Designing Stone Matrix Asphalt Mixtures Volume II(c) – Research Results for Phase II

Final Report

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CHAPTER 1 - INTRODUCTION

1.1 BACKGROUND

Stone matrix asphalt (SMA) has been used successfully within the United States 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 National Cooperative Highway Research Program (NCHRP) contracted with the National Center for Asphalt Technology (NCAT) in 1994 to develop a mixture design procedure for SMA. This research project consisted of two phases. The first phase was completed and submitted as a three volume interim report. Volume I consisted of a comprehensive literature review, Volume II contained the research results, and Volume III was a tentative mixture design procedure for SMA. This three volume report was dated September 1996. The report presented herein provides the results of Phase II of the research work. The report is presented in five volumes. Volume I is an update of the Volume I literature survey. Volume II presents the test results and analysis from Phase I and Phase II. Volume III presents a summary of the findings for the entire study and Volume IV provides the deliverables including mix design method, QC/QA procedures, construction guidelines, etc. Volume V is an appendix which includes the raw data.

1.2 OBJECTIVES

The primary objectives of this study were to:

1. Develop and validate a mixture design procedure for SMA;
2. Develop Quality Control/Quality Assurance (QC/QA) procedures for SMA mixtures; and
3. Develop guidelines for construction of SMA.

1.3 SCOPE

Phase II of the research involved observation of eleven different SMA construction projects throughout the United States. Information obtained from each of these projects was used to help validate the SMA mixture design procedure developed in Phase I, verify roadway density requirements, and evaluate the accuracy and precision of the nuclear density gauge for determining field density. Additionally, laboratory work was conducted to extend the research results obtained during Phase I. Finally, based on all of the information obtained in Phases I and II, QC/QA procedures and SMA construction guidelines were developed.

CHAPTER 2 - WORK COMPLETED IN PHASE I

Phase I research involved developing an SMA mixture design procedure. The Interim Report was delivered to NCHRP in September of 1996. A summary of tasks in Phase I and Phase II is provided in Figure 2.1.

2.1 REVIEW OF PHASE I RESEARCH RESULTS

Results of research findings during Phase I of research are summarized below.

- The dry-rodded test for coarse aggregate is acceptable for measuring the voids in coarse aggregate (VCA) for stone-on-stone contact. Many other methods were evaluated but the dry-rodded test seemed to be the most acceptable.
- Excessive breakdown in some aggregates may prevent them from being used in SMA. Aggregates that are too soft break down excessively, resulting in the VMA being too low. When the L.A. Abrasion of the aggregate is above 30, the aggregate breakdown may be excessive resulting in low VMA.
- SHRP binder tests including dynamic shear, bending beam, and the direct tension device can be used to evaluate the quality of the fine mortar (filler and binder). The Brookfield viscometer had limited use for measuring the stiffness of the mortar based on initial testing. The properties of the fine mortar are related to the total mortar (minus 2.36 mm and binder), hence, testing of the total mortar is not required. Additional mortar testing in Phase II showed the Brookfield viscometer to be useful in evaluating the mortar stiffness at high temperatures.
- The creep test (static and dynamic loading with and without confinement) was not effective in showing the benefits of SMA mixtures over dense graded mixtures.
- The Marshall stability test has limited value as a specification requirement for SMA. However, if it is used the requirement should be lowered from 6200 N (1400 lbs) to approximately 5300 N (1200 lbs).
- It appears that a good aggregate skeleton can be obtained by comparing the VCA in the SMA mixture to the VCA in the dry-rodded condition for coarse aggregate only. Mortar properties can be controlled by using the existing SHRP binder tests. At the present time, the best control of the mixture is the volumetric properties (voids, VMA, and VCA).
- The coarse and fine aggregate used in SMA mixtures should be 100 percent crushed. The fine aggregate angularity should equal or exceed 45 when measured with the NAA flow test.
- The LA Abrasion for the coarse aggregate to be used in SMA mixtures should be set at 30. Higher LA Abrasion values may result in excessive breakdown and make it difficult to meet the VMA requirements.
- Flat and elongated particles should be measured on a 3 to 1 and 5 to 1 ratio. A 2 to 1 can be used but the 3 to 1 ratio appears to rank the various aggregates effectively. The requirement for 3 to 1 should be set at 20 percent. The requirement for 5 to 1 should be set at 5 percent.

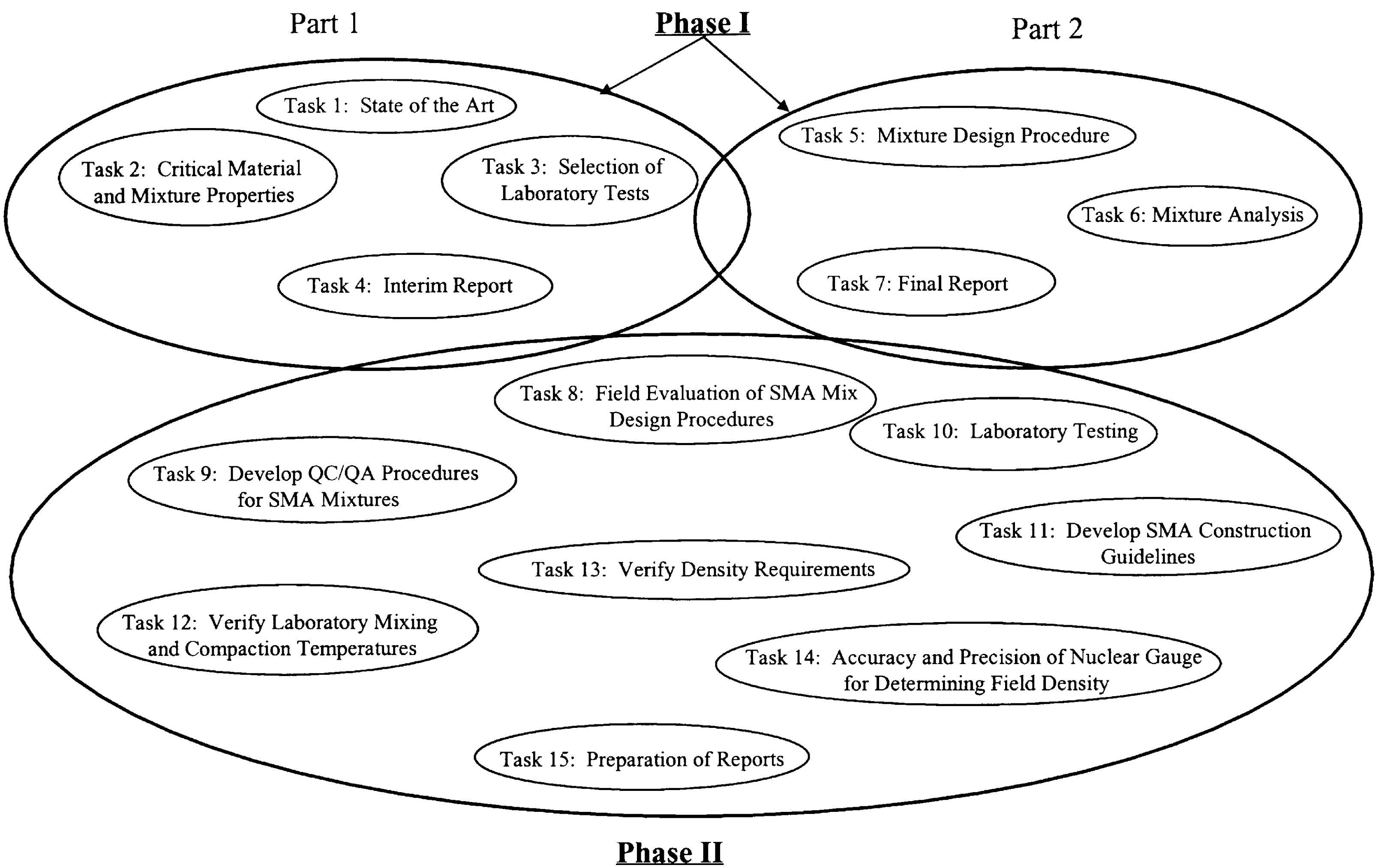


Figure 2.1: Relationship Between Phase I and Phase II of the Research

- The filler requirement that sets a maximum percentage of material passing the 0.02 mm size should be deleted from specifications. There is no relationship between the percentage of material passing the 0.02 mm and its effect on mortar properties. Specifying the mortar properties should identify any filler that is unsatisfactory due to being excessively fine.
- A good screening test for fillers is to ensure that the angularity does not exceed 50 when measured with the modified Rigden voids. Fillers that exceed 50 cause the mortar to be excessively stiff and difficult to work.
- The asphalt cement for SMA mixtures should be the PG grade for the climate in which it will be used. For best performance on high volume roads, the higher temperature grade should be increased.
- For mix design, mortars (material finer than 0.075 mm, asphalt cement, and stabilizer) should be tested at the same high and low temperatures as the PG graded asphalt cement. The requirement at the high end should be approximately 5 times the Superpave requirements on unaged and aged mortars. The requirements on the low end should be limited to approximately 5-6 times the Superpave requirements. Intermediate temperatures and m-value at low temperatures should not be specified.
- The proposed mix design method will produce an acceptable SMA mixture. It is sometimes difficult to meet desired VMA requirements when aggregate breakdown is excessive. The requirements for VCA are generally easily met.
- The minimum VMA requirement should be set at 17 to ensure a durable SMA. If a minimum asphalt content requirement is used, it should require that the effective asphalt content be at least 6.0 percent. This minimum required effective asphalt content should be adjusted downward when the aggregate specific gravity is high and upward when the aggregate specific gravity is low. The best approach is to establish the minimum VMA requirement at 17 and to not specify a minimum asphalt content.
- The 50-blow compactive effort with the Marshall hammer appears reasonable. The compactive effort with the Superpave gyratory compactor should be 100 revolutions. (Further research during Phase II indicated that approximately 80 gyrations better correlated with 50-blows of the Marshall hammer.) This number of revolutions corresponds to the design number of gyrations under Superpave terminology.
- SMA mixtures should be designed to have 3.0 to 4.0 percent air voids. Higher air voids result in excessive permeability and will lead to durability problems. Lower air voids result in excessive fat spots and can lead to rutting.
- The laboratory wheel tracking test appears to be a good approach to evaluate rutting potential of SMA mixtures. This should be considered for inclusion as part of an SMA mix design procedure.
- The tensile strength ratio (TSR) of SMA mixtures is typically lower than for dense graded mixtures. This does not indicate that SMAs are more susceptible to moisture but does indicate that the acceptance limit for TSR should be lower than for dense graded mixtures. Based on many mixtures that were evaluated in this study, a TSR requirement of 70 percent appears reasonable. The test results also show that hydrated lime is usually effective in increasing the TSR. The TSR test should be conducted at an average void level of 6.0 percent. For SGC designed SMA, 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.

- The use of fibers tend to do a better job than polymers in reducing draindown. The 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.
- During compaction , aggregate breakdown for aggregates with L.A. abrasion loss values below 30 percent tends to range from 5-10 percent on the 4.75 mm sieve. For aggregates with loss values above 30 percent this breakdown usually exceeds 10 percent. Limited data indicates that the amount of breakdown in the field is approximately equal to that in the laboratory. The Marshall hammer generally produces more breakdown than the SGC. The breakdown for SMA mixtures is greater than that for dense graded mixtures for the 4.75 mm sieve; however, the breakdown is greater for dense graded mixtures on the 0.075 mm sieve.
- SMA mixtures tend to have higher permeability than dense graded mixtures for the same void level. The permeability appears to become a problem between 6 and 7 percent air voids. The data shows that the permeability tends to double for each 1 percent increase in void level.

CHAPTER 3 - TEST PLAN FOR PHASE II

3.1 INTRODUCTION

This chapter outlines the test plan used in completing Tasks 8 through 15. The first objective of these tasks was to field validate the proposed mix design procedure established in Tasks 1 through 7 in order to produce a final mixture design procedure. The second objective was to develop Quality Control/Quality Assurance (QC/QA) procedures for SMA mixtures. The final objective of Phase II was to develop guidelines for the construction of SMA. Each of these objectives required the collection of field data. Figure 3.1 provides a flow diagram showing the collection of field data for the various tasks accomplished for Phase II.

3.2 TASK 8 - FIELD EVALUATION OF SMA MIX DESIGN PROCEDURES

The main objective of this task was to determine if SMA can be properly produced in the field when designed in the laboratory according to the mixture design procedure developed in Phase I. This was accomplished by evaluating 11 SMA construction projects, sending a questionnaire to both users and potential users of the SMA mixture design procedure to identify weak areas in the procedure, and collecting information from other tasks within this project relating to mixture design. Figure 3.2 shows how these elements were used to evaluate the SMA mixture design procedure.

3.2.1 Field Evaluation of SMA Construction Projects

Data and materials were collected from 11 SMA construction projects to help verify the mixture design procedure and to provide information to make changes to improve the procedure. Site reports for the eleven sites are provided in the Appendix - Volume V. Data collected from each field project included: aggregate type, aggregate quality, aggregate gradation, asphalt binder properties, asphalt cement modifier type, asphalt content, fiber type and amount, mineral filler type and amount, compaction method and effort, volumetrics, mortar properties, workability properties, draindown characteristics, in-place density, and permeability. In order to obtain all of this data, two subtasks were accomplished: field testing and laboratory testing.

3.2.1.1 Field Testing of SMA Construction Projects

For each of the 11 SMA construction projects, NCAT personnel visited the projects to collect data and to obtain the raw materials used in the SMA mixtures. Figure 3.3 illustrates the activities undertaken by NCAT personnel while at each of the SMA construction projects.

Upon arrival at the projects, NCAT personnel collected information about the SMA mixture and construction procedures used in producing the SMA. The raw materials (aggregates, asphalt binder, mineral filler, and stabilizing additives) used to produce the SMA mixture were sampled.

Figure 3.3 shows that the field produced SMA was sampled at six different times. Enough SMA mixture was sampled from each sampling interval to accomplish the following:

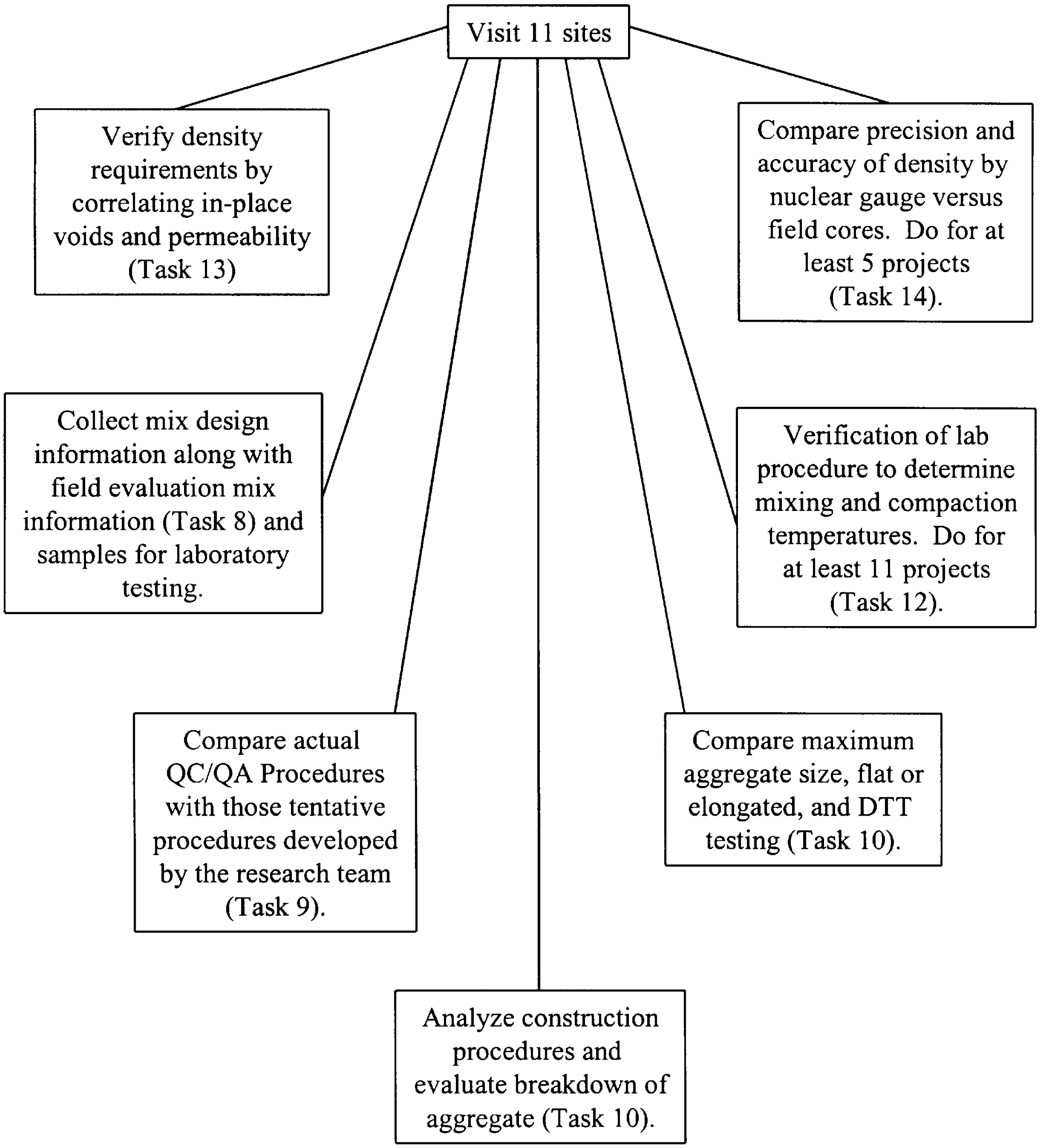


Figure 3.1: Flow Diagram for Collection of Field Data

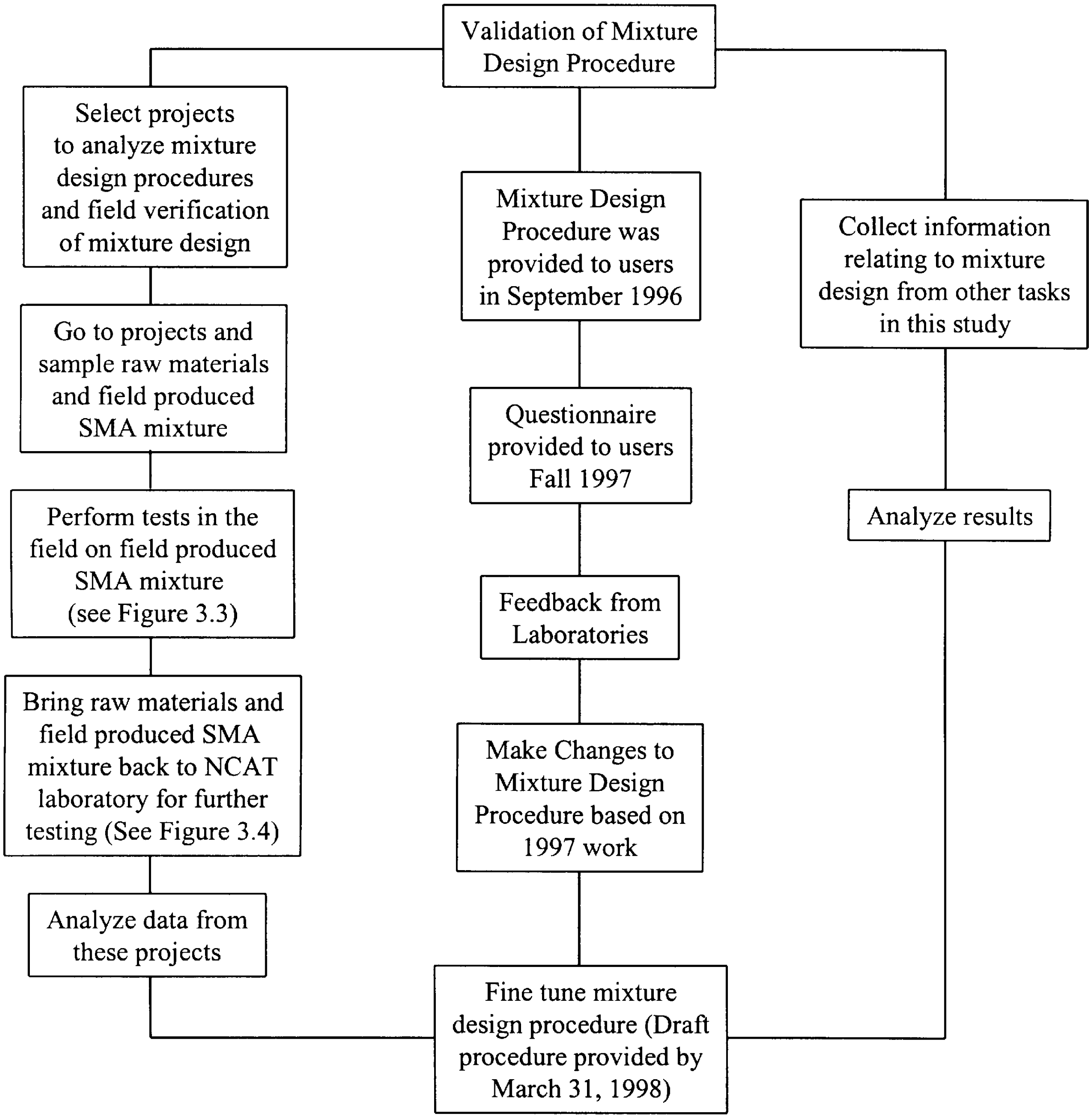


Figure 3.2: Flow Diagram for Validation of Mix Design Procedure

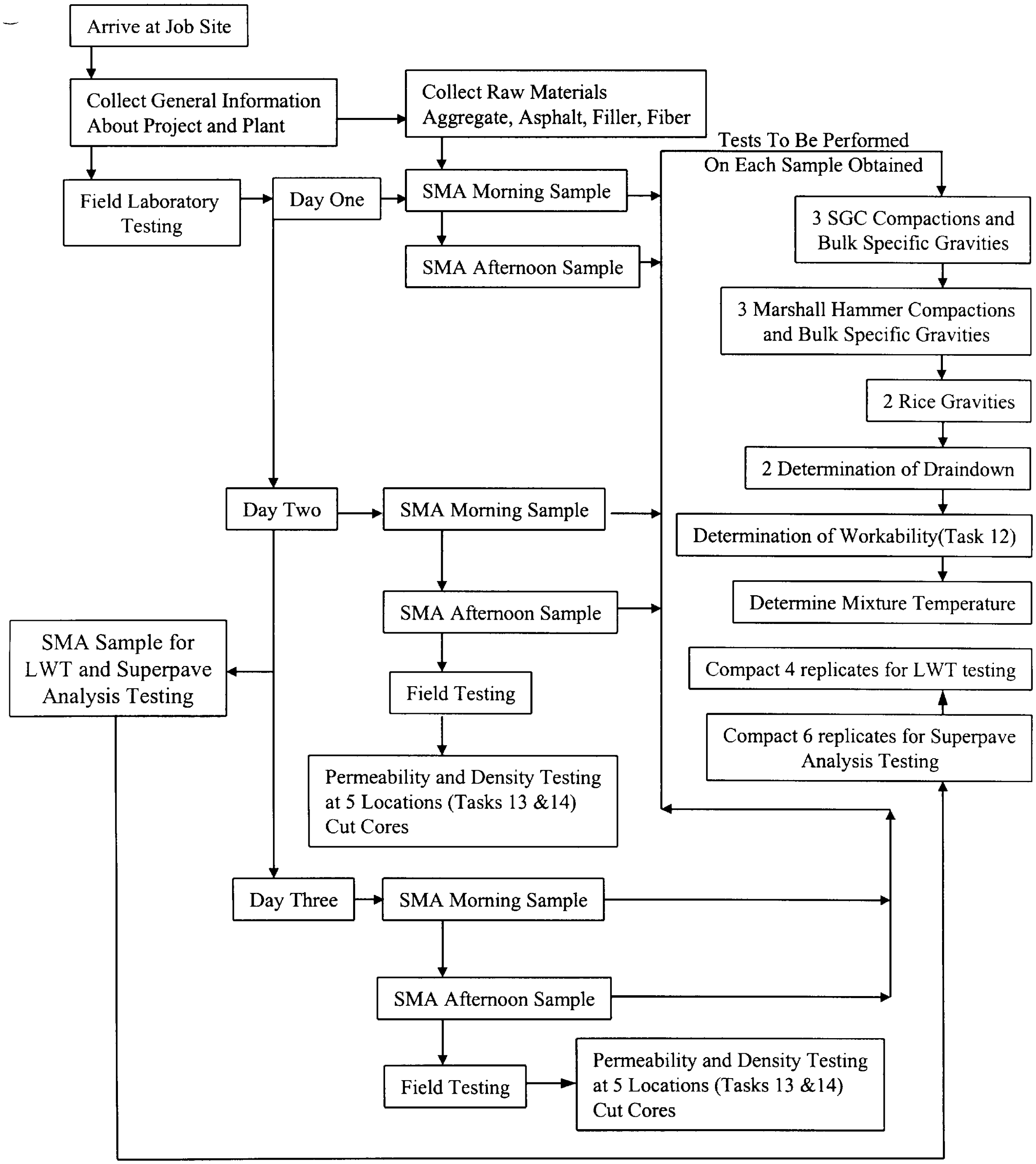


Figure 3.3: Activities and Tests at Each SMA Construction Project

1. Compact 3 replicates with SGC;
2. Compact 3 replicates with Marshall hammer.
3. Determine the theoretical maximum density (G_{mm}) for two replicates;
4. Determine draindown characteristics of the SMA for two replicates; and
5. Determine the temperature of the mixture.

In addition to the testing shown above, SMA was sampled for further testing at the NCAT laboratory. An additional sample (seventh sample) was obtained to compact specimens with the SGC for loaded wheel testing (LWT) and Superpave analysis testing. These compacted specimens were brought back to the NCAT laboratory for testing.

The testing protocol shown on Figure 3.3 was followed at each of the construction projects with only minor modifications. At two projects, the contractor was required to compact Marshall specimens as part of their QC/QA program. Therefore, NCAT personnel did not compact Marshall specimens on these two projects. The contractor’s data and specimens were obtained by NCAT for evaluation.

Also included on Figure 3.3 were permeability and nuclear density gauge testing and the determination of workability. These activities were not part of Task 8 and will be discussed under their respective tasks.

Since samples were obtained at six sampling intervals, a total of 18 specimens were compacted with both the SGC and Marshall hammer at each project. Sixteen of the 18 specimens compacted with the SGC were compacted to 100 revolutions (design number of gyrations). This number of revolutions was used because this is the value that the mixture design procedure requires. The remaining two specimens were compacted to 158 revolutions. This value represents the number of gyrations for $N_{maximum}$ using Superpave methodology. The bulk specific gravity (G_{mb}) for each of the compacted specimens (SGC and Marshall) was determined by AASHTO T 166.

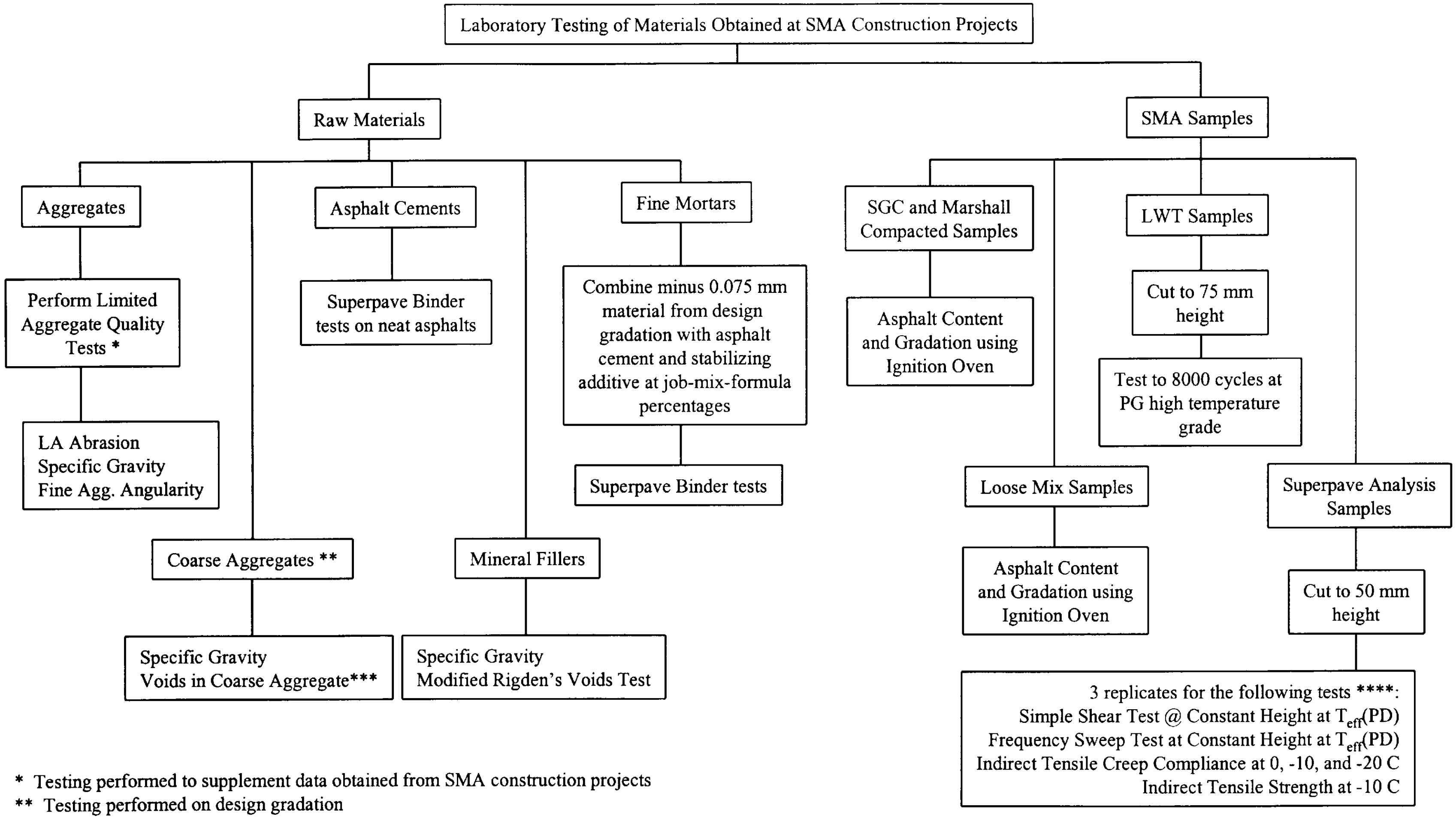
All of the Marshall hammer compacted specimens were compacted using 50 blows per face of a flat-faced, static base, mechanical Marshall hammer (AAASTO T245). The 50-blow compactive effort was selected based on the recommended mixture design procedure for SMA mixtures.

For each of the six sampling intervals, the G_{mm} was determined for two replicates (AASTHO T 209). This value along with the materials properties and G_{mb} for the SGC and Marshall compacted specimens allowed the volumetrics to be calculated.

The draindown characteristics of the uncompacted SMA was determined from two samples for each of the six sampling intervals at each project. Two replicates of uncompacted mixture were tested for each sampling interval. The test procedure used is presented in Volume IV of this report. The test temperature for all draindown testing was the respective mixing temperature for the SMA mixtures.

3.2.1.2 Laboratory Testing of Materials at SMA Construction Projects

Materials obtained from the SMA construction projects for further laboratory testing included: each of the raw materials used to produce the SMA mixtures, SGC and Marshall compacted specimens, loose SMA mixture, four SGC compacted specimens for LWT testing, and six SGC compacted specimens for Superpave analysis testing. Testing accomplished on these materials is illustrated in Figure 3.4.



* Testing performed to supplement data obtained from SMA construction projects
** Testing performed on design gradation
*** Dry-Rodded Test
**** Simple Shear Test and Frequency Sweep Test will be performed on the same samples

Figure 3.4: Laboratory Testing of Materials Obtained at SMA Construction Projects

Test results showing aggregate quality was obtained from each of the projects. However, in some instances test results were not available. Some limited aggregate quality testing was performed as a part of this study for some of the projects. This testing was limited to Los Angeles Abrasion (AASHTO T 96) on coarse aggregates, specific gravities (coarse and fine aggregates), and fine aggregate angularity (AASHTO TP 33, Method A).

Additional testing on the coarse aggregate included determining the voids in coarse aggregate (VCA) using the dry-rodded technique according to AASHTO T19. This testing was performed on coarse aggregates batched to meet the design gradations. Only two of the SMA projects provided data for the VCA for the coarse aggregates, therefore this testing was performed as part of this study for nine SMA construction projects.

Each of the asphalt cements were tested using the Superpave binder tests. For those projects that used performance graded (PG) binders, this testing was limited to verifying the PG grade. For those projects that did not use PG graded binders, the binders were tested to determine the PG grade.

For each of the construction projects the fine mortars were tested using the Superpave binder tests. The fine mortars were comprised of the material passing the 0.075 mm sieve from the design gradation, the asphalt binder, and any stabilizing additives. These constituents were batched to meet the job-mix formula percentages. Actual testing of the fine mortars was conducted at a temperature corresponding to the performance grading temperatures of the asphalt binders for each project.

Laboratory testing on the SGC and Marshall field compacted samples consisted of determining the asphalt content and gradation. Removal of the asphalt was accomplished with the ignition oven. This data along with the gradation of field cores obtained from the projects was used to evaluate aggregate breakdown in the field and in the laboratory.

Testing of the loose mixture obtained from the different projects also consisted of determining the asphalt content and gradation. This data was used as a reference to evaluate aggregate breakdown.

From each construction project, four specimens were compacted in the field to the design number of revolutions (100) for loaded wheel testing (LWT). These specimens were then brought back to the NCAT laboratory and cut to a height of 75 mm for testing. The test temperature for these samples was set at the respective high temperature binder grade designated for each location.

3.2.2 SMA Mixture Design Procedure Questionnaire

A questionnaire (Table 3.1) was sent to agencies with experience designing SMA mixtures. Responses to the Questionnaire were used to further refine the SMA mix design procedure.

3.3 TASK 9 - DEVELOP QUALITY CONTROL/QUALITY ASSURANCE PROCEDURES FOR SMA

Quality Control/Quality Assurance (QC/QA) procedures complete with a troubleshooting guide were developed based on current SMA knowledge. This is provided in Volume III of this report. As part of Task 9, QC/QA data was collected and analyzed from each of the 11 SMA

TABLE 3.1: Questionnaire for SMA Mix Design Procedure

Questionnaire for SMA Mix Design Procedure

The purpose of this questionnaire is to obtain feedback on the acceptability of the mixture design procedure for SMA mixtures developed under NCHRP 9-8. A copy of the procedure is enclosed with this questionnaire. Please take a few minutes and answer the questions provided below. Thank you for your assistance with this important project.

1. Have you used the proposed mixture design procedure for the design of SMA mixtures.
   Yes_____ No _____

2. Please review the enclosed mixture design procedure and send any comments or concerns to NCAT at the address provided on the next page. After your review, please answer the appropriate questions below.

3. Is the mixture design procedure easy to follow?

4. Is the procedure for selecting mixing and compaction temperatures acceptable? Any suggested improvements?

5. Were there any problems in conducting the mortar tests? If yes, please identify.

6. Do the criteria for the mortar test results appear acceptable for your SMA Mixtures? Any suggested changes?

7. Are you able to design mixtures meeting the VCA and VMA requirements?

8. Have you designed mixes for different maximum aggregate sizes? If yes, what sizes?

9. What types of laboratory compaction have you used for SMA mixtures?
   Marshall _____ Gyratory _____ Other (Specify) ____________________

10. Do your SMA mixtures meet the minimum asphalt content requirement? If no, what is a typical range of asphalt contents?

11. Do you have suggestions to improve the mix design procedure?

12. Please provide any general comments (pro or con) about the SMA mixture design procedure.

Test results showing aggregate quality was obtained from each of the projects. However, in some instances test results were not available. Some limited aggregate quality testing was performed as a part of this study for some of the projects. This testing was limited to Los Angeles Abrasion (AASHTO T 96) on coarse aggregates, specific gravities (coarse and fine aggregates), and fine aggregate angularity (AASHTO TP 33, Method A).

Additional testing on the coarse aggregate included determining the voids in coarse aggregate (VCA) using the dry-rodded technique according to AASHTO T19. This testing was performed on coarse aggregates batched to meet the design gradations. Only two of the SMA projects provided data for the VCA for the coarse aggregates, therefore this testing was performed as part of this study for nine SMA construction projects.

Each of the asphalt cements were tested using the Superpave binder tests. For those projects that used performance graded (PG) binders, this testing was limited to verifying the PG grade. For those projects that did not use PG graded binders, the binders

A troubleshooting guide was also included as part of the QC/QA procedures. This guide provides information to aid in determining the specific cause of potential problems. The proper corrective action when one or more mixture properties do not meet specifications was included as well.

3.4 TASK 10 - LABORATORY TESTING

The testing included in this task was designed to extend some of the results in Phase I of the research. Testing included in Task 10 focused on various nominal maximum aggregate sizes, determination of the effects of flat or elongated aggregates using the SGC, and Direct Tension Testing (DTT) of SMA fine mortars.

The SMA mixture design method established by Phase I was developed for only one nominal maximum aggregate size (19.0 mm). However, the method should be applicable to SMA mixtures of various maximum aggregate sizes. Extending the design procedure to any maximum aggregate size only required slight modifications. The major modification was to shift the sieve size that defines the break point between coarse aggregate and fine aggregate. This break point sieve is important because it establishes what portion of the aggregate constitutes the coarse aggregate skeleton. For SMA mixtures with a maximum nominal aggregate size of 19.0 mm, this breakpoint was defined as the 4.75 mm sieve.

Table 3.2 shows the test plan used to develop information on various maximum aggregate sizes. Five different nominal maximum aggregate sizes were used, 4.75, 9.5, 12.5, 19.0, and 25 mm. For the 4.75, 9.5, and 12.5 mm nominal maximum aggregate sizes, two different break point sieves were investigated. For the 25 mm nominal maximum aggregate size, three different break point sieves were employed. In addition, two 19.0 mm nominal maximum aggregate sizes with a 4.75 mm break point sieve were included. One of these combinations is the same mixture used during Phase I with traprock aggregate. The second 19.0 mm mixture was developed during Phase II using the Interim Report mixture design procedure and was designed having a gradation shape (as plotted on a 0.45 power chart) similar the other mixtures developed during Phase II (nominal maximum aggregate size experiment).

For each of the nominal maximum size/break point size combinations, three tests were performed. The voids in coarse aggregate (VCA) was determined for both the coarse aggregate only fraction and for the entire SMA mixture. This test was used to ensure an adequate coarse aggregate skeleton. The permeability test was used to determine if the permeability of SMA mixtures varies with nominal maximum aggregate size, and if so, in what manner. The wheel tracking test was used as a laboratory evaluation to determine when an adequate coarse aggregate skeleton was achieved. In addition to the testing accomplished on SMA mixtures of differing nominal maximum aggregate sizes, four mixture designs (Table 3.3) were performed using the Superpave Volumetric mixture design procedure. Testing of these mixtures was identical to that performed on the SMA mixtures and was accomplished for comparison purposes.

The flat and elongated testing in Task 10 was similar to the flat and elongated testing from Phase I with the exception that the SGC was used to compact the mixture specimens. A quantity of aggregate was obtained that was crushed by different methods. One method produced a mostly cubical particle shape, while the second method produced a large portion of flat and elongated particles. These two particle shapes were combined in various percentages to produce SMA mixtures. The mixtures were then tested to determine the aggregate gradations before and after compaction; the mixtures were also tested to determine their sensitivity to moisture. Table 3.4 shows the test plan used for flat and elongated particle testing.

The fine mortar DTT testing used the eight mortars selected for the mortar mixture from Phase I. This means that a complete set of data has been gathered for these eight SMA mixtures. Each of the fine mortars were tested in the Dynamic Shear (DSR) and Bending Beam Rheometers (BBR) during Phase I. A mixture design has also been completed for SMA mixtures containing each of the fine mortars using both the Marshall hammer and the SGC compaction techniques. Physical testing of these designs was also completed in Phase I. The DTT test plan for Task 10 is shown in Table 3.5.

3.5 TASK 11 - DEVELOP SMA CONSTRUCTION GUIDELINES

The main focus of this task was to develop SMA construction guidelines to assist producers during production and placement of SMA. The guidelines include guidance on aggregate handling, modifier handling, plant operations, laydown procedures, and compaction.

The guidelines were developed in two steps. First, SMA construction guidelines were developed based on the experience of NCAT staff and input from the SMA technical working group (TWG). Some of the guidance provided by the guidelines included aggregate handling, preferred methods to add mineral filler, preferred methods to add stabilizing additives (including pellets), effect of silo storage on SMA mixtures, trucking methods (release agents, tarps, condition of beds, etc.), and placement procedures.

The final step was to produce a final version of SMA construction guidelines. To accomplish this, all 11 SMA construction projects visited under Task 8 were monitored to evaluate construction procedures. Construction procedures evaluated included aggregate handling, plant type, addition of stabilizing additives, addition of mineral filler, method of placement, method of longitudinal joint construction, compaction procedures, etc. Problem areas were identified and solutions noted.

A laboratory study (Table 3.6) was also conducted to evaluate the potential damage to asphalt binder when it is heated to a high temperature. This effort involved six asphalt binders (three modified and three neat asphalt cements). Samples of each binder were heated to five

Table 3.2: Test Plan For Nominal Maximum Aggregate Sizes

Nom. Max. Agg. Size (mm)	4.75		9.5		12.5		19.0		25.0		
Break Point (mm)	2.36	1.18	4.75	2.36	9.5	4.75	4.75[1]	4.75[2]	12.5	9.5	4.75
VCA	3[A]	3	3	3	3	3	3	3	3	3	3
Permeability[B]	3	3	3	3	3	3	3	3	3	3	3
Wheel Tracking	3	3	3	3	3	3	3	3	3	3	3

[A] The number of replicates for each test.
[B] Samples were evaluated at 10, 30, and 50 gyrations.
[1] Gradation used during Phase I.
[2] Gradation developed in Phase II.

Table 3.3: Superpave Mixtures Included with Nominal Maximum Sizes Test Plan

Nominal Max. Agg. Size, mm	9.5		25.0	
Above or Below Restricted Zone	ARZ	BRZ	ARZ	BRZ
VCA	3	3	3	3
Permeability	3	3	3	3
Wheel Tracking	3	3	3	3

ARZ - Above the restricted zone.
BRZ - Below the restricted zone.

Table 3.4: Test Plan For Flat and Elongated Particles			
Aggregate Mixes	SMA Mix Design	Gradations After Compaction	Moisture Susceptibility Tests
100% A1	1[A]	3	3[B]
100% A2	1	3	3
75% A1 - 25% A2	1	3	3
50% A1 - 50% A2	1	3	3
25% A1 - 75% A2	1	3	3

[A] The number of replicates for each test.
[B] AASHTO T283
Aggregate A1 - Aggregate containing flat or elongated particles.
Aggregate A2 - Aggregate from same source as A1 but containing cubical particles.

Table 3.5: Direct Tension Testing Test Plan					
Mortar Composition				Test Temperature (°C)	
Asphalt Cement	Modifier	Filler	Fiber	-6	-18
AC-20	None	10% Traprock	None	4[A]	4
AC-20	4% SBS[B]	10% Traprock	None	4	4
AC-20	8% Polyolefin[C]	10% Traprock	None	4	4
AC-20	None	10% Traprock	Cellulose	4	4
AC-20	None	10% Traprock	Mineral	4	4
AC-20	None	8% Wimpey	Cellulose	4	4
AC-20	None	12% Dankalk	Cellulose	4	4
AC-20	None	10% SE Flyash	Cellulose	4	4

[A] The number of replicates tested.
[B] 4% SBS by mass of asphalt cement.
[C] Polyolefin by mass of asphalt cement.

Table 3.6: Test Plan For Potential Damage To Asphalt Cement Due To High Temperatures

Asphalt Binder	Oven Temperature (°C)				
	150°C	165°C	180°C	195°C	210°C
Neat AC No. 1	2 reps	2 reps	2 reps	2 reps	2 reps
Neat AC No. 2	2 reps	2 reps	2 reps	2 reps	2 reps
Neat AC No. 3	2 reps	2 reps	2 reps	2 reps	2 reps
Modified AC No. 1	2 reps	2 reps	2 reps	2 reps	2 reps
Modified AC No. 2	2 reps	2 reps	2 reps	2 reps	2 reps
Modified AC No. 3	2 reps	2 reps	2 reps	2 reps	2 reps

different temperatures in a forced draft oven using the thin film oven pans. Before and after the five hour heating, the Superpave binder properties (DSR, BBR, and DTT) were evaluated. An effort was made to determine a critical temperature below which no significant damage occurred and above which significant damage did occur.

3.6 TASK 12 - VERIFY LABORATORY MIXING AND COMPACTION TEMPERATURES

The SMA mixture design procedure established under Phase I research stated that laboratory mixing and compaction temperatures be determined using AASHTO T 245 (temperature-viscosity relationship). Task 12 evaluated this procedure to determine its applicability to field work.

To evaluate this procedure, information and materials were obtained from the eleven SMA field projects visited under Task 8. Information obtained included the laboratory mixing and compaction temperatures used for the project and the temperture of the SMA mixtures produced in the field. The Marshall and SGC specimens compacted during the field evaluation of SMA construction projects (Task 8) were compacted using respective laboratory compaction temperatures.

From each of the field projects, the mixture components comprising the SMA mixtures were obtained. To evaluate the mixing and compaction temperature, the fine mortars (minus fibers) were tested in the Brookfield viscometer (BV). Testing of these mortars consisted of determining the viscosity at five temperatures: at plant mixing temperature, ±20°F, and ±40°F. Based on this testing, a temperature-viscosity relationship was determined for the fine mortars from each project.

Another method employed to evaluate mixing and compaction temperatures was the development of a workability device. This equipment measures the torque required to rotate a paddle embedded in an SMA mixture. After performing this test at different mixture temperatures, it was then correlated to mixing and compaction temperatures. The development of the workability device was ongoing during the visitation of SMA construction projects. The final version of this equipment was only used during the construction of two projects. The device was used in the NCAT laboratory to test samples of loose mixture obtained from the remaining projects.

3.7 TASK 13 - VERIFY DENSITY REQUIREMENTS

Most SMA projects have been compacted to an in-place air void content of 5-6 percent. This appears to be acceptable, but there is some evidence that indicates that SMA mixtures are permeable to water at lower air void contents than are dense-graded mixtures. This problem has been observed in the laboratory and on at least two field projects. It may be necessary therefore to modify in-place air void requirements based on the permeability of the SMA mixtures. Task 13 evaluated this possibility.

This task evaluated the work on permeability accomplished in Phase I. As part of Phase I, a study was performed to evaluate the air void level at which SMA mixtures become permeable to water. Dense-graded mixtures become permeable to water at approximately 8 percent air voids. It is believed that SMA mixtures become permeable at around 6 percent air voids or slightly higher.

Tests were conducted on in-place mixtures to evaluate at what air void content SMA mixtures become permeable (Table 3.7). Field permeability tests were conducted at each location that a core was taken under Task 14. The bulk specific gravity of each core was determined to provide air void contents.

Table 3.7: Test Plan For Mixture Permeability

SMA Construction Project	No. of Samples	Voids	Permeability	
			Lab Method	Field Method
2	10	each sample	each sample	each sample
3	10	each sample	each sample	each sample
4	10	each sample	each sample	each sample
5	10	each sample	each sample	each sample
6	5	each sample	each sample	each sample
9	5	each sample	each sample	each sample
10	10	each sample	each sample	each sample
11	10	each sample	each sample	each sample

3.8 TASK 14 - ACCURACY AND PRECISION OF NUCLEAR GAUGE FOR DETERMINING FIELD DENSITY

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

Eight projects were selected on which a nuclear gauge was used to check density, cores were obtained to verify gauge results. On each of the eight projects, five or ten cores were obtained and compared to the nuclear gauge density (calibrated nuclear gauge) with and without leveling sand at the same location (Table 3.8 and Figure 3.5). Accuracy and precision of the nuclear gauge with and without sand was compared to that of the cores.

Table 3.8: Test Plan For Evaluating Precision and Accuracy of Nuclear Gauge

SMA Construction Project	No. of Samples	Core Density	Nuclear Density
2	10	for each sample	for each sample
3	10	for each sample	for each sample
4	10	for each sample	for each sample
5	10	for each sample	for each sample
6	5	for each sample	for each sample
9	10	for each sample	for each sample
10	10	for each sample	for each sample
11	10	for each sample	for each sample

3.9 TASK 15 - PREPARATION OF REPORTS

The requirement of this task was to produce a final report documenting the entire research effort (Phase I and Phase II). This report is written in five volumes. Volume I contains a review of all available, pertinent literature on SMA at the time the final report was written. The literature review was updated based on new information obtained since the interim report. Volume II provides research results developed in Phase I and Phase II. Volume III provides a summary of the entire research. Volume IV contains the final mixture design method, construction guidelines, and QC/QA procedures for SMA, and other deliverables. Volume V is an appendix which provides the raw data developed during the conduct of Phase II.

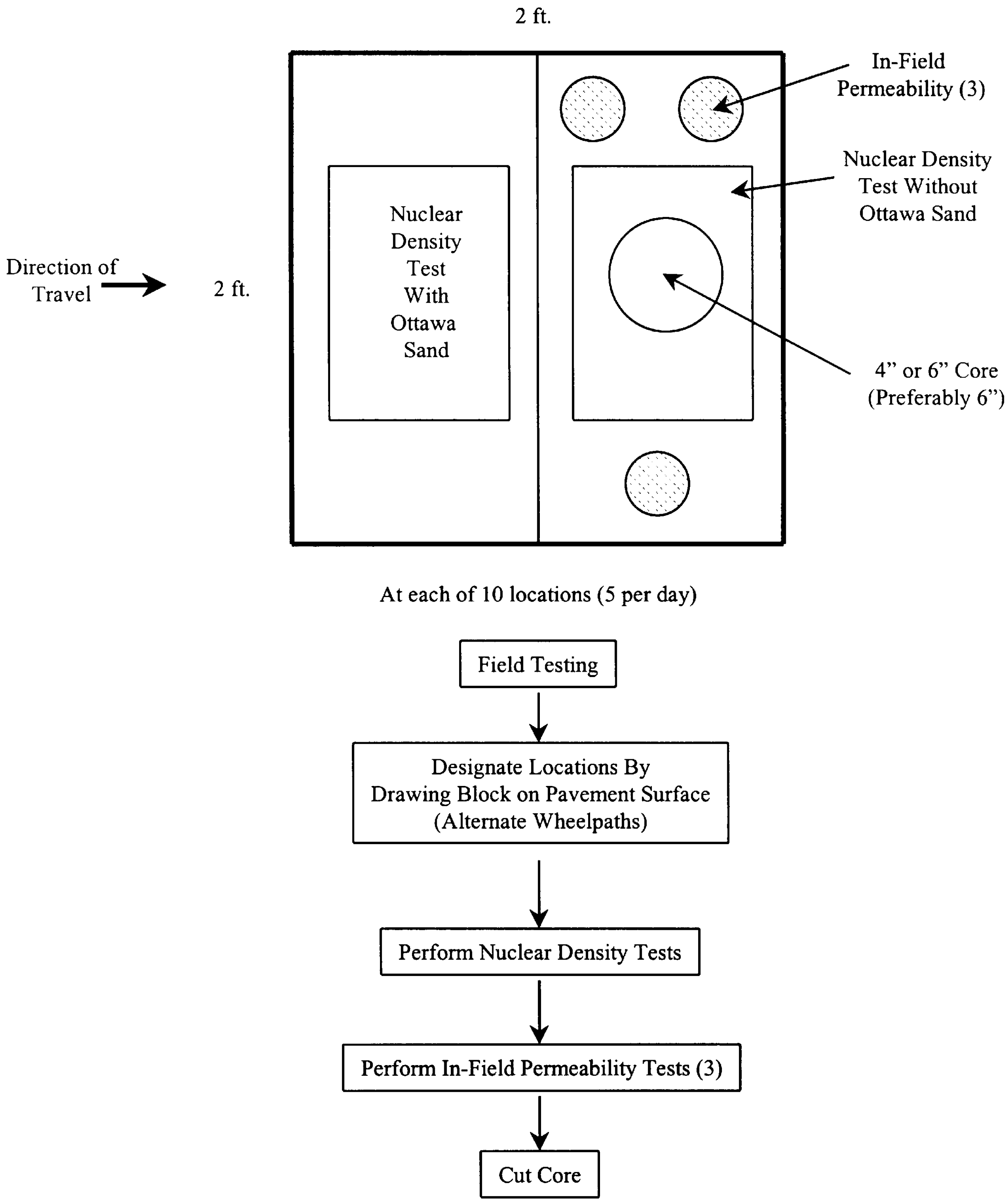


Figure 3.5: Field Density and Permeability Testing

CHAPTER 4 - TEST RESULTS AND ANALYSIS

4.1 FIELD EVALUATION OF SMA MIXTURE DESIGN PROCEDURE

Eleven SMA construction projects were visited to determine if SMA could be properly produced in the field when designed in the laboratory according to the mixture design procedure developed in Phase I. These projects were located in Georgia (2), Colorado, New Mexico, Maryland, Virginia, Missouri (2), Texas, Michigan, and South Carolina. Site reports along with all test results for the different construction projects are presented in the Appendix (Volume V).

4.1.1 SMA Mixture Information

Table 4.1 provides information about the SMA mixtures encountered at the different SMA construction projects visited. This data represents the design values for each of the mixtures.

Of the eleven projects visited, nine used Superpave performance graded (PG) asphalt binders; the other two types included an AC-20 and a multi-grade 20-40. All of the projects used some type of stabilizing additive. Six used both polymer modified asphalt cements and a fiber, four used only fibers, and one used only polymer modified asphalt.

Seven of the ten projects that utilized fibers used them in the loose form. The remaining three used pelletized fibers (sites 3, 5 and 6). For two of these projects, the pellets were comprised of 66 percent asphalt cement and 34 percent fiber (cellulose) while the third project (site 5) used a pellet comprised of only cellulose fiber. These pellets were held together by a small amount of glue. The pelletized fibers containing both asphalt and fiber were introduced at a dosage rate of 0.46 percent of total mix. The pelletized fiber containing only cellulose was introduced at a rate of 0.3 percent of total mix. Of those using loose fibers, two used a mineral fiber while the remaining projects used cellulose. For all projects using cellulose in a loose form, the dosage rate was 0.3 percent of the total mix and for the two projects that used mineral fiber the dosage rate was 0.4 percent of total mix.

Several different types of mineral fillers were used on the different projects, including: marble dust, limestone dust, dolomite dust, fly ash, and agricultural lime. Design percentages of mineral filler ranged from a low of 4.6 percent to a high of 15 percent. Eight of the projects used an anti-stripping additive. Five used hydrated lime, while the remaining three used a liquid anti-strip.

Of the eleven projects, ten used the Marshall hammer as the compactive effort during mixture design. Nine of these used 50 blows per face and one used 51 blows per face. The remaining project utilized a SGC. This mixture was designed using 100 gyrations, which corresponds to the number of gyrations at N_{design} in the SMA mixture design procedure developed under Phase I of this study. Specimens for this design were compacted to 158 gyrations ($N_{maximum}$). From $N_{maximum}$, the volumetric properties of this mixture were back calculated to the design number of gyrations (100 gyrations).

Design voids in total mix (VTM) ranged from 3.1 percent to 4.3 percent. Design voids filled with asphalt (VFA) ranged from 75.1 to 82 percent. All of the projects met the required VMA requirement of 17 percent minimum as specified in the SMA mixture design procedure.

TABLE 4.1: SMA Mixture Information For the Construction Projects Visited Under Task 8

	Site 1		Site 2	Site 3
	JMF-01-1	JMF-01-2		
Type Asphalt Binder Used:	PG 76-22	PG 76-22	PG 76-28	AC-20
Modifier Type:	Stylink	Stylink	Stylink	None
Design Asphalt Binder Content:	6.0 %	5.9 %	7.1 %	7.4 %
Type Fiber Used:	Slag Wool	Slag Wool	None	Cellulose
Form of Fiber:	Loose	Loose	N/A	Pellets (66/34)*
Design Percentage of Fiber:	0.4 %	0.4 %	N/A	0.46 %
Type Mineral Filler Used:	Marble Dust	Marble Dust	Limestone Dust	Perlite
Design Percentage of Mineral Filler:	8.0 %	8.0 %	4.6 %	11.0 %
Type Anti-Strip Additive Used:	Hyd. Lime	Hyd. Lime	Hyd. Lime	Liquid
Design Percentage of Anti-Strip:	1.0 %	1.0 %	1.0 %	1.0 %
Compaction Method:	Marshall	Marshall	Marshall	Marshall
Compactive Effort:	50 Blows/face	50 Blows/face	50 Blows/face	50 Blows/face
Laboratory Mixing Temperature:	335°F	335°F	325°F	312 ± 20°F
Laboratory Compaction Temperature:	325°F	325°F	300°F	292 ± 20°F
Design Voids in Total Mix:	3.7%	3.1 %	3.7 %	3.4 %
Design Voids in Mineral Aggregate:	17.9 %	17.6 %	17.5 %	17.1 %
Design Voids Filled With Asphalt:	79.3 %	82.0 %	78.9 %	80.1 %
Design G_{mb}:	2.430	2.483	2.317	2.111
Design G_{mm}:	2.524	2.563	2.406	2.186
Design Marshall Stability:	2110 lbs	2120 lbs	2590 lbs	1510 lbs
Design Flow (0.01 in):	14.0	13.6	15.0	13.3
Coarse Aggregate VCA:	38.3 percent	39.1 percent	41.3 percent	42.6 percent
SMA Mixture VCA:**	40.7 percent	40.9 percent	42.2 percent	39.2 percent
Design Laboratory Draindown:	----	----	----	----

* Indicates 66 percent asphalt and 34 percent fiber. ** Calculated based on job-mix-formula data.

TABLE 4.1 (continued): SMA Mixture Information For the Construction Projects Visited Under Task 8

	Site 4	Site 5	Site 6	Site 7
Type Asphalt Binder Used:	PG 76-22	PG 70-22	Multi-Grade 20-40	PG 64-28
Modifier Type:	SBS	None	None	SBS
Design Asphalt Binder Content:	5.7 %	6.5 %	7.1 %	6.5 %
Type Fiber Used:	Slag Wool	Cellulose	Cellulose	Cellulose
Form of Fiber:	Loose	Pellets	Pellets (66/34)*	Loose
Design Percentage of Fiber:	0.4 %	0.3 %	0.3 %	0.3 %
Type Mineral Filler Used:	Marble Dust	Limestone Dust	Dolomite Dust	Limestone Dust
Design Percentage of Mineral Filler:	10.0 %	11.0 %	15.0 %	9.2 %
Type Anti-Strip Additive Used:	Hydrated Lime	Liquid	Liquid	None
Design Percentage of Anti-Strip:	1.0 %	0.2 %	1.0 %	N/A
Compaction Method:	Marshall	SGC	Marshall	Marshall
Compactive Effort:	50 Blows/face	100 gyrations	51 Blows/face	50 Blows/face
Laboratory Mixing Temperature:	340°F	324 - 337°F	340°F	330 - 340°F
Laboratory Compaction Temperature:	325°F	299 - 310°F	290°F	312 - 324°F
Design Voids in Total Mix:	4.1 %	3.4 %	4.0 %	4.3 %
Design Voids in Mineral Aggregate:	17.1 %	19.4 %	20.0 %	17.5 %
Design Voids Filled With Asphalt:	75.1 %	79.4 %	80.0 %	75.5%
Design G_{mb}:	2.332	2.340	2.387	2.279
Design G_{mm}:	2.431	2.423	2.486	2.378
Design Marshall Stability:	1537 lbs	NA	----	2182 lbs
Design Flow (0.01 in):	11.2	NA	----	----
Coarse Aggregate VCA:	38.5 percent	41.9 percent	42.3 percent	44.0 percent
SMA Mixture VCA:**	39.8 percent	44.9 percent	39.8 percent	38.1 percent
Design Laboratory Draindown:	0.03 percent	----	----	----

* Indicates 66 percent asphalt and 34 percent fiber NA - Not Applicable

** Calculated based on job-mix-formula data.

TABLE 4.1 (continued): SMA Mixture Information For the Construction Projects Visited Under Task 8

	Site 8	Site 9	Site 10	Site 11
Type Asphalt Binder Used:	PG 64-28	PG 76-22	PG 76-22	PG 58-28
Modifier Type:	SBS	Stylink	Air-Blown	None
Design Asphalt Binder Content:	6.0 %	6.0 %	6.1 %	6.7 %
Type Fiber Used:	Cellulose	Cellulose	Cellulose	Cellulose
Form of Fiber:	Loose	Loose	Loose	Loose
Design Percentage of Fiber:	0.3 %	0.3 %	0.3 %	0.3 %
Type Mineral Filler Used:	Limestone Dust	Agricultural Lime	Limestone Dust	Fly Ash
Design Percentage of Mineral Filler:	10.0 %	10.0 %	9.0 %	7.0 %
Type Anti-Strip Additive Used:	None	Hydrated Lime	Liquid	None
Design Percentage of Anti-Strip:	N/A	1.0 %	0.5 %	N/A
Compaction Method:	Marshall	Marshall	Marshall	Marshall
Compactive Effort:	50 Blows/face	50 Blows/face	50 Blows/face	50 Blows/face
Laboratory Mixing Temperature:	368°F	307 - 320°F	----	315°F
Laboratory Compaction Temperature:	340°F	268 - 282°F	----	295°F
Design Voids in Total Mix:	4.3 %	3.6 %	3.2 %	3.3 %
Design Voids in Mineral Aggregate:	17.7 %	17.4 %	18.9 %	17.2 %
Design Voids Filled With Asphalt:	75.9 %	79.2 %	83.1 %	80.8 %
Design G_{mb}:	2.312	2.445	2.501	2.324
Design G_{mm}:	2.416	2.537	2.584	2.403
Design Marshall Stability:	1801 lbs	2183 lbs	----	----
Design Flow (0.01 in):	----	13.2	----	----
Coarse Aggregate VCA:	42.6 percent	40.9 percent	42.9 percent	40.0 percent
SMA Mixture VCA: **	38.9 percent	40.3 percent	43.7 percent	38.9 percent
Design Laboratory Draindown:	----	----	----	----

** Calculated based on job-mix-formula data.

During Phase I, it was shown that if the voids in coarse aggregate (VCA) of an SMA mixture are less than the VCA of the dry-rodded coarse aggregate, stone-on-stone contact is said to be achieved. Six of the eleven projects had design mixtures that did achieve stone-on-stone contact. However, it should be stated that only two projects reported the VCA of the dry-rodded coarse aggregate (VCA_{DRC}). For those projects not reporting VCA_{DRC} values, materials obtained from the projects were tested at the NCAT laboratory to determine VCA_{DRC}.

Table 4.2 presents the design gradations for each of the SMA mixtures. This table shows that seven (including JMF-01-1 and JMF-01-2) of the design gradations were 12.5 mm nominal maximum aggregate size gradations and five were 19.0 mm nominal maximum aggregate size gradations. Based on the gradation provided on the job-mix-formula for Site 6, it is unclear what the nominal maximum aggregate size would be because of lack of sufficient data.

The percent passing the 4.75 mm sieve for the different projects ranged from 26.1 percent to a high of 33 percent. The percent passing the 0.075 mm sieve ranged from 7.5 to 11.1 percent.

4.1.2 Material Properties

Stone Matrix Asphalt mixtures are made from coarse aggregates, fine aggregates, mineral fillers, asphalt cement, and stabilizing additives. The following paragraphs provide properties of these different constituents (except stabilizing additives) for each of the SMA construction projects visited. Portions of this data were obtained from the owners or contractors for the different projects and portions were determined at the NCAT laboratory.

For the different projects, aggregate quality testing was performed on either individual stockpiles or the combined gradation. Table 4.3 presents the results of the coarse aggregate quality testing obtained from the owners or contractors along with supplemental testing performed by NCAT.

Seven different types of coarse aggregates were encountered: granite, crushed basalt gravel, crushed “river rock”, rhyolite (a very hard granite), limestone, traprock, and a granite gneiss. Over these types of aggregates, the aggregate toughness varied greatly. The Los Angeles Abrasion loss values ranged from a low of 16 for the rhyolite used at site 6 to a high of 42 for the granite used at site 4. The bulk specific gravities ranged from a low of 2.355 for site 3 to a high of 2.84 for site 9. For all of the reported data, all of the projects used coarse aggregates with 100 percent of the particles with at least one crushed face. All of the projects but one used coarse aggregates with 100 percent two or more crushed faces; however, this project (site 2) had 98.5 percent with two or more crushed faces.

Table 4.4 presents the results of the fine aggregate quality testing obtained from the owners or contractors along with supplemental testing performed by NCAT. All of the projects but one used a fine aggregate. Site 6 used only two stockpiles plus a mineral filler. Neither of these two stockpiles can be characterized as a fine aggregate. Site 6 also had the highest percentage of mineral filler in the design gradation.

Results of the Superpave binder tests performed on the asphalt cements sampled from the different construction projects are presented in Table 4.5. The test temperatures used for these results correspond to the respective performance-grade (PG) test temperatures for high, intermediate, and low temperature gradings.

For those projects listed in Table 4.1 that did not use PG graded asphalt cements, part of the laboratory testing was accomplished to determine the PG grade. For site 3 an AC-20 was

TABLE 4.2: Design Gradations For the SMA Construction Projects Visited Under Task 8

Sieve Size, mm	Recommended Limits*	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	100.0	100.0	100.0	100.0	100.0	100.0	**	100.0	100.0	100.0	100.0	100.0
12.5	85-95	98.0	93.0	100.0	86.0	94.0	96.0	**	87.0	89.9	100.0	86.0	90.0
9.5	75 max.	70.0	70.0	87.0	67.0	69.0	83.0	66.7	63.1	63.3	75.5	74.0	59.2
4.75	20-28	28.0	29.0	30.0	27.0	29.0	33.0	26.1	27.5	26.2	27.5	28.0	27.0
2.36	16-24	19.0	19.0	19.0	17.0	19.0	19.0	**	20.0	19.5	20.7	19.0	21.5
1.18	----	15.0	16.0	15.0	**	17.0	16.0	**	16.3	16.0	17.1	**	17.4
0.60	12-16	14.0	14.0	13.0	14.0	16.0	15.0	**	14.1	14.3	15.2	15.0	12.7
0.30	12-15	13.0	13.0	11.0	13.0	14.0	14.0	**	13.1	13.5	13.7	**	10.7
0.150	----	12.0	12.0	9.0	**	12.0	12.0	**	11.4	11.9	11.4	**	8.8
0.075	8-10	10.0	10.0	8.0	9.7	10.0	9.0	11.1	8.9	9.5	8.5	8.0	7.5

* Based on Guidelines for SMA NAPA Publication
** Not Specified in JMF

Table 4.3: Properties of the Coarse Aggregates For the Construction Projects Visited Under Task 8

Property	Site 1			Site 2	Site 3	Site 4
	No. 7's - Granite (JMF-01-1)	No. 7's - Granite (JMF-01-2)	No. 89's - Granite (Both JMFs)	Combined Gradation - Basalt	-3/4" - Crushed "River Rock"	No. 78's - Granite
Los Angeles Abrasion, % Loss	19	18	**	20.6	23.2*	42
Flat & Elongated, %						
3 to 1	11	12	25	**	47*	10
5 to 1	**	**	**	**	3*	**
Bulk/Apparent Sp. Gr.	2.710/2.741	2.824/2.858	2.824/2.858	2.595/**	2.346/2.549*	2.595/2.644
Absorption, %	0.42	0.40	0.40	**	3.4*	0.75
Soundness (5 Cycles) % Loss	0.8[a]	0.2[a]	0.2[a]	3[a]	**	1.1[a]
Crushed Content, %						
One Face	100	100	100	100	**	100
Two Faces	100	100	100	98.5	**	100

** - Data Not Available
* - Performed at NCAT Laboratory
[a] - Sodium Sulfate Soundness

Table 4.3 (Continued): Properties of the Coarse Aggregates For the Construction Projects Visited Under Task 8

Property	Site 4	Site 5	Site 6		Site 7	
	No. 7's - Granite	Combined Gradation - Granite	WD ¾" - Rhyolite	WD Type D - Rhyolite	Limestone	Traprock
Los Angeles Abrasion, % Loss	42	25.2*	16	16	23	19
Flat & Elongated, % 3 to 1	9	**	9.8*	30.7*	27.6*	43.3*
Flat & Elongated, % 5 to 1	**	5.0	2.5*	6.1*	9.9*	17.5*
Bulk/Apparent Sp. Gr.	2.595/2.644	2.675/2.714	2.726/**	2.726/**	2.616/2.734*	2.610/2.645*
Absorption, %	0.75	0.5	**	**	1.7*	0.5*
Soundness (5 Cycles) % Loss	1.1[a]	**	5[b]	5[b]	2.7-6.2[c]	1[c]
Crushed Content, % One Face	100	100	100	100	100*	100*
Crushed Content, % Two Faces	100	100	100	100	100*	100*

** - Data Not Available
* - Performed at NCAT Laboratory
[a] - Sodium Sulfate Soundness
[b] - Magnesium Sulfate Soundness
[c] - Method Similar to AASHTO T103

Table 4.3 (Continued): Properties of the Coarse Aggregates For the Construction Projects Visited Under Task 8

Property	Site 8		Site 9	Site 10		Site 11
	Limestone	Traprock	Granite Gneiss	68's - Traprock	8's - Traprock	Limestone
Los Angeles Abrasion, % Loss	28	19	27	13.0	17.4	33.0
Flat & Elongated, %						
3 to 1	16.0*	43.5*	**	21.2*	16.9*	7.0*
5 to 1	0.0*	11.2*	**	3.1*	0.4*	1.0*
Bulk/Apparent Sp. Gr.	2.640/2.733*	2.604/2.671*	2.82/2.84	2.930/3.006	2.920/3.004	2.625/2.783*
Absorption, %	1.3*	1.0*	0.3	0.9	1.0	2.2*
Soundness (5 Cycles) % Loss	0.1	1[c]	1.2[a]	**	**	**
Crushed Content, %						
One Face	100*	100*	100	100	100	100
Two Faces	100*	100*	100	100	100	100

** - Data Not Available
* - Performed at NCAT Laboratory
[a] - Sodium Sulfate Soundness
[b] - Magnesium Sulfate Soundness
[c] - Method Similar to AASHTO T103

Table 4.4: Properties of the Fine Aggregates For the Construction Projects Visited Under Task 8

Property	Site 1	Site 2	Site 3	Site 4	Site 5	Site 7
	No. 810's - Granite	Combined Aggregates - Basalt	Combined Aggregates - "River Rock"	W10's - Granite	No. 8's - Limestone	Traprock
Bulk/Apparent Sp. Gr.	2.822/2.854	2.595/**	2.353/ 2.608*	2.599/2.623	2.663/**	2.499/2.652*
Absorption, %	0.41	**	4.2*	0.49	**	2.3*
Soundness, % Loss	**	3[b]	**	**	**	0.1[a]
Angularity, %	48.5	47.9*	48.9*	46.7*	46.2*	45.7*
Liquid Limit, %	**	NL	**	**	**	**
Plasticity Index, %	**	NP	**	**	**	NP

** - Data Not Available
* - Performed at NCAT Laboratory
[a] - Method Similar to AASHTO T103
[b] - Sodium Sulfate Soundness

Table 4.4 (continued): Properties of the Fine Aggregates For the Construction Projects Visited Under Task 8

Property	Site 8	Site 9	Site 10	Site 11	
	Traprock	Regular Screenings - Granite	Natural Sand	Manufactured Sand	Fines Crushed
Bulk/Apparent Sp. Gr.	2.401/2.656*	2.819/2.872*	2.610/2.846*	2.560/2.673*	2.481/2.740*
Absorption, %	4.0*	0.01*	3.2*	1.7*	3.8*
Soundness, % Loss	0.1[a]	**	**	**	**
Angularity, %	42.3	49.3*	47.1*	43.5*	41.4
Liquid Limit, %	**	**	**	**	**
Plasticity Index, %	NP	**	**	**	**

N/A - Data Not Available
* - Performed at NCAT Laboratory
[a] - Method Similar to AASHTO T103

Table 4.5: Results of Superpave Binder Testing for Asphalt Cements Used At SMA Construction Projects										
SMA Project	DSR Results					BBR Results (After PAV)			Direct Tension Results	
	G*/sinδ (kPa)			G*sinδ (kPa)		Test Temp. (°C)	S (MPa)	m (slope)	Failure Strain, %	Peak Stress, mPa
	Test Temp. (°C)	No Aging	After RTFO	Test Temp. (°C)	After PAV					
Site 1*	76	1.37	3.01	31	1298	-12	174	0.300	2.23	3.94
Site 2	76	1.40	2.52	28	1150	-18	211	0.372	2.33	3.77
Site 3	64	1.34	2.99	25	1620	-12	98	0.305	1.95	2.38
Site 4	76	1.51	2.74	31	1100	-12	157	0.324	2.59	2.60
Site 5	70	1.30	2.89	28	3636	-12	293	0.350	0.76	3.70
Site 6	64	1.71	4.43	25	1745	-12	123	0.330	2.76	3.32
Site 7	64	1.31	3.01	25	1150	-12	126	0.371	2.83	2.14
Site 8	64	1.46	3.35	22	1722	-18	132	0.304	2.17	3.27
Site 9	76	1.56	3.09	31	1243	-12	122	0.314	1.52	3.35
Site 10	76	1.28	3.88	31	1638	-12	226	0.347	1.52	3.97
Site 11	58	1.51	3.79	19	4739	-18	288	0.307	0.41	1.51

* Used with JMF-01-2

utilized while at site 6 a multi-grade 20-40 was used. Both of these asphalt cements were graded as a PG 64-22 using the Superpave performance grading system. Recall that this testing was performed at the NCAT laboratory on samples of asphalt cement obtained at the different projects. Of those projects that used PG graded binders, only one did not meet the PG grade presented on the job-mix-formula (JMF), site 7. The JMF stated that the asphalt cement was a PG 64-28, while it graded at the NCAT laboratory as a PG 64-22. Therefore, results in Table 4.5 for site 7 are presented at temperatures corresponding to a PG 64-22.

Table 4.6 presents the results of testing accomplished on the mineral fillers sampled from the different SMA construction projects.

4.1.3 Field Testing

Results of field testing associated with Task 8 are presented in Table 4.7. This table provides results for the volumetric properties of both the Marshall and SGC compacted specimens, asphalt contents, draindown, and G_{mm} for each of the samples obtained at the different construction projects. From eight of the field projects, cores were obtained after compaction. Table 4.8 presents air voids for each of the cores obtained. Table 4.9 presents gradation analyses performed for loose mixture samples obtained for each sample. These gradations represent the gradation of the SMA mixtures before any compactive effort. The following subsections provide observations about these results for each of the 11 sites.

4.1.3.1 Site 1 Field Testing

While at construction project No. 1, the contractor used two different job-mix-formulas (JMFs). Samples 1 through 3 were representative of one JMF (JMF-01-1) while samples 4 through 6 were representative of another JMF (JMF-01-2). The primary difference between the two JMFs was the coarse aggregate sources used. Based on Table 4.2, the two gradations were very similar; the primary difference being the percent passing the 12.5 mm sieve (98 percent for JMF-01-1 and 93 percent for JMF-01-2).

The volumetric properties between the two JMFs and the two compactors do appear to be different. For instance, the VCA/VCA_{DRC} values for the SGC compacted specimens were collectively lower than the Marshall compacted specimens. Also, the VMA values (again SGC samples) for the JMF-01-2 specimens seem to be higher than the JMF-01-1 specimens.

Collectively for both JMFs, the data shows that these mixtures did not achieve stone-on-stone contact as all of the VCA/VCA_{DRC} ratios were above 1.0, except sample 1. This indicates that only sample 1 would have stone-on-stone contact. This should be expected as the mixture design values did not obtain the desired ratio. Also, the draindown values are higher than the recommended limit of 0.3 percent (again with the exception of sample 1). Interestingly, fat spots were noticed on the roadway while at this project. Sample 4 was taken very close in time to when these fat spots were noticed. The draindown value for sample 4 was 0.8 percent which is above the recommended limit. One possible reason for the high draindown values could be that the percent passing the 0.075 mm sieve was lower than the design values for samples 2 through 6. The design value of percent passing the 0.075 mm sieve was 10.0 percent. Table 4.8 shows that for samples 2 through 6, the percent passing the 0.075 mm sieve ranged from 8.9 to 9.9 percent.

Table 4.6: SMA Construction Projects Mineral Filler Properties

Size (µm)	SMA Construction Project Mineral Fillers (Cumulative Percent Passing by Volume)[1]										
	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6	Site 7*	Site 8*	Site 9	Site 10	Site 11
2,360	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
1,180	100.0	100.0	100.0	100.0	96.0	100.0	100.0	100.0	93.5	100.0	100.0
600	100.0	100.0	100.0	100.0	91.6	100.0	100.0	100.0	86.5	99.5	100.0
300	98.9	99.7	99.3	99.3	88.1	100.0	100.0	100.0	81.7	95.4	100.0
150	84.3	93.2	96.4	87.5	80.8	99.9	100.0	100.0	72.2	85.0	97.3
75	59.8	78.5	84.9	62.6	54.3	93.5	100.0	100.0	58.4	72.5	85.2
45	45.4	70.8	74.7	47.7	42.2	75.7	96.2	96.2	48.0	65.1	71.7
20	29.5	53.6	51.8	30.8	27.9	48.7	80.3	80.3	33.3	54.4	50.9
ASG[2]	2.679	2.727	2.374	2.717	2.752	2.918	2.753	2.753	2.833	2.729	2.143

[1] - Determinations made using a Coulter LS-200 laser particle size analyzer.
[2] - ASG: Apparent Specific Gravity determined by AASHTO T-100
* Sites 7 and 8 used the same mineral filler.

Table 4.7: Results of Field Testing At SMA Construction Projects

Site 1[A]						
Marshall Hammer Compaction Results						
	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
G_{mb}	2.449	2.397	2.418	2.401		2.422
VTM, %	4.1	6.3	5.3	7.2	Sample	5.6
VMA, %	15.6	17.5	16.8	17.3	Not	16.7
VFA, %	73.9	63.0	67.3	58.5	Obtained	66.6
VCA, %	39.8	41.1	40.9	41.9		41.5
VCA/VCA_{DRC}	0.996	1.028	1.023	1.072		1.062
Superpave Gyratory Compactor Results						
G_{mb} @ N_{des}	2.513	2.400	2.416	2.400	2.422	2.404
% G_{mm} @ N_{ini}	87.2	83.2	84.2	83.4	85.7	84.8
% G_{mm} @ N_{des}	98.4	93.8	94.6	92.8	94.4	93.7
% G_{mm} @ N_{max}	**	94.9[B]	95.8[B]	**	**	**
VTM @ N_{des}	1.6	6.2	5.4	7.2	5.8	6.3
VMA @ N_{des}	13.8	17.7	17.2	18.5	18.2	18.1
VFA @ N_{des}	88.5	65.2	68.9	61.0	69.8	65.9
VCA, %	38.5	41.3	40.9	42.8	42.6	42.7
VCA/VCA_{DRC}	0.969	1.038	1.030	1.093	1.087	1.092
Loose Mixture Results						
Asphalt Content, %	5.7	5.7	5.8	5.7	6.2	5.8
Draindown, %	0.3	0.8	0.7	0.8	0.9	1.2
G_{mm}	2.554	2.559	2.554	2.587	2.565	2.566

[A] Site 1 used two job-mix-formula during sampling. Samples 1, 2, and 3 were sampled from one JMF while samples 4, 5, and 6 were sampled from another JMF.
[B] One of three replicates was compacted to Nmax.
** Not determined.

Table 4.7 (Continued): Results of Field Testing At SMA Construction Projects

Site 2						
Marshall Hammer Compaction Results[A]						
	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
G_{mb}	2.288	2.446				
VTM, %	6.5	7.0	Sample	Sample	Sample	Sample
VMA, %	19.1	19.2	Not	Not	Not	Not
VFA, %	66.5	63.4	Obtained	Obtained	Obtained	Obtained
VCA, %	42.9	43.0				
VCA/VCA_{DRC}	1.038	1.041				
Superpave Gyratory Compactor Results						
G_{mb} @ N_{des}	2.265	2.217	2.186	2.100	2.294	2.256
% G_{mm} @ N_{ini}	82.8	80.6	79.2	77.1	84.3	81.7
% G_{mm} @ N_{des}	93.1	90.8	89.0	86.8	94.9	92.3
% G_{mm} @ N_{max}	**	**	90.4[C]	**	**	**
VTM @ N_{des}	8.9	9.2	11.0	13.2	5.1	7.7
VMA @ N_{des}	18.9	20.9	21.7	24.7	17.9	19.5
VFA @ N_{des}	63.4	56.3	49.5	46.4	72.0	60.4
VCA, %	42.7	44.2	44.8	46.8	42.0	43.2
VCA/VCA_{DRC}	1.034	1.069	1.083	1.134	1.018	1.045
Loose Mixture Results						
Asphalt Content, %[B]	5.8	6.2	5.9	5.7	5.9	6.2
Draindown, %	0.9	1.4	1.2	1.3	1.0	0.8
G_{mm}	2.433	2.440	2.456	2.420	2.416	2.445

[A] Marshall samples compacted by contractor. Data based on information obtained from contractor.
[B] Asphalt Content results based on samples obtained by NCAT.
[C] Two of three replicates were compacted to Nmax.
** Not Determined

Table 4.7 (Continued): Results of Field Testing At SMA Construction Projects

Site 3						
Marshall Hammer Compaction Results[A]						
	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
G_{mb}	2.093	2.091	2.004	1.993		
VTM, %	4.8	4.5	8.7	9.1	Sample	Sample
VMA, %	17.9	18.1	21.8	22.2	Not	Not
VFA, %	73.3	75.1	60.2	58.9	Obtained	Obtained
VCA, %	39.8	39.9	42.6	42.9		
VCA/VCA_{DRC}	0.933	0.936	0.999	1.006		
Superpave Gyratory Compactor Results						
G_{mb} @ N_{des}	2.103	2.073	2.052	2.027	2.048	2.095
% G_{mm} @ N_{ini}	85.3	83.0	82.6	82.2	82.4	83.8
% G_{mm} @ N_{des}	96.5	94.9	94	92.7	93.7	95.6
% G_{mm} @ N_{max}	**	96.4[B]	**	**	**	**
VTM @ N_{des}	3.5	5.1	6.0	7.3	6.3	4.4
VMA @ N_{des}	17.9	19.1	20.0	20.8	19.7	18.3
VFA @ N_{des}	80.5	73.2	69.8	65.0	68.0	76.3
VCA, %	39.7	40.6	41.3	41.9	41.0	40.0
VCA/VCA_{DRC}	0.933	0.954	0.969	0.983	0.964	0.940
Loose Mixture Results						
Asphalt Content, %	7.9	7.9	8.0	7.8	7.5	8.0
Draindown, %	0.1	0.1	0.2	0.1	0.1	0.1
G_{mm}	2.179	2.184	2.184	2.186	2.186	2.190

[A] Marshall samples compacted by contractor. Data based on information obtained from contractor.
[B] Two of three replicates were compacted to Nmax.
** Not Determined

Table 4.7 (Continued): Results of Field Testing At SMA Construction Projects						
Site 4						
Marshall Hammer Compaction Results						
	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
G_{mb}	2.328	2.324	2.332	2.232	2.208	2.317
VTM, %	3.9	3.9	3.6	8.1	9.2	4.2
VMA, %	15.2	15.5	15.1	18.6	19.5	15.7
VFA, %	74.0	74.9	76.2	56.4	52.7	73.6
VCA, %	39.9	40.2	39.9	42.4	43.0	40.3
VCA/VCA_{DRC}	1.038	1.044	1.037	1.102	1.119	1.048
Superpave Gyratory Compactor Results						
G_{mb} @ N_{des}	2.349	2.367	2.362	2.252	2.334	2.380
% G_{mm} @ N_{ini}	84.7	85.0	85.0	81.1	79.9	85.9
% G_{mm} @ N_{des}	96.9	97.9	97.6	92.7	91.8	98.5
% G_{mm} @ N_{max}	**	98.9[A]	**	**	93.7[A]	**
VTM @ N_{des}	3.1	2.1	2.4	7.3	8.2	1.5
VMA @ N_{des}	14.4	13.9	14.0	18.0	18.7	13.4
VFA @ N_{des}	78.6	85.2	83.3	59.1	55.8	88.7
VCA, %	39.4	39.0	39.1	41.9	42.4	38.7
VCA/VCA_{DRC}	1.024	1.015	1.017	1.090	1.103	1.005
Loose Mixture Results						
Asphalt Content, %	5.7	5.9	5.8	5.7	5.7	5.8
Draindown, %	1.0	0.7	0.8	2.7	2.8	0.8
G_{mm}	2.424	2.418	2.419	2.430	2.433	2.417

[A] One of three replicates compacted to Nmax.
** Not Determined

Table 4.7 (Continued): Results of Field Testing At SMA Construction Projects

Site 5						
Marshall Hammer Compaction Results[A]						
	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
G_{mb}	2.267	2.266	2.311	2.301	2.333	2.396
VTM, %	6.8	7.1	5.5	5.8	4.8	2.0
VMA, %	20.5	20.6	19.1	19.5	18.2	16.0
VFA, %	66.7	65.7	71.4	70.3	73.9	87.2
VCA, %	46.5	46.6	45.6	45.8	45.0	43.5
VCA/VCA_{DRC}	1.111	1.112	1.088	1.093	1.073	1.038
Superpave Gyratory Compactor Results[A]						
G_{mb} @ N_{des}	2.292	2.297	2.340	2.328	2.331	2.360
% G_{mm} @ N_{ini}	82.9	83.0	85.0	85.2	84.9	85.0
% G_{mm} @ N_{des}	94.2	94.2	95.7	95.3	95.2	96.4
% G_{mm} @ N_{max}	**	95.2[B]	96.8[B]	**	96.8[C]	98.0[C]
VTM @ N_{des}	5.8	5.8	4.3	4.7	4.8	3.6
VMA @ N_{des}	19.7	19.6	18.1	18.5	18.2	17.4
VFA @ N_{des}	70.5	70.3	76.4	74.9	73.9	79.2
VCA, %	45.9	45.9	44.9	45.2	45.0	44.4
VCA/VCA_{DRC}	1.096	1.095	1.072	1.078	1.073	1.059
Loose Mixture Results						
Asphalt Content, %	6.2	6.3	6.4	6.4	6.2	6.2
Draindown, %	0.1	0.1	0.2	0.2	**	**
G_{mm}	2.433	2.438	2.445	2.442	2.449	2.446

[A] Samples 5 and 6 were compacted by contractor.
[B] One of three replicates compacted to Nmax.
[C] All three replicates compacted to Nmax.
** Not Determined

Table 4.7 (Continued): Results of Field Testing At SMA Construction Projects

Site 6						
Marshall Hammer Compaction Results						
	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
G_{mb}	2.393	2.361	2.389	2.351	2.339	2.373
VTM, %	3.5	6.0	4.1	6.2	6.0	5.1
VMA, %	19.4	19.9	19.4	20.5	21.2	20.1
VFA, %	81.9	69.7	78.7	70.0	72.0	74.5
VCA, %	39.8	40.2	39.8	40.7	41.2	40.4
VCA/VCA_{DRC}	0.940	0.950	0.940	*0.960*	0.972	0.954
Superpave Gyratory Compactor Results						
G_{mb} @ N_{des}	2.364	2.358	2.399	2.317	2.394	2.349
% G_{mm} @ N_{ini}	86.9	84.4	87.2	83.8	87.2	84.8
% G_{mm} @ N_{des}	95.3	93.4	96.3	92.4	96.3	93.9
% G_{mm} @ N_{max}	**	**	**	**	97.1[A]	95.8[A]
VTM @ N_{des}	4.7	6.6	3.7	7.6	3.7	6.1
VMA @ N_{des}	20.4	20.1	19.0	21.7	19.3	20.9
VFA @ N_{des}	77.2	67.1	80.4	65.2	80.9	71.0
VCA, %	40.5	40.3	39.5	41.5	39.7	41.0
VCA/VCA_{DRC}	0.957	0.952	0.933	0.981	0.938	0.967
Loose Mixture Results						
Asphalt Content, %	7.3	6.7	7.1	7.0	7.3	7.4
Draindown, %	**	0.1	0.1	0.03	0.04	0.03
G_{mm}	2.480	2.525	2.492	2.506	2.488	2.502

[A] One of three replicates compacted to Nmax.
** Not Determined

Table 4.7 (Continued): Results of Field Testing At SMA Construction Projects

Site 7						
Marshall Hammer Compaction Results						
	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
G_{mb}	2.339	2.301	2.306	2.325	2.286	2.330
VTM, %	2.1	3.5	3.1	2.6	5.0	2.6
VMA, %	14.8	16.1	16.0	15.0	16.5	14.8
VFA, %	85.6	78.3	80.6	82.5	70.0	82.7
VCA, %	39.2	40.2	40.1	39.4	40.4	39.3
VCA/VCA_{DRC}	0.892	0.913	0.911	0.895	0.919	0.892
Superpave Gyratory Compactor Results						
G_{mb} @ N_{des}	2.358	2.331	2.329	2.338	2.327	2.364
% G_{mm} @ N_{ini}	85.8	85.5	85.8	85.5	85.0	86.0
% G_{mm} @ N_{des}	98.7	97.8	97.8	97.9	96.8	96.8
% G_{mm} @ N_{max}	**	**	**	99.2[A]	**	**
VTM @ N_{des}	1.3	2.2	2.2	2.1	3.2	1.2
VMA @ N_{des}	14.1	15.0	15.2	14.5	14.9	13.6
VFA @ N_{des}	90.4	85.1	85.8	85.7	78.4	91.4
VCA, %	38.7	39.4	39.5	39.1	39.3	38.4
VCA/VCA_{DRC}	0.811	0.895	0.898	0.888	0.894	0.873
Loose Mixture Results						
Asphalt Content, %	6.4	6.3	6.4	6.1	6.1	6.1
Draindown, %	**	**	**	0.03	0.05	0.05
G_{mm}	2.390	2.384	2.380	2.388	2.405	2.392

[A] Two of three replicates compacted to Nmax.
** Not Determined

Table 4.7 (Continued): Results of Field Testing At SMA Construction Projects						
Site 8						
Marshall Hammer Compaction Results						
	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
G_{mb}	2.344	2.304	2.332	2.304	2.330	2.298
VTM, %	4.2	5.4	4.1	5.3	4.0	5.6
VMA, %	14.8	15.7	15.2	16.1	15.2	16.2
VFA, %	71.5	65.7	72.9	67.0	73.4	65.8
VCA, %	38.2	38.9	38.5	39.2	38.5	39.3
VCA/VCA_{DRC}	0.896	0.913	0.904	0.919	0.903	0.921
Superpave Gyratory Compactor Results						
G_{mb} @ N_{des}	2.378	2.359	2.385	2.350	2.370	2.338
% G_{mm} @ N_{ini}	85.1	83.8	86.0	84.0	85.7	83.9
% G_{mm} @ N_{des}	97.2	96.8	98.1	96.6	97.6	96.1
% G_{mm} @ N_{max}	**	**	**	**	**	**
VTM @ N_{des}	2.8	3.2	1.9	3.4	2.4	3.9
VMA @ N_{des}	13.5	13.8	13.3	14.4	13.7	14.7
VFA @ N_{des}	79.2	77.0	85.4	76.4	82.6	73.6
VCA, %	37.3	37.5	37.1	37.9	37.4	38.2
VCA/VCA_{DRC}	0.875	0.879	0.871	0.890	0.878	0.896
Loose Mixture Results						
Asphalt Content, %	6.3	5.7	6.3	6.1	6.1	6.0
Draindown, %	0.02	0.00	0.1	0.0	0.02	0.1
G_{mm}	2.447	2.436	2.432	2.433	2.428	2.433

** Not Determined

Table 4.7 (Continued): Results of Field Testing At SMA Construction Projects						
Site 9						
Marshall Hammer Compaction Results						
	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
G_{mb}	2.453	2.446	2.485	2.448	2.445	2.474
VTM, %	3.9	4.0	2.6	4.3	4.1	3.3
VMA, %	17.5	17.5	16.1	17.9	17.8	16.6
VFA, %	77.7	77.1	83.6	76.0	76.8	80.3
VCA, %	40.3	40.3	39.3	40.6	40.5	39.6
VCA/VCA_{DRC}	0.984	0.984	0.960	0.991	0.989	0.968
Superpave Gyratory Compactor Results						
G_{mb} @ N_{des}	2.517	2.503	2.532	2.489	2.512	2.456
% G_{mm} @ N_{ini}	86.2	85.1	88.0	84.1	85.5	84.5
% G_{mm} @ N_{des}	96.6	96.2	99.2	97.3	96.5	96.1
% G_{mm} @ N_{max}	**	**	**	98.5[A]	99.1[A]	**
VTM @ N_{des}	1.4	1.8	0.8	2.7	1.5	3.9
VMA @ N_{des}	15.3	15.5	14.4	16.5	15.5	17.1
VFA @ N_{des}	91.0	88.7	94.7	83.6	90.3	77.1
VCA, %	38.7	38.9	38.1	39.6	38.9	40.0
VCA/VCA_{DRC}	0.946	0.950	0.930	0.966	0.940	0.978
Loose Mixture Results						
Asphalt Content, %	6.3	6.0	5.9	6.6	6.3	6.1
Draindown, %	0.04	0.1	0.1	0.01	0.1	0.00
G_{mm}	2.552	2.548	2.552	2.558	2.550	2.558

[A] One of three replicates compacted to Nmax.
** Not Determined

Table 4.7 (Continued): Results of Field Testing At SMA Construction Projects						
Site 10						
Marshall Hammer Compaction Results						
	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
G_{mb}	2.427	2.486	2.472	2.439	2.512	2.481
VTM, %	6.5	4.0	3.7	4.7	3.1	4.7
VMA, %	21.7	19.7	20.5	21.0	18.6	19.5
VFA, %	70.1	79.7	81.9	77.5	83.1	75.9
VCA, %	45.7	44.3	44.9	45.2	43.6	44.2
VCA/VCA_{DRC}	1.066	1.033	1.046	1.055	1.015	1.030
Superpave Gyratory Compactor Results						
G_{mb} @ N_{des}	2.409	2.504	2.465	2.423	2.551	2.505
% G_{mm} @ N_{ini}	84.4	87.4	87.4	85.6	88.5	86.7
% G_{mm} @ N_{des}	92.8	96.7	96.0	94.6	98.4	96.2
% G_{mm} @ N_{max}	**	**	97.4[A]	**	**	**
VTM @ N_{des}	7.2	3.3	4.0	5.4	1.6	3.8
VMA @ N_{des}	22.3	19.1	20.7	21.6	17.3	18.7
VFA @ N_{des}	67.8	82.8	80.5	75.2	90.7	79.7
VCA, %	46.1	43.9	45.0	45.6	42.7	43.6
VCA/VCA_{DRC}	1.076	1.023	1.050	1.063	0.994	1.017
Loose Mixture Results						
Asphalt Content, %	6.7	6.5	6.9	6.3	6.2	6.1
Draindown, %	1.1	1.1	3.6	0.6	0.5	0.5
G_{mm}	2.596	2.589	2.568	2.560	2.593	2.604

[A] Two of three replicates compacted to Nmax.

** Not Determined

Table 4.7 (Continued): Results of Field Testing At SMA Construction Projects

Site 11						
Marshall Hammer Compaction Results						
	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
G_{mb}	2.344	2.345	2.341	2.351	2.349	2.329
VTM, %	2.8	2.8	3.5	3.0	3.2	4.6
VMA, %	16.6	16.9	16.9	16.7	16.7	17.3
VFA, %	83.0	82.9	79.5	82.2	80.7	73.6
VCA, %	38.9	39.2	39.1	39.0	39.0	39.4
VCA/VCA_{DRC}	0.973	0.979	0.978	0.975	0.975	0.985
Superpave Gyratory Compactor Results						
G_{mb} @ N_{des}	2.379	2.381	2.372	2.390	2.387	2.378
% G_{mm} @ N_{ini}	86.5	86.8	85.5	85.8	85.3	84.6
% G_{mm} @ N_{des}	98.6	98.6	97.8	98.6	98.3	97.5
% G_{mm} @ N_{max}	**	99.7[A]	98.7[A]	**	**	**
VTM @ N_{des}	1.4	1.4	2.2	1.4	1.7	2.5
VMA @ N_{des}	15.4	15.6	15.8	15.4	15.4	15.5
VFA @ N_{des}	91.2	91.2	86.2	91.1	89.2	83.6
VCA, %	38.0	38.2	38.3	38.0	38.0	38.1
VCA/VCA_{DRC}	0.950	0.955	0.958	0.950	0.950	0.953
Loose Mixture Results						
Asphalt Content, %	6.8	7.2	7.0	7.2	7.1	6.9
Draindown, %	0.03	0.00	0.02	0.04	0.00	0.02
G_{mm}	2.412	2.415	2.425	2.423	2.427	2.440

[A] One of three replicates compacted to Nmax.
** Not Determined

Table 4.8: Percent Air Voids of Cores Obtained From Field Projects

Core Number	Field Project Number and Percent Air Voids							
	2	3	4	5	6	9	10	11
1	6.7	6.3	9.6	10.4	6.0	5.3	9.0	4.2
2	9.9	7.2	11.6	10.2	5.4	7.9	9.3	3.8
3	10.7	7.9	10.8	11.5	6.8	5.5	9.9	5.2
4	9.7	6.8	10.2	10.4	4.3	8.7	5.6	5.5
5	10.2	5.4	12.4	10.4	4.9	7.6	7.7	5.3
6	8.7	8.0	7.5	7.6	**	7.5	9.1	4.0
7	7.7	8.6	8.4	10.4	**	8.0	7.8	5.3
8	7.8	8.7	8.9	9.2	**	6.3	9.5	3.7
9	3.6	8.7	9.3	10.4	**	8.5	12.0	2.7
10	5.7	7.4	9.7	9.8	**	5.4	9.4	3.4

** Cores not obtained.

4.1.3.2 Site 2 Field Testing

While at site 2, the contractor was compacting Marshall specimens as part of quality control. Therefore, NCAT representatives obtained the Marshall data and specimens from the contractor.

For site 2, the air voids for both the Marshall and SGC compacted specimens appear to be too high. The two Marshall sample sets had air void contents of 6.5 and 7.0 percent. The SGC specimens had average air void contents of between 5.1 and 13.2 percent. The probable cause of these low air void contents was a combination of low asphalt contents and low filler (percent passing 0.075 mm sieve) contents. The design asphalt content was 7.1 percent. From Table 4.7 it can be seen that the asphalt contents for the six samples ranged from 5.7 to 6.2 percent. (These values were based on asphalt content ignition tests conducted on the loose mixture samples obtained at the project.) The design filler percentage was 8.0 percent which is on the low side to begin with. From Table 4.8, samples 2, 3, and 4 were between 0.6 and 1.9 percent lower than design. The low amounts of filler may also be the cause for the high draindown values (0.8 to 1.4 percent).

Quality control data was obtained from this project. QC samples obtained by the contractor were taken at the paver. The results of the QC testing indicated that the asphalt contents were not as low as was obtained at the NCAT laboratory; however, the air void contents on Marshall compacted specimens ranged from 6.1 to 7.3 percent. The air void contents of in-place cores obtained as part of the QC program also showed high air void contents. Air void contents of in-place cores obtained while NCAT was on-site ranged from 5.1 to 11.9 percent.

Site 2 did not meet the VCA/VCA_{DRC} ratio maximum of 1.0. Again, similar to site 1, the mix design values did not meet the desired ratio.

Table 4.9: Results of Gradations Performed on Loose Mixture Samples Obtained From SMA Construction Projects

Sieve, mm	Site 1 Percent Passing (Sample No.)[A]						Site 2 Percent Passing (Sample No.)					
	1	2	3	4	5	6	1	2	3	4	5	6
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
12.5	94.0	92.2	96.6	92.8	95.1	93.9	99.9	99.7	99.9	99.8	99.8	99.7
9.5	66.5	62.4	70.0	66.3	69.5	68.7	86.5	86.5	84.5	85.2	83.0	88.0
4.75	30.7	24.8	30.2	26.3	30.3	27.9	31.2	31.2	27.7	27.6	28.4	32.1
2.36	20.5	16.4	17.9	16.1	18.1	17.0	21.2	19.8	17.4	17.9	19.7	21.4
1.18	17.5	13.8	14.8	13.6	15.0	14.2	17.2	15.7	13.3	14.2	16.1	17.4
0.60	16.1	12.6	13.3	12.4	13.7	13.0	14.9	13.3	11.0	11.9	14.1	15.0
0.30	15.4	11.9	12.6	11.9	13.0	12.3	13.0	11.5	9.2	10.2	12.5	13.2
0.150	14.4	11.1	11.8	11.2	12.2	11.6	11.0	9.5	7.9	8.4	10.8	11.0
0.075	11.9	9.0	8.9	9.2	9.9	9.2	8.7	7.4	6.1	6.6	8.8	8.7
	Site 3 Percent Passing						Site 4 Percent Passing					
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
12.5	88.7	84.1	85.6	86.1	85.7	84.0	95.4	95.2	96.1	95.3	94.1	97.1
9.5	68.0	64.7	64.9	67.5	65.1	66.2	69.2	70.9	71.4	69.3	64.8	76.3
4.75	31.5	28.7	29.4	28.7	28.6	30.0	28.9	29.1	29.2	26.4	23.2	32.1
2.36	19.5	17.9	17.5	17.0	17.2	19.4	20.5	21.3	22.0	17.6	14.6	23.7
1.18	16.5	15.1	14.6	14.0	14.3	16.7	18.8	19.9	20.5	16.9	14.1	22.3
0.60	14.9	13.6	13.1	12.4	12.7	15.2	18.2	19.3	20.0	16.7	13.9	21.7
0.30	13.7	12.4	12.0	11.2	11.6	14.2	17.3	18.5	19.1	15.3	13.6	20.9
0.150	12.8	11.3	10.8	9.9	10.5	13.0	14.8	15.7	16.1	12.1	12.5	17.5
0.075	11.2	9.8	9.4	8.4	9.0	11.2	10.3	10.8	11.2	9.4	9.1	11.5

[A] Site 1 used two job-mix-formula during sampling. Samples 1, 2, and 3 were sampled from one JMF while samples 4, 5, and 6 were sampled from another JMF.

Table 4.9 (continued): Results of Gradations Performed on Loose Mixture Samples Obtained From SMA Construction Projects

Sieve, mm	Site 5 Percent Passing (Sample No.)						Site 6 Percent Passing (Sample No.)					
	1	2	3	4	5	6	1	2	3	4	5	6
25.0	100.0	100.0	100.0	100.0	100.0	100.0	A	100.0	A	100.0	100.0	100.0
19.0	100.0	100.0	100.0	100.0	100.0	100.0		100.0		100.0	100.0	100.0
12.5	96.0	97.0	97.1	95.9	97.0	96.3		92.4		90.8	90.7	92.0
9.5	79.0	77.4	83.0	80.8	80.7	81.2		74.1		70.0	67.5	70.5
4.75	29.9	28.1	32.4	31.1	32.7	33.5		31.0		30.0	29.0	30.6
2.36	16.8	16.4	17.8	17.0	17.3	17.7		19.6		19.6	18.8	20.2
1.18	14.4	14.2	15.3	14.7	14.8	15.0		17.8		17.7	17.0	18.2
0.60	13.2	13.0	14.1	13.5	13.7	13.8		173.2		17.1	16.4	17.6
0.30	12.0	11.9	12.9	12.4	12.5	12.6		16.8		16.7	16.0	17.2
0.150	10.6	10.4	11.3	10.9	11.0	11.1		16.4		16.2	15.6	16.6
0.075	8.2	8.2	9.0	8.6	8.7	8.7		13.4		13.1	12.0	12.8
	Site 7 Percent Passing						Site 8 Percent Passing					
25.0	100.0	100.0	**	100.0	100.0	**	100.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	99.9		99.9	99.9		100.0	100.0	99.9	100.0	100.0	99.7
12.5	90.3	88.6		89.9	85.6		92.4	91.2	92.1	93.2	93.4	90.7
9.5	71.5	67.7		68.7	62.0		68.6	65.3	66.6	69.9	69.8	65.1
4.75	34.0	30.4		32.3	27.5		29.8	26.9	30.2	29.36	30.0	27.2
2.36	24.8	21.7		22.5	19.5		21.8	19.3	22.5	21.0	21.5	20.0
1.18	20.0	18.4		18.4	15.9		18.2	16.3	18.8	17.7	18.4	16.8
0.60	17.0	16.4		16.2	14.4		16.3	14.6	16.8	16.0	16.9	15.2
0.30	15.2	15.3		14.9	13.5		15.2	13.6	15.7	15.0	16.0	14.3
0.150	14.0	14.2		13.8	12.7		14.1	12.7	14.6	14.1	15.1	13.5
0.075	12.1	12.4		11.9	11.2		12.0	10.5	12.0	11.8	12.7	11.6

** Not Determined

Table 4.9 (continued): Results of Gradations Performed on Loose Mixture Samples Obtained From SMA Construction Projects

Sieve, mm	Site 9 Percent Passing (Sample No.)						Site 10 Percent Passing (Sample No.)					
	1	2	3	4	5	6	1	2	3	4	5	6
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0	100.0	100.0	100.0	99.4	100.0	99.8	99.0	99.2	99.1
12.5	99.5	100.0	99.5	99.3	98.9	100.0	88.3	88.6	89.1	87.9	88.8	87.5
9.5	70.4	72.1	70.4	74.4	72.7	77.8	72.8	75.9	74.3	73.7	72.9	72.9
4.75	30.5	28.0	30.5	27.5	26.1	32.1	27.0	30.8	27.8	25.7	31.8	30.4
2.36	19.7	17.0	19.7	17.0	16.6	21.0	15.1	19.0	16.7	15.6	20.5	18.7
1.18	**	**	**	**	**	**	13.2	17.2	14.9	13.9	18.4	16.6
0.60	**	**	**	**	**	**	11.2	15.1	13.0	12.1	16.2	14.4
0.30	13.7	12.0	13.7	11.8	11.9	14.2	9.1	12.9	10.8	10.1	13.9	12.1
0.150	**	**	**	**	**	**	7.6	11.6	9.6	8.8	12.6	10.9
0.075	8.9	7.7	8.9	7.7	7.7	9.2	6.6	9.9	8.2	7.6	11.1	9.5
	Site 11 Percent Passing											
25.0	100.0	100.0	100.0	100.0	100.0	100.0						
19.0	99.1	99.9	98.8	100.0	98.6	99.2						
12.5	91.0	94.1	90.0	87.8	88.0	90.0						
9.5	60.7	63.1	59.0	54.6	54.6	59.4						
4.75	28.7	29.3	28.7	26.5	25.1	28.2						
2.36	21.0	20.2	20.9	19.8	18.2	20.4						
1.18	17.4	16.2	17.0	16.2	14.7	16.3						
0.60	15.6	14.2	15.1	14.4	13.0	14.3						
0.30	14.3	12.6	13.7	12.4	11.7	12.8						
0.150	12.2	10.6	11.6	10.3	9.8	10.8						
0.075	10.4	9.0	9.8	8.5	8.1	9.1						

** Not Determined

4.1.3.3 Site 3 Field Testing

Again, the contractor was compacting Marshall specimens as part of their QC program. Information for Marshall compacted specimens in Table 4.7 for site 3 were provided by the contractor. The data for site 3 show that the VCA/VCA_{DRC} ratios were below the recommended 1.0 minimum (except sample 4 of the Marshall compacted specimens). However, there were some air void contents that were high. For the Marshall compacted specimens, samples 3 and 4 had air void contents of 8.7 and 9.1 percent, respectively. Air void contents for the SGC compacted specimens ranged from 3.5 to 7.3 percent. Again, the cause of the higher air void contents was probably low filler contents. Samples 4 and 5 had the highest air void contents at 7.3 and 6.3 percent, respectively. Table 4.8 shows that these two samples had the lowest filler contents at 8.4 and 9.0 percent, respectively (design was 9.7 percent).

Also, all samples for site 3 met the minimum VMA requirement of 17.0 percent. All of the samples also met the draindown requirements.

4.1.3.4 Site 4 Field Testing

Site 4 data shows that none of the samples met the VCA/VCA_{DRC} requirement of 1.0 minimum. The contractor had trouble metering in the proper amount of mineral filler. Samples 4 and 5 were both obtained the time that the contractor was having problems metering the filler. The draindown values were extremely high for these two samples (2.7 and 2.8 percent, respectively). Table 4.8 shows that the percentage of filler was almost 1.0 percent low for these two samples (design filler content was 10.0 percent). This also explains why the VMA increased for samples 4 and 5. The air void contents for the Marshall compacted samples for the other four samples were all very close to the design air void content (4.1 percent).

Also of interest for site 4 was the differences in the volumetric properties between the Marshall hammer and SGC compacted specimens. The air void contents for the SGC compacted specimens were on average 2 percent lower, while the VMA values were on average 1.2 percent lower.

4.1.3.5 Site 5 Field Testing

Referring back to Table 4.1, site 5 was the only project visited in which the SMA mixture was designed using the SGC. The site 5 data shows a similar trend as that of the site 4 data in that the SGC seemed to be providing more compaction than did the Marshall hammer. Collectively, the air voids, VMA, and VCA/VCA_{DRC} were all lower for the SGC compacted specimens.

Similar to sites 1, 2, and 4, site 5 did not meet the VCA/VCA_{DRC} requirements. The mixture design values also did not meet the VCA/VCA_{DRC} requirement.

The reason for the higher air void contents and higher VMA values for samples 1 and 2 appears to be that these two samples had the lowest percent passing the 4.75 mm sieve and the lowest filler content (both had 8.2 percent passing the 0.075 mm sieve).

Site 5 had very good draindown values. This contractor only used fibers to stabilize the mixture. No polymer additive was used.

4.1.3.6 Site 6 Field Testing

The site 6 data did not generally follow the trend of more compaction with the SGC. Four of the six samples compacted with the Marshall hammer had lower air voids than the companion SGC samples. The VCA/VCA_{DRC} ratios were all below 1.0. Site 6 had very good draindown values even with the additional asphalt cement as all were at or below 0.1 percent. Cellulose was used as the stabilizer.

Air void contents for site 6 ranged from 3.5 to 6.2 percent for the Marshall compacted samples and 3.7 to 7.6 for the SGC compacted samples. The VMA values were all above the recommended minimum of 17.0 percent as they ranged from 19.4 to 20.5 for the Marshall compacted samples and 19.0 to 21.7 percent for the SGC compacted samples. The coarse aggregates used for site 6 had an L.A. Abrasion loss value of 16 percent. The percentage passing the 0.075 mm sieve was higher than normal with values ranging from 12.0 to 13.4 percent.

4.1.3.7 Site 7 Field Testing

The VCA/VCA_{DRC} ratios for site 7 were lower than normal but they do indicate that stone-on-stone contact exists. The Marshall ratios ranged from a high of 0.919 to a low of 0.892, while the SGC ratios ranged from a high of 0.898 to a low of 0.811. However, referring back to Table 4.1, the VCA/VCA_{DRC} ratio for the design mixture was 0.866. Referring to the gradations of the companion loose mixtures in Table 4.8, it appears that the gradations are slightly finer than the JMF provided for this project. The design percent passing the 4.75 mm sieve was 27.5 percent and the resulting loose mixture gradations had percent passing the 4.75 mm sieve values of between 27.5 and 34.0. Also, the design percentage of filler was 8.9 percent and the loose mixture gradations had percent passing the 0.075 mm sieve values of between 11.2 and 12.4 percent.

The high filler contents may also explain the low air void contents. The Marshall compacted specimens had air void contents ranging from 2.1 to 5.0 percent with five of the six being at or below 3.5 percent, while the SGC compacted specimens had air void contents ranging between 1.2 and 3.2 percent.

The high filler contents also likely explain the low VMA and low draindown values. The design VMA was 17.5 percent. None of the samples (Marshall or SGC) had VMA values that met the minimum requirement of 17.0 percent. For the three samples tested, the draindown values were less than 0.05 percent.

4.1.3.8 Site 8 Field Testing

Similar to site 7, site 8 had lower VCA/VCA_{DRC} ratios than normally observed. The Marshall ratios ranged from 0.896 to 0.921 while the SGC compacted samples ranged from 0.875 to 0.896. The mixture design VCA/VCA_{DRC} was 0.913. It appears based on the gradations of the companion loose mixture samples that gradations actually produced were slightly finer than the design gradation. The design percent passing the 4.75 mm sieve was 26.2 while in the field the percent passing the 4.75 mm sieve ranged from 26.9 to 30.2 percent. The design percentage of filler was 9.5 percent while the loose mixture samples had percent passing the 0.075 mm sieve values ranging from 11.8 to 12.7 percent. However, the Marshall compacted samples for site 8 did not have the same low air void contents as did site 7. The SGC compacted samples did however have low air void contents (1.9 to 3.9 percent).

Again, site 8 had low VMA values that was probably caused by the high filler contents. The draindown values were very good for site 8 as none were above 0.1 percent.

4.1.3.9 Site 9 Field Testing

Site 9 was a unique project in that NCAT performed the mix design and the control testing for this project. For the mix design, the proposed SMA mixture design procedure was used (Marshall compaction). The data obtained from the project showed that the air voids of the Marshall compacted samples were similar to the design values. Each was within one percent of the design value of 3.6 percent.

Similar to other projects that were designed to have stone-on-stone contact, the site 9 data showed that the produced mixture also had stone-on-stone contact.

The SGC compacted samples seemed to have a higher density than did the Marshall compacted samples. Collectively, the air void contents, VMA, and VCA/VCA_{DRC} values were all lower for the SGC compacted samples. The draindown values for the six samples were all satisfactory as all were below 0.1 percent.

4.1.3.10 Site 10 Field Testing

Of interest for site 10 was that the draindown tests identified mixtures that had high asphalt contents and/or low filler percentages. Samples 1 through 3 all had high asphalt contents (0.4 to 0.8 percent high) along with high draindown values (1.1 to 3.6 percent). Samples 1 and 4 had filler percentages below the design value and also had high draindown values. Collectively, none of the samples met the draindown requirement of 0.3 percent maximum.

The data did not distinctly show whether the Marshall hammer or SGC provided more compaction. For samples 1, 3, and 4 the Marshall hammer provided more compaction while for the remaining samples, the SGC provided more density. Samples 1 and 5 were the extremes for filler content at 6.6 and 11.1 percent, respectively. At the lower filler content, the SGC and Marshall compacted specimens had a higher air void content than they did at the higher filler content.

Referring back to Table 4.1, site 10 was not designed to have stone-on-stone contact as the VCA/VCA_{DRC} ratio was greater than 1.0. During field production, compacted mixtures also did not achieve stone-on-stone contact with the exception of the sample 5 SGC sample.

4.1.3.11 Site 11 Field Testing

Site 11 was the sixth project that was designed to have stone-on-stone contact. During field production, the compacted samples also exhibited stone-on-stone contact as the VCA/VCA_{DRC} ratios were all below 1.0.

The air void contents for site 11 ranged from 2.8 to 4.6 for the Marshall compacted samples. The SGC compacted samples had air void contents ranging from 1.4 to 2.5 percent. The filler contents were from 0.6 to 2.9 percent higher than design while the asphalt contents were 0.1 to 0.5 percent higher.

The high filler contents probably explain the very low draindown values (0.00 to 0.04 percent). In addition, the high filler contents along with the slightly high asphalt content probably explain the lower than design VMA values.

4.1.4 Marshall Hammer Versus Superpave Gyratory

One of the main goals of Task 8 was to compare the Marshall hammer and the Superpave gyratory compactor for both volumetric properties and aggregate breakdown. The following subsections provide a discussion of these comparisons.

4.1.4.1 Comparison of Volumetric Properties

One of the primary goals of the SMA mixture design procedure developed under this study was to provide a mixture design procedure that could use two separate compaction methods: Marshall hammer or SGC. Therefore, one of the primary goals of the field evaluation was to evaluate a number of gyrations with the SGC that would provide similar volumetrics as 50-blows of the Marshall hammer.

In order to accomplish this goal, the results of the field compacted specimens were used to determine the number of gyrations of the SGC required to yield similar volumetrics as 50-blows of the Marshall hammer. This was done by calculating a density ratio based on the bulk specific gravities (or densities) of the Marshall and SGC field compacted specimens. The Marshall and SGC specimens were split from the same truck sample. For each SGC specimen compacted in the field to 100 gyrations and had a companion Marshall sample, the corrected bulk specific gravity using Superpave protocols was calculated at 50, 60, 70, 80, 90, and 100 revolutions. Sites 2 and 3 were omitted from this analysis as for both of these sites the contractor compacted Marshall specimens from mixture sampled from behind the paver. The SGC samples were compacted by NCAT personnel from mixture sampled at the plant-site. Therefore for these two sites, the Marshall and SGC samples were not obtained from the same truck. For a given sample, the bulk specific gravity (G_{mb}) of the Marshall specimen was used to develop G_{mb} ratios with the corrected G_{mb} values determined for the companion SGC compacted specimens. A plot was then developed using the data for all SGC specimens compacted in the field with a companion Marshall specimen that showed the G_{mb} ratio versus the number of gyrations (Figure 4.1).

Based on Figure 4.1 it appears that approximately 80 gyrations of the SGC produced a similar density as 50 blows of the Marshall hammer. Figure 4.1 also shows that there was much variation in the G_{mb} ratio for the different gyration levels. Part of this variation could be caused by errors inherent in the back calculation of bulk specific gravities when using the SGC. This could especially be a problem with SMA mixtures because of the high number of surface voids on samples.

Another possible source of variation in the data was the difference in Los Angeles Abrasion loss values from the different 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 Los Angeles Abrasion loss values were designated the independent variables. Based on the analysis, 47 percent of the variation in Figure 4.1 could be attributed to the two independent variables. Therefore a chart showing the effects of both gyration levels and Los Angeles Abrasion loss values was developed (Figure 4.2). Instead of showing a regression line for all of the projects evaluated, this figure shows the results for LA Abrasion values of 20, 30, and 40 percent loss with the same gyration levels as Figure 4.1.

Based on Figure 4.2, 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 is recommended that for SMA mixtures utilizing aggregates with LA Abrasion loss values of more than 30 percent, a design number of gyrations of 70 be used. For harder aggregates (loss values below 30), 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

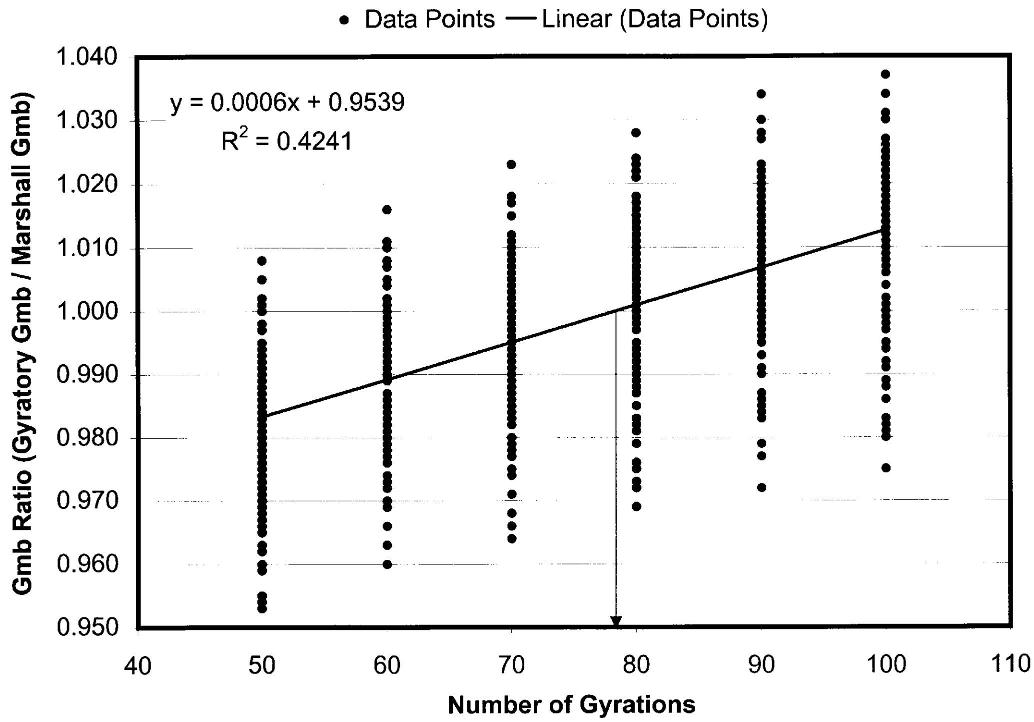


Figure 4.1: SGC/Marshall G_{mb} Ratios for All Data

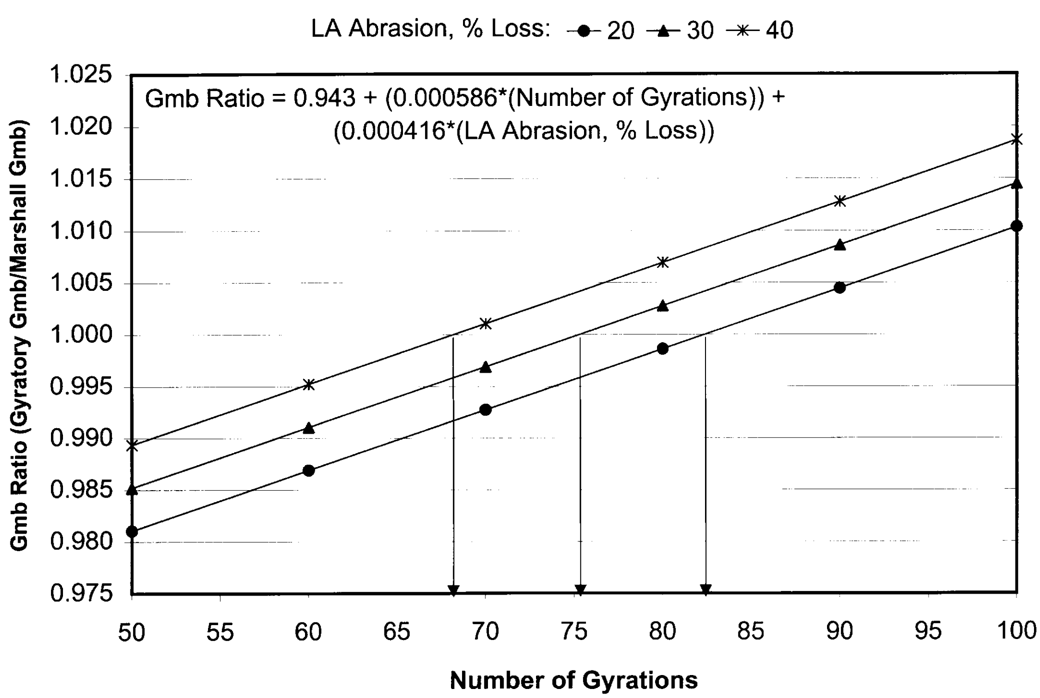


Figure 4.2: G_{mb} Ratio As a Function of Gyration Level and Los Angeles Abrasion

approximately 0.4 percent. The 70 and 100 gyrations were selected to compare with the recommended gyration levels from NCHRP contract 9-9.

4.1.4.2 Comparison of Aggregate Breakdown

An evaluation of aggregate breakdown experienced during both laboratory and field compaction was performed by analyzing the results of gradation tests conducted on the specimens compacted in the field laboratory along with the cores obtained as part of Tasks 13 and 14. All of the compacted specimens (Marshall, SGC, and field cores) from the different projects were brought back to the NCAT laboratory in order to extract the asphalt and determine the gradations of the samples. The gradations after both laboratory and field compaction were then compared to those of the loose mixtures.

Tables 4.10 and 4.11 present the results of the gradation tests performed on the specimens compacted with the SGC and Marshall, respectively. Results of the tests conducted on the field cores obtained from eight projects are presented in Table 4.12. Gradations of the loose mixtures were presented in Table 4.9.

The first analysis was to compare the amount of aggregate breakdown produced by the Marshall hammer and SGC. This was accomplished by comparing the Marshall and SGC compacted specimens to the loose mixture samples from each of the eleven projects visited. This was done by performing an analysis of variance (ANOVA) for each of the eleven projects to determine if significant differences occurred in the percent passing the 4.75 and 0.075 mm sieves between the three types of samples (loose, Marshall, and SGC). Laboratory aggregate breakdown is defined as the gradation of a laboratory compacted sample minus the gradation of the loose mixture. Therefore if significant differences occur between the compacted samples and the loose samples, significant aggregate breakdown occurred.

The actual analysis was performed using the SAS statistical program. Using the ANOVA option, SAS provided the *F*-statistic and the level of significance at which significant differences occur (Probability > F). For instance, if the probability is 0.05, the level of significance at which differences occur would be 95 percent. The actual analysis was accomplished using only the SGC specimens compacted to 100 gyrations.

Results of this analysis (Table 4.13) showed that there existed significant differences between the three types of samples when comparing the 4.75 mm data for all sites except site 1 for a level of significance of 95 percent (α=0.05). However, only two sites showed significant differences in the 0.075 mm data again at a level of significance of 95 percent (sites 9 and 11). These results suggest that both the SGC and Marshall hammer produce significant aggregate breakdown during laboratory compaction on the 4.75 mm sieve; however, in most cases aggregate breakdown does not appear to be significant on the 0.075 mm sieve.

The next step in the analysis was to determine whether the SGC (at 100 gyrations) or the Marshall hammer (50-blows) produced significantly more aggregate breakdown. Again, the data from all eleven projects was used. This was accomplished by again using an ANOVA, except this time the analysis consisted of comparing only the SGC and Marshall compacted specimens. Results of this analysis (Table 4.14) show that the SGC and Marshall hammer produced similar aggregate breakdown for all sites when comparing the 0.075 mm data. Three sites showed significant differences in the percent passing the 4.75 mm data. For each of these three sites, the Marshall hammer compacted specimens had a higher percentage passing the 4.75 mm sieve which indicates the Marshall hammer produced more aggregate breakdown.

Table 4.10: Results of Gradations Performed on SGC Compacted Samples Obtained From SMA Construction Projects

Sieve, mm	Site 1 Percent Passing (Sample No.)[A]						Site 2 Percent Passing (Sample No.)					
	1	2	3	4	5	6	1	2	3	4	5	6
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
12.5	95.3	94.7	96.4	92.5	93.8	93.5	99.9	99.8	97.6	99.8	99.8	100.0
9.5	69.6	64.7	72.3	64.5	68.9	67.0	85.5	86.0	84.4	85.3	86.3	87.6
4.75	34.2	27.7	32.1	27.3	29.0	28.2	33.5	31.9	30.0	30.0	33.0	34.1
2.36	23.0	17.9	19.6	16.9	17.5	17.2	22.5	20.6	18.8	19.6	23.1	23.0
1.18	19.3	14.8	15.8	13.8	13.9	14.0	18.0	16.1	14.2	15.2	18.5	18.2
0.60	17.5	13.1	14.0	12.3	12.7	12.5	15.4	13.5	11.6	12.6	15.9	15.3
0.30	16.5	12.2	13.0	11.6	11.9	11.7	13.4	11.6	9.6	10.6	14.0	13.1
0.150	15.3	11.3	12.1	10.8	11.1	11.1	11.2	9.4	7.9	8.7	11.9	10.9
0.075	12.7	9.1	9.9	8.8	8.8	8.8	8.8	7.3	6.2	6.7	9.5	8.7
	Site 3 Percent Passing						Site 4 Percent Passing					
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	99.8	100.0
12.5	87.9	87.9	88.2	87.9	86.3	86.8	95.9	95.7	96.6	95.7	94.3	97.2
9.5	69.2	69.6	68.9	70.1	69.3	69.8	71.9	72.6	74.1	70.3	65.8	77.9
4.75	35.3	34.3	34.2	38.3	35.1	34.6	34.1	33.9	34.6	29.8	29.5	37.1
2.36	22.3	21.5	21.1	20.1	21.3	22.4	24.3	25.9	25.4	19.9	19.5	26.9
1.18	17.9	17.2	16.7	15.8	16.8	18.0	21.1	23.4	22.7	17.9	17.5	24.3
0.60	15.5	14.8	14.4	13.5	14.4	15.6	19.6	22.1	21.5	17.0	16.8	23.3
0.30	13.9	13.2	12.9	11.9	12.8	14.1	16.3	20.4	19.7	15.8	16.0	21.9
0.150	12.4	11.8	11.5	10.4	11.5	12.7	13.2	16.7	15.8	13.3	13.6	17.8
0.075	10.8	10.1	9.7	8.8	9.8	10.7	10.1	11.7	10.9	9.1	9.4	11.7

[A] Site 1 used two job-mix-formula during sampling. Samples 1, 2, and 3 were sampled from one JMF while samples 4, 5, and 6 were sampled from another JMF.

Table 4.10 (continued): Results of Gradations Performed on SGC Compacted Samples Obtained From SMA Construction Projects

Sieve, mm	Site 5 Percent Passing (Sample No.)[A]						Site 6 Percent Passing (Sample No.)					
	1	2	3	4	5	6	1	2	3	4	5	6
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
12.5	96.6	96.4	96.4	96.2	97.4	97.4	91.3	92.5	90.8	89.7	90.3	90.0
9.5	80.0	80.2	82.5	82.1	83.1	81.6	70.7	73.8	67.4	67.9	69.0	67.6
4.75	35.9	34.7	35.9	36.2	39.4	37.8	32.1	33.5	30.4	30.5	31.0	30.1
2.36	21.3	20.9	21.0	21.0	22.4	21.4	20.5	21.1	19.9	19.8	20.2	19.2
1.18	17.5	17.1	17.2	17.1	18.0	17.6	17.8	18.1	17.3	17.1	17.6	16.4
0.60	15.5	15.1	15.3	15.1	15.8	15.5	16.8	17.1	16.3	16.1	16.6	15.4
0.30	13.7	13.4	13.6	13.3	14.0	13.8	16.1	16.5	15.7	15.6	16.0	14.7
0.150	11.8	11.6	11.6	11.3	11.9	11.8	15.2	16.1	15.0	15.2	15.2	13.6
0.075	9.3	9.1	9.1	8.8	9.2	9.2	12.1	13.1	11.9	12.4	11.4	9.4
	Site 7 Percent Passing						Site 8 Percent Passing					
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0	100.0	100.0	99.8	100.0	100.0	99.8	99.9	100.0	100.0
12.5	91.3	91.1	91.7	88.8	89.0	89.8	92.6	93.8	93.5	93.0	94.9	91.5
9.5	73.7	72.1	74.0	70.6	65.6	70.1	70.1	74.2	69.6	70.4	71.2	69.9
4.75	38.0	36.5	36.4	36.9	31.7	36.9	34.4	33.3	34.8	32.1	33.6	33.1
2.36	27.1	24.9	24.5	25.1	21.9	26.0	24.7	22.9	25.3	22.5	23.5	23.2
1.18	21.4	20.0	19.6	19.8	17.1	20.3	19.8	18.5	20.6	18.2	19.6	18.8
0.60	18.0	17.2	16.8	16.9	14.7	17.0	17.3	16.0	18.1	15.9	17.5	16.4
0.30	15.7	15.6	15.2	15.2	13.4	15.0	15.7	14.6	16.8	14.7	16.4	15.0
0.150	14.3	14.4	14.0	13.8	12.4	13.5	14.4	13.4	15.5	13.6	15.7	14.0
0.075	12.2	12.4	12.1	11.8	10.7	11.4	12.0	11.1	13.3	11.3	13.7	11.9

Table 4.10 (continued): Results of Gradations Performed on SGC Compacted Samples Obtained From SMA Construction Projects

Sieve, mm	Site 9 Percent Passing (Sample No.)						Site 10 Percent Passing (Sample No.)					
	1	2	3	4	5	6	1	2	3	4	5	6
25.0	**	100.0	**	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0		100.0		100.0	100.0	100.0	98.7	99.8	99.4	99.0	99.7	99.2
12.5		99.3		99.4	99.0	99.3	90.7	90.3	90.7	88.1	91.1	88.9
9.5		82.3		77.5	76.9	79.6	75.3	75.8	76.0	73.4	78.2	74.9
4.75		35.5		31.8	35.2	37.2	29.7	32.7	30.6	27.8	35.1	32.2
2.36		23.2		21.1	23.6	24.8	16.9	20.1	18.5	17.1	22.7	20.5
1.18		19.1		17.2	19.5	20.2	14.4	17.8	16.1	14.8	19.9	17.9
0.60		17.4		15.6	17.7	18.2	12.1	15.6	14.0	12.7	17.5	15.5
0.30		16.5		14.7	16.5	16.9	9.8	13.3	11.8	10.5	15.0	13.0
0.150		15.0		13.5	14.7	15.0	8.2	11.9	10.2	9.1	13.3	11.6
0.075		12.2		10.5	11.6	11.8	7.1	10.1	8.7	7.7	11.6	10.0
	Site 11 Percent Passing											
25.0	100.0	100.0	100.0	100.0	100.0	100.0						
19.0	99.4	99.7	98.8	99.1	99.5	99.2						
12.5	91.7	93.9	91.3	86.8	91.9	88.4						
9.5	64.5	65.8	63.8	61.2	64.7	62.1						
4.75	33.4	33.5	32.9	32.0	33.0	32.3						
2.36	23.6	23.4	23.6	23.8	23.3	23.5						
1.18	18.6	18.5	18.5	19.5	18.4	18.7						
0.60	15.8	16.0	15.9	17.1	15.9	16.1						
0.30	13.6	14.2	13.9	15.3	14.0	14.2						
0.150	11.5	12.6	11.9	13.3	12.1	12.3						
0.075	9.5	10.6	10.2	10.9	10.0	10.2						

** Not Determined

Table 4.11: Results of Gradations Performed on Marshall Compacted Samples Obtained From SMA Construction Projects

Sieve, mm	Site 1 Percent Passing (Sample No.)[A]						Site 2 Percent Passing (Sample No.)[C]					
	1	2	3	4	5	6	1	2	3	4	5	6
25.0	100.0	100.0	100.0	100.0	**	100.0	100.0	100.0	**	**	**	**
19.0	100.0	100.0	100.0	100.0		100.0	100.0	100.0				
12.5	94.6	94.4	97.1	91.3		91.8	99.9	99.8				
9.5	70.8	69.6	75.7	68.4		70.9	87.6	85.5				
4.75	33.7	30.1	34.3	29.4		31.6	36.2	34.5				
2.36	21.2	18.5	20.1	17.1		18.0	22.8	22.4				
1.18	16.8	14.4	15.6	13.3		14.0	17.6	17.3				
0.60	15.0	12.6	13.5	11.7		12.2	14.6	14.4				
0.30	14.0	11.7	12.5	10.9		11.3	12.4	12.1				
0.150	13.1	10.9	11.7	10.2		10.6	10.2	9.8				
0.075	10.9	8.9	9.4	8.4		8.5	7.9	7.5				
	Site 3 Percent Passing[B]						Site 4 Percent Passing					
25.0	100.0	100.0	100.0	100.0	**	**	100.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0	100.0			100.0	100.0	100.0	100.0	100.0	100.0
12.5	90.0	90.5	88.0	88.1			96.9	97.0	96.5	96.6	94.4	96.3
9.5	76.5	70.1	69.6	70.4			77.3	76.5	76.5	71.5	69.5	76.5
4.75	35.8	36.5	36.1	36.6			39.6	38.5	37.1	33.0	31.4	37.5
2.36	21.5	22.4	22.8	22.5			28.1	27.2	26.9	21.3	19.6	26.6
1.18	16.7	17.7	18.0	17.7			24.3	23.9	24.0	18.6	17.3	23.7
0.60	14.2	15.3	15.6	15.2			22.5	22.4	22.6	17.4	16.3	22.5
0.30	12.5	13.8	14.1	13.6			20.2	20.3	20.7	16.0	15.1	20.7
0.150	11.1	12.2	12.6	12.1			15.9	16.1	16.8	13.1	12.9	16.6
0.075	9.4	10.4	10.8	10.4			10.5	10.8	11.6	8.8	9.2	10.7

[A] Site 1 used two job-mix-formula during sampling. Samples 1, 2, and 3 were sampled from one JMF while samples 4 and 6 were sampled from another JMF.
[B] Samples compacted by contractor.
** Not Determined

Table 4.11 (continued): Results of Gradations Performed on Marshall Compacted Samples Obtained From SMA Construction Projects

Sieve, mm	Site 5 Percent Passing (Sample No.)						Site 6 Percent Passing (Sample No.)					
	1	2	3	4	5	6	1	2	3	4	5	6
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	D	100.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0		99.9	100.0	100.0	99.7
12.5	96.1	97.1	97.9	96.1	97.0	95.7	92.5		89.8	92.3	90.8	91.4
9.5	80.9	81.1	86.2	83.6	83.0	79.8	77.9		68.3	72.3	74.3	71.8
4.75	37.1	36.7	39.9	37.7	40.2	35.9	36.1		32.0	32.2	33.7	33.5
2.36	21.8	20.9	22.5	21.8	22.5	22.2	22.5		20.8	20.4	21.4	20.9
1.18	17.5	16.7	17.9	17.4	17.9	17.8	19.2		18.0	17.4	18.3	17.7
0.60	15.4	14.6	15.6	15.1	15.6	15.8	18.0		16.9	16.4	17.2	16.5
0.30	13.5	12.7	13.7	13.2	13.7	14.0	17.4		16.4	15.8	16.6	15.9
0.150	11.5	10.7	11.6	11.0	11.8	12.1	16.6		15.9	15.5	16.0	15.2
0.075	8.9	8.3	9.0	8.5	9.1	9.5	14.0		13.3	13.0	12.5	11.5
	Site 7 Percent Passing						Site 8 Percent Passing					
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	99.9	100.0	99.6	100.0	99.7	100.0	100.0	100.0	100.0	100.0	100.0
12.5	91.2	91.1	93.0	92.5	89.7	91.5	95.1	94.6	93.12	94.1	94.3	93.7
9.5	74.4	74.8	75.9	76.8	68.5	73.6	75.2	75.5	73.5	74.8	75.1	72.4
4.75	40.2	39.2	40.3	41.8	33.1	39.5	38.2	36.3	37.1	36.6	37.1	35.1
2.36	27.5	25.6	26.3	27.2	21.4	26.6	25.7	24.4	25.4	24.1	24.5	23.4
1.18	20.9	20.0	20.4	20.9	16.3	20.3	20.2	19.3	20.1	18.8	19.8	18.5
0.60	17.2	17.1	17.3	17.6	13.9	16.8	17.4	16.8	17.4	16.3	17.5	16.1
0.30	15.0	15.6	15.6	16.0	12.6	14.9	15.8	15.4	16.0	14.8	16.3	14.9
0.150	13.5	14.3	14.2	14.6	11.7	13.5	14.6	14.5	15.1	13.7	15.5	13.8
0.075	11.5	12.4	12.2	12.8	10.1	11.7	12.3	12.3	13.1	10.8	13.4	11.8

D - Samples damaged during extrusion.

Table 4.11 (continued): Results of Gradations Performed on Marshall Compacted Samples Obtained From SMA Construction Projects

Sieve, mm	Site 9 Percent Passing (Sample No.)						Site 10 Percent Passing (Sample No.)					
	1	2	3	4	5	6	1	2	3	4	5	6
25.0	100.0	100.0	100.0	**	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0		100.0	100.0	99.2	99.6	100.0	99.4	98.7	99.8
12.5	99.2	99.0	98.7		99.5	99.2	89.0	88.6	93.4	90.2	90.5	89.5
9.5	80.4	79.5	80.3		81.6	81.7	75.9	76.7	79.2	77.9	77.3	78.0
4.75	38.6	37.0	39.3		37.5	39.5	29.0	34.5	34.0	32.4	37.7	36.4
2.36	25.6	23.7	26.6		24.2	25.9	16.9	20.9	19.6	18.9	23.7	21.9
1.18	20.9	18.9	21.7		19.4	20.8	14.0	18.0	16.7	16.1	20.4	18.6
0.60	19.0	17.1	19.8		17.4	18.7	11.7	15.7	14.3	13.8	17.9	16.0
0.30	18.0	16.0	18.7		16.1	17.4	9.3	13.4	11.8	11.7	15.5	13.6
0.150	16.6	14.6	16.9		14.2	15.6	7.8	11.6	10.1	10.2	13.8	12.0
0.075	13.0	11.2	13.2		11.1	12.0	6.6	9.7	8.5	8.6	11.9	10.2
	Site 11 Percent Passing											
25.0	100.0	100.0	100.0	100.0	100.0	100.0						
19.0	98.8	98.9	98.8	100.0	98.9	100.0						
12.5	89.5	92.5	90.7	91.7	91.3	90.0						
9.5	66.7	70.0	66.7	67.9	67.3	63.4						
4.75	36.3	36.2	35.0	36.7	35.1	32.2						
2.36	25.0	24.5	23.9	25.6	24.2	22.4						
1.18	19.6	19.1	18.3	20.2	18.9	17.5						
0.60	17.0	16.6	15.7	17.6	16.3	15.0						
0.30	15.2	14.8	13.9	15.5	14.4	13.3						
0.150	13.3	12.9	12.0	13.4	12.3	11.5						
0.075	11.1	10.6	10.1	11.0	10.1	9.2						

** Not Determined

Table 4.12: Results of Gradations Performed on Cores Obtained From SMA Construction Projects

Sieve Size, mm	Site 2 Percent Passing (Core No.)									
	1	2	3	4	5	6	7	8	9	10
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0	99.8	100.0	100.0	100.0	100.0	100.0	100.0
12.5	99.6	100.0	99.5	99.0	99.9	99.7	100.0	99.4	99.3	99.6
9.5	92.1	88.4	86.1	87.3	88.3	86.4	85.7	88.3	86.3	87.3
4.75	40.4	36.2	36.7	37.3	36.8	32.9	33.1	38.3	34.7	33.8
2.36	26.1	23.9	24.4	25.2	24.9	21.7	22.4	26.7	24.2	22.9
1.18	20.8	19.2	19.6	20.3	19.8	17.1	17.9	21.9	19.6	18.4
0.60	17.6	16.2	16.7	17.3	16.8	14.4	15.2	19.0	16.8	15.7
0.30	15.5	13.8	14.5	14.9	14.4	12.3	12.9	16.6	14.5	13.6
0.150	12.9	11.5	12.1	12.6	12.2	10.7	10.8	14.7	12.2	11.5
0.075	10.6	9.3	9.8	10.3	9.8	8.6	8.6	12.1	9.9	9.3

Table 4.12 (continued): Results of Gradations Performed on Cores Obtained From SMA Construction Projects

Sieve Size, mm	Site 3 Percent Passing (Core No.)									
	1	2	3	4	5	6	7	8	9	10
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	99.7	100.0	99.5	100.0	100.0	100.0	100.0	100.0	100.0	99.5
12.5	87.6	90.9	87.3	92.0	88.5	90.8	92.2	87.6	88.4	89.3
9.5	72.7	77.0	72.4	77.4	74.7	75.0	79.3	73.1	72.7	72.8
4.75	39.0	42.8	41.4	42.0	42.6	39.4	41.9	38.1	37.9	41.1
2.36	23.8	28.2	27.3	26.9	27.8	25.3	26.6	25.0	24.1	27.4
1.18	18.8	22.2	21.7	21.2	22.0	19.8	21.1	19.9	19.9	22.2
0.60	15.8	18.3	18.1	17.7	18.3	16.3	17.7	16.8	17.4	18.8
0.30	13.6	15.4	15.4	15.0	15.5	13.8	15.3	14.4	15.5	16.2
0.150	11.6	13.0	13.0	12.9	13.4	11.7	13.5	12.2	13.8	14.1
0.075	9.5	10.7	10.8	10.7	11.3	9.7	11.5	10.2	11.8	11.7

Table 4.12 (continued): Results of Gradations Performed on Cores Obtained From SMA Construction Projects

Sieve Size, mm	Site 4 Percent Passing (Core No.)									
	1	2	3	4	5	6	7	8	9	10
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	99.9	100.0	100.0	100.0	100.0	100.0	99.8	100.0	100.0	99.7
12.5	96.1	96.6	96.0	96.8	96.9	98.3	96.4	96.9	97.3	94.8
9.5	77.3	75.4	77.0	74.5	79.3	79.9	79.3	78.1	78.8	72.5
4.75	32.9	32.1	33.0	30.3	31.0	32.9	35.8	36.4	34.0	31.2
2.36	22.9	19.3	21.2	19.0	18.4	22.3	25.0	26.4	23.3	21.0
1.18	21.2	17.2	19.2	17.0	16.7	20.4	22.9	24.6	21.6	19.5
0.60	20.6	16.5	18.6	16.3	16.0	19.6	22.2	24.0	21.0	18.8
0.30	19.7	15.4	17.5	15.3	15.1	18.5	21.0	22.9	20.1	17.9
0.150	16.1	12.4	14.7	12.5	12.6	15.4	17.1	19.0	17.2	15.4
0.075	11.4	8.8	10.3	8.5	9.2	10.5	11.8	13.6	12.6	11.2

Table 4.12 (continued): Results of Gradations Performed on Cores Obtained From SMA Construction Projects

Sieve Size, mm	Site 5 Percent Passing (Core No.)									
	1	2	3	4	5	6	7	8	9	10
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0	100.0	100.0	100.0	99.4	100.0	100.0	100.0
12.5	97.6	92.7	98.7	97.0	98.4	99.4	96.3	96.9	96.1	98.6
9.5	86.2	76.2	83.1	84.4	84.4	85.7	81.9	86.6	81.5	83.1
4.75	37.1	35.9	35.0	37.6	38.8	38.7	34.8	37.2	33.5	36.3
2.36	20.5	20.2	20.6	21.5	22.0	21.4	20.1	20.9	20.4	21.1
1.18	17.4	16.6	17.5	17.8	18.7	18.2	17.2	17.7	17.5	17.8
0.60	15.7	14.7	15.9	15.9	16.9	16.5	15.6	16.0	16.0	16.1
0.30	14.2	13.1	14.5	14.3	15.3	15.0	14.1	14.5	14.6	14.5
0.150	12.3	11.2	12.7	12.3	13.4	13.1	12.2	12.7	12.8	12.6
0.075	9.7	8.7	10.1	9.8	10.7	10.6	9.8	10.2	10.4	10.2

Table 4.12 (continued): Results of Gradations Performed on Cores Obtained From SMA Construction Projects

Sieve Size, mm	Site 6 Percent Passing (Core No.)				
	1	2	3	4	5
25.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	99.8	99.9	99.9	100.0
12.5	92.4	93.3	95.3	92.1	93.5
9.5	73.8	73.1	77.4	74.7	74.7
4.75	33.5	33.6	34.2	33.8	34.6
2.36	21.5	21.5	22.2	22.3	22.6
1.18	18.9	18.9	19.5	19.8	19.9
0.60	17.9	17.9	18.4	18.9	19.0
0.30	17.3	17.4	17.8	18.4	18.5
0.150	16.5	16.4	16.8	17.1	17.6
0.075	13.0	12.8	12.8	12.4	13.6

Table 4.12 (continued): Results of Gradations Performed on Cores Obtained From SMA Construction Projects

Sieve Size, mm	Site 10 Percent Passing (Core No.)									
	1	2	3	4	5	6	7	8	9	10
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	99.3	100.0	98.8	98.8	99.1	99.1	100.0	100.0	98.8	98.7
12.5	89.1	93.3	88.9	85.8	84.4	87.5	87.2	88.0	93.7	88.6
9.5	78.2	80.1	77.4	77.6	74.3	75.7	75.8	77.4	84.8	77.1
4.75	35.2	31.8	31.9	40.1	32.0	32.5	32.1	31.6	34.5	32.5
2.36	19.0	17.2	18.5	23.2	19.0	19.1	18.5	17.7	18.8	18.0
1.18	15.5	14.3	15.6	19.2	16.5	16.8	15.7	15.2	15.8	15.3
0.60	12.5	11.7	12.7	16.3	14.1	14.2	13.1	12.6	13.1	12.6
0.30	9.5	9.2	10.0	13.3	11.7	11.8	10.5	10.1	10.5	10.0
0.150	7.8	7.5	8.3	11.4	10.1	10.1	8.9	8.5	8.9	8.4
0.075	6.7	6.3	7.1	9.7	8.8	8.8	7.5	7.3	7.7	7.2

Table 4.12 (continued): Results of Gradations Performed on Cores Obtained From SMA Construction Projects

Sieve Size, mm	Site 11 Percent Passing (Core No.)									
	1	2	3	4	5	6	7	8	9	10
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	99.3	98.6	99.3	99.3	100.0	99.4	98.7	99.0	100.0	98.2
12.5	90.8	93.2	92.1	92.7	91.3	89.6	90.5	92.4	92.0	88.2
9.5	69.0	67.5	67.6	70.1	66.8	65.9	64.9	68.1	67.2	64.0
4.75	35.4	33.2	33.2	32.9	32.6	33.6	33.0	33.6	33.7	32.5
2.36	24.5	23.2	23.0	22.5	22.0	23.8	23.1	24.2	23.8	23.5
1.18	19.4	18.3	18.4	17.7	17.3	18.8	18.2	19.3	18.9	18.7
0.60	16.5	15.6	15.9	15.3	14.9	15.9	15.5	16.2	16.0	15.8
0.30	14.1	13.3	14.0	13.3	13.0	13.2	13.2	13.3	13.4	13.2
0.150	11.4	10.9	11.8	11.1	10.9	10.4	10.7	10.5	10.8	10.5
0.075	9.2	8.7	9.6	9.0	9.0	8.2	8.5	8.2	8.5	8.3

Table 4.13: Results of ANOVA Performed To Compare Aggregate Breakdown on Field Samples

Site No.	Mean Percent Passing 4.75 mm Sieve			*F*-stat	Probability > F	Significant Difference	Mean Percent Passing 0.075 mm Sieve			*F*-stat	Probability > F	Significant Difference
	Loose	SGC	Marshall				Loose	SGC	Marshall			
1	28.4	29.7	31.8	2.55	0.120	No	9.7	9.8	9.2	0.23	0.796	No
2	29.7	32.5	35.4	8.35	0.007	Yes	7.7	8.2	7.7	0.29	0.753	No
3	29.5	35.5	36.3	50.78	0.000	Yes	9.8	10.0	10.3	0.24	0.788	No
4	28.2	33.9	36.2	10.46	0.002	Yes	10.4	10.5	10.3	0.03	0.969	No
5	31.3	37.3	37.9	23.06	0.000	Yes	8.6	9.1	8.9	3.23	0.072	No
6	30.2	31.6	33.5	6.49	0.016	Yes	12.8	12.4	12.9	0.57	0.585	No
7	31.1	35.9	39.0	9.83	0.003	Yes	11.9	11.8	11.8	0.04	0.960	No
8	29.3	33.6	36.7	57.37	0.000	Yes	11.8	12.3	12.3	0.42	0.667	No
9	29.1	34.2	38.4	33.27	0.000	Yes	8.4	11.4	12.1	33.43	0.000	Yes
10	28.9	31.5	34.0	5.01	0.023	Yes	8.8	9.3	9.3	0.13	0.879	No
11	27.8	32.7	35.1	28.94	0.000	Yes	9.2	10.2	10.3	4.43	0.034	Yes

Table 4.14: Results of ANOVA Performed To Compare Aggregate Breakdown Between Marshall Hammer and SGC

Site No.	4.75 mm Sieve			0.075 mm Sieve		
	F-stat	Probability > F	Significant Difference	*F*-stat	Probability> F	Significant Difference
1	1.51	0.258	No	0.31	0.596	No
2	4.89	0.078	No	0.33	0.592	No
3	0.80	0.401	No	0.35	0.574	No
4	2.57	0.144	No	0.00	0.968	No
5	0.29	0.605	No	1.04	0.337	No
6	3.17	0.118	No	0.85	0.388	No
7	3.42	0.098	No	0.00	0.965	No
8	24.20	0.001	Yes	0.00	0.996	No
9	9.20	0.019	Yes	0.95	0.363	No
10	1.94	0.197	No	0.00	0.965	No
11	6.70	0.032	Yes	0.07	0.793	No

Figure 4.3 illustrates average aggregate breakdown values produced by both the Marshall hammer and SGC for each of the eleven projects. Though only three sites showed significant differences in the percent passing the 4.75 mm sieve, this figure shows that for all eleven projects the Marshall hammer yielded higher breakdown values on the 4.75 mm sieve. This figure suggests that 50 blows of the Marshall hammer produces more aggregate breakdown on the 4.75 mm sieve (typically 1 to 3 percent) than does 100 revolutions of the SGC. This was also concluded during Phase I.

The next analysis was to determine which method of compaction produced aggregate breakdown similar to the breakdown exhibited during in-place compaction using rollers. This was accomplished by first using an ANOVA at a significance level of 0.05 to determine if significant differences occurred between either or both of the two laboratory compaction methods and the field cores for each of the eight projects in which cores were obtained (sites 2, 3, 4, 5, 6, 9, 10, and 11).

Results of this analysis (Table 4.15) showed that significant differences occurred on both the 4.75 and 0.075 mm sieves at four of the sites. Sites 2, 3, 9, and 11 all showed significant differences between the percents passing the 4.75 mm sieve while sites 2, 5, 9, and 11 showed significant differences on the 0.075 mm sieve. The differences in aggregate breakdown between the laboratory compacted and field compacted specimens on the 4.75 and 0.075 mm sieves are illustrated in Figures 4.4 and 4.5, respectively.

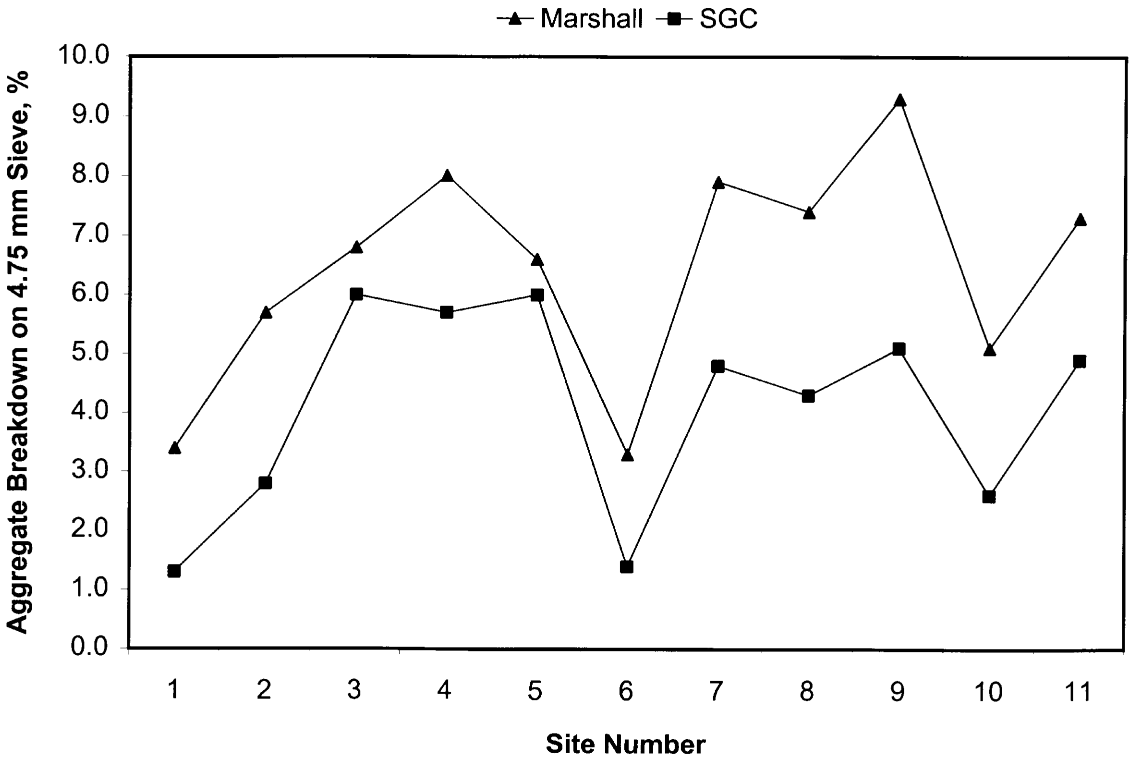


Figure 4.3: Aggregate Breakdown on the 4.75 mm Sieve for Marshall and SGC Compacted Specimens

For each of the sites that showed significant differences between the laboratory and field compacted specimens, a Duncan's Multiple Range Test (DMRT) was used to rank the data. This statistical method was selected because it ranks the different population means to show which are significantly different. Table 4.16 shows the results of this analysis. Based on this table and the fact that only four of the sites showed significant differences in aggregate breakdown between the laboratory compactors and the field, it appears that both 50-blows of the Marshall hammer and 100 gyrations of the SGC produce approximately similar aggregate breakdown when compared to aggregate breakdown in the field.

During Phase I, it was shown that the Los Angeles Abrasion Loss Value was correlated to the amount of aggregate breakdown exhibited in laboratory compacted specimens. Therefore, linear regression analyses were completed for each of the projects for both laboratory compaction methods and the field compaction results. Figures 4.6 through 4.8 present the results of these analyses. Based on these figures, it appears that a relationship exists (though not strong) between the two laboratory compaction methods and the Los Angeles Abrasion loss values; however, a correlation did not exist between field compaction and the Los Angeles Abrasion loss values.

Based on these discussions of volumetrics and aggregate breakdown, it appears that for the Marshall hammer and SGC to provide similar volumetric properties, the design number of gyrations in the mixture design procedure should be set at 70 gyrations for aggregates with high L.A. Abrasion loss values (above 30 percent) and 100 gyrations for aggregates that have loss values below 30 percent.

Table 4.15: Results of ANOVA Performed To Compare Aggregate Breakdown for SGC, Marshall, and In-Place Samples

Site No.	Mean Percent Passing 4.75 mm Sieve			*F*-stat	Probability > F	Significant Difference	Mean Percent Passing 0.075 mm Sieve			*F*-stat	Probability > F	Significant Difference
	SGC	Marshall	Core				SGC	Marshall	Core			
2	32.5	35.4	36.0	4.57	0.030	Yes	8.2	7.7	9.8	6.21	0.012	Yes
3	35.5	36.3	40.6	20.98	0.000	Yes	10.0	10.3	10.8	2.10	0.155	No
4	33.9	36.2	33.0	2.87	0.084	No	10.5	10.3	10.8	0.27	0.765	No
5	37.3	37.9	36.5	1.35	0.286	No	9.1	8.9	10.0	11.70	0.001	Yes
6	31.6	33.5	33.9	3.96	0.051	No	12.4	12.9	12.9	0.83	0.462	No
9	34.2	38.4	33.5	16.19	0.000	Yes	11.4	12.1	10.7	4.37	0.032	Yes
10	31.5	34.0	33.4	1.19	0.328	No	9.3	9.3	7.7	2.87	0.083	No
11	32.7	35.1	33.4	7.97	0.004	Yes	10.2	10.3	8.7	17.63	0.000	Yes

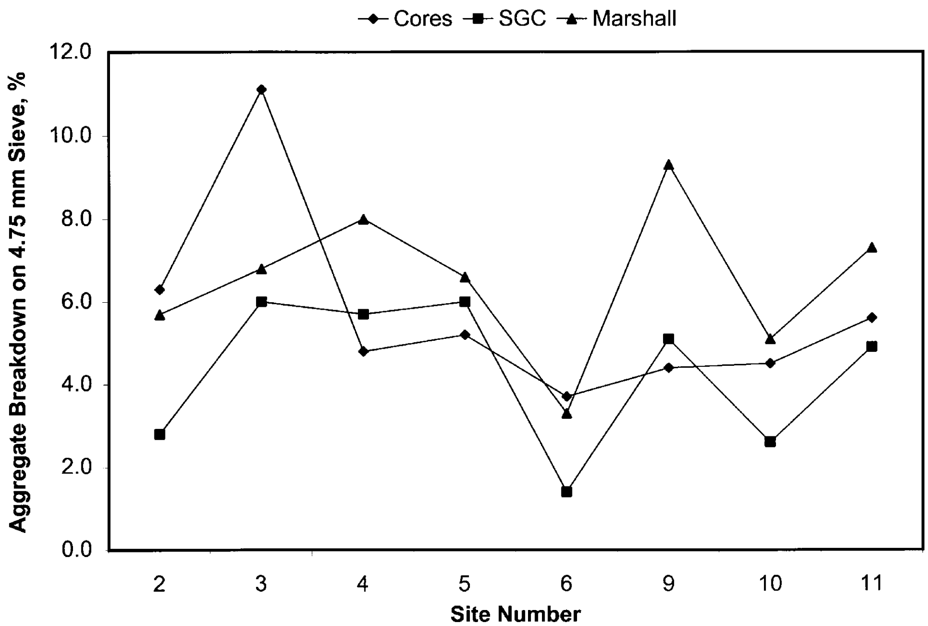


Figure 4.4: Aggregate Breakdown on the 4.75 mm Sieve for Marshall, SGC, and Field Compacted Specimens

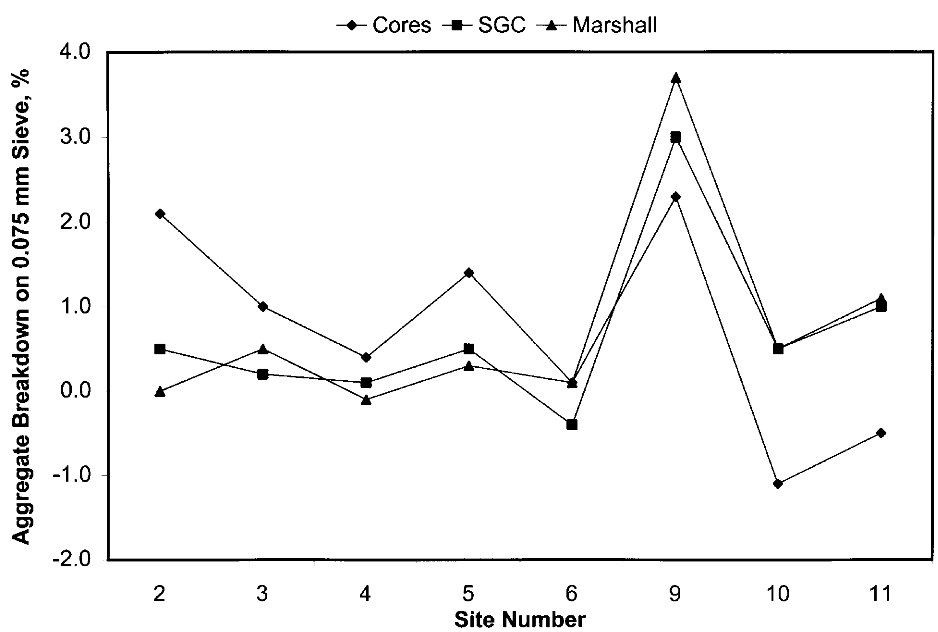


Figure 4.5: Aggregate Breakdown on 0.075 mm Sieve for Marshall, SGC, and Field Compacted Studies

Table 4.16: Results of DMRT Rankings for Sites Showing Significant Differences in Aggregate Breakdown Between Marshall Hammer, SGC, and Cores

Site No.	DMRT Ranking for 4.75 mm Sieve			DMRT Ranking for 0.075 mm Sieve		
	SGC	Marshall	Core	SGC	Marshall	Core
2	B	AB	A	B	B	A
3	B	B	A	Not Significantly Different		
5	Not Significantly Different			B	B	A
9	B	A	B	AB	A	B
11	B	A	B	A	A	B

At 100 gyrations of the SGC the aggregate breakdown is approximately equal to the breakdown produced during field compaction. The Marshall hammer also exhibits similar aggregate breakdown to that produced during field compaction.

4.1.5 Mortar Testing

During Phase I of this project, fine mortars were tested at high, intermediate, and low temperatures using the Superpave binder test equipment. This testing was conducted to evaluate SMA fine mortars and to establish criteria for the mortars using the Superpave binder tests. For testing mortars as part of the mix design process, it was suggested that they be tested at the same high and low temperatures as the PG graded asphalt cements. The tentative recommendation at high temperatures was selected as 5.0 kPa minimum before aging and 11.0 kPa minimum after

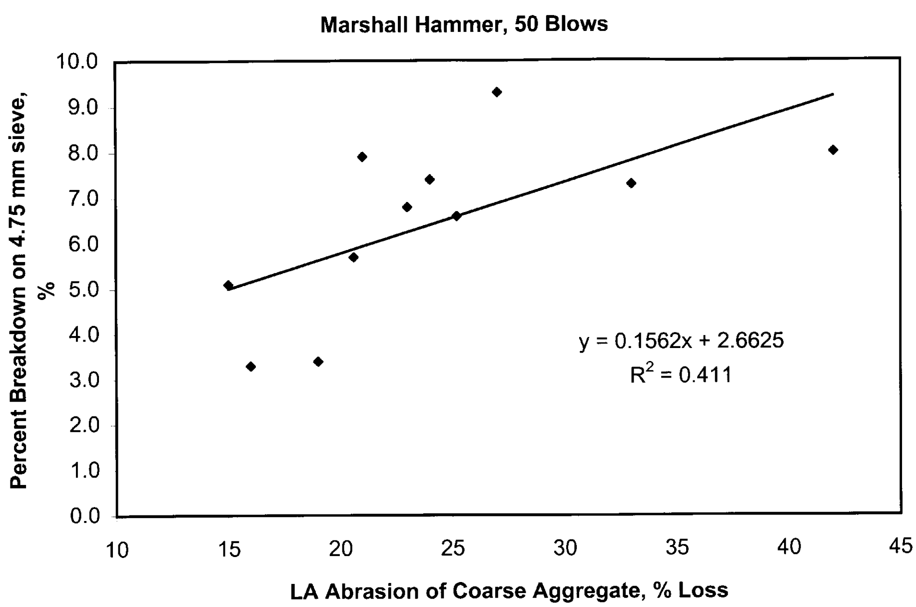


Figure 4.6: Correlation of Los Angeles Abrasion Values and Aggregate Breakdown for 50-Blow Marshall

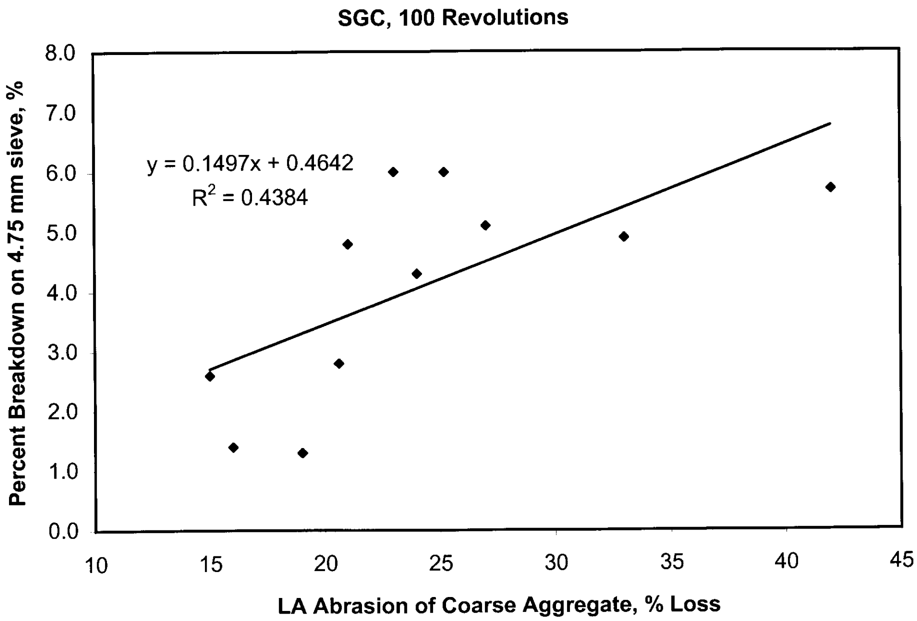


Figure 4.7: Correlation of Los Angeles Abrasion Values and Aggregate Breakdown for 100 Gyrations of SGC

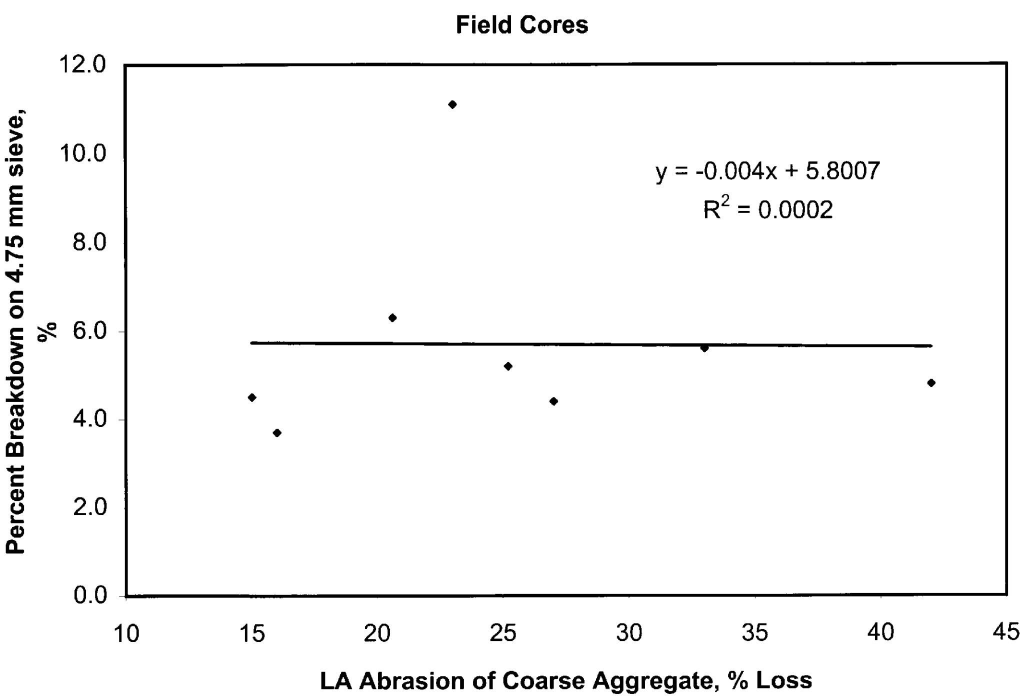


Figure 4.8: Correlation of Los Angeles Abrasion Values and Aggregate Breakdown for Field Compacted Samples

aging. The tentative recommendation at low temperatures was selected to be 1500 MPa maximum. These values were selected from the Phase I work because they set the criteria approximately correct for a number of fillers with known performance history. (Table 4.6 presented the filler properties encountered during Phase II.) During the course of Phase I mortar work, it was found that the addition of fillers and stabilizing additives to an asphalt cement at percentages common to SMA resulted in increases in stiffness of about five times.

Mortar testing for Phase II was conducted to evaluate the suggested mortar requirements developed in Phase I. To do so, fine mortars were created in the laboratory to meet the job-mix-formula percentages from the materials brought back from the eleven projects visited under Task 8. Table 4.17 presents the composition of each of the fine mortars. Testing of these mortars were conducted at high, intermediate, and low temperatures using the Superpave binder test equipment. Actual test temperatures for each site's fine mortars were the high, intermediate, and low temperatures for the respective PG graded asphalts. For instance, if the asphalt cement used was a PG 76-22, testing was conducted at 76, 31, and -12°C (high, intermediate, and low temperatures, respectively).

Table 4.18 presents the results of the mortar testing along with the test temperatures used for each of the eleven fine mortars.

Table 4.17: Composition of Fine Mortars for Each of SMA Construction Projects

SMA Project	Percent of Fine Mortar By Mass				
	Aggregate Filler	Mineral Filler	Asphalt Cement	Stabilizer	Asphalt From Stabilizer**
Site 1[a]	18.8	41.1	37.6	2.5	-
Site 2[a]	23.1	26.2	50.7	-	-
Site 3	2.3	50.3	42.8	2.7	1.8
Site 4[a]	7.0	53.7	36.7	2.6	-
Site 5	-	56.7	41.4	1.9	-
Site 6	-	57.7	38.1	2.5	1.7
Site 7	9.7	45.3	43.0	2.0	-
Site 8	8.3	50.3	39.4	2.0	-
Site 9[a]	21.3	34.7	41.9	2.1	-
Site 10	-	54.0	43.8	2.2	-
Site 11	10.0	40.9	47.0	2.1	-

** The asphalt cement contained in pelletized fibers was included as part of total asphalt.
[a] Aggregate filler included lime used for moisture susceptibility.

4.1.5.1 High Temperature Testing

Figures 4.9 and 4.10 present the results of fine mortar testing at high temperatures for unaged and RTFO aged mortars, respectively. Figure 4.9 shows that only one of the eleven fine mortars did not meet the recommended minimum limit of 5.0 kPa for unaged mortars. From the figure, it can be seen that the fine mortar from site 3 was very high (70.19 kPa). This value is probably not

Table 4.18: Results of Superpave Binder Testing for Mortars Used At SMA Construction Projects

SMA Project	DSR Results					BBR Results (After PAV)			Direct Tension Results*	
	G*/sinδ (kPa)			G*sinδ (kPa)		Test Temp. (°C)	S (MPa)	m (slope)	Failure Strain, %	Peak Stress, mPa
	Test Temp. (°C)	No Aging	After RTFO	Test Temp. (°C)	After PAV					
Site 1*	76	7.61	15.08	31	4452	-12	860	0.242	0.37	6.42
Site 2	76	4.85	8.80	28	2963	-18	695	0.285	1.46	8.24
Site 3	64	70.19	133.31	25	6597	-12	616	0.261	0.39	3.06
Site 4	76	11.93	23.41	31	4746	-12	1140	0.221	0.25	3.85
Site 5	70	5.48	8.60	28	10267	-12	1050	0.263	0.42	7.07
Site 6	64	7.66	52.44	25	8403	-12	473	0.311	0.39	3.55
Site 7	64	8.82	15.26	25	5855	-12	486	0.253	2.39	5.41
Site 8	64	8.85	20.36	22	11179	-18	679	0.269	0.77	7.94
Site 9	76	11.48	22.78	31	5660	-12	667	0.213	0.24	3.32
Site 10	76	8.43	21.69	31	9031	-12	614	0.243	0.55	7.27
Site 11	58	9.84	25.19	19	19397	-18	897	0.220	0.20	2.04

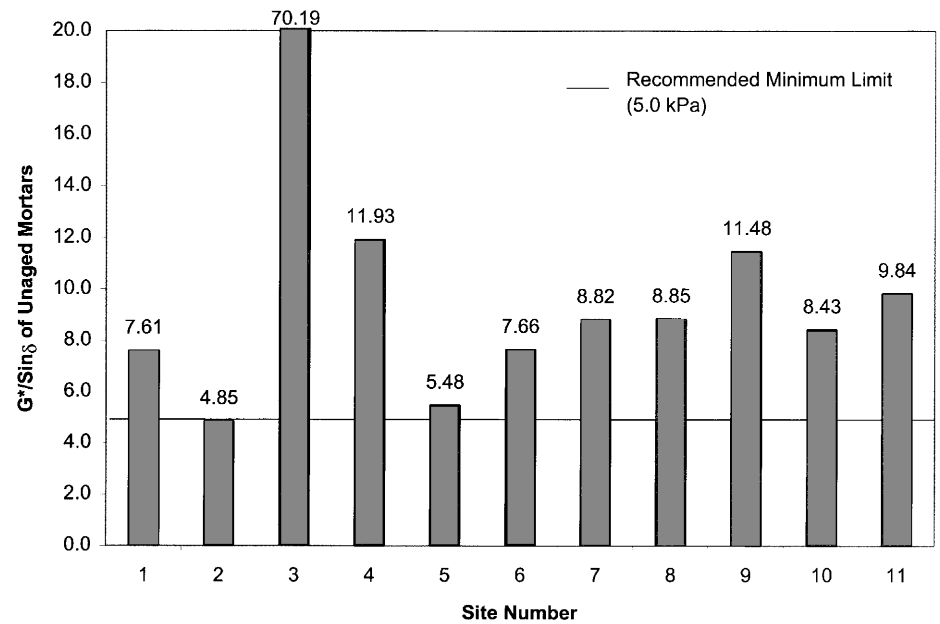


Figure 4.9: Results of Project High Temperature Fine Mortar Testing on Unaged Mortars

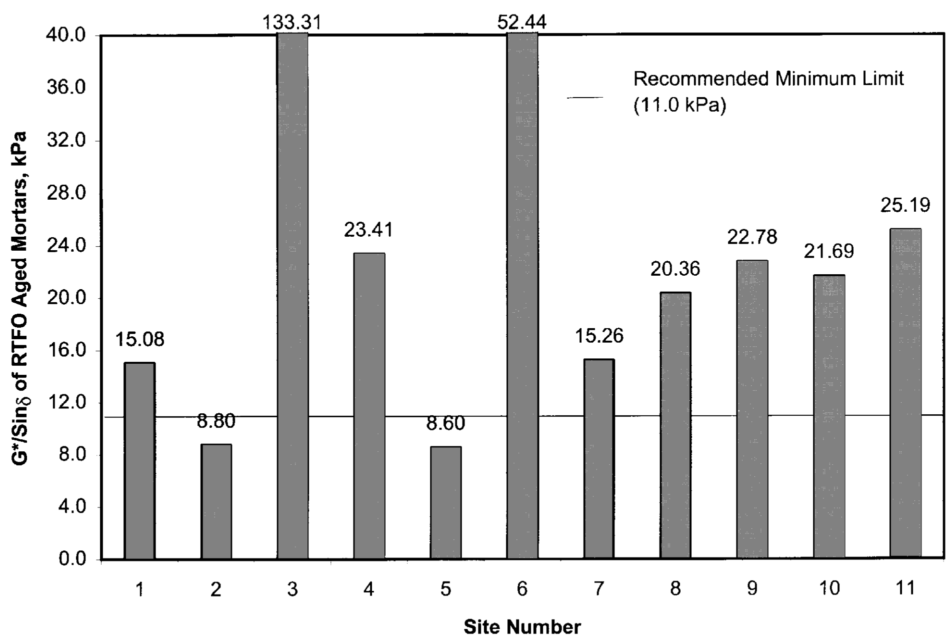


Figure 4.10: Results of Project High Temperature Fine Mortar Testing on RTFO Aged Mortars

representative of the actual mortar as this site was one of the two sites that used asphalt-cellulose pelletized fibers. It was noted during the laboratory preparation of this fine mortar that the pelletized fiber at sites 3 and 6 could not be broken down to make a homogeneous mortar mixture. Interestingly though, site 6 used a similar pelletized fiber which did not seem to affect the test results as the G*/Sinδ value compared favorably to the other mortars.

Figure 4.10 illustrates the results of DSR testing on RTFO aged mortars. Based on this figure, two of the mortars did not meet the recommended minimum limit of 11.0 kPa (sites 2 and 5). Site 2 was the only project that did not use fibers. This site also had the highest percentage of asphalt cement in the mortar (50.7 percent). Site 5 was the other site that did not meet the requirement. This site had a smaller percentage of the filler passing the 0.02 mm size which would result in less stiffening of the mortar. Also based on the figure, it appears that the two sites that used the asphalt-cellulose pelletized fibers had suspect test results. Again, this can probably be attributed to difficulties in preparing the mortars.

4.1.5.2 Intermediate Temperature Testing

Though not recommended during Phase I to be a fine mortar requirement, testing was accomplished at intermediate temperatures using the DSR on RTFO and PAV aged mortars. Test results from DSR testing at intermediate temperatures (Table 4.18) showed that increases in stiffness ranged from 2.8 to 6.5 times over that of the neat asphalt cements with an average increase of 4.5 (results of testing on neat asphalts were presented in Table 4.5). Following the methodology used during Phase I (multiplying by a factor of 5), a recommended maximum limit for G*Sinδ would therefore be 25000 MPa or 5 times the Superpave binder specification for intermediate temperature testing. Figure 4.11 shows the fine mortar test results in relation to the maximum limit of 25000 MPa. Based on this figure, none of the fine mortars would fail to meet the maximum criteria. In fact, only three of the mortars would fail at 2 times the Superpave neat asphalt specification (10000 MPa). Three of the mortars actually meet the Superpave neat asphalt specification of 5000 MPa maximum.

4.1.5.3 Low Temperature Testing

Figure 4.12 shows the results of the BBR stiffness testing on RTFO and PAV aged mortars. This figure shows that none of the eleven mortars exceeded the maximum requirement of 1500 MPa. Based on the data accumulated during Phase II mortar work, the increase in BBR stiffness over that of the neat asphalt cements ranged from 3.29 to 7.26 with an average increase of 4.6 times.

Under Superpave, the slope of the BBR stiffness curve is used to indicate a binder’s ability to relieve tensile stresses at low temperatures. Therefore, it would be expected that the addition of fillers and stabilizing additives would decrease the m-value. This was seen in the test results for the mortars as the m-values all decreased from the neat asphalt test results. The mortar m-values were on average 76.6 percent of the neat asphalt binders. It appears that a reasonable requirement would result in an m-value specification of 0.230 minimum (0.766 * 0.300). Figure 4.13 shows the m-value test results in relation to the minimum limit of 0.230. This figure shows that three of the eleven mortars would fail this criteria.

Additional low temperature testing was conducted using the DTT. Testing with this equipment was not conducted during Phase I. Under Superpave, tensile strain at failure using the DTT is required to be greater than 1.0 mm/mm on neat asphalt cements. Based on the data from Table 4.5, only two of the eleven neat asphalt binders failed to meet this criteria. Based on Tables

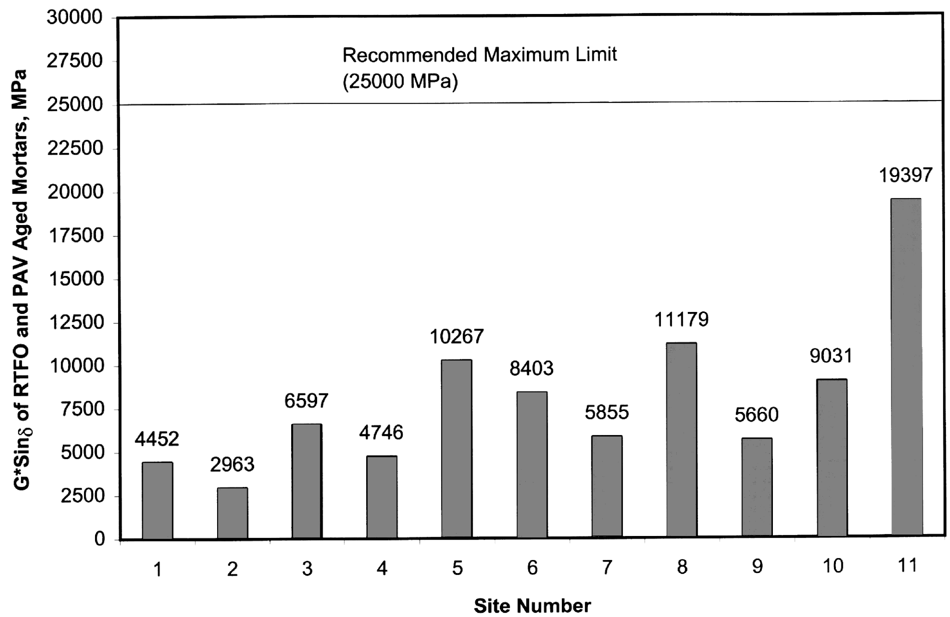


Figure 4.11: Results of Project Intermediate Temperature Fine Mortar Testing With DSR on RTFO and PAV Aged Mortars

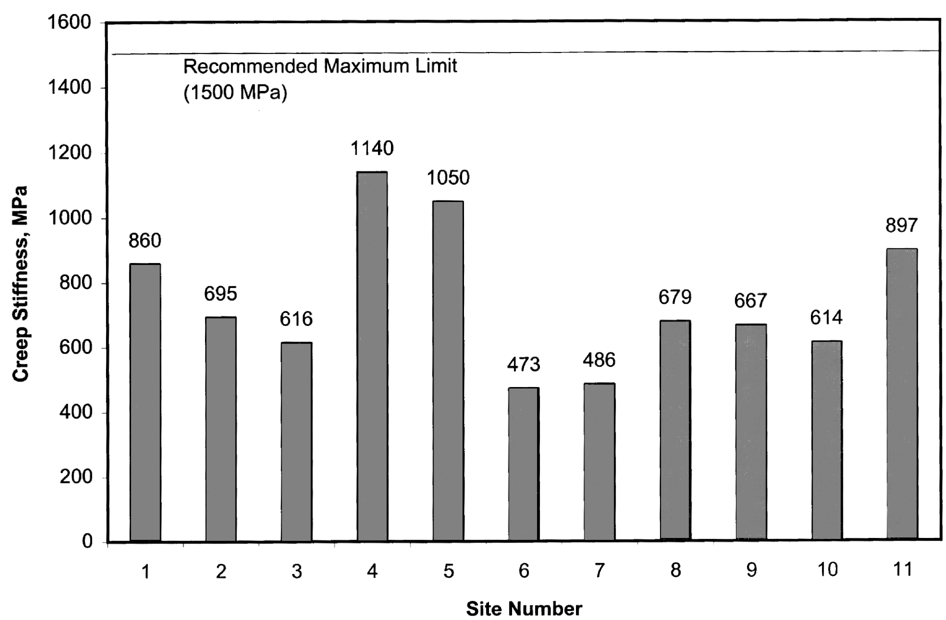


Figure 4.12: Results of Project Low Temperature Fine Mortar Testing With BBR on RTFO and PAV Aged Mortars (Stiffness)

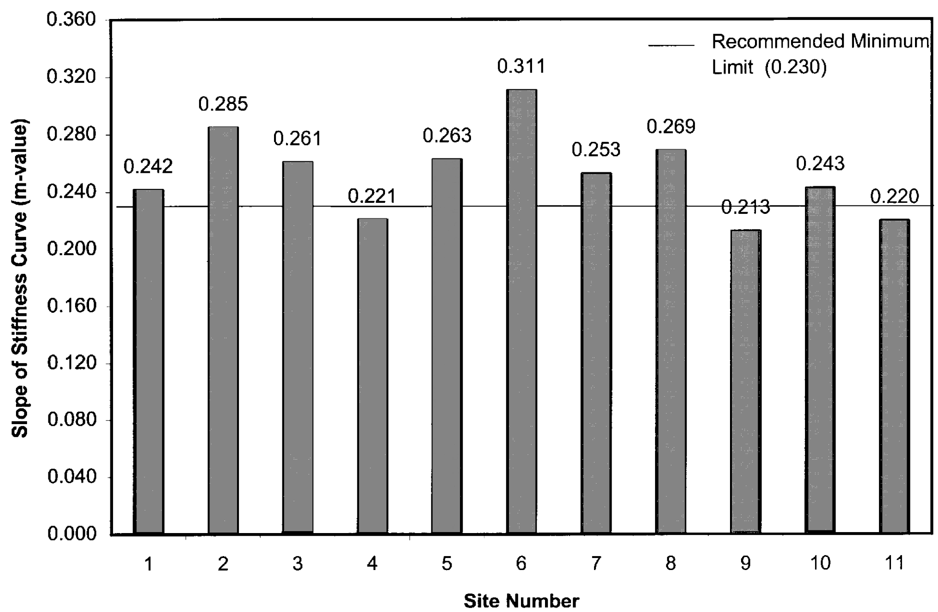


Figure 4.13: Results of Project Low Temperature Fine Mortar Testing with BBR on RTFO and PAV Aged Mortars (m-value)

4.5 and 4.18, the addition of the fillers and stabilizing additives resulted in the expected decrease in percent tensile strain at failure. The DTT testing was performed with the new DTT device but only one replicate was tested for each project before the equipment experienced mechanical problems. Based on the data, no fine mortar criteria is recommended, but the results seem to indicate that criteria may be able to be developed. Additional work is needed.

In summary, it appears that the recommended criteria for SMA fine mortars established during Phase I for high temperature with the DSR and low temperature testing with the BBR (stiffness) are acceptable.

4.1.6 Loaded Wheel Testing

The laboratory loaded wheel test (LWT) device is becoming more widely used to estimate the rutting potential for asphalt mixtures. This testing involved using a laboratory loaded test to apply a specified number of passes at a given temperature and then determining the rut depth. During Phase I of research, the LWT test equipment used was very similar to the Hamburg equipment. For this phase of testing, the Asphalt Pavement Analyzer was used. Table 4.19 presents the results of this LWT performed on mixtures compacted in the field.

Recall from Chapter 3 that the samples used for this testing were compacted while in the field. SMA mixture was compacted to the design number of revolutions (100) for each sample. This explains the high variation in the average air void contents for the different sites as the field sampled mixtures were not necessarily at the design proportions. Also, the testing temperatures used were the same as the high temperature requirements used in selecting the standard PG grade for the sites (64°C for nine sites and 58°C for the remaining two sites).

Table 4.19: Average Results of LWT Testing on Specimens Made at SMA Construction Projects

Cycles	Rut Depths, mm										
	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6	Site 7	Site 8	Site 9	Site 10	Site 11
1000	3.15	2.64	2.67	2.10	1.68	2.60	1.00	0.56	1.08	1.66	2.01
4000	4.04	4.29	3.01	2.37	2.83	4.46	1.71	1.76	1.50	2.42	3.85
8000	4.60	5.15	4.86	3.13	3.53	5.02	2.24	2.38	1.73	3.03	4.81
Avg. VTM, %	5.5	8.6	3.5	5.3	5.0	2.5	2.4	2.7	0.7	5.3	2.2
Test Temp. (C)	64	58	64	64	64	64	64	64	64	64	58

The rut depths at 8000 cycles ranged from a high of 5.15 mm for site 2 to a low of 1.73 mm for site 9. Interestingly, site 2 was the only project visited that did not use fiber stabilizers. Based on experience with the asphalt pavement analyzer, these rut depth values are better than typical values obtained when testing conventional dense-graded mixtures (including Superpave). This information along with the LWT testing accomplished during Task 10 will be used to set guidelines for LWT testing.

4.1.7 Comparison of Field Projects To Phase I Mixture Design Procedure

One of the goals of visiting the different SMA construction projects was to compare the properties of actual SMA mixtures to the recommended requirements of the proposed mixture design procedure. The following sections discuss these comparisons.

4.1.7.1 SMA Gradations

The SMA mixture design procedure developed under Phase I only specified a gradation band for a 19.0 mm nominal maximum aggregate size (NMAS). As discussed earlier, not all of the projects evaluated in this study used 19.0 mm NMAS mixtures. However, all of the SMA mixtures did have 100 percent passing the 19.0 mm sieve (Table 4.20). Most of the mixtures met the majority of the sieves on the specification band. The sieve that had the most projects not meeting the criteria was 4.75 mm. Four projects (JMF-01-2 from site 1 and sites 2, 4, 5) did not meet the suggested requirements. All of these mixtures were above the recommended upper limit of 28 percent passing and ranged from 29 to 33 percent passing. However, each of these projects did meet the minimum VMA requirement of 17 percent but did not meet the VCA requirement. Two of the projects did not meet the requirements for the 0.075 mm sieve. One of these projects was above the upper limit of 10 percent passing and the other was below the lower limit of 8 percent passing.

In summary, a majority of the project gradations met the gradation band requirements. It appears that the requirements of the 19.0 mm gradation band in the SMA mixture design procedure can be met and will produce SMA mixtures that can be placed in the field.

4.1.7.2 Aggregate Quality

As part of the SMA mixture design procedure, requirements were included for both coarse and fine aggregates. Tables 4.21 and 4.22 show that most of the coarse and fine aggregate requirements were met at the different SMA construction projects. These tables also show that some of the specified tests are not commonly performed. For the coarse aggregates (Table 4.21), all of the aggregates met the crushed content requirements for both one face and two or more faces. Of the states reporting aggregate soundness results, all met the suggested requirements. Two projects did not meet the absorption requirements. These sites included projects 3 and 11. Only three of the states reporting test results for flat and elongated particles tested the coarse aggregates at both the 3 to 1 and 5 to 1 ratios. Two of the projects did not meet the 5 to 1 limit of 5 percent maximum with approximately 10 percent at the 5 to 1 ratio. Four projects did not meet the 3 to 1 requirement of 20 percent maximum with values ranging from 20 to 47 percent at the 3 to 1 ratio. All of the projects but two did meet the L.A. Abrasion requirement of 30 percent loss maximum.

Table 4.20: 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 meet.

Table 4.21: 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	**	No	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 4.22: 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.

Table 4.22 shows that there was not as much data collected for the fine aggregates. However, of the three projects reporting fine aggregate soundness, all met the requirements. Nine of the eleven projects met the fine aggregate angularity requirements. All that reported liquid limit and plasticity index values met the requirements.

It appears that the aggregate quality test that most often fails to meet the requirements was the flat and elongated requirements test. All of the other properties were met most of the time. Therefore, the aggregate quality requirements of the SMA mixture design procedure seem to be reasonable but a relative high percentage of projects did not meet the flat and elongated requirements.

4.1.7.3 SMA Mixture Specifications

For each of the projects visited, a copy of the actual mix design data was obtained. These properties were compared to the requirements established in the SMA mixture design procedure. Table 4.23 shows that all but two of the projects had a minimum asphalt content of 6 percent. As noted on the table, two of the projects had asphalt contents below 6 percent but the combined aggregate bulk specific gravities of these mixtures were greater than 2.75; therefore, this minimum asphalt content can be lowered slightly as long as the minimum VMA requirements are met. Three of the projects did not meet the air void content requirement. These three projects had laboratory voids above the 4.0 percent air void level. All mixtures did meet the VMA requirements. As stated previously, six of the mixtures met VCA requirements. Those mixtures that did not meet the VCA requirements (sites 1, 2, 4, 5, and 10) could have been adjusted by reducing the percentage passing the 4.75 mm sieve.

4.1.7.4 Mortar Properties

Table 4.24 presents the comparisons for the fine mortars from each site. Based on this table, only one project did not meet the high temperature requirements for unaged mortars and two did not meet for RTFO aged mortars. All of the projects met the low temperature requirements.

In summary, it appears that the fine mortar requirements are applicable to SMA mortars. The majority of the projects met the criteria.

4.1.8 Results of Questionnaire

In addition to the site projects, a questionnaire was sent to panel members, industry, and state DOTs concerning the mix design procedure developed under Phase I. Information obtained from the questionnaire was then used along with test results from the study to revise the SMA mixture design procedure. The questionnaire was provided in Table 3.1.

Sixteen recipients replied to the questionnaire. The replies can be summarized as follows:

1. No one has used the proposed SMA mixture design procedure verbatim. Those that have used similar procedures have not included the fine mortar properties and/or voids in coarse aggregate (VCA) requirements.
2. Many of the reviewers indicated that the procedure for selecting the mixing and compaction temperatures (temperature-viscosity relationship) may not be applicable to modified asphalt cements.
3. Because none of the reviewers had used the fine mortar testing procedures, there were not many comments about the recommended criteria. However, several reviewers asked that

Table 4.23: 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	Yes	Yes	Yes	No	Yes	Yes	No	No	Yes	Yes	Yes
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 4.24: Comparison of Fine Mortar Properties From Field Projects To SMA Mixture Specification

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

[1] As discussed in Section 4.1.5 of this report.
[2] The asphalt cement used for JMF-01-1 was not obtained.
[3] Results of this test passed the criteria but the test results were suspect.

a standard method of preparing fine mortars be included within the mix design procedure.

4. Most of the reviewers have not yet used the VCA requirement; however, comments about the VMA requirements were favorable. Some reported that lower VMA requirements have been used with success.
5. The majority of SMA mixtures have been designed with the Marshall hammer (50 blows per face).
6. Several reviewers have designed SMA mixtures with different maximum aggregate sizes than recommended in the Phase I mix design procedure and include: 9.5, 12.5, and 25.0 mm maximum aggregate sizes.
7. Some reviewers indicated that SMA mixtures designed with the larger nominal maximum aggregate sizes may not require the minimum asphalt cement content of 6.0 percent.

4.2 LABORATORY TESTING

Laboratory testing included in Phase II included three main subtasks: evaluation of varying nominal maximum aggregate sizes, evaluation of flat and elongated particles using the SGC, and direct tension testing of mortars from Phase I.

4.2.1 Nominal Maximum Aggregate Size

The objective of this subtask was to determine whether the mixture design procedure developed during Phase I could be used for mixtures of varying nominal maximum aggregate sizes. This was done by isolating the effect of different nominal maximum aggregate sizes by keeping all other variables constant. A traprock aggregate source was exclusively used during this subtask. This was the same material used during much of Phase I testing and was selected due to its high quality. Traprock has proven to be particularly suited for SMA primarily due to its toughness (i.e. low L.A. Abrasion loss value) and resistance to excessive breakdown during mixing and compacting. The properties for both the coarse and fine traprock aggregates are given in Tables 4.25 and 4.26, respectively.

The mineral filler and stabilizing fiber used throughout this subtask were limestone and cellulose, respectively. The properties of each are shown in Tables 4.27 and 4.28. A PG 64-22 binder was also used exclusively.

Eleven SMA mix designs were conducted for various nominal maximum sizes (NMS) and break point (BP) sieve combinations (NMS/BP). The break point sieve is identified as the sieve at which the gap grading begins. These mixes were as follows:

1) 25.0 mm NMS/12.5 mm BP (25/12.5)
2) 25.0 mm NMS/9.5 mm BP (25/9.5)
3) 25.0 mm NMS/4.75 mm BP (25/4.75)
4) 19.0 mm NMS/4.75 mm BP, Phase I gradation (19/4.75-I)[A]
5) 19.0 mm NMS/4.75 mm BP, Phase II gradation (19/4.75-II)[B]
6) 12.5 mm NMS/9.5 mm BP (12.5/9.5)
7) 12.5 mm NMS/4.75 mm BP (12.5/4.75)
8) 9.5 mm NMS/4.75 mm BP (9.5/4.75)

Table 4.25: Properties of Coarse Traprock Aggregate		
Property	Test Method	Value
Bulk Specific Gravity	AASHTO T-85	2.967
Apparent Specific Gravity	AASHTO T-85	3.024
Absorption, %	AASHTO T-85	0.7
Los Angeles Abrasion, % Loss	AASHTO T-96	17
Flat or Elongated Particles	ASTM D-4791	
2 to 1		54
3 to 1		15
5 to 1		1
Soundness, % Loss	AASHTO T-104	1.1
Crushed Content, %		
One Face		100
Two Face		100

Table 4.26: Properties of Fine Traprock Aggregate		
Property	Test Method	Value
Bulk Specific Gravity	AASHTO T-84	2.923
Apparent Specific Gravity	AASHTO T-84	3.004
Absorption, %	AASHTO T-84	1.0
Soundness, % Loss	AASHTO T-104	1.1
Angularity, %	AASHTO TP-33	48.3
Liquid Limit, %	AASHTO T-89	**
Plastic Limit, %	AASHTO T-90	NP

**: Liquid Limit could not be determined.
NP: Non-Plastic

9) 9.5 mm NMS/2.36 mm BP (9.5/2.36)
10) 4.75 mm NMS/2.36 mm BP (4.75/2.36)
11) 4.75 mm NMS/1.18 mm BP (4.75/1.18)

[A] - This is the same gradation used during Phase I with the traprock aggregate. It was included for comparison purposes.
[B] - This gradation was developed during Phase II.

Each of the different NMS/BP mixture designs were performed in accordance with the procedure developed during Phase I. For each NMS/BP combination, a minimum of three trial blends were mixed and compacted at an estimated optimum asphalt content. A Superpave gyratory compactor (set for 100 gyrations) was used to compact the specimens. The design gradation was selected from the trial blends based on VMA and VCA/VCA_{DRC}. Some of the

Table 4.27: Limestone Mineral Filler Properties

Particle Size Analysis	
Size (mm)	Cumulative Percent Passing [1]
2.36	100
1.18	100
0.600	100
0.300	98.22
0.150	92.20
0.075	79.25
0.045	69.82
0.020	57.08
Property	Value
Apparent Specific Gravity [2]	2.883
Dry-Compacted Voids [3] (%)	33.5
Surface Area [4] (m^2/g)	1.50

[1] As determined using a Coulter LS-200 laser particle size analyzer.
[2] As determined by AASHTO T-100.
[3] As determined by Penn DOT method.
[4] BET determined using a Coulter SA-3100 surface area analyzer.

Table 4.28: Cellulose Fiber Properties

Property	Value
Bulk Density (kg/m^3)	28
Average Fiber Length (mm)	1.1
Average Fiber Thickness (mm)	0.045
Surface Area [1] (m^2/g)	1.14

[1] BET determined using a Coulter SA-3100 surface area analyzer.

design gradations had to be selected solely on the basis of VCA/VCA_{DRC} as some of the NMS/BP combinations had extremely high VMA values. Once each design gradation was selected, the asphalt content was varied in order to determine the optimum asphalt content for the mixture. Optimum asphalt content was selected on the basis of percent voids in total mix (VTM). The asphalt content that produced 3.75 percent VTM was selected as the optimum mixture.

In addition to the eleven SMA mixture designs, four Superpave mix designs were developed for two different nominal maximum sizes for comparison purposes. These mixes included two 9.5 mm mixes, one with a gradation passing above the restricted zone and one passing below. These will be referred to hereafter as 9.5ARZ and 9.5BRZ. The other two mixes had a nominal maximum size of 25 mm. Again, one gradation passed above the restricted zone while one passed below the restricted zone and will be referred to as 25ARZ and 25BRZ. These Superpave mixes represent two common nominal maximum sizes and were selected since they are toward the higher and lower ends of the Superpave NMS range. These designs were done in accordance with Superpave protocol using a design number of gyrations of 96.

Wheel tracking tests were conducted on all of the optimum mixtures. The mixture properties were held constant and test specimens were compacted to design air voids. An Asphalt Pavement Analyzer (APA) was used to perform the tests. Testing was accomplished at both 55°C and 64°C. The lower temperature was selected since it is currently the one used by many agencies for high performance mixes. Testing at the high temperature (64°C) was done since this is a common high temperature for many states. The specimens were tested in the dry condition and average rut depths in millimeters were obtained after the application of 8000 cycles of a one hundred pound load.

Permeability tests were also conducted on all the optimum mixtures. Specimens at design gradation and asphalt content were prepared at different VTM levels for each mixture. This was accomplished by varying the number of gyrations with the SGC. Specifically, gyration levels of 10, 30, and 50 were used to produce a range of air voids. A falling head permeameter was employed to determine the coefficient of permeability in centimeters per second. Test specimens were subjected to a vacuum in accordance with AASHTO T-209 prior to testing to achieve saturation. The permeability apparatus and test procedure are shown in Volume V.

4.2.1.1 Gradations

The optimum mixture gradations determined for each NMS/BP combination are shown in Table 4.29. These gradations are illustrated on an FHWA 0.45 power chart in Figures 4.14 through 4.18. These figures show that the general trend of each gradation was identical except for the case of the 19.0 mm mixes. The 19/4.75-II gradation (Phase II gradation) was designed similar to the other Phase II gradations in that the gradation line begins at the maximum aggregate size and is straight to the break point sieve (4.75 mm). The 19/4.75-I gradation (Phase I gradation) did have a break in the gradation line at the 12.5 mm sieve.

Table 4.30 shows the optimum mixture gradations for the Superpave mixes. These gradations are shown graphically in Figures 4.19 and 4.20. Note that the respective SMA gradations have been included on these plots for reference. Also note that the BRZ mixes have gradation curves approaching that of an SMA with the exception of a significantly lower dust content (3.5 percent passing the No. 200 sieve instead of 10 percent passing).

4.2.1.2 SMA Mix Designs

The optimum mix properties for the SMA mixes are given by Table 4.31. The percent passing the break point sieve for each NMS ranged from 21 to 32 percent while optimum asphalt contents ranged from 5.5 to 8.3 percent. Design air voids (VTM) ranged from 3.5 to 3.8 percent. VMA and VCA values for each mix are shown in Figures 4.21 and 4.22, respectively. Complete mixture design results are shown in the Appendix, Volume V.

Seven of the eleven SMA mixes were successfully designed without any modification of the Phase I design procedure and included: 9.5/2.36, 12.5/4.75, 19/4.75-I,19/4.75-II, 25/12.5, 25/9.5 and 25/4.75. Thus, it appears that the mix design procedure will work for different nominal maximum aggregate sizes. Five of the NMS/BP combinations, however, presented design problems that required altering the design procedure. Even with the modification, in some

Table 4.29: SMA Design Gradations for Each NMS/BP Combination

Sieve Size (mm)	Cumulative Percent Passing										
	Nominal Maximum Size (NMS)/Break Point (BP) Sieve Combination (mm/mm)										
	4.75/ 2.36	4.75/ 1.18	9.5/ 4.75	9.5/ 2.36	12.5/ 9.5	12.5/ 4.75	19.0/ 4.75-I[A]	19.0/ 4.75-II[B]	25.0/ 12.5	25.0/ 9.5	25.0/ 4.75
25.0									100	100	100
19.0							100	100	68	75.3	82.5
12.5					100	100	90	72.5	28	44.5	60.5
9.5			100	100	26	75	54	56.5	26	26	47
4.75	100	100	30	55.5	21.5	27	24	24	21.5	21.8	21
2.36	31	60	24	23	18.5	22	20	20	18	18.5	18
1.18	24	32	19.5	19	16	18	18	17	16	16	16
0.6	19	24	16.5	16	14.8	16	14	15	14.5	14.5	14
0.3	15	17	14.3	14	13.8	13.5	13	13.5	13	13.3	13
0.15	12	12	12	12	12	12	11	12	12	12	12
0.075	10	10	10	10	10	10	10	10	10	10	10

[A] Design gradation used in Phase I.
[B] Design gradation developed in Phase II.

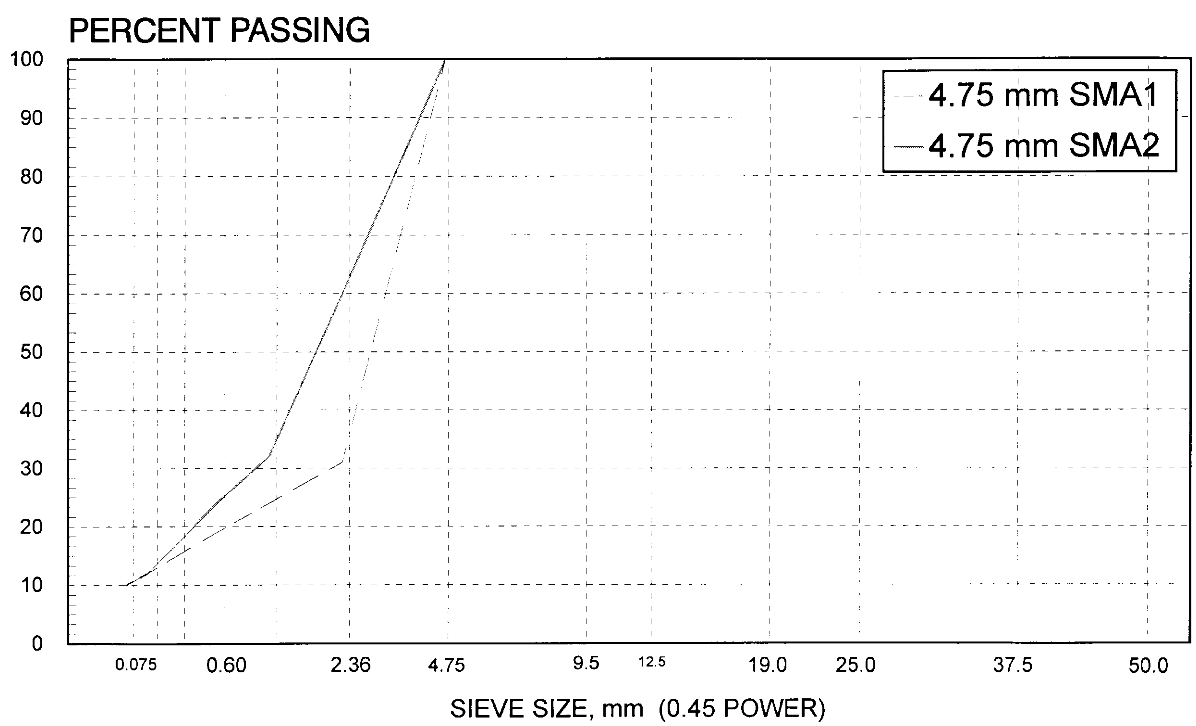


Figure 4.14: 4.75 mm NMS SMA Optimum Mixture Gradations

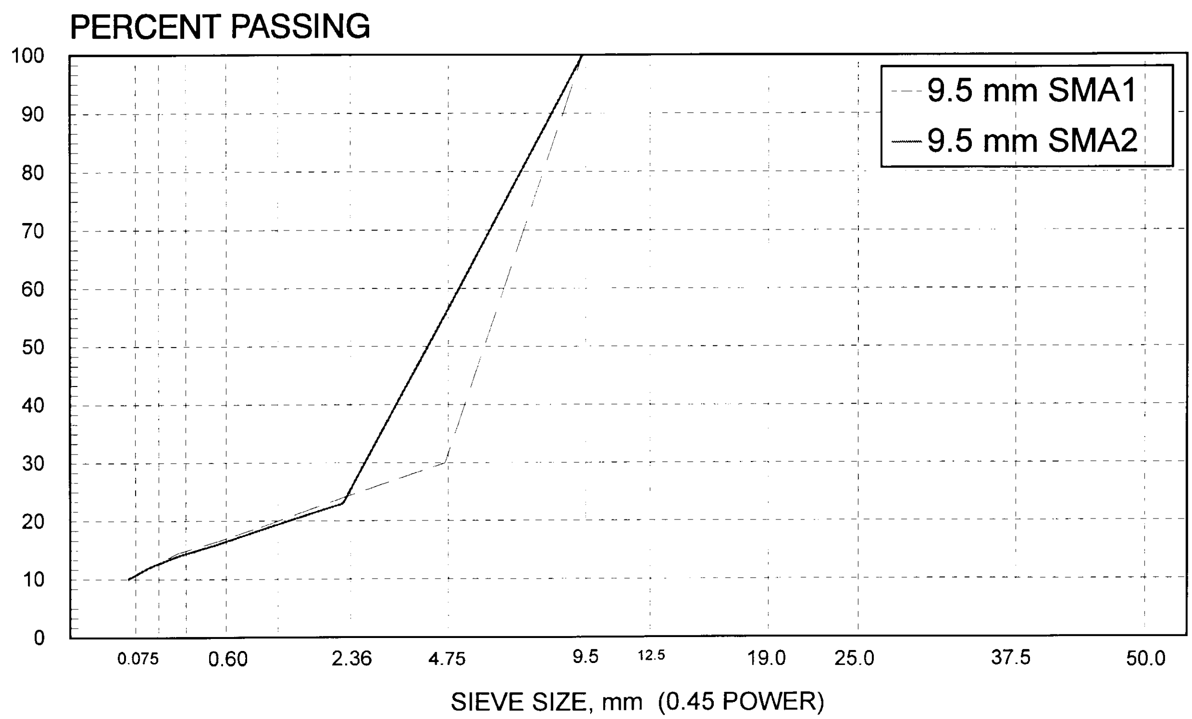


Figure 4.15: 9.5 mm NMS SMA Optimum Mixture Gradations

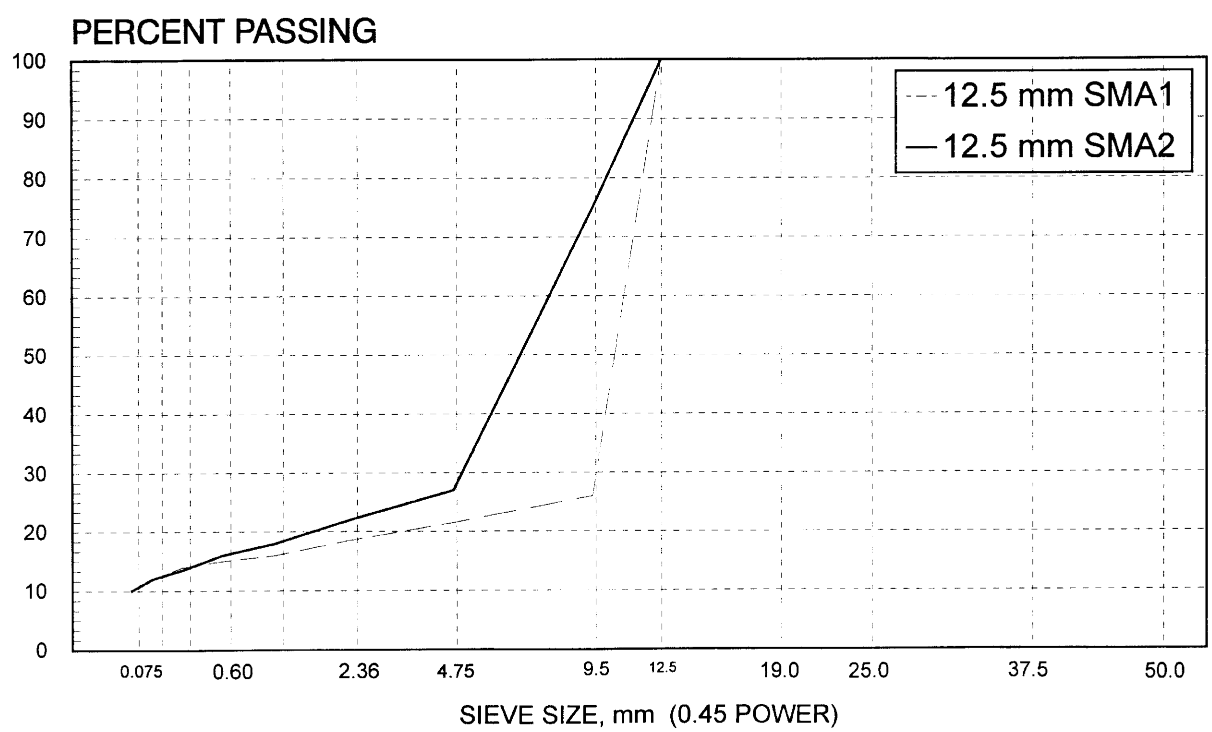


Figure 4.16: 12.5 mm NMS SMA Optimum Mixture Gradations

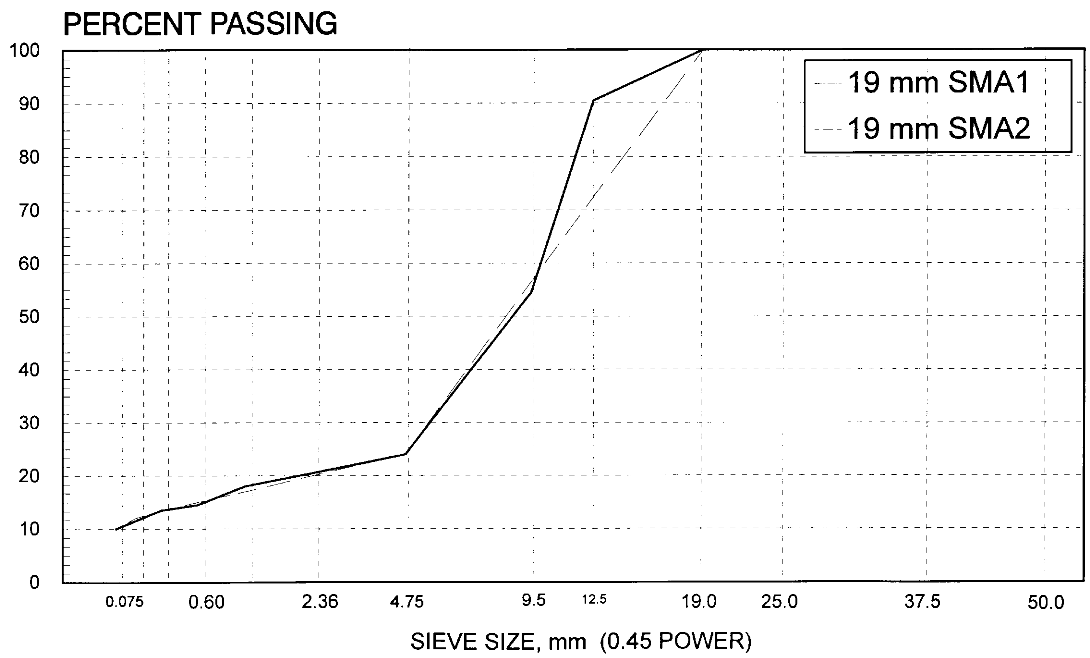


Figure 4.17: 19.0 mm NMS SMA Optimum Mixture Gradations

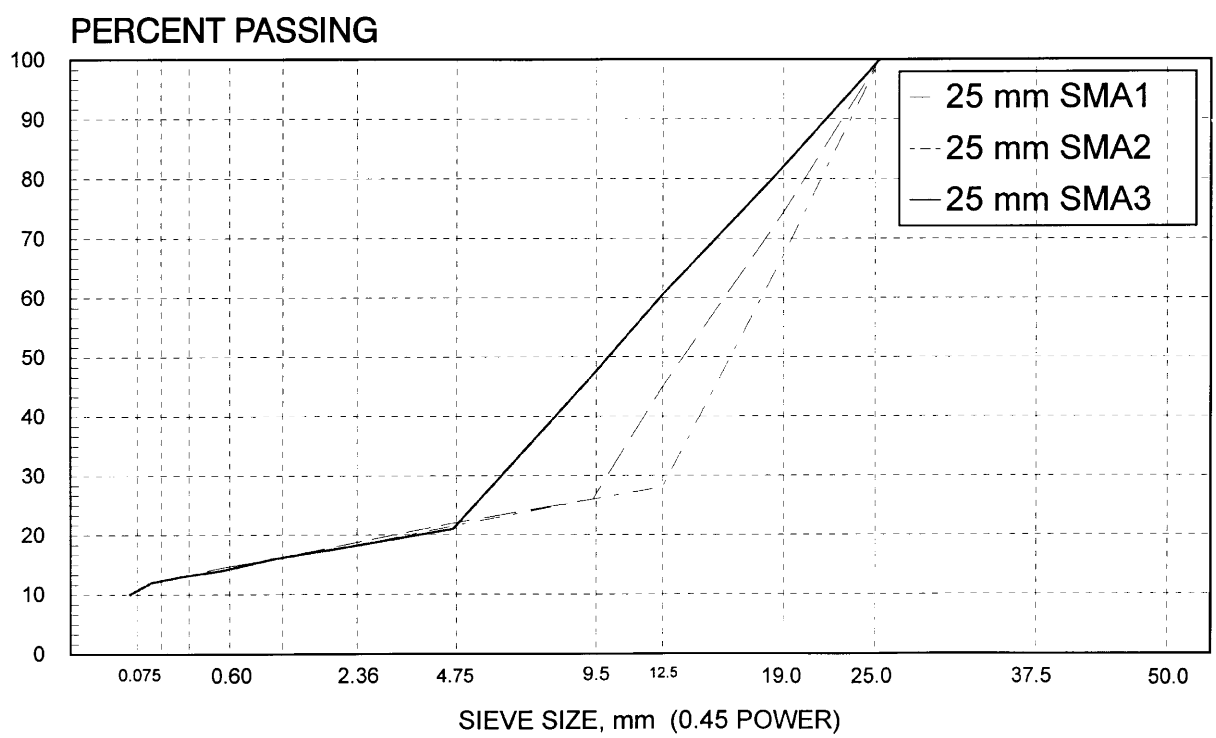


Figure 4.18: 25.0 mm NMS SMA Optimum Mixture Gradations

Table 4.30: Superpave Design Gradations

Sieve Size (mm)	Cumulative Percent Passing			
	9.5 ARZ	9.5 BRZ	25.0 ARZ	25.0 BRZ
37.5			100	100
25.0			95	95
19.0			89	89
12.5	100	100	78	72
9.5	95	95	72	60
4.75	85	85	58	35
2.36	67	40	45	22
1.18	50	23	30	15
0.6	35	16	20	10
0.3	22	12	13	7
0.15	12	8.5	7	5
0.075	6	5	3.5	3.5

ARZ - Above Restricted Zone
BRZ - Below Restricted Zone

cases the final design had undesirable characteristics.

Three mixes, 4.75/2.36, 4.75/1.18 and 9.5/4.75, had to be designed at optimum percentages passing the break point sieve in excess of 28 percent. The mix design procedure calls for the investigation of three trial blends, one with 20 percent passing the break point sieve, one with 24 percent, and one with 28 percent. In the case of these mixes, the blends exhibited high VMA even for the finest trial blend and did not allow for selection of the design gradation based on 17.5 percent VMA without extrapolation. Thus, a finer blend of 32 percent passing the break point sieve was investigated. As shown in Table 4.31 and Figure 4.21, in the case of the 9.5/4.75 mixture, a design gradation yielding 17.5 percent VMA was produced. Thus, the rationale of the design procedure was maintained. However, this was not the case for the two 4.75 mm NMS mixtures. For both BP sieves, a significant reduction in VMA was not achieved by increasing the percentage of fine aggregate. Subsequent trial blends with higher fine aggregate contents were not investigated due to VCA limitations. Table 4.31 shows a VCA ratio for these mixes of 0.982 and 0.996. Gradations finer than those indicated in Table 4.31 for the 4.75 mm mixes would have resulted in VCA ratios greater than 1.0 which is indicative of a loss of stone-on-stone contact. Thus, both of the 4.75 mm mixes were designed on the basis of limiting VCA ratio. Consequently, the high VMA values of 22.5 and 20.2 percent resulted in high asphalt contents (8.3 and 7.3 percent, respectively).

The 12.5/9.5 mix initially was not unlike the three mixes previously described. This mix had a VMA of 19.4 percent at the 28 percent passing the BP sieve trial blend. In addition, the VCA ratio was 0.989. Nevertheless, a 32 percent passing the BP sieve blend was investigated. The VMA dropped to 17.7 percent and the VTM was 3.5 percent at 6.0 percent asphalt cement.

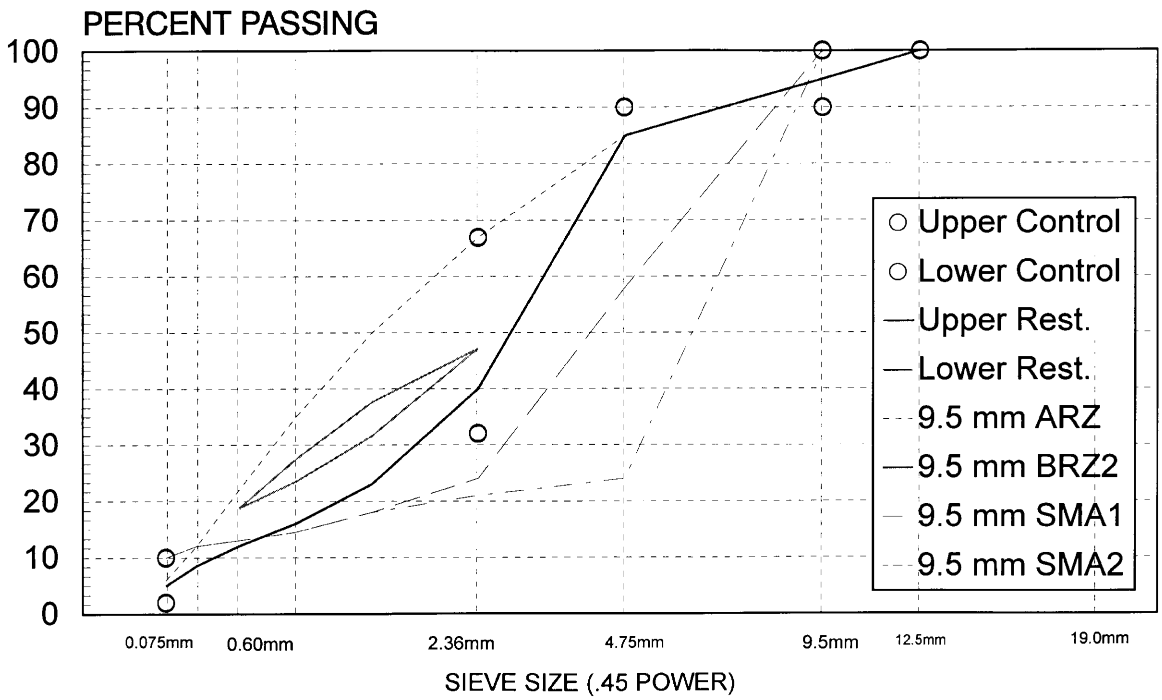


Figure 4.19: 9.5 mm NMS Superpave Optimum Mixture Gradations

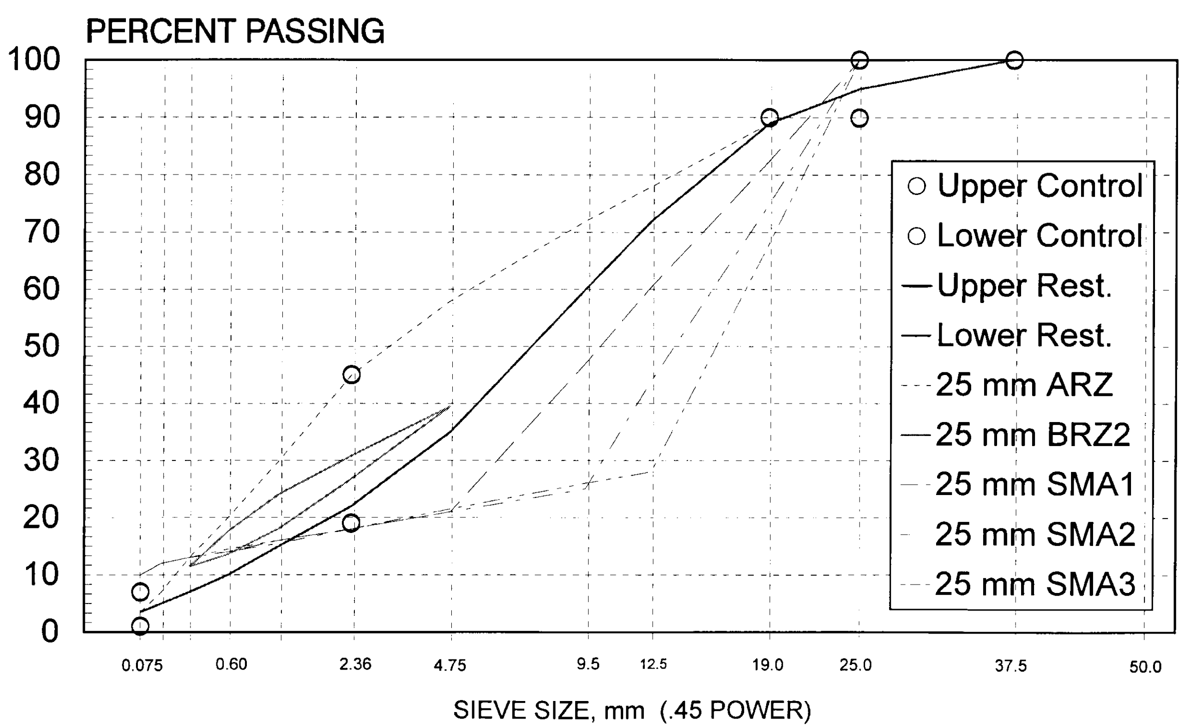


Figure 4.20: 25.0 mm NMS Superpave Optimum Mixture Gradations

Table 4.31: Optimum SMA Mixture Properties						
Mixture (NMS/BP)	% Passing BP Sieve	% Asphalt	VTM %	VMA %	VCA %	VCA Ratio
4.75/2.36	31	8.3	3.8	22.5	46.6	0.982
4.75/1.18	32	7.3	3.8	20.2	45.8	0.996
9.5/4.75	30	5.6	3.8	17.5	41.7	0.932
9.5/2.36	23	5.8	3.8	17.6	37.9	0.864
12.5/9.5	26	8.0	3.8	22.0	42.4	0.996
12.5/4.75	27	6.1	3.8	18.0	40.3	0.948
19/4.75-I	24	5.6	3.6	17.4	36.3	0.876
19/4.75-II	24	5.5	3.6	16.2	36.3	0.825
25/12.5	28	5.6	3.7	17.1	40.7	0.954
25/9.5	26	5.5	3.5	17.1	39.0	0.912
25/4.75	21	6.3	3.7	18.3	35.7	0.859

19/4.75-I - Gradation used in Phase I. 19/4.75-II - Gradation developed during Phase II.

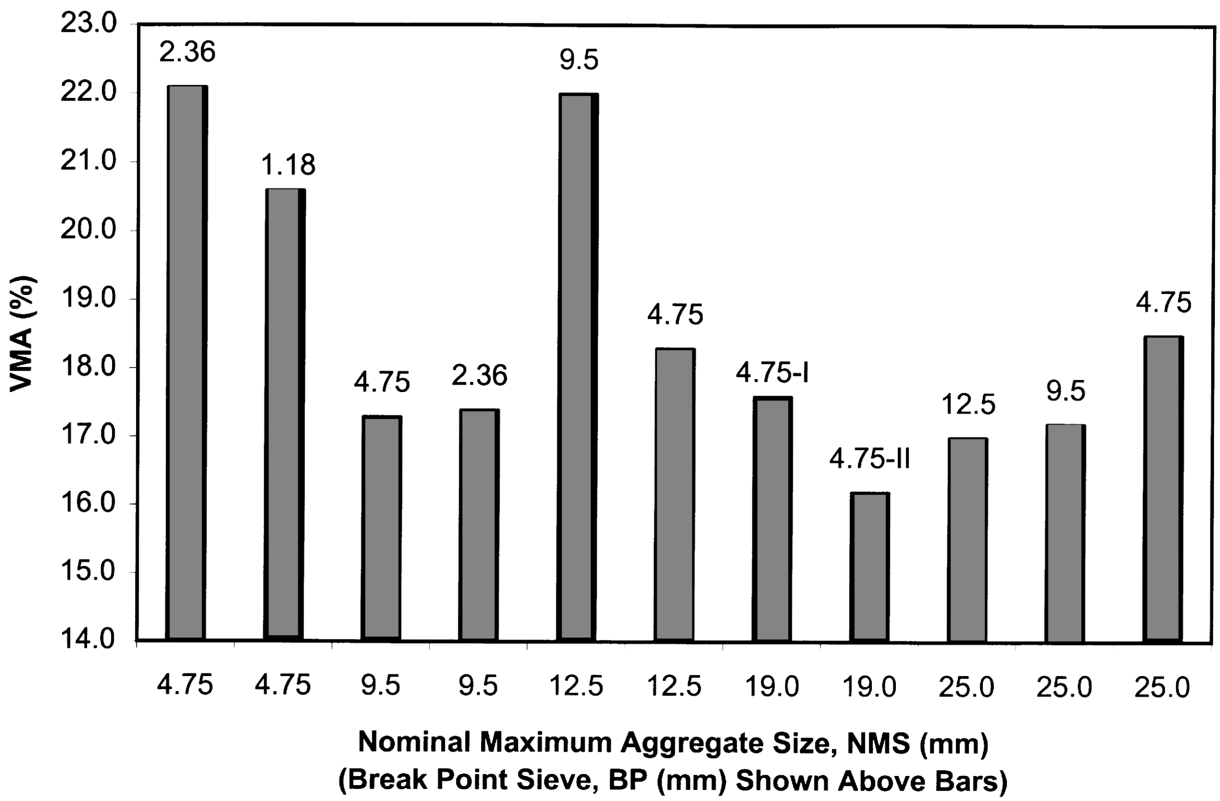


Figure 4.21: SMA Optimum Mixture Voids in Mineral Aggregate

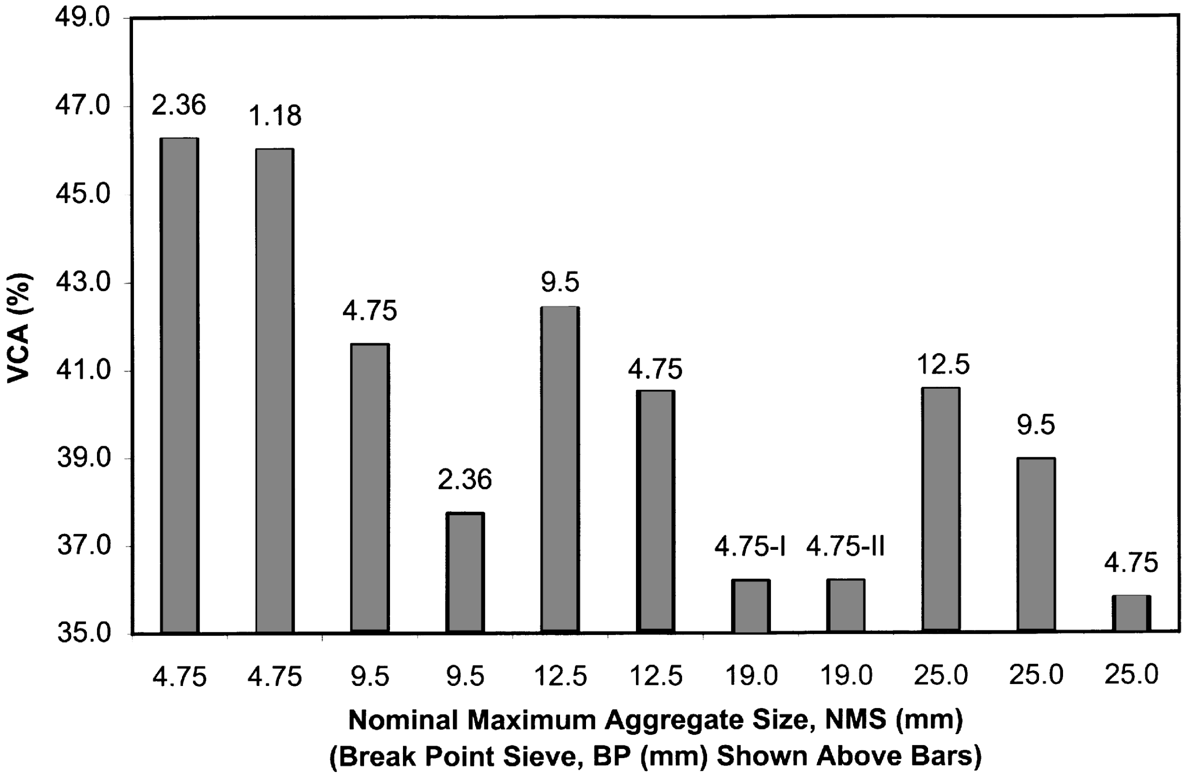


Figure 4.22: SMA Optimum Mixture Voids in Coarse Aggregate

This trial blend would have been acceptable as the design gradation and asphalt content. However, the VCA ratio increased to 1.039 indicating a loss of stone-on-stone contact. The 28 percent passing the BP sieve was therefore selected as the design gradation. However, subsequent attempts to optimize the asphalt content for this gradation revealed further complications. The 28 percent passing the BP sieve had such a high VMA that a asphalt content significantly higher than the trial blend asphalt content was required in order to reduce the VTM to the design level. The problem was that the VCA ratio increased and surpassed the limiting value of 1.0 before the design VTM level could be reached. In other words, the addition of more asphalt was resulting in dilation of the mix and loss of stone-on-stone contact. Thus, it was evident that the mix required a higher dust or filler content in order to reduce the VMA and subsequently lower the asphalt demand such that dilation would not occur. This was not possible however since filler content was being held constant to evaluate and verify the mix design procedure. Thus, although contrary to what was actually needed, the fine aggregate content was reduced in order to provide more volume for the asphalt cement that was needed to reduce to VTM to between 3.5 and 4.0 percent without the detrimental effects of dilation. It was decided, to use a 26 percent passing the BP sieve. This yielded a design with acceptable volumetric properties. The VTM was 3.8 percent, the VMA was 22.0 percent and the VCA ratio was 0.996. The optimum asphalt content of 8.0 percent was not surprising given the high VMA. This mix provides evidence to suggest that some NMS/BP combinations, particularly those having predominantly single size or uniform coarse aggregate gradations, may not be desirable for producing SMA mixtures.

Table 4.31 also shows no significant difference between the Phase I 19.0 mm mix (19/4.75-I) and the Phase II 19.0 mm mix (19/4.75-II). Both gradations had the same percent passing the break point sieve and essentially the same asphalt content indicating essentially the same volumetrics even though the gradations are different.

4.2.1.3 Superpave Mix Designs

The optimum mixture properties for the Superpave mixes are given in Table 4.32 and the complete mix design data is shown in the appendix in Volume V. As shown by Table 4.32, the 9.5ARZ and 25ARZ had optimum asphalt contents of 5.7 percent and 4.4 percent, respectively. In addition, each mix satisfied the Superpave mixture criteria for the various mix parameters. The 9.5BRZ and 25BRZ mixes exhibited high VMA similar to the coarser SMA mixes. The 25BRZ mix satisfied the Superpave mixture criteria; however, the VMA value of 15.8 percent was considerably higher than the minimum required 13.0 percent. This was the result of a combination of a coarse gradation and the tough, traprock aggregate that was resistant to breakdown in the compactor. The 9.5BRZ mixture did not satisfy the Superpave mixture criteria. As shown by Table 4.32, the 9.5 mm BRZ gradation (9.5 BRZ) had high VMA (19.3 percent) and as a result a high optimum asphalt content (7.2 percent). The VFA of 79.0 percent was the parameter that did not satisfy Superpave requirements for VFA of between 65 and 75 percent. The VMA of 19.3 percent was significantly higher than the minimum required 15.0 percent and therefore caused the high VFA.

Table 4.32: Optimum Superpave Mixture Properties

Mixture (NMS)	% Asphalt	VTM %	VMA %	VFA %	% G_{mm} @ N_{ini}	% G_{mm} @ N_{max}
9.5 ARZ	5.7	4.0	16.1	75.0	87.1	97.1
9.5 BRZ	7.2	4.0	19.3	79.0	85.7	97.4
25 ARZ	4.4	4.0	13.5	69.0	87.4	97.2
25 BRZ	5.1	4.0	15.8	74.0	85.8	97.4

4.2.1.4 Wheel Tracking

APA wheel tracking results at 55°C and 64°C are shown in Table 4.33 and illustrated in Figures 4.23 and 4.24. At 55°C all of the mixtures rutted less than 5 mm at 8000 cycles. This is significant since this is the criteria often used for design acceptance of high performance mixes using the APA. It is not possible, due to the relative newness of the test, equipment and test procedure, to make comparisons among the individual mixes. However, it is interesting to note that the SMA and Superpave mixes having high VMA rutted the most as would be expected since the high VMA resulted in high asphalt contents. As would be expected, the rut depths at 64°C are greater than the values obtained at 55°C. However, approximately one-third of all the mixes still satisfy the typical criteria at 64°C (Figure 4.24) even though the criteria was established for 55°C.

Table 4.33: SMA and Superpave Wheel Tracking Results

Mixture Type	Average Rut Depth (mm)[1] (@ 55ºC)	Average Rut Depth (mm)[1] (@ 64ºC)
4.75/2.36	4.21	5.30
4.75/1.18	2.70	5.39
9.5/4.75	2.78	4.43
9.5/2.36	3.47	5.36
12.5/9.5	3.67	4.47
12.5/4.75	4.11	5.42
19/4.75-I	1.68	2.62
19/4.75-II	1.36	2.22
25/12.5	4.65	7.65
25/9.5	2.78	4.43
25/4.75	3.59	6.53
9.5 ARZ	1.74	Not tested.
9.5 BRZ	4.27	Not tested.
25 ARZ	1.68	Not tested.
25 BRZ	2.03	3.00

[1] Average rut depth after 8000 cycles of loading.
19/4.75-I - Gradation used during Phase I.
19/4.75-II - Gradation developed during Phase II.

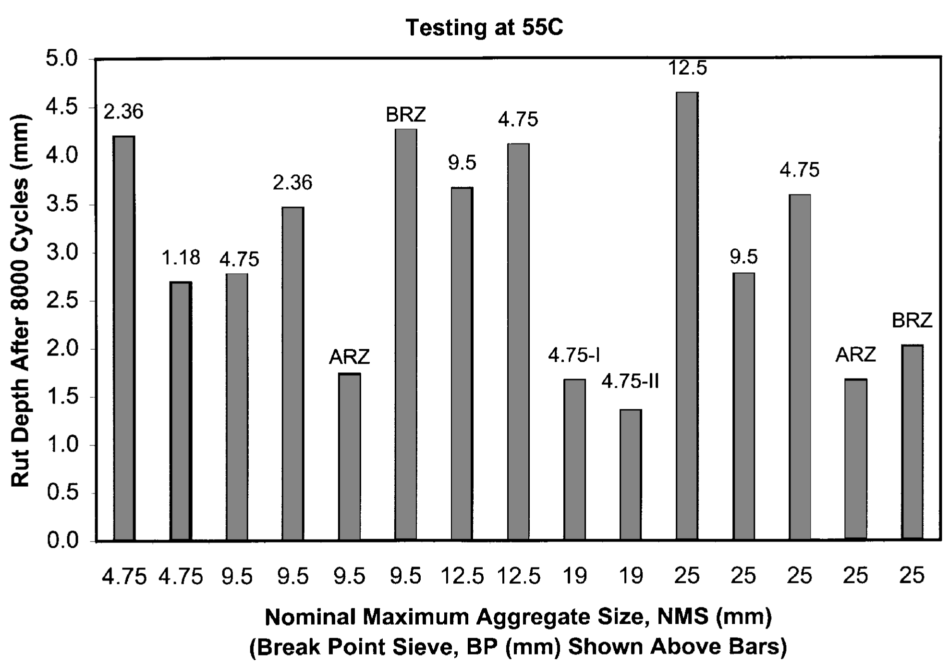


Figure 4.23: APA Wheel Tracking Results for 55°C

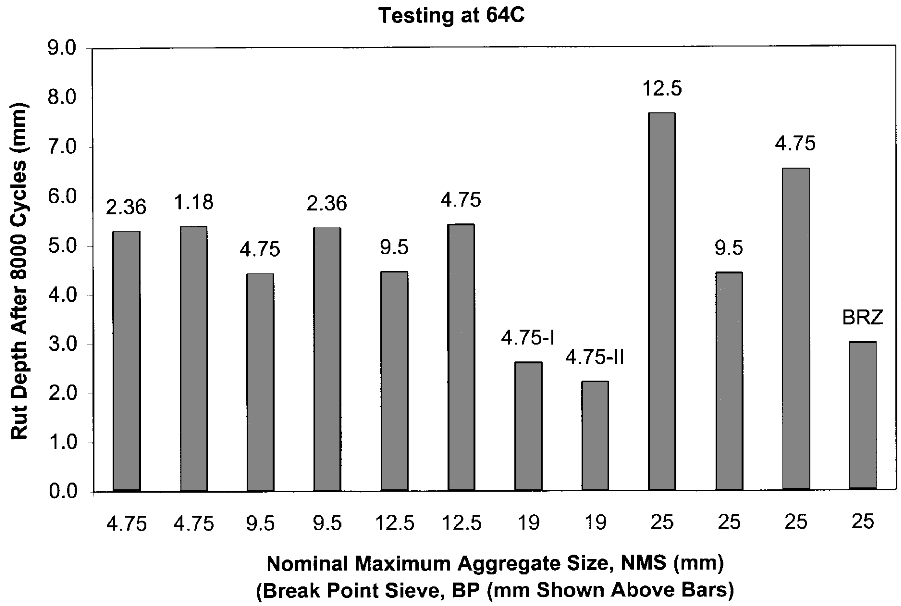


Figure 4.24: APA Wheel Tracking Results for 64°C

4.2.1.5 Permeability

Permeability results for all of the SMA mixtures are shown in Table 4.34 and Figures 4.25 through 4.29. For the 9.5 mm and 25 mm NMS plots, the Superpave results have been included for comparison.

Using the Florida DOT [Page, Musselman & Choubane] criteria of $100x10^{-5}$ cm/sec for limiting permeability as a guideline, it is evident that only the 4.75 mm mixes are considered impermeable over the range of densities usually established in the field. With respect to void content, the 4.75/2.36 mix becomes permeable at about 8 percent voids while the 4.75/1.18 mix becomes permeable at 10 percent voids. All of the other mixes were found to be permeable at less than 7 percent air voids. It is interesting to note that the 4.75 mm mixes had high VMA and thus one would expect higher permeability associated with these mixes. However, this illustrated the fact that it is the character of the voids that is important. In other words, both the size of air voids and the degree of interconnectivity of the voids is what controls the degree of permeability.

Generally speaking, the air void requirements could be set at 8 percent for 4.75 mm mixes for suitable permeability. For the 9.5 mm mixtures it would need to be set at approximately 6 percent, for the 12.5 and 19.0 mm mixtures it should be set at around 4 percent, and for the 25 mm it needs to be set at approximately 3 percent.

Regarding the Superpave mixes, Figure 4.26 shows that the 9.5 mm Superpave mixes were less permeable than the 9.5 mm SMA mixes for equal void levels. Of the two Superpave gradation types, the ARZ gradations were less permeable than the BRZ gradations (Table 4.34). Its also interesting to note that the 9.5 ARZ and 9.5 BRZ gradations satisfy the Florida DOT recommended criteria having less than $100x10^{-5}$ cm/sec even at 7 percent VTM. Neither of the 12.5 mm nor 19 mm mixes met the recommended permeability test results even at 6 percent air voids (Figures 4.27 and 4.28). The only 25 mm mixture that met the recommended permeability requirements at 6 percent air voids was the 25mm ARZ mix (Figure 4.29). The 25 ARZ mix became permeable at about 7.5 percent VTM.

Table 4.34: SMA and Superpave Permeability Results

Permeability Results for Each of Three Gyration Levels (10, 30, and 50)						
Mixture Type	10 Gyrations		30 Gyrations		50 Gyrations	
	Voids, %	Permeability, 10^{-5} cm/sec	Voids, %	Permeability, 10^{-5} cm/sec	Voids, %	Permeability, 10^{-5} cm/sec
4.75/2.36	11.5 11.9	1067 1267	7.3 6.0	68 55	6.9 6.5	20 63
4.75/1.18	14.1 13.6	553 694	9.7 9.7	81 94	7.5 7.4	24 28
9.5/4.75	8.7 9.7	5269 11677	7.4 8.1	1596 3572	9.9 6.7	28393[1] 1052
9.5/2.36	11.9 11.8	5701 7700	8.9 8.8	955 997	6.8 7.6	296 626
12.5/9.5	8.7 8.0	15017 8660	6.7 5.4	2866 2304	5.1 4.6	1624 860
12.5/4.75	10.1 10.1	6323 11638	7.5 7.4	2787 2224	5.7 5.6	882 1158
19/4.75-I	8.3 8.6	14500 11715	7.5 7.9	5141 3314	6.8 6.7	2037 1500
19/4.75-II	8.6 8.2	10645 12290	5.9 6.7	1307 2666	5.4 5.7	1129 370
25/12.5	6.1 6.1	30830 14313	4.5 5.4	2701 1903	5.1 5.0	68 1403
25/9.5	5.6 6.0	36660 22932	4.7 4.4	12738 4826	3.9 3.8	2223 2655
25/4.75	7.2 7.8	19176 11043	6.0 6.5	959 3316	4.5 4.3	649 1364
9.5 ARZ	11.7 11.5	487 467	8.2 8.6	66 57	7.1 7.0	20 21
9.5 BRZ	14.2 14.8	5128 4281	11.0 10.4	643 599	8.8 8.5	184 119
25 ARZ	11.4 12.3	996 2392	8.4 8.8	230 290	7.4 7.5	108 70
25 BRZ	8.3 8.8	15919 16088	8.1 8.7	3573 5437	6.8 6.5	2641 1847

[1] - Outlier

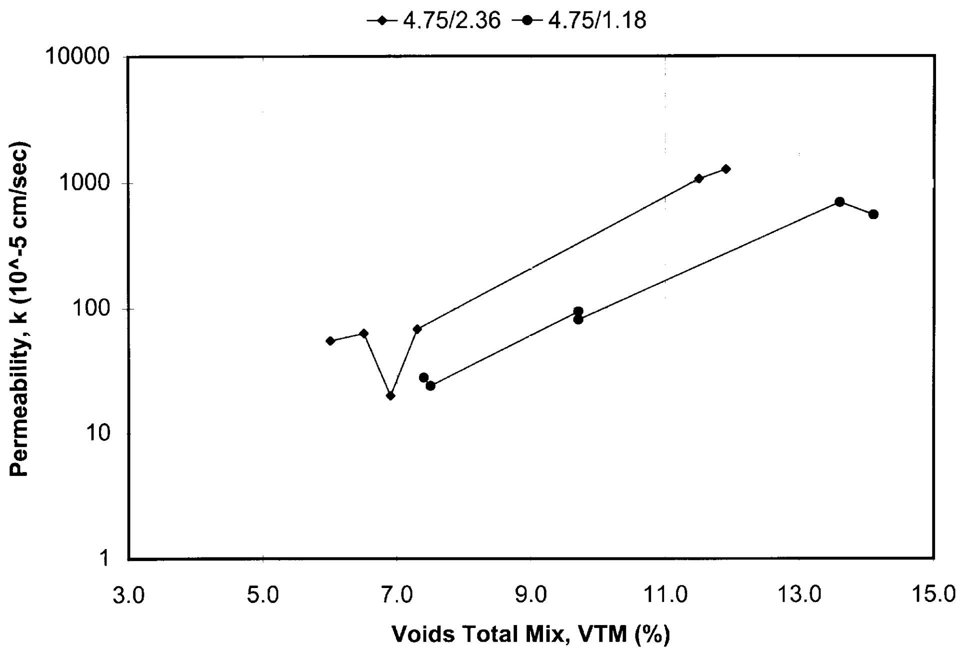


Figure 4.25: 4.75 mm NMS SMA Optimum Mixture Permeability

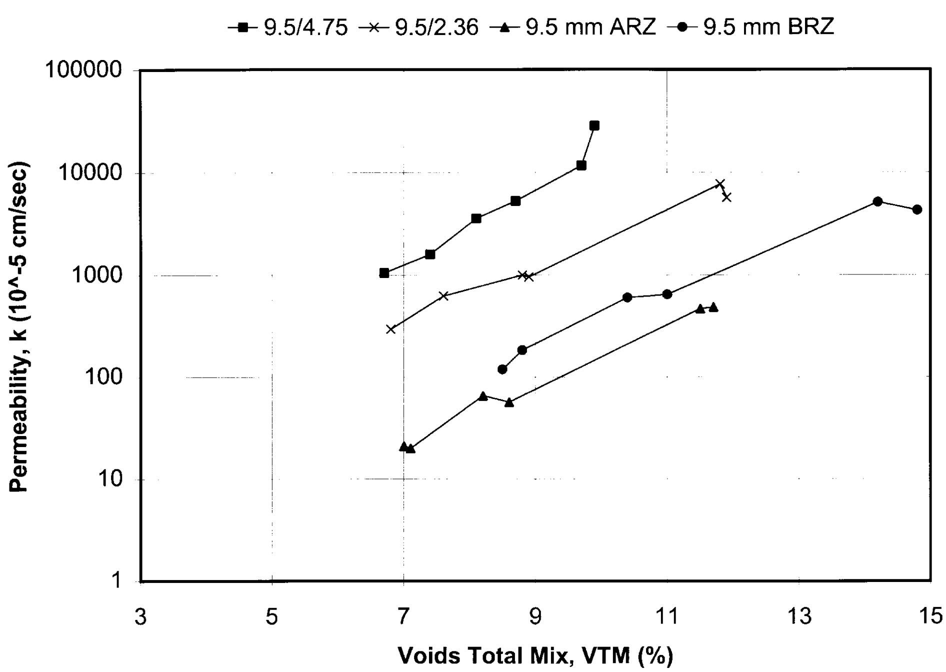


Figure 4.26: 9.5 mm NMS SMA and Superpave Optimum Mixture Permeability

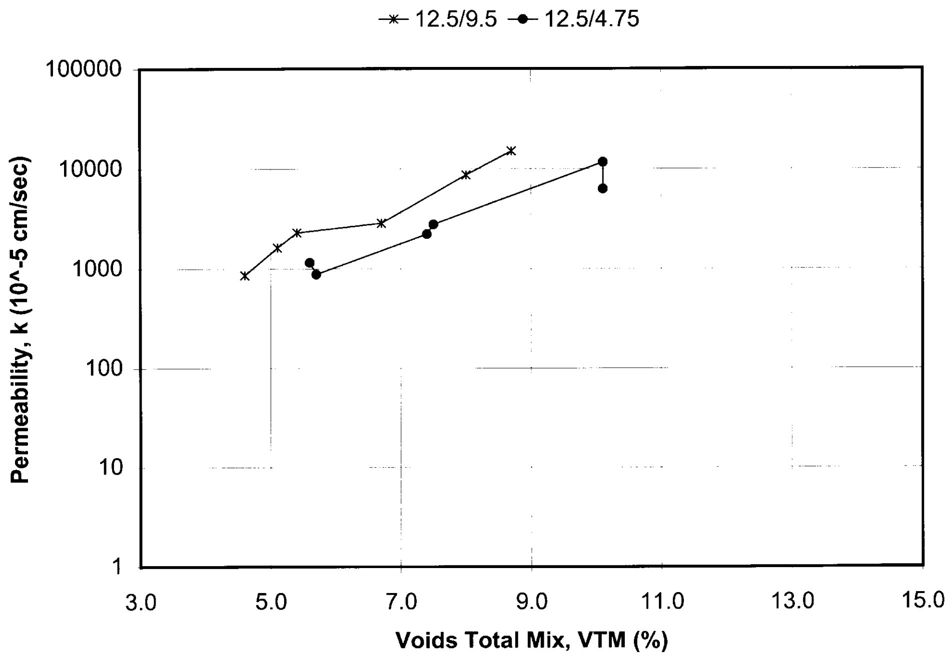


Figure 4.27: 12.5 mm NMS SMA Optimum Mixture Permeability

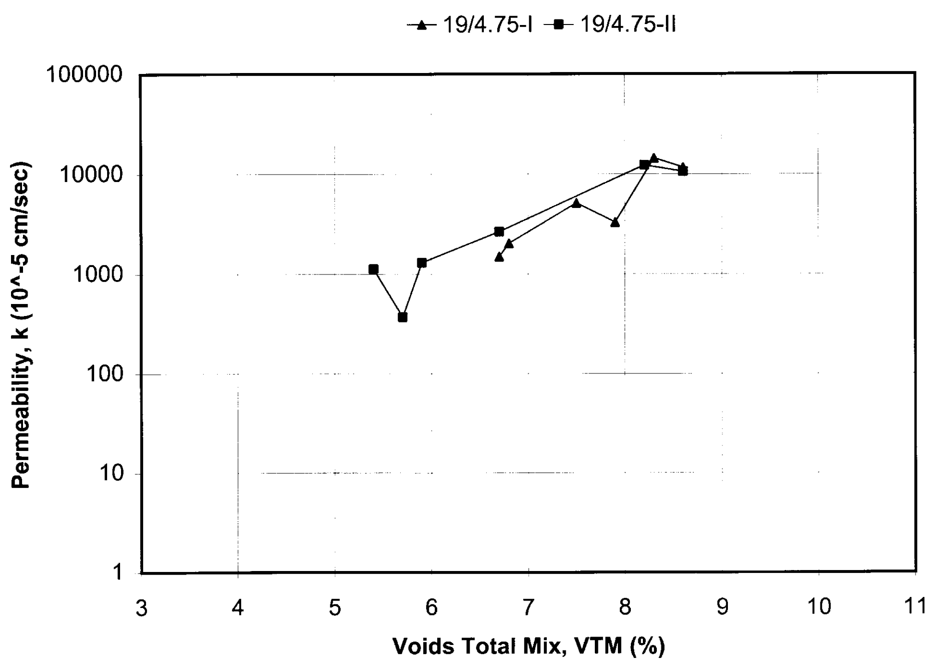


Figure 4.28: 19.0 mm NMS SMA Optimum Mixture Permeability

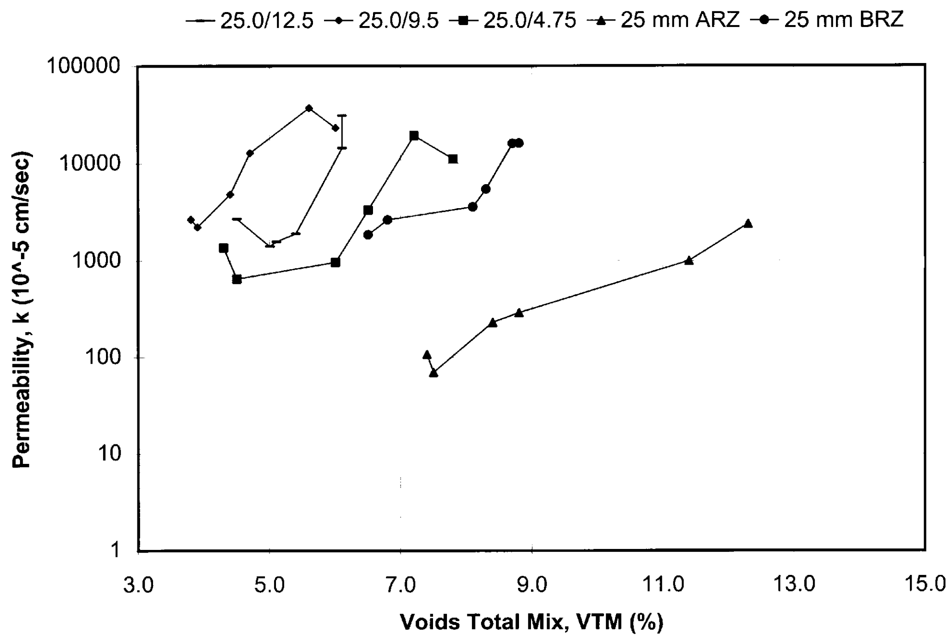


Figure 4.29: 25.0 mm NMS SMA and Superpave Optimum Mixture Permeability

4.2.2 Flat and Elongated Particles

The objective of this subtask was to evaluate the effect of flat and elongated particles on SMA mixtures using the SGC. During Phase I, mixture designs using aggregates with varying amounts of flat and elongated particles were performed using 50-blows of the Marshall hammer. Properties of the materials used are provided in Table 4.35. The coarse aggregate percentage was varied from 100 percent aggregate A1 (aggregate containing a high percentage of flat and elongated particles) to 100 percent of aggregate A2 (aggregate with a smaller percentage of flat and elongated particles).

Table 4.35: Properties of Aggregate Used in Flat and Elongated Subtask

Property	Test Method	Aggregate A1	Aggregate A2
Bulk Specific Gravity	AASHTO T85	2.615	2.593
Apparent Specific Gravity	AASHTO T85	2.651	2.638
Absorption, %	AASHTO T85	0.5	0.7
Flat and Elongated Particles, %	ASTM D4791		
2 to 1	Section 8.4	67	38
3 to 1		25	3
5 to 1		1	0
Crushed Content, %			
One Face		100	100
Two Faces		100	100

Aggregate A1 contained a high percentage of flat and elongated particles.
Aggregate A2 contained a lower percentage of flat and elongated particles.

The percentages of flat and elongated particles for each of the five combinations evaluated are presented in Table 4.36. The optimum gradations determined for the different blends are presented in Table 4.37 . However, for testing purposes, the gradation was held constant for all five mixtures used for testing to eliminate gradation as a variable. This gradation chosen produced SMA mixes that meet VMA and VCA requirements for all mixtures.

Testing of the different mixtures was similar to that performed during Phase I with the only difference being that the SGC was used to compact the specimens. The optimum asphalt contents determined from the Marshall designs were again used during this part of testing. Results of testing performed on these mixtures were then used to determine the effect of flat and elongated particles on VMA, aggregate breakdown, and moisture susceptibility when compacted with the SGC.

Table 4.36: Percent Flat and Elongated for Coarse Aggregates

Aggregate Gradation	Percent Flat and Elongated		
	2 to 1	3 to 1	5 to 1
100% A1	67	25	1
100% A2	38	3	0
75% A1 - 25% A2[1]	59	20	1
50% A1 - 50% A2[1]	52	14	0
25% A1 - 75 % A2[1]	45	8	0

Aggregate A1 contained a high percentage of flat and elongated particles.
Aggregate A2 contained a lower percentage of flat and elongated particles.
[1] Values Interpolated

Table 4.37: Optimum Gradations for Flat and Elongated Study Mixtures

Sieve Size, mm	Aggregate Mixtures				
	100% A1	100% A2	75% A1 - 25% A2	50% A1 - 50% A2	25% A1 - 75% A2
19.0	100.0	100.0	100.0	100.0	100.0
12.5	90.4	90.1	90.5	90.3	90.3
9.5	53.8	52.6	54.5	53.2	53.2
4.75	23.0	21.0	24.0	22.0	22.0
2.36	19.6	17.9	20.5	18.8	18.8
1.18	17.3	15.8	18.0	16.6	16.6
0.6	14.1	13.4	14.5	13.8	13.8
0.3	13.1	12.4	13.5	12.8	12.8
0.15	11.4	11.1	11.5	11.3	11.3
0.075	10.0	10.0	10.0	10.0	10.0

Aggregate A1 contained a high percentage of flat and elongated particles.
Aggregate A2 contained a lower percentage of flat and elongated particles.

4.2.2.1 Voids in Mineral Aggregate (VMA)

The VMA values of the five mixtures are presented in Table 4.38. The VMA is affected by the aggregate properties and thus is the only volumetric property that is almost completely controlled by the aggregate and compactive effort. Therefore, it is the only volumetric property that is discussed here. Using this data, Figure 4.30 was prepared to evaluate the effect of flat and elongated particles on SMA mixtures. Similar to Phase I, the data showed a trend of increasing VMA as the percent flat and elongated particles in the mix increased. The VMA increased by more than 0.5 percent but less than 1.0 percent.

Table 4.38: Flat and Elongated Experiment Optimum Mixture Properties

Mix Property	Aggregate Mixtures				
	100% A1	100% A2	75% A1 -25% A2	50% A1 - 50% A2	25% A1 - 75% A2
AC, %	7.2	6.9	7.0	6.8	6.8
VTM, %	3.1	4.0	3.1	3.2	3.6
VMA, %	18.3	17.6	17.7	17.3	17.6
VFA, %	82.8	77.5	82.5	81.6	79.5
VCA, %	34.8	34.1	34.2	33.9	34.1

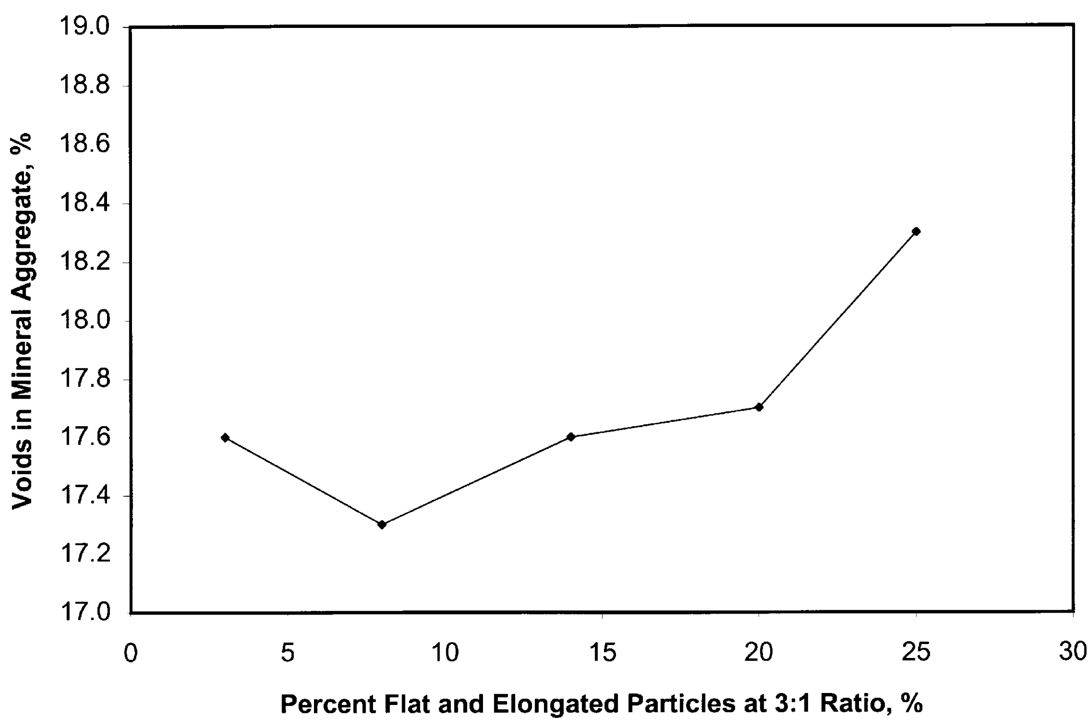


Figure 4.30: VMA Results for Flat or Elongated Particles

4.2.2.2 Aggregate Breakdown

Using the same gradation as was used in Phase I (shown as optimum gradation in Table 4.39), specimens were compacted at optimum asphalt content for each of the five mixtures. The asphalt cement was then removed using the ignition oven. Gradation analyses were conducted according to AASHTO T27 and T11 to determine the aggregate breakdown of each mixture after compaction with the SGC. Six test replicates were used. Table 4.39 shows the mean gradation test results for each mixture after compaction.

Table 4.39: Aggregate Breakdown Sieve Analysis Results

Sieve Size, mm	Optimum Gradation	Aggregate Mixtures				
		100% A1	100% A2	75% A1 - 25% A2	50% A1 - 50% A2	25% A1 - 75% A2
19.0	100.0	100.0	100.0	100.0	100.0	100.0
12.5	90.1	92.0	90.4	92.0	91.5	90.9
9.5	52.6	62.6	56.4	61.6	59.7	58.1
4.75	21.0	27.3	24.4	28.4	26.9	25.7
2.36	17.9	20.7	19.4	20.8	20.4	19.9
1.18	15.8	17.2	16.5	17.4	17.1	16.9
0.6	13.4	13.3	13.8	14.5	14.3	14.1
0.3	12.4	12.8	12.6	13.0	13.1	12.8
0.15	11.1	11.3	11.2	11.6	11.5	11.4
0.075	10.0	9.3	9.1	9.5	9.4	9.3

This data was analyzed using an ANOVA. Results of this analysis (Table 4.40) showed that there are significant differences in the gradation results for the five mixtures. Figures 4.31 and 4.32 demonstrate graphically and confirm that the aggregate breakdown generally increases as the percent flat and elongated particles in the mixture increases. Again, similar to the Phase I analysis, a second statistical analysis was performed to determine if the four mixes containing flat and elongated particles differed with respect to aggregate breakdown (Table 4.40). Unlike mixtures compacted with the Marshall hammer, this analysis indicated that there was a significant difference at the five percent significance level for the 4.75 mm sieve examined between the four mixes having flat and elongated particles.

4.2.2.3 Marshall Hammer vs. Superpave Gyratory

An ANOVA was again used to determine if differences existed for the amount of aggregate breakdown when using the SGC and the Marshall hammer (α=0.05). Results of this analysis showed that there were significant differences between the two compactive efforts for the amount of aggregate breakdown as shown on the 4.75 and 0.075 mm sieves (F-stat=32.39 and Probability > F=0.000 and F-stat=3.00 and Probability > F = 0.009, respectively). Figures 4.33 and 4.34 graphically show these differences. Figure 4.33 illustrates that the Marshall hammer produced more aggregate breakdown on the 4.75 mm sieve than did the SGC. On average, the

Table 4.40: Results of ANOVA Performed To Compare Aggregate Breakdown on Mixtures With Varying Amounts of Flat and Elongated Particles

Analysis No.	Aggregate Breakdown on 4.75 mm Sieve – Mixtures included in Analysis					*F*-stat	Prob. > F	Aggregate Breakdown on 0.075 mm Sieve – Mixtures included in Analysis					*F*-stat	Prob. > F
	0% A1	25% A1	50% A1	75% A1	100% A1			0% A1	25% A1	50% A1	75% A1	100% A1		
1	✓	✓	✓	✓	✓	46.85	0.000	✓	✓	✓	✓	✓	0.63	0.6482
2		✓	✓	✓	✓	39.14	0.000							

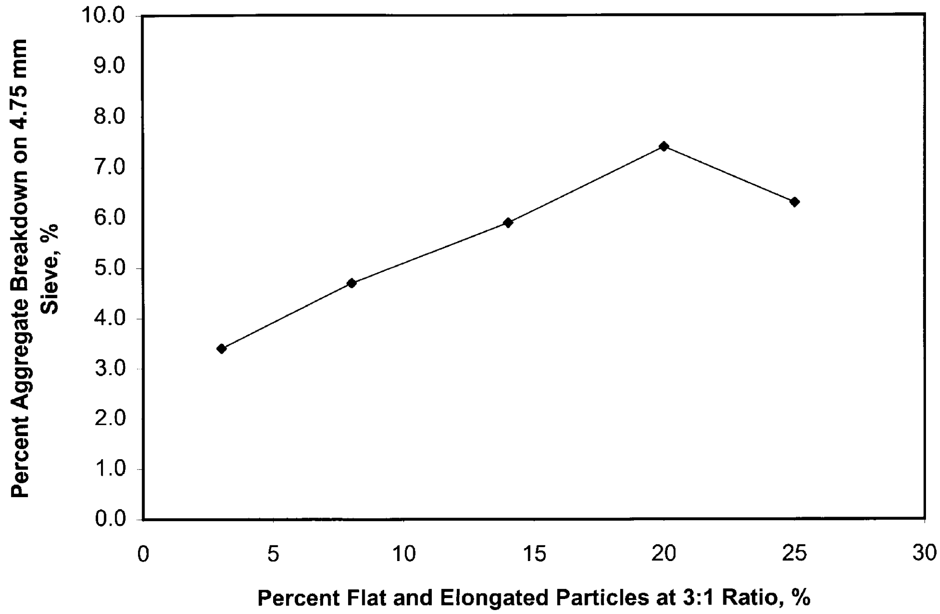


Figure 4.31: Aggregate Breakdown on the 4.75 mm Sieve as a Function of Percent Flat and Elongated Particles

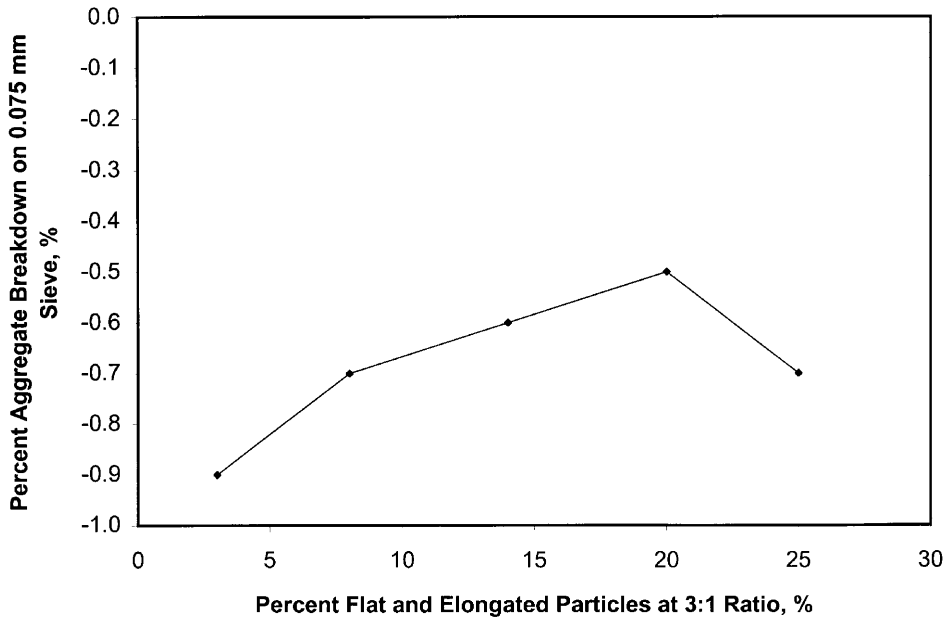


Figure 4.32: Aggregate Breakdown on the 0.075 mm Sieve as a Function of Percent Flat and Elongated Particles

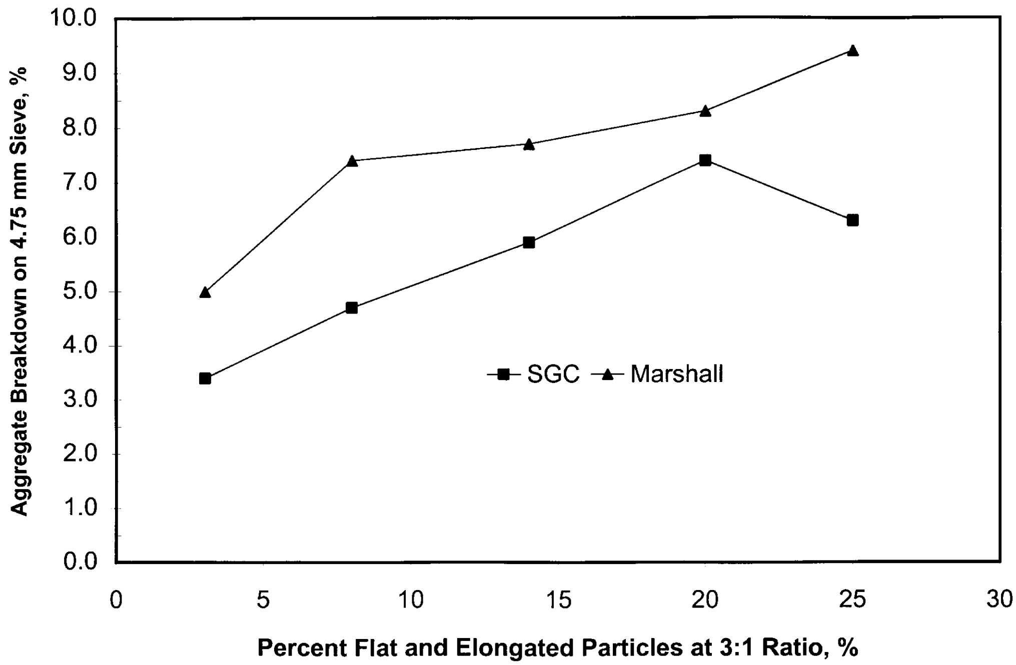


Figure 4.33: Aggregate Breakdown Comparison on the 4.75 mm Sieve Between Marshall Hammer (50-Blows) and SGC (100 Revolutions)

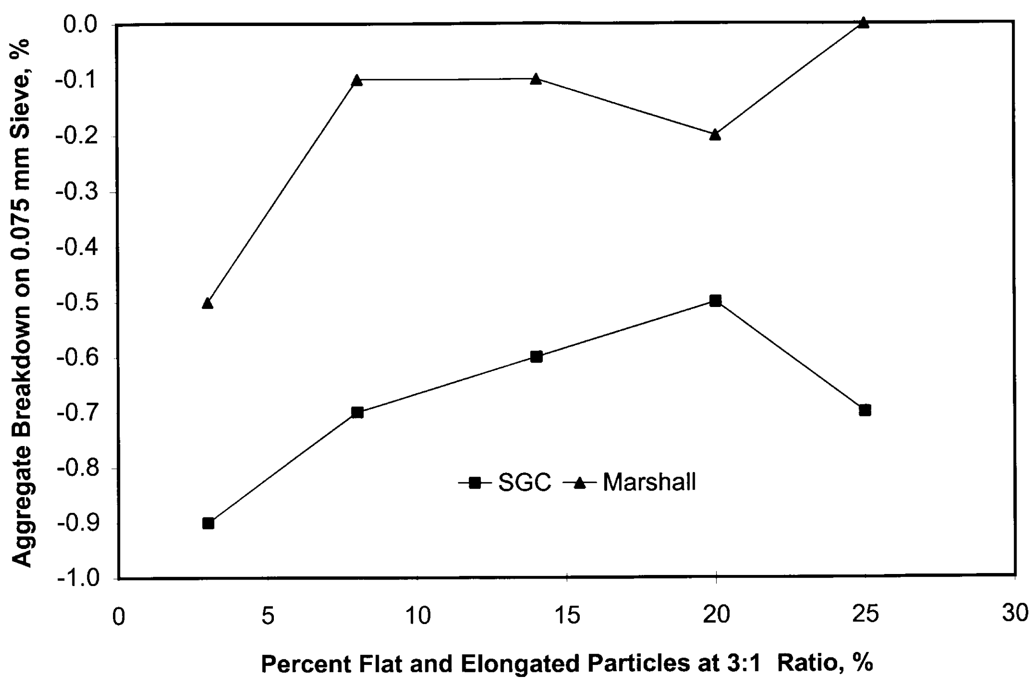


Figure 4.34: Aggregate Breakdown Comparison on the 0.075 mm Sieve Between Marshall Hammer (50-Blows) and SGC (100 Revolutions)

Marshall hammer produced approximately two percent more breakdown on the 4.75 mm sieve and approximately 0.5 percent more breakdown on the 0.075 mm sieve.

4.2.2.4 Moisture Susceptibility Testing

It was thought that flat and elongated aggregates, and particularly the breakdown of those aggregates during compaction, would increase the susceptibility of SMA mixtures to moisture induced damage. Thus, AASHTO T283 was conducted on the five mixtures. The tensile strength ratios (TSR) of each mix along with the average unconditioned and conditioned tensile strengths are given in Table 4.41.

Table 4.41: Moisture Susceptibility (AASHTO T283) Results

Mix Property	Aggregate Mixtures				
	100% A1	100% A2	75% A1 - 25% A2	50% A1 - 50% A2	25% A1 - 75% A2
Unconditioned Tensile Strength, kPa	762	766	762	759	764
Conditioned Tensile Strength, kPa	328	327	332	308	327
Tensile Strength Ratio, %	43.0	42.6	43.8	40.6	42.8

Since only one repetition of the test method was completed, a statistical analysis was not possible. However, a generalized observation of the data was made. Figure 4.35 shows the plot of the data determined in the analysis. The graph of the data suggests that the TSR data, although low, is not influenced by the presence of flat and elongated particles within the mixture. This was also observed during Phase I analysis of the TSR data for Marshall compaction. The TSR data for this phase exhibited a difference between the two extreme values of only 3.0 percent. This value is well within the multi-laboratory standard deviation for TSR testing as given in ASTM D 4867 (8 percent).

In summary, for the aggregate source evaluated (Arkansas limestone), it appears that flat and elongated particles affect the VMA and the aggregate breakdown during compaction but does not affect the moisture susceptibility of mixtures compacted with the SGC. The amount of flat and elongated particles should not be increased to produce higher values of VMA. High percentages of flat and elongated particles were shown to cause excessive aggregate breakdown.

4.3 AGGREGATE BREAKDOWN IN THE FIELD

This subtask was designed to evaluate the aggregate breakdown occurring during field compaction. Two possible variables causing variation in the aggregate breakdown during field compaction are the type of underlying pavement (Portland cement concrete or HMA) and the type of rollers used. The following subsections discuss these two variables.

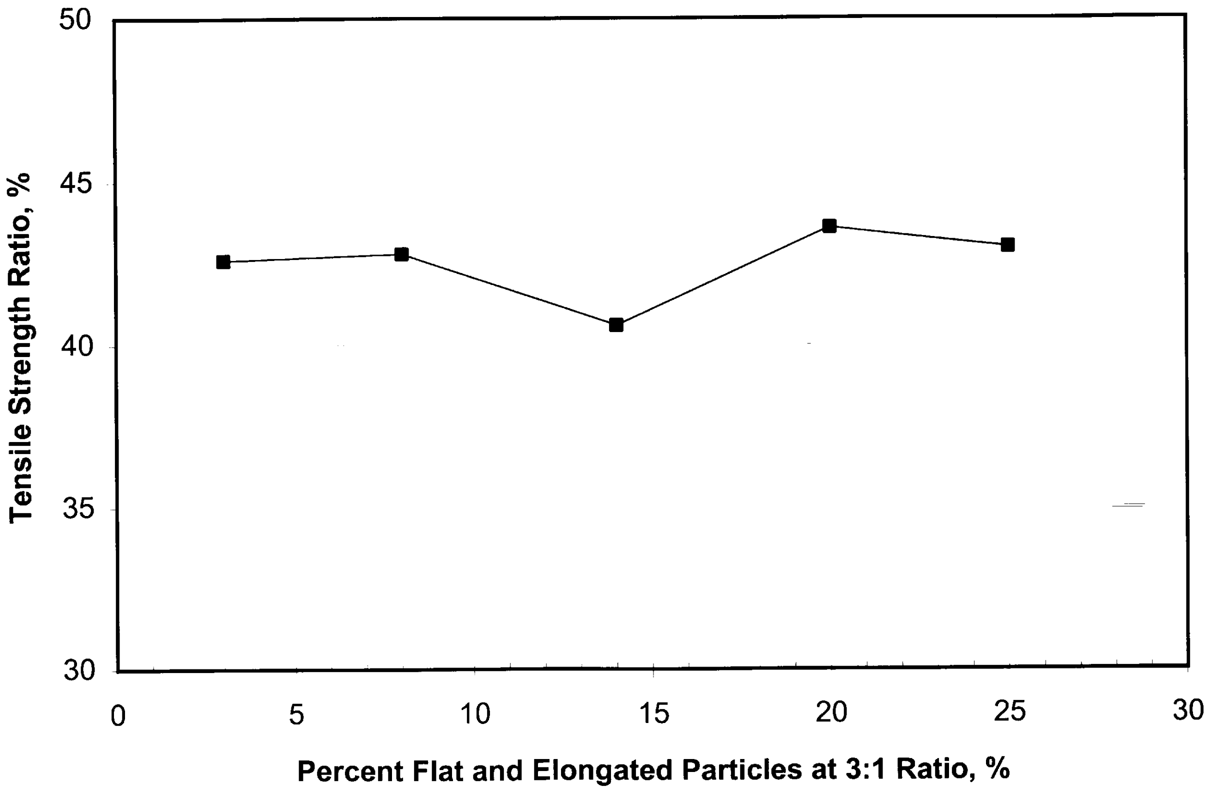


Figure 4.35: Moisture Susceptibility Results for the Flat and Elongated Subtask

4.3.1 Effect of Underlying Pavement

This subtask was included to determine if SMA shows more breakdown when compacted on Portland cement concrete (PCC) pavements than on HMA pavements; however, only one project (site 6) in which cores were obtained was constructed over a PCC pavement. The breakdown data is provided in Table 4.42. Site 6 actually had the least amount of breakdown of all mixtures for the 4.75 mm sieve. Site 5 was constructed over a PCC pavement but several HMA overlays had been placed over it. In addition, the HMA overlays were not milled and therefore could not be used in this comparison.

4.3.2 Effect of Roller Type

The objective of this subtask was to determine whether or not the type of roller affects the amount of aggregate breakdown during field compaction. This analysis consisted of comparing the aggregate breakdown values for each of the eight projects in which cores were obtained. Similar to previous aggregate breakdown analyses, both the 4.75 and 0.075 mm sieves were investigated. Aggregate breakdown for each core was defined as the percent passing for the core minus the percent passing the average loose mixture gradation.

Of the eight projects in which cores were obtained, five used vibratory rollers, two used only static steel-wheel rollers, and one used both pneumatic tire rollers and static steel-wheel rollers. The analysis was accomplished using an ANOVA at a level of significance of 0.05. Since

Table 4.42: Aggregate Breakdown Exhibited During Field Compaction

Site	Avg. Breakdown on 4.75 mm Sieve, %	Avg. Breakdown on 0.075 mm Sieve, %
2	6.3	2.1
3	11.1	1.0
4	4.8	0.4
5	5.2	1.4
6	3.7	0.1
9	4.4	2.3
10	4.5	-1.1
11	5.6	-0.5

only one site used pneumatic tire rollers with static steel-wheel rollers (site 3), the analysis was performed to determine if significant differences occurred in the amount of aggregate breakdown for the vibratory and static steel-wheel rollers. Results of this analysis showed that no significant differences occurred in the aggregate breakdown data for the 4.75 or 0.075 mm sieve data (F-stat = 0.45 and Probability > F = 0.504 and F-stat = 0.34 and Probability > F = 0.563, respectively). Figure 4.36 illustrates the differences in aggregate breakdown on the 4.75 mm sieve for the three different types of rollers. However, since the sample size is small more data is needed before a conclusion can be made comparing static and vibratory rollers.

Several other factors could have caused the significant differences in the 4.75 mm aggregate breakdown data; most prevalently the hardness of the aggregate (LA Abrasion). The hardness of the different aggregates were presented in Table 4.3. Of the eight sites shown in Figure 4.36 , sites 6 and 10 had the lowest LA Abrasion loss values (16 and 15, respectively). From the figure, this seems logical as both had low breakdown values. Interestingly, site 4 had the highest LA Abrasion loss value at 42 percent but the aggregate breakdown was not excessively high. Site 3 had an LA Abrasion of 23 and yet had the greatest amount of breakdown. Based on the data it is unclear why site 3 had the highest breakdown values. Therefore, it appears for the projects included in this analysis, the LA Abrasion loss values did not influence the aggregate breakdown.

Based on this discussion, it appears that the type of roller, whether vibratory or static, does not affect the aggregate breakdown of SMA.

4.4 POTENTIAL DAMAGE TO ASPHALT CEMENT AT HIGH TEMPERATURES

High mixing temperatures are sometimes associated with SMA because of the relative stiff binders used in these mixtures and the high filler contents. This part of the research effort was performed to evaluate the effect of high temperatures on asphalt cements. The effort involved six asphalt binders: three unmodified and three modified. Samples of each binder were heated using five different temperatures in a forced draft oven utilizing thin film oven pans. For each of the binders, the Superpave binder tests were conducted before and after the aging procedures. Testing with the DSR was conducted at 64°C, while BBR and DTT testing was conducted at -

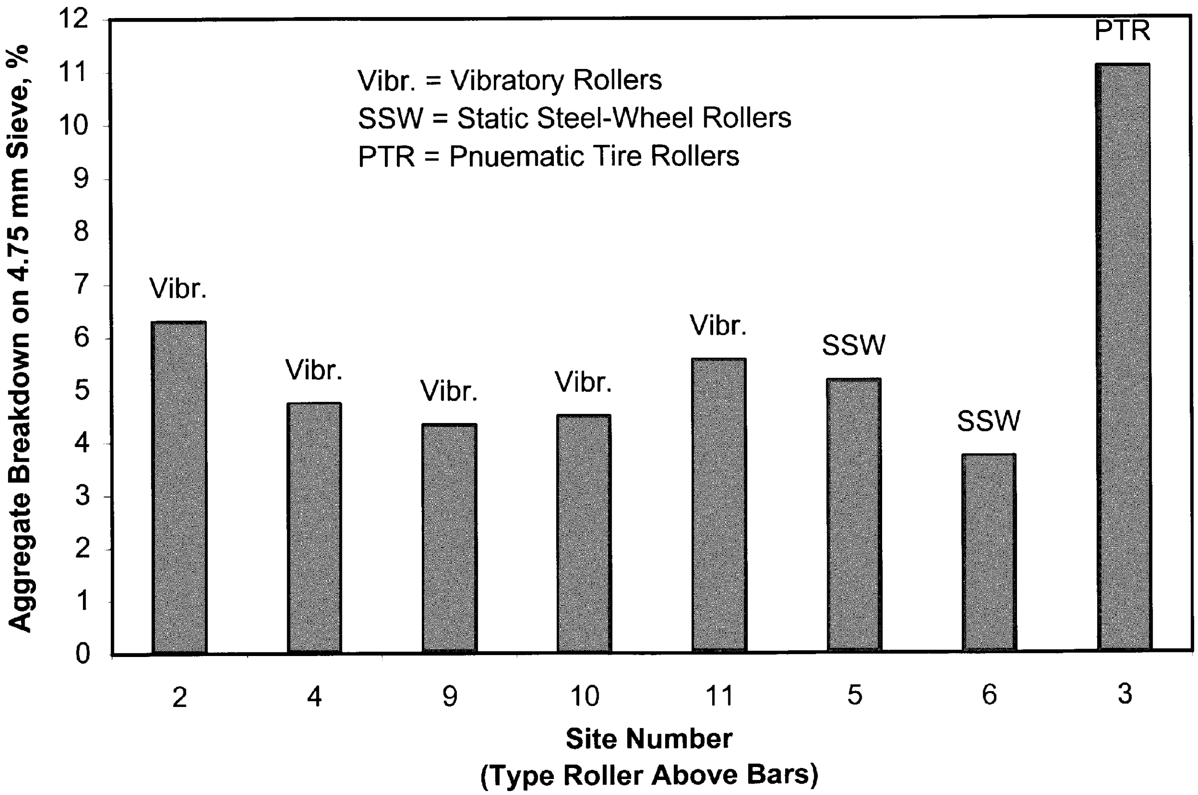


Figure 4.36: Aggregate Breakdown on the 4.75 mm Sieve as a Function of Roller Type

12°C. Results of this testing are presented in Tables 4.43 through 4.46.

In order to statistically analyze the test results, an ANOVA was performed to determine if significant differences existed in the means of the different test results for the different aging temperatures. Results of the ANOVA indicated that there were significant differences in the test results for the different aging temperatures (Table 4.47). The differences are shown graphically in Figures 4.37 through 4.40. To further investigate at which temperature damage occurs to an asphalt cement, Duncan's Multiple Range Test rankings were performed for each of the test

Table 4.43: DSR Results to Evaluate Potential Damage to Asphalt Cement at High Temperatures

Asphalt Binder	Oven Temperatures (G*/sinδ, kPa)					
	No Aging	150°C	165°C	180°C	195°C	210°C
Neat AC No. 1	1.14	2.06	3.96	4.65	15.20	14.35
Neat AC No. 2	1.10	2.01	3.15	3.70	8.56	13.07
Neat AC No. 3	1.27	2.18	3.45	3.84	10.96	11.79
Modified AC No. 1	3.14	5.17	7.78	7.01	15.57	18.38
Modified AC No. 2	4.81	5.44	8.35	6.82	10.77	9.79
Modified AC No. 3	1.75	3.75	3.53	4.30	10.46	11.42

Table 4.44: BBR Creep Stiffness Results to Evaluate Potential Damage to Asphalt Cement at High Temperatures

Asphalt Binder	Oven Temperatures (Creep Stiffness, MPa)					
	No Aging	150°C	165°C	180°C	195°C	210°C
Neat AC No. 1	81	108	162	171	284	274
Neat AC No. 2	58	62	81	80	100	103.5
Neat AC No. 3	118	141	162	171	213	199
Modified AC No. 1	118	120	173	154	318	297
Modified AC No. 2	63	70	76	76	81	73
Modified AC No. 3	103	122	141	145	200	194

Table 4.45: BBR m-value Results to Evaluate Potential Damage to Asphalt Cement at High Temperatures

Asphalt Binder	Oven Temperatures (m-value)					
	No Aging	150°C	165°C	180°C	195°C	210°C
Neat AC No. 1	0.528	0.481	0.431	0.428	0.357	0.358
Neat AC No. 2	0.495	0.450	0.426	0.416	0.389	0.373
Neat AC No. 3	0.478	0.444	0.405	0.412	0.372	0.373
Modified AC No. 1	0.449	0.405	0.364	0.369	0.316	0.299
Modified AC No. 2	0.439	0.426	0.384	0.411	0.400	0.416
Modified AC No. 3	0.493	0.471	0.438	0.438	0.391	0.389

Table 4.46: DTT Results to Evaluate Potential Damage to Asphalt Cement at High Temperatures

Asphalt Binder	Oven Temperatures (% Strain)					
	No Aging	150°C	165°C	180°C	195°C	210°C
Neat AC No. 1	0.6337	0.4006	0.2281	0.2265	0.1950	0.3073
Neat AC No. 2	0.8152	0.4253	0.5158	0.2678	0.5334	0.5467
Neat AC No. 3	0.8194	0.4795	0.3985	0.6725	0.5079	0.3576
Modified AC No. 1	0.5415	0.1603	0.2159	0.2424	0.2730	0.2211
Modified AC No. 2	0.2427	3.3810	1.9511	2.2337	1.4092	0.7909
Modified AC No. 3	1.0479	0.8696	0.5104	0.5440	0.2183	0.3521

Table 4.47: ANOVA Results for Superpave Binder Tests Performed to Evaluate Potential Damage to Asphalt Cement at High Temperatures

Test and Property	*F*-stat	Probability > *F*	Significant Difference
DSR, G*/Sinδ	12.83	0.000	Yes
BBR, Creep Stiffness	39.52	0.000	Yes
BBR, m-value	37.91	0.000	Yes
DTT, Percent Strain	17.03	0.000	Yes

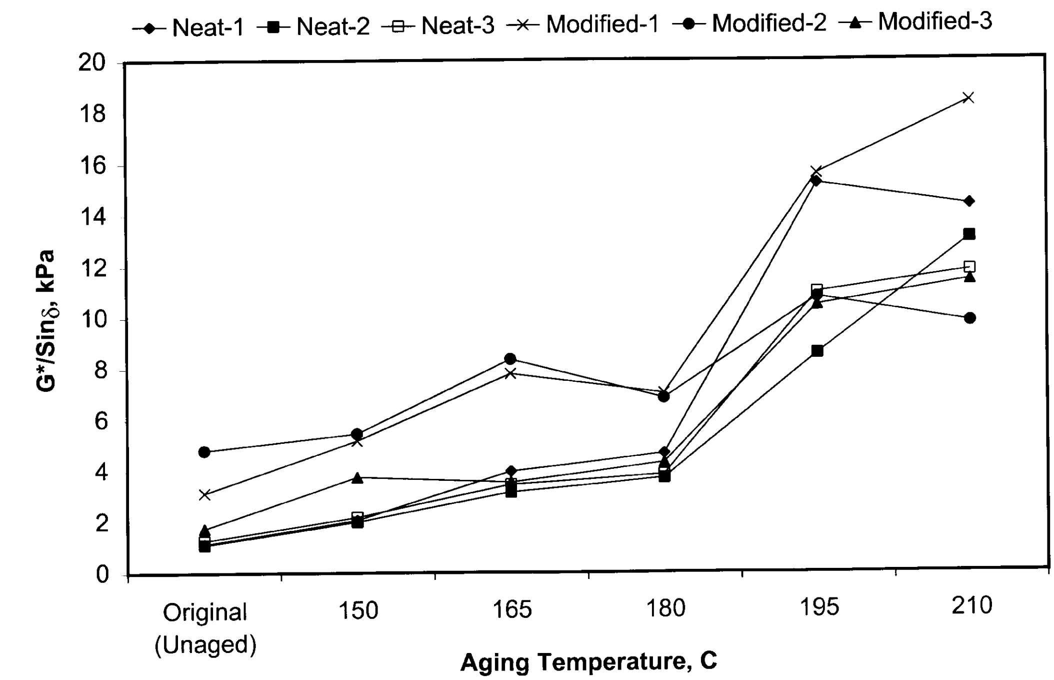


Figure 4.37: Results of DSR Testing to Evaluate the Potential Damage to Asphalt Cement at High Temperatures

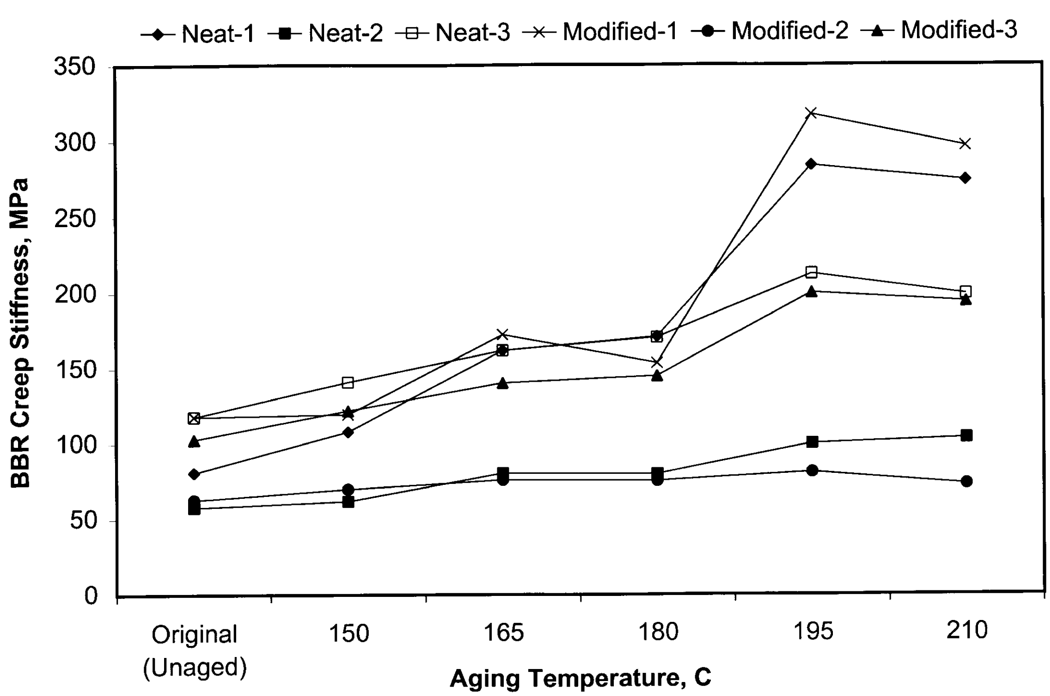


Figure 4.38: Results of BBR Creep Stiffness to Evaluate the Potential Damage to Asphalt Cement at High Temperatures

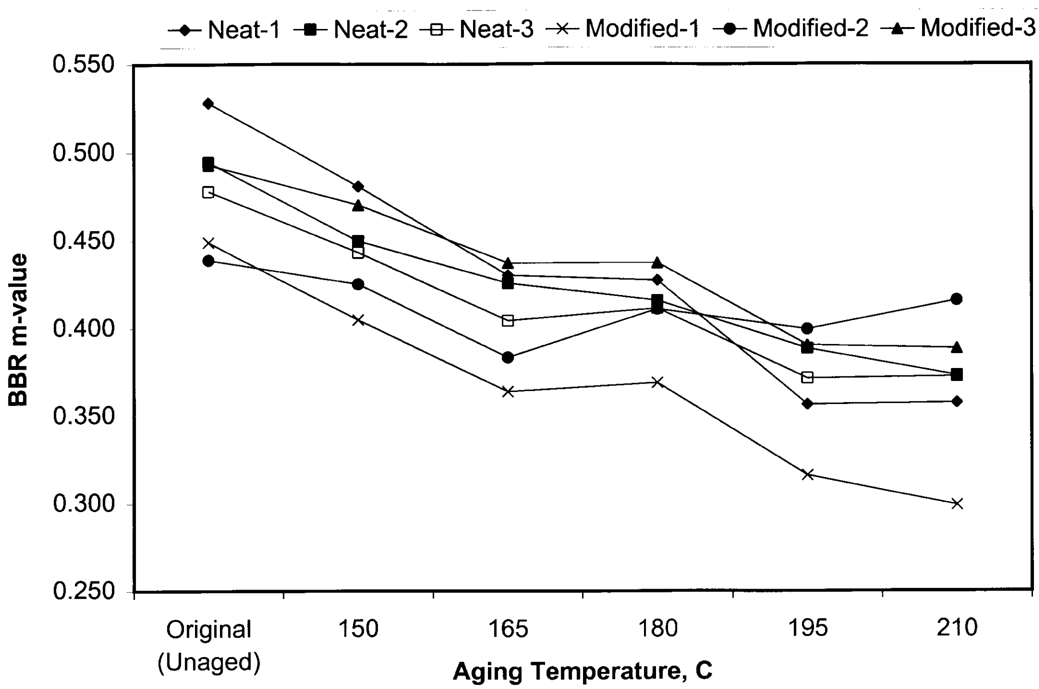


Figure 4.39: Results of BBR m-value to Evaluate the Potential Damage to Asphalt Cement at High Temperatures

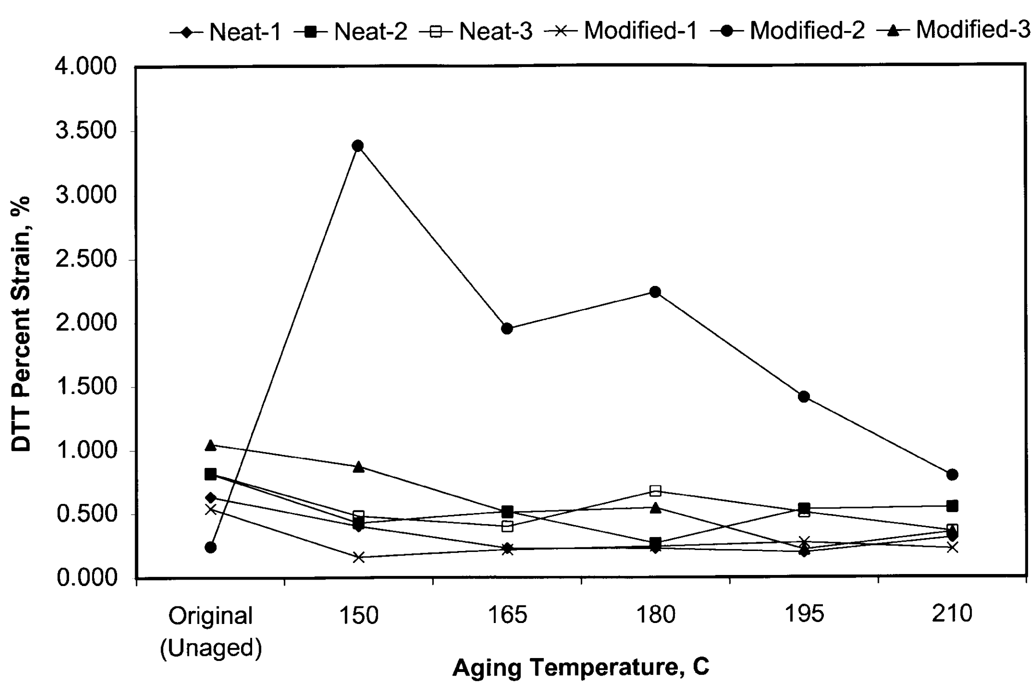


Figure 4.40: Results of DTT Testing to Evaluate the Potential Damage to Asphalt Cement at High Temperatures

properties. This was accomplished to determine whether a particular aging temperature provided a break point below which no damage occurred. Results of these analyses are presented in Tables 4.48 through 4.51.

Table 4.48 presents the results of DMRT ranking of the G*/Sinδ testing. These rankings show that a break point between 180 and 195°C seems to exist. Five of the six asphalt binders have the same ranking up to 180°C. At 195°C, the ranking changed for 5 of the binders which indicates that the test results are significantly different. Figure 4.37 illustrates this break point between 180 and 195°C. The figure also shows the expected trend of increasing stiffness with increasing aging temperatures. Also, it appears that a distinction in the effects on high temperature properties of modified versus neat asphalts
can not be made. The asphalts seem to react differently to the increasing aging temperatures.

Table 4.48: Results of DMRT Ranking on DSR Testing Performed to Evaluate Potential Damage to Asphalt Cement at High Temperatures

Asphalt Binder	Oven Temperatures (DMRT Ranking [1])				
	150°C	165°C	180°C	195°C	210°C
Neat AC No. 1	B	B	B	A	A
Neat AC No. 2	C	C	C	B	A
Neat AC No. 3	B	B	B	A	A
Modified AC No. 1	B	B	B	A	A
Modified AC No. 2	B	AB	AB	A	AB
Modified AC No. 3	B	B	B	A	A

[1] Means with the same letter ranking are not significantly different.

A distinctive break point between 180 and 195°C also was exhibited in the BBR creep stiffness DMRT rankings (Table 4.49). For each of the six binders except Modified-2, there existed significant differences between the 180 and 195°C data. This is illustrated in Figure 4.38. As would be expected, this figure shows that the stiffness of the different binders increased as the aging temperatures increased. Results of the DMRT rankings for the BBR m-values are presented in Table 4.50. Again, a break point between 180 and 195 °C seems to exist as five of the six binders exhibited significant differences between these two temperatures. There was a break point between 150 and 165°C. The BBR m-values, as shown in Figure 4.39, decreased with increasing aging temperatures. This decrease in m-values was as expected since the asphalt stiffened due to oxidation caused by the increasing temperatures. Again, the data does not indicate that a difference occurs between the modified versus neat asphalts for either the creep stiffness or m-values.

Table 4.51 presents the DMRT rankings for the DTT test results. Based on this table, it is unclear if the aging temperatures affected the results of this test. Four of the six binders did not show significant differences in the test results. Figure 4.40 presents the results of the DTT testing. Based on this figure, it appears that for all of the asphalts except Modified-2, the effects of higher aging temperatures are minimal. It is unclear why the Modified-2 asphalt behaved in the manner that it did.

Overall, the data supports the conclusion that the properties of all the asphalts tested

Table 4.49: Results of DMRT Ranking on BBR Creep Stiffness Testing Performed to Evaluate Potential Damage to Asphalt Cement at High Temperatures

Asphalt Binder	Oven Temperatures (DMRT Ranking [1])				
	150°C	165°C	180°C	195°C	210°C
Neat AC No. 1	B	B	B	A	A
Neat AC No. 2	C	B	B	A	A
Neat AC No. 3	C	C	BC	A	AB
Modified AC No. 1	C	B	B	A	A
Modified AC No. 2	A	A	A	A	A
Modified AC No. 3	B	B	B	A	A

[1] Means with the same letter ranking are not significantly different.

Table 4.50: Results of DMRT Ranking on BBR m-value Testing Performed to Evaluate Potential Damage to Asphalt Cement at High Temperatures

Asphalt Binder	Oven Temperatures (DMRT Ranking [1])				
	150°C	165°C	180°C	195°C	210°C
Neat AC No. 1	A	B	B	C	C
Neat AC No. 2	A	B	B	C	D
Neat AC No. 3	A	B	B	C	C
Modified AC No. 1	A	B	B	C	D
Modified AC No. 2	A	B	AB	AB	AB
Modified AC No. 3	A	B	B	C	C

[1] Means with the same letter ranking are not significantly different.

Table 4.51: Results of DMRT Ranking on DTT Testing Performed to Evaluate Potential Damage to Asphalt Cement at High Temperatures

Asphalt Binder	Oven Temperatures (DMRT Ranking [1])				
	150°C	165°C	180°C	195°C	210°C
Neat AC No. 1	A	B	B	B	AB
Neat AC No. 2	A	A	A	A	A
Neat AC No. 3	A	A	A	A	A
Modified AC No. 1	A	A	A	A	A
Modified AC No. 2	A	BC	AB	BC	C
Modified AC No. 3	A	A	A	A	A

[1] Means with the same letter ranking are not significantly different.

changed as a result of aging within the forced draft oven. This was shown by the increasing G*/Sinδ, increasing BBR stiffness, and decreasing BBR m-value and DTT strain at failure. The data also suggests that an aging temperature somewhere between 180 and 195°C seems to cause significant changes in the properties of the different asphalt binders.

4.5 LABORATORY 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 based on viscosity ranges. Because of the high filler contents and the frequent use of polymer modified asphalt cements, this method of determining the mixing and compaction temperatures may not be the best. Therefore, two different methods were evaluated to determine if they provided more usable results. The first method entailed combining the material finer than 0.075 mm from the design gradation and the asphalt binders (no fibers included) for each of the eleven projects visited under Task 8. These mortars were then tested in the Brookfield viscometer at the mixing temperatures used in the field, ± 11.1°C (± 20°F), and ± 22.2°C (± 40°F). The second test was a workability device that was developed to measure the stiffness of SMA mixtures. Using this device at different temperatures allowed the stiffness of a mixture in relation to temperature to be determined. This information was then evaluated to see if it could be related to mixing and compaction temperatures. The following sections describe the test results obtained using these two methods to obtain mixing and compaction temperatures.

4.5.1 Brookfield Viscometer Testing

Using materials obtained from each of the projects visited, filler-asphalt mortars were fabricated in the laboratory for testing in the Brookfield viscometer (BV). Fibers were not included in this testing. The overall objective of this testing was to establish a critical filler-asphalt viscosity at which a mixing temperature could be established.

Table 4.52 and Figures 4.41 through 4.43 present the results of the BV testing performed on the filler-asphalt mortars. Figure 4.41 illustrates the results of testing on the PG 76-XX mortars. This figure shows that there was much variation in the test results at the lower temperatures. Mortars created from the materials from sites 4 and 9 yielded BV test results that were significantly higher than the other three sites. Referring back to Table 4.18, sites 4 and 9 did have the highest G*/Sinδ values in both the unaged and aged conditions (neglecting the suspect values for sites 3 and 6). These two mortars were the only two mortars with unaged values above 10.0 kPa. This explains why these two mortars had higher viscosities than did the remaining three. For sites 1, 2, and 10 it appears that a much lower viscosity would be appropriate.

Figure 4.42 illustrates the results of testing on the PG 64-XX mortars. Test results for these four sites show less variation than did the PG 76 mortars.

Figure 4.43 illustrates the BV measurements on the remaining two mortars. Both of these sites were the only ones using the respective asphalt binder PG grades and were therefore included on one figure. Site 5 used an unmodified PG 70 while site 11 utilized an unmodified PG 58.

Table 4.52: Results of Brookfield Viscosity Testing on Site Mortars (Minus Fibers)

Site	Neat Asphalt Mixing Temp. °C	Temperature, °C and Brookfield Viscosity, cP				
1	168	157.2	168.3	179.4	190.6	201.7
		8350	5125	3475	2475	1800
2	163	140.6	151.7	162.8	173.9	185.0
		8650	5050	3175	2100	1475
3	155	133.3	144.4	155.6	166.7	177.8
		6150	3625	2275	1525	1175
4	171	148.9	160.0	171.1	182.2	193.3
		47150	16775	9525	5875	4250
5	166	138.3	149.4	160.6	171.7	182.8
		2875	1875	1075	675	475
6	171	148.9	160.0	171.1	182.2	193.3
		2600	1500	1125	875	725
7	168	148.9	160.0	171.1	182.2	193.3
		2200	1525	975	700	525
8	187	165.6	176.7	187.8	198.9	210
		2350	1700	1350	1087	875
9	156	137.8	148.9	160.0	171.1	182.2
		35000	20450	10475	6675	4425
10	NA	140.6	151.7	162.8	173.9	185.0
		4250	2325	1375	875	575
11	157	135.0	146.1	157.2	168.3	179.4
		3250	2050	1200	875	575

NA - Not Available

The Brookfield viscometer data is compared to the actual mixing temperature and the suggested mixing temperature (in accordance with AASHTO T245) in Table 4.53. The viscosity from the Brookfield Test that correlates to the mid-range of the actual mixing temperatures is provided. The data shows that for most mixes the mortar viscosity should be approximately 2,000 cP for suitable mixing. This would be reasonably close to the desired viscosity for most of the mixes but would have significant error for 2 to 3 of the mortars. The AASHTO T245 approach clearly provides the best estimate of mixing temperature for the data shown.

Based on test results with the Brookfield viscometer, it appears it has potential to be used to help select mixing temperatures. However, the present method presented in AASHTO T245 seems to do a better job of selecting the mixing temperature.

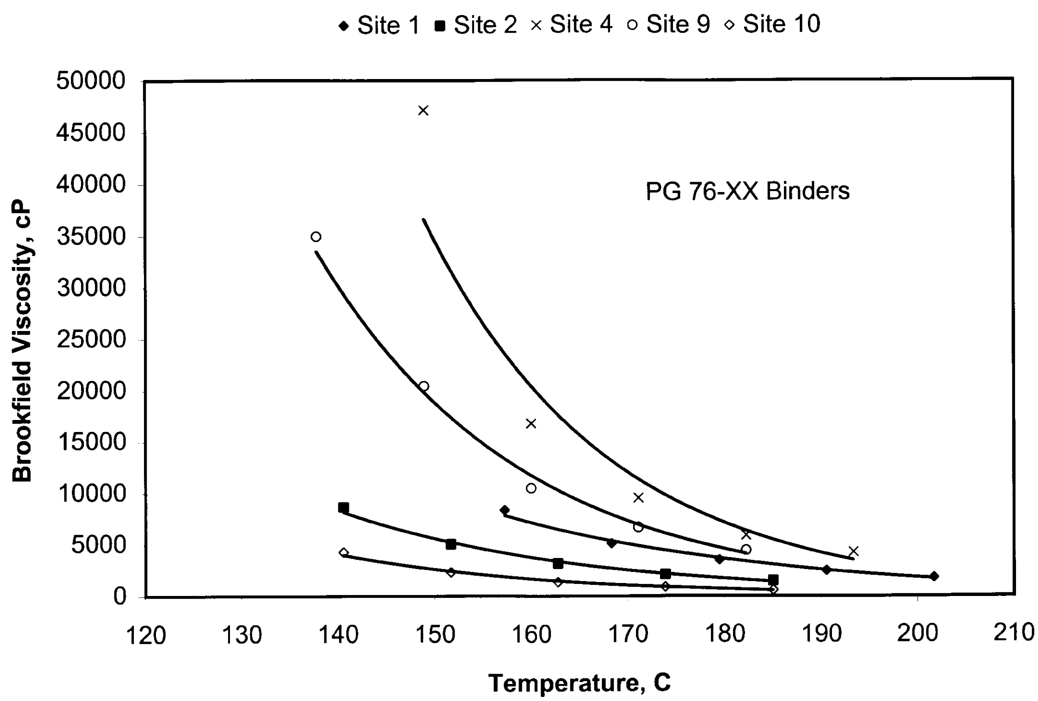


Figure 4.41: Results of Brookfield Viscosity Testing on PG 76-XX Project Fine Mortars

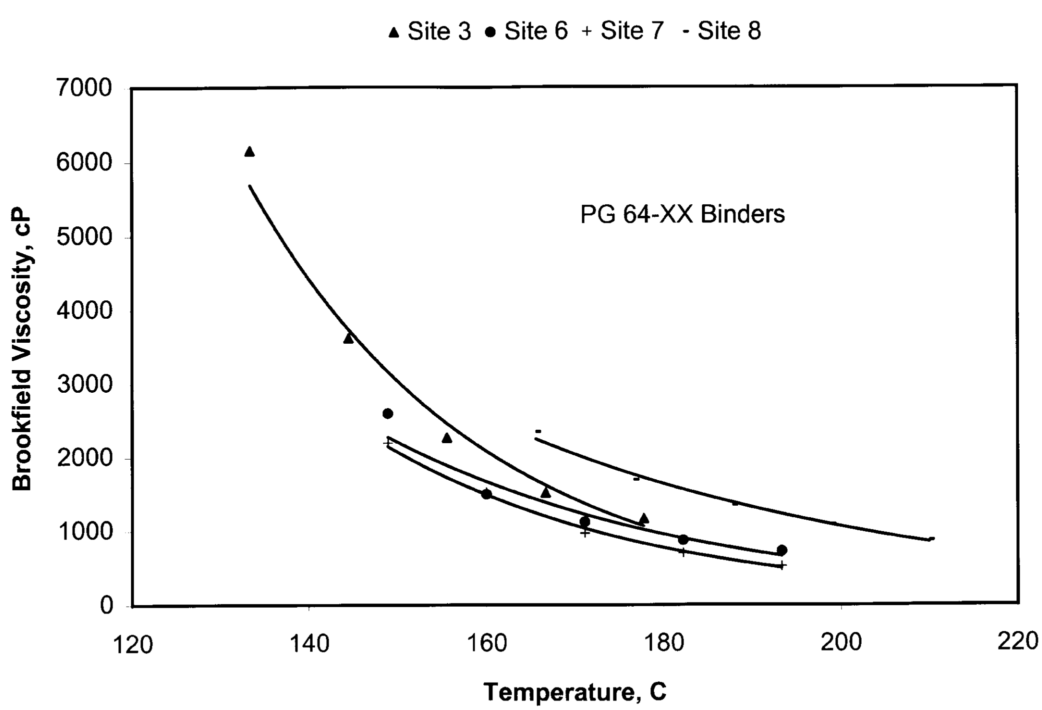


Figure 4.42: Results of Brookfield Viscosity Testing on PG 64-XX Project Fine Mortars

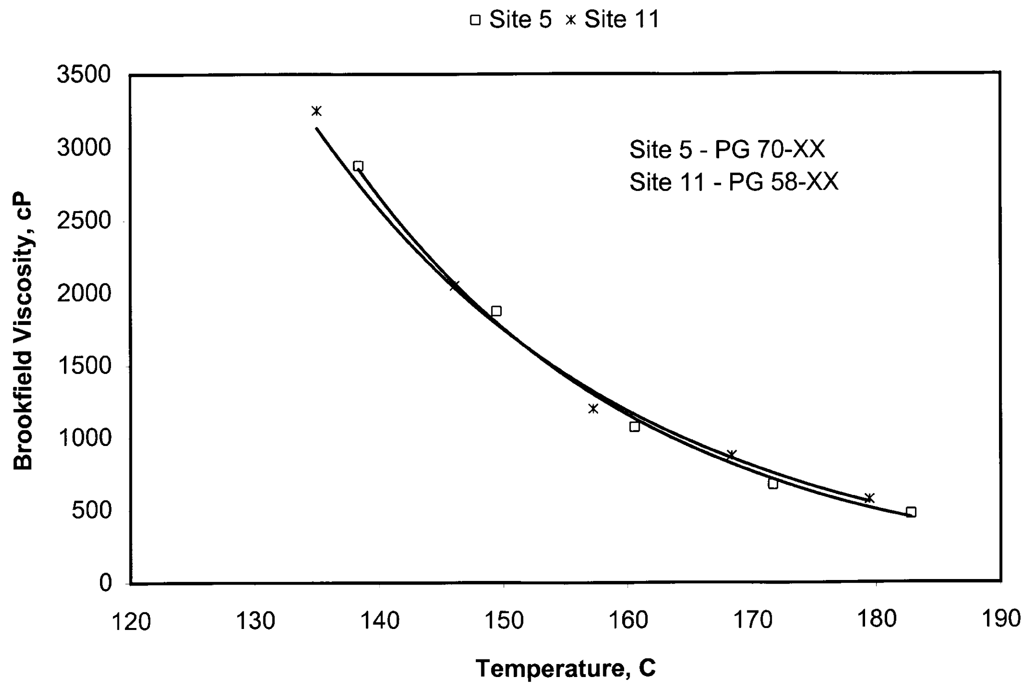


Figure 4.43: Results of Brookfield Viscosity Testing on PG 70-XX and PG 58-XX Project Fine Mortars

Table 4.53: Comparison of Mixing Temperatures and Viscosity

Site Number	AASHTO T245 Mixing Temperature, °C	Actual Field Mixing Temperature, °C	Brookfield Viscometer Measurement at Mixing Temperature, cP
1	168	176 to 182	3000
2	163	163 to 171	2500
3	155	157 to 166	2000
4	171	163 to 182	9400
5	166	157 to 163	1200
6	171	151 to 160	2000
7	168	174 to 188	1000
8	187	177 to 188	1500
9	156	174 to 188	5000
10	NA	154 to 171	1500
11	157	143 to 149	2000

NA - Not Available

4.5.2 Workability Testing

Another approach that was evaluated to help establish the mixing and compaction temperatures was a workability test. This test consisted of placing SMA mix into a bucket and turning a paddle through the mixture. The torque required to turn the paddle was measured at various temperatures. It was hopeful that this test would help to select the mixing and compaction temperatures and it was also desirable to quantify the stiffness of the mixture at various temperatures.

Development of the test was on-going during the evaluation of the eleven construction projects. Therefore, the data collected during the construction projects had a lot of scatter and due to testing problems the data was not collected during construction of all projects. However, samples of SMA mix were brought back to the laboratory and tested after the test procedure had been finalized. Data was collected for all projects except project number 9. There was insufficient material available from the project for testing.

The workability was measured in the laboratory at the field mixing temperatures and below. The mix was heated to the respective field mixing temperatures and tested. The mix was then allowed to cool to various temperature levels and tested again. The data is shown in Tables 4.54 through 4.65. The workability data did not compare well with the Brookfield viscometer data (Figure 4.44). For this plot, the viscosity at 154 °C was interpolated for the Brookfield viscometer.

A plot of the workability (torque) for the mixes at various temperatures is shown in Figure 4.45. This data, as expected, clearly shows that the torque required to turn the paddle increases as the mix temperature decreases. The test does a good job of showing stiffness of a particular mix in comparison to other mixes at any desired temperature. However, the data does not provide information about the best temperature for mixing and compacting. The device has some promise to do this but additional work is needed. This additional work is beyond the scope of this project.

Until a better method is developed, the best approach for determining the mixing and compaction temperatures is to use AASHTO T245 for neat asphalts and to seek guidance from manufacturers when modifiers are used.

Table 4.54: Results of Workability Testing Conducted at NCAT Laboratory for Site 1

Replicate 1		Replicate 2	
Temperature (F)	Average Torque (ft-lbs)	Temperature (F)	Average Torque (ft-lbs)
350	25	350	20
340	27	340	30
320	37	325	32
310	40	315	37
300	42	290	38

Table 4.55: Results of Workability Testing Conducted at NCAT Laboratory for Site 2

Replicate 1		Replicate 2	
Temperature (F)	Average Torque (ft-lbs)	Temperature (F)	Average Torque (ft-lbs)
340	20	340	20
320	27	330	25
310	30	310	32
290	35	300	37
280	35	290	32

Table 4.56: Results of Workability Testing Conducted at NCAT Laboratory for Site 3

Replicate 1		Replicate 2	
Temperature (F)	Average Torque (ft-lbs)	Temperature (F)	Average Torque (ft-lbs)
330	22	330	25
325	32	320	35
325	30	315	38
310	38	300	37
300	40	270	42
270	47		

Table 4.57: Results of Workability Testing Conducted at NCAT Laboratory for Site 4

Replicate 1		Replicate 2	
Temperature (F)	Average Torque (ft-lbs)	Temperature (F)	Average Torque (ft-lbs)
350	30	350	30
340	48	340	33
325	53	330	50
310	50	315	40
300	53	300	67

Table 4.58: Results of Workability Testing Conducted at NCAT Laboratory for Site 5

Replicate 1		Replicate 2	
Temperature (F)	Average Torque (ft-lbs)	Temperature (F)	Average Torque (ft-lbs)
325	25	325	20
320	25	320	27
310	27	305	28
300	30	290	30
290	28	285	30
270	30	265	37

Table 4.59: Results of Workability Testing Conducted at NCAT Laboratory for Site 6

Replicate 1		Replicate 2	
Temperature (F)	Average Torque (ft-lbs)	Temperature (F)	Average Torque (ft-lbs)
325	30	325	30
310	40	315	40
300	42	305	40
290	47	295	40
275	53	275	48

Table 4.60: Results of Workability Testing Conducted at NCAT Laboratory for Site 7

Replicate 1		Replicate 2	
Temperature (F)	Average Torque (ft-lbs)	Temperature (F)	Average Torque (ft-lbs)
360	25	360	25
350	37	350	30
335	35	330	35
310	38	310	40

Table 4.61: Results of Workability Testing Conducted at NCAT Laboratory for Site 8

Replicate 1		Replicate 2	
Temperature (F)	Average Torque (ft-lbs)	Temperature (F)	Average Torque (ft-lbs)
360	30	360	30
350	40	350	37
340	40	340	40
330	48	325	48
310	48	310	50

Table 4.62: Results of Workability Testing Conducted On-Site for Site 9

Replicate 1		Replicate 2	
Temperature (F)	Average Torque (ft-lbs)	Temperature (F)	Average Torque (ft-lbs)
355	30	370	30
340	33	330	43
320	40	310	45
310	45	300	55
Replicate 3		Replicate 4	
360	25	340	33
310	40	320	43
280	45	300	45
		280	59
Replicate 5			
340	33		
320	40		
315	48		
300	55		

Table 4.63: Results of Workability Testing Conducted at NCAT Laboratory for Site 10

Replicate 1		Replicate 2	
Temperature (F)	Average Torque (ft-lbs)	Temperature (F)	Average Torque (ft-lbs)
340	23	340	20
330	29	330	27
310	29	310	32
305	34	300	30
290	37	290	37

Table 4.64: Results of Workability Testing Conducted On-Site for Site 10

Replicate 1		Replicate 2	
Temperature (F)	Average Torque (ft-lbs)	Temperature (F)	Average Torque (ft-lbs)
320	30	325	27
300	30	315	30
280	40	300	38
260	51	285	43
Replicate 3		Replicate 4	
320	27	325	23
305	33	305	30
290	40	285	30
270	55		
Replicate 5		Replicate 6	
305	31	315	27
290	33	300	33
275	38	280	35

Table 4.65: Results of Workability Testing Conducted at NCAT Laboratory for Site 11

Replicate 1		Replicate 2	
Temperature (F)	Average Torque (ft-lbs)	Temperature (F)	Average Torque (ft-lbs)
310	19	310	20
300	21	300	25
285	25	290	25
273	26	280	23
260	29	270	27
		260	28
		250	31
		240	35

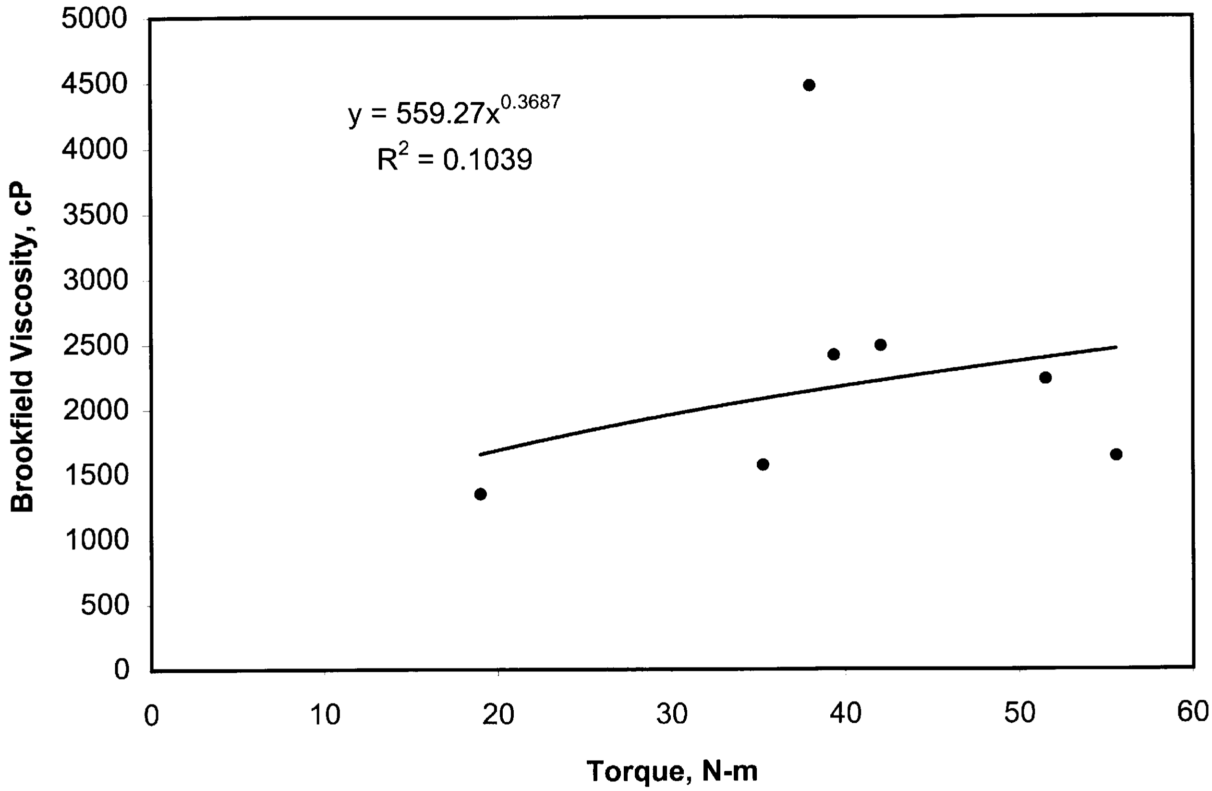


Figure 4.44: Comparison of Workability and Brookfield Viscosity Testing at 154°C

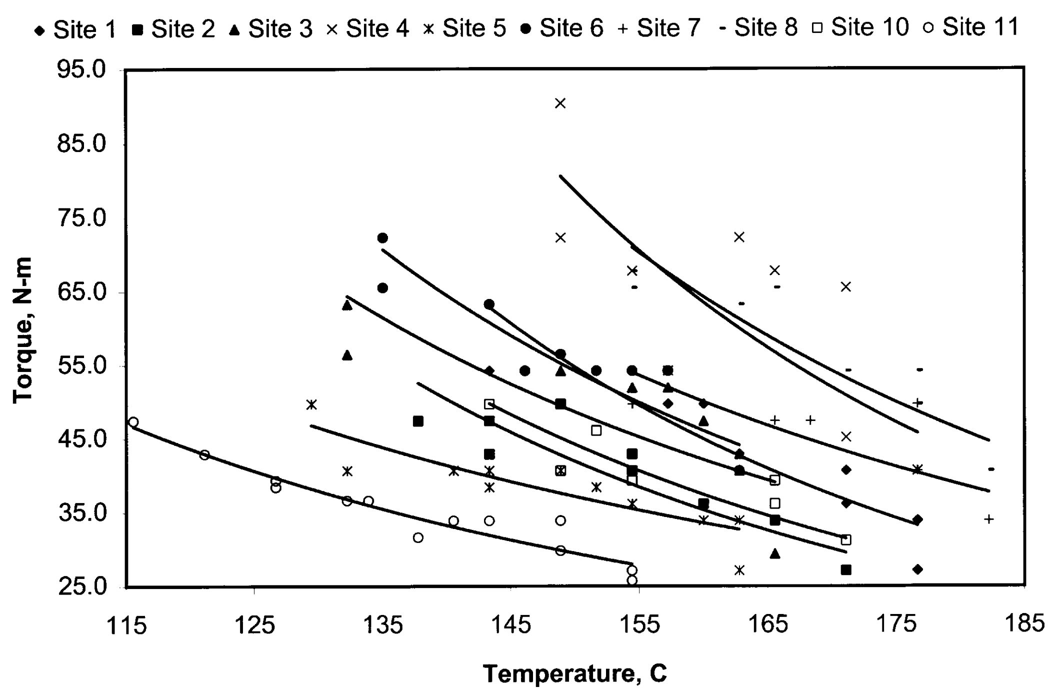


Figure 4.45: Results of All Field Workability Testing

4.6 DENSITY REQUIREMENTS

Task 13 was intended to provide guidance for the verification of field density of SMA mixes. This was to be accomplished using permeability testing. The thought was that SMA mixes should be compacted in place to a density level that produced an impermeable pavement. It has been postulated that SMA mixes become permeable to water at approximately six percent air voids. Support for this theory was accumulated in Phase I of this study.

For Phase II, ten core samples were taken from eight of the Task 8 field sites described previously in this report. These samples were tested and compared to the in-situ measure of permeability obtained in the field. Laboratory tests were performed in accordance with the permeability procedure outlined in Volume V on cores obtained from the projects. The in-situ permeability was determined using a falling head method that consisted of sealing a plastic graduated cylinder to the pavement. Silicone caulk was used as a sealant.

The in-situ and laboratory permeability values are shown in Figures 4.46 through 4.53 for sites 2, 3, 4, 5, 6, 9, 10, and 11 respectively. Considerable variability is exhibited in most of the graphs. This was most likely due to a number of factors. Among these, for the in-situ measurements, the SMA layer was probably not fully saturated prior to testing. This is important for determining the actual amount of flow through the medium. However, this could not be avoided given the practical incapability of physically being able to saturate the entire pavement layer. Also, in the field flow was unbounded and thus was permitted in any direction. In some of the tests, some water was observed coming to the surface of the pavement in the vicinity of the permeameter while in other tests this did not occur. The variability associated with the laboratory specimens was primarily due to sample preparation. Sawing to separate the pavement layers tended to seal the undersides of the specimens somewhat resulting in some specimens being less permeable than others for a given site. In particular, Site 2 was determined to be impermeable (i.e. no appreciable flow after 20-30 minutes in the lab permeameter).

In addition to the variability, the in-situ results tended to be relatively constant as shown on the semi-log graphs. In each case, with the exception of the Site 2 results shown in Figure 4.46, the in-place field data do not indicate a general increase in permeability with decreasing mat density. As a result, the validity of the in-situ results is certainly questionable.

Increasing permeability with decreasing core density (or increasing core air voids) was generally observed in the laboratory tests. Site 2 was an exception where the laboratory permeability could not be determined due to sawing effects. Figure 4.46 shows a flat line on the horizontal axis to indicate this. Otherwise, the results from the other sites showed the increasing permeability for increasing air voids.

Saturation in the laboratory was achieved by pulling a vacuum on each core using the AASHTO T209 procedure. Using the Florida DOT laboratory test based limiting k-value again, the in-situ results indicate that the SMA mixtures become permeable to water between in place air voids of 5.0 and 6.5 percent. For example, site 3 required less than 6.3 percent air voids, site 4 less than 7.5 percent, site 5 less than 7 percent, site 9 less than 6.3 percent, site 10 less than 5.5 percent, and site 11 less than 5.3 percent. This confirms the notion that SMA requires higher densities than do dense-graded mixtures in the field.

Major changes must be made in the field test for it to be acceptable for use.

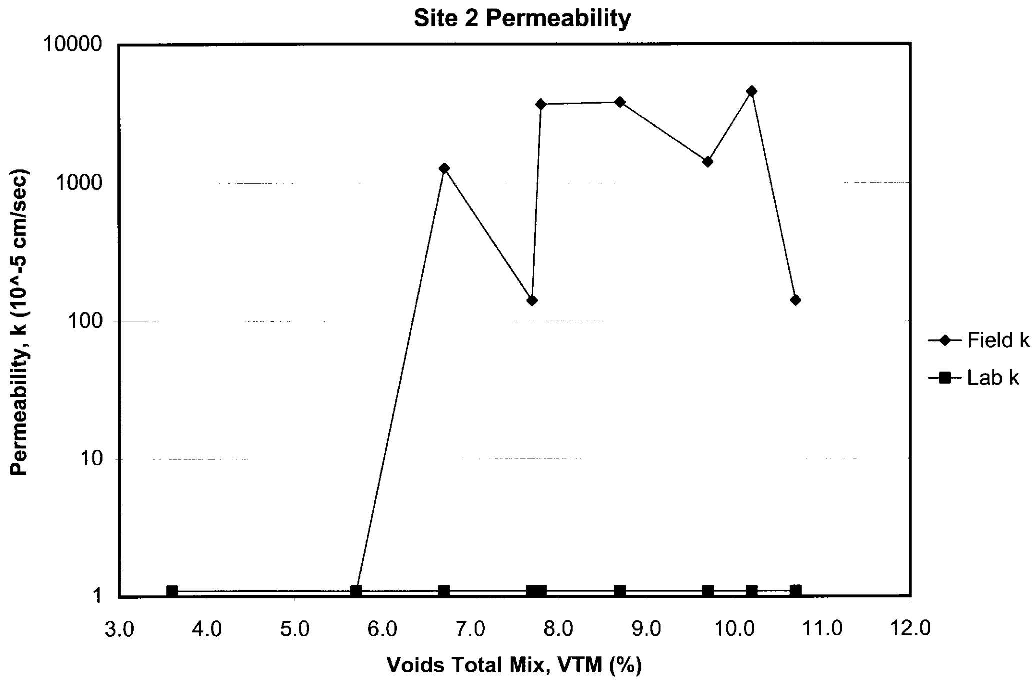


Figure 4.46: Field and Laboratory Permeability for Site 2

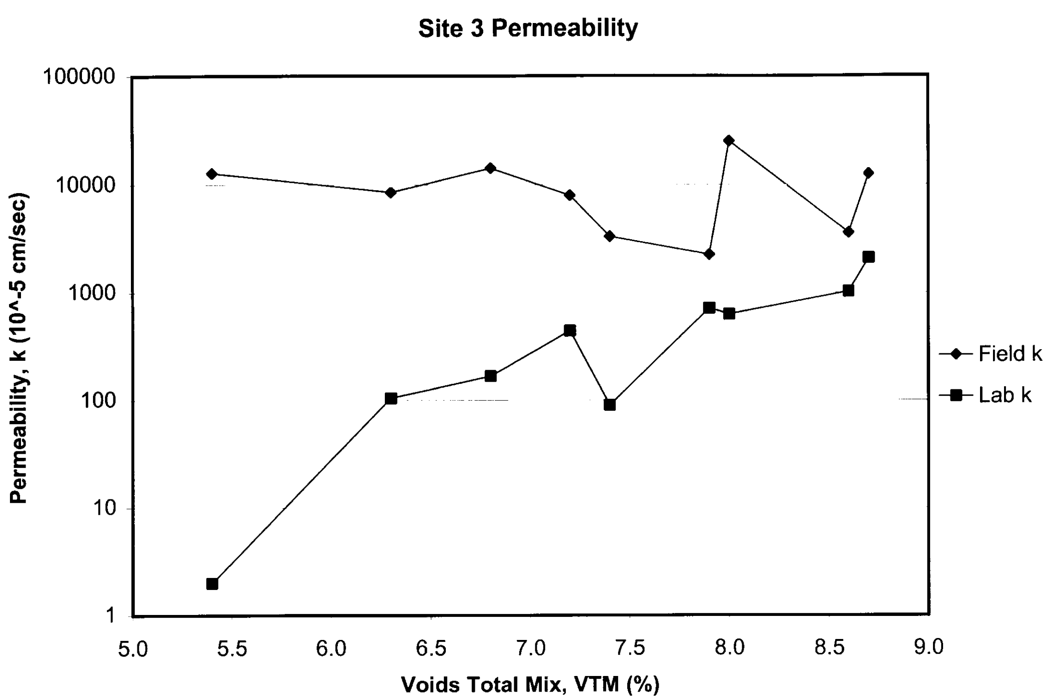


Figure 4.47: Field and Laboratory Permeability for Site 3

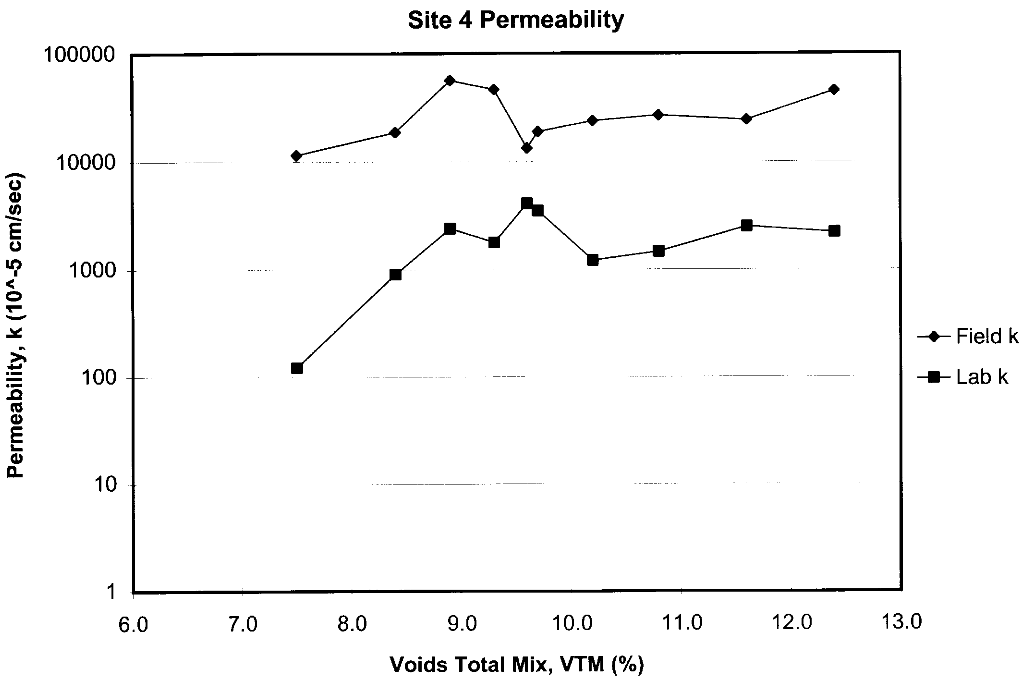


Figure 4.48: Field and Laboratory Permeability for Site 4

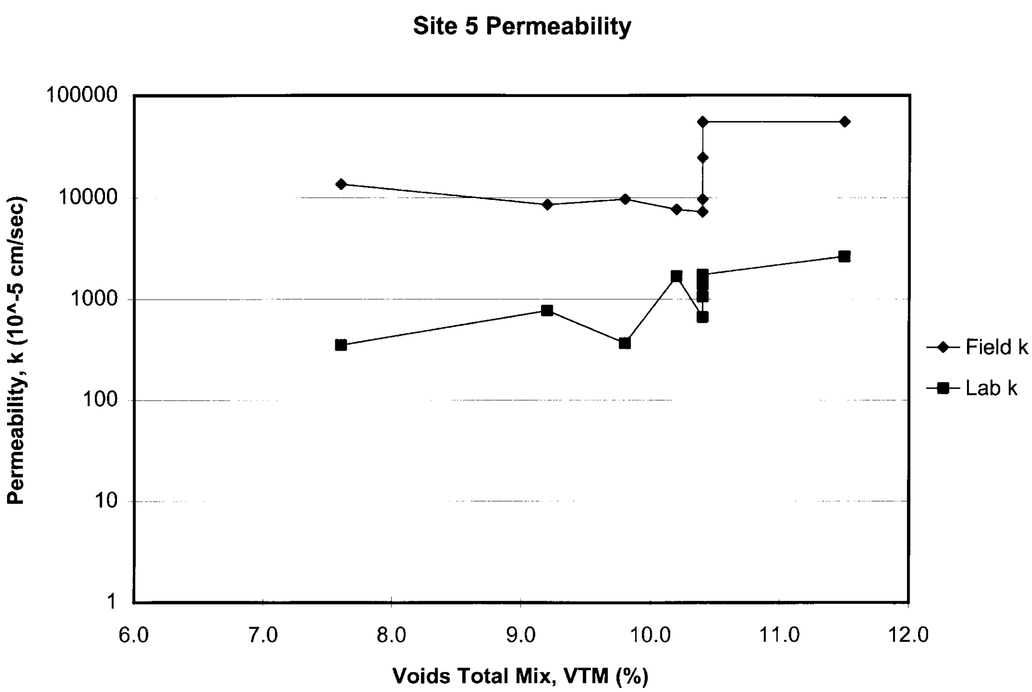


Figure 4.49: Field and Laboratory Permeability for Site 5

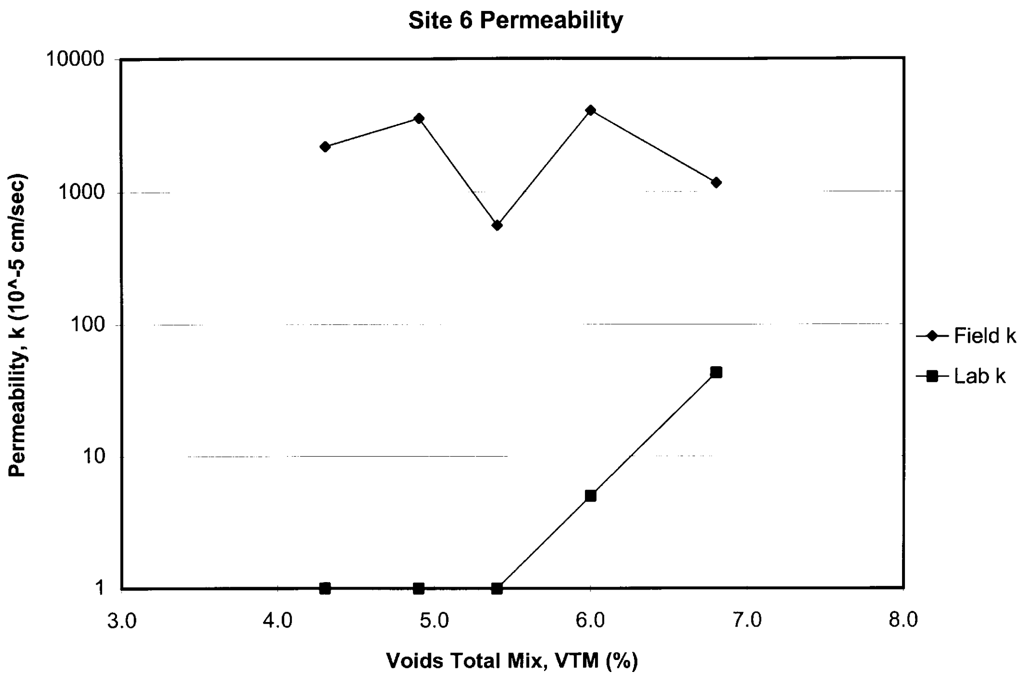


Figure 4.50: Field and Laboratory Permeability for Site 6

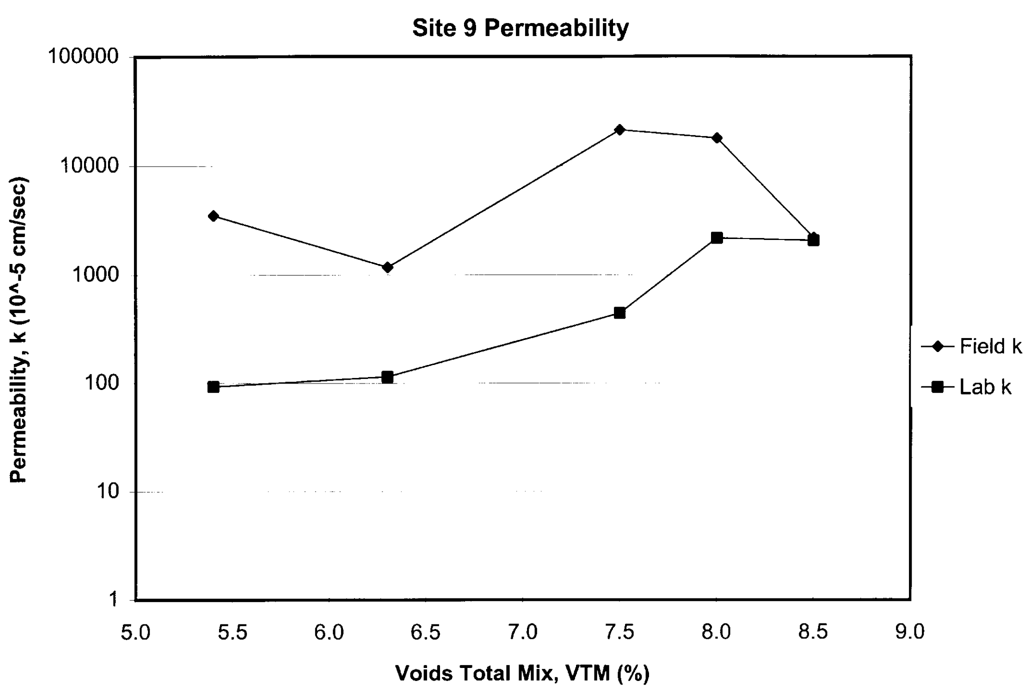


Figure 4.51: Field and Laboratory Permeability for Site 9

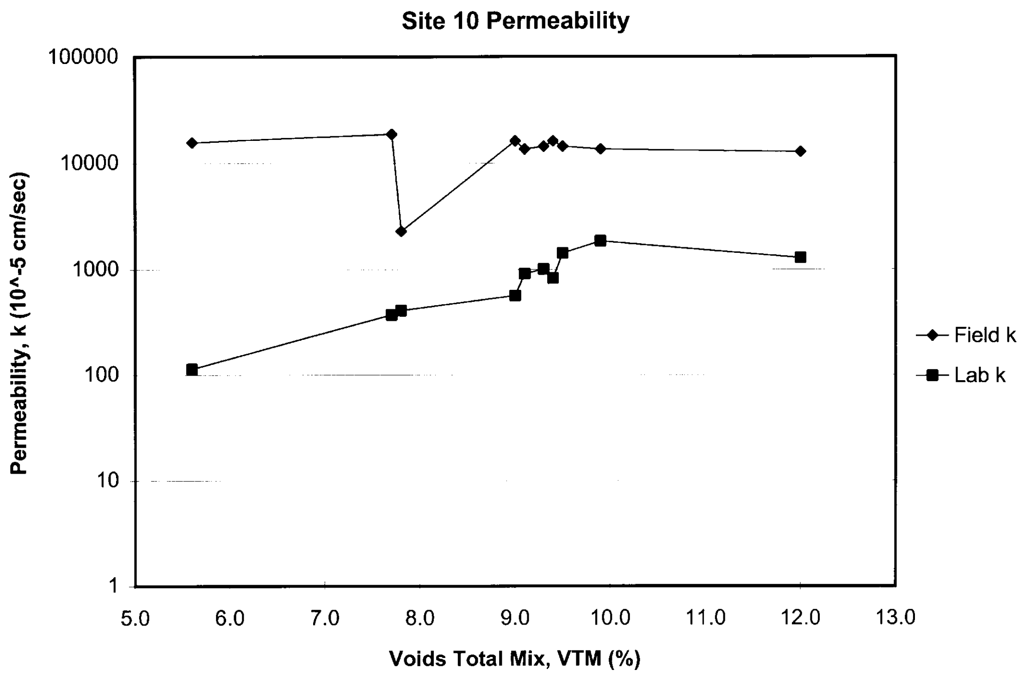


Figure 4.52: Field and Laboratory Permeability for Site 10

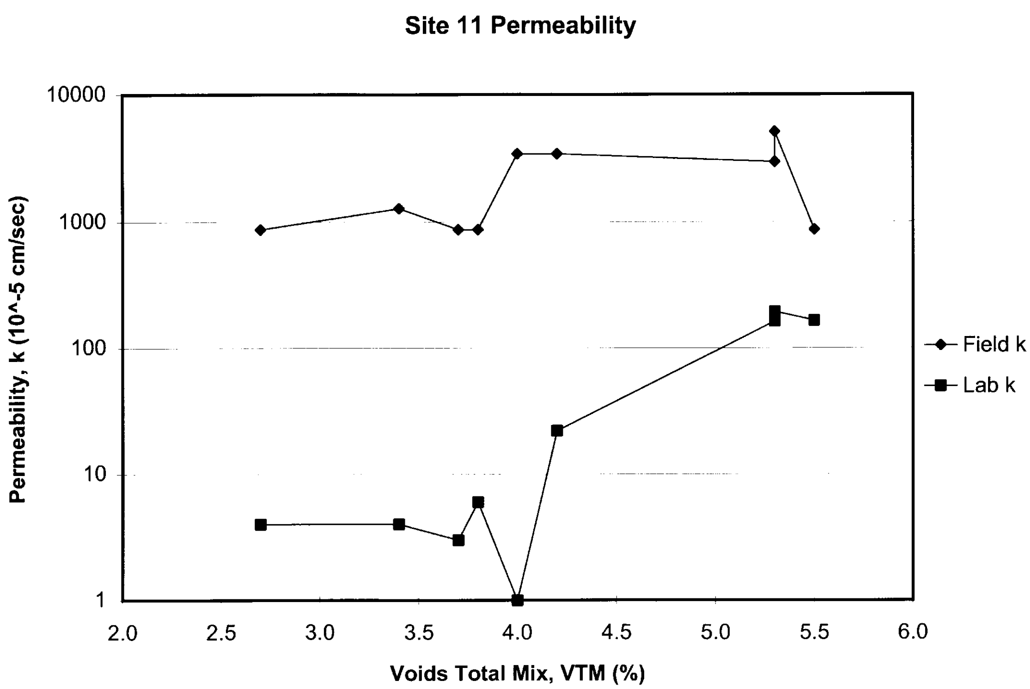


Figure 4.53: Field and Laboratory Permeability for Site 11

4.7 ACCURACY AND PRECISION OF NUCLEAR DENSITY 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 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. As stated previously, at one of the eight projects only five cores were obtained while at other projects 10 were taken. 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. For each of the eight sites, the nuclear density gauge measurements (both with and without sand) were subtracted from the core densities to establish individual bias values. These individual bias values were then averaged (both with and without sand) to produce the correction factor. The premise of this analysis was that the final corrected nuclear gauge density would be equal to the core density. Results of this analysis are presented in tabular form in Tables 4.66 through 4.73 and illustrated in Figures 4.54 through 4.61.

The second method of evaluating the data consisted of determining a dynamic correction factor. For each of the eight sites, the core densities were divided by the nuclear density measurements (both with and without sand) to produce individual density ratios. Next, the individual density ratios (both with and without sand) were plotted versus the nuclear density gauge measurements and a simple linear regression performed (separate regression analyses were performed for the with and without sand data). An example of this type of plot is presented for site 3 in Figure 4.62. Based on the regression line, a correction factor can then be determined for any individual nuclear gauge reading. Multiplying the correction factor and the nuclear density gauge measurements results in a corrected nuclear gauge reading. Unlike the average bias correction factor, the premise of this analysis is that a linear correlation exists between the nuclear density readings and the core densities. Results of this analysis are presented in Tables 4.74 through 4.81 and illustrated in Figures 4.63 through 4.70.

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 sand and without sand readings. Using the method of static correction factors (average bias analysis), the standard error of the estimate was based on the line of equality since 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 predicted core densities (determined from the regression lines in Figures 4.63 through 4.70) and the core densities. Table 4.82 presents the standard errors of the estimate for both methods and for both types of nuclear density gauge readings.

Table 4.82 shows that the dynamic correction factors provide a better estimate of true densities. The lower the standard errors of the estimate, the closer the nuclear gauge comes to producing the actual density. Standard errors of the estimate ranged from 0.61 to 2.75 lbs per cu ft for the dynamic correction factors. For illustrative purposes, if a SMA pavement has a G_{mm} of 150 lbs per cu ft, based on these standard errors of the estimate, the nuclear gauge could be

Table 4.66: Results of Average Bias Analysis for Accuracy and Precision of Nuclear Density Gauge
Site No.: __2__

Core Number	Core Density, pcf	Nuclear Densities		Ind. Bias Values		Corrected Nuclear Densities		Residuals	
		With Sand, pcf	Without Sand, pcf	With Sand	Without Sand	With Sand	Without Sand	With Sand	Without Sand
1	142.6	138.5	139.6	4.1	3.0	142.7	143.8	0.1	1.2
2	137.7	138.7	142.3	-1.0	-4.6	142.9	146.5	5.2	8.8
3	136.4	134.2	131.8	2.2	4.6	138.4	136.0	2.0	-0.4
4	138.0	133.8	134.2	4.2	3.8	138.0	138.4	0.0	0.4
5	137.2	133.7	133.6	3.5	3.6	137.9	137.8	0.7	0.6
6	138.2	133.9	133.9	4.3	4.3	138.1	138.1	-0.1	-0.1
7	139.7	134.6	133.5	5.1	6.2	138.8	137.7	-0.9	-2.0
8	139.6	132.3	131.6	7.3	8.0	136.5	135.8	-3.1	-3.8
9	145.9	140.0	138.8	5.9	7.1	144.2	143.0	-1.7	-2.9
10	142.8	136.1	136.4	6.7	6.4	140.6	140.6	-2.5	-2.2
			Average Bias:	4.23	4.24		Standard Err. of the Est.:	6.76	10.35

Table 4.67: Results of Average Bias Analysis for Accuracy and Precision of Nuclear Density Gauge
Site No.: __3__

Core Number	Core Density,Pcf	Nuclear Densities		Ind. Bias Values		Corrected Nuclear Densities		Residuals	
		With Sand, pcf	Without Sand, pcf	With Sand	Without Sand	With Sand	Without Sand	With Sand	Without Sand
1	127.8	126.4	124.3	1.4	3.5	129.2	128.4	1.4	0.6
2	126.6	126.5	121.4	0.1	5.2	129.3	125.5	2.7	-1.1
3	125.6	123.5	121.3	2.1	4.3	126.3	125.4	0.7	-0.2
4	127.1	125.5	121.1	1.6	6.0	128.3	125.2	1.2	-1.9
5	129.0	127.5	125.2	1.5	3.8	130.3	129.3	1.3	0.3
6	125.5	121.1	121.2	4.4	4.3	123.9	125.3	-1.6	-0.2
7	124.7	121.1	123.6	3.6	1.1	123.9	127.7	-0.8	3.0
8	124.5	118.8	119.6	5.7	4.9	121.6	123.7	-2.9	-0.8
9	124.6	122.1	122.4	2.5	2.2	124.9	126.5	0.3	1.9
10	126.3	120.9	120.4	5.4	5.9	123.7	124.5	-2.6	-1.8
			Average Bias:	2.83	4.12		Standard Err. of the Est.:	1.98	1.65

Table 4.68: Result of Average Bias Analysis for Accuracy and Precision of Nuclear Density Guage
Site No.: 4

Core Number	Core Density,Pcf	Nuclear Densities		Ind. Bias Values		Corrected Nuclear Densities		Residuals	
		With Sand, pcf	Without Sand, pcf	With Sand	Without Sand	With Sand	Without Sand	With Sand	Without Sand
1	136.7	142.7	141.8	-6.0	-5.1	137.2	138.0	0.5	1.3
2	133.7	138.6	138.4	-4.9	-4.7	133.1	134.6	-0.6	0.9
3	134.9	141.1	137.7	-6.2	-2.8	135.6	133.9	0.7	-1.0
4	135.8	142.7	140.4	-6.9	-4.6	137.2	136.6	1.4	0.8
5	132.5	140.0	138.4	-7.5	-5.9	134.5	134.6	2.0	2.1
6	140.0	144.3	141.3	-4.3	-1.3	138.8	137.5	-1.2	-2.5
7	138.6	141.9	141.5	-3.3	-2.9	136.4	137.7	-2.2	-0.9
8	137.9	142.7	140.0	-4.8	-2.1	137.2	136.2	-0.7	-1.7
9	137.3	142.5	141.1	-5.2	-3.8	137.0	137.3	-0.3	0.0
10	136.3	142.3	141.2	-5.7	-4.6	136.8	137.4	0.2	0.8
			Average Bias:	-5.48	-3.78		Standard Err. of the Est.:	1.32	1.55

Table 4.69: Results of Average Bias Analysis for Accuracy and Precision of Nuclear Density Gauge
Site No.: 5

Core Number	Core Density,Pcf	Nuclear Densities		Ind. Bias Values		Corrected Nuclear Densities		Residuals	
		With Sand, pcf	Without Sand, pcf	With Sand	Without Sand	With Sand	Without Sand	With Sand	Without Sand
1	136.1	135.3	124.4	0.8	11.7	136.4	131.4	0.3	-4.7
2	136.5	135.8	129.7	0.7	6.8	136.9	136.7	0.4	0.2
3	134.5	135.1	128.5	-0.6	6.0	136.2	135.5	1.7	1.0
4	136.1	133.0	130.0	3.1	6.1	134.1	137.0	-2.0	0.9
5	136.2	132.6	128.0	3.6	8.2	133.7	135.0	-2.5	-1.2
6	140.9	141.2	134.7	-0.3	6.2	142.3	141.7	1.4	0.8
7	136.6	135.1	130.5	1.5	6.1	136.2	137.5	-0.4	0.9
8	138.5	136.3	131.7	2.2	6.8	137.4	138.7	-1.1	0.2
9	136.6	134.7	128.9	1.9	7.7	135.8	135.9	-0.8	-0.7
10	137.6	139.2	133.3	-1.6	4.3	140.3	140.3	2.7	2.7
			Average Bias:	1.13	6.99		Standard Err. of the Est.:	1.76	2.08

Table 4.70: Results of Average Bias Analysis for Accuracy and Precision of Nuclear Density Gauge
Site No.: __6__

Core Number	Core Density,Pcf	Nuclear Densities		Ind. Bias Values		Corrected Nuclear Densities		Residuals	
		With Sand, pcf	Without Sand, pcf	With Sand	Without Sand	With Sand	Without Sand	With Sand	Without Sand
1	145.3	142.1	140.9	3.2	4.4	145.0	145.9	-0.30	0.64
2	146.3	146.6	144.0	-0.3	2.3	149.5	149.0	3.20	2.74
3	144.1	141.5	136.7	2.6	7.4	144.4	141.7	0.30	-2.36
4	147.9	143.7	144.3	4.2	3.6	146.6	149.3	-1.30	1.44
5	147.1	142.3	139.6	4.8	7.5	145.2	144.6	-1.90	-2.46
			Average Bias:	2.90	5.04		Standard Err. of the Est.:	2.29	2.68

Table 4.71: Results of Average Bias Analysis for Accuracy and Precision of Nuclear Density Gauge
Site No.: __9__

Core Number	Core Density,Pcf	Nuclear Densities		Ind. Bias Values		Corrected Nuclear Densities		Residuals	
		With Sand, pcf	Without Sand, pcf	With Sand	Without Sand	With Sand	Without Sand	With Sand	Without Sand
1	150.6	148.7	149.1	1.9	1.5	152.1	150.8	1.5	0.2
2	146.5	142.0	146.6	4.5	-0.1	145.4	148.3	-1.1	1.8
3	150.3	148.9	149.2	1.4	1.1	152.3	150.9	2.0	0.5
4	145.2	139.0	144.1	6.2	1.1	142.4	145.8	-2.8	0.6
5	147.0	143.6	143.8	3.4	3.2	147.0	145.5	0.0	-1.5
6	147.1	145.0	144.4	2.1	2.7	148.4	146.1	1.3	-1.0
7	146.3	139.8	144.1	6.5	2.2	143.2	145.8	-3.1	-0.6
8	149.1	148.2	147.0	0.9	2.1	151.6	148.7	2.5	-0.4
9	145.5	140.0	144.6	5.5	0.9	143.4	146.3	-2.1	0.8
10	150.5	149.3	148.7	1.2	1.8	152.7	150.4	2.2	-0.2
			Average Bias:	3.36	1.65		Standard Err. of the Est.:	6.16	2.87

Table 4.72: Results of Average Bias Analysis for Accuracy and Precision of Nuclear Density Gauge
Site No.: _10__

Core Number	Core Density,Pcf	Nuclear Densities		Ind. Bias Values		Corrected Nuclear Densities		Residuals	
		With Sand, pcf	Without Sand, pcf	With Sand	Without Sand	With Sand	Without Sand	With Sand	Without Sand
1	147.9	142.5	141.1	5.4	6.8	146.3	146.9	-1.6	-1.1
2	147.4	156.9	144.4	-9.5	3.0	160.7	150.2	13.3	2.8
3	146.4	138.1	150.6	8.3	-4.2	141.9	156.4	-4.5	9.9
4	153.4	146.8	146.0	6.6	7.4	150.6	151.8	-2.8	-1.7
5	150.0	145.4	143.1	4.6	6.9	149.2	148.9	-0.8	-1.2
6	147.7	143.1	139.5	4.6	8.2	146.9	145.3	-0.8	-2.4
7	149.8	143.9	141.1	5.9	8.7	147.7	146.9	-2.1	-3.0
8	147.1	142.0	139.9	5.1	7.2	145.8	145.7	-1.3	-1.4
9	142.9	139.6	139.9	3.3	3.0	143.4	145.7	0.5	2.8
10	147.1	143.4	136.6	3.7	10.5	147.2	142.4	0.1	-4.8
			Average Bias:	3.80	5.75		Standard Err. of the Est.:	5.19	4.45

Table 4.73: Results of Average Bias Analysis for Accuracy and Precision of Nuclear Density Gauge
Site No.: _11__

Core Number	Core Density,Pcf	Nuclear Densities		Ind. Bias Values		Corrected Nuclear Densities		Residuals	
		With Sand, pcf	Without Sand, pcf	With Sand	Without Sand	With Sand	Without Sand	With Sand	Without Sand
1	144.6	142.4	139.7	2.2	4.9	146.0	145.4	1.4	0.8
2	145.3	141.5	139.1	3.8	6.2	145.1	144.8	-0.2	-0.5
3	143.2.	138.9	139.7	4.3	3.5	142.5	145.4	-0.7	2.2
4	142.7	141.2	137.7	1.5	5.0	144.8	143.4	2.1	0.7
5	143.0	141.6	139.7	1.4	3.3	145.2	145.4	2.2	2.4
6	145.7	142.6	140.0	3.1	5.7	146.2	145.7	0.5	0.0
7	143.7	141.8	139.9	1.9	3.8	145.4	145.6	1.7	1.9
8	146.2	140.8	139.1	5.4	7.1	144.4	144.8	-1.8	-1.4
9	147.7	139.3	137.6	8.4	10.1	142.9	143.3	-4.8	-4.4
10	146.7	142.4	139.7	4.3	7.0	146.0	145.4	-0.7	-1.3
			Average Bias:	3.36	5.66		Standard Err. of the Est.:	2.28	2.20

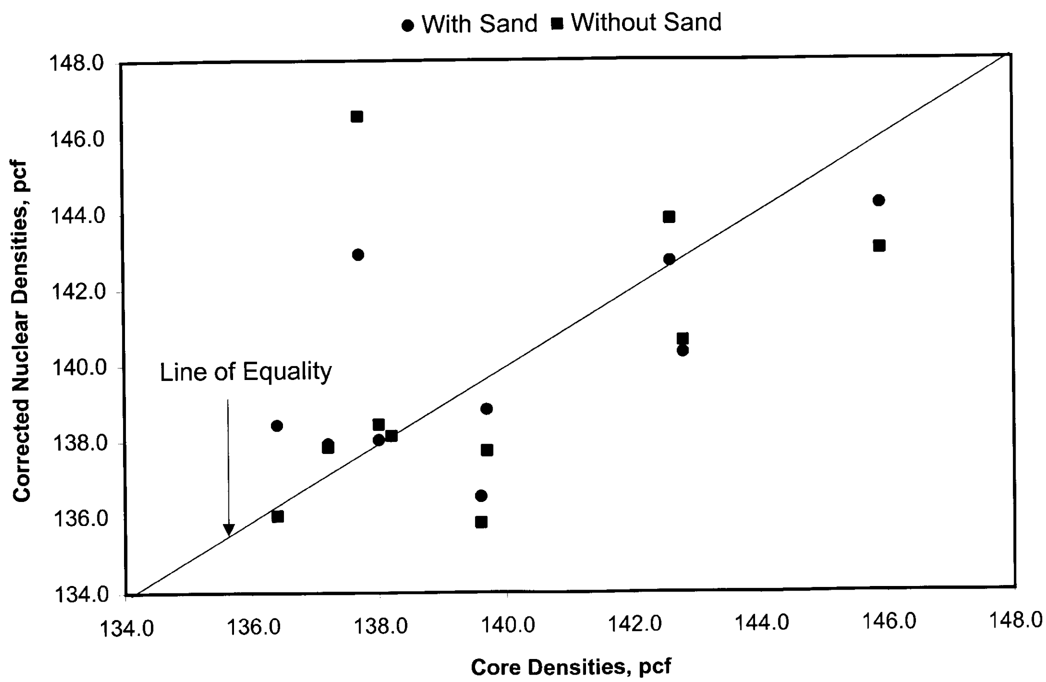


Figure 4.54: Results of Average Bias Analysis on Nuclear Density Gauge Results for Site 2

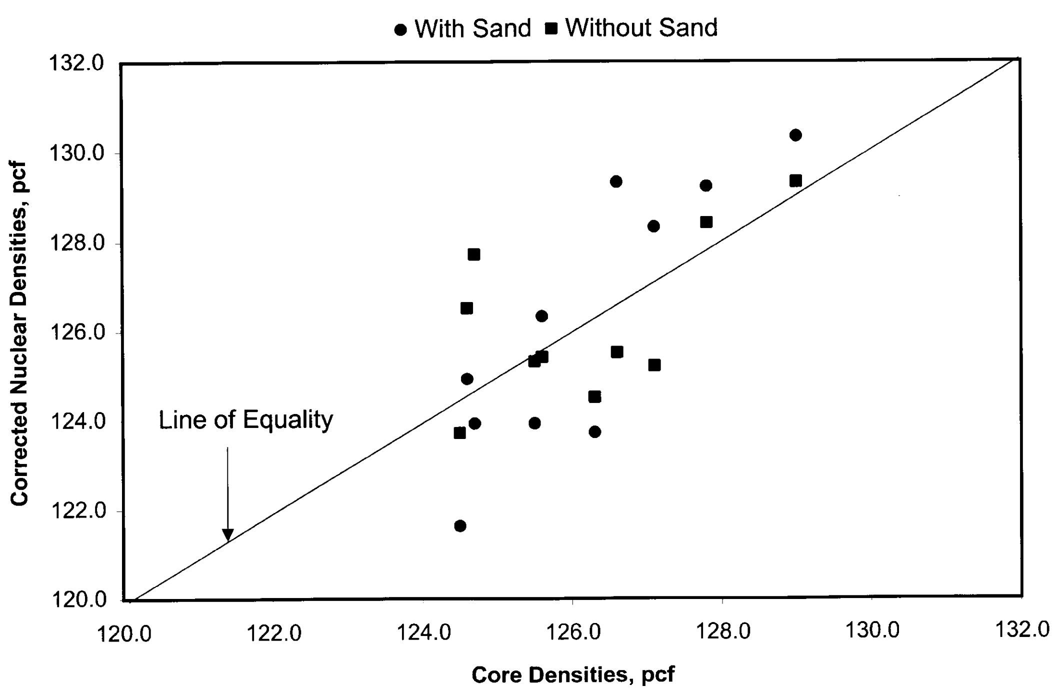


Figure 4.55: Results of Average Bias Analysis on Nuclear Density Gauge Results for Site 3

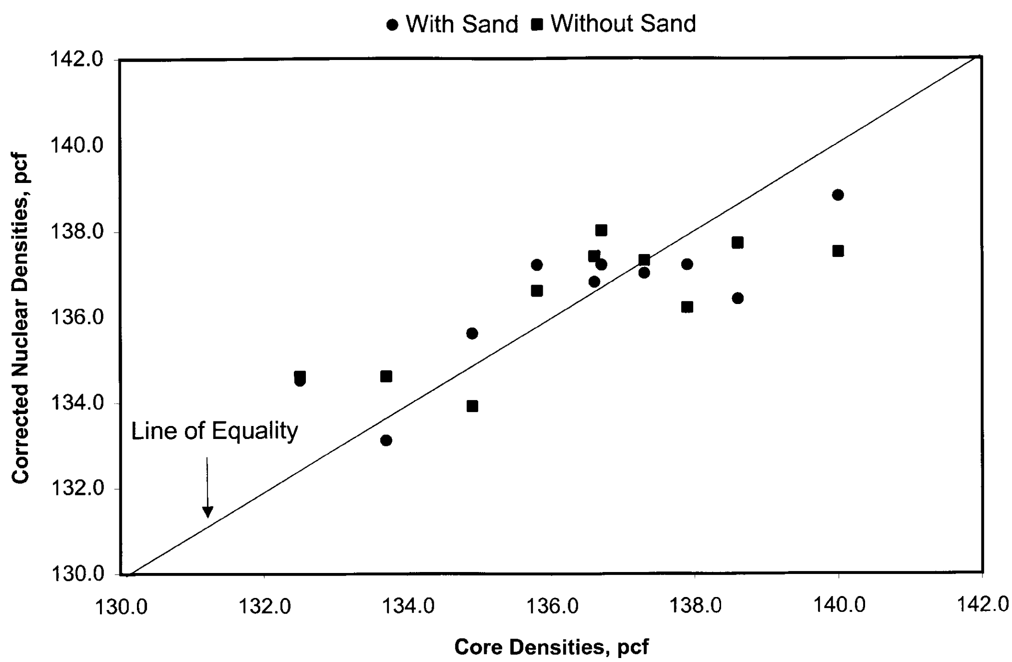


Figure 4.56: Results of Average Bias Analysis on Nuclear Density Gauge Results for Site 4

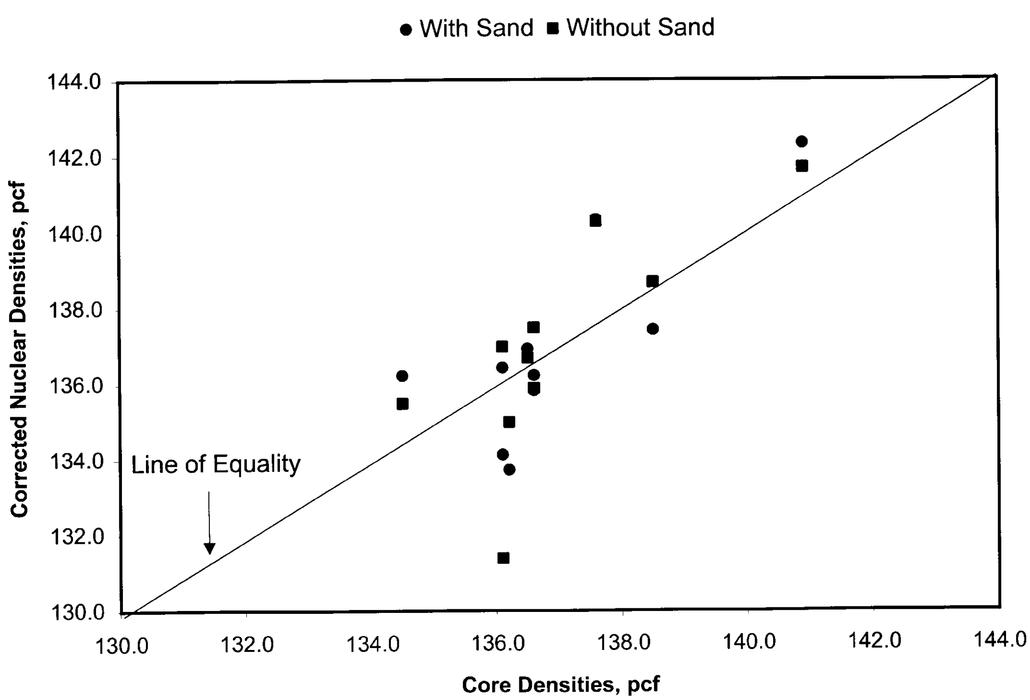


Figure 4.57: Results of Average Bias Analysis on Nuclear Density Gauge Results for Site 5

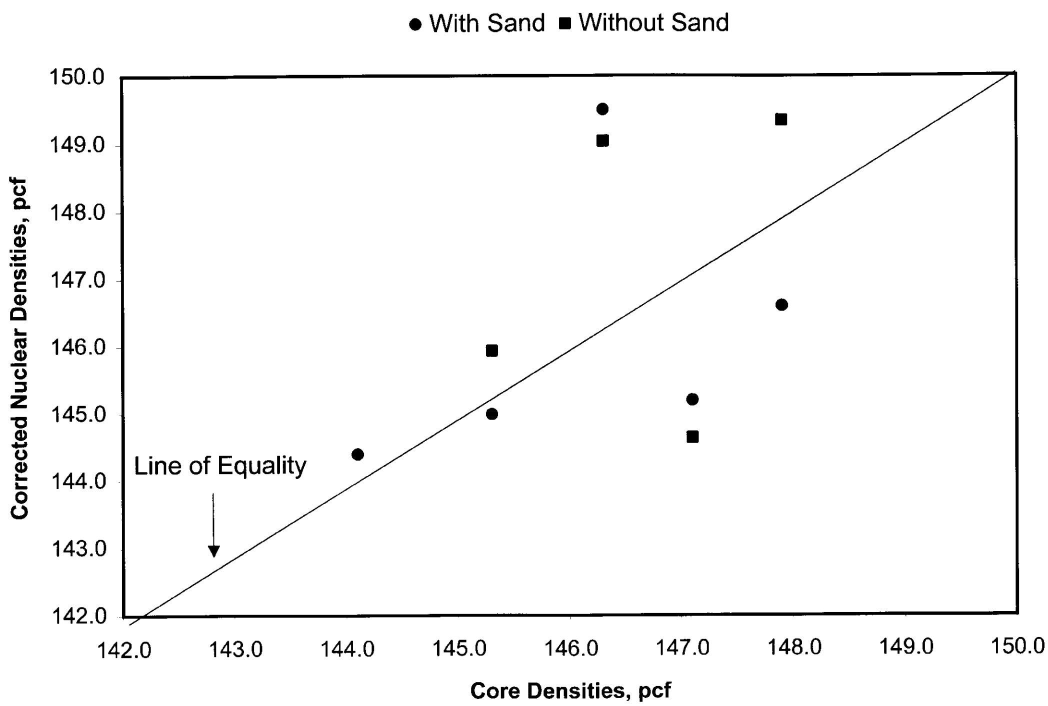


Figure 4.58: Results of Average Bias Analysis on Nuclear Density Gauge Results for Site 6

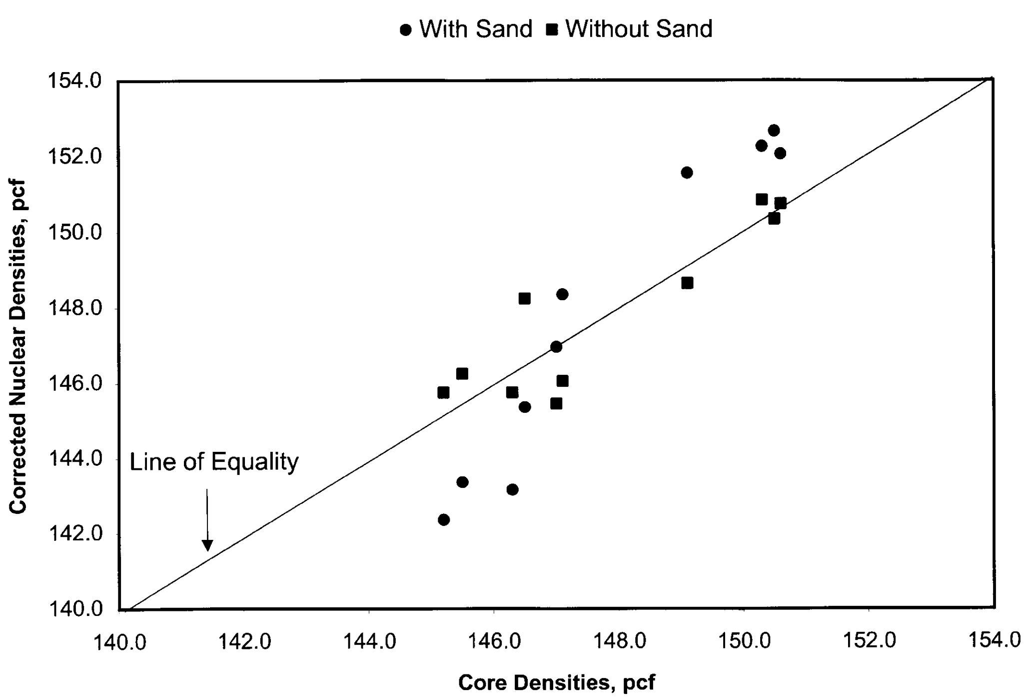


Figure 4.59: Results of Average Bias Analysis on Nuclear Density Gauge Results for Site 9

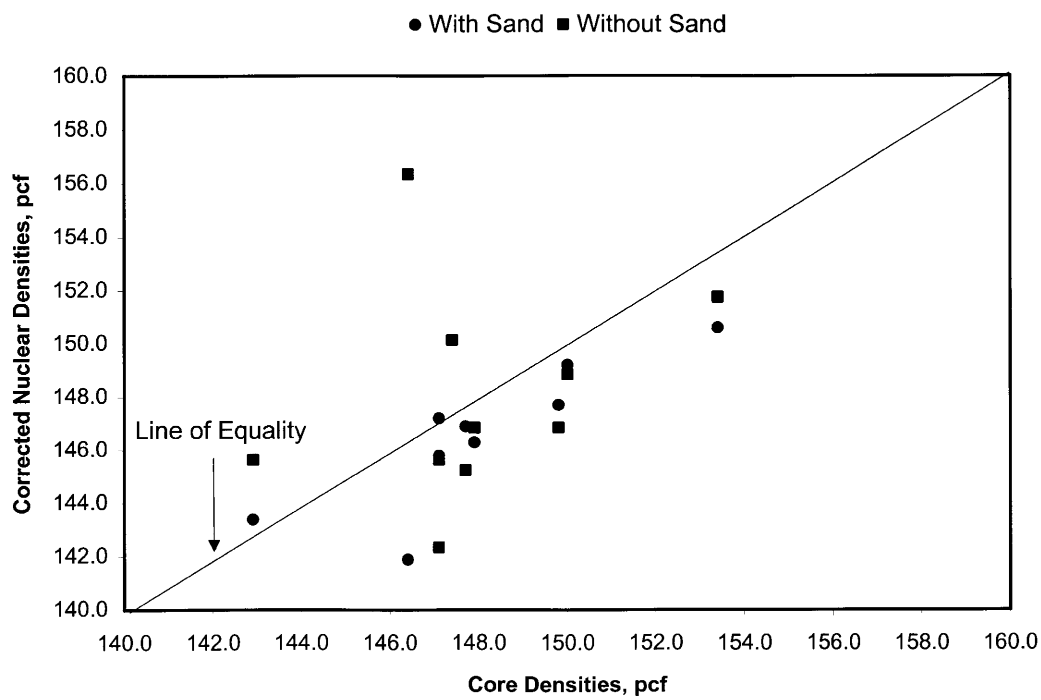


Figure 4.60: Results of Average Bias Analysis on Nuclear Density Gauge Results for Site 10

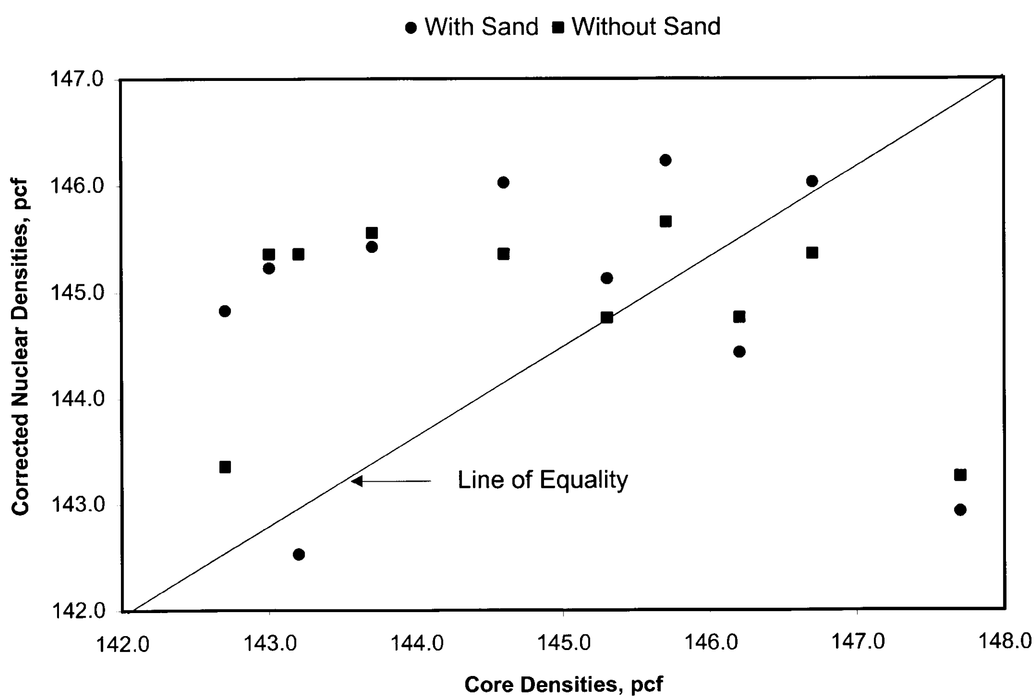


Figure 4.61: Results of Average Bias Analysis on Nuclear Density Gauge Results for Site 11

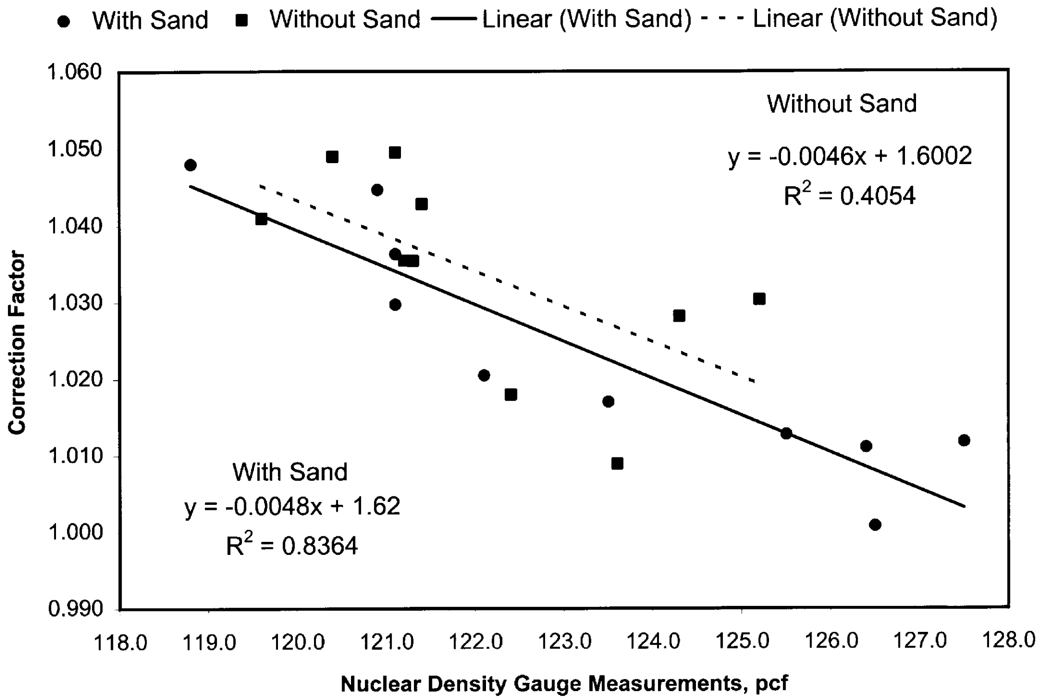


Figure 4.62: Example Plot Determining Dynamic Correction Factors (Site 3)

expected to be from 0.4 to 1.8 percent of G_{mm} in error when using the dynamic correction factor method. Using the static correction factors, the standard errors of the estimate ranged from 1.02 to 5.19 lbs per cu ft. Again using a SMA pavement with a 150 lbs per cu ft G_{mm}, based on these standard errors of the estimate, the nuclear gauge could be expected to be from 0.7 to 3.5 percent of G_{mm} in error when using the static correction factor. Additionally from Table 4.82, it appears that the nuclear density gauge readings made with Ottawa sand gave a slightly better estimate of true densities than did the measurements without sand. For the dynamic correction factors, the with sand readings had lower standard errors of the estimate for seven of the eight sites.

Based on this discussion, it appears that the nuclear density gauges can be used to estimate the density of SMA pavements in the field. A dynamic correction factor should be used to correct the nuclear density gauge measurements. The use of Ottawa sand will improve the accuracy of the test, however the use of this sand requires effort. The same accuracy can be obtained by simply conducting a higher number of tests when testing without sand.

Table 4.74: Results of Dynamic Correction Factor Analysis for Accuracy and Precision of Nuclear Density Gauge
Site No.: __2__

		Nuclear Densities		Relationship to Cores		Correction Factors		Corrected Nuclear Densities		Predicted Core Density		Residuals	
Core No.	Core Density, pcf	With Sand, pcf	Without Sand, pcf	With Sand	Without Sand	With Sand	Without Sand	With Sand, pcf	Without Sand, pcf	With Sand, pcf	Without Sand, pcf	With Sand	Without Sand
	A	B	C	D	E	F	G	H	I	J	K	L	M
				A/B	A/C							J-A	K-A
1	142.6	138.5	139.6	1.030	1.021	1.029	1.011	142.5	141.2	142.0	141.4	-0.57	-1.19
2	137.7	138.7	142.3	0.993	0.968	1.028	0.998	142.6	142.1	142.2	142.4	4.48	4.66
3	136.4	134.2	131.8	1.016	1.035	1.037	1.048	139.2	138.1	138.8	138.3	2.38	1.88
4	138.0	133.8	134.2	1.031	1.028	1.038	1.037	138.9	139.1	138.5	139.3	0.47	1.30
5	137.2	133.7	133.6	1.026	1.027	1.038	1.039	138.8	138.9	138.4	139.1	1.20	1.85
6	138.2	133.9	133.9	1.032	1.032	1.038	1.038	139.0	139.0	138.5	139.2	0.35	0.98
7	139.7	134.6	133.5	1.038	1.046	1.037	1.040	139.5	138.8	139.1	139.0	-0.62	-0.69
8	139.6	132.3	131.6	1.055	1.061	1.041	1.049	137.8	138.0	137.3	138.2	-2.28	-1.41
9	145.9	140.0	138.8	1.042	1.051	1.026	1.015	143.6	140.9	143.1	141.1	-2.75	-4.78
10	142.8	136.1	136.4	1.049	1.047	1.034	1.026	140.7	140.0	140.2	140.2	-2.58	-2.60

Standard Error of the Estimate: 2.44 2.85

Table 4.75: Results of Dynamic Correction Factor Analysis for Accuracy and Precision of Nuclear Density Gauge
Site No.: __3__

		Nuclear Densities		Relationship to Cores		Correction Factors		Corrected Nuclear Densities		Predicted Core Density		Residuals	
Core No.	Core Density, pcf	With Sand, pcf	Without Sand, pcf	With Sand	Without Sand	With Sand	Without Sand	With Sand, pcf	Without Sand, pcf	With Sand, pcf	Without Sand, pcf	With Sand	Without Sand
	A	B	C	D	E	F	G	H	I	J	K	L	M
				A/B	A/C							J-A	K-A
1	127.8	126.4	124.3	1.011	1.028	1.013	1.028	128.1	127.8	127.5	127.2	-0.34	-0.59
2	126.6	126.5	121.4	1.001	1.043	1.013	1.042	128.1	126.5	127.5	125.9	0.90	-0.72
3	125.6	123.5	121.3	1.017	1.035	1.027	1.042	126.9	126.4	126.3	125.8	0.67	0.23
4	127.1	125.5	121.1	1.013	1.050	1.018	1.043	127.7	126.3	127.1	125.7	0.00	-1.36
5	129.0	127.5	125.2	1.012	1.030	1.008	1.024	128.5	128.2	127.9	127.6	-1.11	-1.39
6	125.5	121.1	121.2	1.036	1.035	1.039	1.043	125.8	126.4	125.2	125.8	-0.27	0.28
7	124.7	121.1	123.6	1.030	1.009	1.039	1.032	125.8	127.5	125.2	126.9	0.53	2.20
8	124.5	118.8	119.6	1.048	1.041	1.050	1.050	124.7	125.6	124.2	125.0	-0.33	0.51
9	124.6	122.1	122.4	1.020	1.018	1.034	1.037	126.2	126.9	125.7	126.3	1.07	1.75
10	126.3	120.9	120.4	1.045	1.049	1.040	1.046	125.7	126.0	125.1	125.4	-1.16	-0.90
											Standard Error of the Estimate:	0.83	1.31

Table 4.76: Results of Dynamic Correction Factor Analysis for Accuracy and Precision of Nuclear Density Gauge
Site No.: __4__

		Nuclear Densities		Relationship to Cores		Correction Factors		Corrected Nuclear Densities		Predicted Core Density		Residuals	
Core No.	Core Density, pcf	With Sand, pcf	Without Sand, pcf	With Sand	Without Sand	With Sand	Without Sand	With Sand, pcf	Without Sand, pcf	With Sand, pcf	Without Sand, pcf	With Sand	Without Sand
	A	B	C	D	E	F	G	H	I	J	K	L	M
				A/B	A/C							J-A	K-A
1	136.7	142.7	141.8	0.958	0.964	0.963	0.972	137.5	137.8	137.4	138.3	0.68	1.60
2	133.7	138.6	138.4	0.965	0.966	0.957	0.967	132.6	133.8	132.5	134.3	-1.19	0.61
3	134.9	141.1	137.7	0.956	0.980	0.961	0.966	135.6	133.0	135.5	133.5	0.57	-1.40
4	135.8	142.7	140.4	0.952	0.967	0.963	0.970	137.5	136.2	137.4	136.7	1.58	0.86
5	132.5	140.0	138.4	0.946	0.957	0.959	0.967	134.3	133.8	134.2	134.3	1.67	1.81
6	140.0	144.3	141.3	0.970	0.991	0.966	0.971	139.4	137.2	139.3	137.7	-0.71	-2.29
7	138.6	141.9	141.5	0.977	0.980	0.962	0.971	136.5	137.4	136.4	137.9	-2.17	-0.65
8	137.9	142.7	140.0	0.966	0.985	0.963	0.969	137.5	135.7	137.4	136.2	-0.52	-1.71
9	137.3	142.5	141.1	0.964	0.973	0.963	0.971	137.2	137.0	137.1	137.5	-0.16	0.18
10	136.6	142.3	141.2	0.960	0.967	0.963	0.971	137.0	137.1	136.9	137.6	0.30	1.00

Standard Error of the Estimate: 1.28 1.52

Table 4.77: Results of Dynamic Correction Factor Analysis for Accuracy and Precision of Nuclear Density Gauge
Site No.: __5__

		Nuclear Densities		Relationship to Cores		Correction Factors		Corrected Nuclear Densities		Predicted Core Density		Residuals	
Core No.	Core Density, pcf	With Sand, pcf	Without Sand, pcf	With Sand	Without Sand	With Sand	Without Sand	With Sand, pcf	Without Sand, pcf	With Sand, pcf	Without Sand, pcf	With Sand	Without Sand
	A	B	C	D	E	F	G	H	I	J	K	L	M
				A/B	A/C							J-A	K-A
1	136.1	135.3	124.4	1.006	1.094	1.008	1.088	136.4	135.3	136.7	135.6	0.61	-0.47
2	136.5	135.8	129.7	1.005	1.052	1.006	1.049	136.7	136.1	137.0	136.9	0.47	0.43
3	134.5	135.1	137.7	0.996	0.977	1.009	0.991	136.3	136.4	136.6	137.5	2.10	3.04
4	136.1	133.0	130.0	1.023	1.047	1.016	1.047	135.2	136.1	135.5	137.0	-0.62	0.88
5	136.2	132.6	128.0	1.027	1.064	1.018	1.062	135.0	135.9	135.3	136.6	-0.94	0.39
6	140.9	141.2	134.7	0.998	1.046	0.987	1.013	139.3	136.4	139.7	137.5	-1.22	-3.40
7	136.6	135.1	130.5	1.011	1.047	1.009	1.043	136.3	136.2	136.6	137.1	0.00	0.46
8	138.5	136.3	131.7	1.016	1.052	1.005	1.035	136.9	136.3	137.2	137.2	-1.27	-1.26
9	136.6	134.7	128.9	1.014	1.060	1.010	1.055	136.1	136.0	136.4	136.8	-0.21	0.18
10	137.6	139.2	133.3	0.989	1.032	0.994	1.023	138.4	136.4	138.7	134.4	1.10	-0.19

Standard Error of the Estimate: 1.15 1.73

Table 4.78: Results of Dynamic Correction Factor Analysis for Accuracy and Precision of Nuclear Density Gauge
Site No.: __6__

		Nuclear Densities		Relationship to Cores		Correction Factors		Corrected Nuclear Densities		Predicted Core Density		Residuals	
Core No.	Core Density, pcf	With Sand, pcf	Without Sand, pcf	With Sand	Without Sand	With Sand	Without Sand	With Sand, pcf	Without Sand, pcf	With Sand, pcf	Without Sand, pcf	With Sand	Without Sand
	A	B	C	D	E	F	G	H	I	J	K	L	M
				A/B	A/C							J-A	K-A
1	145.3	142.1	140.9	1.023	1.031	1.019	1.042	144.9	146.8	145.8	144.2	0.49	-1.09
2	146.3	146.6	144.0	0.998	1.016	0.997	1.027	146.1	147.8	147.2	145.5	0.88	-0.83
3	1441.1	141.5	136.7	1.018	1.054	1.022	1.062	144.7	145.2	145.6	144.0	1.49	-0.07
4	147.9	143.7	144.3	1.029	1.025	1.0114	1.025	145.3	147.9	146.3	144.7	-1.59	-3.22
5	147.1	142.3	139.6	1.034	1.054	1.018	1.048	144.9	146.3	145.9	144.3	-1.24	-2.83

Standard Error of the Estimate: 1.56 2.10

Table 4.79: Results of Dynamic Correction Factor Analysis for Accuracy and Precision of Nuclear Density Gauge
Site No.: __9__

		Nuclear Densities		Relationship to Cores		Correction Factors		Corrected Nuclear Densities		Predicted Core Density		Residuals	
Core No.	Core Density, pcf	With Sand, pcf	Without Sand, pcf	With Sand	Without Sand	With Sand	Without Sand	With Sand, pcf	Without Sand, pcf	With Sand, pcf	Without Sand, pcf	With Sand	Without Sand
	A	B	C	D	E	F	G	H	I	J	K	L	M
				A/B	A/C							J-A	K-A
1	150.6	148.7	149.1	1.013	1.010	1.006	1.003	149.5	149.6	149.9	150.3	-0.71	-0.28
2	146.5	142.0	146.6	1.032	0.999	1.031	1.006	146.3	147.5	146.6			
3	150.3	148.9	149.2	1.009	1.007	1.005	1.003	149.6	149.7	150.0	150.4	-0.32	0.10
4	145.2	139.0	144.1	1.045	1.008	1.042	1.009	144.8	145.4	145.1	146.0	-0.12	0.84
5	147.0	146.3	143.8	1.024	1.022	1.025	1.009	147.1	145.1	147.4	145.8	0.45	-1.22
6	147.1	145.0	144.4	1.014	1.019	1.019	1.009	147.8	145.6	148.1	146.3	1.04	-0.80
7	146.3	139.8	144.1	1.046	1.015	1.039	1.009	145.2	145.4	145.5	146.0	-0.79	-0.26
8	149.1	148.2	147.0	1.006	1.014	1.008	1.006	149.3	147.8	149.7	148.5	0.56	-0.57
9	145.5	140.0	144.6	1.039	1.006	1.038	1.008	145.3	145.8	145.6	146.5	0.11	0.97
10	150.5	149.3	148.7	1.008	1.012	1.003	1.004	149.8	149.3	150.2	150.0	-0.34	-0.52
										Standard Error of the Estimate:		0.61	0.96

Table 4.80: Results of Dynamic Correction Factor Analysis for Accuracy and Precision of Nuclear Density Gauge
Site No.: 10

		Nuclear Densities		Relationship to Cores		Correction Factors		Corrected Nuclear Densities		Predicted Core Density		Residuals	
Core No.	Core Density, pcf	With Sand, pcf	Without Sand, pcf	With Sand	Without Sand	With Sand	Without Sand	With Sand, pcf	Without Sand, pcf	With Sand, pcf	Without Sand, pcf	With Sand	Without Sand
	A	B	C	D	E	F	G	H	I	J	K	L	M
				A/B	A/C							J-A	K-A
1	147.9	142.5	141.1	1.038	1.048	1.043	1.047	148.6	147.7	147.6	147.8	-0.28	-0.11
2	147.4	156.9	144.4	0.939	1.021	0.965	1.027	151.4	148.3	150.8	148.5	3.36	1.14
3	146.4	138.1	150.6	1.060	0.972	1.066	0.990	147.3	149.2	146.2	149.5	-0.23	3.13
4	153.4	146.8	146.0	1.045	1.051	1.019	1.018	149.6	148.6	148.8	148.8	-4.58	-4.56
5	150.0	145.4	143.1	1.032	1.048	1.027	1.035	149.3	148.1	148.5	148.3	-1.55	-1.74
6	147.7	143.1	139.5	1.032	1.059	1.039	1.056	148.7	147.3	147.8	147.4	0.10	-0.32
7	149.8	143.9	141.1	1.041	1.062	1.035	1.047	148.9	147.7	148.0	147.8	-1.76	-2.01
8	147.1	142.0	139.9	1.036	1.051	1.045	1.054	148.4	147.4	147.5	147.5	0.37	0.39
9	142.9	139.6	139.9	1.024	1.021	1.058	1.054	147.7	147.4	146.7	147.5	3.79	4.59
10	147.1	143.4	136.6	1.026	1.077	1.038	1.073	148.8	146.6	147.9	146.5	0.79	-0.56
											Standard Error of the Estimate:	2.57	2.75

Table 4.81: Results of Dynamic Correction Factor Analysis for Accuracy and Precision of Nuclear Density Gauge
Site No.: __11__

		Nuclear Densities		Relationship to Cores		Correction Factors		Corrected Nuclear Densities		Predicted Core Density		Residuals	
Core No.	Core Density, pcf	With Sand, pcf	Without Sand, pcf	With Sand	Without Sand	With Sand	Without Sand	With Sand, pcf	Without Sand, pcf	With Sand, pcf	Without Sand, pcf	With Sand	Without Sand
	A	B	C	D	E	F	G	H	I	J	K	L	M
				A/B	A/C							J-A	K-A
1	144.6	142.4	139.7	1.015	1.035	1.018	1.039	145.0	145.1	145.0	144.7	0.36	0.09
2	145.3	141.5	139.1	1.027	1.045	1.025	1.045	145.0	145.3	144.9	144.9	-0.43	-0.37
3	143.2	138.9	139.7	1.031	1.025	1.044	1.039	145.1	145.1	144.8	144.7	1.57	1.49
4	142.7	141.2	137.7	1.011	1.036	1.027	1.059	145.0	145.8	144.8	145.4	2.15	2.74
5	143.0	141.6	139.7	1.010	1.024	1.024	1.039	145.0	145.1	144.9	144.7	1.88	1.69
6	145.7	142.6	140.0	1.022	1.041	1.017	1.036	145.0	145.0	145.0	144.6	-0.71	-1.12
7	143.7	141.8	139.9	1.013	1.027	1.023	1.037	145.0	145.0	144.9	144.6	1.20	0.92
8	146.2	140.8	139.1	1.038	1.051	1.030	1.045	145.1	145.3	144.8	144.9	-1.38	-1.27
9	147.7	139.3	137.6	1.060	1.073	1.041	1.060	145.1	145.9	144.8	145.5	-2.93	-2.22
10	146.7	142.4	139.7	1.030	1.050	1.018	1.039	145.0	145.1	145.0	144.7	-1.74	-2.01

Standard Error of the Estimate: 1.82 1.78

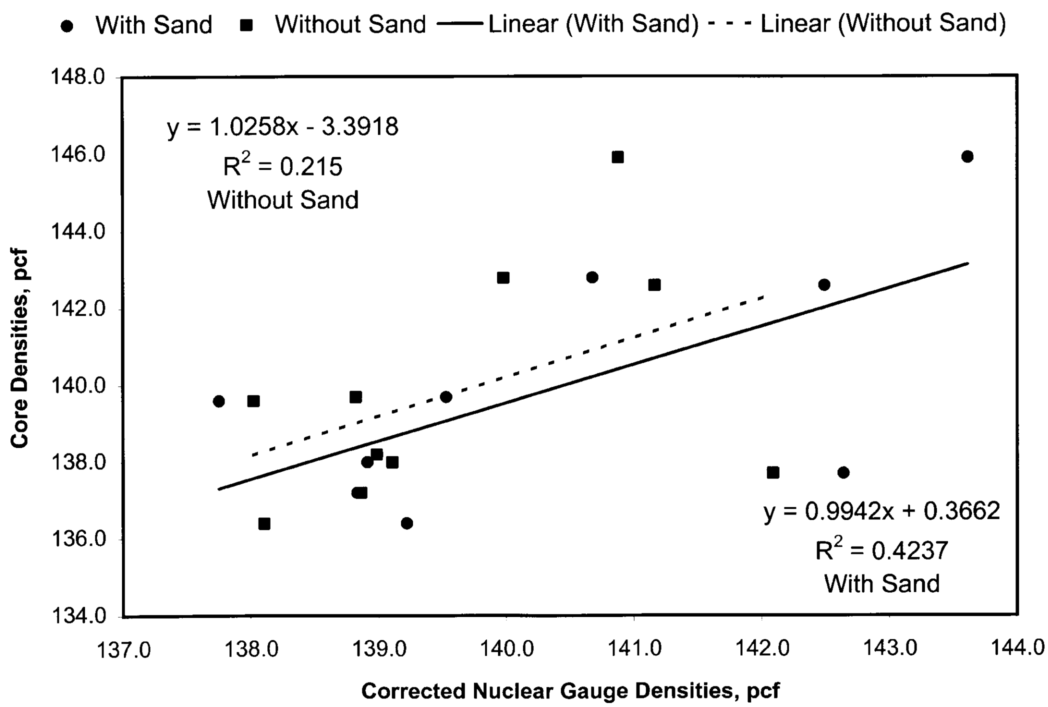


Figure 4.63: Results of Dynamic Analysis on Nuclear Density Gauge Results for Site 2

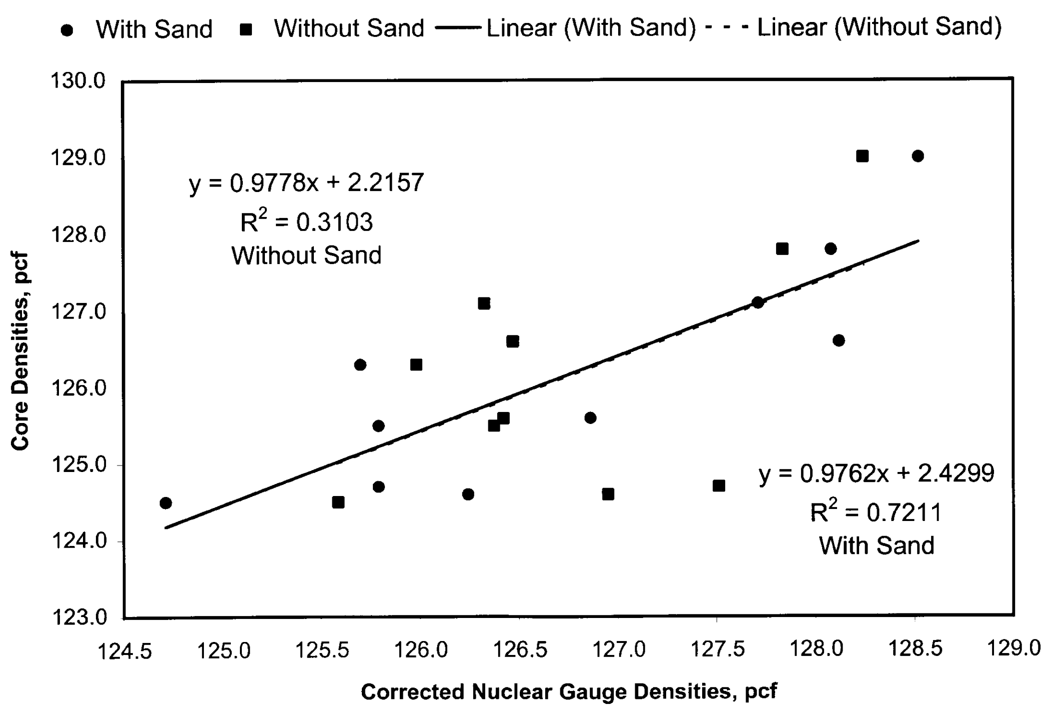


Figure 4.64: Results of Dynamic Analysis on Nuclear Density Gauge Results for Site 3

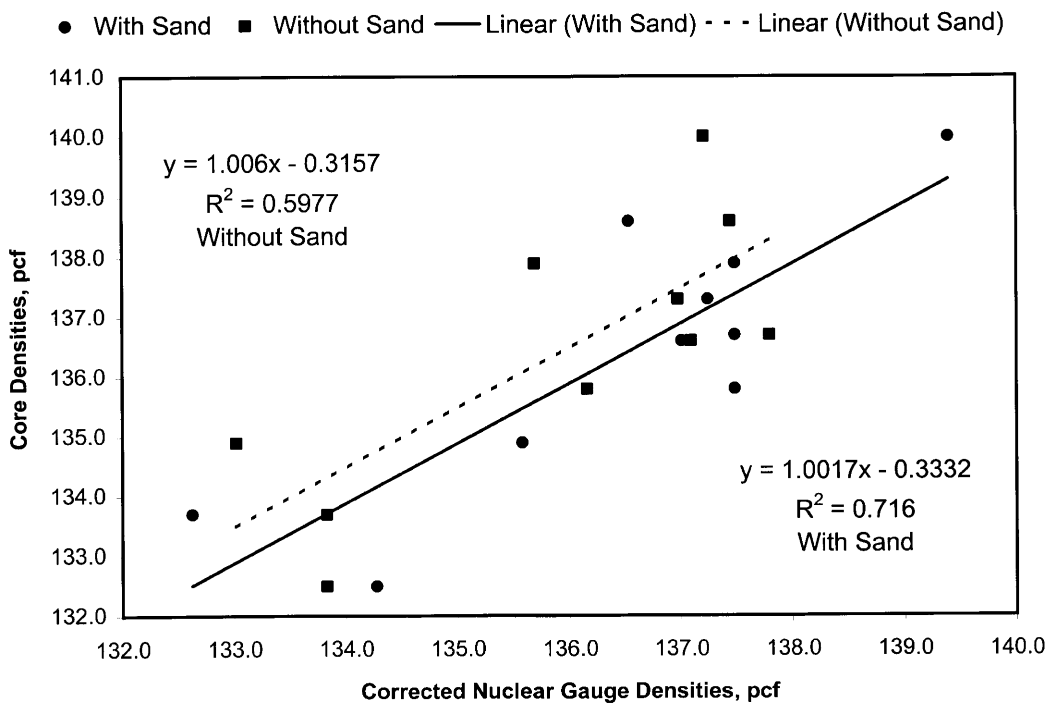


Figure 4.65: Results of Dynamic Analysis on Nuclear Density Gauge Results for Site 4

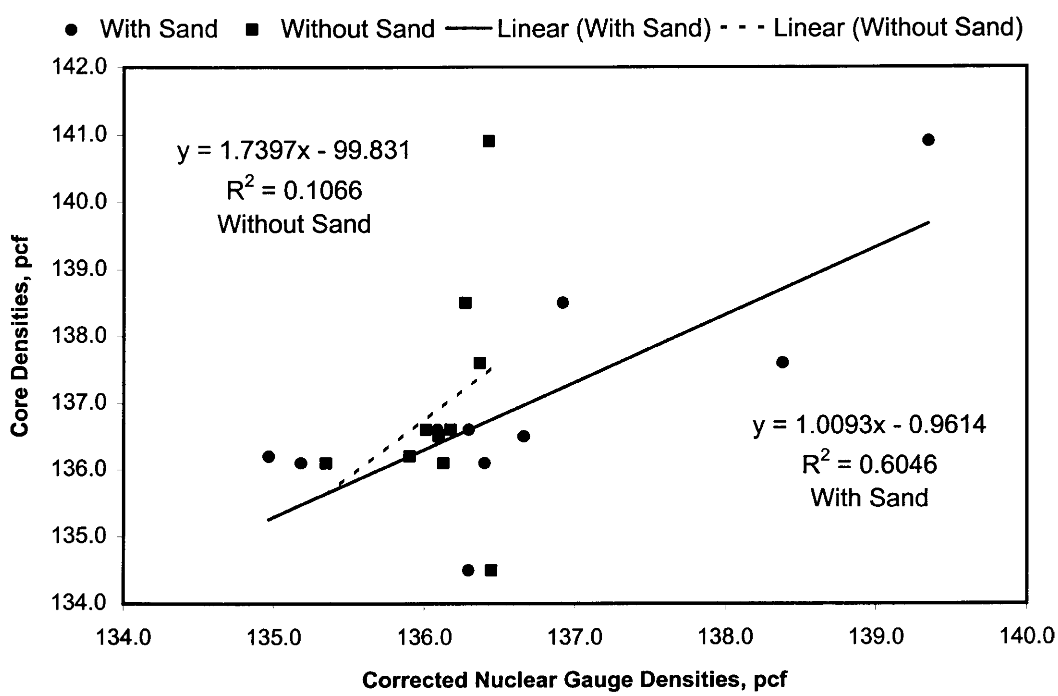


Figure 4.66: Results of Dynamic Analysis on Nuclear Density Gauge Results for Site 5

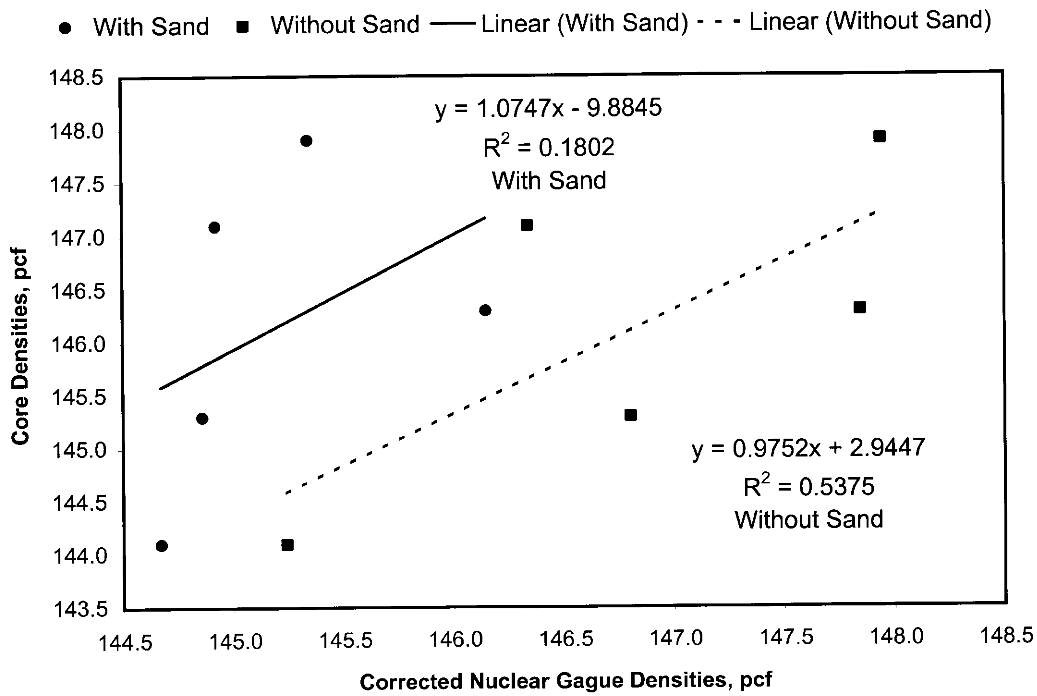


Figure 4.67: Results of Dynamic Analysis on Nuclear Density Gauge Results for Site 6

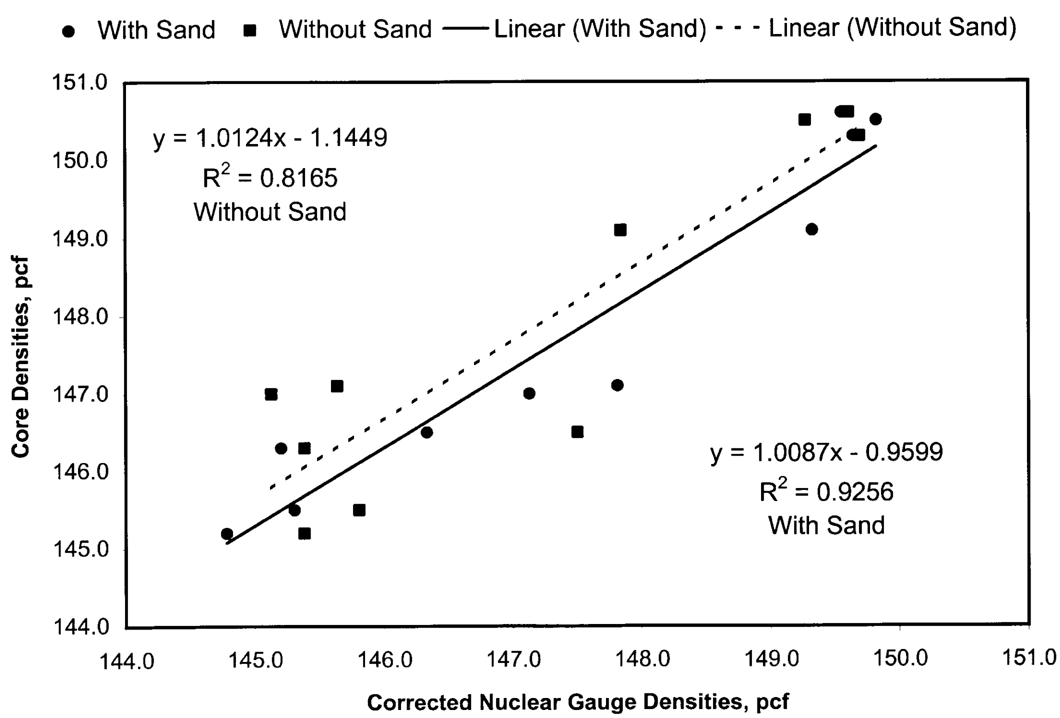


Figure 4.68: Results of Dynamic Analysis on Nuclear Density Gauge Results for Site 9

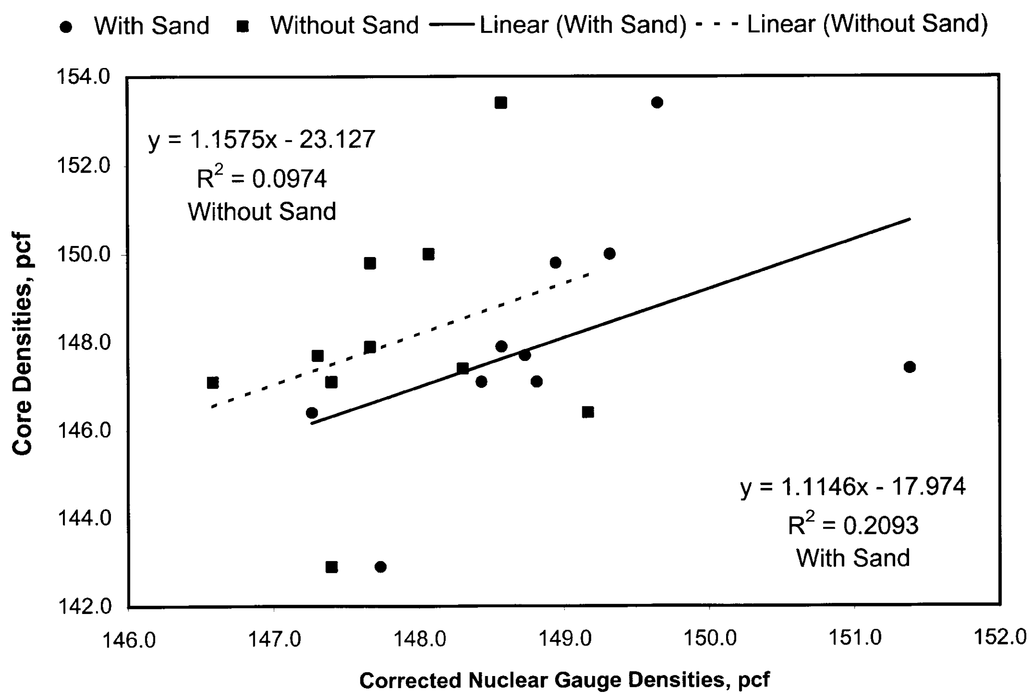


Figure 4.69: Results of Dynamic Analysis on Nuclear Density Gauge Results for Site 10

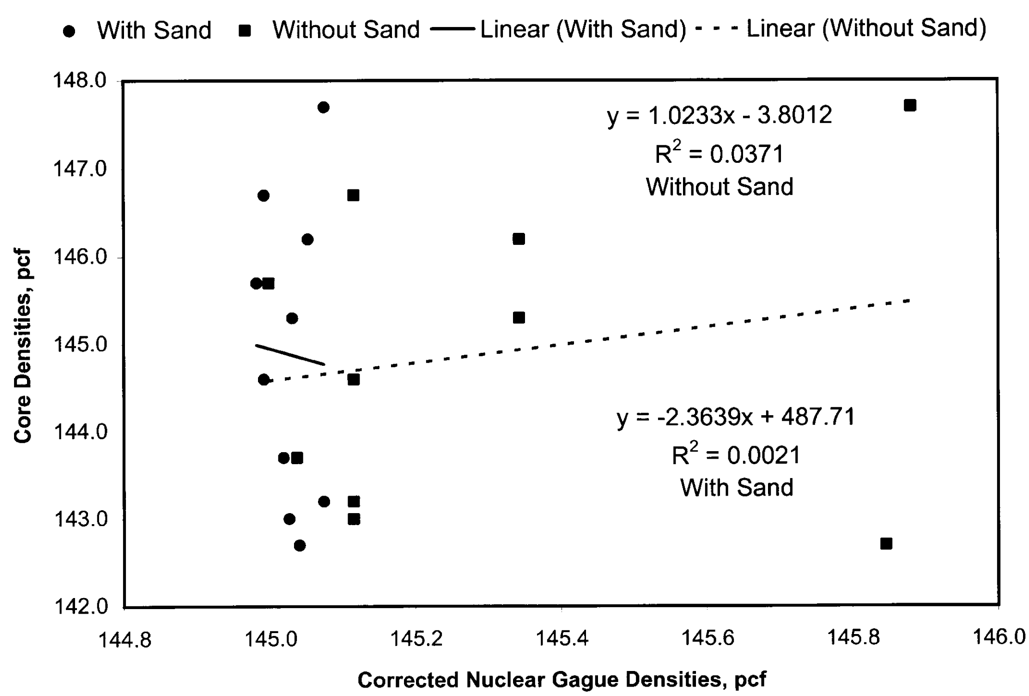


Figure 4.70: Results of Dynamic Analysis on Nuclear Density Gauge Results for Site 11

Table 4.82: 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

CHAPTER 5 - DISCUSSION OF RESULTS

The results obtained and reported earlier in this report are discussed below. This discussion is aimed at explaining the rationale for the final conclusions and recommendations based on the test results presented.

5.1 FIELD EVALUATION OF SMA MIX DESIGN PROCEDURE

It was important to verify that the proposed mix design procedure from the Phase I work could be verified on a number of projects. For most projects, the SMA mixtures were not designed using the recommended procedures. However, they were typically designed using an approach similar to the recommended procedure so the job mix formula was not that different from what would have been obtained using the proposed procedure. Most designs were performed using 50-blows with the Marshall hammer. Another goal of the field tests was to verify that the procedure developed for the Superpave gyratory compactor was applicable on the projects.

Many other objectives were identified to be evaluated in these field tests. To accomplish all of the testing necessary it was essential for NCAT to have a field laboratory at each of the eleven field projects. A field laboratory was set up in a rental truck to be transported to each site. Since the field laboratory was available, compaction data from the Superpave gyratory compactor and the Marshall hammer was available from most projects.

The location of these eleven projects were selected to include all regions of the country. There was a wide variation in materials and mix designs. These mixes should be representative of typical mixes that may be utilized for SMA. Some projects used fibers, some used polymers, and some used both. Some projects used pelletized fiber while others used loose fibers. A wide range of mineral filler types was used and these fillers were added to the mixture using different techniques. Eight of the projects used anti-stripping agents with some using lime and others using liquid anti-strip materials.

Some of the projects met the volumetric properties suggested under Phase I of the study. Even though some of the projects did not meet the volumetric properties, a small mix adjustment would have allowed those mixes to meet the suggested requirements. Very few of the projects evaluated the voids in coarse aggregate and none evaluated the mortar properties. However, test results by NCAT indicated that most of the SMA mixtures would meet the recommended requirements for these properties.

The field work did indicate that the recommended mix design procedure does develop mixes that can be produced and placed in the field. Some of the tests developed as a part of this mix design procedure can be used as a part of quality control testing during construction.

Based on the field projects, the draindown test was easy to conduct and seemed 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. On at least one project where fat spots were observed in the field, the laboratory draindown was greater than 0.3.

5.2 CORRELATION BETWEEN SUPERPAVE GYRATORY COMPACTOR AND MARSHALL HAMMER

Work in Phase I of this project indicated that 100 gyrations of the SGC would provide about the same density as 50-blows of the Marshall hammer. Since past experience with SMA has been good and since most SMA mixtures in the past were designed using 50-blows with the Marshall hammer, it was determined in this study that the compaction effort with the SGC should be set to provide a density approximately equal to that with the Marshall. Comparisons were made on the field projects to verify that 100 gyrations with the SGC was equal to 50-blows with a Marshall hammer. There was a lot of scatter in the data. Some projects needed more than 100 gyrations to produce density equal to that provided with a Marshall hammer. On other projects less than 60 gyrations were needed to produce a density equal to the density with 50-blow Marshall. When all of the data was averaged, it showed that approximately 80 gyrations were required to provide equal density. Generally, very little additional density is obtained between 80 and 100 gyrations. Based on the results shown in Figure 4.2 it is anticipated that changing the compaction effort from 100 to 70 gyrations (based on recommendation from NCHRP 9-9) will result in a change of less than 1 percent in air voids. This change in air voids will result in a change in optimum asphalt content of approximately 0.4 percent when going from 100 to 70 gyrations.

Data also showed that fewer gyrations were required to compact the SMA with softer aggregates (L.A. Abrasion more than 30). More gyrations were required for harder aggregates (L.A. Abrasion less than 30). To be consistent with the work developed under NCHRP 9-9 and the data presented in this report, it is recommended that 70 gyrations be used when the L.A. Abrasion is above 30 and 100 gyrations when the L.A. Abrasion is below 30. The difference between 70 and 100 gyrations will result in an approximate change in optimum asphalt content of 0.4%. Since this difference is small, it is not anticipated that a major problem will occur as a result of selecting an incorrect compaction level.

5.3 AGGREGATE BREAKDOWN

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

Statistical analyses showed that there was a significant difference in gradation after compaction and prior to compaction. The amount of breakdown in the laboratory was related to the aggregate hardness. Analysis also showed that the Marshall compactor consistently resulted in more breakdown than the gyratory compactor. The data also showed that the breakdown underneath rollers was approximately equal to that observed with the Marshall and with the SGC. So even though the Marshall provides more aggregate breakdown there is no significant advantage (as far as aggregate breakdown) in using either type of laboratory compactor since both provide approximately equal breakdown to what is observed in the field. There is other reasons for wanting to use a SGC but from an aggregate breakdown point of view, there is no obvious advantage when comparing results to breakdown in the field.

5.4 MORTAR TESTING

SMA mixes consist of an aggregate skeleton partially filled with a binder rich mortar (filler and binder). This mortar makes up at least 20 percent by volume of the SMA, so its properties are important to ensure good mixture performance.

The best way to determine how well the mortar will perform is to test the mortar. In this project, tests were conducted on the filler, fiber, and asphalt binder individually and based on these tests some indication of performance was obtained. One problem with this approach however is that it provides no indication of the best proportioning of the mortar.

Establishing mortar tests and setting limits on the results will allow an evaluation of the entire mortar which indirectly provides some control over material properties and proportions. As indicated in the test results, the addition of filler in proportions used in SMA will increase the stiffness of the binder approximately 5 times at low (BBR) and high (DSR) temperatures. This indicates that limits for the mortar can be set at 5 times the limit for the asphalt binder. Generally, the tests conducted at intermediate temperature results in less than 5 times increase in stiffness. So the intermediate temperature does not seem to be critical. For this reason, the intermediate temperature does not need to be included in the testing requirements.

The mortar stiffness is affected by the filler properties (fineness, shape, texture, etc) and by the amount of filler. These mortar tests will provide some guidance on the amount of filler to be used for specific mix designs. For example, a coarse filler will likely have to be used at a higher percentage than a finer filler to provide equal mortar properties. Also, a filler with a low specific gravity can be used at a lower percentage than one with a higher specific gravity to provide similar mortar properties.

It is reasonable to use the mortar testing for mix design but it will be difficult to use these tests for quality control during production. Preparation of the mortar requires that the filler (from aggregate and from commercial filler) be blended with the asphalt binder. The mortar can not be obtained from the SMA mixture so it has to be blended in the laboratory. It is recommended that the mortar testing be done for the mix design or when significant mixture changes are made. Mortar testing does not need to be done as a part of the daily QC testing. However, if at any time one believes that the filler or binder have changed, mortar tests can be conducted to verify its acceptability.

5.5 WHEEL TRACKING TESTS

Specimens were prepared at each of the 11 sites for laboratory wheel track testing. The wheel track testing indicated that the mixes at all sites performed well. These tests were conducted to help evaluate the mixture quality and to have a point of reference if problems with these pavements occur in the future. Interestingly, the only site that did not use fiber was the site with the highest rutting in the wheel track test. Again, this may have no significance since wheel track rutting at all sites was low.

5.6 EVALUATION OF NOMINAL MAXIMUM SIZE AND BREAK POINT SIEVE

Most SMA mixtures that have been placed in the US have been 19.0 mm nominal maximum size. 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 on the properties of SMA mixtures.

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 that contain small percentages is considered the break point sieve. For example, a typical SMA mix will have a large percentage of material on all sieves down to the 4.75 mm sieve. After this 4.75 mm sieve size, each sieve typically has a low percentage of material. So for a 19.0 mm mixture is there a difference if the break point sieve is 4.75 mm or 9.5 mm?

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 break point 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 designed using the proposed mix design method. The 4.75 mm mixes had very high VMAs and required a high asphalt content to produce satisfactory volumetrics. This high optimum asphalt content may make these mixes uneconomical. As expected the VMA and optimum asphalt contents reduced as the nominal maximum aggregate size increased.

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

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 (below the restricted zone) at the same nominal maximum size were less permeable than the SMA mixes for the same void level. The fine graded Superpave mixes (above the restricted zone) were even much less permeable. This data seems to indicate that mixes with a high percentage of coarse aggregate are more permeable than similar mixes with less coarse aggregate. The size of the coarse aggregate as well as the amount is important.

5.7 FLAT AND ELONGATED PARTICLES

The effect of flat and elongated particles was evaluated. Mixes with various amounts of flat and elongated particles were produced and tested to evaluate aggregate breakdown, effect on volumetrics, and water susceptibility. There was some increase in breakdown as the amount of flat and elongated particles increased. The VMA did increase as the amount of flat and elongated particles increased. The flat and elongated particles had no effect on moisture susceptibility.

Some amount of flat and elongated particles can be used without any detrimental effect on mixture properties however after some point is reached the effect becomes more severe. It appears that the current criteria of no more than twenty percent 3 to 1 and no more than five percent 5 to 1 is satisfactory. However, four of the seven projects evaluated during construction did not meet the twenty percent limit on the 3 to 1 requirements. This requirement may limit the use of SMA in some states.

5.8 EFFECT OF UNDERLYING LAYER AND ROLLER TYPE ON AGGREGATE BREAKDOWN

At the beginning of the study it was anticipated that the effect of overlaying PCC in comparison to HMA would be evaluated. It was believed that overlying PCC would result in more aggregate breakdown during compaction. However, a lack of SMA overlays of PCC made it impossible to evaluate the effect of pavement type being overlaid on aggregate breakdown.

The effect of roller type was evaluated. Ten of the eleven projects were rolled with static steel wheel rollers or vibratory rollers. The remaining project also used a rubber tire roller along with a static steel wheel roller. A statistical analysis of the data indicated that there was not a significant difference in the breakdown underneath static and vibratory steel wheel rollers. The results also indicated that the L.A. Abrasion properties of the aggregate being rolled had very little effect on the breakdown.

5.9 POTENTIAL DAMAGE TO ASPHALT CEMENT AT HIGH TEMPERATURES

SMA projects tend to use a lot of modified asphalts. These modified asphalts require a relatively high mixing and compaction temperature. Since these mixes require high temperatures, it was determined to evaluate the effect these high temperatures on binder properties. The binders were aged in thin films in a conventional oven at various temperatures. The binders were then tested with the DSR and BBR to determine the effect of heating on binder properties.

The data indicated that heating had very little effect on the binder properties up to approximately 180°C. However, when binders were heated above 180°C a significant difference in the binder properties were indicated with the DSR at high temperatures and the BBR at low temperatures. This trend seemed to be consistent for most of the binders. This data indicates that 180°C is the highest that asphalt binders should be heated without concern for damage of the asphalt binder. Specifications should clearly specify a maximum temperature for binder.

5.10 LABORATORY MIXING AND COMPACTION TEMPERATURES

Work was performed to determine a method to establish the mixing and compaction temperatures of SMA mixes. The Brookfield viscometer was used and a workability device was evaluated. The data indicates that both tests (viscosity with Brookfield viscometer and workability) do a good job of measuring the stiffness of the mortar and mix, respectively.

However, neither test appeared to provide a method to determine the mixing and compaction temperatures. The best approach for determining these temperatures is AASHTO T245.

5.11 DENSITY REQUIREMENTS

SMA mixtures have been constructed in the U.S. since 1991. There have been at least 2 to 3 SMA projects that have been identified with permeability problems. The primary cause of high permeability is in-place air voids but the type of mixes used is also important. It has been shown that SMA mixes with higher percentages of large coarse aggregates tend to be permeable even at relatively low air voids.

The data has shown that SMA mixes above 5 to 7 percent air voids are permeable to water. For these coarse mixtures (19 to 25 mm) this number is closer to 5 and for fine mixes the number is closer to 7. The data collected from the eleven projects clearly indicated that the in-place voids in many cases were significantly higher than even 7 percent. More emphasis must be placed on good compaction or long term durability due to water related problems is likely to occur. The density of SMA mixes should be required to be high enough to produce no more than 6 percent air voids in the field.

The field permeability test that was used in this study appeared to provide inaccurate data. In most cases, the laboratory procedure appeared to provide good results.

5.12 ACCURACY AND PRECISION OF NUCLEAR DENSITY GAUGE

The nuclear density gauge was evaluated to determine its accuracy in determining in-place density. The SMA mixture has a rough surface texture and it was believed that this texture may cause error when using the nuclear gauge.

Density tests were conducted with nuclear density gauges with and without leveling sand and the results compared to the density of cores. As part of the process, the nuclear gauge was calibrated by using a constant calibration factor and by using a calibration factor that was a function of the measured density.

The results indicated that the nuclear gauge can be used to measure density but there is some error. The error can be minimized by using a calibration based on measured density. Using a constant calibration factor provides more error.

The use of leveling sand did reduce the error some but the required effort in most cases can not justify the improved accuracy. The increased error without the sand can be offset by increasing the number of locations tested.

CHAPTER 6 - 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 on-going 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 Superpave Gyratory Compactor should be 100 gyrations for mixtures having aggregates with Los Angeles 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 tend 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 insure 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. 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 a 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)