 5	G*/Sinδ	5.0 kPa min.
RTFO Aged Mortar			
Dynamic Shear	TP 5	G*/Sinδ	11.0 kPa min
PAV Aged Mortar			
Bending Beam	TP 1	S	1500 MPa Max.

10. **SMA Design Requirements**
    10.1 The combined aggregates shall conform to the gradation requirements of Table 6. These gradations are based on percent by volume passing the respective sieves.
    10.2 The designed SMA mixture shall meet the properties of Tables 7 or 8 depending on the compaction method used. Table 7 presents required properties of SMA mixtures designed using 50-blows of a flat-faced, static base, mechanical Marshall hammer (T245) while Table 8 presents required properties of SMA mixtures designed with 100 gyrations of the SGC. (NOTE 1)

NOTE 1 - When aggregates having a Los Angeles Abrasion loss value greater than 30 percent or the design ESALs are less than 1 million, the number of gyrations of the SGC should be 70 for design. Required properties from Table 8 also apply to SMA mixtures designed with 70 gyrations.

TABLE 6 Stone Matrix Asphalt Gradation Specification Bands (Percent Passing by Volume) [1].

Sieve, mm	25 mm NMAS[2]		19 mm NMAS		12.5 mm NMAS		9.5 mm NMAS		4.75 mm NMAS	
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
37.5	100	100								
25.0	90	100	100	100						
19.0	30	86	90	100	100	100				
12.5	26	63	50	74	90	100	100	100		
9.5	24	52	25	60	26	78	90	100	100	100
4.75	20	28	20	28	20	28	26	60	90	100
2.36	16	24	16	24	16	24	20	28	28	65
1.18	13	21	13	21	13	21	13	21	22	36
0.6	12	18	12	18	12	18	12	18	18	28
0.3	12	15	12	15	12	15	12	15	15	22
0.075	8	10	8	10	8	10	8	10	12	15

[1] - An example problem illustrating how to blend gradations based on volume can be found later in this report.
[2] NMAS - Nominal Maximum Aggregate Size- one sieve size larger than the first sieve that retains more than 10 percent.

TABLE 7 SMA Mixture Specifications for Marshall Hammer Compacted (T245) Designs.

Property	Requirement
Air Voids, %	4.0 (NOTE 2)
VMA, %	17 min.
VCA_{MIX}[2], %	Less Than VCA_{DRC}[2]
Stability, N[1]	6200 min.[1]
TSR, %	70 min.
Draindown @ Production Temperature, %	0.30 max.

[1] Successful SMA mixtures have been designed with Marshall Stability values below 6200 N, therefore this requirement can be waived based on experience.
[2] VCA_{MIX} and VCA_{DRC} are defined in the, Standard Practice for Designing Stone Matrix Asphalt (SMA).

TABLE 8 SMA Mixture Specifications for SGC Compacted (TP4) Designs.

Property	Requirement
Air Voids, %	4.0 (NOTE 2)
VMA, %	17 min.
VCA_{MIX}, %	Less Than VCA_{DRC}
TSR, %	70 min.
Draindown @ Production Temperature, %	0.30 max.

NOTE 2 - For low traffic volume roadways or colder climates, air void contents less than 4.0 percent can be used, but should not be less than 3.0 percent.

10.3 The minimum asphalt binder content for designed SMA mixtures shall meet the requirements of Table 9.

TABLE 9 Minimum Asphalt Content Requirements for Aggregates With Varying Bulk Specific Gravities

Combined Aggregate Bulk Specific Gravity	Minimum Asphalt Content, %
2.40	6.8
2.45	6.7
2.50	6.6
2.55	6.5
2.60	6.3
2.65	6.2
2.70	6.1
2.75	6.0
2.80	5.9
2.85	5.8
2.90	5.7
2.95	5.6
3.00	5.5

10.4 The retained tensile strength level of the SMA shall be at least 70 percent at 6±1 percent air voids and tested in accordance with T283. When the compaction effort utilized is the SGC, the specimen shall be 150 mm in diameter and 95 mm in height.

10.5 Draindown sensitivity shall be determined on the designed SMA mixture in accordance with the, Determination of Draindown Characteristics in Uncompacted Bituminous Mixtures. Draindown sensitivity shall be determined at the anticipated plant production temperature and shall not exceed 0.3 percent.

Standard Practice for the Construction of Stone Matrix Asphalt (SMA)

AASHTO Format

Standard Practice for the Construction of Stone Matrix Asphalt (SMA)

AASHTO Format

1. **Scope**

1.1 This practice provides guidance for the three main construction phases of SMA: plant production, placement and compaction procedures, and quality control/quality assurance (QC/QA). Plant production encompasses those procedures performed at the plant to manufacture the mixture. Examples of such procedures are aggregate handling, mixing times, and plant calibration. Placement and compaction procedures involve everything from transporting the mixture to placing the mixture to compaction of the mixture. Quality control/quality assurance procedures are provided to ensure that the mixture produced and placed in the field actually meets the specifications. This standard addresses issues inherent with the production of SMA mixtures. The suggestions discussed herein should be used in conjunction with acceptable construction and QC/QA practices.

1.2 The values stated in SI are to be regarded as the standard. The English units in parentheses are for information only.

1.3 *This practice does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this practice to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. **Referenced Documents**

2.1 AASHTO Standards:

T27 Sieve Analysis of Fine and Coarse Aggregates
T164 Quantitative Extraction of Bitumen from Bituminous Paving Mixtures
T166 Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface-Dry Specimens
T209 Maximum Specific Gravity of Bituminous Paving Mixtures
T245 Resistance to Plastic Flow of Bituminous Materials Using Marshall Apparatus
T269 Percent Air Voids in Compacted Dense and Open Bituminous Paving Mixtures
TP4 Preparing and Determining the Density of Hot-Mix Asphalt Specimens by Means of the SHRP Gyratory Compactor
TP53 Test Method for Determining the Asphalt Content of Hot Mix Asphalt (HMA) by Ignition Method

3. **Terminology**

3.1 Stone Matrix Asphalt (SMA) - SMA is a hot mix asphalt consisting of two parts, a coarse aggregate skeleton and an asphalt binder rich mortar. The mixture must have an aggregate skeleton with coarse aggregate-on-coarse aggregate contact (generally referred to as stone-on-stone contact). The coarse aggregate is generally considered to be that fraction of the aggregate retained on the 4.75 mm (No. 4) sieve but may be designated as other sizes. Sufficient mortar of the desired consistency must be provided. Satisfactory mortar consistency and thus good SMA performance requires that a relatively high asphalt binder content to be used. For this reason, the voids in mineral aggregate (VMA) must exceed some minimum requirement.

3.2 SMA Mortar - a mixture of filler, asphalt binder, and stabilizing materials that binds the coarse aggregate matrix in SMA and fills the voids in the matrix.

4. Plant Production

4.1 Materials

4.1.1 Aggregates - To obtain the coarse aggregate-on-coarse aggregate contact inherent in SMA, the mixture must contain a high percentage of coarse aggregate. While it is typical to blend two or three different aggregate stockpiles in the mixture (coarse aggregate, intermediate aggregate, and fine aggregate), the coarse aggregate (defined as the material retained on a 4.75-mm (No.4) sieve) content is usually 72-80 percent of the blend. Thus the gradation of the coarse aggregates can have a tremendous effect on the quality of the mixture produced. Therefore, it is imperative that the aggregates be carefully handled and stockpiled. Each coarse aggregate stockpile may need to be fed through more than one cold feeder since a high percentage of material is being fed. Using more than one feeder will also minimize variability in the gradation of coarse aggregate stockpiles.

4.1.2 Mineral Filler - Mineral filler, other than that naturally occurring in the aggregate, is not routinely used today within HMA produced in the United States. By contrast, SMA gradation specifications usually require approximately 10 percent passing the 0.075-mm (No. 200) sieve. Even with the return of baghouse fines, this high fines content requirement typically means that at least 5 percent commercial mineral filler must be added to the mixture. For most SMA construction projects, the ability to add commercial mineral filler to the mixture has governed the plant production rate. For example, a hot mix plant producing at the rate of 275 Mg per hour (300 tons per hour) will need the mineral filler delivered at the approximate rate of 14 Mg per hour (15 tons per hour) or higher. In addition, SMA mixtures are very sensitive to the mineral filler content. Handling, storing, and introduction of the mineral filler into SMA mixtures is therefore an important concern. The best method of handling and storing mineral filler is in bulk quantities. With this method the filler can be purchased, shipped, and stored in bulk, thus reducing cost. To do this, a mineral filler silo is necessary at the plant site. These silos are closed systems in which the filler is stored until needed. To deliver the filler to the plant, a vane feeder containing an airlock system or auger system is attached. These systems meter the proper amount of filler. A blower is mounted beneath the feeder system to blow the filler into the pugmill or weigh hopper of a batch plant or the drum of a drum mix plant. These methods are heavily taxed because of the high amount of filler required but have given excellent results as long as the feed system is properly calibrated. The bulk mineral filler should generally not be added by conveyor belt from the cold feeder or from the RAP feeder. The rate of feed cannot be closely controlled using this conveyor belt/feeder method. It may also create a dust problem during windy weather conditions. It is imperative that the filler be captured by asphalt binder or coarser aggregate as soon as it is added to the mixture. In a batch plant this is normally done by adding fine or coarse aggregate to the pugmill immediately before and after introduction of the filler. In a drum plant, the mineral filler line into the drum should be next to the asphalt binder line so that the filler is coated with asphalt binder before it is exposed to the high velocity gas flow through the drum.

Introduction in this manner helps keep the filler in the mixture rather than loosing it to the fines recovery system.

4.1.3 Liquid Asphalt - The handling and storage of liquid asphalt binder for SMA production is similar to that for any HMA mixture. When modified asphalt binders are used, typically the storage temperatures may increase slightly from those of neat asphalt binders. However, contractors should follow the manufacturers recommendations for circulation and storage of modified asphalts. Metering and introduction of the asphalt binder into the mixture may be done by any of the standard methods using a temperature compensating system. It is very important however that the asphalt binder be metered accurately.

4.1.4 Stabilizing Additives - Due to the high asphalt binder contents in the SMA mortar, a stabilizing additive of some type must be used to hold the mortar on the coarse aggregate during hauling and placement. If a stabilizing additive is not used the asphalt binder will drain from the mix during haul and placement resulting in fat spots in the finished pavement. Two types of stabilizing methods have been used. One method is the use of fibers such as cellulose or mineral fibers. The second stabilizing method is to modify the asphalt binder in some manner. This may be done by modifying the asphalt binder at the refinery or by adding an asphalt binder modifier to the SMA mixture during production. Some projects have utilized both a fiber and a modified asphalt binder.

4.1.4.1 *Fibers* - Both cellulose and mineral fibers have been used. Typical dosage rates are 0.3% for cellulose and 0.4% for mineral fiber by total mixture mass. Fibers can generally be purchased in two forms, loose fibers and pellets. Fibers in a dry, loose state come packaged in plastic bags or in bulk. Both fiber types have been added into batch or drum mix plants with good success. For batch plant production, loose fibers are sometimes delivered to the plant site in bags. The bags are usually made from a material which melts easily at mixing temperatures. The bags can therefore be added to the pugmill during each dry mix cycle. When the bags melt only the fiber remains. Addition of the bags of fibers is done by workers on the pugmill platform. At the appropriate time in every dry mix cycle, the workers add the correct number of bags to the pugmill. The bags of fiber can be elevated to the pugmill platform by the use of a conveyor belt. While this method of manual introduction works satisfactorily, it is labor intensive.

Another method for addition of fibers into a batch plant is by blowing them into the plant using a machine typically designed and supplied by the fiber manufacturer. The dry, loose fiber is placed in the hopper of the machine where it is fluffed by large paddles. The fluffed fiber next enters the auger system which conditions the material to a known density. The fiber is then metered by the machine and blown into the pugmill or weigh hopper at the appropriate time. These machines can meter in the proper amount of fiber by mass or blow in a known volume. This fiber blowing method can also be used in a drum mix plant. The same machine is used and the fibers are simply blown into the drum. When using this method in a drum mix plant, the fiber line may be placed in the drum beside the asphalt binder line and merged into a mixing head. This allows the fibers to be

captured by the asphalt binder before being exposed to the high velocity gases in the drum. If the fiber does get into the gas stream in large quantities, it will enter the dust control system of the plant.

The pelletized form of fibers can be used in both drum mix and batch plants. The pellets are shipped to the plant in bulk form and when needed are placed in a hopper. From the hopper they can be metered and conveyed to the drum or pugmill via a calibrated conveyor belt. Addition of the pellets occurs at the RAP collar of the drum mix plant or they are added directly into the pugmill of a batch plant. Here the pellets are mixed with the aggregate where the heat from the aggregates causes the asphalt binder in the pellets to become fluid allowing the fiber to mix with the aggregate.

The pelletized fibers do contain a given amount of asphalt binder that must be accounted for in the overall asphalt content of the mixture. Check with the fiber manufacturer to determine the asphalt contents of the pellets. It is again imperative that the fiber addition, whether it be bulk or pelletized, be calibrated to ensure that the mixture continually receives the correct amount of fiber. If the fiber content is not accurately controlled at the proper level, fat spots will almost certainly result on the surface of the finished pavement. For assistance with the fiber storage, handling, and introduction into the mixture, the fiber manufacturer should be consulted.

4.1.4.2 *Asphalt Cement Modifiers* - Another method of providing stabilization to SMA is with the use of asphalt binder modifiers. The asphalt binder in SMA can be modified at the refinery, or, in some cases, the modifier is added at the hot mix plant. For the first method, the hot mix producer takes delivery of the modified asphalt binder and meters it into the SMA mixture in a traditional manner. Special storage techniques and/or temperatures may be required. With the second method, the contractor must ensure that the proper amount of modifier is added and thoroughly mixed with the asphalt binder.

When an asphalt binder modifier is added at the hot mix plant, two different methods are utilized. The modifier is either blended into the asphalt binder before it is injected into the mixture or it is added directly to the dry aggregates during production. Addition of the modifier to the asphalt binder is accomplished by in-line blending or by blending the two in an auxiliary storage tank. If the modifier is added to the aggregates rather than the asphalt binder, it can be added directly into the pugmill or, in a drum mix plant, it can be delivered to the drum via the RAP delivery system. Use of the RAP belt weigh bridge is not recommended because of poor accuracy and a special metering device may be necessary if the RAP feeder cannot be calibrated.

When a modifier is added directly into the plant and not premixed with the asphalt cement, it is impossible to measure the properties of the modified asphalt binder. The properties of the modified asphalt binder can be estimated in the laboratory by mixing the desired proportion of the asphalt cement and modifier and testing. Regardless of the form of stabilization, advice and assistance should be sought from the stabilizer supplier. It is

imperative that the system used to add the modifier be calibrated to ensure the mixture receives the proper dosage.

4.2 Mixture Production - Production of SMA is similar to production of standard HMA from the standpoint that care should be taken to ensure a quality mixture is produced. Production of SMA is discussed in this section with special emphasis on production areas where SMA quality may be significantly affected.

4.2.1 Plant Calibration - It is important that all the feed systems of the plant be carefully calibrated prior to production of SMA. The aggregate cold feeds can make a large difference in the finished mixture even in a batch plant where hot bins exist. Calibration of the aggregate cold feed bins should therefore be performed with care.

The stabilizing additive delivery system should be calibrated and continually monitored during production. Whether fibers, an asphalt binder modifier, or both are being utilized, variations in the amount of additive can have a detrimental impact on the finished pavement. Stabilizing additive manufacturers will usually assist the hot mix producer in setting up, calibrating, and monitoring the stabilizing additive delivery system.

Two very important, interrelated systems in SMA production are the mineral filler feed system and the dust collection/return system. These two systems must be working properly in order to ensure quality in the mixture. If the mineral filler is not being delivered to the mixture in proper quantities or in a proper manner, it can find its way into the fines recovery system. This can plug the system or at the very least cause an improper amount of fines to be added to the SMA mixture. If the fines recovery system completely removes fines from the plant, a wet collector for example, then the mineral filler feed system should be calibrated for any loss of fines that may be removed by the wet collector. As stated earlier, for most SMA projects, the ability to add mineral filler to the mixture has governed mix production rates.

4.2.2 Plant Production Temperature - Production temperatures of SMA mixtures vary somewhat according to aggregate moisture contents, weather conditions, grade of asphalt binder and type of stabilizing additive used. However, experience indicates that normal HMA production temperatures or slightly higher are adequate. Experience also shows less fuel demand to dry SMA aggregates at the same temperature as typical HMA. Typically, a temperature of 145-155°C (293-310°F) can be used when a polymer is not included. Temperatures higher than this may be needed on some occasions, such as when a polymer modifier is added, but should be used with caution as rapid oxidation begins to occur at higher temperatures. The SMA mixture should never be heated above 176°C (350°F) since this may excessively damage the asphalt binder and may increase plant emissions. As the mixture temperature is increased, the chance of the mortar draining from the coarse aggregate also increases. The temperature should be chosen to ensure a uniform mixture that allows enough time for transporting, placing, and compaction of the mixture.

4.2.3 Mixing Time - When adding fibers to the SMA mixture, experience has shown that the mixing time should be increased slightly over that of conventional HMA. This additional time allows for the fiber to be sufficiently distributed in the mixture. In a batch plant this requires that both the dry and wet mix cycles be increased from 5 to

15 seconds each. In a parallel flow drum mix plant, the asphalt binder injection line may be relocated, usually extended, if necessary to provide improved mixing. When blowing fibers into a drum mix plant, it is imperative that the fiber line be placed in the drum beside the asphalt binder line and merged into a mixing head.

The proper mixing times can be estimated by visual inspection of the mixture. If clumps of fibers or pellets still exist intact in the mixture at the discharge chute, or if aggregate particles are not sufficiently coated, mixing times should be increased or other changes made. For other plants such as double barrels and plants with coater boxes, the effective mixing time can be adjusted in a number of ways including reduction in production rate, slope reduction of the drum, etc.

4.2.4 Mixture Storage - The SMA mixture should not be stored at elevated temperatures for extended periods of time. This could facilitate draindown. In general, experience has shown that SMA can be stored for 2-3 hours without detriment. Storage time in excess of 12 hours should not be allowed.

5. Placement and Compaction Procedures

5.1 Weather Limitations - In order to achieve proper placement and compaction, SMA should not be placed in cold or inclement weather. It is recommended that a minimum pavement temperature of 10°C (50°F) be achieved prior to placement of SMA mixture. However, the ability to place SMA will also depend on wind conditions, humidity, the lift thickness being placed and the temperature of the existing pavement.

5.2 Mixture Transportation - Haul times for SMA should be as short as possible. It is important that the temperature of the SMA mixture not be raised arbitrarily high in order to facilitate a longer haul time. The increased temperature in coordination with the vibration provided during haul can serve to separate the mortar from the coarse aggregate. The mixture should arrive at the paving site so that it is placed at temperatures of approximately 140-150°C (280-300°F). Higher temperatures may be needed if polymer modifiers are used. These temperatures help ensure that proper compaction is achieved.

Due to its thick mortar, SMA may adhere to truck beds somewhat more than conventional HMA mixtures. This is particularly true when asphalt binder modifiers are employed in the mixture. It is therefore prudent to use a release agent and clean the truck beds frequently. Most agencies have approved lists of release agents. However, if not carefully utilized, these agents may cause problems with SMA mixtures. If the agent is allowed to pool in the bottom of the truck it may cause the mortar to flush from the SMA pavement surface. The bed of the truck should be coated and the excess agent removed before loading. This can be accomplished by raising the truck bed after the agent has been sprayed into the truck. Any excess agent will then be discharged. Use of fuel oils in any form should be strictly prohibited.

5.3 Pavement Surface Preparation - When placing SMA, preparation of the surface to be covered will depend on the type of surface; this preparation is generally the same as for conventional HMA. SMA is normally applied in any of several situations: over an old HMA pavement, over an old Portland cement concrete pavement, over rubblized Portland cement concrete, or over a new HMA binder course. Some states have also placed SMA surface courses over SMA binder courses.

If an old pavement surface is to be covered by SMA then proper repair should first be performed. Areas containing large permanent deformations should be milled or filled using a leveling course. If the condition of the in-place mix is sufficiently bad it may have to be removed to some predetermined depth. Any distressed areas should be properly repaired. While SMA has shown superior performance, it cannot be expected to perform long term when it is used to cover up existing pavement problems. For old and new surfaces, a tack coat should be used. Types of materials and their application rates need not vary from that of conventional HMA construction.

5.4 Paver Operation

5.4.1 Charging the Paver - The SMA mixture is normally delivered to the paver in the traditional manner of backing in trucks or by means of a windrow elevator. A material transfer vehicle (MTV) can also be used for SMA.

5.4.2 Paver Calibration - Prior to placement of the SMA, the paver should be correctly calibrated. This is no different than when placing conventional HMA and involves the flow gates, the slat conveyors, and the augers. The flow gates should be set to allow the slat conveyors to deliver the proper amount of mixture to the augers so that the augers turn 85-90 percent of the time.

5.4.3 Paving Speed - When placing SMA, the paving speed is for the most part dictated by the ability of the rolling operation to compact the mixture. It is critical that the plant production, mixture delivery, placement, and compaction be coordinated so that the paver does not have to continually stop and start. Paver stops and starts should be held to an absolute minimum because they will likely have a significant negative impact on the ride. In addition to continuous paver movement, the SMA mixture delivery and paver speed should be calibrated so that the augers can be kept turning 85-90 percent of the time. This helps ensure the slowest possible speed for the augers. Running the augers very fast for short periods of time should be avoided. The high auger speed may have a tendency to shear the mortar from the coarse aggregate thus causing fat spots in the pavement. Generally, the paver wings should not be lifted except when the material is to be discarded.

5.4.4 Lift Thickness - The majority of SMA pavements have been placed 40-mm (1-1/2 inches) thick. It is imperative that the construction inspector not try to balance yield by adjusting the thickness of the SMA lift. This can cause unsatisfactory pavements to be built. A tolerance of ± 6-mm (1/4 inch) in the lift thickness is allowable.

5.4.5 Placement and Finishing - Immediately behind the paver, SMA mixtures are known to be harsh and very sticky. For this reason a minimum of raking and hand working should be performed. When needed, hand placement of the material can be accomplished with care.

5.5 Rolling - It is recommended that a target density of 95 percent and a minimum density of 94 percent of maximum theoretical density be achieved for the completed mat. To achieve this goal, densification of the SMA mixture should be accomplished as quickly as possible after placement. By its very nature SMA compacts readily when at proper mix temperature and immediately behind the paver. However, SMA becomes difficult to compact once it begins to cool. For this reason it is imperative that the rollers be kept immediately behind the paver.

Polymer modified SMA has a more narrow temperature window than does fiber stabilized SMA. Polymer mixtures should generally be completely compacted before they cool below 127°C (260°F).

Rolldown of SMA mixtures is slightly more than one-half that for conventional HMA mixtures. While conventional HMA mixtures rolldown approximately 20-25 percent of the lift thickness, SMA will normally rolldown 10-15 percent of the lift thickness.

Breakdown rolling should begin immediately behind the paver and the roller should stay close behind the paver at all times. Experience has shown that two rollers working in tandem running straight forward and back provides very effective breakdown rolling. If the rolling operation gets behind, placement of SMA should slow down until the rollers catch up with the paver. Two or three rollers are typically used in conventional HMA construction. This number normally serves well for SMA also. If density cannot be achieved quickly enough then an additional roller should be added to the project.

A roller pattern should be established and followed that provides a density that meets the specification requirements. Steel wheeled rollers weighing at least 9 Mg (10 tons) should be used when compacting the SMA mixture. Roller speed should not exceed 5 km/hr (3 miles/hr) and the drive roll should be kept nearest the paver. Six to eight passes of the breakdown rollers should be sufficient. If it becomes necessary for the rollers to sit idle they should be taken off the mat if possible. Idle rollers sitting on the mat can cause unnecessary roughness in the finished surface.

It is normal practice to mix a minimum amount of release agent with the water in the roller drum to prevent the asphalt binder from sticking to the drum. Excessive amounts of water should not be used. Most experience with vibratory rollers to date indicates that they can be used successfully on SMA. If a vibratory is allowed, it must not excessively break aggregate and/or force the mortar to the surface of the mat. Vibratory rollers should be used in a high frequency, low amplitude mode. Vibratory rollers have been used on SMA projects with success, but their performance must be closely monitored during compaction. The use of more passes in the vibratory mode should not be used to replace the need for another roller. Pneumatic-tired rollers are not recommended for use on SMA. The rubber tires tend to pick up the mortar causing surface deficiencies.

6. Quality Control/Quality Assurance

6.1 Aggregates - As with conventional HMA, the producer should periodically monitor the aggregate stockpiles being used for the production of the SMA mixture. Stockpile gradations can change as additional material is added to the stockpile during mixture production. Even if the stockpiles do not receive additional aggregates during mixture production, their gradations may change due to stockpiling and/or load out procedures. Therefore the monitoring program must be frequent enough to warn the producer that a change has taken place before a significant amount of the aggregate has been used in SMA mixtures. In batch plant operations, hot bin analyses should also be performed. This testing serves as a further check of aggregate gradations. In both batch and drum mix plants the cold feed gradations should be monitored. Variations or deviations of aggregate gradation from the specified job-mix-formula are often more critical to the performance of SMA mixtures than they are for HMA. Therefore, close control of gradation must be accomplished for these mixtures.

6.2 Asphalt Binder - The asphalt binder used in the SMA should be tested as it is for any conventional HMA project. Some modified asphalt binders may require special testing techniques. The SMA mortar properties (binder, filler, and stabilizing additive) can be evaluated using the Superpave binder tests. This is not recommended for field control but can be useful in the mix design process.

6.3 Trial Sections - Prior to full scale production and placement of SMA mixtures, a trial section of the mixture should be produced and placed by the contractor. This trial section should be at the actual construction site and should be at least two paver widths wide. The trial section should consist of between 200 and 500 Mg (220 and 550 tons) of mixture. The length of the trial section will depend upon the capacity of the plant and other variables in the mixture production and placement. However, the trial section needs to be of sufficient size to allow the plant components to operate to the point of producing consistent mixture. The trial section is a good opportunity to determine any proportioning problems with the final job-mix-formula (JMF). The trial section should also be used to determine the proper rolling pattern to be used. However, if the trial section is placed at a location other than the actual site to be paved during full scale production, care should be taken in drawing conclusions about achieved densities due to the possibility of dissimilar base characteristics. The trial section should be constructed in advance of the production paving so as to allow for testing and adjustment in the JMF and to allow for a second trial section if major adjustments need to be made.

6.4 Mixture Sampling - Most agencies have established their own requirements for where and how mixture sampling must be done. SMA should be sampled according to these recognized procedures. Experience has shown that quartering of SMA can be difficult due to its tendency to stick to the tools thus potentially causing a low asphalt content to be measured. Frequency of sampling and testing is usually established by the owner. As a minimum, at least two test series per day (gradations, asphalt contents, volumetrics, draindown, and in-place density) should be performed. More frequent testing is advisable in order to maintain good quality control. Many agencies divide the mixture into lots and sublots and require two or three test series per lot. In addition, the time at which samples are taken should be obtained randomly so as not to bias the results.

6.5 Mixture Tests -Certain test data on the mixture must be collected to allow the producer of SMA to control the mixture as well as to allow the owner the ability to accept or reject the mixture. These tests are generally similar to those performed on conventional HMA.

6.5.1 *Laboratory Compacted Specimens* - Laboratory compacted specimens should be examined for compliance with volumetric properties established for the mixture. These tests consist of compacting specimens using 50 blows of a static base, flat-faced Marshall hammer as described in AASHTO T245, or with 100 gyrations of the Superpave Gyratory Compactor (SGC) (TP4). (70 gyrations when L.A. Abrasion loss is above 30 percent or when design traffic level is below 1 million ESALs) The bulk specific gravity of the specimens can be determined by AASHTO T166, while the maximum theoretical specific gravity is determined by AASHTO T209. Air voids may then be determined according to the method described for dense-graded bituminous mixtures in AASHTO T269. The resulting air voids should be within the specified range, typically between 3-4 percent, and VMA should meet or exceed the specified minimum requirements. SMA mixtures become permeable to water at a lower air void level than that for dense-graded mixtures. When the in-place air voids in SMA are approximately 6-7 percent or greater the mixture is permeable to water. A method for measuring the density of porous mixtures should be used at these higher air void levels. If this is not done, a lower air void content will be measured than actually exists in the mixture.

6.5.2 *Asphalt Content and Gradation* - The stabilizing additives used in SMA can sometimes hinder the extraction process and some experimentation may be necessary to determine the optimum method of extracting the mixture. Most agencies allow one or more of any of the methods discussed in AASHTO T164. Note that while Method B is very reliable, it is generally not suited for field work due to the length of time needed for the test. The asphalt content by ignition method (TP53) has also been shown to work well for determining the asphalt binder content of SMA. After removing the asphalt binder the aggregate should be graded according to AASHTO T27. The resulting gradation and asphalt content should meet the JMF established for the mixture within the tolerance limits specified. Typical tolerance limits for gradations are shown in Table 1.

TABLE 1: Gradation Tolerances for Extracted SMA Samples.

Sieve Size	Percent Passing Tolerance
19.0-mm (3/4 Inch)	± 4.0
12.5-mm (½ Inch)	± 4.0
9.5-mm (3/8 Inch)	± 4.0
4.75-mm (No. 4)	± 3.0
2.36-mm (No. 8)	± 3.0
0.60-mm (No. 30)	± 3.0
0.30-mm (No. 50)	± 3.0
0.075-mm (No.200)	± 2.0
Asphalt Content (%)	± 0.3

6.5.3 In-Place Density - In-place density of the mixture should be checked continually throughout pavement construction to ensure that proper density is being obtained. Current density specifications normally call for a minimum density of 94 percent of theoretical maximum density. However, a good target density for SMA is 95 percent of theoretical maximum density. The pavement density can be monitored using a nuclear density gauge. The nuclear gauge is not as accurate for SMA pavements as it is for conventional HMA pavements because of SMA's rough surface texture. It should therefore be calibrated and periodically checked by comparison to cores taken from the pavement. The nuclear gauge density locations, as well as the coring locations, should be randomly chosen. Care should be exercised when measuring core densities due to the higher permeability of SMA at lower air void content as discussed previously.

6.5.4 *Draindown Test* - Since the SMA mixture must have some stabilizing additive to prevent draindown of the mortar from the coarse aggregate, mixture samples should also routinely be checked for compliance with draindown requirements

(Determination of Draindown Characteristics in Uncompacted Asphalt Mixtures). The draindown test should be performed at the plant mixing temperatures. When adding fiber or polymer, draindown can result when either is added at an improper proportion. Generally, a fiber will do a better job of preventing draindown than a polymer. There is no known test to measure the amount of fiber in a mixture. Hence there needs to be a method to visually observe the fiber being added into the mixture so that it can be confirmed that fiber is actually being added. This approach will not allow measurement of the fiber content, but it does at least let the inspector know that fiber is being added. The draindown test cannot quantify the amount of fiber in the mixture but it does provide a good indicator of the effects fiber have in draindown.

6.6 Troubleshooting - Table 2 provides guidance for possible problems encountered during the construction of SMA.

TABLE 2: Troubleshooting Chart

Observed Problem	Likely Cause	Corrective Action
Low VMA	Percentage Passing 4.75 mm too high Percentage Passing 0.075 mm too high Excessive breakdown of aggregate Incorrect bulk specific gravity of aggregate	1.Verify accuracy of results 2. Reduce the percentage passing the 4.75 mm and/or 0.075 mm sieves
High VMA	Percentage Passing 4.75 mm too low Percentage Passing 0.075 mm too low Incorrect bulk specific gravity of aggregate	1.Verify accuracy of results 2. Increase the percentage passing the 4.75 mm and/or 0.075 mm sieves
Low Voids	Low VMA High Asphalt Content	1. Verify accuracy of results 2. Decrease the asphalt content or increase VMA
High Voids	High VMA Low Asphalt Content	1. Verify accuracy of results 2. Increase the asphalt content or decrease VMA
High VCA	High Percentage Passing 4.75 mm Incorrect bulk specific gravity of aggregate	1. Verify accuracy of results 2. Decrease percentage passing 4.75 mm
Increased Stiffness in Mortar Properties	Increased Binder Stiffness Increased Amount of Filler Increased Filler Fineness	1. Verify accuracy of results 2. Decrease filler content 3. Use coarser filler
Decreased Stiffness in Mortar Properties	Decreased Binder Stiffness Decreased Amount of Filler Increased Filler Coarseness	1. Verify accuracy of results 2. Increase filler content 3. Use finer filler

TABLE 2: Troubleshooting Chart

Observed Problem	Likely Cause	Corrective Action
High Draindown (test)	Increased Temperature Decreased Filler Content Insufficient Stabilizer Increased Percentage of Coarse Aggregates Moisture in Mix	1. Verify accuracy of results 2. Increase stabilizer content 3. Change type of stabilizer 4. Reduce moisture in mixture 5. Modify gradation 6. Decrease temperature
Fat Spots	High Draindown (test) Long Haul Distance Long Storage Time Bad Paving Practices	1. Follow steps to reduce draindown 2. Minimize storage time 3. Calibrate paver 4. Adjust mix delivery and paving speed to maintain augers turning 85-90 percent of the time
Low In-Place Density	Insufficient Rollers Rollers Not Keeping Up Cold Weather and/or High Winds Insufficient Layer Thickness	1. Two breakdown rollers close to paver 2. Layer thickness should be at least 3 times nominal maximum aggregate size 3. Operate rollers correctly 4. Increase size of rollers and/or number of rollers

Standard Test Method for

Determination of Draindown Characteristics in Uncompacted Asphalt Mixtures

AASHTO Format

Standard Test Method for

Determination of Draindown Characteristics in Uncompacted Asphalt Mixtures

AASHTO Format

1. SCOPE

1.1 This test method covers the determination of the amount of draindown in an uncompacted asphalt mixture sample when the sample is held at elevated temperatures comparable to those encountered during the production, storage, transport, and placement of the mixture. The test is particularly applicable to mixtures such as porous asphalt (open-graded friction course) and Stone Matrix Asphalt (SMA).

1.2 The values stated in the gram-millimeter 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 AASHTO Standards:

T 245 Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus

M 92 Wire Cloth Sieves for Testing Purposes

3. DEFINITIONS

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

4. SUMMARY OF METHOD

4.1 A sample of the asphalt mixture to be tested is prepared in the laboratory or obtained from field production. The sample is placed in a wire basket which is positioned on a plate or other suitable container of known mass. The sample, basket, and plate or container are placed in a forced draft oven for one hour at a pre-selected temperature. At the end of one hour, the basket containing the sample is removed from the oven along with the plate or container and the mass of the plate or container is determined. The amount of draindown is then calculated.

5. SIGNIFICANCE AND USE

5.1 This test method can be used to determine whether the amount of draindown measured for a given asphalt mixture is within acceptable levels. The test provides an evaluation of the draindown potential of an asphalt mixture during mixture design and/or during field production. This test is primarily used for mixtures with high coarse aggregate content such as porous asphalt (open-graded friction course) and Stone Matrix Asphalt (SMA).

6. APPARATUS

6.1 Forced draft oven, capable of maintaining the temperature in a range from 120-175°C (250-350°F). The oven should maintain the set temperature to within ±2°C (± 3.6°F).
6.2 Plates or other suitable containers of appropriate size. The plates or containers used should be of appropriate durability to withstand the oven temperatures. Cake pans or pie tins are examples of suitable types of containers.
6.3 Standard basket meeting the dimensions shown in Figure 1. The basket sides and bottom shall be constructed using standard 6.3 mm (0.25 inches) sieve cloth as specified in AASHTO M 92.
6.4 Spatulas, trowels, mixer, and bowls as needed.
6.5 Balance accurate to 0.1 gram.

7. SAMPLE PREPARATION

7.1 Laboratory Prepared Samples

7.1.1 *Number of Samples*—For each mixture tested, the draindown characteristics should be determined at two different temperatures. The two temperatures should be the anticipated plant production temperature as well as 15°C (27°F) above (Note 1). For each temperature, duplicate samples should be tested. Thus for one asphalt mixture, a minimum of four samples will be tested.

Note 1—When using the test as part of the mixture design procedure, the test should be performed at two temperatures in order to determine the potential effect that plant temperature variation may have on the mixture during production. When the test is used in the field during production, it should be necessary to perform the test at the plant production temperature only.

7.1.2 Dry the aggregate to constant mass and sieve it into appropriate size fractions as indicated in AASHTO T 245, section 3.2.
7.1.3 Determine the anticipated plant production temperature or select a mixing temperature in accordance with AASHTO T 245, section 3.3.1.
7.1.4 Place into separate pans for each test sample the amount of each size fraction required to produce completed mixture samples having a mass of 1200±200 grams. The aggregate fractions shall be combined such that the resulting aggregate blend has the same gradation as the job-mix-formula. Place the aggregate samples in an oven and heat to a temperature not to exceed the mixing temperature established in 7.1.3 by more than approximately 28°C (50°F).
7.1.5 Heat the asphalt binder to the temperature established in 7.1.3.
7.1.6 Place the heated aggregate in the mixing bowl. Add any stabilizers (Note 2) and thoroughly mix the dry components. Form a crater in the aggregate blend and add the required amount of asphalt. The amount of asphalt shall be such that the final sample has the same asphalt content as the job-mix-formula. Using a spatula (if mixing by hand) or a mixer, mix the aggregate (and stabilizer if any) and asphalt binder quickly until the aggregate is thoroughly coated. After mixing, the temperature of the aggregate and asphalt binder shall be within the limits of the mixing temperature established in 7.1.3.

Note 2—Some types of stabilizers such as fibers or some polymers must be added directly to the aggregate prior to mixing with the asphalt binder. For laboratory prepared mixes, loose fibers should

be used. Asphalt-fiber pellets will not blend into the aggregate under low-shear laboratory mixing conditions. Other types of stabilizers must be added directly to the asphalt binder prior to blending with the aggregate.

7.2 Plant Produced Samples

7.2.1 *Number of Samples*—For plant produced samples, duplicate samples should be tested at the plant production temperature.
7.2.2 Samples may be obtained during plant production by sampling the mixture at any appropriate location such as the trucks prior to the mixture leaving the plant. Samples obtained during actual production should be reduced to the proper test sample size by the quartering method.

Note 3—Caution should be exercised when sampling from surge or storage bins because draindown may already have taken place.

8. PROCEDURE

8.1 Transfer the hot laboratory produced or plant produced uncompacted mixture sample to a tared wire basket described in 6.3. Place the entire sample in the wire basket. Do not consolidate or otherwise disturb the sample after transfer to the basket. Determine the mass of the sample to the nearest 0.1 gram. Care should be exercised to ensure that the sample does not cool more than 25°C below the test temperature (see section 8.2).
8.2 Determine and record the mass of a plate or other suitable container to the nearest 0.1 gram. Place the basket on the plate or container and place the assembly into the oven at the temperature as determined in 7.1.1 or 7.2.1 for 1 hour ± 5 minutes. If the sample has cooled more than 25°C but less than 50°C below the test temperature, the test should be conducted for 70 ± 5 minutes. If the sample has cooled more than 50°C below the test temperature, the sample should be discarded.
8.3 After the sample has been in the oven for the time specified in section 8.2, remove the basket and plate or container from the oven. Determine and record the mass of the plate or container plus drained asphalt binder to the nearest 0.1 gram.

9. CALCULATIONS

9.1 Calculate the percent of mixture which drained by subtracting the initial plate or container mass from the final plate or container mass and divide this by the initial total sample mass. Multiply the result by 100 to obtain a percentage.

10. REPORT

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

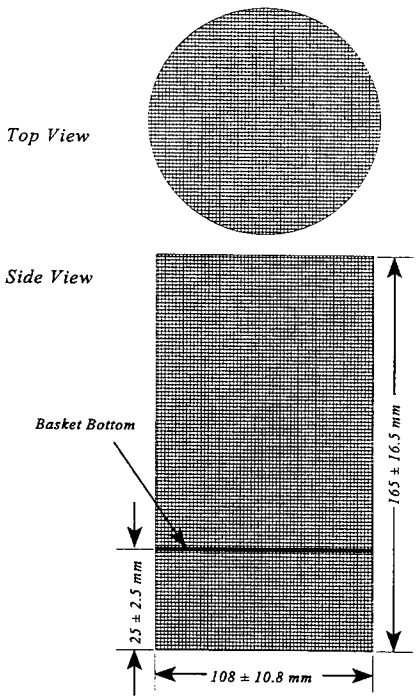

FIGURE 1 Wire Basket Assembly (Not to Scale).

Standard Practice for

Testing of HMA Mortars

AASHTO Format

Standard Practice for

Testing of HMA Mortars

AASHTO Format

1. SCOPE

1.1 This method covers the blending and specimen preparation of HMA mortars to predetermine the physical characteristics of mortars used in Hot Mix Asphalt.

1.2 *This standard does not purport to address all of the safety problems, if any, 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 *AASHTO Standards*:

PP1 Standard Practice for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)

TP1 Standard Test Method for Determining the Flexural Creep Stiffness of an Asphalt Binder Using the Bending Beam Rheometer (BBR)

TP5 Standard Test Method for Determining the Rheological Properties of an Asphalt Binder using the Dynamic Shear Rheometer DSR)

T240 Effect of Heat and Air on a Moving Film of Asphalt (RTFO)

2.2 *ASTM Standards*:

D4402 Test Method for Viscosity Determinations of Unfilled Asphalts Using the Brookfield Thermosel Apparatus

D4753 Specification for Evaluating, Selecting, and Specifying Balances and Scales for Use in Soil and Rock Testing

E11 Specification for Wire-Cloth Sieves for Testing Purposes

3. Summary of Test Methods

3.1 The mineral filler is heated and blended with preheated asphalt binder into a homogenous mixture. The mixture (mortar) is then capable of being tested as a binder.

3.2 The mortar will be significantly stiffer than an asphalt binder so different techniques from that used in binder testing will be adopted.

4. Significance and Use

4.1 This method can be used for laboratory preparation of mortars to predetermine the physical characteristics. The mortar is prepared and tested as a binder using various conventional tests and Superpave binder test methods. The results are primarily used for mix design evaluation, but may also be used for quality control.

5. Apparatus for Preparation

5.1 *Balance*, 2-kg capacity, sensitive to 0.1 g. The balance shall conform to the requirement of ASTM D 4753, Class GP2.

5.2 *Oven*, capable of maintaining the temperature at 175 ± 5° C.

5.3 *Oven*, capable of maintaining the temperature at 165 ± 5° C.

5.4 *Hot Plate*, electric, 700-W continuous or low, medium, and high settings.

5.5 *Sample Containers*, capable of holding 100 g of mineral filler and up to 200 g of asphalt binder. A 6 oz. (0.18 cm^3) tin box or seamless ointment tin is recommended.

5.6 *Mixing Tool*, a spatula is recommended for hand mixing.

5.7 *Gloves*, for handling hot equipment.

6. Procedure

6.1 *Preparation of filler*. Dry respective aggregate fractions containing filler to constant weight at 110 ±5° C and separate the filler portion by dry-sieving over a 0.075 mm sieve (See Note 1). Collect filler (material passing the 0.075 mm sieve) from dry sieving of the aggregate as well as the commercial filler. Blend the fillers, by volume, to meet percentages on job-mix-formula. (See Note 2)

6.2 Place a quart can of asphalt binder into an oven set at 165 ± 5° C.

6.3 *Preparation of Mortars:*

6.3.1 Weigh 100 ± 0.1 g of minus 0.075 mm recombined filler into a 6 oz. (0.18 cm^3) sample tin and place into the 175 ± 5° C oven. The material should remain in the oven for at least 30 minutes.

6.3.2 After the allocated time, remove tin containing filler from oven and tare on balance. Weight the proper amount of asphalt binder into sample tin to the nearest 0.1 g.

6.3.3 Place tin on hot plate and hand-mix with spatula. Care must be exercised to prevent loss of the mortar during mixing and subsequent handling. Mix until homogeneous.

6.3.4 When mortar is homogenous, the mixture is ready for testing. If fibers are to be used, they are to be added slowly to the mortar and mixed until homogeneity is achieved. (See Note 3)

NOTE 1 - Detailed requirements for this sieve are given in ASTM Specification E 11.

NOTE 2 - An example problem illustrating how to blend based on volumes can be found later in this report.

NOTE 3 - When asphalt-fiber pellets are to be used, loose fiber of the same type should be incorporated into the mixing process. Asphalt-pellet fibers will not blend into the aggregate under low-shear mixing conditions.

7. Specimen Testing of Mortars

7.1 When performing Superpave Binder Testing, the asphalt binder should be aged following AASHTO T240 and PP1 prior to blending with fillers.

7.2 Follow ASTM D4402, except readings should be taken as soon as the material stabilizes in temperature. The fillers will sink to the bottom of the sample chamber over time and will create invalid data.

7.3 Follow AASHTO TP 5, except in order for the specimen to adhere strongly to both plates, PAV DSR testing will require a higher preheat temperature of +58°C.

7.4 Follow AASHTO TP1, except preparation of specimens are as follows:

7.4.1 *Aluminum Molds:*

7.4.1.1 Place the mold over the corner of the hot plate so the base plate is on the hot plate and the rubber O-rings are not. Care should be taken to insure O-rings do not come in contact with the hot plate.

7.4.1.2 Using a wooden tongue depressor, gently tamp the mortar into the mold. A light coating of a release agent such as a glycerin and talc mixture on the depressor will assist in this procedure.

7.4.1.3 Repeat 7.4.1.2 until mold is full of mortar.

7.4.1.4 Continue testing according to TP1.

7.4.2 *Silicone Molds:*

7.4.2.1 Disassemble the mold apparatus and place silicone mold on flat surface.

7.4.2.2 Trowel mortar into mold, overfilling the mold on the top as well as the open end.

7.4.2.3 Care shall be taken while trimming to assure a smooth, straight surface.

7.4.2.4 Continue testing according to TP1.

Example Mixture Design

INTRODUCTION

This example provides a step by step discussion of the mixture design procedure outlined by the "Standard Practice for Designing Stone Matrix Asphalt (SMA)" and "Standard Specification for Designing Stone Matrix Asphalt (SMA)" provided earlier in this report. These draft standards ensure that sufficient VMA, VCA, and air voids exist in the mixture. It also ensures that an aggregate skeleton with stone-on-stone contact is produced. The design was accomplished for a 19.0 mm nominal maximum aggregate size (NMAS) SMA. The 20 year design ESAL level for this project was 20 million.

MATERIALS SELECTION

Materials available for this design are listed below and the individual aggregate stockpile gradations are shown in Table 1.

Coarse Aggregate:	#78 Limestone
Fine Aggregate:	#810 Limestone
Mineral Filler:	Limestone Dust
Asphalt Binder:	PG 64-22
Stabilizer:	Cellulose Fiber (Dosage rate of 0.3% by total mix weight)

MATERIALS TESTING

The aggregates, mineral filler, PG 64-22, and cellulose fiber were tested for compliance with the applicable specifications in the "Standard Specification for Designing Stone Matrix Asphalt (SMA)" provided earlier in this report. All materials were found suitable for use in SMA. The Los Angeles Abrasion loss for the coarse aggregates was determined to be 25 percent.

SELECT TRIAL GRADATIONS

In accordance with the AASHTO draft "Standard Practice for Designing Stone Matrix Asphalt (SMA)", three trial gradations were chosen such that they were within the master gradation band for a 19.0 mm NMAS SMA. The three trial blends were along the coarse and fine limits of the gradation band along with one gradation falling in the middle. Because the bulk specific gravities of the different aggregates differ by more than 0.02, the blended trial gradations were based on volumetric percentages. (Blending SMA gradations based on volume is discussed later in this report.) These trial gradations are shown in Table 2. The percentages of aggregates in each blend are shown in Table 3.

Determination of Voids in the Coarse Aggregate - Dry-Rodded Condition (VCA_{DRC})

For each of the three trial blends, the VCA_{DRC} was determined for the coarse aggregate fraction according to AASHTO T 19. Since the gradation was a 19.0 mm NMAS, the VCA_{DRC} was determined for the aggregate fraction coarser than the 4.75 mm sieve. Two replicates for each test were performed. The average results are given in Table 4.

TABLE 1: Available Aggregate Gradations.

	Percent Passing by Mass			
Sieve Size	#78	#810	Mineral Filler	19.0 mm SMA Band
25	100.0	-	-	100
19	96.7	-	-	90-100
12.5	61.2	100.0	-	50-74
9.5	44.0	99.8	-	25-60
4.75	9.0	76.0	-	20-28
2.36	5.0	65.0	-	16-24
1.18	3.0	50.2	100.0	13-21
0.60	2.0	37.6	98.8	12-18
0.30	2.0	27.5	88.6	12-15
0.075	1.0	10.4	77.6	8-10
G_s	2.704	2.711	2.803	-

G_s - bulk specific gravity of the aggregate

TABLE 2: Gradations of the Three Trial Blends.

	Percent Passing by Volume		
Sieve Size (mm)	Trial Blend 1	Trial Blend 2	Trial Blend 3
25.0	100.0	100.0	100.0
19.0	97.2	97.3	97.5
12.5	67.3	68.8	70.4
9.5	52.8	55.0	57.2
4.75	21.8	24.5	27.2
2.36	17.8	20.2	22.6
1.18	15.2	17.1	19.0
0.60	13.5	14.9	16.4
0.30	11.9	12.9	14.0
0.075	9.0	9.4	9.7
G_{sb}	2.715	2.715	2.715

G_{sb} - combined aggregate bulk specific gravity

TABLE 3: Percent of Aggregates Used in Each of the Three Trial Blends.

	Percent by Mass Used in Blend		
Blend No.	#78	#810	MF
1	84	6	10
2	80	10	10
3	76	14	10

TABLE 4: Density and VCA_{DRC} for the Three Trial Blends.

Blend No.	VCA_{DRC} (%)	Density (kg/m³)
1	40.8	1598
2	40.3	1610
3	39.9	1623

The calculation for VCA_{DRC} for blend 2 is shown below.

$$VCA_{DRC} = \frac{G_{CA}\gamma_w - \gamma_s}{G_{CA}\gamma_w} \; x \; 100$$

$$VCA_{DRC} = \left(\frac{(2.704)(998) - 1610}{(2.704)(998)} \right) \; x \; 100$$

$$VCA_{DRC} = 40.3\%$$

where,

γ_s - unit weight of the coarse aggregate fraction in the dry rodded condition (kg/m³) (Table 4),

γ_w - unit weight of water (998 kg/m³), and

G_{CA} - combined bulk specific gravity of the coarse aggregate (Table 1).

As can be seen, the VCA_{DRC} does not vary much from blend to blend. This is because the coarse aggregate gradations are nearly identical for each of the three trial blends. In practice, this will nearly always be the case. The result is that the VCA_{DRC} may need only be determined for one of the trial blends.

Compact Specimens

For each of the trial blends, three samples were produced at 6.5% asphalt binder by total mix mass using 100 gyrations of the Superpave Gyratory Compactor (SGC). This gyration level was selected because the mixture is to be designed for a pavement with an anticipated 20 year ESAL level of 30 million and the Los Angeles Abrasion loss was below 30 percent. The bulk specific gravities (G_{mb}) of these specimens were then determined after compaction according to AASHTO T 166. Also for each trial blend the maximum theoretical specific gravity (G_{mm}) was determined for one sample according to AASHTO T 209. The air voids, VMA, and VCA were then determined. These results are summarized in Table 5.

TABLE 5: Test Results for Three Trial Gradation Blends.

Property	Trial Blend 1	Trial Blend 2	Trial Blend 3
G_{mb}	2.379	2.383	2.383
G_{mm}	2.468	2.460	2.455
Air Voids, %	3.6	3.1	2.9
VMA, %	18.1	17.9	17.9
VCA,%	35.7	37.8	40.1

An example of the VCA calculation for the SMA mixtures is given here for blend 2.

$$P_{bp} = P_s \; x \; PA_{bp}$$

$$P_{bp} = (93.5)(0.755)$$

$$P_{bp} = 70.6\%$$

$$VCA = 100 - \left(\frac{G_{mb}}{G_{ca}} \; x \; P_{bp} \right)$$

$$VCA = 100 - \left(\frac{2.382}{2.704} \times (70.6) \right)$$

$$VCA = 37.8\%$$

where,

P_{bp} - percent aggregate by total mixture mass retained on the breakpoint sieve (4.75 mm),
P_s - percent aggregate in the mix,
PA_{bp} - percent aggregate by total aggregate mass retained on the breakpoint sieve (4.75 mm),
G_{ca} - combined bulk specific gravity of the coarse aggregate, and
G_{mb} - bulk specific gravity of compacted specimens.

Based on Table 5, trial blends 1 and 2 meet the requirements for VMA and VCA (VMA > 17 percent and $VCA<VCA_{DRC}$). Trial blend 3 did not meet the VCA requirements. However, none of these blends met the air voids requirement of 4.0 percent. Based on the "Standard Practice for Designing Stone Matrix Asphalt (SMA)", the minimum asphalt content for this SMA mixture is 6.1 percent. This is based on the combined bulk specific gravity of the aggregates of 2.715. Therefore, from the three trial blends, it appears that trial blend 1 would be the easiest of the three to achieve this minimum asphalt content at the required air void content of 4.0 percent. For this reason, trial blend 1 was selected and the optimum asphalt content was determined.

SELECT OPTIMUM ASPHALT BINDER CONTENT

With the optimum gradation determined, the asphalt content is now varied to determine optimum asphalt content. For this phase eight samples were prepared, four at an asphalt binder content of 5.7 percent and four at 6.1 percent. The third asphalt binder content data point was 6.5% (already completed as trial gradation). Three samples at each of these asphalt binder contents were compacted while the remaining sample was used to determine the theoretical maximum density. The average results are shown in Table 6.

The air voids, VMA, and VCA from Table 6 are plotted as a function of asphalt binder content in Figure 1. Upon evaluation of the test results, it was determined that an asphalt binder content of 6.4 percent produced a mixture meeting all the requirements.

TABLE 6: Test Results From Trial Blend 1 at Various Asphalt Binder Contents.

Property	5.7 % AC	6.1% AC	6.5% AC
G_{mb}	2.359	2.369	2.379
G_{mm}	2.498	2.483	2.468
Air Voids, %	5.6	4.6	3.6
VMA, %	18.2	18.2	18.1
VCA, %	35.7	35.7	35.7

MORTAR PROPERTIES (Optional Requirement)

The fine mortar properties for this mixture were evaluated according to AASHTO TP 5 and TP 1, methods for the Dynamic Shear and Bending Beam Rheometers, respectively. These mortars were prepared and tested in accordance with the "Standard Practice for Testing of HMA Mortars" provided earlier in this report. The asphalt binder was a PG 64-22, therefore testing with the DSR was conducted at 64°C and at -12°C for the BBR. The results of the testing are shown in Table 7.

TABLE 7: Fine Mortar Properties.

Test	Temperature (°C)	Result	Requirement
Unaged DSR, G*/sinδ (kPa)	64	6.23	5 min.
RTFO Aged DSR, G*/sinδ (kPa)	64	14.85	11 min.
PAV Aged BBR, Stiffness (MPa)	-12	895	1500 max.

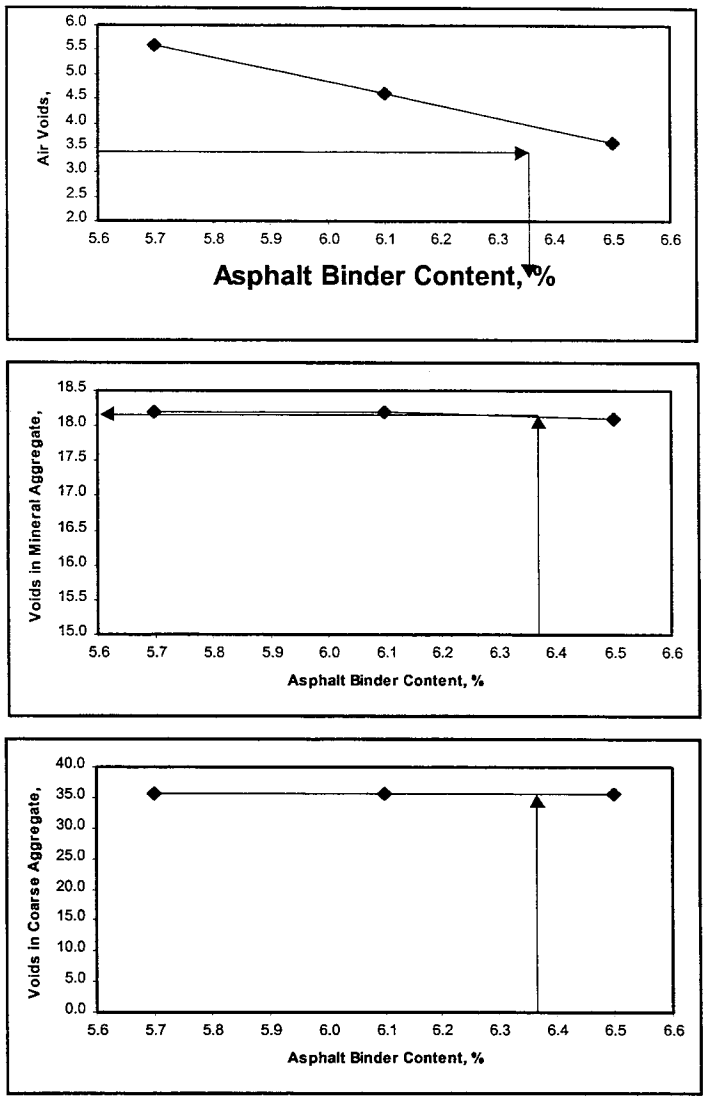

FIGURE 1: Plots of Properties as a Function of Asphalt Content

MOISTURE SUSCEPTIBILITY

Moisture susceptibility was determined in accordance with ASHTO T 283. The compactive effort necessary to produce an average of 6 percent air voids was determined. Eight samples were compacted to this average air void content with no one sample having less than 5 percent or more than 7 percent. Four of the specimens were tested for dry strength while the remaining four were tested for wet strength. A freeze-thaw cycle was not used. The average results are shown in Table 8. The average TSR was measured to be 94.4% which exceeds the 70 percent minimum requirement. The mixture therefore complies with moisture susceptibility requirements.

TABLE 8: Moisture Susceptibility Test Results for the SMA Mixture.

Ave. Dry Strength, kPa	445
Ave. Wet Strength, kPa	420
TSR, %	94.4

DRAINDOWN TESTING

Draindown susceptibility was performed according to the method described in the "Standard Test Method for Determination of Draindown Characteristics in Uncompacted Asphalt Mixtures" provided earlier in this report. Duplicate samples were tested at each of three temperatures, 137°, 152°, and 167°C. The test results are shown in Table 9. For this project the plant production temperature was anticipated to be 152°C.

TABLE 9: Draindown Sensitivity Test Results for the SMA Mixture.

Temperature, °C	Ave. Draindown, %
137	0.05
152	0.06
167	0.09

None of the samples exceeded the requirement of 0.30 percent maximum. The mixture is therefore deemed not to be susceptible to draindown.

DESIGN SUMMARY

The mixture properties for the designed mixture, gradation and optimum asphalt binder content, component percentages, and mortar properties are shown in Tables 10, 11, 12, 13, and 14. Table 12 shows the design gradation based on mass percentages. This gradation would be used during QC/QA testing. Table 13 presents the percentages of each stockpile based on mass. This table would be used when dialing in stockpile percentages during production. Both Tables 12 and 13 were determined following the procedures outlined in the example problem for blending gradations based on volumes provided later in this report.

TABLE 10: SMA Mixture Properties.

Mixture Property	Test Result	Requirement
G_{sb}	2.715	-
Asphalt Content, %	6.4	6.1 minimum
Fibers, %	0.3	0.30
Air Voids, %	4.0	4.0
VMA, %	18.1	17.0 min.
VCA, %	35.7	40.8[1] max.
TSR, %	94.4	70.0 min.
Draindown @ 152°C, %	0.06	0.30 max.

[1] - VCA_{DRC}

TABLE 11: Aggregate Gradation Based on Volume

Sieve Size (mm)	Percent Passing by Volume
25.0	100.0
19.0	97.2
12.5	67.3
9.5	52.8
4.75	21.8
2.36	17.8
1.18	15.2
0.60	13.5
0.30	11.9
0.075	9.0

TABLE 18: Aggregate Gradation Based on Mass.

Sieve Size (mm)	Percent Passing by Mass
25.0	100.0
19.0	97.2
12.5	67.4
9.5	52.9
4.75	22.1
2.36	18.1
1.18	15.5
0.60	13.8
0.30	12.2
0.075	9.2

TABLE 19: Component Percentages in the Selected Mixture.

Component	Percent by Mass of Aggregate	Percent By Total Mix Mass
#78 Stone	84	78.4
#810 Stone	6	5.6
Mineral Filler	10	9.3
Cellulose Fiber	-	0.3
PG 64-22	-	6.4

TABLE 20: Fine Mortar Properties.

Test	Temperature (°C)	Result	Requirement
Unaged DSR, G*/sinδ (kPa)	64	6.23	5 min.
RTFO Aged DSR, G*/sinδ (kPa)	64	14.85	11 min.
PAV Aged BBR, Stiffness (MPa)	-12	895	1500 max.

Example Problem
Determining Percent Passing by Volume

INTRODUCTION

As with any hot mix asphalt (HMA), specified aggregate gradations should be based on aggregate volume and not aggregate mass. However, for most conventional HMA mixtures (dense-graded), the specific gravities of the different aggregate stockpiles are close enough to make the gradations based on mass percentages similar to that based on volumetric percentages. With SMA, the specific gravities of the different aggregate components are not always similar. This is especially true for mineral fillers that are used in SMA. Therefore, the specified gradation bands presented in the mixture design procedure are based on percent passing by volume. This example problem provides guidance on how to blend aggregate components based on volumes to meet the SMA gradation bands. However, if the bulk specific gravities of the different stockpiles to be used within the mixture vary by 0.02 or less, gradations based on mass percentages can be used.

STEP 1

The first step as with any blending problem is to perform gradation tests based on mass for the various stockpiles to be used in the SMA mixture. AASHTO T27 is used for this testing. For this example, a 19.0 mm nominal maximum aggregate size SMA mixture is to be blended. Table 1 provides the results of gradation tests performed on four stockpiles which are to be blended for this example problem. Also needed to determine aggregate gradations based on volume are the bulk specific gravities (G_{sb}) of the different stockpiles. Table 1 also provides the G_{sb} values for each stockpile. Notice that the G_{sb} values differ by more than 0.02.

TABLE 1: Results of Gradation and Specific Gravity Tests for Stockpiles To Be Used

Sieve, mm	Stockpile and Percent Passing Based on Mass, %			
	Aggregate A	Aggregate B	Aggregate C	Mineral Filler
25.0	100.0	100.0	100.0	100.0
19.0	95.0	100.0	100.0	100.0
12.5	66.0	71.0	97.4	100.0
9.5	43.0	46.0	84.6	100.0
4.75	9.0	6.0	48.9	100.0
2.36	5.0	4.0	27.8	100.0
1.18	2.0	4.0	16.6	100.0
0.60	2.0	3.0	10.7	100.0
0.30	2.0	3.0	7.6	100.0
0.075	1.0	1.5	4.6	72.5
G_{sb}	2.616	2.734	2.736	2.401

STEP 2

The second step is to determine the percent mass retained on each sieve for the different stockpiles. For a given sieve, this is done by subtracting the percent passing the given sieve from the percent passing the next larger sieve. For example, using Aggregate C the percent mass retained on the 4.75 mm sieve would be calculated as:

$$\%\ Retained\ on\ 4.75\ mm\ Sieve = 84.6 - 48.9 = 35.7\ \%$$

where,

84.6 = percent mass passing 9.5 mm sieve (Table 1)
48.9 = percent mass passing 4.75 mm sieve (Table 1)
35.7 = percent mass retained on 4.75 mm sieve

Table 2 presents the values for percent mass retained for all sieves and stockpiles. Note that a row has been added to reflect that material finer than the 0.075 mm (- 0.075) sieve is included.

TABLE 2: Percent By Mass Retained On Each Sieve

Sieve, mm	Percent Mass Retained per Sieve			
	Aggregate A	Aggregate B	Aggregate C	Mineral Filler
25.0	0.0	0.0	0.0	0.0
19.0	5.0	0.0	0.0	0.0
12.5	29.0	30.0	2.6	0.0
9.5	23.0	24.0	12.8	0.0
4.75	34.0	40.0	35.7	0.0
2.36	4.0	2.0	21.1	0.0
1.18	3.0	0.0	11.2	0.0
0.60	0.0	1.0	5.9	0.0
0.30	0.0	0.0	3.1	0.0
0.075	1.0	1.5	3.0	27.5
- 0.075	1.0	1.5	4.6	72.5
Total, Σ	100	100	100	100

STEP 3

For step 3 a simple assumption is made, "Assume the Mass of Each Aggregate Stockpile is 100 grams." Using this assumption allows for the mass that would be retained on each sieve for each stockpile to be determined and can be shown to be equal to the numbers shown in Table 2.

STEP 4

Step 4 converts the masses determined in Step 3 to volumes per sieve. To make this conversion the bulk specific gravity of the individual stockpiles is needed. The volume of aggregate retained on each sieve for each stockpile can be determined from the following equation:

$$\begin{matrix} Volume\ of\ aggregate\ for\ a \\ given\ sieve\ (cm^3) \end{matrix} = \frac{Mass\ of\ aggregate\ Retained,\ grams}{Specific\ Gravity\ x\ \gamma_w}$$

where,

γ_w - unit weight of water (1.0 g/cm^3)

The following calculation demonstrates how the volume is calculated for the aggregate retained on the 4.75 mm sieve of Aggregate C.

$$Volume = \frac{35.7\ grams}{2.736\ x\ 1\ g/cm^3} = 13.05\ cm^3$$

where,

35.7 = mass of Aggregate C retained on 4.75 mm sieve (Table 2)
2.736 = bulk specific gravity of Aggregate C (Table 1)
1 g/cm^3 = unit weight of water
13.05 cm^3 = volume of Aggregate C retained on 4.75 mm sieve

The volumes retained on all sieves for each of the four stockpiles are provided in Table 3.

TABLE 3: Volumes of Aggregate Retained on Each Sieve

Sieve, mm	Volume of Aggregate Retained per Sieve, cm^3			
	Aggregate A	Aggregate B	Aggregate C	Mineral Filler
25.0	0.00	0.00	0.00	0.00
19.0	1.91	0.00	0.00	0.00
12.5	11.09	10.97	0.95	0.00
9.5	8.79	8.78	4.68	0.00
4.75	13.00	14.63	13.05	0.00
2.36	1.53	0.73	7.71	0.00
1.18	1.15	0.00	4.09	0.00
0.60	0.00	0.37	2.16	0.00
0.30	0.00	0.00	1.13	0.00
0.075	0.38	0.55	1.10	11.45
- 0.075	0.38	0.55	1.68	30.20

STEP 5

The values provided in Table 3 are used to blend the different stockpiles to meet the desired gradation based on volumes. In this procedure the aggregate is blended by mass, then the final gradation is determined based on volume. As with gradations based on mass, this is a trial and error process. To perform the blending, select the estimated percentages by mass of the different stockpiles to be used. For this example, the following percentages will be tried first:

Stockpile	% Blend by Mass
Aggregate A	30
Aggregate B	30
Aggregate C	30
Mineral Filler	10

Notice that the percentages above are based on mass. This indicates that the volume represented by 30 percent of Aggregate A by mass will be used in the blending of stockpiles based on volumes.

The percent of each stockpile in the blend is multiplied by the volume retained on a given sieve for each stockpile to determine the total volume retained on that sieve. Using the 4.75 mm sieve, the total volume retained on the 4.75 mm sieve would be calculated as follows:

$$\textit{Total Volume Retained on 4.75 mm Sieve} = (0.30 \times 13.00) + (0.30 \times 14.63) + (0.30 \times 13.05) + (0.10 \times 0.00)$$

$$\textit{Total Volume Retained on 4.75 mm Sieve} = 12.20\ cm^3$$

where,

0.30, 0.30, 0.30, and 0.10 = percentages by mass of each aggregate in blend expressed as decimal

13.00, 14.63, 13.05, and 0.00 = volume retained on 4.75 mm sieve for each stockpile (Table 3)

This calculation is performed for each of the sieves in the gradation. Table 4 presents the total volume retained for each of the sieves in the gradation.

TABLE 4: Total Volumes Retained Per Sieve

Sieve, mm	Volume Retained per Sieve, cm^3
25.0	0.00
19.0	0.57
12.5	6.90
9.5	6.67
4.75	12.20
2.36	2.99
1.18	1.57
0.60	0.76
0.30	0.34
0.075	1.75
- 0.075	3.80
Total Volume, Σ	37.57

Now, based on the total volume retained per sieve and the summed total volume of the blended aggregates, the percent retained per sieve by volume can be determined for the blend. This is accomplished for a given sieve by dividing the volume retained on that sieve by the total volume of the blend. The following equation illustrates this calculation for the 4.75 mm sieve.

$$\%\ Volume\ Retained\ on\ 4.75\ mm\ Sieve = \frac{12.20}{37.57}\ x\ 100 = 32.50\ \%$$

where,

12.20 = volume retained on 4.75 mm sieve (Table 4)
37.57 = summed total volume of blend (Table 4)
32.50 = percent volume of blend retained on 4.75 mm sieve

Table 5 provides the percents retained based on volumes for each of the sieves and converts this to percent passing.

TABLE 5: Percents Passing Based on Volumes

Sieve, mm	Percent Retained Per Sieve	Cumulative Percent Retained	Percent Passing by Volume
25.0	0.0	0.0	100
19.0	1.5	1.5	98.5
12.5	18.4	19.9	80.1
9.5	17.8	37.7	62.3
4.75	32.5	70.1	29.9
2.36	8.0	78.1	21.9
1.18	4.2	82.3	17.7
0.60	2.0	84.3	15.7
0.30	0.9	85.2	14.8
0.075	4.7	89.9	10.1
- 0.075	10.1	100.0	0.0

Using the percent retained per sieve based on volume, the percent passing by volume for the gradation can be determined similar to the method used with gradations based on mass. Determine the cumulative percent retained for each sieve and then subtract from 100.

Now, the blended gradation is compared to the required gradation band (also based on volume). Table 6 compares the gradation band for a 19.0 mm nominal maximum aggregate size SMA to the gradation shown in Table 5.

TABLE 6: Comparison of Gradation Blend Based on Volume To Specified Gradation Band

Sieve, mm	Gradation Band	Blend Percent Passing
25.0	100	100
19.0	90-100	98.5
12.5	50-74	80.1*
9.5	25-60	62.3*
4.75	20-28	29.9*
2.36	16-24	21.9
1.18	13-21	17.7
0.60	12-18	15.7
0.30	12-15	14.8
0.075	8-10	10.1*

* Does not meet requirements.

Based on Table 6, the blended gradation did not meet the specified gradation band for a 19.0 mm nominal maximum aggregate size SMA. Therefore, different blending percentages for the various stockpiles are needed. Below are the percentages of the four stockpiles used for the second trial.

Stockpile	% Blend by Mass
Aggregate A	40
Aggregate B	41
Aggregate C	10
Mineral Filler	9

Table 7 presents the blending of the four aggregates for the second trial. The second trial blend percentages were used along with the values of Table 3 to determine the percent passing by volume for the blend.

TABLE 7: Percents Passing Based on Volumes

Sieve, mm	Percent Retained Per Sieve by Volume	Cumulative Percent Retained by Volume	Percent Passing by Volume	Percent Passing by Mass (For Comparison)	Gradation Band by Volume
25.0	0.0	0.0	100.0	100.0	100
19.0	2.0	2.0	98.0	98.0	90-100
12.5	24.0	26.0	74.0	74.3	50-74
9.5	20.1	46.1	53.9	53.5	25-60
4.75	33.2	79.3	21.7	20.0	20-28
2.36	4.5	83.7	16.3	15.4	16-24
1.18	2.3	86.0	14.0	13.1	13-21
0.60	1.0	87.0	13.0	12.1	12-18
0.30	0.3	87.3	12.7	11.8	12-15
0.075	4.0	91.3	8.7	8.0	8-10
- 0.075	8.7	100.0	-	-	-

Based on Table 7, the following percentages produce a gradation based on volume which meets the 19.0 mm nominal maximum aggregate size gradation band for SMA.

Stockpile	% Blend by Mass
Aggregate A	40
Aggregate B	41
Aggregate C	10
Mineral Filler	9

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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