EVALUATION OF STONE MATRIX ASPHALT (SMA) FOR AIRFIELD PAVEMENTS

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The AAPTP and the FAA thank the Project Technical Panel that willingly gave of their expertise and time for the development of this report. They were responsible for the oversight and the technical direction. The names of those individuals on the Project Technical Panel follow.

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ABSTRACT

A study was conducted to evaluate the potential of stone matrix asphalt (SMA) for use as a surface course on airfield pavements. SMA is a gap-graded mixture with a high (> 70) percent of coarse aggregate. The coarse aggregate forms a stone skeleton, which carries imposed loads, while the inter-particle voids are filled with mastic consisting of mineral filler, fiber, and asphalt binder. Typical binder contents range from 6 to 7.5 percent by total weight of mix. This high binder content offers improved durability while the stone skeleton ensures good rutting resistance.

The objectives of this study were three-fold: 1) evaluate and document the performance of SMA for airfields; 2) develop a design and construction specification for airfields; and 3) develop an implementation plan to expand the use of SMA on airfields, where appropriate. The study was conducted by performing a literature review, collecting data on in-service airfields using SMA, and conducting a laboratory study. The laboratory study was designed to compare the performance of SMA with conventional dense-graded P401 mixes and refine specification parameters for the SMA. Based on the results of the study, SMA offers equal rutting performance and improved resistance to cracking, moisture damage, and fuel spills when compared to conventional mixes.

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Brian Prowell, Ray Brown and Graham Hurley were employed by the National Center for Asphalt Technology when this project was awarded. Dr. Prowell was originally the project principal investigator. Don Watson took over as principal investigator when Dr. Prowell joined Advanced Materials Services. Dr. Prowell was primarily responsible for assembling and writing this report. Mr. Watson, Mr. Hurley, and Dr. Brown were contributing authors. Mr. Watson edited the final report. The laboratory testing was primarily conducted by the National Center for Asphalt Technology. Advanced Materials Services, LLC designed the P401 mixtures and conducted the fuel resistance and deicer tests. The Rutgers Asphalt Pavement Laboratory at Rutgers University conducted the overlay tester tests.

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

1.1 INTRODUCTION

Stone Matrix Asphalt (SMA) was developed in Germany over 30 years ago. Its success has led to its usage throughout Europe on both highway and airfield pavements. In 1990, AASHTO led an European Asphalt Study Tour introduced SMA to the United States (U. S.) SMA has demonstrated good performance on highway pavements in the U. S., but has seen little use on airfields. Recently, there has been resurgence in interest in SMA in the U. S. as a more durable paving option than Superpave mixes.

This project documents the use and performance of SMA on airfields in Europe, Asia, Australia, and the U. S. There are several unique differences between highway and airfield pavements which may affect the performance of SMA on airfields. Specific concerns include potential for acceptability of grooving, foreign object damage (FOD), resistance to deicing chemicals, resistance to fuel spillage, rubber build up, skid resistance, and winter maintenance requirements. Where possible, these concerns were addressed within the research.

1.2 OBJECTIVE

The objectives of this study were three-fold: 1) evaluate and document the performance of SMA for airfields; 2) develop a design and construction specification for SMA for airfields; and 3) develop an implementation plan to expand the use of SMA on airfields, where appropriate.

1.3 RESEARCH APPROACH AND REPORT OVERVIEW

The first tasks of this research were to perform a literature review on SMA and to survey the use of SMA on airfields. Field testing of SMA during construction was not conducted as part of this research. Therefore results from the literature review and survey of use on airfields were used to determine construction specification parameters. The literature review and survey of use on airfields were also used to refine the experimental factors for the laboratory testing. The results of the literature review are presented in Chapter 2. Chapter 2 includes some important information on the long-term performance of SMA mixtures. Chapter 3 presents the findings of an international survey on the use of SMA on airfield pavements. The results of Chapters 2 and 3, in terms of the design of SMA mixtures are summarized at the beginning of Chapter 4. Chapter 4 presents the experimental plan for the laboratory testing, the actual test results and a summary of the findings including a summary comparison with the P401 (dense-graded) control mixes. Chapter 5 presents an implementation plan and recommendations for additional research. Chapter 6 presents the conclusions from this study. The SMA mix design data is presented in Appendix A and the

P401 data in Appendix B. A draft FAA advisory circular for SMA for airfields is presented in Appendix C.

CHAPTER 2

LITERATURE REVIEW

2.1 DESIGN OF SMA MIXTURES

SMA is a gap-graded asphalt mixture with a high percentage (> 70 percent) of coarse aggregate. Gap-graded refers to the fact that SMA mixtures typically have very little material retained on the sand size sieves (e.g. between 2.36 mm and 0.075 mm). SMA is differentiated from dense-graded mixes by its coarse aggregate skeleton, consisting of a limited number of particle sizes, which carries the load. Mastic, consisting of mineral filler, fibers, and asphalt binder, fills the voids between the coarse aggregate skeleton. The percentage by weight passing the 0.075 mm sieve is typically greater than 8 percent. Asphalt contents range from 6 to 7.5 percent by weight of total mix. Fiber, either cellulose or mineral, is generally added to prevent draindown of the binder during construction.

The following section describes the evolution of the SMA design procedure to date, starting with a brief overview of the technology when it was initially brought over from Europe. The section will provide an overview of materials selection and mix design for SMA. The section will highlight areas that need to be addressed specifically for the use of SMA on airfields, as opposed to highway pavements, including:

- What materials properties, e.g. aggregate, binder, and fiber, should be specified for SMA,
- What design criteria, e.g. gradation and volumetric properties, should be specified for designing SMA, and
- What laboratory compaction effort should be used to design SMA.

2.1.1 Early European Experience

"Splittmastixasphalt", commonly called SMA in the United States, was developed in Germany during the 1960's as a durable asphalt mixture which was resistant to studded tire wear and permanent deformation. In 1990, the American Association of State Highway and Transportation Officials (AASHTO) European Asphalt Study Tour brought back the German asphalt mix technology known as "Splittmastixasphalt." Two country's specifications, Germany and Sweden, primarily influenced the early U.S. specifications. Germany tended to primarily use 8 and 11 mm nominal maximum aggregate size (NMAS) SMA, with limited use of 5 mm NMAS SMA. Sweden specified a 16 mm NMAS SMA. Germany had previously used 16 mm NMAS SMA, but had mainly discontinued its use by the early 1990's. It should be noted that one of the major reasons that Sweden was using SMA was to resist studded tire wear. Studded tire use was reported to be 50 percent in the Götenborg area. The larger NMAS was reported to be more resistant to studded tire wear.

Highly durable coarse aggregates, such as granite, basalt, gabbro, diabase, gneiss, phorphory, and quartzite are used in Europe (*1*). Aggregates for SMA should be cubical, provide a rough texture, and be resistant to breakdown of the contact points under load. Aggregates used in SMA are generally 100 percent crushed. Rounded gravel particles are required to be double crushed in Germany. Double crushing is interpreted to mean two or more crushed faces. German specifications require a minimum of 90 percent crushed coarse aggregate.

In Germany, aggregates for HMA (including SMA) are tested to ensure that not more than 20 percent of the particles exceed a length to thickness ratio of 3:1. Stuart (*1*) noted that the German's indicated that some elongated or other irregularly shaped particles are desirable to improve aggregate interlock. Flat and elongated particles are considered undesirable because they can lead to variability in volumetric properties in the laboratory (particularly if the percentage of flat and elongated particle varies), can break during compaction exposing uncoated faces, and may align themselves during compaction, possible altering stability or causing bleeding.

As noted previously, aggregate durability, or hardness, is an important consideration in the design of SMA. Excessive aggregate breakdown during mixing and compaction could alter the SMA gradation, potentially causing a loss of stone-on-stone contact between the coarse aggregate particles. Secondly, the contact points between the coarse aggregate particles provide stability to the mixture. If the aggregate is too soft or brittle, these points could break down under load. The Los Angeles (L.A.) Abrasion Test, ASTM C131, is typically used to characterize aggregate breakdown during construction in the U. S. However, the L.A. Abrasion test was not used to characterize aggregate breakdown by either Germany or Sweden when the SMA concept was brought to the U.S. The Germans use the Schlagversuch Impact Test to assess aggregate breakdown (*1,2*). Sweden used abrasion tests for both the aggregate and mixture and an impact test for the aggregate (*1,2*).

The European design gradation bands for SMA varied by NMAS and were fairly wide. Stuart (*1*) noted that the Germans believed that both good and poor performing mixes could be designed within their design limits. Aggregate is fractionated in Germany, allowing precise control of the gradation. Scherocman (*3*) notes that early SMA projects in the U. S. often used the "30:20:10 rule" for the percent passing the 4.75 mm, 2.36 mm and 0.075 mm sieves, respectively.

At the time of the 1990 European Asphalt Study Tour, the asphalt binders (or bitumen as they are called in Europe) were typically 65, 80, or 85 penetration grade binders (*1*). Penetration grade binders are still used in Europe. In 1990, these were approximately equivalent to AC-10 or AC-20 viscosity grades. Today this would be approximately equivalent to a PG 58-28 or a PG 64-22. The softer (80 or 85) penetration grades were typically used in northern Europe and the stiffer (65) penetration grade was used in southern Europe. Sweden and Norway have reportedly used binders as soft as 160 to 220 penetration grade. These binders were subject to attack by potassium acetate and potassium formate used as deicing chemicals (*4*). Research has indicated that polymer modified PG 64-28 is resistant to deicing chemicals (*5*).

Stabilizing additives are typically added to SMA to prevent binder draindown during storage, hauling and laydown. Excessive draindown, in essence a form of segregation, can result in so called “fat spots”, or areas with apparent bleeding of asphalt on the surface immediately after construction. Fat spots or the thick film of asphalt binder on the coarse aggregate particles may initially cause reduced skid resistance.

Cellulose and mineral fibers are the most common type of stabilizer. Typically, fibers are added at 0.3 percent by weight of total mix. The Schellenberg Bitumen Segregation Test can be used to assess draindown. A 1000 g sample of SMA is placed in a beaker at a temperature of 170 °C (338 °F) for one hour. At the end of the hour, the mixture is dumped from the beaker and the material is reweighed. The weight retained in the beaker (binder that has drained off the aggregate) is divided by the original weight of the sample and this is reported as the drainage. Losses less than 0.2 percent indicate that no segregation should occur, however values up to 0.3 percent are considered acceptable (*1*). Mineral fibers are slightly less absorptive than cellulose fibers and therefore may require a somewhat higher dosage. The use of fibers increases measured voids in mineral aggregate (VMA) and asphalt demand (optimum binder content). Fibers serve no real purpose after the mix is compacted in-place.

The European Asphalt Study Tour (*6*) noted that the Marshall mix design method was (and still is) used in Sweden and Germany for the design of SMA mixtures. The design is supplemented by the experience of the contractor such that mix designs have been referred to as “recipes.” Samples are compacted with 50 blows on each face. An increased compaction effort (higher blows) is not recommended as it may result in additional fractured aggregate in the sample with little increase in density. Stuart (*1*) noted that SMA mixtures in Sweden were generally designed at 3 percent air voids for high traffic pavements and closer to 2 percent air voids for low traffic pavements. German SMA mixtures were typically designed at 3 percent air voids with a tolerance of ± 1 percent. German specifications required a binder content of 6.5 to 7.5 percent and Swedish specifications targeted 6.6 percent for an 11 mm SMA. Stability and flow measurements were not routinely used for SMA.

Information on specific field construction guidelines from the 1990 European Asphalt Study Tour was limited (*6*). Dry mixing time was increased for batch plants to disperse the fibers. In-place air voids were targeted as less than 6 percent (greater than 94 percent of G_{mm}). The use of vibratory rollers should be limited to the first few passes and the amplitude should be low and the frequency high. The use of rubber tire rollers was not recommended. In Germany, an overlay thickness of 25 to 50 mm was recommended for the German 11 mm (U.S. 12.5 mm NMAS) mixture.

It was noted that SMA provides good skid resistance over time. However, the initial skid resistance, typically for the first month of service, could be reduced until the thick film of asphalt binder wears off the coarse aggregate particles. Crushed sand, with particle sizes ranging from 1 to 3 mm, may be rolled into the hot mat to improve skid resistance before the thick binder film wears off the coarse aggregate. The crushed sand is sometimes pre-coated with binder.

2.1.2 FHWA SMA Technical Working Group

In 1994, the Federal Highway Administration (FHWA) SMA Technical Working Group (TWG) developed model Material and Construction Guidelines for SMA (*7*). The guide specifications recommended a maximum L.A. Abrasion loss of 30 percent. A single gradation was specified that corresponded to the European 16 mm SMA with 100 percent passing the 19.0 mm sieve and 85 to 95 percent passing the 12.5 mm sieve (Table 2.1). Asphalt grades for heavy duty pavements were recommended with notes that AC-20 (approximately PG 64-22) generally corresponded to what was used in Europe. The design laboratory compaction effort was 50 Marshall blows on each face. The design volumetric properties are summarized in Table 2.2.

The National Center for Asphalt Technology (NCAT) developed a test method to measure draindown (binder segregation) potential (*8*). The test uses a 100 mm diameter basket constructed of wire mesh with 6.3 mm openings. The test is performed in a similar fashion as the Shellenberg test except that the sample of known mass is placed in the wire basket over a tared pie plate at the specified temperature for a period of one hour. The mass of binder retained on the pie plate divided by the original sample mass is expressed as the percent draindown. This method was later adopted as ASTM D 6390.

TABLE 2.1 FHWA SMA TWG Recommended Gradation

Sieve Size, mm (in)	Percent Passing
19.0 (3/4)	100
12.5 (1/2)	85 – 95
9.5 (3/8)	75 Max.
4.75 (No. 4)	20 – 28
2.36 (No. 8)	16 – 24
0.600 (No. 30)	12 – 16
0.300 (No. 50)	12 – 15
0.075 (No. 200)	8 – 10
0.020 (No. 635)	< 3

TABLE 2.2 FHWA SMA TWG Recommended Volumetric Properties

Property	Design Range
Marshall Compaction	50 Blows on each face
Air Voids, %	3 – 4
Asphalt Content, %	6.0 Min.
VMA, %	17 Min.
Stability, N (lbs)	6200 (1400) suggested Min.
Flow, 0.25 mm (0.01 in)	8 – 16
Draindown, %	0.3 Max. (1 hour)

2.1.3 NCHRP 9-8 Designing Stone Matrix Asphalt Mixtures

NCHRP 9-8, “Designing Stone Matrix Asphalt Mixtures,” was conducted by NCAT between 1994 and 1998. The goal of this study was to develop a repeatable mixture design method suitable for either Marshall or Superpave gyratory compaction. The study was conducted in two phases where a preliminary design procedure was developed and then refined in Phase I, and then the design procedure was field verified in Phase II.

The degree of breakdown during laboratory compaction was evaluated in Phase I with respect to laboratory compaction. Figure 2.1 shows the relationship between the coarse aggregate L.A. Abrasion loss and the measured breakdown on the 4.75 mm sieve after laboratory compaction. The Marshall method resulted in greater aggregate breakdown. The coefficient of determination (R^2) values indicate a reasonable correlation between L.A. Abrasion loss and breakdown during laboratory compaction for both compaction methods. Based on this analysis, the authors concluded that the L.A. Abrasion loss specification of 30 percent max. developed by the FHWA SMA TWG was reasonable and that the use of aggregate with higher L.A. Abrasion loss values would result in excessive breakdown. (*9*).

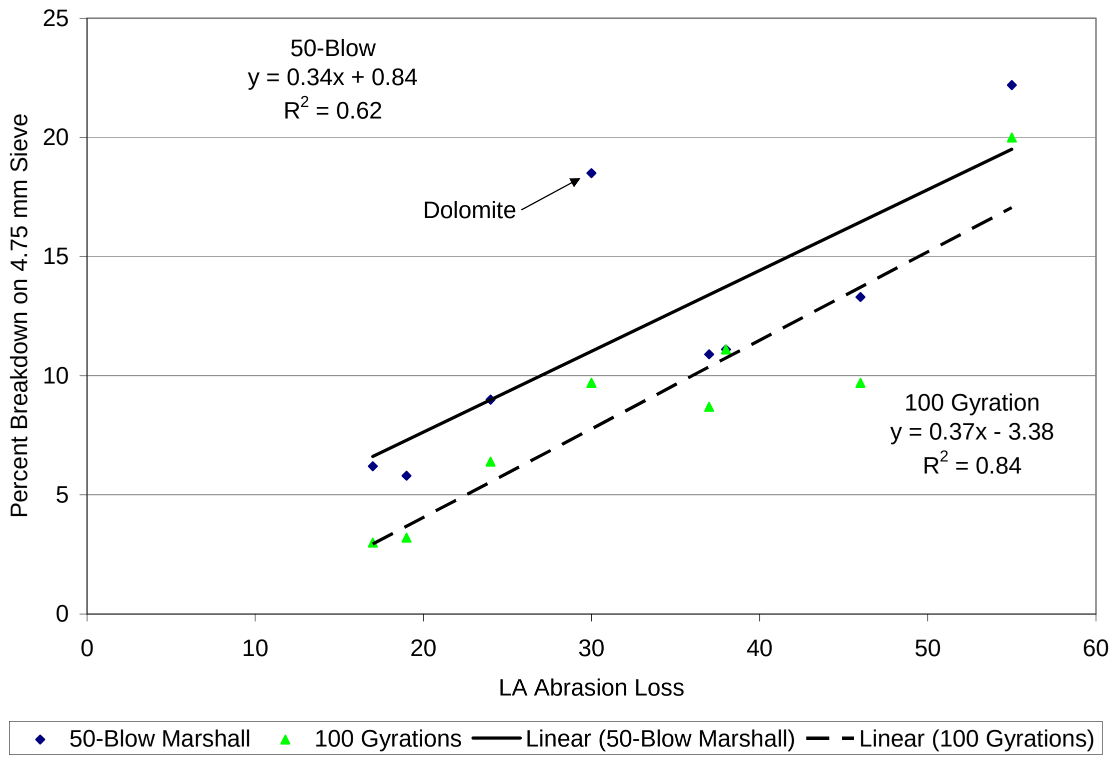

FIGURE 2.1 Aggregate Breakdown as a Function of L.A. Abrasion.

In Phase II of the study, the recovered gradation from 50-blow Marshall, 100 gyrations and field compaction samples were compared using Analysis of Variance (ANOVA) (Table 2.3). Although significant differences were noted in four of eight cases, both laboratory compaction methods generally approximated breakdown in the field.

TABLE 2.3 ANOVA to Compare Aggregate Breakdown (*10*)

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

Excessive aggregate breakdown can make it difficult to meet the minimum VMA requirements (minimum 17). Finally, the authors (*10*) concluded that, "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."

Testing was conducted on a variety of mineral fillers at different concentrations. The effect of the fillers on the resulting mastic (binder, fiber, and filler) was tested using the original and rolling thin-film oven (RTFO) aged residue in the dynamic shear rheometer (DSR) and the RTFO followed by pressure aging vessel aging in the bending beam rheometer (BBR). These same tests are used to characterize binders in the performance grading (PG) system. The data indicated that the mortars were typically five times stiffer than the asphalt binders.

The Rigden voids test was identified as a good screening tool for mineral fillers. The Rigden voids test requires inexpensive testing equipment compared to the Superpave binder testing equipment. Mineral fillers with Rigden voids in excess of 50 percent were identified as causing the mastic to be excessively stiff and difficult to work. No correlation between the percent passing the 0.020 mm sieve and the mastic performance was found. Therefore, it was recommended that gradation specifications on the percent passing the 0.020 mm sieve be eliminated.

As noted previously, the European design gradation specifications for SMA were relatively liberal. Researchers (*11,12*) were concerned that the wide gradation bands did not guarantee that the mixture would have a coarse aggregate skeleton. Without a coarse aggregate skeleton to carry the load, the high binder content of an SMA mixture could potentially make it susceptible to permanent deformation. A methodology was developed to ensure stone-on-stone contact (*13,14*). The coarse aggregate fraction is dry-rodded in three lifts according to AASHTO T19 (ASTM C29). The coarse aggregate fraction is considered to be the blended material retained on the 4.75 mm sieve for 12.5 mm NMAS and larger SMA mixtures. The voids in coarse aggregate in the dry rodded condition is calculated according to Equation 1:

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

where,

G_{CA} = dry bulk specific gravity of the coarse aggregate fraction determined according to ASTM C127,
γ_w = density of water (999 kg/m^3), and
γ_s = Unit weight of coarse aggregate in the dry-rodded condition (kg/m^3).

In a similar manner, the voids in coarse aggregate of the compacted SMA mixture can be calculated according to Equation 2:

$$VCA_{Mix} = 100 - (G_{mb} \div G_{CA})P_{CA} \tag{2}$$

where,

G_{mb} = bulk specific gravity of the compacted SMA sample measured according to AASHTO T166 (ASTM D 2726), and
P_{CA} = percent coarse aggregate (percent retained on the 4.75 mm (No. 4) sieve for SMA mixtures with NMAS greater than 12.5 mm (1/2 inch)).

The theory is that the coarse aggregate in the dry-rodded condition, without any fine aggregate or mastic, represents a stone (coarse aggregate) skeleton, as there is nothing to hold the coarse aggregate particles apart. Then the VCA_{Mix} must be less than the VCA_{DRC}. This assures that the coarse aggregate particles are still in contact with one another and have not been pushed apart by either fine aggregate particles or mastic. This procedure has been adopted as AASHTO PP-41, *Standard Practice for Designing Stone Matrix Asphalt (SMA)* (*15*).

A study was conducted to determine how VMA and VCA changed as a function of the percent passing the 4.75 mm (No. 4) sieve (*8*). Two aggregate sources were used, a gravel and a limestone. The mixes were designed with various percentages passing the 4.75 mm (No. 4) sieve while holding the filler content constant at 10 percent. Optimum asphalt content was determined for each mixture at 3 percent air voids using a 50-blow Marshall compactive effort. VMA and VCA as a function of the percent passing the 4.75 mm (No. 4) sieve are presented in Figures 2.2 and 2.3 for the gravel and limestone aggregates, respectively. Based on Figures 2.2 and 2.3, the VCA_{Mix} becomes less than the VCA_{DRC} somewhere close to 30 percent passing the 4.75 mm (No. 4) sieve for both aggregate sources. However, the minimum VMA requirements are not achieved for the limestone source until the percent passing the 4.75 mm (No. 4) sieve is reduced to approximately 19 percent. The L.A. Abrasion loss for the aggregate sources are not given in the report. VMA alone is not a suitable indicator of stone-on-stone contact.

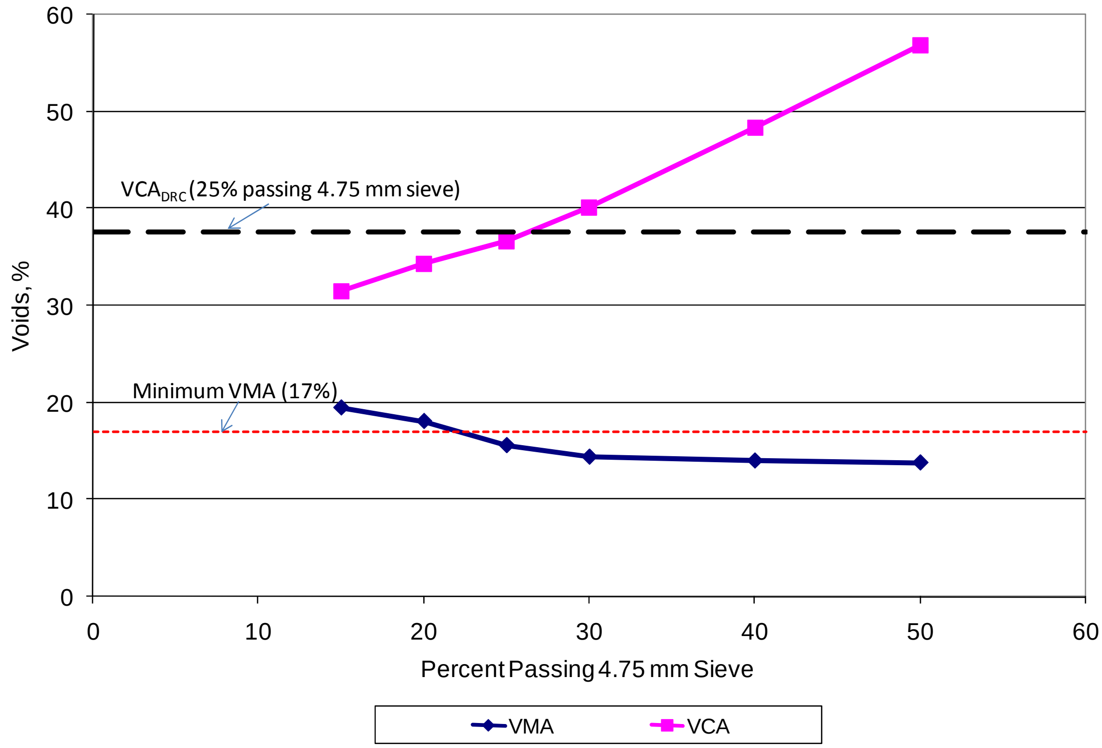

FIGURE 2.2 VMA and VCA versus Percent Passing 4.75 mm (No. 4) Sieve for Gravel Aggregate (after *8*).

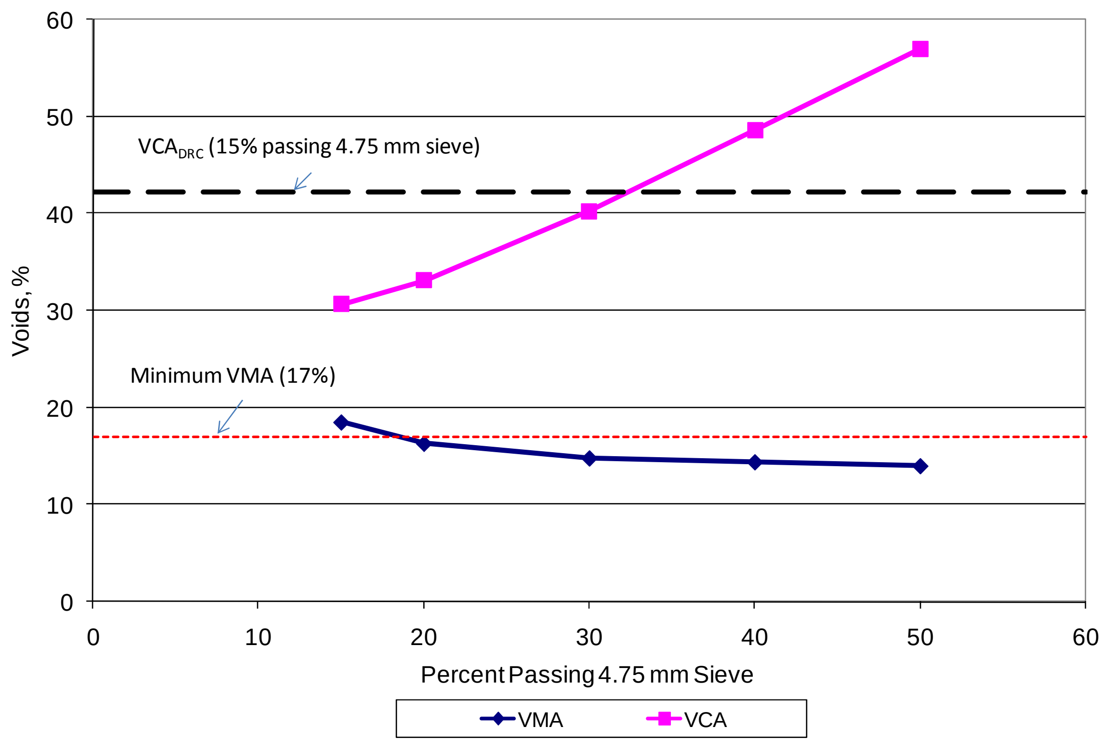

FIGURE 2.3 VMA and VCA versus Percent Passing 4.75 mm (No. 4) Sieve for Limestone Aggregate (after *8*).

NCHRP 9-8 recommended increasing Performance Grade (PG) one or two high temperature grade bumps above the binder recommended to meet the climatic conditions for a project (*10*). The climatic binder grade for projects constructed in North America can be determined from a program developed by FHWA entitled "LTPPBind" (*16*). High temperature grade bumps help to ensure the resistance to permanent deformation under slow moving or turning traffic.

A significant portion of the NCHRP 9-8 research effort was used to determine the appropriate laboratory compaction effort for the Superpave gyratory compactor (SGC). Based on European practice, the 50-blow Marshall compaction effort was selected as the standard. The results from the SGC were compared to the 50-Blow Marshall results. In Phase II of NCHRP 9-8, samples were collected from eleven field projects. Six samples were taken from each project. Three replicates each were compacted with the 50-blow Marshall and 100 gyration SGC compaction effort in the field for each sample. The sample density for the SGC was back calculated at lower numbers of gyrations to determine the numbers of gyrations necessary to match the 50-Blow compactive effort (*17*). The overall data is shown in Figure 2.4. Based on Figure 2.4, approximately 80 gyrations with the SGC are required to match the 50-Blow Marshall compactive effort. However, there is a great deal of scatter in the data. Additional analyses indicated that the relationship between Marshall and SGC compaction varied as a function of the L.A. Abrasion loss. Figure 2.5 shows the relationship between number of gyrations and G_{mb} ratio as a function of L.A. Abrasion loss. Based on Figure 2.5, a design compactive effort of 100 gyrations was recommended for mixtures with coarse aggregate having an L.A. Abrasion loss less than 30 percent although the data suggests that a lower compactive effort could be used. A design compactive effort of 70 gyrations was recommended for SMA mixtures having coarse aggregate with an L.A. Abrasion loss greater than 30 percent.

Permeability tests were conducted during both Phases I and II of NCHRP 9-8. Permeability tests were conducted using both the laboratory and field falling head devices. Permeability and water absorption test results indicated that permeability increased rapidly above six percent air voids (*18*). SMA mixes were generally found to be more permeable than coarse-graded Superpave mixes at the same void content and much more permeable than fine-graded Superpave mixes. Permeability at a given air void content tends to increase as a function of increasing NMAS. Based on this data, a maximum in-place air void content of six percent (94 percent G_{mm}) was recommended. Field permeability testing in Phase II confirmed the need for in-place air void contents less than six percent, except for 4.75 mm (No. 4) NMAS SMA mixtures for which up to nine percent in-place air voids would be acceptable (*10*).

Tensile Strength Ratio (TSR) testing was conducted according to AASHTO T283. The target sample air voids were adjusted to 6 ± 1 percent [the air void tolerance was higher at the time this study was conducted]. A freeze-thaw cycle was not used [a freeze-thaw cycle was not required at the time this study was conducted]. The TSR values for SMA mixes were typically lower than the TSR for corresponding dense-graded mixtures produced with the same aggregate source. This does not indicate that SMA mixtures are susceptible to moisture damage, but does indicate that the acceptance criteria should be lower. A minimum TSR value of 0.70 is recommended for SMA mixtures (*10*).

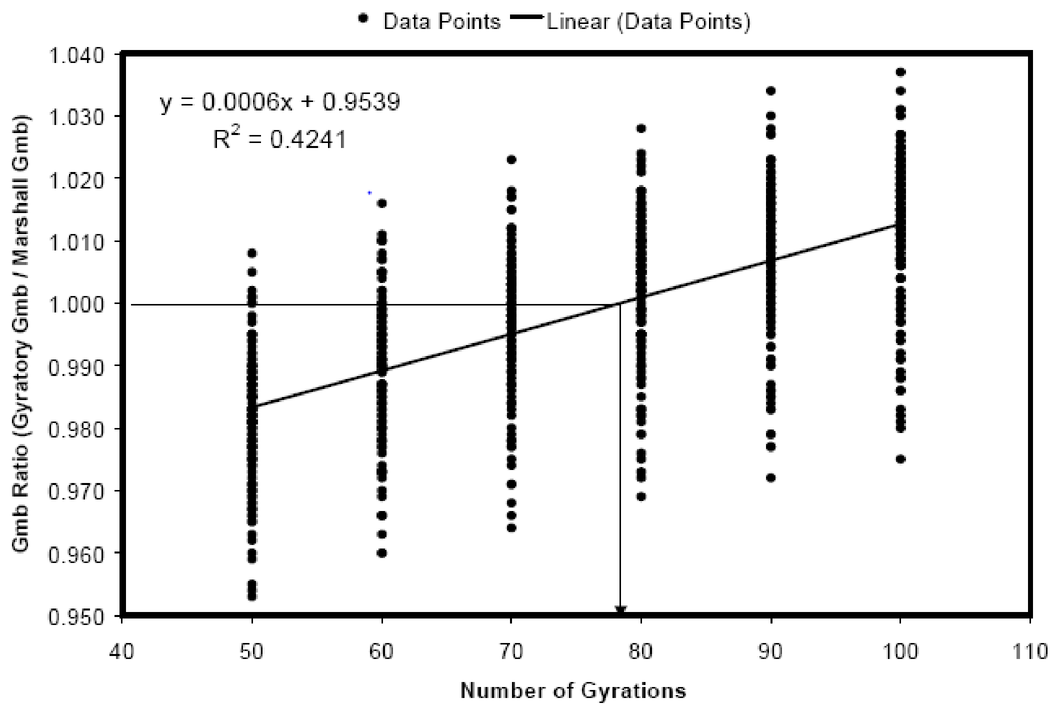

FIGURE 2.4 SGC/ Marshall G_{mb} Ratios for All Data (*17*).

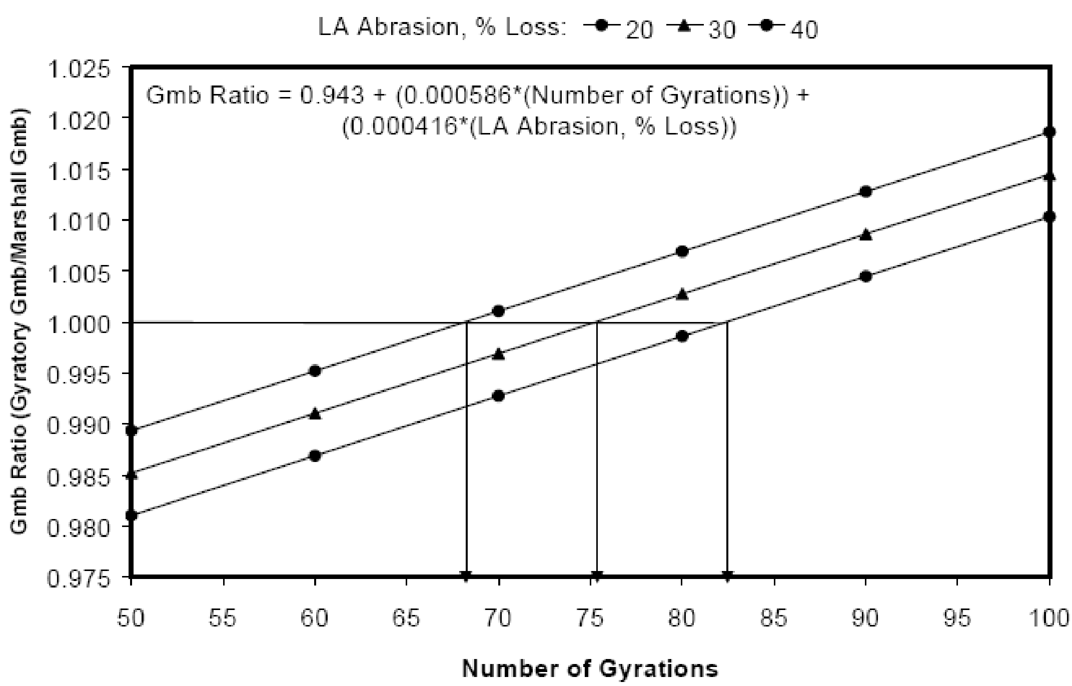

FIGURE 2.5 G_{mb} Ratio as a Function of Gyration Level and L.A. Abrasion Loss (*17*).

2.1.4 Current U.S. Specifications for the Design of SMA

The SMA specifications developed by the FHWA SMA TWG were revised based on the research conducted as part of NCHRP 9-8 and interim field experience. A provisional specification for SMA was developed by AASHTO in 1999. A provisional mix design practice for SMA was developed by AASHTO in 2000. A Unified Facilities Guide specification for SMA for airfields was developed in 2004 and significantly revised in 2006. The following section provides a summary of these two specifications. These two specifications represent an overview of what would be recommended for the design of SMA on a national basis.

2.1.4.1 Aggregate Properties

The aggregate properties required in the two specifications are shown in Table 2.4. Both specifications limit the L.A. Abrasion loss to a maximum of 30 percent. The flat and elongated particle requirements are also identical for both specifications. The AASHTO specification notes that the requirement for flat and elongated particles applies to the design aggregate blend, not the individual coarse aggregate stockpiles. The AASHTO specification determines the percentage of fractured faces using ASTM D5821 and requires a minimum of 90 percent of particles with two or more fractured faces. The Unified Facilities specification determines the percentage of fractured faces according to Corp of Engineers test method CRD-C 171 and requires 100 percent of the coarse aggregate particles to have two or more crushed faces. Such a specification virtually eliminates crushed gravel sources unless the

TABLE 2.4 Aggregate Requirements for SMA

Test	AASHTO (*15*)	Unified Facilities (*19*)
Coarse Aggregate		
L.A. Abrasion, % loss	30 max.	30 max.
Flat and Elongated Particles, % 3:1	20 max.	20 max.
Flat and Elongated Particles, % 5:1	5 max.	5 max.
Water Absorption, %	2.0 max.	2.0 max.
Soundness loss, % (5 cycles)	15/20[1]	NA/18[1]
Crushed Content, % one face/two faces	100/90	NA/100
Fine Aggregate		
Soundness loss, % (5 cycles)	15/20[1]	NA
Sand Equivalent Value, %	NA	45 min.[2]
Uncompacted Voids Content, Method A, %	NA	45.0 min.
Water Absorption, %	NA	2.0 max.
Liquid Limit, %	25	25
Plasticity Index, %	Non-plastic	Non-plastic

[1]Sodium and Magnesium sulfate soundness, respectively. Only one type needs to be run.
[2]Each stockpile; NA = No Data Available

gravel cobbles are exceptionally large prior to crushing. Both specifications require the fine aggregate to be crushed manufactured fines. In addition, the Unified Facilities specification requires the fine aggregate to meet a minimum uncompacted voids (45 percent) and sand equivalent value (min. 45 percent). Some manufactured fines produce uncompacted voids less than 45 percent (*2*).

2.1.4.2 Mineral Filler

The AASHTO specification recommends that mineral filler consist of finely divided mineral matter such as crusher fines or fly ash. The plasticity index [method not specified, but most likely AASHTO T90] should not be greater than 4. The Unified Facilities specification requires the mineral filler to meet the requirements of ASTM D 242. The AASHTO specification recommends that mineral fillers with modified Rigden voids greater than 50 percent not be used in SMA. The modified Rigden voids test is described in the National Asphalt Pavement Association's Information Series No. 127.

2.1.4.3 Asphalt Binder

PG binder grades are specified (where possible) by both specifications. PG grades are specified based on their anticipated in-service temperature range. The first number in the PG grade represents the highest expected average pavement temperature over a seven-day period to resist permanent deformation or rutting. The second number in the PG grade represents the lowest expected pavement temperature to resist low temperature cracking. Both pavement temperatures are generally selected to provide 98 percent reliability.

AASHTO specifies the grade that is appropriate for the climate and traffic loading conditions, selected according to AASHTO M323. High temperature "bumps" are applied to the high temperature climatic grade for slow or standing traffic (one and two grade bumps, respectively) or design traffic in excess of 30 million equivalent single axle loads (ESALs). The Unified Facilities specification recommends the PG grade used by the local state highway agency for traffic less than 10 million ESALs with a two-grade high temperature bump. Caution is recommended if climatic data recommends use of a low temperature grade warmer than -22 °C (-8 °F). This may be especially important for airfield pavements, which may be more susceptible to thermal fatigue cracking due to the large paved expanse and limited traffic repetitions.

2.1.4.4 Stabilizing Additives

Either cellulose or mineral fibers are typically added to SMA to prevent draindown or segregation of the binder during construction. The AASHTO specification says that a stabilizer may be added to the mix and recommends a dosage rate for cellulose fibers of approximately 0.3 percent by total weight of mix. The literature review noted that the required dosage rate for mineral fibers is typically higher owing to the fact that they add surface area, but do not readily absorb binder. The Unified Facilities specification requires the addition of either cellulose or mineral fibers. Both specifications have identical requirements for cellulose and mineral fibers. There has been question as to whether or not fibers need to meet these exact specifications in order to be effective in reducing draindown.

The origin of these specifications has not been clearly identified although they are generally attributed to German and Swedish requirements.

2.1.4.5 Design Gradation

As noted previously, the original FHWA SMA TWG gradation specification (*7*) was approximately equivalent to the 16 mm SMA used in Sweden. NCHRP 9-8 provided design gradation ranges for 4.75, 12.5, 19.0 and 25.0 mm (No. 4, ½, ¾, and 1.0 inch) NMAS SMA mixtures (*20*). The AASHTO specification includes design ranges for 9.5, 12.5 and 19.0 mm (3/8, 1/2, and 3/4 inch) NMAS SMA. The Unified Facilities specification includes only a single gradation. It is believed that a design gradation between an 11 and 12.5 mm NMAS is the most appropriate size for airfields. This is based on considerations related to permeability, macro-texture, and the propensity for foreign object damage.

The design gradation ranges for the various specifications are shown in Table 2.5. The German 0/11 and FHWA TWG represent the two extremes with the German 0/11 specification being finer. The United Facilities specification most closely approximates the German 0/11 specification, but allows a wider design range, particularly on the 4.75 and 2.36 mm (No. 4 and No. 8) sieves. There are two potential reasons for allowing a wider design range: first, aggregate in Germany is fractionated as compared to blended stockpile sizes most commonly found in the U.S.; and secondly, somewhat lower quality aggregates have been used to produce SMA in the U.S. Typically available stockpile gradations should be considered when developing gradation specifications for SMA produced in the U.S. However, recent advances in portable screening equipment make it feasible for the contractor to fractionate aggregate for SMA on-site. Aggregates with higher L.A. Abrasion losses have been

TABLE 2.5 Design Gradation Ranges for 12.5 mm (1/2 inch) NMAS SMA

Sieve Size, mm (in)	Percent Passing				
	German 0/11[1]	FHWA TWG[2]	NCHRP 9-8 12.5 mm	AASHTO 12.5 mm	Unified Facilities
19.0 (3/4)	100	100	100	100	100
12.5 (1/2)	93-100	85-95	90-100	90-100	90-100
9.5 (3/8)	<80	75 max.	26-78	50-80	50-85
4.75(No. 4)	29-39	20-28	20-28	20-35	20-40
2.36 (No. 8)	22-29	16-24	16-24	16-24	16-28
1.18 (No. 16)	-	-	13-21	-	-
0.600 (No. 30)	14-19	12-16	12-18	-	-
0.300 (No. 50)	-	12-15	12-15	-	-
0.075 (No. 200)	8.7-12.6	8-10	8-10	8-11	8-11
0.020 (No. 635)	-	< 3.0	-	-	-

[1]German 0/11 SMA is specified using the 11.2, 8, 5, 2, and 0.09 mm sieves. The U.S. sieve sizes have been interpolated using this data.
[2]FHWA TWG is 16 mm NMAS.

successfully used to produce SMA in the U.S. However, the use of aggregates with higher L.A. Abrasion loss result in a greater degree of aggregate breakdown during both laboratory and field compaction. This can make it more difficult to achieve the desired volumetric properties, particularly minimum VMA. VMA can be increased by producing a mixture which is coarser (lower percent passing the 4.75 mm (No. 4) sieve). Hence lower percents passing the 4.75 mm (No. 4) sieve have been adopted for U.S. SMA mixes as compared to the German specifications.

2.1.4.6 Volumetric Properties

The AASHTO and Unified Facilities specifications for mixture properties are summarized in Table 2.6. The laboratory compaction effort needs to be considered when evaluating the specified volumetric properties. Although NCHRP 9-8 recommended the use of either Marshall or gyratory compaction, the AASHTO specification adopted only gyratory compaction. A design compactive effort of 100 gyrations was recommended for aggregates with L.A. Abrasion loss less than 30 percent and a design compactive effort of 75 gyrations for aggregates with L.A. Abrasion loss greater than 30 percent. By comparison, Unified Facilities specifies a hand Marshall hammer be used for design of SMA for airfield pavements. A calibration between the hand Marshall hammer and an automatic Marshall hammer can be developed for the specific SMA mix for production testing. The SGC may be used for roadways under the United Facilities specification.

Some differences from the European practice, discussed previously, should be noted in Table 2.6. Both Germany and Sweden target 3.0 percent air voids when designing SMA. For low volume roads, 2.0 percent air voids is targeted in Sweden. European practice and the research conducted as part of NCHRP 9-8 emphasize the importance of in-place air voids less than 6 percent (> 94 percent G_{mm}). Higher laboratory compaction efforts or design air void contents may make it difficult to achieve the required in-place density. When a strong aggregate skeleton is formed there is less than 6 percent air voids in the in-place pavement.

TABLE 2.6 Mixture Properties

Property	AASHTO MP8 (*15*)	Unified Facilities (*19*)
Air Voids, %	4.0[2]	3.0-4.0
VMA, %	17.0 min.	17.0 min.
VCA_{Mix}, %	< VCA_{DRC}	NA[3]
TSR	0.80	0.75
Draindown, %[1]	0.30 max.	0.3 max.
Asphalt Binder Content, %	6.0 min.[4]	NA

[1]Determined at the anticipate production temperature.
[2]For low volume roadways or cold climates air void contents less than 4.0 percent can be used. Air voids should not be less than 3.0 percent.
[3]The mix design is to be completed according to AASHTO MP8 and PP41 and the VCA_{Mix} and VCA_{DRC} are to be reported therefore the requirement is implied.
[4]Guidelines are presented in AASHTO PP41 for mixes with varying aggregate G_{sb}. Higher aggregate gravities, in excess of 2.75, may allow lower asphalt contents.

Once the mixture has been compacted to the point where a coarse aggregate skeleton forms in the in-place pavement, additional densification is only possible through aggregate breakdown. Therefore, there must be sufficient mastic in the mixture to fill the voids once aggregate interlock is achieved.

VMA is calculated in both specifications using the aggregate dry bulk specific gravity (G_{sb}). Although most specifications specify a minimum VMA of 17 percent, lower design values have been successfully used (*3*). Both specifications use the VCA tests developed as part of NCHRP 9-8 to ensure stone-on-stone contact. However, the AASHTO specification explicitly includes criteria that VCA_{Mix} be less than the VCA_{DRC}, whereas the criteria is implicit in the Unified Facilities specification.

AASHTO MP8 includes a specification for minimum binder content of 6.0 percent. The minimum binder content may be adjusted for aggregates with combined bulk specific gravities different from 2.75. The minimum design asphalt content would typically be decreased for aggregates having very high gravities since SMA is proportioned by mass. An aggregate with a higher specific gravity has less volume for a given mass of material and therefore less surface area to coat. However, even the AASHTO minimum asphalt content is lower than European practice. For 11 mm (7/16 inch) SMA, Germany specifies 6.5 to 7.5 percent binder and Sweden 6.6 percent binder. As discussed previously, the original European SMA mixes were based more on recipe or experience as well as volumetric criteria.

Draindown of the binder can occur while hauling the SMA. During design, draindown testing is required at the anticipated production temperature by both specifications. Draindown is tested according to the methodology developed by NCAT using a mesh basket (AASHTO T305 or ASTM D6390). In addition, the Unified Facilities specification requires the addition of fibers, regardless of the draindown. The use of cellulose fibers has been shown to increase VMA and the resulting design asphalt content. Although draindown can be minimized by avoiding excessive production temperatures and over asphalted mixes, most practitioners agree that the inclusion of fibers is good insurance against draindown or binder segregation.

NCHRP 9-8 noted that SMA mixes tended to have reduced TSR values and therefore recommended a minimum TSR value of 0.70. Both the AASHTO and Unified Facilities specifications require higher TSR values, 0.80 and 0.75, respectively.

2.1.5 Additional Research on SMA Design

Additional research has been conducted on SMA design since the completion of NCHRP 9-8. This section discusses research related to three key areas: L. A. Abrasion requirements, laboratory compaction effort, and field density/permeability.

2.1.5.1 Research Related to L.A. Abrasion Loss

For best performance from a rutting (or resistance to studded tires) standpoint, SMA should be produced with hard aggregates. However, if excessive breakdown does not occur during field compaction, SMA has been successfully produced using aggregates with higher L. A. Abrasion losses. This would potentially allow SMA to be constructed on airfields using locally available materials in a more economical manner. This cost savings might allow airfields to take advantage of SMA's improved resistance to cracking in areas which would not be possible (from an economic standpoint) if aggregates needed to be imported.

Stuart (*1*) recommended a maximum L.A. Abrasion loss of 40 percent. Georgia DOT has been one of the leaders in the use of SMA in the U.S. on highways. Georgia DOT has a long history of using coarse aggregates with L.A. Abrasion loss greater than 30 percent. The coarse aggregate used on Georgia DOT's first SMA project had an L.A. Abrasion loss of 35 percent (*21*). In 1992, Georgia constructed Test sections of SMA to evaluate its performance when applied as an overlay on Portland cement concrete pavement (*21*). The L.A. Abrasion loss of the coarse aggregate used on this project was 41 percent. Georgia DOT's current specifications allow L.A. Abrasion loss up to 45 percent for SMA (*22*).

Cross (*23*) conducted a study for Kansas DOT on aggregate specifications for SMA. Based on aggregate breakdown that occurred during compaction, Cross (*23*) concluded that the L.A. Abrasion loss would need to be less than 16 percent to produce SMA in Kansas, based on the degree of breakdown that occurred during the mix design process. However, this was based on the fact that Kansas DOT's acceptance practices at the time were based on recovered gradations from field compacted material. Because the aggregates would be expected to breakdown under the roller during compaction of the SMA, this would result in the recovered sample being out of specification. The L.A. Abrasion loss of aggregates tested in Kansas ranged from 22 to 46 percent. Therefore, a specification limiting L.A. Abrasion loss to 16 percent would be unreasonable. Missouri DOT allows LA Abrasion losses up to 35 percent (*24*).

Virginia Department of Transportation allows coarse aggregates with an L.A. Abrasion loss up to 40 percent in SMA (*25*). Wisconsin DOT allows aggregates with an L.A. Abrasion loss up to 45 percent in SMA. Projects were constructed between 1992 and 1994 to evaluate the performance of SMA (*26*). Agency records indicated three primary regions of aggregate hardness: Region One, in the northern half of the state generally characterized by igneous gravels with L.A. Abrasion loss values between 15 and 30; Region Two, in the southwestern part of the state with softer dolomite or gravels with L.A. Abrasion loss values ranging between 30 and 60; and Region Three, in the southeastern part of the state generally consisting of limestone/dolomite or crushed gravel sources with L.A. Abrasion loss values between 20 and 40. Six projects were selected for the study, two in each region. Three additional "adjunct" projects were also included in the study. Aggregate hardness was identified as the factor most correlated to reflective cracking. Region Two, with the softer aggregate, averaged 62 percent reflective cracks after five years, compared to Region One, with the hardest aggregates, which averaged 19 percent reflective cracks after five years. Rutting was negligible in all three regions after five-years of traffic.

Xie and Watson (*27*) reported on a laboratory study conducted for FHWA on the degradation of SMA mixtures. The study evaluated SMA mixtures produced with five aggregate sources with L.A. Abrasion loss values ranging from 16.6 to 36.4 percent and flat and elongated particle counts (3:1 ratio by mass) ranging from 12.8 to 37.7. Mix designs were produced using 9.5, 12.5, and 19.0 mm (3/8, 1/2, and 3/4 inch) NMAS aggregates. A constant gradation was used for all of the aggregate sources. Samples were compacted using both a 50-Blow Marshall (calibrated to a hand hammer, 59 blows actual), and 100 gyration SGC compaction efforts. Aggregate breakdown was measured on the critical or break point sieve based on samples extracted using the ignition furnace (calibrated using loose mix for any breakdown occurring due to the extraction procedure). The critical sieve was the 4.75 mm (No. 4) for the 12.5 and 19.0 mm (1/2 and 3/4 inch) NMAS and the 2.36 mm (No. 8) for the 9.5 mm (3/8 inch) NMAS. Both L.A. Abrasion loss and the percentage of flat and elongated particles were correlated to the breakdown during laboratory compaction for the 12.5 and 19.0 mm (1/2 and ¾ inch) NMAS mixes. Only L.A. Abrasion loss was correlated to the breakdown of the 9.5 mm (3/8 inch) NMAS mixes. The degree of breakdown was similar in trend to the research conducted during NCHRP 9-8. More breakdown was noted for the samples compacted with the Marshall hammer, even with very low L. A. Abrasion losses. Measured VMA was found to decrease with increasing L. A. Abrasion loss based on samples prepared using the same gradation. Previously, Collins et al. (*28*) recommended adjusting the gradation for expected breakdown during the mix design process.

The International Center for Aggregate Research (ICAR) evaluated new techniques to assess aggregate resistance to degradation in SMA (*29*). The study evaluated the Micro-Deval Abrasion Test, Aggregate Imaging System (AIMS), and X-Ray Tomography to characterize aggregate breakdown during compaction and under loading. Mix designs were produced using six aggregate sources with a range of abrasion, angularity, shape, and texture. Four of the six mixtures failed the minimum VMA requirements for SMA. Unconfined repeated load permanent deformation tests were conducted on the compacted mixtures at a temperature of 37.8 °C (100 °F) and a vertical stress of 310 kPa (45 psi) [although the authors state that this is a "higher" stress to evaluate degradation; this is 380 kPa (55 psi) lower than what NCAT has used in previous SMA testing. However, the samples were apparently run unconfined]. Aggregate degradation was evaluated before and after the flow number testing. No significant degradation resulted from the dynamic loading. The applied stress may have been too low to cause aggregate breakdown. The expected stress from a larger aircraft would be much higher. A method was also proposed to evaluate aggregate breakdown based on a combination of breakdown under laboratory compaction and Micro-Deval Test results.

2.1.5.2 Research Related to Mineral Filler

The Queensland (Australia) Department of Main Roads conducted research to evaluate the affect of various fillers on the workability of SMA mixtures (*30*). A variety of commonly used fillers from Australia and the United States (primarily Maryland and Virginia) were studied. Experience indicated that in-place density could be more difficult to achieve with certain fillers, leading to pavement permeability problems. Scanning electron microscope images indicated that fly ash was more single-sized, rounded, and porous than other fillers.

Testing showed that filler to binder ratio, by mass, had a poor correlation ($R^2 = 0.3$) with the resulting mastic viscosity. This parameter is commonly specified for dense-graded mixes (Superpave specifications recommend a dust to effective binder ratio of 0.6 to 1.2). The ratio of filler to binder by volume had a slightly better correlation ($R^2 = 0.5$). The filler fixing factor (FFF) or percent of the binder absorbed (fixed) by the filler, calculated as shown in Equation 3, produced a strong correlation with the measured mastic viscosities ($R^2 = 0.9$).

$$FFF = \frac{1}{G_{se}} \frac{V}{(1 - V)} \tag{3}$$

where,
G_{se} = filler effective gravity, and
V = Rigden Voids.

A minimum free binder volume of 8 to 11 percent is specified (7.5 to 11 percent in production). The free binder volume is calculated as the total binder volume minus the percent absorbed by the aggregates minus the theoretical percent fixed by the filler.

2.1.5.3 Research Related to Laboratory Compaction Effort

The 50-blow Marshall compaction effort has been the standard for the design of SMA in Europe and early U.S. projects. Airfield pavements are still primarily constructed with mixes designed using the Marshall method. However, many contractors are losing their experience base with the Marshall method. Research is being conducted to adapt the Superpave mix design system, including the gyratory compactor, for the design of airfield pavements. Therefore, when developing specifications for SMA for airfields, SGC laboratory compactive efforts should be considered as well as the Marshall method.

Prowell et al. (*31*) evaluated field mix samples of a 9.5 mm (3/8 inch) NMAS SMA that were compacted to both 75 and 100 gyrations. The VCA_{Mix} was less than the VCA_{DRC} for both laboratory compaction efforts. It was concluded that since stone-on-stone contact was achieved at 75 gyrations, additional compaction was the result of aggregate breakdown. The reduction in gyrations increased air voids by approximately 0.4 percent. Based on the project data, it was estimated that the optimum asphalt content could be increased by 0.2 percent to maintain the same air void content. Testing with the Asphalt Pavement Analyzer (APA) indicated that field samples of the mixture, produced with a PG 76-22 binder, were rut resistant and insensitive to binder content (from a rutting standpoint) over a range of binder contents from approximately 7 to 8 percent.

James (*32*) evaluated gyratory compaction levels for SMA for Alabama DOT. In the laboratory stage, three aggregate sources: granite, sandstone, and limestone, were used to design both 9.5 and 19.0 mm (3/8 and 3/4 inch) NMAS SMA mixtures. The L.A. Abrasion loss ranged from 25.8 for the sandstone to 36.1 for the granite. Optimum asphalt contents were determined with 50, 75, and 100 gyrations using the SGC and with a 50-blow Marshall compaction effort. Gradations were adjusted to produce Marshall designs with passing volumetric properties (VMA > 17 percent). VCA_{Mix} was less than VCA_{DRC} for all of the compaction efforts studied. The ratio of the VMA at various SGC compaction level (number of gyrations) and the VMA determined from the 50-blow Marshall were used to estimate a design number of gyrations. The best fit line of the data produced a VMA ratio of 1.0 at 70 gyrations.

Additional testing was conducted on samples from four field projects. Each project was sampled four times. The same laboratory compactive efforts were applied as used in the laboratory study. Comparisons were made between the G_{mb} ratio obtained from the SGC and 50-blow Marshall compaction efforts. The best fit line of the data indicated that 63 gyrations with the SGC provided the same G_{mb} as a 50-blow Marshall compactive effort.

Aggregate breakdown on the breakpoint sieve (4.75 mm (No. 4) for 12.5 and 19.0 mm (1/2 and 3/4 inch) NMAS and 2.36 mm (No. 8) for 9.5 mm (3/8 inch) NMAS) was compared between field cores and the four laboratory compaction efforts. Projects 1 and 2 had similar breakdown for all compaction efforts (lab and field). For Project 3, the Marshall hammer produced the greatest breakdown (6 percent) and the 50 gyration compaction effort best matched the field cores. The aggregate breakdown based on field cores for Project 4 was between that which occurred for 50 and 75 gyrations. The Marshall Hammer produced approximately 1 percent higher breakdown than the field cores did.

Rut testing using the APA during the laboratory testing portion of the study indicated that rut resistant SMA mixes could be designed using 75 gyrations. Based on the data collected during the project, 70 gyrations were recommended for the design of SMA in Alabama with the caveat that this number may need to be adjusted based on changes in the internal angle of gyration for the SGC (*32*).

Xie (*33*) conducted a study funded by FHWA to determine the optimum laboratory compaction effort for SMA. Five aggregates with a range of L.A. Abrasion loss values were selected for the study including crushed gravel, two granite sources, a limestone and a traprock source. The L.A. Abrasion loss ranged from 16.6 to 36.4. A marble dust mineral filler, cellulose fibers, and PG 76-22 were used for all of the mixes. Mix designs were conducted using a 50-blow Marshall compaction effort for three NMAS for each aggregate source. The aggregates with higher L.A. Abrasion loss values were designed with gradations near the middle to coarse side of the design range. Finer gradations were used for the two aggregate sources with lower L.A. Abrasion loss values. The optimum asphalt contents for the 50-blow Marshall designs ranged from 5.8 to 6.8 percent by total weight of mix. The two designs below 6.0 percent were both for the limestone aggregates with an L.A. Abrasion loss of 26.5 percent.

Two SGC compaction levels were used for comparison to the 50-blow Marshall effort, 65 and 100 gyrations. On average, the optimum asphalt content increased 0.7 percent when the design gyrations were reduced from 100 to 65. All of the Marshall and 65 gyration SGC designs met the minimum VMA requirement (17). The VCA ratios for the Marshall and 65 gyration mixes were similar.

Performance testing was conducted to evaluate the rutting potential of the mixes designed with the SGC. Testing was conducted with both the APA and simple performance tests (SPT). The APA indicated that rutting potential increased with decreasing N_{design} gyrations. Eighty-seven percent of the mixtures designed with 65 gyrations met Georgia DOT's criteria for a maximum APA rut depth of 5.0 mm after 8,000 cycles. The 12.5 mm (1/2 inch) NMAS crushed gravel and limestone mixes exceeded the maximum rutting criteria. The repeated load permanent deformation test was conducted at a temperature of 60 °C, with a vertical stress of 827 kPa (120 psi) and a 138 kPa (20 psi) confinement pressure. The load was applied with a 0.1 second haversine pulse followed by a 0.9 second rest period for a total of 10,000 load cycles [dynamic modulus and static creep tests were also conducted, but are not discussed herein]. Based on testing conducted in this study and a literature review of other studies, a cumulative strain criterion of 5 percent after 10,000 cycles was recommended. The accumulated strain after 10,000 cycles averaged 2.2 and 3.0 percent respectively for the mixes designed at 100 and 65 gyrations. Statistically, the results were significantly different. Only one of fifteen mixes designed at 65 gyrations failed the 5 percent permanent strain criterion. A good relationship was found between the uncompacted voids in coarse aggregate and the secondary creep slope from the repeated load permanent deformation test. The uncompacted voids in coarse aggregate test was originally developed by Ahlrich (*34*) for the design of HMA for heavy-duty airfield pavements. Based on the testing completed, Xie (*33*) recommended a 65 gyration design compaction effort using the SGC to maximize durability and rutting resistance.

West et. al. (*35*) conducted a study to evaluate Georgia DOT's design compaction requirements for SMA. Five aggregate sources were selected for the study with a range of L.A. Abrasion loss values from 16 to 44 percent. Four design compaction efforts were used in the study: 50-Blow Marshall and 50, 75, and 100 gyrations with the SGC. Type C Fly Ash, 0.3 percent cellulose fibers and PG 76-22 binder were used to prepare all of the mixes. Design gradation varied between the aggregate sources. The design gradations mimicked existing mix designs. The percent passing the 4.75 mm (No. 4) sieve varied from 23 to 25 percent and the percent passing the 0.075 mm sieve varied from 8.4 to 10.3 percent. This study found that 35 gyrations with the SGC produced the same compacted sample density as a 50-blow Marshall. (Figure 2.6). A good correlation was found between the equivalent gyrations to match the 50-blow Marshall compaction effort and L.A. Abrasion ($R^2 = 0.98$). Georgia DOT specifies a design binder content of 5.8 to 7.5 percent for 12.5 mm (1/2 inch) NMAS SMA mixtures with a voids filled with asphalt (VFA) range of 70 to 90 percent. Three of the five mixes designed at 75 gyrations failed the minimum asphalt content [the gradation was not altered from the Marshall design]. All of the mixes designed with the SGC met Georgia DOT's APA rut depth criterion (max. 5 mm (0.2 inch)).

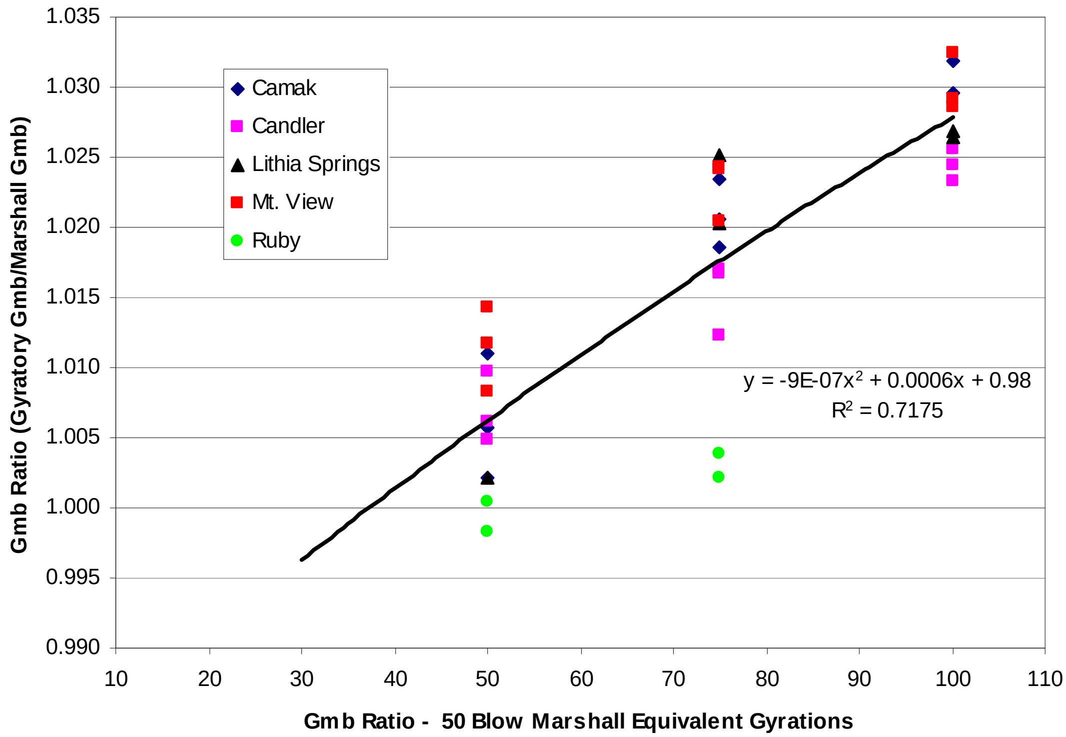

FIGURE 2.6 G_{mb} Ratio versus Gyration for Georgia SMA Study (*35*).

Three additional projects were used for field verification. Samples were taken from four consecutive lots from each project. Samples were compacted using the same four compaction levels (Marshall and SGC) described previously. During the field verification an average of 34 gyrations was predicted to match the 50-blow Marshall compaction effort. Based on Georgia's successful use of aggregates with relatively high L.A. Abrasion loss values (45 percent max.) to produce SMA, a 50 gyration N_{design} value is recommended for designing SMA with the SGC (*35*).

2.2 CONSTRUCTION OF SMA

There is considerably less information on the construction of SMA in the literature as compared to the design of SMA. Some of the early information from Europe has been discussed previously, but will be briefly repeated here. Fractionated aggregate is generally used to produce SMA in Europe. In Germany, aggregates are fractionated into 8-11, 5-8, 2-5, and 0-2 (sand) mm size fractions. This allows precise control of the SMA gradation, particularly on the critical or breakpoint sieve. When using blended sizes common in the U.S., if a high proportion of a single stockpile is used, it should be split into two cold feed bins. If the gradation of that particular stockpile being supplied by the aggregate producer varies during production, it may be difficult to adjust the gradation of the mix to maintain volumetric properties. Portable screening equipment has been developed to allow the contractor to fractionate aggregate on site, thereby improving control.

Scherocman (*3*) recommends that mineral filler be added into the mixing chamber on a drum plant so that it does not get caught in the air stream and be sucked into the baghouse. Similarly, it should be treated as a fifth hot bin for a batch plant and added directly into the pugmill. However, other states have followed different practices. In Maryland (personal communication with Larry Michael) and Virginia (personal communication with Richard Schreck), mineral filler is commonly added through the cold-feed bins. It is important for the filler to be kept dry in order for it to flow. Teflon liners or vibrators can help prevent the mineral filler from bridging. Care must be taken that the cold-feed bins are in good condition or the mineral filler may flow out of any hole in the bin. This same caution applies to the mixing chamber or pugmill of a batch type plant. Any wear on the liners of the pugmill gate will allow mineral filler to flow through without being coated.

The European Asphalt Study Tour (*6*) noted that cellulose fibers were added directly into the pugmill by hand. The plastic bag that the fibers were contained in readily melted. The aggregate and fibers were dry mixed for a period of six to ten seconds prior to adding the binder. Today, fibers are typically added through a weight reduction feeder system tied to the plant controls. This allows the fiber feed rate to vary with the plant production rate.

Brown and Greene (*36*) noted the increasing use of materials transfer vehicles (MTVs) to increase smoothness and decrease segregation problems. Not only will an MTV with remixing capabilities decrease segregation of the mixture components (e.g. draindown), but it will also reduce thermal segregation or crusting of the mix during haul. Reduction in thermal segregation helps to improve the uniformity of in-place density. Many U.S. states require the use of MTVs when placing SMA.

In Europe, SMA is generally placed by heavy "tamping bar" screed pavers. A picture of the tamping bars is shown in Figure 2.7. Tamping bar screed pavers provide a higher degree of initial compaction of the SMA, immediately behind the paver. They have been used to place SMA in Virginia and Indiana, among other places in the U.S. Tamping bar screed pavers are also readily adaptable to pave wide widths [6 m (20 feet) has been routinely used on commercial projects in the author's experience]. This provides an advantage when paving airfields by reducing the number of longitudinal joints. The use of a tamping bar screed paver can also improve smoothness, particularly if the thickness varies. The higher degree of compaction minimizes differential rolldown as thickness varies. Rolldown is the degree of thickness change between the depth immediately behind the paver and the depth after compaction. With a conventional paver, roll down is typically estimated at 6 mm per 25 mm (0.25 inches per inch) of compacted thickness (*37*). With a tamping bar screed paver, roll down is typically reduced to 3 mm per 25 mm (0.125 inches per inch) of compacted thickness.

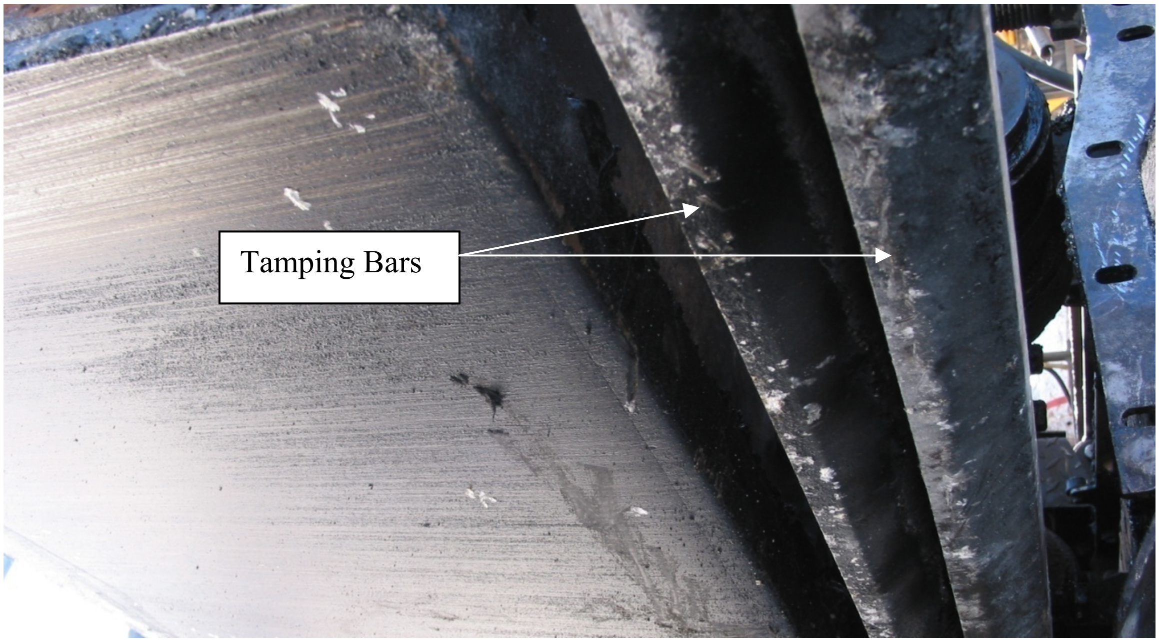

FIGURE 2.7 Tamping Bars on Tamping Bar Screed Paver.

The European Asphalt Study Tour (*6*) noted that vibratory compaction was not recommended when compacting SMA, especially for the first pass of the roller. It was felt that vibratory compaction on the first pass could bring excess binder to the surface. Scherocman (*3*) recommends using vibratory rollers set at low amplitude and high frequency for breakdown rolling of SMA mixtures. The use of vibratory rollers once a stone skeleton has formed seems more likely to fracture aggregate. In a 2003 study tour of SMA sponsored by the Virginia Asphalt Association (VAA), Prowell observed vibratory rollers being used on two projects. The breakdown roller would complete the first pass in static mode and then "vibe out" using low amplitude and high frequency.

Wilson (*38*) discussed the use of static rollers on SMA. Rollers with wide (2.1 m [84 inch]) drums are often ballasted to approximately 12,247 kg (27,000 lbs). The use of these wide drum rollers has increased since they can cover a 3.7 m (12-foot) wide mat in two passes. The compactive effort of a static roller can be measured by pounds-per-linear-inch (PLI). The PLI can be determined by dividing the effective operating weight by 2 (for two drums) and then by the width of the drum in inches. Thus a 2.1 m (84 inch) wide double drum vibratory roller ballasted to 12,247 kg (27,000 lbs) operated in static mode would produce a PLI of 161. Some states require static rollers with PLI in excess of 300 for compacting SMA. Higher PLI is more readily achieved on rollers with narrower drum widths (which are more typical in Europe). For instance a large steel wheel static drum roller with a 1.4 m (54 inch) drum width ballasted to 8,687 kg (28,500 lbs) produces 264 PLI. However, typically the drive wheel of such rollers is heavier, often 60 percent of the total roller weight. Thus the drive wheel may produce 317 PLI. Although these narrower drums will require an additional pass to cover the width of the mat, they produce a greater compactive effort per pass, while the mix is hot. Rubber tire rollers are generally not recommended for SMA due to concerns about the potential for pickup.

In-place density is one of the most important factors in the construction of SMA pavements. The FHWA SMA Technical Working Group Guide specifications specified an in-place density of greater than 94 percent of theoretical maximum density (G_{mm}) (*7*). NCHRP 9-8 Phase I and II presented data that indicated SMA pavements required higher in-place densities over dense-graded mixes. Hence, it was recommended that the in-place air void content of SMA be less than six percent (*20*). This was to ensure an impermeable pavement, thus a longer life over conventional HMA pavements. Prowell et al. (*31*) confirmed this maximum recommended air void content for SMA pavements.

The European Asphalt Study Tour (*6*) reported lift thickness varied with NMAS in Germany: 25 to 50 mm (1 to 2 inches) for 11 mm (7/16 inch) NMAS and 15 to 30 mm (0.6 to 1.2 inches) for 5 mm (approximately No. 4) NMAS. The standard thickness for Autobahn paving appeared to be 40 mm (1.5 inches) for 11 mm (7/16 inch) NMAS. Swedish lift thicknesses were observed to be 38 mm (1.5 inches) for 16 mm (5/8 inch) NMAS SMA. NCHRP 9-27 conducted intensive research into determining a minimum lift thickness to NMAS (t/NMAS) ratio that would result in an optimum performing pavement; one that had high in-place density and was impermeable. Results from this study recommended a t/NMAS of 4:1 for most SMA pavements (*39*).

As noted previously, early skid resistance can be of some concern with SMA pavements due to the high film thickness of binder on the coarse aggregate. The European Asphalt Study Tour noted that sand is sometimes added to the surface of SMA in Germany and rolled in while it is hot. This was observed on every project visited by the 2003 VAA SMA Study Tour. Schreck (*40*) states 1-3 mm grit is applied at a rate of 1.5 to 3 lbs per square yard on 0/8 mm and smaller NMAS SMA and 2-5 mm grit is applied at a rate of 3-5 lbs per square yard on 0/11 mm and larger NMAS SMA. The grit is often precoated with 0.8 percent asphalt binder to control dust. This is not enough binder to cause the particles to stick together and it can be stockpiled. Grit is applied to the mat surface while it is still hot, typically in the range of 65 to 93 °C (150 to 200 °F) and then rolled in. If the mat is too cold, the grit will not stick. Figure 2.8 shows the grit being applied, and Figure 2.9 shows the difference in surface appearance before and after gritting. The grit acts to absorb excess binder on the surface of the SMA, improving early skid resistance. It is also believed to reduce permeability. Two concerns with the use of grit on airfield pavements would be the reduction in macrotexture which may necessitate grooving, and the potential for FOD. The grit can also be used as a release agent in truck beds.

FIGURE 2.8 Application of Grit to SMA on Autobahn 3, near Passau, Germany.

FIGURE 2.9 Gritted (Foreground) and Non-Gritted SMA Surface.

2.3 PERFORMANCE OF SMA

SMA has been produced in the United States since 1991. Originally from Europe, SMA was introduced into the United States due to its rut resistant characteristics. Other benefits from the use of SMA have since been discovered, ranging from crack resistance, greater durability, improved friction, reduced noise generation, and improved ride quality. It is due to these performance benefits over conventional hot mix asphalt that more than 28 states have placed SMA in high-traffic applications. The following discusses the performance of SMA since its inception in the United States.

After the European Asphalt Study Tour in 1990, the Georgia DOT (*41*) produced two SMA research projects to evaluate the performance of SMA in the state. Since then, additional research has been conducted to better understand the performance of SMA. From this research, SMA has proven to be 30-40 percent more rut resistant than standard dense-graded mixtures. Fatigue life, based on laboratory studies, was reported to be three to five times that of conventional mixes. Friction values obtained from field test sections also indicated that SMA pavements provide good performance, once the thicker asphalt film wears off. The performance benefits can be summarized by Georgia DOT's life-cycle cost analysis of SMA pavements versus conventional HMA. This research indicated the SMA pavements will have a lower annualized cost of $50,095 over $79,532 for conventional HMA. The analysis was based on a four-lane roadway over a 30 year period with overlay intervals of 10 years for the SMA as compared to 7.5 years for conventional mixes.

Brown et al. (*42*) published the results from a national study to evaluate the performance of SMA pavements that were constructed from 1991 to 1996, during the early stages of SMA implementation. A total of over 100 different SMA pavements located in 19 states were evaluated based on several factors, including rutting, cracking, raveling, and fat spots. Conclusions from this study indicated that 90 percent of the projects evaluated had less than 4 mm of rutting, including 25 percent that had no measurable rutting. Cracking (both thermal and reflective) were determined to be of no concern, as the relatively high asphalt content in an SMA mixture produces a more crack resistant pavement. The authors did state that the only area of concern with an SMA mixture was fat spots, possibly due to segregation, draindown, high asphalt content, excessive production temperatures, and improper type or amount of stabilizer.

Watson (*43*) performed a follow-up to the study that Brown et al. (*42*) conducted in 1995 so that the long-term performance of these SMA mixtures could be better evaluated. Thirteen SMA projects in 5 states were revisited, and their performance characteristics were recorded. Watson concluded that due to the rut-resistant benefits of SMA, several state DOTs have made the construction of SMA pavements a standard practice. Cracking observed in some of the SMA pavements were attributed to mix design or material property errors. Watson also stated that SMA pavements seemed to reduce the propagation rate of reflective cracking, leading to a longer expected life span compared to Superpave mixtures.

Campbell (*44*) published results from several SMA trials that were conducted on two airfields in Australia. Also contained within this report, Campbell discussed the performance of SMA on general roadways throughout the world. To date, fourteen countries in Europe, the United States, Canada, South Africa, China, New Zealand, and Australia have used SMA on roadways in some capacity. All European countries reported very positive experience in using SMA, most noticeably the surface characteristics, durability, and riding comfort.

Schmiedlin and Bischoff (*26*) published a report on the performance of six SMA pavements in Wisconsin after five years of trafficking and compared the results to a control dense-graded pavement. Among the performance measures were amount of cracking, friction characteristics, overall pavement distress, amount of rutting, noise impact, and ride quality. Results indicated that, after the end of the five-year evaluation period, the SMA pavements were performing better than the conventional asphalt pavements in the majority of the performance measures. Specifically, SMA produced 19 percent less reflective cracking than typical HMA pavements. The sections constructed with a high percentage of elastomeric polymer performed marginally better than the other sections. The larger (16 mm [5/8 inch]) NMAS mixes also performed better than the smaller (9.5 mm [3/8 inch]) NMAS mixtures. Rutting values for both mix types were inconclusive due to the uniformly low values for all pavements. Regarding overall pavement distress (PDI), a unitless numerical value between 1 and 100 is used. The lower the number, the lower the presence of pavement distress. Table 2.7 presents PDI data for the different pavements evaluated, along with the control pavement. The sections in Table 2.7 refer to different types and levels of stabilizers or modifiers. From the data, it was determined that the SMA pavements are performing 38 percent better than the control pavements, in terms of overall pavement distress.

TABLE 2.7 Pavement Distress Index Analysis at Five Years

	Region 1		Region 2		Region 3		
Aggregate	3/8" (9.5 mm)	5/8" (16 mm)	3/8" (9.5 mm)	5/8" (16 mm)	3/8" (9.5 mm)	5/8" (16 mm)	
Section	STH 63	USH 45	STH 21	USH 151	I-43 Wauk	I-43 Walw	Mean
F1	32	7	27	36	32	10	24
F2	47	32	27	19	33	18	29
E1	13	34	6	19	14	27	19
E2	13	13	27	20	6	15	16
P1	13	13	27	26	14	6	17
P2	23	17	6	32	27	31	23
Mean	24	19	20	25	25	18	21
Control	48	13	27	30	30	47	34

F1 = Cellulose fiber, F2 = Mineral fiber, P1 = Low % Thermoplastic polymer, P2 = High % Thermoplastic polymer, E1 = Low % Elastomeric polymer, and E2 = High% Elastomeric polymer.

Friction tests were conducted with a locked wheel skid trailer using a ribbed tire [presumably according to ASTM E274]. Tests were conducted at both 64 and 80 km/hr (40 and 50 mph). The SMA mixes had a slightly lower average friction number after 5-years of traffic as compared to the dense grade control mixes (45 versus 48). However, the average speed gradient or reduction in friction with increased speeds is smaller for the SMA mixes than for the dense graded mixes (0.22 versus 0.29) (*26*).

Michael et al. (*45*) published a report that documented the performance of SMA in Maryland. Using over 1000 sets of construction quality control test data and nearly 300 sets of pavement performance measurements, a performance analysis was performed on SMA pavements up to 10 years in age. The performance analysis included rut depth, roughness using the International Roughness Index (IRI), and skid resistance using the Friction Number (FN). Table 2.8 presents the summary statistics for the annual changes in performance for the three factors. From the data, in practical terms, the annual changes are so small that they can be taken as zero. Michael et al. concluded that the SMA pavements that have been constructed over the past 10 years have performed very well. Other noticeable benefits of SMA were also observed, these being reduced tire splash and reduced tire noise.

TABLE 2.8 Summary Statistics for Annual Changes in Performance (*45*)

ΔRutting/Year (inches/year)						
Mix Size	n	Min	Max	Mean	Std Dev	COV
9.5 mm (3/8 inch)	1			0.025		
12.5 mm (1/2 inch)	12	0.007	0.077	0.034	0.021	62%
19 mm (3/4 inch)	56	0.087[1]	0.069[1]	0.008[1]	0.02	250%
ΔIRI/Year (in/mile-year)						
Mix Size	n	Min	Max	Mean	Std Dev	COV
9.5 mm (3/8 inch)	1			3.2		
12.5 mm (1/2 inch)	12	-3.0	9.2	1.3	3.4	262%
19 mm (3/4 inch)	50	-4.4	13.6	1.8	3.6	200%
ΔFriction/Year (FN/year)						
Mix Size	n	Min	Max	Mean	Std Dev	COV
9.5 mm (3/8 inch)	1			0.4		
12.5 mm (1/2 inch)	12	-0.3	5.3	1.3	1.5	114%
19 mm (3/4 inch)	576	-2.7	3.4	0.3	1.1	335%

[1]One or more of these values appear to be erroneous. They are reproduced from the original text.

Note: 1 inch = 25.4 mm; 1 mile = 1.6 km

The NCAT Pavement Test Track was initially constructed in 2000 (Phase I) and then after the application of 10 million ESALs portions of the track were reconstructed in 2003 (Phase II). In both Phase I and Phase II, SMA test sections were constructed and evaluated on the NCAT Test Track. Timm et al. (*46*) published overall findings from Phase II of the test track. Also included in this report were findings from the evaluation of five SMA test sections from Phase I, and these findings showed that the SMA sections had excellent performance. Table 2.9 presents rutting data for the SMA sections after 10 million ESALs were applied. It was also noticed that no cracking appeared for any of the SMA sections. Only minor raveling of the coarse aggregate for one section was noticed at the end of the first cycle.

TABLE 2.9 Rut Data for SMA Test Sections, NCAT Test Track Phase I

Section	Description	Rut Depth, mm
N12	12.5 mm NMAS, Granite, SBS	2.7
N13	12.5 mm NMAS, Gravel, SBS	4.2
W1	12.5 mm NMAS, Granite, SBR	3.2
W2	12.5 mm NMAS, Limestone & Slag, SBR	4.3
W8	12.5 mm NMAS, Sandstone, Limestone & Slag, SBR	4.8

For Phase II of the NCAT Test Track, SMA test sections N12 and W1 remained in place to receive the next cycle of testing. Timm et al. reported that these two sections continued to perform very well with only minimal additional rutting and no signs of cracking after nearly 19 million total ESALs. Seven new SMA test sections were constructed for Phase II, and like the ones from the first cycle, performed extremely well. No signs of cracking were observed and minimal rutting was determined after 9 million ESALs, as shown in Table 2.10.

TABLE 2.10 Rut Data for SMA Test Sections, NCAT Test Track Phase II

Section	Description	Rut Depth, mm
N7	9.5 mm NMAS, Granite	4.7
N9	9.5 mm NMAS, Limestone	5.1
N10	9.5 mm NMAS, Limestone & Chert	6.6
N13	12.5 mm NMAS, Granite	3.0
S1	12.5 mm NMAS, Granite	5.6
E1	12.5 mm NMAS, Limestone	6.3
W2	12.5 mm NMAS, Porphry & Limestone	6.6

Note: 1 inch = 25.4 mm

Clark et al. (*47*) documented initial performance characteristics of SMA pavements that were constructed in Virginia in 2003. Among the performance characteristics documented were roughness, friction, and texture. Over 25,000 tons of SMA were placed and evaluated from late summer 2002 through the following summer. In addition, friction and roughness data was collected from older SMA pavements to support the expected long-term performance of SMA. Results from this research indicated that the ride quality of SMA varied from project to project, possibly due to the different SMA types. Overall the ride quality was generally good and was predicted to improve as construction experience grew. Friction numbers were reported to be good and tended to increase with time. A slight decrease in texture was determined, but overall texture characteristics are predicted to provide low noise SMA pavements.

In a follow-up study, McGhee and Clark (*48*) evaluated the predicted service lives and estimated the life-cycle costs of the asphalt mixtures most commonly used in Virginia, including SMA. Service life estimates were developed from a database of critical condition index values. The critical condition index is determined from windshield surveys by a panel of raters. A windshield survey is a visual assessment of conditions based on a set of predetermined criteria. The predicted service lives are shown in Table 2.11 as a function of underlying structure. In the mix designations, SM refers to Superpave surface mixes. The number in the designations is the NMAS in mm. The letter at the end of the Superpave designations refers to the binder grade with "A" being PG 64-22 and "D" being PG 70-22. The overall weighted average of the service lives was 8.5 years for the Superpave mixes and 17.3 years for the SMA mixes. Based on equivalent uniform annual cost analyses, SMA mixes could cost 82 to 94 percent more than comparable Superpave mixes when used in bituminous pavement structures and still be more cost effective over the pavements life-cycle.

TABLE 2.11 Predicted Service Life (Years) Based on Highway System Data through 2006 (*48*)

Mix	Service Life, years			No. of Sections Evaluated		
	Underlying Structure					
	BIT	BOJ	BOC	BIT	BOJ	BOC
SM 9.5A	10.5			46		
SM 9.5D	8.2	7.6	12.1[a]	644	55	3
SM 12.5A	11.3			165		
SM 12.5D	7.6	7.6	18.2[a]	346	33	2
SMA 9.5	22.2			14		
SMA 12.5	17.7	9.4	23.1	66	31	27

BIT = bituminous; BOJ = bituminous over jointed; BOC = bituminous over continuously reinforced concrete.
[a]Only one representative section per year in database.

2.3.1 Friction

FAA requires that HMA runways and certain high-speed taxiways be grooved to reduce the potential for hydroplaning. The groove dimensions are specified as 6 mm (0.25 inch) wide by 6 mm (0.25 inch) deep with a spacing of 38 mm (1.5 inches) center to center (*49*). The International Civil Aviation Organization requires pavements to have a minimum macrotexture of 1.0 mm (0.04 inch) to reduce the hydroplaning hazard.

The friction of a pavement surface is a function of the surface textures that include the wavelength ranges described by microtexture, consisting of wavelengths of 1μm to 0.5 mm, and macrotexture, with wavelengths of 0.5 mm to 50 mm (*50*). Microtexture provides a gritty surface to penetrate thin water films and produce good frictional resistance between the tire and the pavement. Macrotexture provides drainage channels for water expulsion between the tire and the pavement thus allowing better tire contact with the pavement to improve frictional resistance and prevent hydroplaning. Macrotexture also affects the friction component of hysteresis, or deformation of the tire rubber by the pavement macrotexture. Hysteresis contributes to high speed sliding friction.

Texture measurements were made at the NCAT Test Track on five SMA sections at the conclusion of the Phase I trafficking. Macrotexture was measured with both the ASTM E965 sand patch test and ASTM E 2157 CT Meter (*51*). The two methods produced average macrotexture readings of 1.28 and 1.26 mm, respectively, based on tests conducted on the five sections. Eight SMA sections from Phase II were tested prior to trafficking with the CT Meter. The six 12.5 mm NMAS SMA sections produced an average surface texture of 1.02

mm with a range of 0.60 to 1.29 mm. Two 4.75 mm (No. 4) NMAS SMA sections produced an average macrotexture of 0.58 mm. Macrotexture would be expected to decrease with decreasing NMAS. SMA pavements have been successfully diamond ground to improve smoothness (Figure 2.10). This produces a tighter pattern of groves than what is specified for runways. Observations of grooves formed in SMA by diamond grinding at the NCAT Test Track indicate that such grooves are durable under heavy traffic.

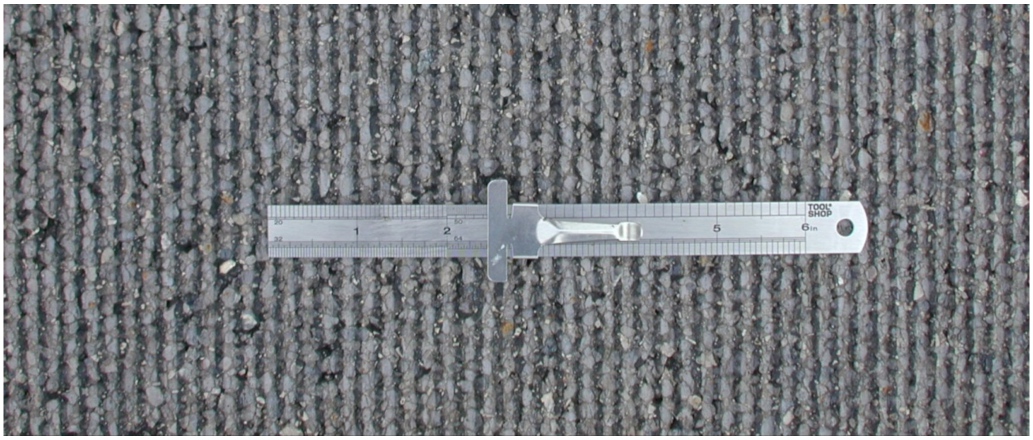

FIGURE 2.10 Diamond Ground 4.75 mm (No. 4) NMAS SMA at Indianapolis Motor Speedway.

After Phase I of the NCAT Test Track, data was recorded with regards to friction and surface texture (*52)*. Five different gradation types were evaluated: OGFC, SMA, and three Superpave gradations (below, through, and above the restricted zone). Basically, the Superpave gradations represent a coarse gradation, a fine gradation, and a gradation similar to those prior to Superpave. In terms of smoothness, it was determined that a smooth pavement can be achieved, regardless of mix type, based on average IRI values. For friction, it was determined that SMA does exhibit lower friction values particularly immediately after construction; however, these values are still more than adequate. Once the film thickness decreases, SMA maintained a higher skid number with traffic than other mix types. Georgia DOT (*41)* reported a slight increase in friction numbers over five years of traffic, confirming that SMA's will indeed provide good friction.

There has been some concern over the early friction SMA surfaces in Ireland and the United Kingdom (*53,54*). The concerns arise from the high film thickness of binder on the aggregates and the use of heavily modified asphalts. Under this combination, the microtexture of the coarse aggregate is not exposed until the binder film has been worn off by traffic. However, it was noted that although friction values of new SMA pavements may be lower than might be expected for a new pavement, they are typically above minimum friction "threshold" values (*54*). As noted previously, the Germans address this concern by gritting the pavement.

Overall, it seems that the performance of Stone Matrix Asphalt has been extremely positive. They have proven to be rut-resistant, provide adequate friction and texture characteristics; all while producing less noise than the conventional hot mix asphalt pavements typically produced in the United States. Resistance to cracking has also improved through the use of

SMA as well. Even though the typical SMA cost approximately 15-20 percent more than conventional asphalt pavements, SMA pavements have been shown to last longer, making them more cost effective than conventional pavements.

CHAPTER 3

USE OF SMA ON AIRFIELDS

Campbell (*44*) reported that while SMA had been used on roadways in 25 countries around the world, its usage on airfields was limited to 15 countries. Efforts have been made to update the work done by Campbell in order to document the usage on airfields in the intervening years.

3.1 AUSTRALIA

3.1.2 Cairns International Airport

In 1998, 1600 m^2 representing half of Bay 19 of the Domestic Apron was paved with SMA. Approximately 200 tons of SMA were placed. The performance of this section is reported to be very good and better than conventional HMA used on other portions of the airfield. It has required little to no maintenance up through 2007. However, in the first four to six weeks after construction the apron needed to be swept frequently to remove loose stones, often after every movement. Stones were apparently plucked out of the surface by hot airplane tires. Watering the surface reduced this problem. This problem may have been avoided by using a stiffer binder.

In 2005, the entire International Apron (approximately 32,000 m^2) was resurfaced using a 50 mm lift (approximately 4,000 tons) of 12 mm maximum aggregate size SMA. The gradation specifications are shown in Table 3.1. A 320/1000 Multigrade binder was specified for the project. The mixture included 0.3 to 0.4 percent fibers.

TABLE 3.1 Specification for Cairns International Taxiway

Sieve Size, mm (in)	Percent Passing
19.0 (3/4)	100
13.2 (0.525)	100
9.50 (3/8)	47-59
6.70 (No. 3)	32-42
4.75 (No. 4)	26-34
2.36 (No. 8)	19-25
1.18 (No. 16)	14-20
0.600 (No. 30)	12-18
0.300 (No. 50)	10-14
0.150 (No. 100)	8.5-11.5
0.075 (No. 200)	7.5-9.5
Binder Content	6.0 – 6.4

3.1.2 Sydney International Airport

A production trial of SMA was constructed at Sydney International Airport in 1999. There was a concern that the introduction of fibers into the SMA would require additional mixing time and a corresponding impact on productivity. The gradation specification for the production trial is shown in Table 3.2 (*44*). An open surface texture was produced on a portion of the trial (Figure 3.1). The open texture is believed to have been caused by low mix temperatures due to operational delays in shipping the SMA to the site (personal communication with Garry Wickham). In 2003, the surface exhibited rutting in a limited area and a small patch was placed. Due to surface raveling resulting in the production of FOD, a surface treatment was placed on the section in 2004 (personal communication with John Dardano). Sydney International Airport has no plans for additional SMA trials.

TABLE 3.2 Specifications for Sydney Airport Trial (*44*)

Sieve Size, mm (in)	Percent Passing
19.0 (3/4)	100
13.2 (0.525)	90-100
9.50 (3/8)	50-65
6.70 (No. 3)	30-44
4.75 (No. 4)	21-32
2.36 (No. 8)	16-26
1.18 (No. 16)	
0.600 (No. 30)	
0.300 (No. 50)	11-18
0.150 (No. 100)	
0.075 (No. 200)	8-12
Binder Content	5.7-6.7
Fiber Content	0.2-0.3
Minimum VMA	16

FIGURE 3.1 Typical Open Texture Area at Sydney International Airport.

3.2 CHINA

China is a leader in the use of SMA on airfields. As old pavements are overlaid, SMA is the mix that is typically used based on information from the China Airport Construction Group Corporation of the Civil Aviation Administration of China (CAAC). The Beijing Capital Airport was first paved with SMA ten years ago. Since that time, an additional ten airfield pavements, for a total of eleven out of twenty-two, have been constructed with SMA (personal communication with Mr. Su Xin). Some of the benefits Chinese Airport Construction Group attributes to SMA are: resistance to damage from oil and fuel spills, improved skid resistance, greater durability, low maintenance, reduced reflective cracking, and lower life-cycle cost.

The Chinese specifications for SMA are presented in two documents, "Specifications for Asphalt Concrete Pavement Construction of Civil Airports (*55*), and Specifications for Asphalt Concrete Pavement Design of Civil Airports (*56*). Some of the design parameters have been updated as described in (*57*). The design specifications are summarized below. The specifications appear to be a combination of method and performance based specifications.

3.2.1 Specifications

A maximum L.A. Abrasion loss of 30 percent is specified. Flat and elongated particles are limited to a maximum of 12 percent for surface layers and 15 percent for other layers using the 3:1 maximum to minimum dimension. When asked about the unusually high standard of quality and the difficulty quarry producers must have in meeting those requirements, it was stated that aggregate could be used that did not meet that level of quality, but it would require a letter of disposition by the Engineer explaining why the lower quality material was used. Figure 3.2 shows an example of SMA coarse aggregate used in China.

FIGURE 3.2 Chinese SMA Aggregate Sample.

There is no specification regarding the use of natural sand. However, the sand equivalent value must exceed 60 percent. The plasticity index for the fine aggregate/filler should be less than 4. The apparent specific gravity of the aggregates should be greater than 2.5. [Apparent specific gravity is always greater than bulk specific gravity (G_{sb})]. Baghouse dust is not allowed as mineral filler. (*55*)

China uses penetration grades for asphalt binders. The penetration grade depends on the climatic zone of the airport. Both low-density polyethylene (LDPE) and Styrene-Butadiene-Styrene are used as modifiers. Base asphalts for modification have a penetration grade of 100-120; although newer projects require the base asphalt to have a penetration grade of 80-100. LDPE is used in warmer areas of the country where low-temperature cracking is not a concern because it is more economical.

Gradation specifications are supplied for two NMAS, 13 and 16 mm (0.51 and .63 inch). The gradation ranges are shown in Table 3.3. These have been modified with experience on specific projects as described below.

TABLE 3.3 SMA Gradation Requirements for Airfields in China (*56*)

Sieve Size, mm (in)	Percent Passing	
	SMA-16	SMA-13
19 (3/4)	100	
16 (5/8)	90 – 100	100
13.2 (.525)	60 – 80	90 – 100
9.5 (3/8)	40 – 60	45 – 65
4.75 (No. 4)	20 – 32	22 – 34
2.36 (No. 8)	18 – 27	18 – 27
1.18 (No. 16)	14 – 22	14 – 22
0.6 (No. 30)	12 – 19	12 – 19
0.3 (No. 50)	10 – 16	10 – 16
0.15 (No. 100)	9 – 14	9 – 14
0.075 (No. 200)	8 - 12	8 – 12

The Chinese design specification for SMA for airfields is shown in Table 3.4. Laboratory mix designs are conducted using the Marshall method. The minimum asphalt content varies with aggregate specific gravity (Table 3.5) similar to AASHTO specifications. The specification includes two criteria related to permanent deformation: Marshall stability and flow, and a wheel tracking test. As noted previously, flow values can be high with SMA mixtures. The range in the Chinese specifications reflects this (20-50, 0.1 mm)

TABLE 3.4 SMA Mix Properties Requirements for Airfields in China (*57*)

Index	Criteria
Marshall Blows	50 on each face
Stability, Min N, (lb)	7,000 (1,574)
Flow, 0.1mm (0.01 in)	20-50 (8-20)
Design Air Voids, %	3-5
Minimum VMA, %	17
Minimum Dynamic Rutting Stability Index[1]	When using modified asphalt: 3000; When using unmodified asphalt: 1500
Retained Marshall Stability[2] after submerged in water, %, Min	80
Minimum TSR, %	75
Draindown (170 °C [338 °F], 1h), %, no greater than	0.15
Cantabro Abrasion Test [3] (-10 °C [14 °F]), %, not greater than	20

[1]Dynamic rutting stability is the results from a wheel track rutting test (Japanese rutting machine), that measures the rate of rutting and unit is cycle/mm of rutting.
[2]Retained Marshall Stability is usually run after the sample is submerged in 60 °C (140 °F) water for 48 hours. The test condition is not stated in the specification.
[3]Cantabro abrasion test is a test that uses an L.A abrasion machine to test the integrity of a compacted Marshall sample under the repeated impact without steel balls. The percent of mass loss due to abrasion after 300 cycles will be the test result. This test was originally developed in Spain.

TABLE 3.5 Minimum Asphalt Content (Percent) for SMA Based on G_{sb} (*Personal Communication with Xie)*

G_{sb}	Standard AC	Polymer-Modified AC
2.9	5.5	5.6
2.8	5.7	5.8
2.7	5.9	6.0
2.6	6.1	6.2

Steel drum rollers with a minimum weight of 8 tons are specified for compaction. Vibratory rollers are recommended for the first two passes. There is no joint density requirement, but a target density of 99 percent of the 50-blow Marshall lab density is used. The remainder of the mat must be compacted to at least 97 percent of the theoretical maximum density.

3.2.2 Beijing Capital International Airport

SMA was first used on the east runway (36R/18L) at the Beijing Capital International Airport. The runway was originally constructed with concrete pavement in 1954 and was 40 cm (15.75 in) thick on the ends and 35 cm thick (13.8 in) at the midpoint of the runway.

Problems with alkali-silica reactivity (ASR) required that it be repaired in 1996. A felt fabric 50 cm (20 inches) wide was applied over all the joints and cracks prior to overlaying with hot mix asphalt. The overlay consisted of 8 cm (3.15 in) of conventional hot mix asphalt (HMA) base layer with AC-25, 7 cm (2.75 in) of intermediate layer with AC-20 and 6 cm (2.4 in) of SMA-16 surface mix. Although the mix is 11 years old, it is still performing well. There are ruts only about 1 cm (0.4 in) in depth at the end of the runway where planes sit waiting to take off. The runway was sealed about five years ago with "SealMaster" seal coat to restore the dark color. A high pressure water spray (no chemicals) is used every two months to remove rubber buildup. Glycol is used during the winter for snow and ice removal.

In 1980, the west runway (18R/36L) was constructed of HMA. The structure was 21 cm (8.25 in) thick at the centerline of the runway, 18 cm (7.1 in) thick at 5 m (16.4 ft) from the center, and 13 cm (5.1 in) thick at the shoulders. It was overlaid with SMA in 2000. Trinidad Lake asphalt modified with styrene-butadiene-styrene (SBS) was used for 200 m (656 ft) at the ends of the runway. There were some problems with mixture quality control during construction that resulted in spots of high dust content from inconsistent feed of mineral filler and dirt from the natural sand that was used. Repairs have been made two to three times on the end of the runway, but the runway cannot be shut down long enough to make full-depth repairs. As a result, light mill and inlay applications are used on a periodic basis to maintain the surface as necessary. Figure 3.3 shows the surface texture of the east and west runways.

(A)

(B)

FIGURE 3.3 SMA Texture on East Runway (A) and West Runway (B).

The SMA design used for the overlay construction is as follows:

a. Binder: A base asphalt with a penetration value of 100-120 was modified with 3 percent low density polyethylene (LDPE) and 3 percent SBS to produce PG 70-22.
b. Aggregate: The coarse aggregate consisted of basalt and the fine aggregate was limestone and natural sand. Manufactured sand is normally used in SMA, but no manufactured sand was available, so a decision was made to allow 15 percent natural sand.

c. Mineral Filler: Limestone dust was used for the filler.
d. Stabilizing additive: Viatop 66 cellulose pellets were used to stabilize the thick asphalt film and prevent draindown. The pellets consisted of 66 percent cellulose fibers and 34 percent bitumen. The fiber was furnished in 4.5 kg (10 lb) plastic bags that were added directly into the weigh hopper of the batch plant during production of each batch. The dry aggregate mixing time was increased 5 to 15 seconds and the wet mix cycle was increased at least 5 seconds in order to ensure adequate blending.
e. Gradation:

TABLE 3.6 Beijing SMA Gradation Range

Sieve Size, mm (in)	% Passing
19 (3/4)	100
16 (5/8)	95-100
13.2 (.525)	72-92
9.5 (3/8)	54-72
4.75 (No. 4)	25-40
2.36 (No. 8)	17-31
1.18 (No. 16)	14-26
0.6 (No. 30)	10-22
0.3 (No. 50)	8-17
0.15 (No. 100)	7-15
0.075 (No. 200)	7-11

The mix was delivered to the construction site at a minimum temperature of 160°C (320°F). Placement temperatures of 170 to 180°C (338 to 355°F) were recommended. The mix was placed by four pavers working in echelon. Some of the mixture was screened and the resulting finer mixture was sprinkled along the longitudinal joint to prevent raveling.

3.2.3 Xiamen International Airport

The runway at Xiamen, 05R/23L, does not have an SMA surface, but was evaluated for a comparison of the performance of regular dense-graded HMA. The original pavement was Portland cement concrete, but the concrete had become badly cracked and was overlaid with dense-graded hot mix asphalt in 1994 using a 9.5 mm (3/8 inch) NMAS mix. The asphalt cement was modified with 6 percent LDPE. Prior to overlay, the concrete slabs were stabilized with pressurized grout injected under the slabs. Joints were covered with a geotextile fabric to help reduce the potential for reflective cracking.

The original 2,000 m (6,560 ft) runway had an additional 700 m (2,296 ft) extension added in 1997. However, the extension was constructed in a built-up area of reclaimed land and there have been some consolidation issues since construction. As a result, a maintenance repair was made in 2005 on the high speed exit ramp and a few recent repairs have been made near the end of the runway due to isolated consolidation (Figure 3.4). The repairs consisted of removing and replacing 7 cm (2.75 in) in depth with LDPE modified HMA.

A Pavement Condition Index (PCI) survey in 2004 indicated the runway had only two years of remaining life. A seal coat was applied in 2005 to extend the service life of the pavement, and a follow-up PCI in 2007 indicated the pavement had a remaining service life of ten years. A seal coat obviously would not extend the pavement life so significantly, but there was no explanation for the apparent error in the 2004 PCI. In 2004 the transverse joints were routed 2 cm (0.8 in) deep and 1 cm (0.4 in) wide and the cracks were sealed with "Seal Master" joint sealer.

The pavement appeared to be performing quite well during this evaluation with only 1.2 cm (0.5 in) of rutting on the end of the runway where planes sit waiting for permission to take off (Figure 3.5). Rubber build up is removed two times a year and there are no grooves in the pavement.

FIGURE 3.4 2005 Repair on Runway Extension.

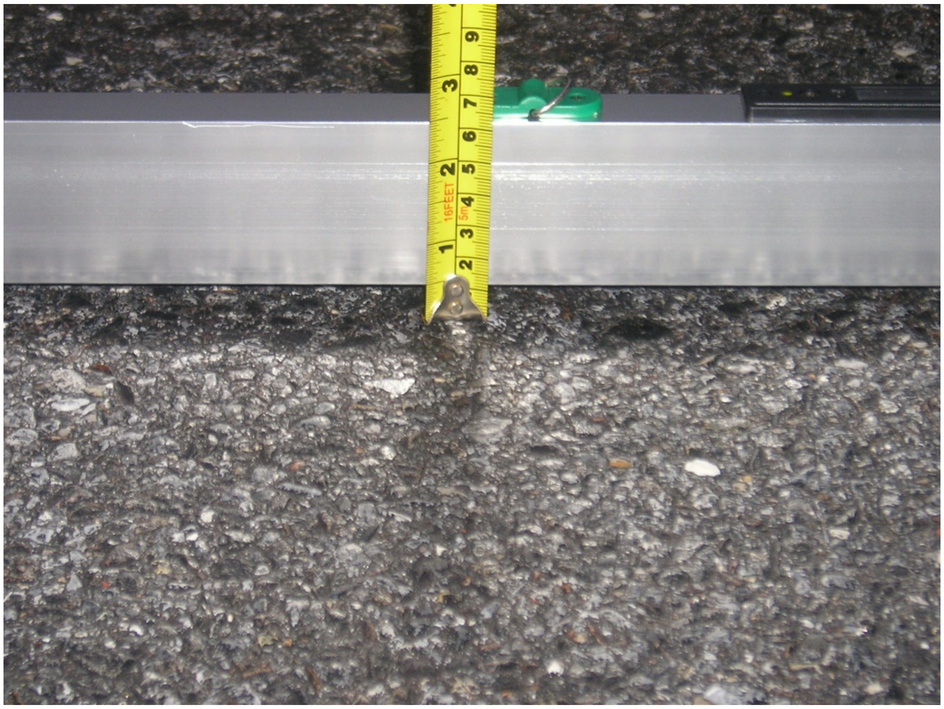

FIGURE 3.5 Maximum Rutting 1.2 cm (0.5 in).

3.2.4 Harbin Taiping International Airport

The Harbin airport was originally concrete pavement constructed in 1979 and is 2200 m (7218 ft) long and 45 m (148 ft) wide. The overall structural thickness is 31 to 34 cm (12.2 to 13.4 in) of concrete pavement, 6 cm (2.4 in) of asphalt base course, 7 cm (2.8 in) of SMA-20 and 5 cm (2 in) of SMA-13. The asphalt overlay, including the SMA layers, was placed in 2002. Prior to the overlay, high pressure grouting was used to under seal the concrete slabs and felt fabric was placed over all of the cracks to help retard reflective cracking. Harbin experiences very cold winters with a low of -40°C (-40 °F). For that reason, several steps have been taken to reduce thermal cracking in cold weather:

- Use high penetration base asphalt (130 pen) with the addition of 8 percent SBS modifier to produce a Superpave PG 64-32.
- Use 0.5 percent fiber stabilizer (other areas normally use 0.3 percent).
- Evaluate binder and modifier compatibility by use of softening point, flash point, and linear vs. star-shaped molecular chain.

The asphalt base layer used AC-25 (80-100 pen). The SMA mixes used 120-140 penetration asphalt before polymer modification. All mixes required the ductility to be at least 150 cm (59 in) when tested at 15°C (59 °F). To determine the amount of polymer modifier needed, samples are prepared at 3, 5, 7, and 9 percent SBS by mass of AC and selection is based on penetration, softening point, and ductility results. The elastic recovery test is also used and samples must have greater than 95 percent recovery when tested at 15°C (59 °F).

The mix consisted of basalt coarse aggregate, limestone fine aggregate, manufactured sand, and limestone dust for filler.

The gradation of the SMA-13 is given in the following table.

TABLE 3.7 Gradation Range for SMA-13

Sieve Size, mm (in)	% Passing
16 (5/8)	100
13.2 (0.525)	90 – 100
9.5 (3/8)	50 – 75
4.75 (No. 4)	20 – 34
2.36 (No. 8)	15 – 26
0.075 (No. 200)	8 – 12

One month after construction four cracks developed in one night and grew to 1 cm (0.4 in) wide after another month. Eleven cracks developed by the end of the second winter and the cracks grew in width to about 1 to 2 cm (0.4 to 0.8 in) in width with the widest cracks being up to 4 cm (1.6 in) during the coldest months. At the time of this review there were 21 transverse cracks that had developed and the cracks averaged about 2 cm (0.8 in) wide. There was some concern among Harbin personnel that the cracks may be top-down cracking and there were some thoughts that the mix may be permeable due to low density. However, no cores had been taken to evaluate the in-place density or the permeability of the asphalt layer.

While a few cracks appeared to be related to thermal cracking (Figure 3.6), during the review it seemed apparent that most of the cracking was reflective cracking along the old concrete transverse joints (Figure 3.7). Based on the width and location of the cracks, it appears the entire concrete slab and asphalt overlay is moving as a whole and that frost-heave is a major cause of distress.

It was also apparent by the many discolored spots (Figure 3.8) that some significant stripping has occurred due to moisture trapped in the underlying structure. AASHTO T 283 is conducted during mix design to check for moisture susceptibility and at least 90 percent TSR is required with one freeze-thaw cycle. However, the leaching of fine aggregate observed on the surface mixture is an indication of underlying moisture problems.

FIGURE 3.6 Apparent Thermal Crack Between Reflective Concrete Joint Crack, Harbin.

FIGURE 3.7 Transverse Reflective Cracking at Harbin.

FIGURE 3.8 Surface Staining from Moisture Damage in Underlying Layers at Harbin.

3.3 EUROPE

3.3.1 Belgium – Brussels National Airport

The main runway of Brussels National Airport, 07L-25R, is 3,200 m (10,496 ft) long and 45 m (148 ft) wide. The original concrete pavement was overlaid with 180 to 340 mm (7 to 13.4 inches) of HMA in 1980. An anti-skid layer was added in 1988. Due to extensive cracking, the runway was overlaid in 1996; 60 mm (2.4 inch) was milled over the whole runway and an additional 70 mm (2.8 inch) was removed in the center of the runway for 2/3 of its length to alter the cross-slope (*58*). An anti-cracking layer (SAMI) was placed at a depth of 130 mm (5.1 inch), 70 mm (2.8 inch) of dense graded HMA, and 60 mm (2.4 inch) of SMA surface.

SMA was selected as the surface course for runway 07L-25R for two main reasons: 1) relatively low air voids for durability, and 2) potential for good skid resistance. It was thought that the use of SMA might provide sufficient macrotexture to meet the International Civil Aviation Organization's (ICAO) requirements, without using the expensive anti-skid layer (*58*).

The target design gradation for the SMA is shown in Table 3.8. The mix incorporated 0.3 percent cellulose fibers. The mixture was produced with 6.85 percent of a modified (elastomeric polymer) binder. The in-place density was measured at 2,108 locations using a nuclear density gauge. The average in-place air voids was 3.8 percent (*personal communication with C. De Backer*). The specifications required that the average in-place air voids for a lot be between 3 and 5 percent and that no individual test exceed 8 percent.

TABLE 3.8 Target Gradation for Brussels National Airport (*Personal communication with C. De Backer*)

Sieve Size, mm (in)	Percent Passing
14 (0.55)	96.6
10 (0.39)	77.1
7 (0.28)	49.8
4 (0.16)	28.8
2 (0.08)	23.8
1 (0.04)	19.5
0.5 (0.02)	15.8
0.25 (0.01)	13.3
0.125 (0.005)	11.6
0.080 (0.003)	10.3

Friction tests were conducted on both the SMA and the anti-skid layer to assess the need for winter maintenance by the Belgium Road Research Center (BRRC) (*58*). The British Pendulum test was used to monitor the friction values in the laboratory. Testing was performed on samples having an average temperature between -4 and -6 °C (25 and 21 °F). The water temperature that led to ice formation was between 2 and 3 °C for both the SMA and the anti-skid layer. The British Pendulum tests were performed 20 minutes after spraying the deicing agent. A "slippery" condition was defined as a British Pendulum Number (BPN) of less than 30; a "safe" condition was achieved when the BPN equaled or exceeded 70. Testing indicated that the thickness of the ice glaze which caused the slippery condition was approximately 0.6 mm (0.02 in) for the SMA whereas the thickness of ice glaze that caused a slippery condition for the anti-skid layer was 1.2 to 1.5 mm (0.05 to 0.06 inch). Starting with the same initial ice thickness (1.5 mm [0.06 inch]), the number of applications of deicing agent required to achieve a safe condition was double for the SMA (six sprayings at 100 g/m^2 compared to three sprayings of 100 g/m^2) (personal communication with C. De Backer). Since the laboratory testing indicated that approximately double the amount of deicing agent would be required for the SMA as compared to the anti-skid layer, further use of the SMA as a surface course was suspended and an anti-skid layer was placed on Brussels National Airport.

In terms of pavement performance, the SMA placed on runway 07L-25R has performed well. The condition of the runway is still good and no repairs have been required (*personal communication with C. De Backer 2006*).

3.3.2 France

SMA is not used on airfields in France. The French have a true performance based specification for HMA for airfields. The French use a performance based specification called BBA, class 1, 2, or 3. The specification does not include parameters for gradation or volumetric properties. Instead, it specifies performance related test criteria such as resistance to moisture damage, a wheel-tracking rutting test, complex modulus, and fatigue resistance. The mixes do tend to be coarse-graded (personal communication with Jean-Paul Michaut).

3.3.3 Germany

SMA was developed in Germany. Although SMA is used on airfields in Germany, dense-graded mixes are also used. Specifications in Germany are developed by the "FGSV," which roughly translates to Research Group for Street and Traffic Construction. In 2005, the FGSC developed a "Merkblatt", or guidelines for the construction of airfields with asphalt (*59*). A copy of the Merkblatt was provided by Dr. Heinrich Els, manager of the German Asphalt Association (DAV). Two SMA gradations are recommended, 0/8S and 0/11S, which approximately correspond to a U.S. 9.5 mm nominal maximum aggregate size (NMAS) and a 12.5 NMAS based on the ZTV-Asphalt StB 01 specifications. The "S" designation stands for "schwer" or heavy (*21*) and refers to the fact that natural sand is not allowed in these particular SMA mixes. The 0/8S gradation is specified for areas with lower loadings. The ZTV-Asphalt StB 2000 specifications (*59*) are summarized in Table 3.9.

TABLE 3.9 German SMA Specifications

SMA	0/11S	0/8S
Sieve Size, mm (in)	Percent Passing	
11.2 (0.44)	90-100	100
8.0 (0.31)	≤60	90-100
5.0 (0.20)	30-40	30-45
2.0 (0.08)	20-27	20-27
0.09 (0.0035)	9-13	10-13
Ratio of crushed to natural sand	1:0	1:0
Binder Grade (penetration grade)	50/70 (PmB 45)[1]	50/70 (PmB 45)[1]
Binder content % by mass	≥6.5	≥7.0
Fiber, % by mass	0.3-1.5	
Marshall Compaction Temperature, °C[2] (°F)	135 ± 5 (270 ± 9)	
Marshall Air Voids, %	3.0-4.0	3.0-4.0
Layer Thickness, cm	3.5-4.0	3.0-4.0
Application Rate, kg/m^3	85-100	70-100
In-Place Density, % Marshall	≥ 97	
In-Place Air Voids, %	≤6.0	

[1]PmB 45, a polymer modified binder, roughly equivalent to a 76-XX is specified for air fields.
[2]Compaction temperature for PmB 45 is 145 ± 5 °C (293 ± 9)

Marshall stability and flow values are not specified. They are "unfit" for SMA. Two specific changes from the highway specifications are recommended: 1) PmB 45 is specified for the binder, and 2) there is an alteration to the void content. Based on the translation, it is believed that the in-place air void content is reduced to ≤ 5.0 percent.

In Germany, grit (clean crushed fine aggregate) is applied to the surface of the SMA while it is still hot and rolled into the surface to "deaden" it. Based on a 2003 SMA study tour to Germany on SMA sponsored by the Virginia Asphalt Association, the grit is applied to increase the initial and long-term skid resistance as well as reduce permeability. A grit size of ≤ 4.0 mm (No. 5) is recommended to reduce the potential for foreign object damage (FOD) (*59*).

3.3.3.1 Hamburg Airport

SMA was used to resurface a runway at the Hamburg Airport in 2001. The mix design is shown in Table 3.10 (Personal communication with Prem Naidoo). The sieve sizes have been converted to those commonly used in the U. S. The mixture is not as gap-graded as typical SMA mixtures.

TABLE 3.10 2001 Hamburg Runway Mix Design Gradation

Design Gradation	
Sieve Size, mm (in)	Percent Passing
12.5 (1/2)	100
9.5 (3/8)	98
4.75 (No. 4)	58
2.36 (No. 8)	42
0.600 (No. 30)	24
0.075 (No. 200)	11
Mixture Properties	
Cellulose Fiber	0.4%
Binder Grade	Sasobit modified with 35 pen[1]
Binder Content	7.0%
Marshall VTM, %	3.3
VMA, %	19.2
VFA, %	82.8
Hamburg Wheel Tracking Rut Depth, mm (in)	3.2 (0.125 in)

[1]82.0°C (180 °F) Softening Point – In the researcher's experience this would be a PG 82-XX binder.

3.3.3.2 Spangdahlem U. S. Air Force Base

The U.S. Air Force maintains a base at Spangdahlem, Germany. In 2007, a runway received a 50 mm (2-inch) mill and overlay of SMA. The SMA was constructed in accordance with ZTV Asphalt StB 01 using the 0/11 gradation. The mix was produced using two different asphalt plants (both using same aggregates and mix design). The production gradation, asphalt content, and laboratory air voids are shown in Table 3.11 based on 22 samples. The data in Table 3.11 indicates that the mix was very consistent. The average laboratory air voids were lower than what might typically be expected in production in the U.S. Twenty-five cores taken from the pavement had an average air void content of 2.3 percent with a standard deviation of 0.85 percent. Grit was applied to the surface and rolled in. Loose grit particles were removed by water blasting prior to opening the runway to help prevent foreign object damage (*60*).

TABLE 3.11 Spangdahlem Production Data

Sieve Size, mm (in)	Average Percent Passing	Standard Deviation
19.0 (3/4)	100.0	0.00
12.5 (1/2)	100	0.30
9.5 (3/8)	79	1.85
4.75 (No. 4)	37	1.95
2.36 (No. 8)	25	1.06
1.18 (No. 16)	19	0.76
0.600 (No. 30)	16	0.59
0.300 (No. 50)	13	0.49
0.150 (No. 100)	11	0.45
0.075 (No. 200)	8.8	0.39
AC,%	6.9	0.25
Lab Voids	1.9	0.82

Moisture was commonly observed migrating up through cracks in the pavement in warm weather prior to placing the overlay. After milling the pavement, there was heavy rain in the area for approximately two weeks before placing the SMA, allowing the surface to become saturated. Although the milled surface was allowed to dry prior to paving, moisture was still trapped in the pavement structure. The low air void SMA apparently does not readily allow the water vapor to escape. In warm weather, small diameter blisters have formed in the pavement surface (Figure 3.9). The blisters have been deflated by drilling a small hole in them and then rolling the affected area to ensure it bonds with the underlying layer (*60*)

FIGURE 3.9 Small Blister under Straightedge (*60*).

3.3.4 Italy

The U.S. Air Force maintains a base at Aviano, Italy. In 1999, a runway received a 50 mm (2-inch) overlay of SMA. The SMA basically followed the UFGS 6S-32 12 17 specifications (Tables 5 and 6) with the following exceptions (*61*):

- Gradation was finer than specified [on the 2.36 mm (No. 8)sieve],
- A 75-blow compaction effort was used instead of a 50-blow compaction effort [similar to the Chinese specifications],
- VMA was apparently lower than specified, resulting in a lower than specified design asphalt content (5.4 percent).
- Contractor's Marshall air void contents were low; 1.6 percent with 75-blow Marshall (as expected) and 3.6 percent with 50-blow Marshall.
- Fiber was not used. It is recommended but not required. Fat spots were observed after construction.

The Italian specifications for 0/15 SMA and UFGS 6S-32 12 17, and the mix design gradation used for Aviano AFB are shown in Table 3.12. The mix design properties are shown in Table 3.13 (Personal communication with Al Fraga).

TABLE 3.12 Mix Design Gradation and Specifications for Aviano Air Force Base

Sieve Size, mm	Italian 0/15 Specification % Passing	Sieve Size, mm	Aviano JMF % Passing	Unified Specification % Passing
		19.0	100	100
15.0	80 – 100			
		12.5	96.7	90 – 100
10.0	46 – 66			
		9.5	69.8	50 – 85
5.0	30 – 44			
		4.75	36.6	20 – 40
		2.36	28.3	16 – 20
2.0	20 – 36			
		1.18	20.9	NA
		0.600	15.3	NA
0.420 (#40)	10 – 17			
		0.300	13.6	NA
0.180	9 – 15			
		0.150	12.0	NA
0.075	8 – 13	0.075	10.8	8 – 11

Note: 1 inch = 25.4 mm

TABLE 3.13 Mix Design Properties and Specifications for Aviano Air Force Base

Property	Italian SMA 0/15 Specification	Aviano JMF		Unified Specification
		75 - Blow	50 - Blow	
Binder Content, %	5.5 – 7.0	5.4		6.0 Min.
Air Voids, %	1 – 4	1.6	3.2	3 – 4
Layer Thickness, mm	40 – 50	50		NA
Marshall Stability, N	13,000 Min.	16,200	12,500	6,200 Min.
Stiffness, N/mm	2,000 Min.	3,188	2,307	NA
Indirect Tensile Stiffness[1], N/mm^2	0.80	0.79	0.74	NA
Indentation Test DIN 1996[1], mm	1.0 Max.	0.40	0.81	
VMA[2], %	NA	19.1		17 Min.
Flow, 0.01 inch	NA	20.1	22.4	8-16
LA Abrasion, % loss	NA	21.9		30 Max.
Sodium Sulfate Soundness Loss	NA	0.60% 4/8 agg., 0.33% 8/16 agg.		15% Max.
In-place Voids	NA	8.47% Paraffin Coated		NA
Field Compaction	NA	96.6% Mat, 92.4% Joints		94% Mat 92% Joints

[1] No details are provided on this test method
[2] VMA appears to have been determined from a blend of the dry aggregate, not from a compacted HMA sample

The contractor for the project was Dell'Agnese. The binder used on the project was a modified penetration graded binder. The specified penetration was 45 to 55 dmm. The measured pen was 61 dmm at 25°C (77 °F). The softening point of the binder was 87°C (189 °F) [in the authors experience, this is a heavily modified binder, typically in excess of PG 82-XX. The coarse aggregate was a limestone source and a limestone (calcium carbonate) mineral filler was used. The mix temperature at the plant was specified at 175-180°C (347-356 °F), with a minimum compaction temperature of 160°C (320 °F).

The airfield was examined in September 2006 (*62*). No maintenance has been done in the last seven years except rubber removal. The friction numbers are reported to be good, even though the pavement was not grooved. It has been observed that it takes longer for the SMA to dry after a rainfall event compared to dense-graded HMA. This is probably due to the high macrotexture (Figure 3.10).

FIGURE 3.10 Typical SMA Surface Texture at Aviano.

There is a fair amount of rubber build-up on the ends of the runway (Figure 3.11). The rubber build-up is reportedly removed twice a year by water blasting. No tendency for raveling was observed resulting from the water blasting. Foreign object damage has not been a problem either. This may be due to the higher binder film-thickness resulting from the SMA coupled with the use of an SBS modified binder. There was some variability observed between paving lanes indicating that the SMA mix was changing during construction (Figure 3.12). Some lanes look more open and others look tighter. Even though this problem has not resulted in performance issues to date it does indicate a need to have better control during construction.

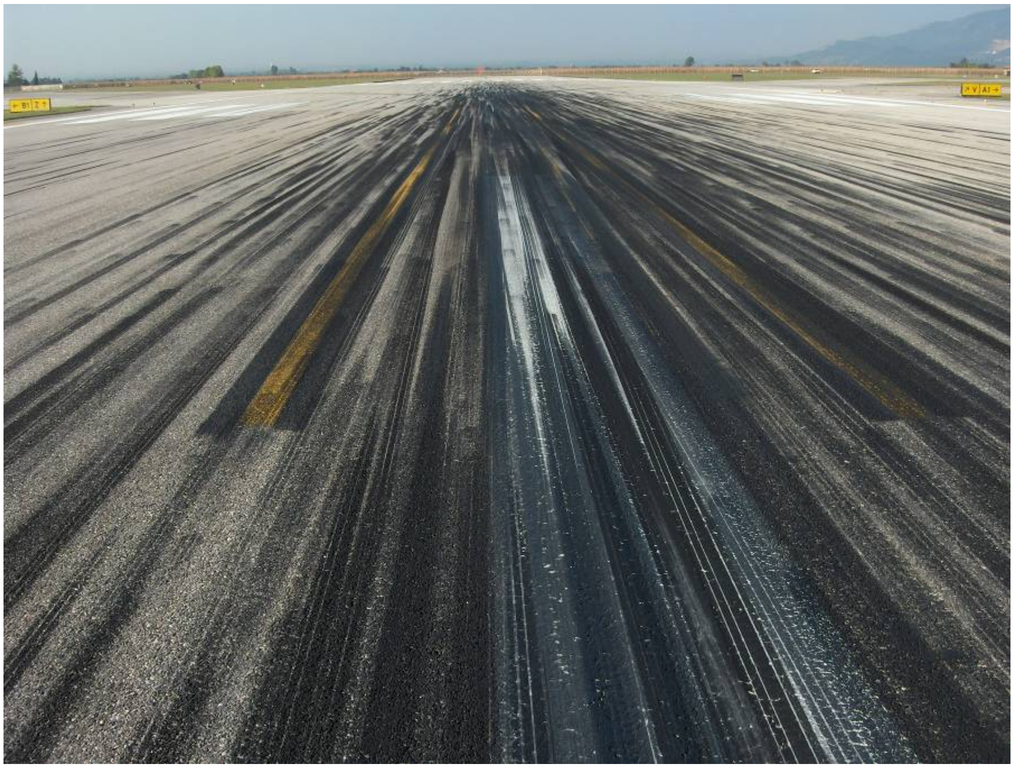

Figure 3.11 Rubber Build-Up at Aviano.

FIGURE 3.12 Differences in Surface Texture between Paving Lanes.

The pavement was closely inspected for cracking, particularly at the longitudinal joints. The longitudinal joints appeared to be in good condition with no cracking. The only cracking that was observed was a transverse reflective crack a few feet from where the asphalt tied into the concrete ends (Figure 3.13). This is likely the result of a buried slab or some similar underlying condition that would cause a crack to reflect through the overlay. On the surface of the entire runway, there was only one other small crack, approximately 1-foot long that was observed. Little to no raveling was observed.

FIGURE 3.13 Transverse Crack Near the Concrete End.

When this pavement was inspected in 2000, it was noted that moisture was migrating up through the pavement surface. Some water stains were noted (in 2006) near the runway shoulder, indicating that some water was continuing to migrate to the surface during the hot portions of the year. There was no deterioration in these stained areas, so this flow of water through the surface did not appear to be a major problem.

3.3.5 Norway

Avinor, the Norwegian Civil Aviation Authority, owns and operates 46 airports in Norway with 7.6 million m^2 of pavement, 97 percent of which is surfaced with asphalt. Since 1992, SMA has been used on 15 airports in Norway (Figure 3.14), including some 16 runways (Personal communication with Geir Lange). For the first three years, a 0/16 mm SMA gradation was used. Then Avinor switched to a 0/11 mm gradation. The last runway, constructed in 2002, used a 0/8 mm gradation. A minimum of 6.4 percent asphalt binder is specified. Avinor has changed its practices in the last five years due to problems with deicing agents (liquid) and asphalt pavements. Raveling and moisture damage were reported in both Norway and Sweden when they changed from deicing with urea to deicing with potassium acetate and potassium formate. The problems occurred with both dense-graded and SMA pavements (*4*).

Avinor's design procedure is still unclear. They stated that they got low stability with 75-blow Marshall designs. For the last ten years, they have apparently used wheel-tracking tests to design their SMA. 0/11 mm SMA is compared to a standard 0/11 mm dense graded HMA produced with 5.6 percent of a 160/200 pen asphalt. Other literature suggests problems with the deicing agents, noted above, may be related to the use of soft binder. The problem apparently improved with the use of the equivalent of a PG 64-28 binder.

There have also been some maintenance concerns. Mr. Lange reports that the SMA surface stays wet longer than dense-graded mixtures with lower macrotexture and therefore require a greater usage of deicing agents (recall this was also a concern in Brussels). They believe the increased usage of deicing agents leads to a more rapid deterioration of the pavement, particularly in the form of raveling. Raveling was reported after six years of service. Fog seals and rejuvenators have been used with success to maintain Avinor's SMA pavements. The oldest SMA pavement, Molde Airport, received rejuvenator treatments in 1997, 2000, and 2005.

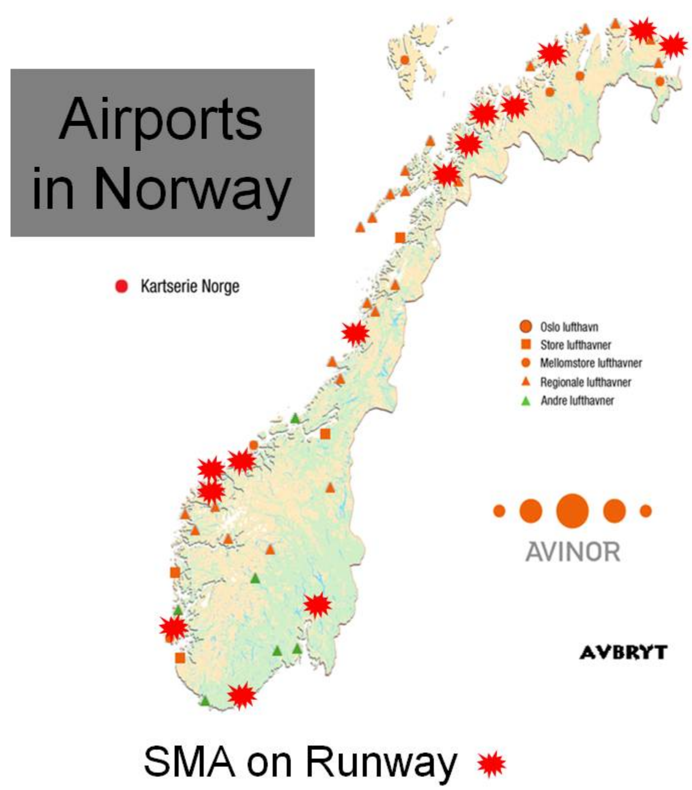

FIGURE 3.14 Location of Norwegian Airfields with SMA Surfaces.

3.3.6 Sweden

Although SMA is used extensively on roadways in Sweden, Fredrik Nilsson with the Swedish Civil Aviation Administration (CAA) reports that SMA has not been used on the civil aviation fields. Instead, the Swedish CAA uses an almost open graded mixture with a 16 mm maximum aggregate size designed with the Marshall method.

3.4 NORTH AMERICA

3.4.1 Mexico

SMA has been used on at least two airfields in Mexico: Mexicali, Baja California and Guadalajara, Jalisco. SMA was placed on a runway at Mexicali in 2004 and 2005. The SMA placed at Mexicali was an overlay of existing Portland cement concrete slabs. SMA was placed on runway 10-28 of Guadalajara airport in 2005.

Both the Guadalajara and Mexicali projects used a polymer modified AC-20 binder. The binder specifications are shown in Table 3.14. In Guadalajara, the polymer was required to be SBS. Cellulose fibers are required at a rate of 0.7 kg per cubic meter of binder. The fiber is added in a pellatized form. The pellets are composed of cellulose fiber and modified asphalt. A minimum of 50 percent of the pellet must be cellulose. The binder in the pellets must be compatible with the binder used in the SMA mixture.

TABLE 3.14 Polymer Modified AC-20 Binder Specifications for Guadalajara and Mexicali (*76, 77*)

Property	Specification Value
Original Binder	
Rotational Viscosity at 135 °C	4 Pa.s, maximum
Penetration at 25 °C	40 0.1 mm, minimum
Penetration at 4 °C (200g, 60 s)	25 0.1 mm, minimum
Softening Point	55 °C, minimum
Polymer Separation based on Softening Point	3 °C, maximum
Thin-Film Oven Residue	
Retained Penetration at 4 °C	65%
Elastic Recovery	50%
Dynamic Shear Rheometer, G*/sin δ	2.2 kPa, minimum
Dynamic Shear Rheometer Phase Angle	70-75, degrees

The aggregate quality requirements for SMA in Mexico are very high. The requirements are summarized in Table 3.15. Additionally, the natural sand content is limited to 5 percent.

The design aggregate gradation for Guadalajara and Mexicali are shown in Table 3.16. Both mixes are a 12.5 mm (1/2 in) NMAS. One notable difference between the Mexican SMA specifications and those used in other countries is the dust content. Most SMA specifications specify 8 to 12 percent passing the 0.075 mm (No. 200) sieve.

TABLE 3.15 Aggregate Quality Requirements for Guadalajara and Mexicali (*76, 77*)

Property	Specification Value
Percent Crushed by Impact	100 %
Shape – Flat Indices (NLT 354/91)	20 %, maximum
L. A. Abrasion Loss (IRAM 1532	25 %, maximum
Sand Equivalent of –4.75 mm (No. 4) material (VN E10-82)	60 %, minimum
Adhered Dust (VN E68-75)	0.5 %, maximum
Boil Test for Binder Adhesion Retained Coating (ASTM D3625-96)	95 %, minimum
Sodium Sulfate Soundness loss (ASTM C 88)	10 %, maximum
Water absorption (ASTM C 127 and C 128)	2 %, maximum

TABLE 3.16 Design Gradation Bands for Guadalajara and Mexicali (*76, 77*)

Sieve Size , mm (in)	Percent Passing	
	Guadalajara	Mexicali
19.0 (3/4)	100	100
12.5 (1/2)	70-90	100
4.75 (No. 4)	30-45	30-45
2.36 (No. 8)	20-30	20-30
0.075 (No. 200)	3-6	3-6

SMA is designed using the Marshall method with 75 blows per face. The design volumetric properties are shown in Table 3.17. The design compaction is higher than typically seen for SMA. The design VMA and corresponding VFA are lower. Most agencies specify a minimum VMA of 17 percent. The Mexican specification includes both Marshall stability and a Marshall stability/flow ratio.

TABLE 3.17 Design Volumetric Requirements for Guadalajara and Mexicali (*76, 77*)

Property	Specification
Design air voids, %	4.0
VMA, %	14 minimum
VFA, %	70 – 76
Marshall Stability	900 kg (1,980 lb) minimum
Stability/Flow ratio	2,600-3,900 kg/cm

The pavement is to be compacted to 3 to 6 percent air voids, based on theoretical maximum density. In place air voids are based on a minimum of six tests per day and a minimum of one test every 100 linear meters.

The SMA pavement at Guadalajara has reportedly had problems with raveling. The asphalt layer softened when it was saturated with moisture. Approximately 1-inch deep raveling occurred where the engine exhaust impinged on the runway during takeoff. Patches were applied to these areas. When the asphalt dried out, it recovered its original characteristics (*personal communication with Marcos Javier Ochoa Gonzalez*).

The areas saturated with moisture may be an indication of low in-place density. The low in-place density may be exacerbated by certain specification parameters including: lower design minimum VMA, higher laboratory compaction effort, and low percentage of filler passing the No. 200 (0.075 mm) sieve.

3.4.2 United States

SMA was placed on Taxiway "H" at Indianapolis International Airport in Indianapolis, Indiana during the fall of 2005. The section is approximately 100 feet wide by 1,832 feet long. The rehabilitation consisted of a mill and inlay with 2.75 inches of a 19.0 mm NMAS Superpave mix and 1.75 inches of a 12.5 mm NMAS SMA (*63*). The 19.0 mm P-401 Modified Superpave mix was produced according to Engineering brief #59 using PG 76-22 binder. Indiana DOT highway specifications were used for the SMA (INDOT Section 410). Indiana typically specifies a 9.5 mm NMAS SMA, however, due to the recommended lift thicknesses for an FAA surface mix, a 12.5 mm NMAS was selected. INDOT Section 410 specifies that the SMA be designed in accordance with AASHTO PP 41 to meet the specifications of AASHTO MP 8. The mixture was designed using the gyratory compactor (N_{design} = 100). A PG 76-22 binder was specified for the SMA. The coarse aggregate was a #11 steel slag with an L.A. Abrasion loss of 12.5 percent. A #9 slag and two limestone fine aggregate sources were also used. The combined aggregate G_{sb} = 3.223. The mixture included 0.3 percent cellulose fiber. The design gradation by both mass and volume is shown in Table 3.18. When SMA contains aggregates with very different G_{sb} values, percent by volume provides a better representation of the gradation. The design VMA was 18.2 percent, resulting in an optimum asphalt content of 5.6 percent at 4.0 percent air voids. The VCA_{DRC} = 42.7 percent and the VCA_{Mix} = 33.5 percent.

TABLE 3.18 Taxiway "H" Job Mix Formula

Sieve Size, mm (in)	Percent Passing		
	12.5 mm SMA JMF, mass (volume)	INDOT 12.5 mm SMA Specifications	AASHTO 12.5 mm SMA Specifications
19.0 (3/4)	100 (100)	100.0	100
12.5 (1/2)	96.9 (97.3)	90.0 – 99.0	90-100
9.5 (3/8)	76.8 (78.2)	50.0 – 85.0	50-80
4.75 (No. 4)	30.2 (33.7)	20.0 – 40.0	20-35
2.36 (No. 8)	19.5 (22.7)	16.0 – 28.0	16-24
1.18 (No. 16)	15.6 (17.9)	-	-
0.600 (No. 30)	12.8 (14.6)	-	-
0.300 (No. 50)	10.9 (12.2)	-	-
0.150 (No. 100)	9.3 (10.4)	-	-
0.075 (No. 200)	7.1 (8.0)	8.0 – 11.0	8-11

INDOT 410 specifications require a target mat density of 93 percent of G_{mm} based on cores. Joint densities are not specified. After construction, some white and brown stains were noted on the surface of the pavement, sometimes accompanied by raised spots. The stains were more prominent in the outer lanes, particularly near the joints. The stained areas were mapped in August 2006. The stained areas have been attributed to water moving through the pavement and possibly reacting with deleterious materials.

In December 2006, the pavement was inspected by one of the authors. Figure 3.15 shows an overview of Taxiway "H", facing the new midfield terminal (under construction). Figure 3.16 shows a close-up of the surface texture and of the longitudinal joints, indicated by the steel ruler. Limited staining and raised areas were still visible (Figure 3.17), but to a lesser extent than reported in August (*63*). Friction test results were reportedly better than other surfaces at the airport. Reportedly, additional paint was required when painting the line markings to provide adequate coverage.

FIGURE 3.15 Overview of Taxiway "H," Indianapolis International Airport.

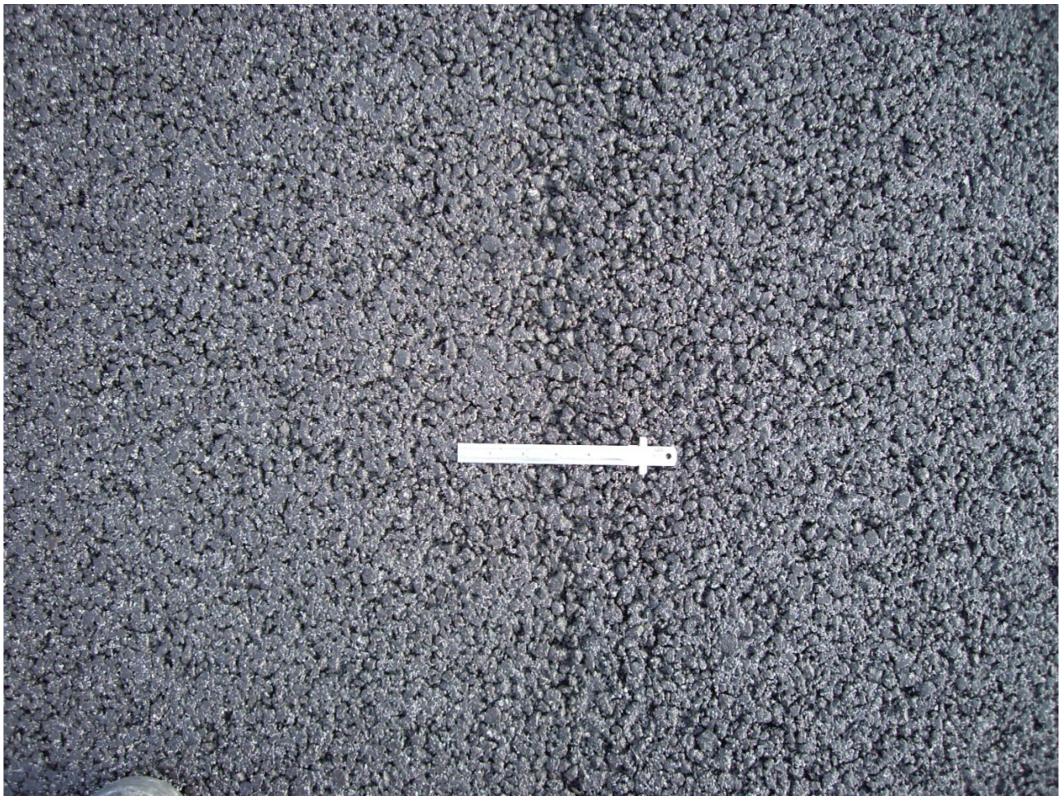

FIGURE 3.16 Close-up of SMA Surface Texture.

FIGURE 3.17 Stained Areas on Outside Lane.

CHAPTER 4

LABORATORY EVALUATION OF SMA FOR AIRFIELDS

4.1 SUMMARY OF LITERATURE REVIEW AND CURRENT USE OF SMA ON AIRFIELDS

The following section summarizes the information gained from Tasks 1 and 2 and presented in Chapters 2 and 3 with regard to SMA mix design. This summary was used to develop the experimental design. When discussing potential specification limits for investigation during the laboratory testing phase, an effort was made to include of the widest range of materials possible, if the resulting performance is acceptable. This was done in order to maximize the use of locally available materials.

The coarse aggregate for SMA mixtures needs to be angular (crushed), cubical, and hard. Although some specifications require 100 percent crushed particles, AASHTO MP-8 (*15*) only requires 90 percent two-crushed faces, determined according to ASTM D5821. This seems to be a reasonable specification since it would potentially allow the use of crushed gravel sources.

There is an interaction between the percent of flat and elongated particles and aggregate breakdown. With the exception of Georgia DOT, all of the specifications which specified flat and elongated particles specified a maximum of 5 percent 5:1, and 20 percent 3:1 for the maximum to minimum dimension. Georgia DOT's specification is slightly more restrictive (based on the measurement technique).

The FHWA SMA Technical Working Group (TWG) (*7*) specified a maximum L.A. Abrasion loss of 30 percent. Stuart (*1*) recommended a maximum L.A. Abrasion loss of 40 percent based on his review of European practice. States, such as Georgia and Wisconsin, have allowed aggregates with up to 45 percent L.A. Abrasion loss, although Schmiedlin and Bischoff (*26*) noted an increased rate of reflective cracking with increased L.A. Abrasion loss. This appears to be an area that should be investigated as part of the research. Higher L.A. Abrasion loss specifications would allow the use of more locally available aggregates and thus reduce cost. However, the higher tire pressures found on large commercial and military aircraft may cause a breakdown of the aggregate contact points under load. The maximum L.A. Abrasion loss allowed may impact the required gradation limits. When considering the breakpoint sieve (the 4.75 mm (No. 4) sieve for 12.5 mm (1/2 inch) NMAS SMA), it is anticipated that coarser mixes would be required for aggregates with higher L.A. Abrasion loss values and finer mixes for aggregates with lower L.A. Abrasion loss values.

German guidelines for the use of asphalt on airfields specify 8 or 11 mm nominal maximum aggregates size mixtures (NMAS), with 11 mm NMAS being used for heavier loading conditions (*59*). The FHWA SMA TWG (*7*) gradation specification was for a 16 mm NMAS. Norway reports moving toward smaller NMAS mixtures with time. The current

Unified Facilities specifications are for a 12.5 mm NMAS SMA mixture (*19*). Based on the gradations reported for in-service SMA airfield, shown in Table 4.1, a 12.5 mm NMAS seems to be most common. China uses both 13 and 15 mm NMAS SMA mixtures on airfields. The 12.5 mm NMAS SMA was selected for the laboratory portion of this study. Table 4.2 shows the range of specifications close to a 12.5 mm NMAS. The gradation for a given aggregate source in this study was adjusted to meet the volumetric requirements.

A variety of mineral fillers have been used in SMA. Limestone fillers are most commonly used in Germany. The modified Rigden voids tests can be used to assess the stiffening potential of various fillers. Fibers are typically added to SMA mixes at the rate of 0.3 percent by total weight of mix.

Both Germany and the U.S. have trended towards increased use of polymer modified binder in SMA. The Unified Facilities specification requires a two-grade high temperature bump from the recommended climatic grade determined with LTPPBind (*19*). PG 64-22 is the most common base climatic binder grade in the U.S. Therefore a PG 76-22 would meet the United Facilities specification.

A 50-blow Marshall effort was originally used to design SMA mixtures in Germany and when the technology was initially brought to the U.S. The 50-blow Marshall compaction effort is still used in Germany and China. Italy specifies a 75-blow Marshall compaction effort for SMA for airfields. Numerous research studies have been conducted to determine an appropriate laboratory compaction effort using the Superpave Gyratory Compactor (SGC). NCHRP 9-8 recommended 100 gyrations for aggregates with L.A. Abrasion loss values less than 30 percent and 70 gyrations for aggregates with L.A. Abrasion loss values greater than 30 percent (*10*). A recent study for Georgia DOT recommended a design compactive effort of 50 gyrations for the SGC (*35*). This recommendation has been adopted in Georgia DOT's specifications (as an alternative to a 50-blow Marshall compaction effort). NCHRP 9-9(1) recently recommended 50, 65, 80, and 100 gyrations for dense-grade mixes. The Marshall hammer generally causes more aggregate breakdown than the SGC. Aggregate breakdown increases with increasing gyration levels. It is important that the mix can be compacted to low air voids in the field.

Design air voids are generally specified between 3 and 4 percent for SMA. A minimum voids in mineral aggregate (VMA) of 17 is generally specified for SMA. Research conducted as part of NCHRP 9-8 recommended the use of voids in coarse aggregate (VCA) to ensure that a stone-on-stone skeleton is achieved (*10*). The VCA_{Mix} should be less than the VCA_{DRC} (dry-rodded condition) determined according to AASHTO T19.

TABLE 4.1 Design Gradations for Airfields Using SMA (Converted to U.S. Sieve Sizes)

Airfield	Cairns	Sydney	Brussels	Hamburg	Spangdahlem	Aviano	Indianapolis	Range		Average
Sieve Size, mm (in)	Percent Passing									
	Range	Range	JMF	JMF	Production	JMF	JMF	Lower	Upper	Average
19 (3/4)	100	100	100	100	100.0	100	100	100	100	100
12.5 (1/2)	91-93	83- 94	90	100	100	97	97	83	100	94
9.5 (3/8)	47-59	50-65	73	98	79	70	78	47	98	69
4.75 (No. 4)	25-34	21-32	35	58	37	37	34	21	58	35
2.36 (No. 8)	19-25	16-26	24	42	25	28	23	16	42	25
1.18 (No.16)			20		19	21	18	18	21	20
0.600 (No. 30)	12-18		17	24	16	15	15	12	24	17
0.300 (No. 50)		11-18	14		13	14	12	11	18	14
0.150 (No. 100)			12		11	12	10	10	12	11
0.075 (No. 200)	7.5-9.5	8-12	10.1	11	8.8	10.8	8	7.5	12.0	9.5

TABLE 4.2 Specification Ranges for Approximately 12.5 mm NMAS SMA (Converted to U.S. Sieve Sizes)

Airfield	Germany		Italy		China		Indiana DOT		Unified		AASHTO		Range	
Sieve Size, mm	Percent Passing													
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
19	100	100	96	100	100	100	100	100	100	100	100	100	96	100
12.5	93	100	70	89	82	94	90	99	90	100	90	100	70	100
9.5		80			45	65	50	85	50	85	50	80	45	85
4.75	29	39	29	43	22	34	20	40	20	40	20	35	20	43
2.36	22	29	22	30	18	27	16	28	16	20	16	24	16	30
1.18					14	22							14	22
0.600	14	19	13	19	12	19							12	19
0.300			10	16	10	16							10	16
0.150			9	14	9	14							9	14
0.075	8.7	12.6	7	13	8	12	8	11	8	11	8	11	7	13

Note: 1 inch = 25.4 mm

Based on the literature review, airfield specifications for SMA, and the data collected from in-service SMA on airfields to date, the following primary areas were recommended for additional study:

- Establishing limits for L.A. Abrasion - particularly with regard to breakdown of coarse aggregate contact points under the stresses induced from aircraft with high pressure tires.
- Binder Grade – Although modified asphalts are highly recommended for commercial or military aircraft, the base climatic grade may be suitable for general aviation fields.
- Laboratory compaction level - although the 50-blow Marshall effort has been the standard, higher compaction efforts are used for airfields in China and Italy. Further many contractors are losing their experience with the Marshall method in the U.S. since the advent of Superpave. It is important to balance the compaction effort to allow field compaction while preventing permanent deformation under heavy loads from commercial or military aircraft.
- Macrotexture and grooving – SMA offers increased macrotexture compared to dense-graded mixes. This increased macrotexture may alleviate the need for grooving. However, grooving SMA should be evaluated to ensure that the grooves will not breakdown over time and create FOD. The early friction of SMA pavements, until the surface binder film has worn off or is absorbed is also a concern.

4.2 RESEARCH APPROACH

The design parameters shown in Table 4.3 were selected for the laboratory study. The laboratory study was developed to evaluate the performance of SMA using a range of aggregate types with a corresponding range of L. A. Abrasion values in terms of rutting, cracking resistance, and moisture susceptibility. Testing was conducted over a range of asphalt contents corresponding to a range of laboratory compaction levels. P401 mixes were produced with PG 76-22 as control mixes. One subset was tested with PG 64-22 binder and a limestone aggregate for potential use on General Aviation airfields. Experiments were also conducted to assess fuel resistance, deicing resistance, and the durability of grooves in SMA. The complete experimental design is shown in Table 4.4.

TABLE 4.3 Design Parameters

Parameter	Design Range
Coarse Aggregate Angularity ASTM D5821	100% 1 face crushed min. 90% 2 face crushed min.
Flat and Elongated Particles ASTM D4791 by weight on blend of coarse aggregates	5% 5:1 max. 20% 3:1 max.
L.A. Abrasion Loss	45% max.
Binder Grade	PG 76-22
Laboratory Compaction Effort	50-Blow Marshall 50, 65, 80, 100 gyrations
Design Air Voids	3.0%
Minimum VMA	17.0
VCA_{Mix}	$< VCA_{DRC}$

TABLE 4.4 Testing Completed

Aggregate	Mix Design	Asphalt Content, %	Design Samples	Stability and Flow	Repeated Load Permanent Deformation			Hamburg	TTI Overlay Tester	Fuel Resistance	Deicing Resistance
					100 psi	200 psi	350 psi				
Diabase L.A.<20 PG 76-22	75-Blow P401	5.1	6	6				4	5		
	50-Blow SMA	7.5	24	3				4	5		
	50 Gyr. SMA	8.1	9					4			
	65 Gyr. SMA	7.6	9								
	80 Gyr. SMA	6.4	9								
	100 Gyr. SMA	6.7	7								
Ruby Granite L.A.≈25 PG 76-22	75-Blow P401	5.3	12	6				4	5		
	50-Blow SMA	7.3	67	3					5		
	50 Gyr. SMA	7.5	12								
	65 Gyr. SMA	7.4	4								
	80 Gyr. SMA	7.0	11								
	100 Gyr. SMA	6.6	8								
Gravel L.A. 20-30 PG 76-22	75-Blow P401	5.4	12	6	2	3		4	5	6	6
	50-Blow SMA	8.0	9	3				4	5	6	6
	50 Gyr. SMA	7.2	8		3	1		4			
	65 Gyr. SMA	7.0	10					NA			
	80 Gyr. SMA	6.8	10		3			4			
	100 Gyr. SMA	6.4	5		2	3		4			
Limestone L.A. 20-30 PG 76-22	75-Blow P401	5.4	18	6	2	3		4	5		
	50-Blow SMA	7.4	6	6					5		
	50 Gyr. SMA	7.8	6		3	3		4			
	65 Gyr. SMA	7.2	9					4			
	80 Gyr. SMA	7.0	11								
	100 Gyr. SMA	6.5	3		2	3		4			
Limestone L.A. 20-30 PG 64-22	75-Blow P401	5.5	9	6	3			4	5		
	50-Blow SMA	7.4	3	3				4	5		
	50 Gyr. SMA	7.6	7								
	65 Gyr. SMA	7.2	2		3	3					
Columbus Granite L.A. ≈40 PG 76-22	75-Blow P401	5.3	9	9	3	3	2	4	5	6	6
	50-Blow SMA	6.8	21	21	2	3	3	4	5	6	6
	50 Gyr. SMA	7.6	6		3	3	3	4			
	65 Gyr. SMA	7.3	9					4			
	80 Gyr. SMA	7.1	9		2	2	2				
	100 Gyr. SMA	6.8	6		2	3	2				
Totals			366	68	35	33	12	76	60	24	24

4.3 MATERIAL PROPERTIES

As noted in Chapters 1 and 2, SMA is normally produced with hard, cubical, crushed aggregates. However, agencies have adjusted aggregate specifications to accommodate locally available materials, often with great success. Since FAA specifications are used across the United States, it was desirable to evaluate as wide of a range of aggregate properties as possible. The primary factor considered in the range of aggregates was L.A. Abrasion, with a second factor being flat and elongated particles. The coarse aggregate properties are summarized in Table 4.5. The L.A. Abrasion loss values ranged from 18 for the diabase to 37 for one of the granite sources. The gravel source had an L. A. Abrasion loss of 30, the maximum limit in several SMA specifications.

All of the aggregate sources except the gravel met the maximum of 20 percent 3:1 and maximum of 5 percent 5:1 flat and elongated particles. The gravel source exceeded the flat and elongated percentages and had an L.A. Abrasion value of 30 percent. Recall that aggregate breakdown is expected to be more of a problem with higher percentages of flat and elongated particles.

The voids in coarse aggregate were determined using the design gradation and the material retained on the 4.75 mm sieve with the exception of the diabase source. There was insufficient 12.5 mm material in the diabase coarse aggregate to design a 12.5 mm NMAS SMA. Therefore, a 9.5 mm NMAS design was produced. The VCA_{DRC} of a 9.5 mm NMAS SMA mix is determined using the material retained on the 2.36 mm sieve. This, in conjunction with the high aggregate bulk specific gravity (G_{sb}) of the diabase source accounts for its higher VCA_{DRC}.

TABLE 4.5 Coarse Aggregate Properties

Aggregate Source	L.A. Abrasion Loss, ASTM C131, %	Flat and Elongated Particles ASTM D4791, %		Coarse Aggregate Angularity ASTM D5821,%		Voids in Coarse $Aggregate_{DRC}$, %[2]
		3:1	5:1	≥ 1FF[1]	≥ 2FF[1]	
Diabase	18	9.7	0.4	100	100	46.2
Columbus Granite	37	7.8	0	100	100	42.3, 42.4[3]
Ruby Granite	20	3.3	0	100	100	40.6
Gravel	30	49.3	9.8	97	77	42.2
Limestone	25	10.1	0	100	100	41.5

[1]FF = Fractured Faces
[2]Voids in Coarse $Aggregate_{DRC}$ for 50-blow Marshall gradation
[3]Blends 1 and 2, respectively.

The predominant binder used in the project was a PG 76-22. A PG 64-22 was also used with the limestone SMA and P401 mixtures to assess the possibility of using SMA on general aviation fields. The binders were graded according to AASHTO M320. In AASHTO M320, the failure criteria remain the same between grades, only the temperature changes. By testing binders at different temperatures, the actual failure temperatures can be determined or "true grade" of the binder. The failure temperatures for the different binder tests are reported in Table 4.6.

TABLE 4.6 Binder Properties

Test Method	PG 64-22	PG 76-22
Original Binder		
Rotational Viscosity at 135 °C, AASHTO T316	0.49	2.14
Dynamic Shear Rheometer, AASHTO T315, G*/sinδ =1.0 kPa	68.2[1]	82.5[1]
Rolling Thin Film Oven (RTFO) Aged Binder, AASHTO T240		
Mass Change, %	-0.046	-0.138
Dynamic Shear Rheometer, AASHTO T315, G*/sinδ =2.2 kPa	68.4[1]	82.9[1]
Pressure Aging Vessel (PAV) Aged Binder, AASHTO R28		
Dynamic Shear Rheometer, AASHTO T315, G*sinδ =5000 kPa	24.0[1]	19.4[1]
Bending Beam Rheometer, AASHTO T313, S(t) =300 Mpa	-24.8[1]	-27.1[1]
Bending Beam Rheometer, AASHTO T313, m=0.300	-23.2[1]	-24.1[1]
True Grade	68.2-23.2	82.5-24.1

[1]Failure Temperature, °C

All of the SMA mixtures contained 0.3 percent of cellulose fibers by total weight of mixture.

4.4 MIX DESIGNS

4.4.1 Design Gradations

Trial blends were established using stockpile gradations for each of the aggregate sources. Although some of the stockpile gradations were not ideal for the production of SMA, the gradations were not artificially altered in the laboratory to produce an ideal gradation as it was felt that contractors may face similar difficulties in production. Typically in an SMA mix design, the percent passing the 4.75 mm (No. 4) sieve is varied with a relatively constant percentage of material passing the 0.075 mm (No. 200) sieve to determine a design with the lowest acceptable VMA which meets the VCA requirements. For instance trial blends may be produced with 24, 28, and 32 percent passing the 4.75 mm (No. 4) sieve and the mineral filler adjusted to provide approximately 10 percent passing the 0.075 mm (No. 200) sieve. VMA is expected to drop during production, often up to 1.0 percent, due to breakdown of the aggregate. Achieving a gradation toward the center of the range of the sand-size sieves proved most difficult in some cases.

SMA designs were initially performed with each aggregate source using 50-blow (on each face) Marshall compaction effort. P401 control mixes were compacted with a 75-blow Marshall effort. Automatic hammers with flat faces and fixed bases were used for the designs. The volumetric properties from trial blends were evaluated against the volumetric properties shown in Table 4.3. Since higher design VMA values result in higher design asphalt contents, contractors in low-bid systems tend to design toward the minimum VMA value. Thus attempts were made to design mixtures with VMA values approximately 1.0

percent above the minimum to account for breakdown, but less than 19 percent. This was not possible in all cases.

The selected design gradations are shown in Table 4.7. The gradations reported are based on washed gradations performed on batched samples. Once a blend was determined with acceptable volumetric properties using the Marshall method, samples were compacted with the SGC, starting with 50 gyrations. It was expected that as gyrations increased, mixtures would fail volumetric properties and require adjustments to the design blend. This only occurred for the Columbus Granite source. The diabase blend falls outside the specification design range presented in Table 4.2 on the 9.5 mm (3/8 inch) sieve. This is because the diabase mixture was designed as a 9.5 mm (3/8 inch) NMAS SMA.

TABLE 4.7 SMA Design Gradations

	Diabase	Columbus Granite		Ruby Granite	Gravel	Limestone	Design Range[1]
Sieve Size	Blend 2	Blend 2	Blend 1	Blend 8B	Blend 1	Blend 4	
19.0 (3/4)	100	100	100	100	100	100	96-100
12.5 (1/2)	100	97	94	100	95	90	70-100
9.5 (3/8)	95	68	62	69	65	64	45-85
4.75 (No. 4)	32	29	25	26	28	23	20-43
2.36 (No. 8)	22	24	18	20	22	12	16-30
1.18 (No. 16)	20	21	17	17	20	10	14-22
0.600 (No. 30)	18	19	13	15	16	9	12-19
0.300 (No. 50)	16	17	11	13	15	9	10-16
0.150 (No. 100)	13	15	10	12	13	8	9-14
0.075 (No. 200)	9.8	12.5	8.7	11.0	9.4	7.8	7-13

[1]From Table 4.2

The 4.75 mm (No. 4) and 2.36 mm (No. 8) sieves are the typical breakpoint sieves for 12.5 and 9.5 mm (1/2 and 3/8 inch) NMAS SMA mixtures, respectively. The breakpoint sieve, along with the percent passing the 0.075 mm (No. 200) sieve tend to have a large influence on the volumetric properties of an SMA mixture. The aggregate retained on the breakpoint sieve is used to determine the VCA_{DRC}. Coarser mixes, with lower percents passing the 4.75 mm (No. 4) sieve, tend to have lower VCA_{Mix} and often higher VMA. Mineral filler can be increased to reduce VMA or decreased to increase VMA.

The trends in the design gradations are generally as expected; except possibly for the gravel mixture. Typically, it would be expected that the gravel and limestone mixes would require lower percents passing the 4.75 mm (No. 4) and 0.075 mm (No. 200) sieves in order to achieve the minimum VMA. This is due to the fact that these aggregates tend to be less angular. By comparison, harder and more angular aggregates such as the diabase and granite would be expected to allow higher percents passing the 4.75 mm (No. 4) and 0.075 mm (No. 200) sieve. Recall the diabase mixture is actually a 9.5 mm (3/8 inch) NMAS. This accounts for the higher percent passing the 4.75 mm (No. 4) sieve. This mixture almost exactly follows the 30:20:10 guidelines for percents passing the 4.75, 2.36, and 0.075 mm sieves, respectively, noted in Chapter 2 (*3*). The gravel mixture has 28 percent passing the No. 4 and 9.4 percent passing the No. 200 and still has a high VMA for the 50-blow Marshall compaction effort (19.4 percent). It is believed that this is due to the high percent of flat and elongated particles (49.3 percent 3:1).

The Ruby Granite mixture was the most challenging to design. A total of 67 samples were prepared representing 9 trial blends with percents passing the No. 4 sieve ranging from 21 to 28 percent. Mixtures with a percent passing the 4 sieve greater than 26 percent failed VCA_{Ratio}. At 26 percent passing the No. 4 sieve, the design VMA is still high, even with 11.0 percent passing the No. 200 sieve. It is believed that part of the difficulty in obtaining a passing VCA_{Ratio} for the Ruby granite mixtures was due to the cubical nature of the aggregates. The Ruby granite only had 3.3 percent particles more flat and elongated than the 3:1 ratio.

The P401 designs used the same aggregate sources as the SMA mixtures. Local natural sands were not collected along with the SMA aggregates. Therefore, a single natural sand source was used for all of the P401 mixtures. An initial target blend was based on historical data of good performing airfield mix designs. The design gradations for the ¾ inch maximum P401 mixtures are shown in Table 4.8. The reported gradations are based on washed gradations performed on batched samples.

TABLE 4.8 19.0 mm (¾ inch) Maximum P401 Design Gradations

Sieve Size, mm (in)	Diabase	Columbus Granite	Ruby Granite	Gravel	Limestone	Design Range
	Blend 1	Blend 1	Blend 1	Blend 1	Blend 4	
19.0 (3/4)	99	100	100	100	100	100
12.5 (1/2)	81	97	98	90	94	79-99
9.5 (3/8)	70	77	84	81	86	68-88
4.75 (No. 4)	49	51	52	63	67	48-68
2.36 (No. 8)	38	41	37	49	44	33-53
1.18 (No. 16)	31	34	27	37	30	20-40
0.600 (No. 30)	23	25	19	26	19	14-30
0.300 (No. 50)	13	15	11	15	10	9-21
0.150 (No. 100)	8	9	7	9	7	6-16
0.075 (No. 200)	4.6	5.2	4.9	5.7	5.0	3-6

4.4.2 Volumetric Properties

4.4.2.1 SMA Mixtures

In-place air voids are critical to the performance of SMA. If in-place density is not achieved, the SMA may be permeable. Based on initial discussions between the research team and the project panel, 3 percent design air voids were initially targeted for determining optimum asphalt content. It was felt that the lower design air voids would correspond to improved density in the field.

Optimum asphalt content, VMA, and VCA_{Ratio} (VCA_{Mix} / VCA_{DRC}) for the SMA blends are presented in Table 4.9. The complete results are presented in Appendix A. The properties are presented at both 3 and 4 percent design air voids. In many cases the results in Table 4.9 were interpolated from actual design points which bracketed 3 and 4 percent air voids. Several trial blends for different aggregate sources were prepared with trial asphalt contents which initially produced air void contents above 3 percent. The rule of thumb used for Superpave mixes is that a 0.4 percent change in asphalt content will produce a 1 percent change in air voids. This approximation seems to be fairly good for other mixes too. However, for some of the SMA mixes, large increases in asphalt content did not produce 3 percent design voids and as the asphalt content was increased, the mixture would reach a point where it would fail VCA_{Ratio}. Closer examination indicated that the mixtures were on the so-called "wet side" of the VMA curve. This is illustrated in Figures 4.1 and 4.2. For the 50-gyrations samples in Figure 4.1 the air void content at 7 percent asphalt is 4.4 percent and at 8 percent asphalt the air void content only decreases to 3.9 percent. At the same time, the VMA has increased from 19.4 to 21.0 percent. This indicates that the additional asphalt is pushing the aggregate skeleton apart, creating more VMA. This is also indicated by an increase in the VCA_{Ratio} from 0.997 to 1.027. Examination of Table 4.9 indicates that the VMA is higher at 3 percent design voids in every case except the Columbus granite mixture with the 100 gyration compaction effort. The measured VMA is the same at 3 and 4 percent air voids for the 50-blow gravel mixture and 65 gyration diabase mixture. All of the remaining combinations of aggregate source and laboratory compaction were selected on the wet side of the VMA curve. Since the additional asphalt required to reduce the air voids from 4 to 3 percent must overwhelm the resulting increase in VMA, the optimum asphalt contents increased on average 0.6 percent from 7.0 to 7.6 percent for the 50-blow Marshall mixes.

TABLE 4.9 Summary of Volumetric Properties for SMA Mixtures

Aggregate	Blend	Lab Compaction	3% Air Voids			4% Air Voids		
			AC, %	VMA, %	VCA_{Ratio}	AC, %	VMA, %	VCA_{Ratio}
Columbus Granite	2	50-Blow	6.8	17.5	0.99	5.9	16.6	0.97
	2	50 Gyration	6.4	16.6	0.97	NA	NA	NA
	2	65 Gyration	6.3	16.4	0.97	NA	NA	NA
	1	50-Blow	7.7	19.6	0.93	7.1	19.0	0.92
	1	50 Gyration	7.6	19.4	0.93	7.1	19.1	0.92
	1	65 Gyration	7.3	18.9	0.92	6.5	18.1	0.90
	1	80 Gyration	7.1	18.5	0.91	6.7	18.4	0.91
	1	100 Gyration	6.8	17.6	0.90	6.4	17.8	0.90
Gravel	1	50-Blow	8.0	19.4	1.00	7.6	19.4	1.00
	1	50 Gyration	7.2	18.3	0.98	6.8	18.2	0.98
	1	65 Gyration	7.0	17.8	0.97	6.4	17.3	0.96
	1	80 Gyration	6.7	17.2	0.96	6.2	16.9[1]	0.95
	1	100 Gyration	6.4	16.6[1]	0.95	6.0	16.5[1]	0.94
Limestone PG 76-22	4	50-Blow	7.4	19.5	0.88	6.9	19.3	0.88
	4	50 Gyration	7.8	20.3	0.90	7.3	20.2	0.90
	4	65 Gyration	7.2	19.1	0.88			
	4	80 Gyration	7.0	18.6	0.87	6.6	18.5	0.86
	4	100 Gyration	6.5	17.6	0.85			
Limestone PG 64-22	4	50-Blow	7.4	19.6	0.89			
	4	50 Gyration	7.6	19.9	0.89	7.2	19.8	0.89
	4	65 Gyration	7.2	19.1	0.88			
Diabase	2	50-Blow	8.0	22.0	0.85	7.5	21.7	0.85
	2	50 Gyration	8.5	23.1	0.87	8.1	23.0	0.86
	2	65 Gyration	8.1	22.1	0.85	7.6	22.1	0.85
	2	80 Gyration	8.2	22.4	0.86	6.4	19.0	0.80
	2	100 Gyration	7.5	21.0	0.83	6.7	20.0	0.81
Ruby Granite	8-B	50-Blow	7.8	20.0	1.01[2]	7.3	19.6	1.00
	8-B	50 Gyration	8.4	21.4	1.03[2]	7.5	20.2	1.01[2]
	8-B	65 Gyration	8.1	20.6	1.02[2]	7.4	19.8	1.00
	8-B	80 Gyration	8.3	21.0	1.02[2]	7.0	19.1	0.99
	8-B	100 Gyration	7.2	18.6	0.98	6.6	18.3	0.97

[1]Fails minimum VMA
[2]Fails VCA_{Ratio}

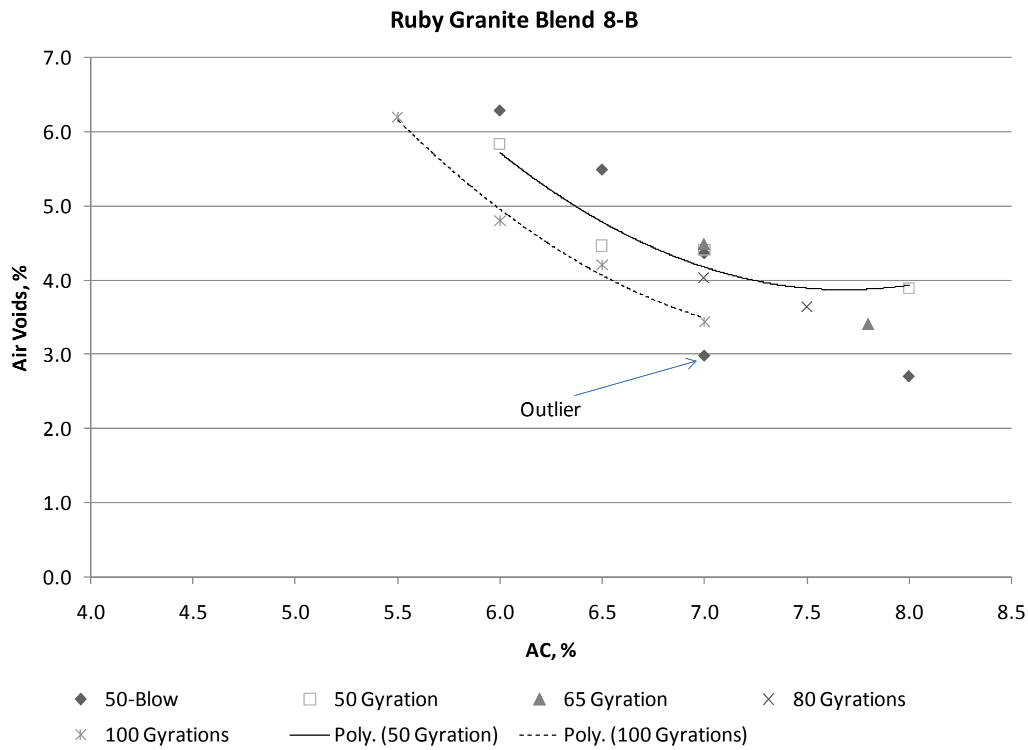

FIGURE 4.1 Air Voids as a Function of Asphalt Content for Ruby Granite.

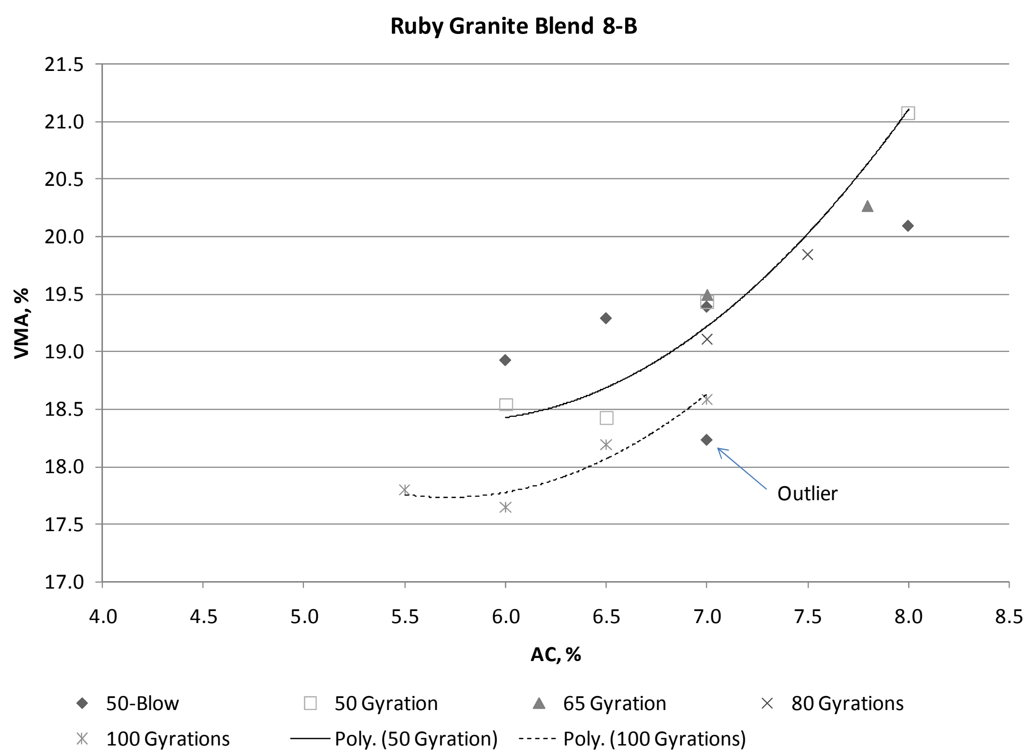

FIGURE 4.2 VMA as a Function of Asphalt Content for Ruby Granite.

4.4.2.2 P401 Control Mixtures

The volumetric properties for the P401 control mixes are summarized in Table 4.10. The complete results are shown in Appendix B. The P401 control mixes were designed at 3.5 percent air voids.

TABLE 4.10 Summary of Volumetric Properties for P401 Mixtures

Aggregate	Binder	AC%	VMA	VFA
Diabase	PG 76-22	5.1	16.0	78
Columbus Granite	PG 76-22	5.3	15.4	77
	PG 64-22	5.3[1]	15.3	76
Ruby Granite	PG 76-22	5.3	14.8[2]	77
Gravel	PG 76-22	5.3	14.7[2]	76
Limestone	PG 76-22	5.4	15.4	77
	PG 64-22	5.5	15.2	77

[1]At 3.7 percent air voids.
[2]Meets minimum VMA at 4 percent air voids.

4.5 RUTTING SUSCEPTIBILITY

In the literature review, it was noted that SMA mixes were developed to resist studded tire wear, which produces a form of rutting. SMA mixes have proven to be resistant to shear flow rutting in the field, even though the optimum asphalt content of SMA mixes is typically 1.0 percent or more higher than dense-graded mixes. Laboratory testing has typically shown SMA mixtures to have comparable performance to dense-graded mixtures (*64*). Therefore, the objective in this study was to demonstrate that SMA mixtures produced comparable performance to dense-graded mixtures even with the higher contact pressures associated with commercial and military aircraft. A modified binder, PG 76-22, was used in the majority of the SMA and P401 control mixes. The use of a modified binder is expected to improve rutting performance compared to an unmodified or "neat" binder.

The rutting susceptibility of the SMA mixtures and P401 control mixtures was assessed in three ways: stability and flow, repeated load permanent deformation, and Hamburg wheel-tracking. Stability and flow tests are the historic method used in the Marshall design procedure to assess rutting potential. The repeated load permanent deformation test was first used by Ahlrich (*34*) to evaluate the influence of aggregate properties on the rutting performance of asphalt mixtures for airfields. A version of this test was recommended as one of the simple performance tests (SPT) for asphalt mixtures (*65*) The Hamburg wheel-tracking tests were conducted wet. Wet Hamburg wheel-tracking tests provide information about both the rutting susceptibility and moisture susceptibility of asphalt mixtures.

4.5.1 Stability and Flow

The average stability and flow results for the SMA and P401 mixtures are shown in Table 4.11. The results for the SMA mixes are shown for the 50-blow Marshall laboratory compaction effort at both 3 and 4 percent design air voids. The P401 specifications note that the flow values may need to be modified for polymer modified binder such as PG 76-22. The average stability is 910 lbs higher with PG 76-22 as compared to PG 64-22 for the Columbus granite and limestone P401 control mixtures. The flow (measured in 0.01 inches) of the control mixes produced with PG 76-22 average 13 compared to 10 for the PG 64-22. The average stability of the diabase and Columbus granite SMA mixtures exceed the minimum requirements for P401 mixtures for aircraft with gross weights in excess of 27,200 kg (60,000 lbs) or tire pressures in excess of 689 kPa (100 psi). All of the SMA mixture's flow values exceed the P401 specifications. The German specifications note that stability and flow are not applicable to SMA mixtures.

TABLE 4.11 Summary of Stability and Flow Values

Aggregate	High PG	SMA			SMA			P401		
		3% Air Voids			4% Air Voids			3.5% Air Voids		
		AC%	Stability, N (lbs)	Flow 0.25 mm (0.01 in)	AC %	Stability, N (lbs)	Flow 0.25 mm (0.01 in)	AC %	Stability, N (lbs)	Flow 0.25 mm (0.01 in)
Diabase	76	8.0	10,231 (2,300)	34	7.5			5.1	21,556 (4,846)	11
Columbus Granite	76	6.5	10,453 (2,350)	28	5.9	12,580 (2,828)	23	5.3	23,086 (5,190)	13
Ruby Granite	76	7.8	8,785 (1,975)	23	7.3	7,998 (1,798)	21	5.3	20,996 (4,720)	11
Gravel	76	8.0	6,819 (1,533)	27	7.6	8,042 (1,808)	29	5.4	16,899 (3,799)	11
Limestone	76	7.5	7,206 (1,620)	20	6.9	6,570 (1,477)	24	5.4	17,526 (3,940)	12
Columbus Granite	64	NA	NA	NA	NA	NA	NA	5.3	18,683 (4,200)	11
Limestone	64	7.4	7,415 (1,667)	18				5.5	13,838 (3,111)	8
Average	76	7.4	8,7001 (1,956)	26	7.0	8,799 (1,978)	24	5.3	20,013 (4,499)	12
Average	64	7.4	7,415 (1,667)	18				5.4	16,263 (3,656)	10

4.5.2 Repeated-Load Deformation

Ahlrich (*34*) first used the confined, repeated-load deformation test to assess the affect of aggregate shape, angularity, and texture on the rutting performance of heavy-duty asphalt mixtures for airfields. Marshall samples, 63.5 mm (2.5 inches) tall and 100 mm (4.0 inches) in diameter, were tested with a 276 kPa (40 psi) confining pressure and 1,379 kPa (200 psi) deviator stress at 60 °C (140 °F). The deviator or repeated load was applied for 0.1 second followed by a 0.9 second rest period. The samples were tested for one hour (3,600 cycles). Good correlations were found between the various test parameters (permanent strain, creep modulus, and creep slope) and measures of coarse aggregate shape, angularity and texture.

The confined, repeated-load deformation test was one of the tests selected for assessing the performance of Superpave mixtures (*65*). Some changes, however, were recommended to the test procedure used by Ahlrich (*34*). Oversize samples were to be compacted in the SGC. The center of the SGC samples was to be cored out and the ends sawed to produce a sample 150 mm (6 inches) tall by 100 mm (4 inches) in diameter. The taller sample is supposed to reduce end effects and produce an approximately uniform stress state over the middle of the sample height. Linear variable differential transformers (LVDTs) are attached to gauge points glued onto the sample with a gauge length of 100 mm (4 inches). The test procedure does not specify confining pressure, deviator stress or test temperature. The performance measure identified for this test procedure was the flow number, or number of cycles at which the sample entered tertiary flow. This will be described in more detail below.

In this study, samples were prepared according to the draft AASHTO test procedure (*66*). The samples were 150 mm (6 inches) in height by 100 mm (4 inches) in diameter, cored and sawed from an oversize SGC sample. The SMA samples were prepared at 5 ± 0.5 percent air voids. As noted previously, SMA must be compacted to a high degree of in-place density to prevent permeability. A sample density of 95 percent of theoretical maximum density is representative of required field in-place densities. The P401 mixtures were prepared at 6 ± 0.5 percent air voids. Using a typical standard deviation of core densities of 1.1 percent, 94 percent of theoretical maximum density should provide 100 percent pay when using the P401 specifications. Gauge points to mount LVDTS were glued to the samples to produce a 100 mm (4-inch) gauge length. Three LVDTs were mounted on each sample. The samples were encased in a latex membrane to provide confinement. A greased latex disk was used on each end of the sample to reduce friction. The samples were tested at 58 °C (136.4 °F) with a 276 kPa (40 psi) confining pressure. Three different deviator stresses were initially used: 689, 1,379, and 2,413 kPa (100, 200, and 350 psi). The deviator stresses are consistent with tire pressures on general aviation, commercial, and military aircraft, respectively.

The data were analyzed for three primary parameters: flow number, secondary creep slope and number of cycles to 2 percent accumulated strain. Two methods were used to determine the flow number. A graphical estimation of the flow number was also determined. The flow number from the Francken model (*67*) is reported herein. This methodology will be implemented in version 3.0 of the Simple Performance Tester (SPT) specifications (*Personal Communication with Ray Bonaquist*).

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The Franken Model is a composite mathematical model which allows primary consolidation, secondary creep, and tertiary flow to be modeled (*67*). The Franken Model is represented by the following equation:

$$\varepsilon_p(N) = AN^B + C(e^{DN} - 1) \tag{4}$$

where:
$\varepsilon_p(N)$ = permanent deformation or permanent strain,
N = number of loading cycles, and
A, B, C, and D = regression constants.

The regression constants were determined by a non-linear regression, least-squares procedure using Microsoft Excel Solver. The Francken Model is differentiated once with respect to N to determine the strain slope. The model is differentiated a second time to determine the gradient of the strain slope. The flow number is the point where the gradient of the strain slope changes from a negative to a positive value. The regression constant "B" represents the secondary creep slope on a log scale. The B values were used by Xie (*33*) to evaluate performance. An example of a typical repeated-load deformation test result is shown in Figure 4.3. The secondary creep slope and flow number are shown in the figure.

The higher contact pressures associated with commercial and military aircraft are expected to cause a higher rutting rate compared to lower contact pressures associated with general aviation aircraft or highway trucks. However, the number of expected coverages in a given year for a busy airfield is most likely measured in the tens-of-thousands compared to millions for a heavily travelled highway pavement. Only a small portion of these repetitions are likely occur in the warmest weather (in most climates) when damage is most likely to occur. Therefore when tested at higher contact pressures, the rutting rate can be higher or number of loading cycles until tertiary flow or a critical level of strain occurs may be lower for airfield pavements as compared to highway pavement mixtures tested at a lower contact pressure.

The repeated-load deformation data is shown in Table 4.12. As indicated in Table 4.4 and 4.12, no repeated-load testing was conducted on the diabase or Ruby granite. Overall, the repeated-load deformation results were more variable than expected. Prior experience suggested that triaxial confinement of gap-graded mixes, like SMA, was more important than with dense-graded mixtures, like P401. The confinement pressure is designed to act like the surrounding asphalt mixture in a pavement. For SMA mixtures, the stone skeleton is designed to carry the load. Particularly with the taller sample height, if the coarse aggregate particles in the center of the specimen are allowed to dilate in the radial direction, or expand out, the sample may fail. In an actual pavement the coarse aggregate particles in the surrounding asphalt pavement act to prevent this dilation and to spread the load. When conducting the testing, it was observed that if the latex membrane used to apply the confining

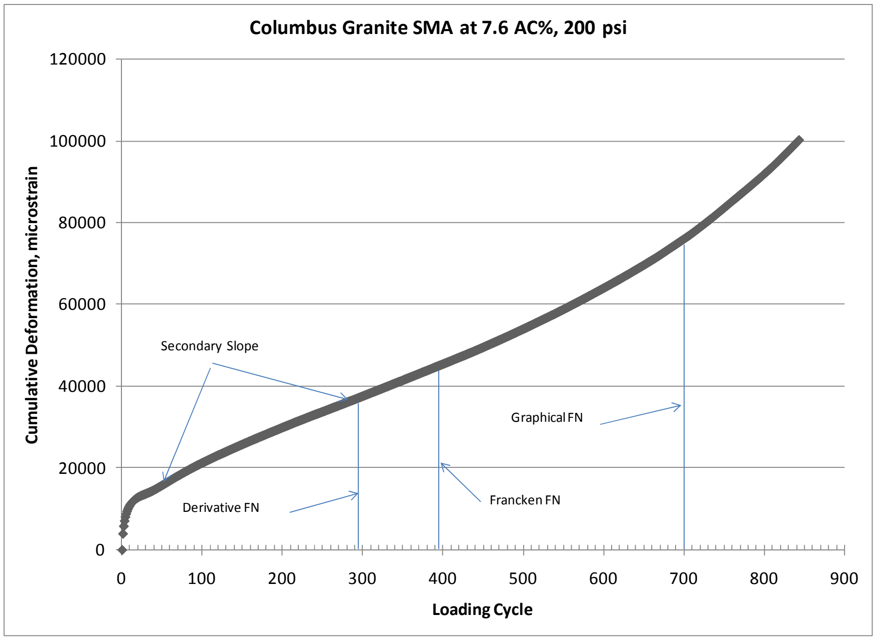

FIGURE 4.3 Typical Output from Repeated Load Permanent Deformation Test.

pressure did not seal tightly around the samples when the confining pressure was applied, the sample failed quickly.

4.5.2.1 P401 Repeated Load Analyses

Analyses were conducted to see which response, e.g. flow number, secondary slope, or cycles to 2 percent permanent strain, was best explained by the factors used in the experiment. The PG 76-22 P401 data at 689 and 1,379 kPa (100 and 200 psi) deviator stress were examined first since this represented the most complete data set (16 of 18 samples). Aggregate source, deviator stress, and the interaction between aggregate source and deviator stress were used as factors. Francken flow number, secondary creep slope, Francken B coefficient (slope on a log basis), and number of cycles to 2 percent permanent strain were used as responses (individually, one at a time). ANOVA was performed using the general linear model (GLM) performed with Minitab statistical software. GLM is a regression-based ANOVA technique which handles incomplete data sets. Since it is regression-based, an R^2 value is determined. The *p-values* for the factors and R^2 values are summarized in Table 4.13. *P-values* less than 0.05 indicate the factors are significant. The R^2 values indicate how well the selected factors describe the response data with higher R^2 values indicating a better model.

The flow number, determined by the Francken model, for all of the P401 mixes tested at 689 kPa (100 psi) deviator stress was greater than 20,000 cycles. For analysis purposes, the Francken flow number was reported as 20,000 cycles. Other samples apparently failed very rapidly at a certain point and a Francken flow number was not indicated. For analysis purposes, the maximum number of cycles tested was reported as the flow number if one was not otherwise identified.

TABLE 4.12 Repeated Load Deformation Test Results												
Aggregate	Mix	Binder	AC, %	Gyrations	Deviator Stress, psi	Secondary Slope, ms/cycle	Franken B Coefficient	Francken Flow Number	Total Strain, (mm/mm)	Cycles for Total Strain	Cycles for 2% Total Strain	Strain @ 10,000 cycles
Col. Granite	P401	PG 76-22	5.3	75 Blows	100	0.77	1.07	> 20,000	0.0169	20,001	28,491	0.0017
Col. Granite	P401	PG 76-22	5.3	75 Blows	100	0.02	0.25	> 20,000	0.0018	20,002	6.45E+07	0.0158
Col. Granite	P401	PG 76-22	5.3	75 Blows	100	0.02	0.23	> 20,000	0.0020	20,002	1.14E+08	0.0019
Col. Granite	P401	PG 76-22	5.3	75 Blows	200	259.62	1.05	0	0.0170	1,251	1,315	
Col. Granite	P401	PG 76-22	5.3	75 Blows	200	38.17	0.72	> 5,001	0.0159	5,001	6,885	
Col. Granite	P401	PG 76-22	5.3	75 Blows	200	1212.16	0.81	> 1,001	0.0134	1,001	1,637	
Col. Granite	P401	PG 76-22	5.3	75 Blows	350	1611.75	1.09	> 9	0.0014	9	325	
Col. Granite	P401	PG 76-22	5.3	75 Blows	350	2139.17	1.21	> 13	0.0023	13	886	
Col. Granite	SMA	PG 76-22	6.8	50 Blows	100	0.43	0.19	> 17,003	0.0102	17,003	64,826	0.0126
Col. Granite	SMA	PG 76-22	6.8	50 Blows	100	0.09	0.28	> 20,000	0.0070	20,002	152,535	0.0064
Col. Granite	SMA	PG 76-22	6.8	50 Blows	200	1838.36	0.52	> 177	0.0060	177	526	
Col. Granite	SMA	PG 76-22	6.8	50 Blows	200	440.85	0.47	> 601	0.0063	601	2,855	
Col. Granite	SMA	PG 76-22	6.8	50 Blows	200	830.43	0.79	> 202	0.0158	202	261	
Col. Granite	SMA	PG 76-22	6.8	50 Blows	350	5198.30	0.69	> 17	0.0073	17	77	
Col. Granite	SMA	PG 76-22	6.8	50 Blows	350	4749.07	0.80	> 15	0.0054	15	67	
Col. Granite	SMA	PG 76-22	6.8	50 Blows	350	3226.32	0.66	> 11	0.0036	11	249	
Col. Granite	SMA	PG 76-22	6.8	100	100	0.02	0.19	> 20,000	0.0011	20,002	1.52E+07	0.0010
Col. Granite	SMA	PG 76-22	6.8	100	100	0.03	0.22	> 20,000	0.0016	20,002	7.60E+07	0.0013
Col. Granite	SMA	PG 76-22	6.8	100	200	103.79	0.48	> 2,251	0.0130	2,251	8,735	
Col. Granite	SMA	PG 76-22	6.8	100	200	123.10	0.74	251	0.0196	1,501	1,574	
Col. Granite	SMA	PG 76-22	6.8	100	200	130.59	0.87	351	0.0095	1,252	2,605	
Col. Granite	SMA	PG 76-22	6.8	100	350	2543.60	0.62	> 13	0.0038	13	133	
Col. Granite	SMA	PG 76-22	6.8	100	350	2726.53	0.61	> 20	0.0055	20	143	
Col. Granite	SMA	PG 76-22	7.2	80	100	0.05	0.10	> 20,000	0.0030	20,002	1.36E+07	0.0026

TABLE 4.12 Repeated Load Deformation Test Results												
Aggregate	Mix	Binder	AC, %	Gyrations	Deviator Stress, psi	Secondary Slope, ms/cycle	Franken B Coefficient	Francken Flow Number	Total Strain, (mm/mm)	Cycles for Total Strain	Cycles for 2% Total Strain	Strain @ 10,000 cycles
Col. Granite	SMA	PG 76-22	7.2	80	100	0.04	0.21	> 20,000	0.0022	20,002	3.93E+06	0.0025
Col. Granite	SMA	PG 76-22	7.2	80	200	36.98	0.41	502	0.0094	1,502	32,103	
Col. Granite	SMA	PG 76-22	7.2	80	200	15.37	0.39	> 5,001	0.0124	5,001	10,688	
Col. Granite	SMA	PG 76-22	7.2	80	350	6476.00	0.90	> 37	0.0145	37	52	
Col. Granite	SMA	PG 76-22	7.2	80	350	3725.10	0.98	> 18	0.0038	18	94	
Col. Granite	SMA	PG 76-22	7.6	50	100	74.3	0.31	324	0.1002	918	98	
Col. Granite	SMA	PG 76-22	7.6	50	100	49.1	0.35	372	0.1000	1210	119	
Col. Granite	SMA	PG 76-22	7.6	50	100	42.7	0.43	499	0.1000	1420	151	
Col. Granite	SMA	PG 76-22	7.6	50	200	49.1	(negative)	1,285	0.1000	1285	115	
Col. Granite	SMA	PG 76-22	7.6	50	200	80.1	0.82	843	0.1000	843	89	
Col. Granite	SMA	PG 76-22	7.6	50	200	52.9	0.44	394	0.1002	1106	127	
Col. Granite	SMA	PG 76-22	7.6	50	350	5853.20	0.84	60	0.0198	60	51	
Col. Granite	SMA	PG 76-22	7.6	50	350	4021.54	0.77	71	0.0156	71	74	
Col. Granite	SMA	PG 76-22	7.6	50	350	12,844.10	1.06	> 20	0.0130	20	31	
Gravel	P401	PG 76-22	5.4	75 Blows	100	0.19	0.40	> 20,000	0.0065	20,002	237,984	0.0054
Gravel	P401	PG 76-22	5.4	75 Blows	100	0.14	0.34	> 20,000	0.0045	20,002	1.40E+06	0.0036
Gravel	P401	PG 76-22	5.4	75 Blows	200	1456.62	0.90	> 302	0.0098	302	786	
Gravel	P401	PG 76-22	5.4	75 Blows	200	304.54	1.33	5	0.0106	352	423	
Gravel	P401	PG 76-22	5.4	75 Blows	200	657.59	0.70	> 901	0.0136	901	9,279	
Gravel	SMA	PG 76-22	6.4	100	100	0.03	0.39	> 20,000	0.0026	20,002	4.84E+07	0.0023
Gravel	SMA	PG 76-22	6.4	100	100	83.75	0.90	> 2,751	0.0144	2,751	2,674	
Gravel	SMA	PG 76-22	6.4	100	200	92.40	0.41	> 1,252	0.0133	1,252	2,029	
Gravel	SMA	PG 76-22	6.4	100	200	31.99	0.22	> 5,001	0.0133	5,001	12,226	
Gravel	SMA	PG 76-22	6.4	100	200	13.59	0.01	> 5,001	0.0055	5,001	8,114	

TABLE 4.12 Repeated Load Deformation Test Results												
Aggregate	Mix	Binder	AC, %	Gyrations	Deviator Stress, psi	Secondary Slope, ms/cycle	Franken B Coefficient	Francken Flow Number	Total Strain, (mm/mm)	Cycles for Total Strain	Cycles for 2% Total Strain	Strain @ 10,000 cycles
Gravel	SMA	PG 76-22	6.8	80	100	0.48	0.87	> 20,000	0.0118	20,002	30,582	0.0068
Gravel	SMA	PG 76-22	6.8	80	100	128.38	0.45	> 8,086	0.0109	8,086	26,623	
Gravel	SMA	PG 76-22	6.8	80	100	67.56	1.24	152	0.0145	2,252	2,985	
Gravel	SMA	PG 76-22	7.2	50	100	0.26	0.87	> 20,000	0.0069	20,002	174,363	0.0048
Gravel	SMA	PG 76-22	7.2	50	100	0.21	0.51	> 20,000	0.0093	14,503	52,369	0.0093
Gravel	SMA	PG 76-22	7.2	50	100	0.11	0.24	> 20,000	0.0067	20,002	712,313	0.0061
Gravel	SMA	PG 76-22	7.2	50	200	1398.06	0.76	> 251	0.0147	251	364	
Limestone	P401	PG 76-22	5.4	75 Blows	100	0.20	0.91	> 20,000	0.0043	20,002	216,627	0.0025
Limestone	P401	PG 76-22	5.4	75 Blows	100	0.13	0.61	> 20,000	0.0035	20,002	645,428	0.0025
Limestone	P401	PG 76-22	5.4	75 Blows	200	0.55	0.45	> 20,000	0.0145	20,001	36,794	0.0102
Limestone	P401	PG 76-22	5.4	75 Blows	200	162.00	0.58	> 552	0.0087	552	1,655	
Limestone	P401	PG 76-22	5.4	75 Blows	200	36.86	0.56	> 7,502	0.0179	7,502	8,640	
Limestone	SMA	PG 76-22	6.5	100	100	0.16	0.39	> 20,000	0.0059	20,002	1.04E+06	0.0050
Limestone	SMA	PG 76-22	6.5	100	100	0.04	0.17	> 20,000	0.0027	20,002	1.05E+07	0.0024
Limestone	SMA	PG 76-22	6.5	100	200	36.79	0.11	4,501	0.0099	4,501	24,601	
Limestone	SMA	PG 76-22	6.5	100	200	499.70	0.39	652	0.0059	652	8,643	
Limestone	SMA	PG 76-22	6.5	100	200	1220.86	0.88	> 351	0.0141	351	559	
Limestone	SMA	PG 76-22	7.2	65	100	0.03	0.39	> 20,000	0.0026	20,002	1.00E+09	0.0023
Limestone	SMA	PG 76-22	7.2	65	100	0.05	0.17	> 20,000	0.0038	20,002	1.62E+07	0.0034
Limestone	SMA	PG 76-22	7.2	65	100	0.04	0.18	> 20,000	0.0026	20,002	1.93E+08	0.0023
Limestone	SMA	PG 76-22	7.8	50	100	3.66	0.40	1,502	0.0036	4,503	207,079	
Limestone	SMA	PG 76-22	7.8	50	100	0.13	0.19	> 20,000	0.0082	20,002	434,478	0.0074
Limestone	SMA	PG 76-22	7.8	50	100	0.25	0.30	> 20,000	0.0056	20,002	250,285	0.0037
Limestone	SMA	PG 76-22	7.8	50	200	81.54	0.46	451	0.0118	1,001	3,786	

TABLE 4.12 Repeated Load Deformation Test Results												
Aggregate	Mix	Binder	AC, %	Gyrations	Deviator Stress, psi	Secondary Slope, ms/cycle	Franken B Coefficient	Francken Flow Number	Total Strain, (mm/mm)	Cycles for Total Strain	Cycles for 2% Total Strain	Strain @ 10,000 cycles
Limestone	SMA	PG 76-22	7.8	50	200	233.08	0.41	301	0.0154	552	735	
Limestone	SMA	PG 76-22	7.8	50	200	1968.33	0.58	> 97	0.0120	97	297	
Limestone	P401	PG 64-22	5.5	75 Blows	100	0.05	0.19	> 20,000	0.0024	20,002	6.65E+08	0.0022
Limestone	P401	PG 64-22	5.5	75 Blows	100	0.13	0.32	> 20,000	0.0045	20,001	2.42E+06	0.0035
Limestone	P401	PG 64-22	5.5	75 Blows	100	0.06	0.42	> 20,000	0.0027	20,002	5.09E+07	0.0023
Limestone	SMA	PG 64-22	7.2	65	100	0.20	1.34	16,000	0.0150	20,002	21,392	0.0034
Limestone	SMA	PG 64-22	7.2	65	100	0.36	0.38	> 20,000	0.0208	20,002	9,502	0.0201
Limestone	SMA	PG 64-22	7.2	65	100	0.10	0.08	> 20,000	0.0050	20,002	3.15E+06	0.0025
Limestone	SMA	PG 64-22	7.2	65	200	6326.30	0.58	151	0.0112	151	301	
Limestone	SMA	PG 64-22	7.2	65	200	4456.20	0.66	58	0.0416	58	17	
Limestone	SMA	PG 64-22	7.2	65	200	8260.27	1.21	0	0.0131	78	128	

Note: 1 psi = 6.89476 kPa

TABLE 4.13 ANOVA *p-values* for P401 Mixes

Factor	DF[1]	Francken FN	Secondary Slope	B Coefficient	Cycles for 2% Strain
Aggregate Source	2	0.306	0.365	0.968	0.136
Deviator Stress	1	0.000	0.042	0.123	0.151
Aggregate*Deviator Stress	2	0.306	0.365	0.100	0.136
Error	10				
Total	15				
R^2		0.85	0.53	0.48	0.57

[1]DF = degrees of freedom

Deviator stress is one of the factors expected to have a large influence on the repeated load permanent deformation results. Deviator stress is significant based on both flow number and the secondary creep slope responses. Aggregate source was not significant for any of the responses. AC 150/5370-10B specifies that P401 mixes designed for aircraft with gross weights in excess of 27,216 kg (60,000 lbs) use coarse aggregate with greater than 70 percent two fractured faces and 85 percent one fractured face. The gravel source meets these requirements with 77 and 97 percent two and one-fractured faces, respectively. The interaction between aggregate source and deviator stress is not significant at the 5 percent level for any of the responses. The analyses indicate that this level of fractured faces is not detrimental to the gravel mixes' performance compared to the other aggregate types used in this study. This confirms the fracture face requirements currently included in the P401 specifications. The R^2 value for the Francken flow number response was moderate at 0.85. Thus for the P401 mixes, flow number appeared to be the response which best explained the variation from the experimental factors.

4.5.2.2 Gyratory Design SMA Mixes Repeated Load Analyses

A similar series of analyses were performed for the 50, 80, and 100 gyration SMA mixes. There was insufficient data to analyze the 65 gyration or 50-blow Marshall SMA mixes. Aggregate source, design gyrations, deviator stress (689 and 1,378 kPa [100 and 200 psi]), and the interaction between gyrations and deviator stress were analyzed. The interaction between design gyrations and aggregate source would be of interest but there were insufficient data points to perform this analysis. Realistically, design gyrations are unlikely to be altered for differing aggregate sources. The same responses as described previously for the P401 mixes were used for the SMA mixes. The *p-values* and the GLM R^2 values for the different responses are shown in Table 4.14.

From Table 4.14, it is evident that flow number appears to be the best response to use to analyze the mixtures performance. Deviator stress is significant at the 5 percent level based on two of the responses: Francken FN and Francken B Coefficient. Aggregate is again insignificant at the 5 percent level for all of the responses; however, it is significant at the 10 percent level for the Franken FN.

TABLE 4.14 ANOVA *p-values* for Gyratory SMA Mixtures

Factor	DF[1]	Francken FN	Secondary Slope	B Coefficient	Cycles for 2% Strain
Aggregate Source	2	0.083	0.218	0.276	0.256
Design Gyrations	2	0.067	0.519	0.511	0.004
Deviator Stress	1	0.000	0.093	0.082	0.016
Aggregate*Deviator Stress	2	0.260	0.511	0.723	0.006
Error	27				
Total	34				
R^2		0.74	0.31	0.26	0.54

[1]DF = degrees of freedom

The number of design gyrations is significant at the 5 percent level for the number of cycles to 2 percent permanent strain and at the 10 percent level for Francken flow number. Generally, higher gyrations provided better resistance to permanent deformation. Although the fitted mean for Franken flow number suggests that the 80 gyration mixes had slightly better performance (Figure 4.4). It should be emphasized that the mixes were designed at 3 percent air voids. The asphalt content determined at 100 gyrations at 3 percent air voids would be selected at approximately 72, 71, or 85 gyrations respectively, for the Columbus granite, gravel, or limestone mixtures at 4 percent air voids.

The interaction between aggregate and deviator stress is significant at the 5 percent level for the number of cycles to 2 percent permanent strain. The model fit for this interaction is nonsensical, since negative cycles are predicted for the 80 gyration mixes. The test data produced an average of 21,396 cycles to 2 percent permanent strain for the 80 gyration Columbus granite samples.

Figure 4.4 shows the main effects plot for the Francken flow number. The plot helps to visualize the noted effects. In some cases, a trend can be seen in the data which may not be significant when testing variability is considered.

Both measures of flow number indicate that the gravel provides the best resistance to rutting. This may be due to its high flat and elongated content. Shape, as well as texture, can contribute to an aggregate's overall angularity. However, too high of a percentage of flat and elongated particles may cause difficulties with field compaction. The Columbus granite, which is the most cubical, has the lowest flow numbers. The Columbus granite also has the highest LA Abrasion loss. Contact points may be degrading under load.

It should be noted that the improved performance of the 80 gyration mixes at 1,379 kPa (200 psi) in Figure 4.4 is driven solely by the performance of the Columbus granite samples. No limestone samples were tested at 80 gyrations and 1,379 kPa (200 psi). Thus 100 design gyrations at 3 percent air voids appear to produce the SMA mixture with the best resistance to permanent deformation.

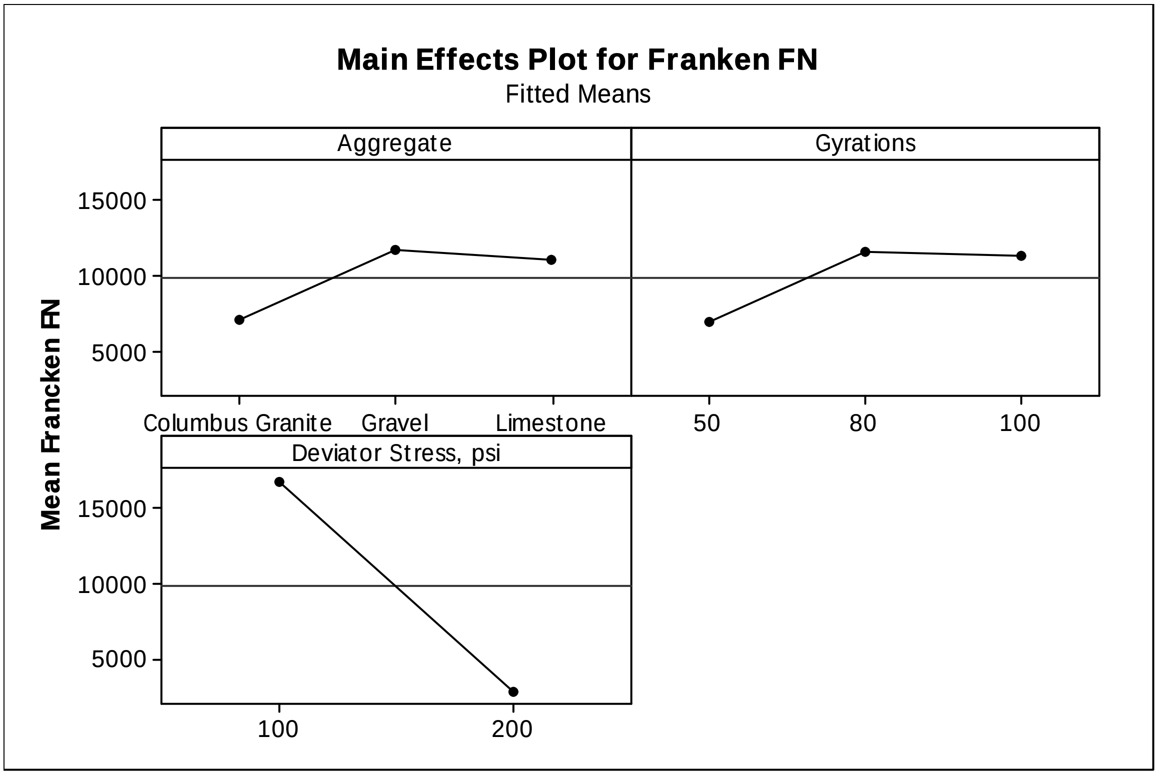

FIGURE 4.4 Main Effects Plots for Gyratory SMA Flow Number.

The data indicate that the best performance of the SMA mixtures, in terms of permanent deformation resistance, resulted from the 80 and 100 gyration mixes. The data also indicate that SMA mixtures designed at 3 percent air voids using 100 gyrations are approximately equivalent to SMA mixtures designed at 4 percent air voids using 71 to 85 gyrations. Therefore, the next set of analyses compared the permanent deformation performance of the SMA mixtures designed at 3 percent air voids using 100 gyrations with the P401 mixes.

4.5.2.3 SMA and P401 Repeated Load Comparison Data

Based on the previous analyses, the Francken flow numbers were selected as the permanent deformation response. A simple evaluation of the data can be made by looking at the Francken flow number results. At 100 psi deviator stress, all of the P401 samples from all three aggregate sources tested made it to 20,000 cycles without experiencing tertiary flow. By comparison for the 100 gyration SMA mixtures at 689 kPa (100 psi), only one of two gravel samples experienced tertiary flow. This sample failed very early, most likely due to a membrane failure.

Next, a series of ANOVAs were conducted using the GLM. Aggregate source, mix type (P401 and100 gyration SMA), deviator stress and the interaction between aggregate and mix were selected as factors. The ANOVA results are shown in Table 4.15 and the main effects plot is shown in Figure 4.5. The only significant factor was deviator stress. The effect of mix, either SMA or P401, was clearly not significant. This indicates that SMA and P401 mixes should provide equal rutting performance, even though the asphalt content of the SMA mixtures is much higher, thereby providing better durability.

TABLE 4.15 ANOVA (GLM) Results for 100 Gyration SMA and P401 Comparison

Source	DF	Francken flow number		
		Adjusted mean squares	F-statistic	*p-value*
Aggregate	2	17,234,619	1.34	.282
Mix	1	7,880,111	0.61	0.442
Deviator Stress	1	2,047,250,357	158.88	0.000
Aggregate*Mix	2	26,194,253	2.03	0.154
Error	22	12,885,318		
Total	28			
R^2				0.88

Figure 4.6 shows the interaction plot between mixture and aggregate source for the Francken flow number. The performance of the SMA mixtures appears to be less reliant on the aggregate type as compared to the P401 mixes. The mean flow number for the limestone P401 mixture is larger than that of the limestone SMA mixture. This may be due to breakdown of the contact points of the limestone aggregate during loading, even though the LA Abrasion loss for the limestone aggregate is relatively low. The mean flow number for the Gravel SMA mixture is larger than that of the gravel P401 mixture. It should be emphasized that the differences described are not statistically significant, except for deviator stress.

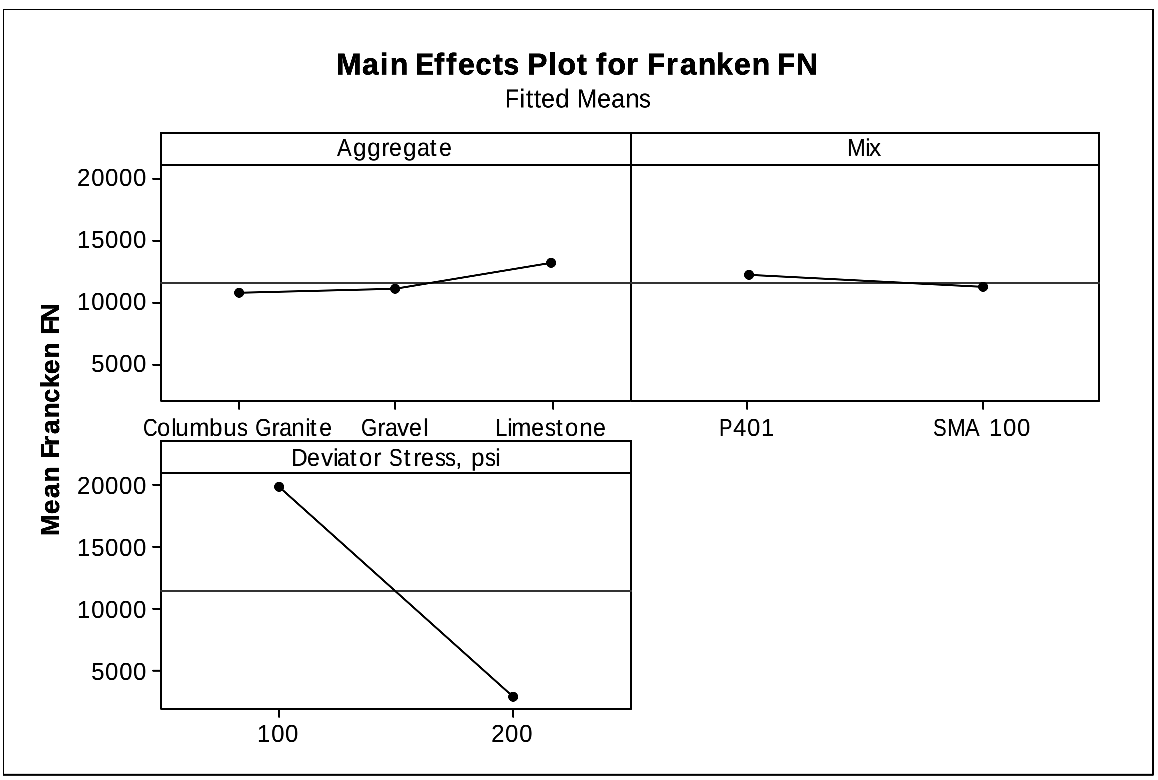

FIGURE 4.5 Main Effects Plot for Francken Flow Number.

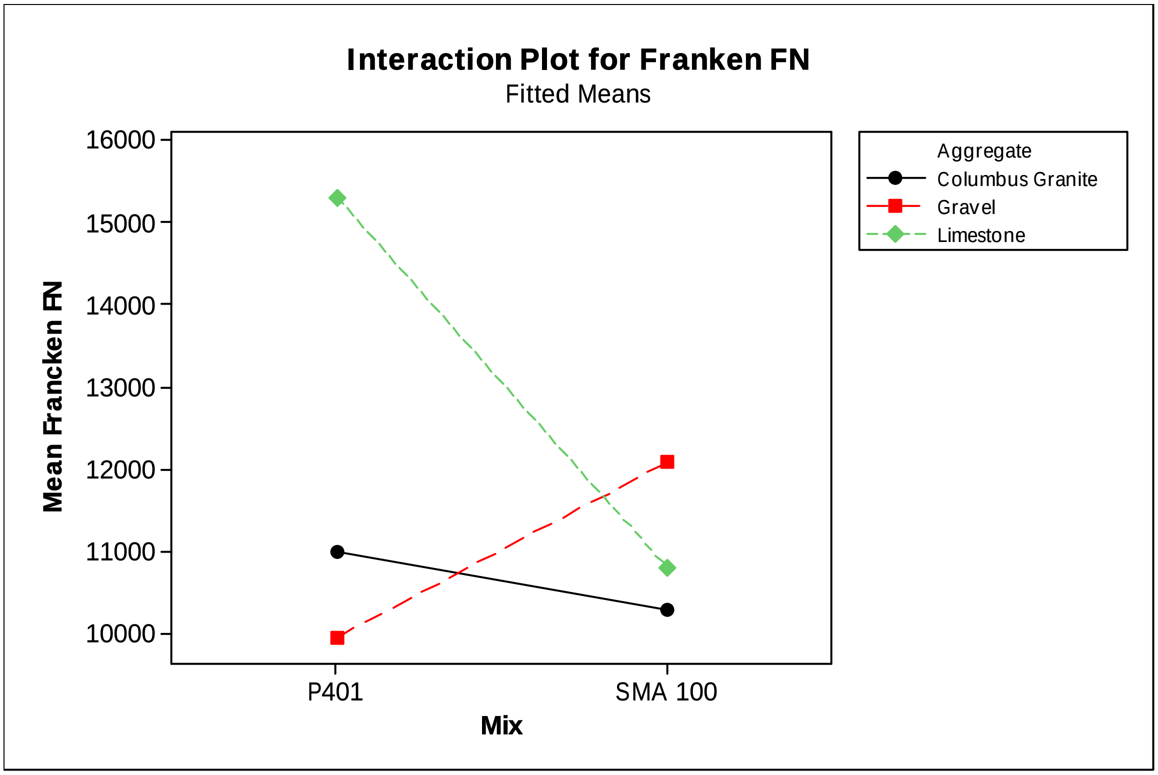

FIGURE 4.6 Interaction Plot for Aggregate and Mix Type.

4.5.2.4 PG 64-22 versus PG 76-22 Repeated Load Analyses

The final analyses compared the limestone SMA and P401 mixtures produced with PG 64-22 and PG 76-22. Examination of the Francken flow numbers indicates that only one sample, a 65 gyration SMA mix sample with PG 64-22, tested at 689 kPa (100 psi) deviator stress failed to achieved 20,000 cycles without incurring tertiary flow. The limestone P401 mix was not tested at 1,379 kPa (200 psi) deviator stress, so comparisons could not be made using the 1,379 kPa (200 psi) deviator stress data. All of the ANOVAs conducted using the data for samples tested with 689 kPa (100 psi) deviator stress indicated that the factors analyzed (mix, binder grade, and their interaction) were insignificant. Although many studies have shown the benefits of polymer modification in terms of rutting resistance, this data indicates that good performing SMA and P401 mixes can be designed for general aviation airfields with neat binders.

4.5.2.5 Summary of Repeated Load Data

The analyses of the repeated load permanent deformation test data indicate the following:

- Francken flow number was the response from the repeated load test which was most sensitive to experimental factors such as deviator stress.
- Deviator stress was altered between 689 and 2,413 kPa (100 and 350 psi) to simulate different aircraft tire pressures. Increased tire pressure, as evidenced by deviator stress, has a significant effect on permanent deformation. The average Francken flow

numbers are summarized by aggregate type as a function of deviator stress in Figure 4.7.

- Repeated load tests were performed on samples from three aggregate sources: Columbus granite, gravel and limestone. Aggregate source was not a significant factor for either the P401 or SMA mixes. This indicates that good performing mixes can be designed for airfield pavements using gravel aggregate sources with as low as 77 percent two crushed faces. The high flat and elongated particle content may have contributed to the gravel mixture's performance.
- Design gyrations were somewhat significant in the rutting performance of the SMA mixtures based on the Francken FN and number of cycles to 2 percent permanent strain. Higher gyrations provided better rutting performance. It should be noted that the optimum asphalt content selected using 100 gyrations at 3 percent air voids is approximately equivalent to the asphalt content which would be selected between 71 and 85 gyrations using 4 percent design air voids.
- The permanent deformation performance of SMA mixtures designed at 3 percent air voids using 100 design gyrations and P401 mixtures were not significantly different, nor did there appear to be any practical difference in the results based on observation of the main effects plots (Figure 4.6).
- At 689 kPa (100 psi) deviator stress, there was no significant difference in the rutting performance of the limestone P401 and SMA mixes produced with either PG 64-22 or PG 76-22. This suggests that modified binders are not required to produce mixes with good rutting performance for general aviation fields serving aircraft with tire pressures less than 689 kPa (100 psi).

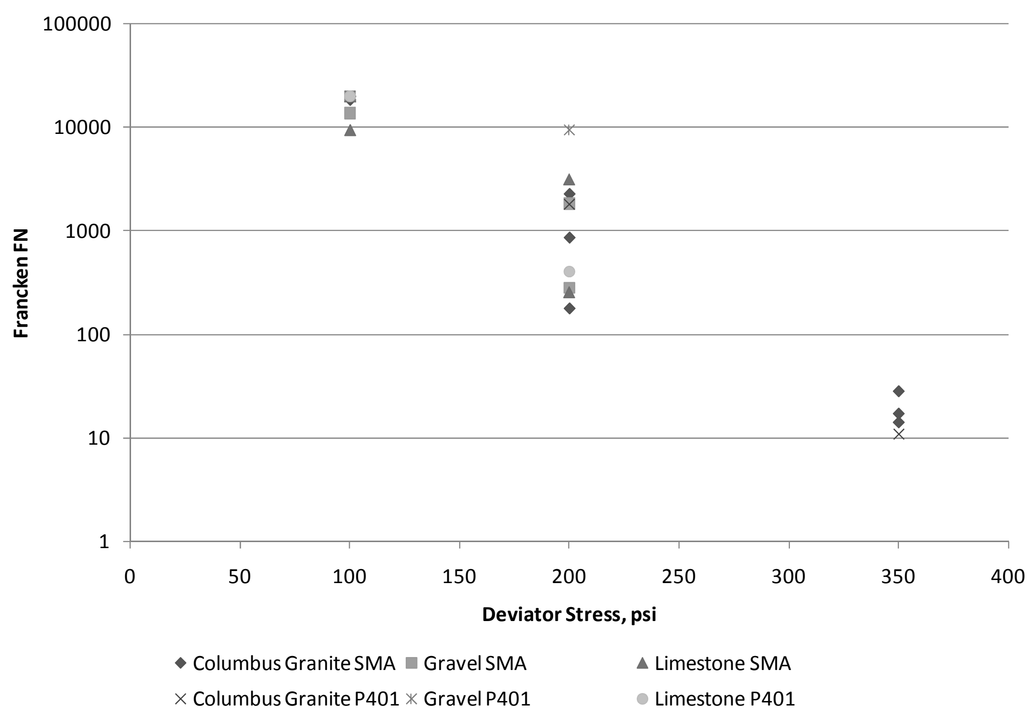

FIGURE 4.7 Francken Flow Number as a Function of Deviator Stress (Tire Pressure).

4.5.3 Hamburg Wheel-Tracking Device

The Hamburg wheel-tracking device (HWTD) was developed in the 1980's to assess both the rutting and moisture damage potential of asphalt mixtures. HWTD test results of field mixed, field compacted samples produced a correlation with an R^2 = 82 percent to the field performance of WesTrack test sections (*68*).

In this study, samples were tested for 20,000 passes (10,000 cycles) at a temperature of 50 °C (122 °F). The SMA test samples were produced at 5 ± 0.5 percent air voids and the P401 samples at 6 ± 0.5 percent air voids. Samples were not tested for every laboratory compaction level due to the fact that some of the optimum asphalt contents were very close together. The HWTD data is summarized in Table 4.16. Primarily three results were analyzed: the existence of a stripping inflection point, the secondary creep slope, and the total rutting after 10,000 cycles (20,000 passes).

The stripping inflection point is similar to the flow number described previously. It may occur due to the onset of moisture damage (stripping) or tertiary flow. For the PG 76-22 mixtures, stripping inflection points were observed for five samples, two SMA and three P401 control mixes. Granite sources can be susceptible to moisture damage; one P401 and one SMA sample from the Columbus granite exhibited a stripping inflection point. One of two gravel samples at the optimum asphalt content for the 50-blow Marshall compaction effort at 3.0 percent air voids indicated a stripping inflection point at a high number of cycles (8,300). This may be due to shear flow rutting. The limestone P401 mixes produced with both PG 64-22 and PG 76-22 binder and the limestone SMA produced with PG 64-22 binder exhibited stripping inflection points. Problems with moisture damage tend to be less common with limestone sources, but previous studies with this source indicated poor performance in the HWTD (*68*).

Figure 4.8 shows the average rutting rates as a function of asphalt content. The HWTD rutting rate is similar to the secondary creep slope for the repeated load test described previously. A lower rate indicates better performance. Figure 4.8 shows that the SMA mixes generally have similar rutting rates across a range of asphalt contents. The one exception is the Columbus granite P401 mix, which has a higher rutting rate, most likely due to moisture damage. The thicker asphalt film of the SMA mixes should improve moisture resistance. It is interesting to note that the rutting rate increases at the extremes of the SMA asphalt contents. The low asphalt contents represent a 100 gyration lab compaction effort at 3 percent design voids or approximately an 80 gyration lab compaction effort at 4 percent design voids. The higher asphalt contents generally represent the 50 gyration lab compaction effort at 3 percent air voids. Analysis of variance (ANOVA) was performed on the rutting rates using mix type and aggregate source as factors. Separate comparisons were made between the 50-blow Marshall asphalt content and gyratory asphalt contents (excluding 100 gyrations) and P401 performance. Mix type was not significant in either case (P-value = 0.947 and 0.223, respectively). Aggregate type was significant in both cases.

TABLE 4.16 Hamburg Wheel-Tracking Device Results

Aggregate	Mix Type	PG Grade	AC, %	Stripping Inflection Point, cycles	Avg. Stripping Inflection Point, cycles	Rutting Rate, mm/hr	Avg. Rutting Rate, mm/hr	Total Rutting @ 10,000 cycles, mm	Avg. Total Rutting, mm
Diabase	SMA	76-22	8.1	> 10,000	> 10,000	0.23	0.82	7.16	8.11
Diabase	SMA	76-22	8.1	> 10,000		1.41		9.06	
Diabase	SMA	76-22	8.5	> 10,000	> 10,000	0.83	1.00	9.57	10.14
Diabase	SMA	76-22	8.5	> 10,000		1.17		10.70	
C. Granite	SMA	76-22	7.2	> 10,000	> 10,000	0.57	0.62	6.37	7.18
C. Granite	SMA	76-22	7.2	> 10,000		0.67		7.98	
C. Granite	SMA	76-22	7.6	> 10,000	> 10,000	1.00	0.80	5.85	5.94
C. Granite	SMA	76-22	7.6	> 10,000		0.60		6.03	
C. Granite	SMA	76-22	6.8	> 10,000	8,200	0.94	1.91	18.90	14.27
C. Granite	SMA	76-22	6.8	6,400		2.880		9.63	
Gravel	SMA	76-22	6.4	> 10,000	> 10,000	0.39	0.98	7.57	8.87
Gravel	SMA	76-22	6.4	> 10,000		1.56		10.16	
Gravel	SMA	76-22	6.8	> 10,000	> 10,000	0.50	0.54	5.39	7.71
Gravel	SMA	76-22	6.8	> 10,000		0.58		10.03	
Gravel	SMA	76-22	7.2	> 10,000	> 10,000	0.72	0.69	8.53	7.25
Gravel	SMA	76-22	7.2	> 10,000		0.66		5.96	
Gravel	SMA	76-22	8.0	> 10,000	9,150	1.90	1.42	11.48	10.10
Gravel	SMA	76-22	8.0	8,300		0.94		8.71	
Limestone	SMA	76-22	6.5	> 10,000	> 10,000	1.92	2.45	10.68	14.99
Limestone	SMA	76-22	6.5	> 10,000		2.99		19.30	
Limestone	SMA	76-22	7.2	> 10,000	> 10,000	0.86	0.93	8.99	9.81
Limestone	SMA	76-22	7.2	> 10,000		0.99		10.62	
Limestone	SMA	76-22	7.8	> 10,000	> 10,000	0.93	1.17	9.19	8.86
Limestone	SMA	76-22	7.8	> 10,000		1.40		8.52	
Limestone	SMA	64-22	7.6	3,300	3,050	10.93	12.21	198.35	349.06
Limestone	SMA	64-22	7.6	2,800		13.48		499.76	
Diabase	P401	76-22	5.1	> 10,000	> 10,000	0.26	0.33	4.21	4.47
Diabase	P401	76-22	5.1	> 10,000		0.40		4.73	
C. Granite	P401	76-22	5.3	5000	7500*	3.79	3.40	34.34	24.86
C. Granite	P401	76-22	5.3	> 10,000		3.01		15.37	
Ruby Granite	P401	76-22	5.3	> 10,000	> 10,000	0.59	0.59	5.25	5.42
Ruby Granite	P401	76-22	5.3	> 10,000		0.59		5.58	
Gravel	P401	76-22	5.4	> 10,000	> 10,000	0.45	0.52	4.73	5.31
Gravel	P401	76-22	5.4	> 10,000		0.59		5.88	
Limestone	P401	76-22	5.4	6,400	6,500	0.91	0.92	20.24	16.91
Limestone	P401	76-22	5.4	6,600		0.92		13.58	
Limestone	P401	64-22	5.4	4,200	3,500	3.90	4.39	291.45	399.97
Limestone	P401	64-22	5.4	2,800		4.89		508.49	

Note: 1 inch = 25.4 mm

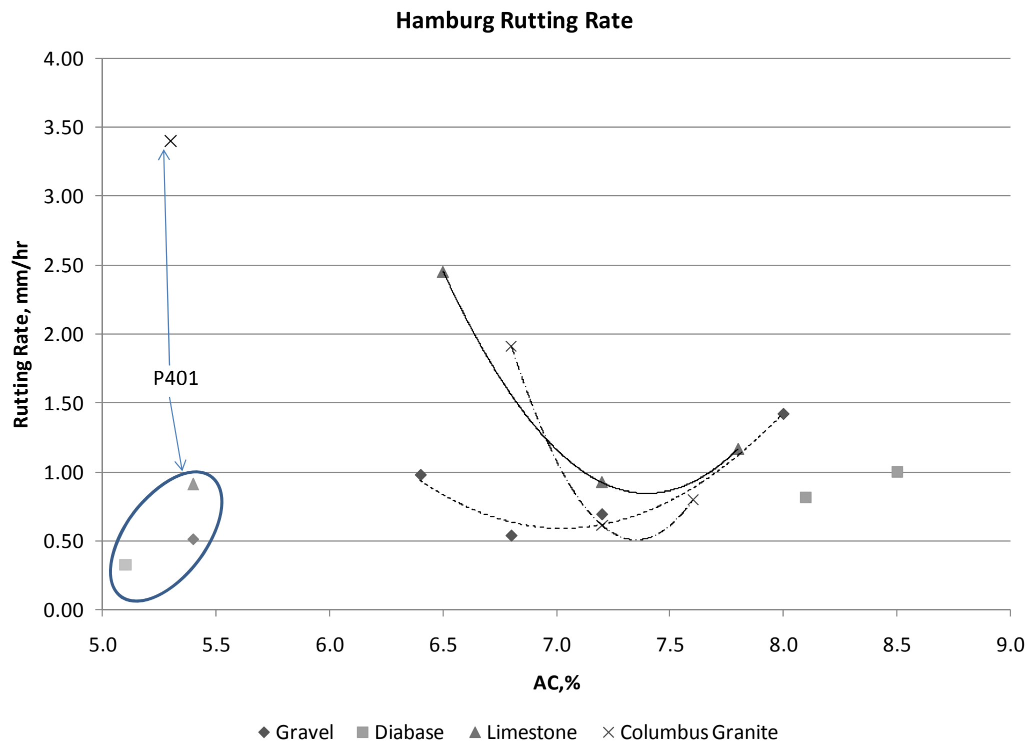

FIGURE 4.8 HWTD Rutting Rates as a Function of Asphalt Content.

Figure 4.9 shows the average total rutting at 10,000 cycles for the PG 76-22 mixes as a function of asphalt content. If the test was stopped prior to 10,000 cycles, the rut depth was extrapolated using a best-fit polynomial regression. The diabase and gravel P401 mixes provide better performance (less total rutting) than the SMA mixtures and the Columbus granite and limestone mixes provide worse performance than the SMA mixtures. The total rutting response of the SMA mixtures appears to be relatively insensitive to asphalt content. ANOVA performed using the total rutting at 10,000 cycles from the PG 76-22 SMA mixtures with aggregate type and lab compaction effort as factors showed that neither compaction effort nor aggregate source were significant ($p = 0.15$ and $p = 0.08$, respectively). A separate ANOVA compared the total rut depths of the PG 76-22 SMA and P401 mixtures. The 100 gyration SMA mixtures were excluded. Aggregate source, mix type and the interaction of aggregate source and mix type were all considered as factors. All three factors were significant. Overall, the total rutting of the SMA mixtures is more consistent, regardless of aggregate source, whereas the performance of some P401 mixtures was better and others worse as described previously.

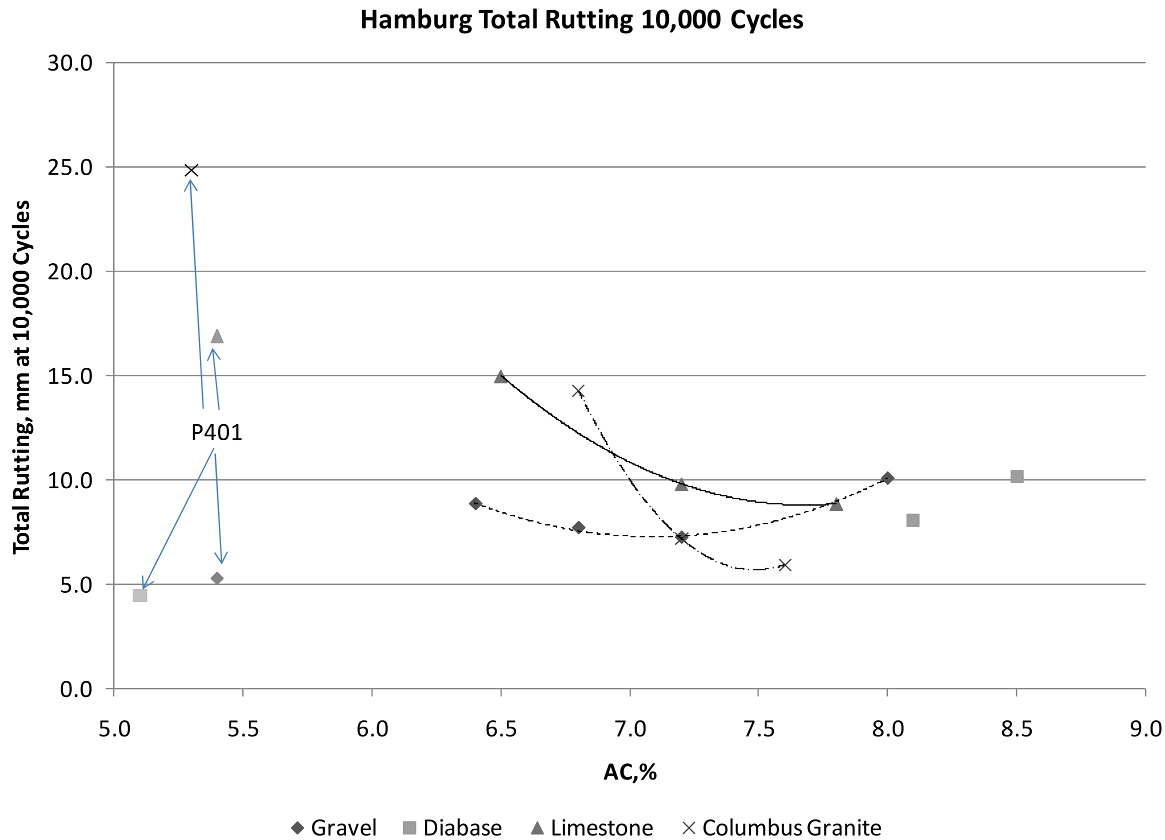

FIGURE 4.9 Total HWTD Rutting as a Function of Asphalt Content.

Finally, comparisons were made between the limestone SMA and P401 mixtures produced with PG 76-22 and PG 64-22. Research conducted as part of the National Cooperative Highway Research Project 9-33, "A Mix Design Manual for Hot Mix Asphalt," indicates that on average HMA produced with a polymer modified binder can withstand 7.1 time more traffic than the same HMA produced with a neat asphalt of the same PG grade (*69*). This relationship was considered as part of the recommendations for selecting polymer modified binders for airfields (*70*).

In an SMA mixture, the aggregate skeleton is expected to carry the load. Therefore, it may be expected that SMA mixtures would be less sensitive to binder grade than dense graded mixes are. However, previous experience with SMA mixtures suggests they may be sensitive to slow speed or turning movements with softer binders. The average rutting rate for the PG 64-22 limestone SMA mixture was 10.4 times that of the PG 76-22 mixture. Recall that the PG 76-22 true graded in excess of a PG 82-22. By comparison, the rutting rate of the PG 64-22 limestone P401 mixture was only 4.8 times that of the PG 76-22 mixture. On average, the PG 64-22 mixtures have a rutting rate 7.6 times that of the PG 76-22 mixtures, very close to the NCHRP 9-33 estimate.

An ANOVA was performed on the rutting rates for the limestone mixtures using mixture type, binder grade, and their interaction as factors. Both factors and their interaction were significant. An interaction plot is shown in Figure 4.10.

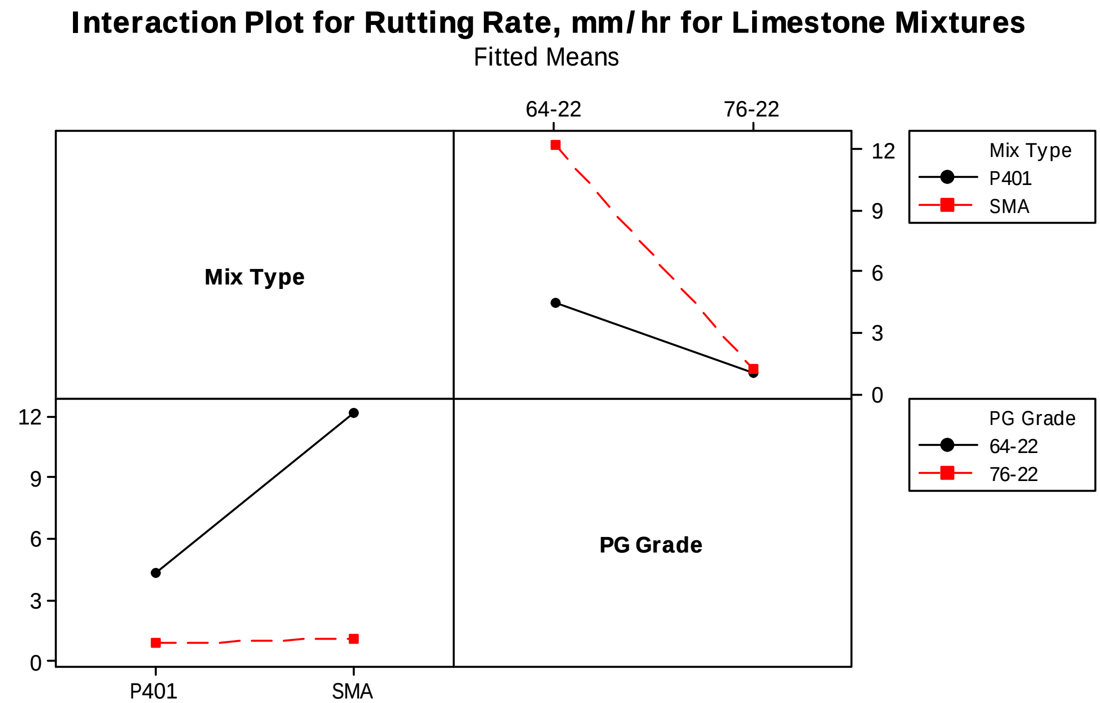

FIGURE 4.10 Interaction Plot for HWTD Rutting Rate for Limestone Mixtures.

Both limestone P401 mixtures and the PG 64-22 limestone SMA mixture experienced stripping inflection points. As discussed previously, if the test was stopped prior to 10,000 cycles, the rut depth at 10,000 cycles was extrapolated. For both the P401 and SMA PG 65-22 mixtures, this resulted in total rut depths greater than the sample thickness. In field conditions, the total rutting of a given layer would be limited to its thickness. Therefore, no further analysis was performed on the total rut depths for the limestone mixtures.

4.6 RECOMMENDATION OF LABORATORY COMPACTION EFFORT

Based on the literature review and international survey of SMA use presented in Chapters 2 and 3, the 50-blow Marshall effort is the standard for the design of SMA. As noted previously, due to the introduction of the Superpave design system in the United States, many contractors and consultants are losing their expertise with the Marshall design system. Thus an effort was made in this study to determine a gyratory compaction effort equivalent to the 50-blow Marshall effort.

VMA was used to compare the compaction efforts. Mixes with the same aggregates and VMA should have the same asphalt content. Figure 4.11 shows VMA as a function of design gyrations, determined at 3 percent air voids. The equivalent number of gyrations to match the 50-blow Marshall compaction effort were determined by linear regression and are summarized in Table 4.17. Examination of the data in Table 4.17 suggested that the L.A. Abrasion loss and percent flat and elongated particles of the aggregate influenced the predicted gyrations. A multiple linear regression was performed using L. A. Abrasion loss and percent flat and elongated particles at the 3:1 ratio as predictors for equivalent gyrations.

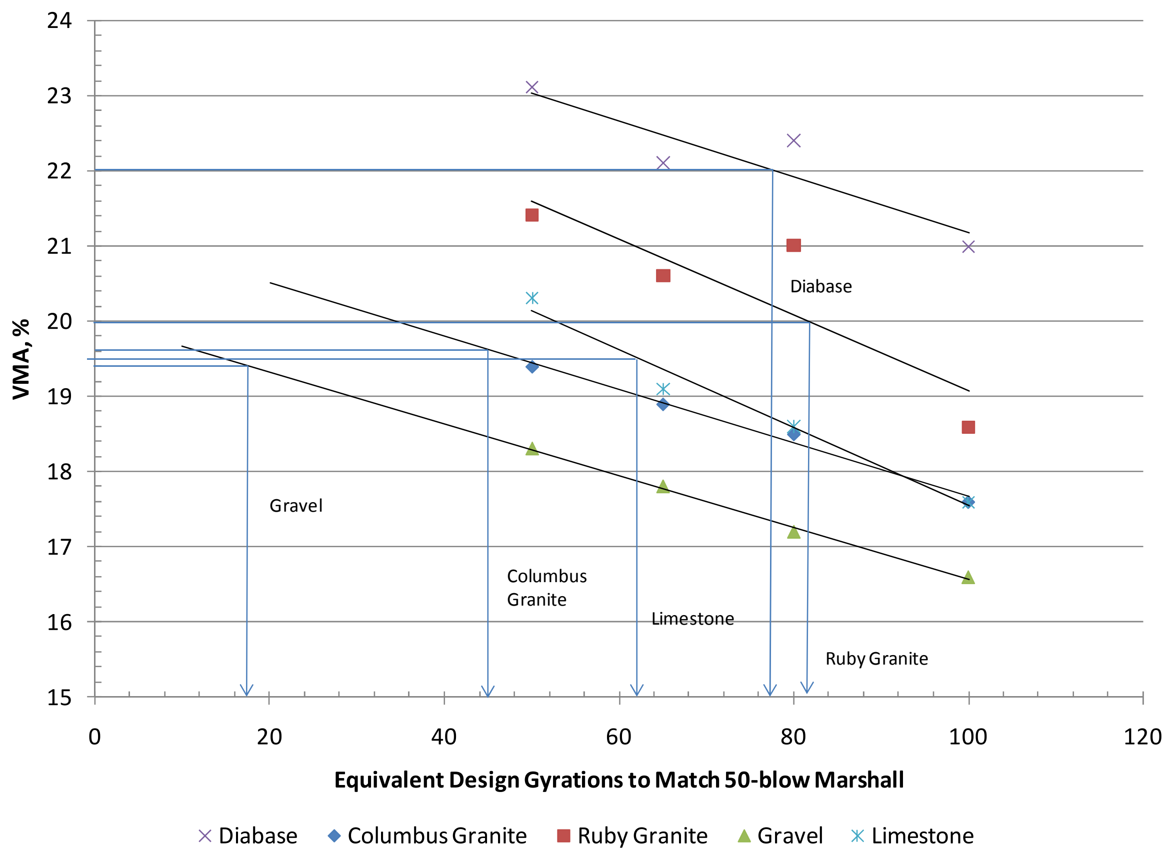

FIGURE 4.11 Equivalent Design Gyrations based on VMA.

TABLE 4.17 Equivalent Gyrations As a Function of Aggregate Properties

Aggregate	LA Abrasion Loss, %	% Flat and Elongated Particles > 3:1	Gyrations to Match VMA
Diabase	18	9.7	77
Columbus Granite	37	7.8	46
Ruby Granite	20	3.3	80
Gravel	30	49.3	18
Limestone	25	10.1	62

Equation 5 produced an $R^2 = 0.99$ with a standard error of 1.23.

$$Equivalent\ Gyrations = 117 - 1.72 \times LA\ Abrasion\ Loss - 0.944\ \times 3{:}1\ F\&E \quad (5)$$

where,
Equivalent gyrations = the number of gyrations to match the 50-blow Marshall result,
LA Abrasion loss = LA Abrasion loss, %, and
3:1 F&E = percent flat and elongated particles exceeding the 3:1 ratio.

Equation 5 suggests that as the LA Abrasion loss or percent flat and elongated particles increase, equivalent gyrations decrease. As the LA Abrasion loss increases, more aggregate breakdown may be expected at higher gyration levels. It is expected that the larger sample size used in the gyratory compactor allows flat and elongated particles to orient better (flatter) reducing VMA as compared to the smaller Marshall samples.

Using Equation 5, it is possible to show a range of potential gyration levels based on aggregate requirements. For instance, the equivalent gyrations for a mixture with 20 percent LA Abrasion loss and 5 percent 3:1 particles, a hard cubicle aggregate, would be 78. For 30 percent LA Abrasion loss and 20 percent 3:1 particles Equation 5 produces 47 equivalent gyrations and 40 percent LA Abrasion loss with 5 percent 3:1 particles produces 43 equivalent gyrations. The higher quality aggregates allow higher gyration levels. This range compares well with previous studies which recommended: 70 (*17*), 70 (*32*), 65 (*33*), and 50 (*35*). This does not say that satisfactory mixes cannot be designed if the design gyrations were higher than the equivalent gyrations for a given set of aggregate properties, just that the resulting VMA would be lower, which may make the mix more difficult to compact in the field.

As described previously, optimum asphalt contents were initially selected at 3 percent air voids. It was noted that in several cases, 3 percent air voids was on the extreme wet side of the VMA curve. As asphalt content was increased, both VMA and VCA_{Ratio} increased. This was problematic in the design process. While it is felt that production air voids near 3 percent are valuable to help achieve in-place density, in production air voids will probably decrease due to extra dust created from aggregate breakdown, breakdown of aggregate contact points, or both. This will decrease voids (and VMA) without increasing the asphalt content. Therefore, 4 percent air voids are recommended for the selection of optimum asphalt content.

The repeated load permanent deformation tests indicated that the 100 gyration mixes were the most rut resistant followed closely by the 80 gyration mixes. The Hamburg wheel-tracking device data indicated that the 80 gyration mixes provided better rutting and moisture resistance than the 100 gyration mixes. Figures 4.12 through 4.14 illustrate the equivalent gyrations for 4 percent design air voids. The equivalent gyrations for 4 percent air voids range from 71 to 85 gyrations for the 100 gyration mixes at 3 percent air voids and 50 to 64 for the 80 gyration mixes designed at 3 percent air voids.

The aggregate data suggests that 50 gyrations would be appropriate for a wider range of LA Abrasion loss and flat and elongated particles. Based on the permanent deformation tests, this expanded range of aggregate properties would not be detrimental to performance. Analysis of the permanent deformation data suggests that 50 to 64 gyrations would match 80 gyrations at 3 percent air voids and still provide good rutting performance. Thus 65 gyrations are recommended as a conservative compromise. During field production, it would be appropriate to target lower laboratory air voids for aggregates with higher L.A. Abrasion loss or higher percentages of flat and elongated particles. Similarly, higher production air voids may be warranted for harder or more cubical aggregates.

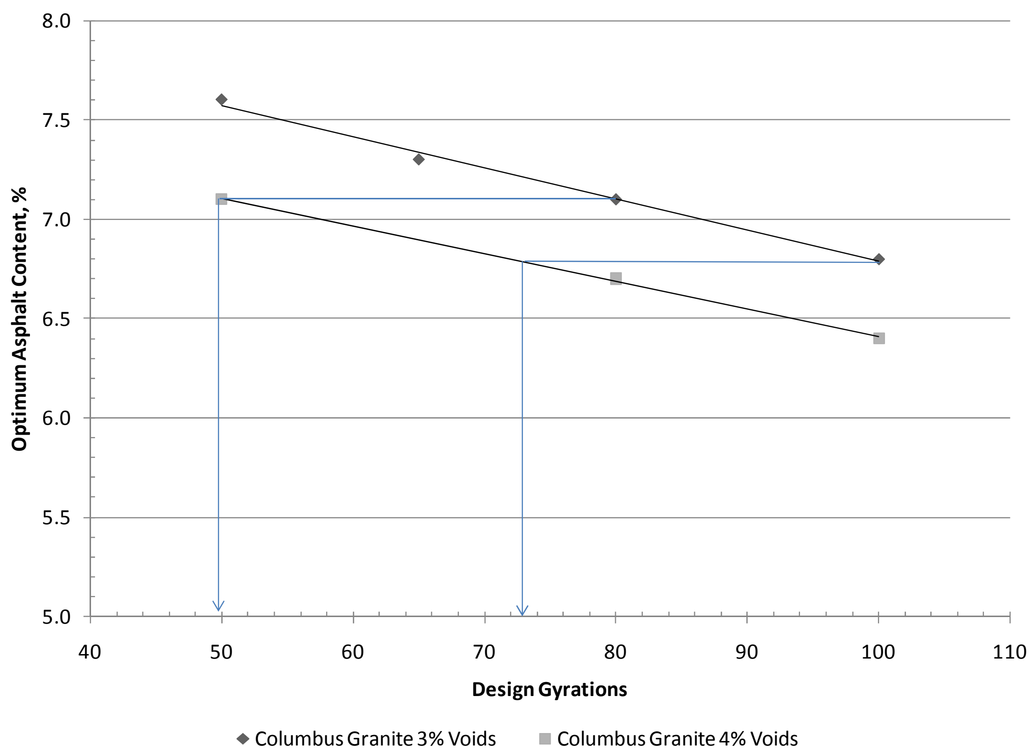

FIGURE 4.12 Equivalent Gyrations for 3 and 4 percent Design Air Voids for Columbus Granite.

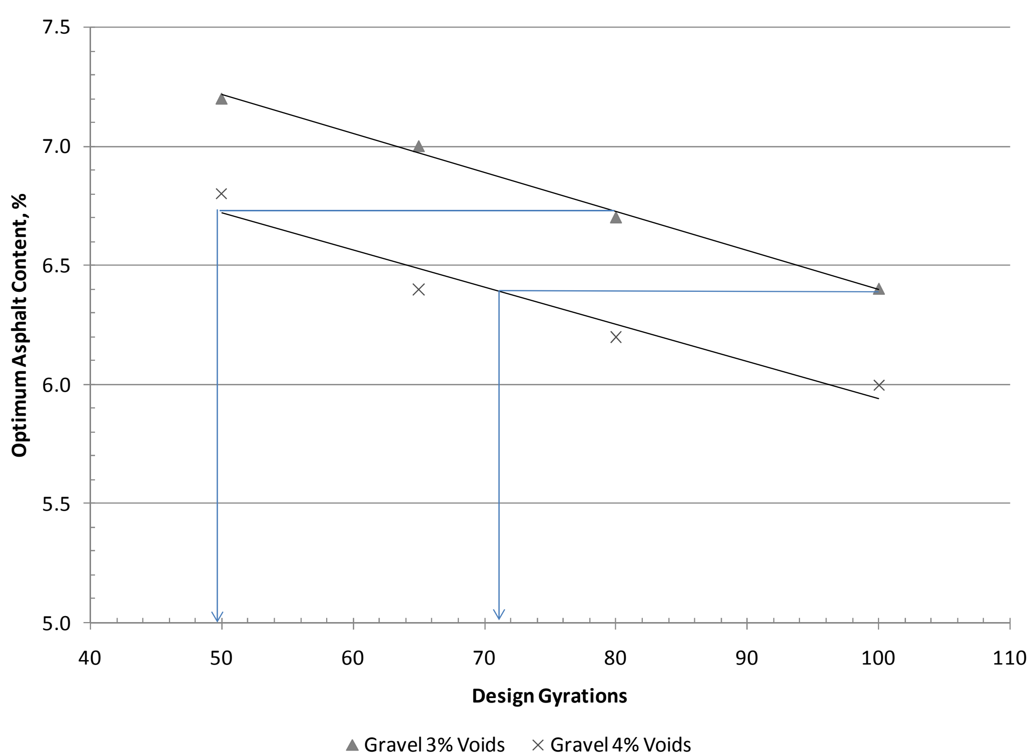

FIGURE 4.13 Equivalent Gyrations for 3 and 4 percent Design Air Voids for Gravel.

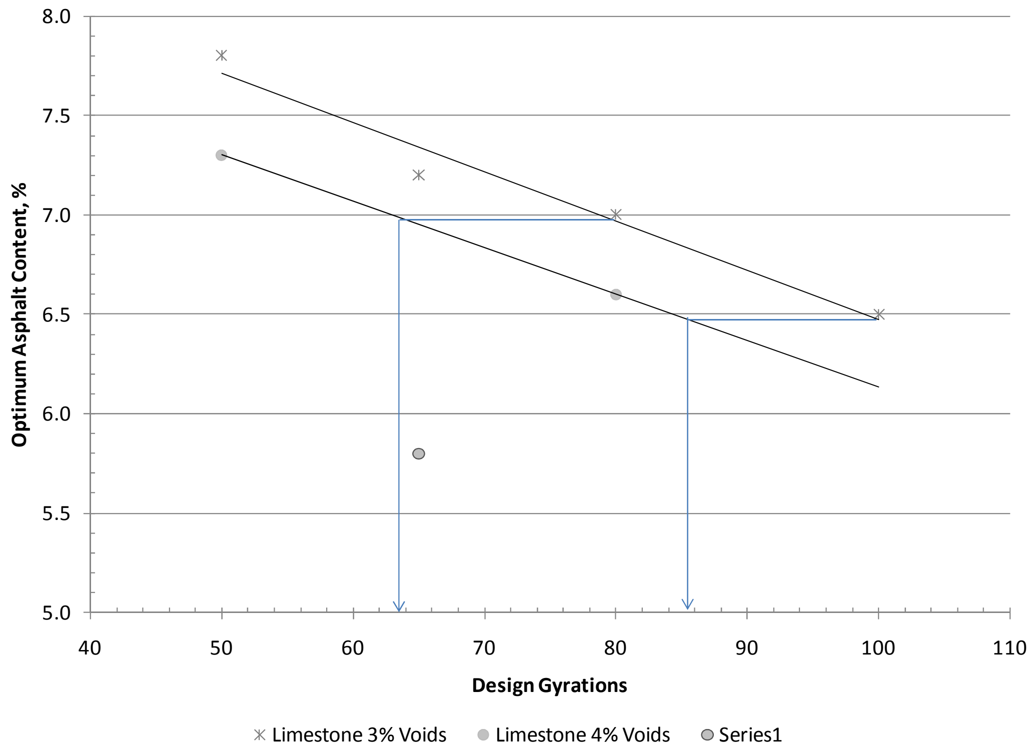

FIGURE 4.14 Equivalent Gyrations for 3 and 4 percent Design Air Voids for Limestone.

4.7 OVERLAY TESTS FOR CRACKING RESISTANCE

Historically, resistance to age related and fatigue cracking has been difficult to quantify in the laboratory. A device called the overlay tester was developed to test the cracking resistance of asphalt mixtures. The device, shown in Figure 4.15, simulates the opening and closing of a joint in a hydraulic cement concrete pavement or existing crack in an asphalt pavement due to environmental stresses. The device does not simulate the bending associated with traffic loads on flexible pavements or load transfer across joints in composite pavements. However, the device was used to correctly rank the fatigue performance of flexible pavement test sections from the Federal Highway Administration's Accelerated Load Facility (ALF) (*71*).

Test samples of the 50-blow Marshall SMA and P401 control mixes were prepared in the SGC at 5 ± 0.5 and 6 ± 0.5 percent air voids, respectively. The test sample is sawed out of the SGC sample using a double-bladed wet saw. The samples were tested according to Texas Department of Transportation Test Method Tex-248-F at 25 °C (77°F) using a maximum cracking opening (deflection) of 0.64 mm (0.025 inches). The samples were prepared by both the National Center for Asphalt Technology (SMA) and Advanced Materials Services, LLC (P401) and tested by the Rutgers University Asphalt Pavement Laboratory.

FIGURE 4.15 Overlay Tester.

Test results for the overlay tester are presented in Table 4.18. Both the SMA and P401 mixes lasted considerably longer than the Superpave mixes previously tested by Rutgers University (Personal communication with Tom Bennert). On average for the mixtures containing PG 76-22, the cycles to failure for the SMA mixtures were 435 percent higher than for the P401 mixtures. This increase clearly demonstrates the potential benefits of SMA in terms of durability. ANOVA indicated that both mix and aggregate type were significant factors (p = 0.000 and 0.017, respectively).

TABLE 4.18 Overlay Tester Test Results

Aggregate Type	Sample ID	Fatigue Life, cycles	
		P401	**SMA**
Columbus Granite	1	788	469
	2	1,219	1,311
	3	1,436	1,438
	4	1,233	943
	5	277	1,708
	6	745	
	Average[1]	**996**	**1,231**
Standard Deviation[1]		**266**	**257**
Ruby Granite	1	1,545	6,779
	2	1,084	29,035
	3	2,508	7,231
	4	2,739	12,900
	5	2,478	2,093
	Average[1]	**2,177**	**8,970**
Standard Deviation[1]		**548**	**3,411**

TABLE 4.18 Overlay Tester Test Results (Continued)

Aggregate Type	Sample ID	Fatigue Life, cycles	
		P401	**SMA**
Diabase	1	962	39,623
	2	3,112	13,467
	3	849	4,694
	4	1,551	4,875
	5	800	13,510
Average[1]		**1,121**	**10,617**
Standard Deviation[1]		**377**	**4,973**
Gravel	1	1,131	3,716
	2	1,491	7,378
	3	822	2,221
	4	683	5,119
	5	1,534	3,489
Average[1]		**1,148**	**4,108**
Standard Deviation[1]		**335**	**883**
Limestone (PG 67-22)	1	915	782
	2	639	532
	3	703	1,651
	4	974	1,422
	5	472	842
Average[1]		**752**	**1,015**
Standard Deviation[1]		**144**	**353**
Limestone (PG 76-22)	1	2,000	30,367
	2	1,371	18,204
	3	1,826	4,183
	4	14,012[2]	9,039
	5	1,666	18,661
	6		
Average[1]		**1,831**	**15,301**
Standard Deviation[1]		**167**	**5,428**

[1]Calculated using a trimmed mean, ignoring highest and lowest reading for fatigue life.
[2]Appears to be an outlier.

Table 4.18 also presents a comparison of overlay tester results using a single aggregate (Limestone) with two different binder grades (PG 64-22 and PG 76-22). It can be seen that the modified binder dramatically improved the cracking resistance compared to the unmodified PG 64-22 binder. Figure 4.16 presents the cracking data in graphical form, showing the percent increase in fatigue life for SMA for each aggregate. The PG 64-22 SMA produced a 35 percent increase in fatigue life. A t-test for equal sample variance indicated no significant difference in the results. Evaluating solely the influence of the modified binder on overlay cracking results, the PG 76-22 produced more than a 1,400 percent increase in fatigue life. This demonstrates the value of an elastic polymer in terms of cracking resistance.

Figure 4.17 presents the relationship between the number of cycles to failure from the overlay tester to the LA Abrasion values for each of the five aggregates. A general trend of decreasing failure cycles to increasing LA Abrasion values can be seen from the graph, with the exception being the Limestone aggregate with PG 76-22 asphalt binder, which had the highest cycles to failure.

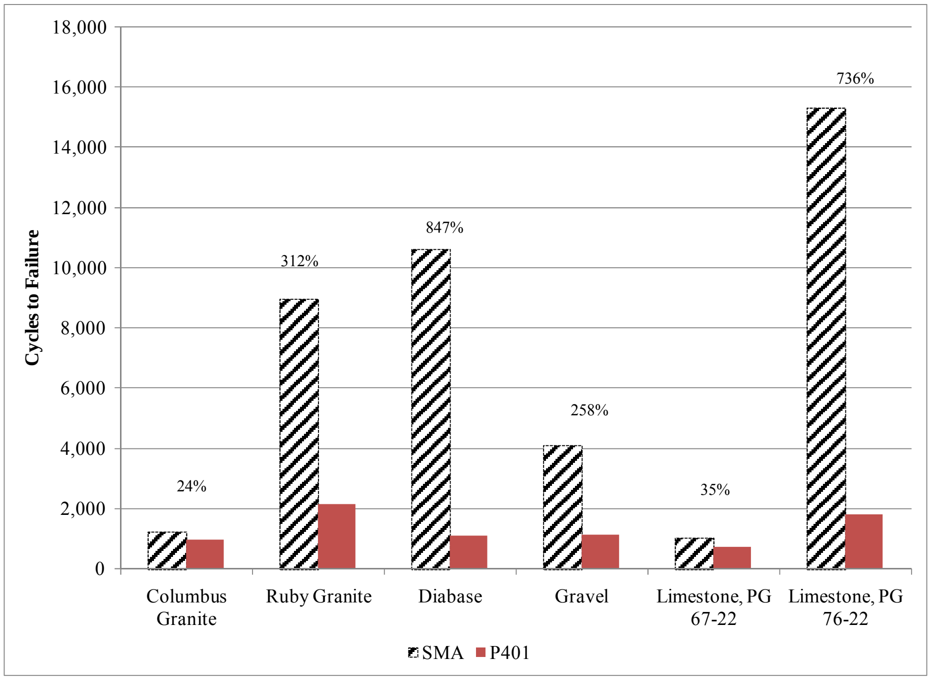

FIGURE 4.16 Overlay Tester Results.

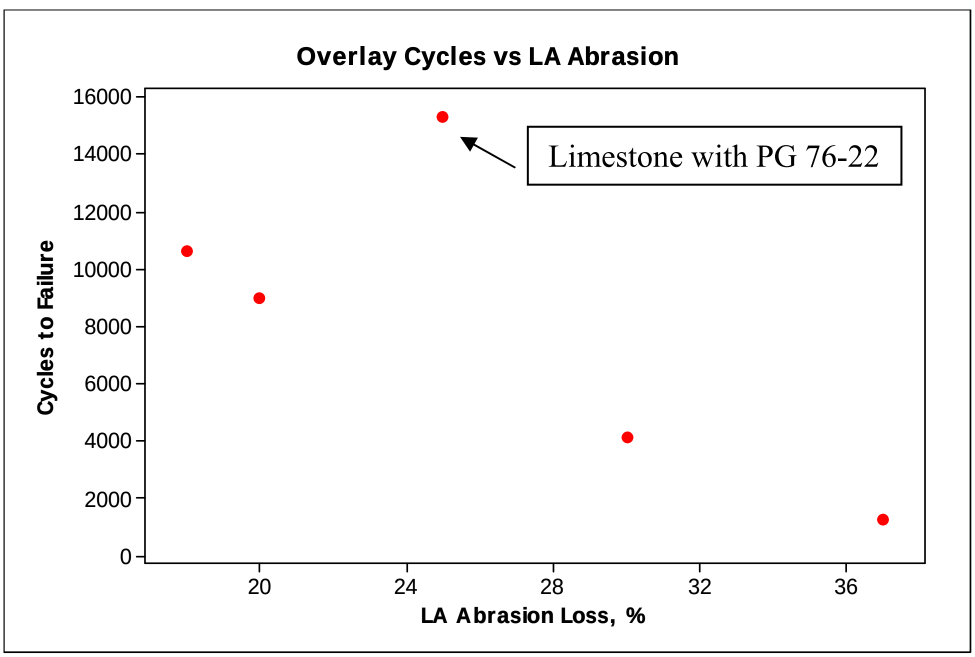

FIGURE 4.17 Overlay Tester Cycles to Failure versus LA Abrasion.

4.8 FUEL RESISTANCE TESTING

In order to evaluate Stone Matrix Asphalt's resistance to fuel-induced failures, samples were prepared and evaluated according to the CITGO Fuel Soak Test Procedure (*72*). The only variations were in the sample air voids and method of producing the test samples. The CITGO Fuel Soak Test calls for test samples to have an air void content of approximately 2.5 percent; samples for this project were compacted to an air void content of approximately 5 ± 0.5 percent for the SMA samples and 6 ± 0.5 percent for the control P401 mixes. Test samples for this project were also produced with the Marshall hammer instead of a Superpave gyratory compactor. A PG 76-22 grade binder was also used for this evaluation. A quick summary of the test procedure is listed below:

- Compact test samples according to the appropriate air void range,
- Submerse the samples in kerosene for two minutes,
- After submersion for two minutes, surface dry the samples with a clean paper towel. Weigh the sample. Record this weight as the initial weight,
- Submerse the samples in kerosene again, this time for a period of 24 hours,
- After 24 hours, remove the samples from the kerosene and allow them to dry under a fan for a period of 24 hours,
- After the 24 hour drying period, weigh the samples again. Record this weight as the final weight,
- Calculate the percent weight loss using the following calculation:

  Percent of weight loss by fuel immersion = [(A-B)/A] * 100

  Where A = initial weight,
  B = final weight.

- Determine the tensile strength of each of the test samples.

Figure 4.18 shows a sample after it has been evaluated by the CITGO Fuel Soak Test. It can be seen from the photo that the kerosene did not fully saturate the sample, but rather only affected the outer portion of the test sample. This allowed the sample to retain approximately 80 percent of its original strength. Table 4.19 presents the test results from the CITGO Fuel Soak Test. The SMA mixtures resulted in 42 and 43 percent less mass loss for the Columbus granite and gravel, respectively. Previous studies suggest that a mixture with a maximum mass loss of 5 percent should be resistant to damage from fuel spills (*Personal communication with Doug Hanson*). The granite SMA mixture meets this criterion. The retained tensile strengths for the SMA and P401 control mixtures were similar.

FIGURE 4.18 Lab Gravel Fuel Resistance Samples After Immersion.

TABLE 4.19 CITGO Fuel Soak Test Results

Aggregate	Mix Type	Treatment	Mass Loss, %	Avg. Failure Load, N (lbs)	Avg. Tensile Strength, kPa (psi)	Tensile Strength Retained, %
Columbus Granite	P401	Fuel	7.8	6717 (1510)	645 (93.5)	51.2
		Control	--	13015 (2926)	1260 (182.8)	--
	SMA	Fuel	4.5	5849 (1314)	542 (78.6)	59.8
		Control	--	9826 (2209)	906 (131.4)	--
Gravel	P401	Fuel	11.6	5667 (1274)	544 (78.9)	79.6
		Control	--	7019 (1587)	684 (99.2)	--
	SMA	Fuel	6.6	4079 (917)	351 (50.9)	73.6
		Control	--	5400 (1214)	476 (69.1)	--

4.9 DEICER EVALUATION

Based on prior evaluations conducted as part of AAPTP Project 05-03 (*73*), test samples were produced and submerged in a potassium acetate solution to evaluate the SMA's resistance to DIAIC-related damage. DIAIC-related damage refers to the damage caused by deicing and anti-icing chemicals. From the research performed as part of AAPTP Project 05-03, the Immersion Tension Test (ITT) was established. A short summary of the ITT test follows:

- Compact test samples according to the appropriate air void range,
- Soak test samples in a 2% potassium acetate solution for a period of four (4) days at a temperature of 60 °C (140 °F),
- Soak control samples in water at 60 °C (140 °F) for four (4) days,

- Determine the IDT strength of each sample after the four (4) day submersion at 25 °C (77 °F),
- Calculate the DIAIC-damage index (DDI). DDI values are calculated as the percent loss or gain in tensile strength after the initial submersion period.

Table 4.20 presents the data obtained from the ITT testing conducted on both the P401 and SMA mixes. In the table, both the average indirect tensile strength values as well as the DDI values for each of the mixes are reported. DDI values of over 20 percent indicate that the pavement may be susceptible to DIAIC-related damage. From the data, it is seen that neither the SMA nor the P401 samples demonstrated any DIAIC-related damage.

TABLE 4.20 Immersion Tensile Test (ITT) Results

Aggregate	Mix Type	Sample Set	Avg. Failure Load, N (lbs)	Avg. Tensile Strength, kPa (psi)	DDI, %
Lab Granite	P401	Dry Control	13015 (2926)	1260 (182.8)	--
		Soaked Control	8136 (1829)	782 (113.4)	--
		2% Potassium Acetate	7931 (1783)	765 (111.0)	2.1
Lab Gravel	P401	Dry Control	7059 (1587)	684 (99.2)	--
		Soaked Control	10017 (2252)	938 (136.0)	--
		2% Potassium Acetate	10066 (2263)	947 (137.4)	0.0
Lab Granite	SMA	Dry Control	9826 (2209)	918 (133.2)	--
		Soaked Control	8447 (1899)	794 (115.1)	--
		2% Potassium Acetate	8176 (1838)	765 (111.0)	3.6
Lab Gravel	SMA	Dry Control	5400 (1214)	482 (69.9)	--
		Soaked Control	6303 (1417)	546 (79.2)	--
		2% Potassium Acetate	6788 (1526)	591 (85.7)	0.0

4.10 EVALUATION OF TEXTURE, FRICTION, AND GROOVING

To evaluate the ability of SMA mixtures to be grooved, a 51 x 51 cm (20 x 20 in) slab of the Columbus Granite SMA (Blend 2) was produced using a linear kneading compactor. Texture and friction measurements were taken using the ASTM E 2157 Circular Texture (CT) Meter and the ASTM E 1911 Dynamic Friction (DF) Tester, respectively. The slab was then grooved to FAA standards (6 x 6 mm (0.25 x 0.25 in) groove on 40 mm (1.5 in) center-to-center spacing) (*49*). Both the CT Meter and the DF Tester measure the surface characteristics in a circular path 284 mm (11.2 in) in diameter. The slab was grooved radially such that the groove spacing would be correct at the diameter measured by the devices and such that the measurements would be normal (at 90 degrees) to the measurement path. Figure 4.19 shows an overview of the grooved slab after polishing.

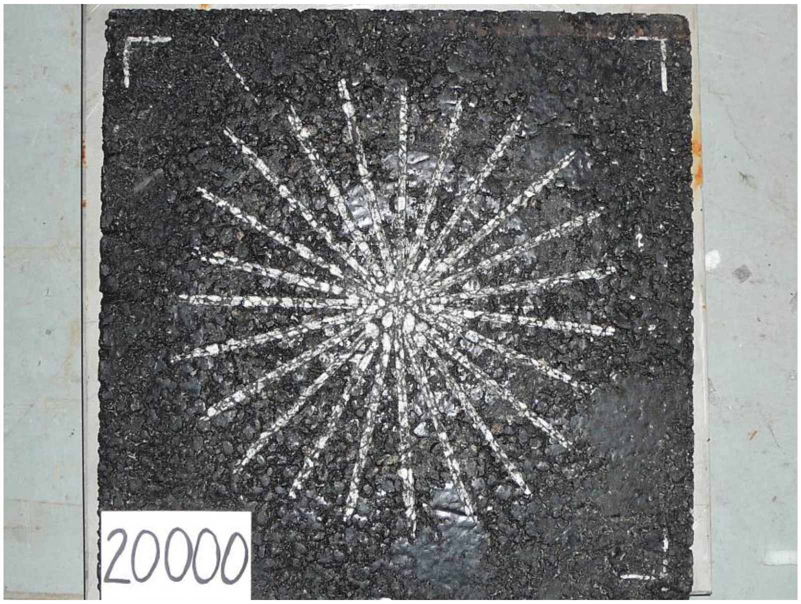

FIGURE 4.19 Grooved Slab after Polishing.

The primary purpose of the experiment was to investigate how the grooves in the SMA would stand up to traffic. The National Center for Asphalt Technology previously developed a three-wheel polishing device to simulate surface wear on a pavement, shown in Figure 4.20 (*74*). The device consists of three pneumatic wheels mounted on a rotating turntable to track along the same path as that measured by the CT Meter and DF Tester. The normal force applied to the wheel is adjustable through the addition of steel plates. For highway traffic, the wheels are loaded with 20 kg (45 lbs). The load was increased to 61 kg (135 lbs) to simulate aircraft. The pneumatic tires were inflated to their maximum pressure of 345 kPa (50 psi). Each rotation of the device results in three tire passes.

FIGURE 4.20 Three-Wheel Polishing Device.

Texture and Friction measurements were obtained before and after grooving and periodically during the polishing process. The friction results are an average of five tests. Texture readings were taken less frequently than friction measurements since the slab must be allowed to dry prior to taking texture measurements. The results are shown in Table 4.21. Texture measurements are expressed in terms of mean profile depth. FAA (*49*) does not include grooves when measuring macrotexture, although they were included in this experiment. Raw DF Tester friction results (mu values) are reported at 20 km/hr and interpolated at 65 km/hr (45 mph). The speed constant and international friction index (IFI) were calculated according to ASTM E1960 using coefficients reported by Wambold (*75*). IFI allows comparisons to be made between various friction measurement devices. FAA does not currently have recommended friction ranges for the DF Tester or IFI.

TABLE 4.21 Friction and Texture Results with Polishing for Columbus Granite SMA

Three-Wheel Polisher Revolutions	MPD, mm	Speed Constant (Sp)	DF Tester, mu		IFI	
			20 km/hr	65 km/hr (40 mph)	F60	F65 (40 mph)
Pre-grooving	0.67	72.73	0.20	0.12	0.16	0.16
0	1.43	157.70	0.22	0.18	0.20	0.20
50	NA	157.70	0.26	0.21	0.23	0.23
250	NA	157.70	0.39	0.30	0.30	0.29
500	NA	157.70	0.42	0.31	0.32	0.31
1000	NA	157.70	0.42	0.36	0.32	0.31
1500	NA	157.70	0.45	0.34	0.34	0.33
2000	1.49	163.92	0.43	0.35	0.33	0.32
3000	NA	163.92	0.41	0.34	0.32	0.31
6000	1.43	157.70	0.40	0.33	0.31	0.30
12000	NA	157.70	0.41	0.36	0.31	0.31
20000	1.49	163.51	0.41	0.36	0.31	0.31

NA = not tested.

The data indicates a significant increase in both the mean profile depth and speed constant after grooving. As noted previously, FAA grooved areas are not typically used when determining macrotexture. As the thick asphalt film associated with SMA mixtures wears off the surface of the pavement, the friction values increase. After 250 cycles, both the DF Tester and IFI friction values appear to remain relatively constant. There was no evidence of groove chipping or disintegration after 20,000 cycles (Figure 4.21). However, the testing was conducted at ambient lab temperatures, not an elevated temperature which might be associated with summer groove closure. Wear of the binder film in the wheel-path is visible in Figure 4.21.

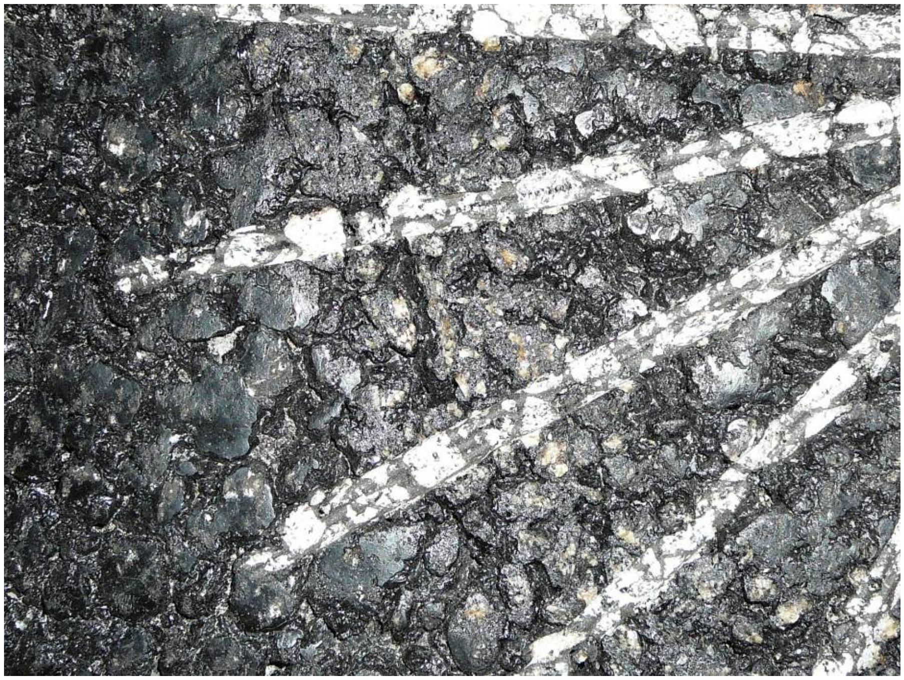

FIGURE 4.21 Close-up of Groove after 20,000 Revolutions.

4.11 SUMMARY OF LABORATORY EXPERIMENTS

Laboratory experiments were conducted to compare the performance of SMA and P401 mixtures. The experiments also examined the appropriate Superpave gyratory compactor compaction effort for SMA mixes in addition to the traditional 50-blow Marshall compaction effort. A range of aggregate sources were included in the experiments including ones with LA Abrasion loss and percentage of flat and elongated particles in excess of that typically specified for SMA. A polymer modified PG 76-22 binder was primarily used for the study. A limited comparison was performed with PG 64-22.

Field experience indicates that it is important to achieve good in-place density with SMA mixtures in order to prevent pavement permeability. To help facilitate in-place density, optimum asphalt contents were initially selected at 3 percent air voids. Evaluation of the volumetric properties as a function of asphalt content suggested that SMA mixes designed at 3 percent air voids were on the wet side of the VMA curve. The wet side of the VMA curve indicates that VMA and VCA_{Ratio} increased with increasing asphalt content. This caused some mixtures to fail VCA_{Ratio} forcing a coarser gradation which may be a disadvantage for airfield pavements.

Three sets of tests were conducted to compare the permanent deformation characteristics of SMA and P401 mixes, including: Marshall stability and flow, repeated load permanent deformation, and the Hamburg wheel-tracking device. SMA was developed in Germany. German specifications state that Marshall stability and flow is “unsuitable” for SMA. Only two of five aggregate sources produced SMA mixtures which met FAA’s stability requirements for P401 mixes for aircraft with gross weights in excess of 27,216 kg (60,000 lbs). All of the SMA mixtures had high flow values.

Repeated load permanent deformation tests were conducted over a range of asphalt contents corresponding to a range of laboratory compaction efforts. The Francken method of determining flow number was best correlated with the experimental factors. Tire pressures of general aviation through military aircraft were simulated by adjusting the deviator stress. Primarily two deviator stresses, 689 and 1,379 kPa (100 and 200 psi), were used. Limited testing was conducted at 2,413 kPa (350 psi). Deviator stress had a significant effect on the rutting performance of both SMA and P401 mixtures. Aggregate source was not a significant factor in the rutting performance of the mixtures analyzed. This suggests that good performing mixtures can be produced using a range of locally available aggregates including gravel sources. Laboratory compaction effort was a significant factor for the SMA mixtures. SMA designed with 80 and 100 gyrations showed the best performance with optimum asphalt content selected at 3 percent air voids. This is equivalent to 50 to 85 gyrations at 4 percent design voids. Overall, the SMA and P401 mixtures produced with PG 76-22 showed similar performance. A comparison of limestone mixtures produced with both PG 64-22 and polymer modified PG 76-22 showed no significant difference in performance at a 689 kPa (100 psi) deviator stress indicative of general aviation airfields.

Similar to the repeated load permanent deformation tests, the Hamburg wheel-tracking device tests were conducted over a range of asphalt contents representing a range of laboratory compaction efforts. The HWTD assess performance in terms of both rutting and moisture sensitivity. Slightly higher asphalt contents, representative of 65 and 80 gyrations at 3 percent air voids, produced the best performance for the SMA mixtures. The SMA mixtures performance on average was approximately the same as the P401 mixtures performance; however the performance of the SMA mixtures was more consistent across aggregate types. This is probably due to the added moisture resistance provided by the thicker binder film-thickness of an SMA mixture.

Worldwide, the 50-blow Marshall compaction effort is the standard for designing SMA mixtures. Due to loss of experience with the Marshall system in the United States resulting from the implementation of the Superpave mix design system, it was desirable to identify an equivalent gyratory compaction effort. The volumetric and permanent deformation data were analyzed to determine a gyratory compaction effort that would be equivalent to the 50-blow Marshall and still provide good rutting performance. A model was developed to estimate an equivalent number of gyrations based on LA Abrasion loss and percent of flat and elongated particles at the 3:1 ratio. Harder, cubical aggregates could be designed with a higher number of gyrations. Softer, or flat and elongated aggregates require lower design gyrations. At 4 percent design air voids, 50 to 85 gyrations provide similar asphalt content to those determined using 80 or 100 gyrations at 3 percent design air voids. Mixes designed using 80 to 100 gyrations at 3 percent air voids performed well in terms of permanent deformation. A laboratory compaction effort of 65 gyrations with optimum asphalt content selected at 4 percent air voids should provide a balance of allowing locally available aggregate sources while still providing good rutting performance.

Additional testing was conducted to assess cracking resistance, fuel resistance, and resistance to deicing chemicals. Increased durability in terms of cracking resistance is expected to be one of the primary benefits of SMA mixtures. Limited testing completed to date indicates a 35 percent increase in resistance to cracking for mixtures with unmodified binders.

Overall, the performance of SMA compared to P401 control mixtures is summarized in Table 4.22. The performance summary is based on the literature review, performance of in-service airfields, and the laboratory testing.

TABLE 4.22 Summary of SMA and P401 Performance Comparison

Property	Performance worse than P401	Performance similar to P401	Performance better than P401
Permanent Deformation		✓ [1]	✓ [2]
Moisture Damage			✓
Cracking			✓
Fuel Resistance			✓
Deicer Resistance		✓	
Texture			✓ [2]

[1]Based on laboratory tests performed as part of this study.
[2]Based on review of the literature or in-service performance.

CHAPTER 5 – IMPLEMENTATION PLAN AND RECOMMENDATIONS FOR FURTHER RESEARCH

Research conducted as part of AAPTP Project 04-03 indicated that cracking was the major form of deterioration on airfields. Rutting, in some cases exacerbated by fuel drippings, was a secondary concern. SMA has the potential to greatly improve cracking performance while offering similar rutting performance. For this research to be effective, the information must be disseminated to airfield managers, military officials and consultant engineers. Additional research is needed to further refine specifications for construction.

5.1 IMPLEMENTATION PLAN

The implementation plan consists of three parts:

- Development of a draft FAA technical advisory for SMA for airfields (Appendix C),
- Development and dissemination of a one-hour Power Point Presentation summarizing the results of this research and providing a basic overview of SMA design and construction issues,
- Construction of two demonstration projects, which will also address some of the requirements for additional research.

A Power Point presentation will be developed summarizing the findings of this research and providing an overview of SMA design and construction. The target audience for the presentation will be airfield managers and design consultants who are not necessarily materials specialists. A draft version of this presentation will be presented at the Task 8 panel meeting for review. A longer course could be offered for materials specialists. A copy of the presentation should be provided to the Asphalt Institute for inclusion in their airfield pavement seminars.

Two airfield repaving or new construction projects should be selected as demonstration projects for SMA. The demonstration projects will allow additional information to be collected during construction to refine the SMA specification. The areas for recommended data collection are defined below. Secondly, the demonstration projects will allow airfield managers, military personnel and consultants to view an SMA construction project first hand. Members of the research team could provide presentations to the project visitors providing an overview of the project, and overview of SMA design and construction, and specifics of the demonstration project design and construction data.

5.2 RECOMMENDATIONS FOR ADDITIONAL RESEARCH

- Perform repeated-load permanent deformation and Hamburg tests on the Columbus granite SMA and P401 mixes with PG 64-22 to help better define the need for modified binders.
- Overlay tester tests were conducted on the P401 mixes and SMA mixes at one asphalt content. The asphalt content for the SMA mixes was determined using the 50-blow

Marshall effort at 3 percent air voids. Based on analyses of the project data 4 percent air voids are now recommended. Additional overlay tester tests should be performed on the SMA mixtures using either the 50-blow Marshall or 65 gyration optimum asphalt content determined at 4 percent air voids to provide an estimate of the increased performance in terms of cracking.

- Field data should be collected during the demonstration projects to quantify:
  - Field variability of gradation and asphalt content,
  - Draindown performance,
  - Field variability of laboratory air voids and VCA_{Ratio},
  - Field variability of mat and joint densities,
  - Documentation of joint construction techniques,
  - Permeability tests on Field Cores.
- Friction and texture tests should be conducted on the demonstration projects immediately after construction, after six months, and then yearly for three years.
- The performance of the demonstration projects should be monitored for a period of three years, preferably longer. Monitoring should include rutting, cracking, and texture. Other U.S. airfield SMA projects such as Indianapolis International and Naval Air Station Oceana should also be monitored.
- Additional research is needed to evaluate the significance of L. A. Abrasion loss, especially in regard to particle shape requirements, and its effect on reflective cracking.

CHAPTER 6 – CONCLUSIONS AND RECOMMENDATIONS

SMA was developed in Germany during the 1960's as a durable asphalt mixture which was resistant to studded tire wear and permanent deformation. In 1990, the American Association of State Highway and Transportation Officials (AASHTO) European Asphalt Study Tour brought the German asphalt mix technology known as "Splittmastixasphalt" to the United States. The initial interest in the use of SMA on highway pavements in the U.S. was later overwhelmed by the release and subsequent implementation of the Superpave Mix Design System. Durability concerns for Superpave resulted in a resurgence in interest in SMA for highway pavements. SMA was identified as having been used on 34 airfields in seven countries to date.

6.1 CONCLUSIONS

Although rutting can be a problem on airfields, cracking is a more prevalent problem. One of the major advantages of SMA mixtures compared to dense-graded mixtures is its durability. The results of the literature review and survey of SMA use on airfields support the following conclusions:

- SMA is rut resistant.
- Field surveys of in-service SMA pavements indicate cracking was minimal and could be explained by design choice(s); SMA reduced the rate of propagation of reflective cracks.
- State department of highways estimated that SMA provided 33 to 103 percent longer service lives than conventional dense-graded mixes. Life-cycle cost analysis using performance data from in-service pavements indicted that SMA would still be cost effective even if 82 to 94 percent more expensive initially than conventional mixes.
- Limited problems were observed with the use of SMA on airfields primarily related to:
    - low in-place density (permeability),
    - too high of in-place density (blistering – easily resolved), and
    - too soft of binder (picking of aggregate particles by hot aircraft tires until binder aged).
- SMA has a higher macro-texture than dense-graded pavements (average 1.26 mm reported). SMA placed on Aviano and Spangdahlem Air Forces Bases has not been grooved and reportedly provides good friction. Rubber build-up is removed twice a year at Aviano Air Force Base.

The laboratory study focused on refining the SMA design procedure in terms of aggregate properties, laboratory compaction effort, volumetric properties, rutting performance, cracking performance, resistance to fuel, and resistance to deicing agents. Based on the laboratory study (and supported by the literature review and survey of the use of SMA on airfields) the following conclusions can be made:

- SMA can successfully be designed with coarse aggregates having LA Abrasion loss in excess of 30 percent. Several state departments of transportation support LA

Abrasion loss of 40 percent. However, the use of aggregates with higher L.A. Abrasion loss appears to reduce durability.

- SMA can be designed with flat and elongated particles at the 3:1 ratio in excess of 20 percent. Although these mixes showed good performance, the effect on field compaction was not determined.
- SMA can be designed with gravel aggregate sources. The source used in this study had 77 percent two fractured faces, but provided good rutting performance. The high percentage of flat and elongated particles may have contributed to its performance.
- All of the mixtures tested had VCA_{Ratios} less than 1.0. VCA_{Ratio} is a concept developed in the United States to help ensure stone-on-stone contact.
- Although lower laboratory air voids may help facilitate field compaction, optimum asphalt content should be selected at 4.0 percent air voids. Lower air voids are selected on the wet side of the VMA curve, which could increase rutting potential. This can cause problems with achieving a VCA_{Ratio} less than 1.0 and may result in a mixture that is very sensitive to changes in asphalt content or compaction effort. Further, VMA is expected to decrease during production due to aggregate breakdown, which will also result in a reduction in voids. Mixes that are either under, or over- asphalted may be difficult to compact in the field.
- Stability values of SMA mixtures were lower than those from dense-graded P401 mixes produced with the same aggregate source. Flow values for SMA mixtures were much higher than for dense-graded P401 mixtures produced with the same aggregate source. Both of these conditions would indicate susceptibility to rutting. Field performance and other laboratory tests do not support this assumption. Stability and flow does not appear to be applicable to SMA mixtures and is not recommended based on German experience.
- The Francken flow numbers was the best response for assessing mixture performance from repeated load permanent deformation tests. Based on the flow number results:
    - Tire pressure has a significant effect on rutting performance,
    - Aggregate source, for the range of aggregate used in this study, did not have a significant effect on permanent deformation,
    - For optimum asphalt contents selected at 3 percent air voids, SMA mixes designed with 80 and 100 gyrations provided an improved level of rutting performance. This is an equivalent optimum asphalt content at 4 percent air voids for mixes designed with 50 to 85 gyrations,
    - The permanent deformation performance of SMA and dense-graded P401 mixes was not significantly different,
    - When tested at 689 kPa (100 psi) deviator stress, both SMA and P401 mixtures produced with both PG 64-22 and PG 76-22 performed equally well. This indicates neat binders can be successfully used at general aviation airfields.
- The Hamburg wheel-tracking device tests indicated that SMA improved the performance of certain aggregate sources and provided a more consistent level of performance across aggregate sources.

- For optimum asphalt contents selected at 3 percent air voids, SMA mixes designed with 65 and 80 gyrations provided an improved level of rutting performance.
    - The rutting rate of the limestone (SMA and P401) mixes produced with PG 64-22 was 7.6 times that of the same mixes produced with PG 76-22.
- Analysis of the aggregate and volumetric data indicate that LA Abrasion loss and percent flat and elongated particles at the 3:1 ratio have a strong influence on the equivalent number of gyrations to the 50-blow Marshall compaction effort.
- Analysis of the volumetric and permanent deformation data indicate that mixtures designed at 4.0 percent air voids using either the 50-blow Marshall or 65 gyration laboratory compaction effort should produce similar volumetric properties, allow a range of aggregate properties including more local materials and provide for good rutting performance.
- The overlay tester indicates a 435 percent increase in cycles until cracking occurs for SMA mixtures as compared to dense-graded mixtures (produced with polymer modified binder). This increase in durability is the most significant benefit of SMA mixtures.
- The fuel and deicer resistance of SMA and P401 mixtures are approximately equal in the laboratory. Experience in China indicates improved fuel resistance for SMA mixtures.
- A laboratory-scale sample of grooved SMA withstood accelerated loading using NCAT's three-wheel polishing device without groove closure. Ridges, resulting from diamond grinding of SMA test sections at the NCAT Test Track have withstood heavy truck traffic.
- Since the SMA mixes had 1.5 to 2.0 percent higher asphalt content, the durability of SMA should be much greater than conventional mixes.

6.2 RECOMMENDATIONS

Based on the results of the literature review, survey of SMA use on airfields, and laboratory testing, the following design and construction parameters are recommended for SMA for airfields. These recommendations are implemented in the draft advisory circular, presented in Appendix C.

TABLE 6.1 Recommended Coarse Aggregate Properties

Property	Airfield pavements with gross aircraft weights < 27, 216 kg (60,000 lbs) and with tire pressures < 689 kPa (100 psi)	Airfield pavements with gross aircraft weights > 27, 216 kg (60,000 lbs) or with tire pressures > 689 kPa (100 psi)
LA Abrasion Loss ASTM C131, %	30[1]	30[1]
Flat and Elongated Particles ASTM D 4791 percent > than 5:1, 3:1	5, 20[2]	5, 20[2]
Fractured Faces (FF) ASTM D5821 percent >2 FF, >1 FF	85, 95	90, 100

[1] Coarse aggregate sources with higher L.A. Abrasion loss values may be considered on a case-by-case basis.

[2] Laboratory testing indicates mixes can be designed with percentages of flat and elongated particles greater than that shown. The ability to construct these mixes in the field has not been verified. The Engineer may allow a higher percentage, not to exceed 10% 5:1 and 50% 3:1.

TABLE 6.2 Design Gradations

Sieve Size, mm (in)	Percent Passing by Mass		
	9.5 mm NMAS	12.5 mm NMAS	19.0 mm NMAS[1]
19.0 (3/4)	100	100	90-100
12.5 (1/2)	100	90-100	50-88
9.5 (3/8)	70-95	50-85	25-60
4.75 (No. 4)	26-40	20-32	20-28
2.36 (No. 8)	20-28	16-24	16-24
0.075 (No. 200)	8-12	8-12	8-11

[1]Not recommended for use on the surface of the pavement. 19.0 mm SMA has been successfully used on airfields in China and highways in Virginia and Maryland below the pavement surface.

TABLE 6.3 Recommended Binder Grades

Expected Loading	Recommended PG Binder Grade
Airfield pavements with gross aircraft weights < 27, 200 kg (60,000 lbs) and with tire pressures < 689 kPa (100 psi)	The same grade PG binder used by the state highway department in the area should be considered as the base grade for the project (e.g. the grade typically specified in that specific location for dense graded mixes on highways with design Equivalent Standard Axle Loads (ESALS) less than 10 million). The exception would be that grades with a low temperature higher than PG XX-22 should not be used (e.g. PG XX-16 or PG XX-10), unless the Engineer has had successful experience with them.
Airfield pavements with gross aircraft weights > 27, 200 kg (60,000 lbs) and with tire pressures > 689 kPa (100 psi) but < 1,378 kPa (200 psi)	Increase (bump) high temperature by two grades, e.g. PG 76-22 instead of PG 64-22, if PG 64-22 is the base climatic grade.
Taxiways or ends of runways subject to stacking for Airfield pavements with gross aircraft weights > 27, 200 kg (60,000 lbs) and with tire pressures > 689 kPa (100 psi) but < 1,378 kPa (200 psi) or for airfield pavements with design aircraft tire pressures > 1,378 kPa (200 psi).	Increase (bump) high temperature by two grades, e.g. PG 76-22 instead of PG 64-22, if PG 64-22 is the base climatic grade.

NOTE: Various highway agencies are currently evaluating the multiple stress creep and recovery (MSCR) test for use in the PG binder grading system. This test will better address the unique characteristics of modified binder than the current DSR tests at high temperature. Once the MSCR is implemented in the PG binder grading system, the grade adjustments given in the table above will need to be modified to reflect the changes in the PG binder grading system.

TABLE 6.4 Recommended Design and Acceptance Properties

Property	Requirement
Cellulose or mineral fiber	Required (dosage rate typically 0.3% by total mix weight).
Draindown, ASTM D 6390, at 13.9 °C (25 °F) above anticipated production temperature	< 0.3 percent
Laboratory compaction effort	50-Blow Marshall or 65 design gyrations
Minimum VMA	17.0 percent
VCA_{Ratio}	< 1.0, VCA_{DRC} determined according to ASTM C 29
Air voids for optimum asphalt content selection	4.0 percent
Acceptance air void range	2.8 to 4.2
In-place density Specification Tolerance Limit for Mat Density, L[1]	96.8 percent of G_{mb} or 93.5 percent of G_{mm}
In-place density Specification Tolerance Limit for Joint Density, L[1]	93.9 percent of G_{mb} or 90.5 percent of G_{mm}

[1]Recommended specification tolerances are based upon the use of FAA's percent within limits (PWL) specification. An average mat density of 95 percent of Gmm or 98.4 percent of Gmb with a standard deviation of 1.3 percent or less will produce 90 PWL (100 percent pay). An average joint density of 93 percent of Gmm or 96.4 percent of Gmb with a standard deviation of 2.1 percent or less will produce 90 PWL.

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APPENDIX A – SMA MIX DESIGNS

TABLE A1. Diabase SMA Mix Design Summary, 50 Blow Marshall

Project: AAPTP 04-04 SMA for Airfields				Aggregate Type: Diabase	Percent Retained on #8 sieve: 78	Date
AC Sp. Gr. (Gb) = 1.028	App. Sp. Gr. (Gsa) 3.043	Eff. Sp. Gr. (Gse): 2.990	Bulk Sp. Gr. (Gsb): 2.971	Trial Blend: 2	Bulk Sp. Gravity of CA: 2.973	8/25/2008
				NMAS: 12.5	Density of CA in DRC (kg/m^3): 1596.5	
				% Passing #8 Sieve: 22.0	Compactive Effort: 50 blow Marshall	

		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS									
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %	Height, in	Stability, lbs	Flow, 0.01 in
1	6.0	1200.5	725.8	1209.0	2.484	2.684	78.6	14.5	155.0	7.4	21.4	65	38.7	46.2	0.838	5.8	2.44		
2	6.0	1200.1	724.1	1209.5	2.472	2.684	78.2	14.4	154.3	7.9	21.8	64	39.0	46.2	0.845	5.8	2.45		
3	6.0	1214.9	734.1	1220.8	2.496	2.684	79.0	14.6	155.8	7.0	21.0	67	38.4	46.2	0.832	5.8	2.46		
Avg.					2.484	2.684	78.6	14.5	155.0	7.4	21.4	65	38.9	46.2	0.842	5.8	2.45		
1	7.0	1222.5	737.4	1226.2	2.501	2.638	78.3	17.0	156.1	5.2	21.7	76	39.0	46.2	0.844	6.8	2.49		
2	7.0	1214.5	733.0	1218.0	2.504	2.638	78.4	17.1	156.3	5.1	21.6	77	38.9	46.2	0.842	6.8	2.44		
3	7.0	1216.1	737.3	1219.2	2.524	2.638	79.0	17.2	157.5	4.3	21.0	79	38.4	46.2	0.832	6.8	2.44		
Avg.					2.510	2.638	78.6	17.1	156.6	4.9	21.4	77	38.9	46.2	0.843	6.8	2.46		
1	8.0	1210.8	734.8	1213.2	2.531	2.595	78.4	19.7	157.9	2.5	21.6	89	38.9	46.2	0.842	7.8	2.41	2500	35.0
2	8.0	1224.6	739.4	1227.0	2.511	2.595	77.8	19.5	156.7	3.2	22.2	86	39.4	46.2	0.853	7.8	2.47	2200	33.5
3	8.0	1220.4	736.9	1222.8	2.512	2.595	77.8	19.5	156.7	3.2	22.2	86	39.4	46.2	0.852	7.8	2.47	2200	33.5
Avg.					2.518	2.595	78.0	19.6	157.1	3.0	22.0	87	39.1	46.2	0.847	7.8	2.45	2300	34.0

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
6.0	7.4	21.4	65	38.9	0.842	155.0	5.8
7.0	4.9	21.4	77	38.9	0.843	156.6	6.8
8.0	3.0	22.0	87	39.1	0.847	157.1	7.8
7.5	4.0	21.7	82	39.0	0.845	156.8	7.3

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
8's	2.990	76
10's	3.030	15
Fly Ash	2.667	0
Mineral Filler	2.718	8
Lime	2.350	1
		100

Comb. Gsb = 2.964

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
8's		76
10's		15
Fly Ash	2.713	0
Mineral Filler	2.718	8
Lime	2.350	1
		100

Comb. Gsa =

TABLE A2. Diabase SMA Mix Design Summary, 50 Gyrations

Project:	AAPTP 04-04 SMA for Airfields								Aggregate Type:	Diabase		Percent Retained on #8 sieve:	78			Date
AC Sp. Gr. (Gb) = 1.028		App. Sp. Gr. (Gsa) 3.043		Eff. Sp. Gr. (Gse): 2.990		Bulk Sp. Gr. (Gsb): 2.964			Trial Blend:	2		Bulk Sp. Gravity of CA:	2.973			8/25/2008
									NMAS:	12.5		Density of CA in DRC (kg/m ^{3}):	1596.5			
									% Passing #8 Sieve:	22.0		Compactive Effort:	50 Gyrations			
		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS						
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %
1	8.0	4833.7	2906.6	4843.3	2.496	2.595	77.5	19.4	155.7	3.8	22.5	83	39.8	46.2	0.861	7.7
2	8.0	4874.7	2925.1	4882.5	2.490	2.595	77.3	19.4	155.4	4.0	22.7	82	39.9	46.2	0.864	7.7
3	8.0	4856.1	2907.6	4865.8	2.480	2.595	77.0	19.3	154.7	4.4	23.0	81	40.1	46.2	0.869	7.7
Avg.					2.489	2.595	77.2	19.4	155.3	4.1	22.8	82	39.8	46.2	0.862	7.7
1	8.4	4875.1	2926.3	4882.8	2.492	2.578	77.0	20.4	155.5	3.3	23.0	85	40.1	46.2	0.868	8.1
2	8.4	4890.8	2937.6	4895.3	2.498	2.578	77.2	20.4	155.9	3.1	22.8	86	40.0	46.2	0.865	8.1
3	8.4	4864.5	2914.6	4870.9	2.487	2.578	76.8	20.3	155.2	3.5	23.2	85	40.2	46.2	0.871	8.1
Avg.					2.492	2.578	77.0	20.4	155.5	3.3	23.0	86	40.0	46.2	0.867	8.1
1	8.5	4852.1	2918.3	4861.1	2.497	2.574	77.1	20.7	155.8	3.0	22.9	87	40.0	46.2	0.867	8.2
2	8.5	4884.3	2946.3	4893.6	2.508	2.574	77.4	20.7	156.5	2.6	22.6	89	39.8	46.2	0.861	8.2
3	8.5	4864.5	2921.1	4875.4	2.489	2.574	76.8	20.6	155.3	3.3	23.2	86	40.2	46.2	0.871	8.2
Avg.					2.498	2.574	77.1	20.7	155.9	2.9	22.9	87	39.9	46.2	0.864	8.2

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
8.0	4.1	22.8	82	39.8	0.862	155.3	7.7
8.4	3.3	23.0	86	40.0	0.867	155.5	8.1
8.5	2.9	22.9	87	39.9	0.864	155.9	8.2
8.1	4.0	22.8	83	39.9	0.863	155.3	7.8

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
8's	2.990	76
10's	3.030	15
Fly Ash	2.667	0
Mineral Filler	2.718	8
Lime	2.350	1
		100

Comb. Gsb = 2.964

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
8's		76
10's		15
Fly Ash	2.713	0
Mineral Filler	2.718	8
Lime	2.350	1
		100

Comb. Gsa = 3.043

TABLE A3. Diabase SMA Mix Design Summary, 65 Gyrations

Project: AAPTP 04-04 SMA for Airfields			Aggregate Type: Diabase	Percent Retained on #8 sieve: 78	Date
AC Sp. Gr. (Gb) = 1.028	App. Sp. Gr. (Gsa) 3.043	Eff. Sp. Gr. (Gse): 2.990	Bulk Sp. Gr. (Gsb): 2.964	Trial Blend: 2	Bulk Sp. Gravity of CA: 2.973	8/25/2008
				NMAS: 12.5	Density of CA in DRC (kg/m ^{3}): 1596.5	
				% Passing #8 Sieve: 22.0	Compactive Effort: 65 Gyrations	

		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS						
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %
1	7.7	4870.9	2940.8	4879.5	2.512	2.608	78.2	18.8	156.8	3.7	21.8	83	39.2	46.2	0.848	7.4
2	7.7	4832.8	2912.5	4844.5	2.501	2.608	77.9	18.7	156.1	4.1	22.1	82	39.4	46.2	0.853	7.4
Avg.					2.507	2.608	78.1	18.8	156.4	3.9	21.9	82	39.3	46.2	0.851	7.4
1	8.0	4809.2	2904.8	4818.7	2.513	2.595	78.0	19.6	156.8	3.2	22.0	86	39.3	46.2	0.852	7.7
2	8.0	4827.8	2912.8	4836.0	2.510	2.595	77.9	19.5	156.6	3.3	22.1	85	39.4	46.2	0.853	7.7
Avg.					2.510	2.595	78.0	19.3	156.6	3.2	22.0	84	39.3	46.2	0.851	7.6
1	8.2	4851.3	2930.7	4861.2	2.513	2.590	77.8	20.0	156.8	3.0	22.2	87	39.5	46.2	0.855	7.9
2	8.2	4867.0	2946.6	4872.8	2.527	2.590	78.3	20.2	157.7	2.4	21.7	89	39.1	46.2	0.847	7.9
3	8.2	4863.9	2947.2	4871.9	2.527	2.590	78.3	20.2	157.7	2.4	21.7	89	39.1	46.2	0.847	7.9
Avg.					2.522	2.590	78.1	20.1	157.4	2.6	21.9	88	39.3	46.2	0.851	7.9

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
7.7	3.9	21.9	82	39.3	0.851	156.4	7.4
8.0	3.2	22.0	84	39.3	0.851	156.6	7.6
8.2	2.6	21.9	88	39.3	0.851	157.4	7.9
8.1	3.0	21.9	86	39.3	0.851	157.0	7.7
7.6	4.0	22.0	81	39.3	0.851	156.2	7.3

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
8's	2.990	76
10's	3.030	15
Fly Ash	2.667	0
Mineral Filler	2.718	8
Lime	2.350	1
		100

Comb. Gsb = 2.964

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
8's		76
10's		15
Fly Ash	2.713	0
Mineral Filler	2.718	8
Lime	2.350	1
		100

Comb. Gsa =

TABLE A4. Diabase SMA Mix Design Summary, 80 Gyrations

Project:	AAPTP 04-04 SMA for Airfields			Aggregate Type:	Diabase	Percent Retained on #8 sieve:	78
AC Sp. Gr. (Gb) = 1.028	App. Sp. Gr. (Gsa) 3.043	Eff. Sp. Gr. (Gse): 2.990	Bulk Sp. Gr. (Gsb): 2.964	Trial Blend:	2	Bulk Sp. Gravity of CA:	2.973
				NMAS:	12.5	Density of CA in DRC (kg/m ^{3}):	1596.5
				% Passing #8 Sieve:	22.0	Compactive Effort:	80 Gyrations
Date	8/25/2008						

Specimen Number	Asphalt Content	Masses: In Air (gms)	Masses: In Water (gms)	Masses: SSD (gms)	Specific Gravities: Bulk (Gmb)	Specific Gravities: TMD (Gmm)	Volumes at Ndes: Aggregate Volume cc	Volumes at Ndes: AC by Volume %	Unit Weight, pcf	Voids: VTM, %	Voids: VMA, %	Voids: VFA, %	Voids: VCAmix, %	Voids: VCAdrc, %	Voids: VCAmix / VCAdrc	Eff. AC Content %
1	5.0	4754.4	2943.2	4777.6	2.592	2.730	83.1	12.6	161.7	5.1	16.9	70	35.4	46.2	0.766	4.7
2	5.0	4700.1	2916.6	4726.2	2.597	2.730	83.2	12.6	162.1	4.9	16.8	71	35.3	46.2	0.763	4.7
3	5.0					2.730										4.7
Avg.					2.595	2.730	83.2	12.6	161.9	5.0	16.8	71	35.3	46.2	0.765	4.7
1	7.0	4778.5	2910.1	4789.1	2.543	2.638	79.8	17.3	158.7	3.6	20.2	82	37.9	46.2	0.822	6.7
2	7.0	4776.5	2916.9	4785.4	2.556	2.638	80.2	17.4	159.5	3.1	19.8	84	37.6	46.2	0.815	6.7
Avg.					2.550	2.638	80.0	17.4	159.1	3.3	20.0	83	37.8	46.2	0.818	6.7
1	7.5	4781.4	2902.4	4792.1	2.530	2.616	79.0	18.5	157.9	3.3	21.0	84	38.6	46.2	0.836	7.2
2	7.5	4808.5	2915.9	4820.0	2.525	2.616	78.8	18.4	157.6	3.5	21.2	84	38.7	46.2	0.838	7.2
3	7.5	4657.7	2832.7	4664.7	2.542	2.616	79.3	18.5	158.6	2.8	20.7	86	38.3	46.2	0.829	7.2
Avg.					2.533	2.616	79.0	18.5	158.0	3.2	21.0	85	38.7	46.2	0.837	7.2
1	7.7	4815.9	2912.3	4823.3	2.520	2.608	78.5	18.9	157.3	3.4	21.5	84.3	39.0	46.2	0.844	7.4
2	7.7	4873.2	2955.2	4879.3	2.533	2.608	78.9	19.0	158.0	2.9	21.1	86.3	38.7	46.2	0.837	7.4
Avg.					2.526	2.608	78.7	18.9	157.6	3.1	21.3	85	38.8	46.2	0.840	7.4

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
5.0	5.0	16.8	71	35.3	0.765	161.9	4.7
7.0	3.3	20.0	83	37.8	0.818	159.1	6.7
7.5	3.2	21.0	85	38.7	0.837	158.0	7.2
7.7	3.1	21.3	85	38.8	0.840	157.6	7.4
8.2	3.0	22.2	87	39.6	0.857	156.7	7.9
6.4	4.0	19.0	79	37.0	0.801	160.0	6.1

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
8's	2.990	76
10's	3.030	15
Fly Ash	2.667	0
Mineral Filler	2.718	8
Lime	2.350	1
		100

Comb. Gsb = 2.964

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
8's		76
10's		15
Fly Ash	2.713	0
Mineral Filler	2.718	8
Lime	2.350	1
		100

Comb. Gsa =

TABLE A5. Diabase SMA Mix Design Summary, 100 Gyrations

Project:	AAPTP 04-04 SMA for Airfields			Aggregate Type: Diabase	Percent Retained on #8 sieve: 78	Date
AC Sp. Gr. (Gb) = 1.028	App. Sp. Gr. (Gsa) 3.043	Eff. Sp. Gr. (Gse): 2.990	Bulk Sp. Gr. (Gsb): 2.964	Trial Blend: 2	Bulk Sp. Gravity of CA: 2.973	8/25/2008
				NMAS: 12.5	Density of CA in DRC (kg/m ^{3}): 1596.5	
				% Passing #8 Sieve: 22.0	Compactive Effort: 100 Gyrations	

Specimen Number	Asphalt Content	Masses: In Air (gms)	Masses: In Water (gms)	Masses: SSD (gms)	Specific Gravities: Bulk (Gmb)	Specific Gravities: TMD (Gmm)	Volumes at Ndes: Aggregate Volume cc	Volumes at Ndes: AC by Volume %	Unit Weight, pcf	Voids: VTM, %	Voids: VMA, %	Voids: VFA, %	Voids: VCAmix, %	Voids: VCAdrc, %	Voids: VCAmix / VCAdrc	Eff. AC Content %
1	6.5	4784.3	2928.8	4797.7	2.560	2.661	80.8	16.2	159.7	3.8	19.2	80	37.2	46.2	0.805	6.2
2	6.5	4783.6	2919.8	4798.5	2.546	2.661	80.3	16.1	158.9	4.3	19.7	78	37.5	46.2	0.813	6.2
Avg.					2.553	2.661	80.54	16.1	159.3	4.1	19.5	79	37.4	46.2	0.809	6.2
1	7.0	4782.3	2908.1	4792.1	2.538	2.638	79.6	17.3	158.4	3.8	20.4	81	38.1	46.2	0.824	6.7
2	7.0	4785.5	2913.3	4795.7	2.542	2.638	79.8	17.3	158.6	3.6	20.2	82	38.0	46.2	0.822	6.7
3	7.0	4766.3	2896.9	4781.8	2.529	2.638	79.3	17.2	157.8	4.1	20.7	80	38.3	46.2	0.829	6.7
Avg.					2.536	2.638	79.584	17.271	158.273	3.9	20.4	81	38.0	46.2	0.823	6.7
1	7.5	4812.0	2927.7	4821.9	2.540	2.616	79.3	18.5	158.5	2.9	20.7	86.1	38.3	46.2	0.830	7.2
2	7.5	4826.0	2932.1	4835.5	2.535	2.616	79.1	18.5	158.2	3.1	20.9	85.3	38.5	46.2	0.833	7.2
Avg.					2.538	2.616	79.203	18.516	158.366	3.0	20.8	86	38.4	46.2	0.831	7.2

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
6.5	4.1	19.5	79	37.4	0.809	159.3	6.2
7.0	3.9	20.4	81	38.0	0.823	158.3	6.7
7.5	3.0	20.8	86	38.4	0.831	158.4	7.2
6.7	4.0	19.8	80	37.6	0.814	158.9	6.4

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
8's	2.990	76
10's	3.030	15
Fly Ash	2.667	0
Mineral Filler	2.718	8
Lime	2.350	1
		100

Comb. Gsb = 2.964

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
8's		76
10's		15
Fly Ash	2.713	0
Mineral Filler	2.718	8
Lime	2.350	1
		100

Comb. Gsa =

TABLE A6. Columbus Granite SMA Mix Design, 50 Blow Marshall

Project:	AAPTP 04-04 SMA for Airfields		Aggregate Type: Granite	Percent Retained on #4 sieve: 70.6
App. Sp. Gr. (Gsa): 2.725	Eff. Sp. Gr. (Gse): 2.726	Bulk Sp. Gr. (Gsb): 2.687	Trial Blend: 1	Bulk Sp. Gravity of CA: 2.691
AC Sp. Gr. (Gb) = 1.028			NMAS: 12.5	Density of CA in DRC (kg/m^3): 1546.8
			% Passing #4 Sieve: 29.4	Compactive Effort: 50 blow Marshall
Date	8/25/2008			

		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS									
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %	Height, in	Stability, lbs	Flow, 0.01 in
1	6.3	1161.7	674.5	1163.4	2.376	2.470	82.9	14.6	148.3	3.8	17.1	78	41.6	42.4	0.981	5.8	2.49	2600	20.5
2	6.3	1191.4	693.1	1192.7	2.385	2.470	83.2	14.6	148.8	3.5	16.8	79	41.4	42.4	0.976	5.8	2.48	2775	22.0
3	6.3	1171.7	682.0	1173.4	2.384	2.470	83.1	14.6	148.8	3.5	16.9	79	41.4	42.4	0.976	5.8	2.51	2550	18.0
Avg.					2.382	2.470	83.1	14.6	148.6	3.6	16.9	79	41.5	42.4	0.978	5.8	2.49	2642	20.2
1	6.8	1194.5	692.7	1195.4	2.376	2.452	82.4	15.7	148.3	3.1	17.6	82	41.9	42.4	0.988	6.3	2.48	2450	31.0
2	6.8	1200.7	698.0	1201.6	2.384	2.452	82.7	15.8	148.8	2.8	17.3	84	41.7	42.4	0.983	6.3	2.52	2600	30.0
3	6.8	1189.1	689.5	1189.9	2.376	2.452	82.4	15.7	148.3	3.1	17.6	82	41.9	42.4	0.988	6.3	2.52	2000	23.0
Avg.					2.379	2.452	82.5	15.7	148.4	3.0	17.5	83	41.8	42.4	0.986	6.3	2.51	2350	28.0
1	7.3	1188.0	691.6	1188.6	2.390	2.433	82.5	17.0	149.2	1.8	17.5	90	41.9	42.4	0.987	6.8	2.50	2400	22.5
2	7.3	1190.0	692.7	1190.6	2.390	2.433	82.5	17.0	149.1	1.8	17.5	90	41.9	42.4	0.987	6.8	2.50	1800	16.0
3	7.3	1187.9	691.8	1189.1	2.389	2.433	82.4	17.0	149.1	1.8	17.6	90	41.9	42.4	0.988	6.8	2.52	2225	20.5
Avg.					2.390	2.433	82.4	17.0	149.1	1.8	17.6	90	41.9	42.4	0.987	6.8	2.51	2142	19.7

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
6.3	3.6	16.9	79	41.5	0.978	148.6	5.8
6.8	3.0	17.5	83	41.8	0.986	148.4	6.3
7.3	1.8	17.6	90	41.9	0.987	149.1	6.8
5.9	4.0	16.6	76	41.3	0.973	148.8	5.4

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.678	0
7's	2.691	76
89's	2.667	0
Mineral Filler	2.718	8
M-10's	2.673	15
Lime	2.350	1
		100
Comb. Gsb =		2.687

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.720	0
7's	2.741	76
89's	2.713	0
Mineral Filler	2.718	8
M-10's	2.681	15
Lime	2.350	1
		100
Comb. Gsa =		2.725

TABLE A7. Columbus Granite SMA Mix Design, 50 Gyrations

Project:	AAPTP 04-04 SMA for Airfields									Aggregate Type:	Granite			Percent Retained on #4 sieve:	75.5		Date
AC Sp. Gr. (Gb) = 1.028		App. Sp. Gr. (Gsa) 2.731		Eff. Sp. Gr. (Gse): 2.709		Bulk Sp. Gr. (Gsb): 2.688				Trial Blend:	1			Bulk Sp. Gravity of CA:	2.691		8/25/2008
										NMAS:	12.5			Density of CA in DRC (kg/m ^{3}):	1550		
										% Passing #4 Sieve:	24.5			Compactive Effort:	50 Gyrations		
		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS							
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %	
1	7.8	4675.2	2677.8	4682.1	2.333	2.410	80.0	17.7	145.6	3.2	20.0	84	39.7	42.3	0.938	7.5	
2	7.8	4861.4	2798.1	4867.2	2.350	2.410	80.6	17.8	146.6	2.5	19.4	87	39.2	42.3	0.928	7.5	
3	7.8	4824.6	2779.7	4827.2	2.356	2.410	80.8	17.9	147.0	2.2	19.2	88	39.0	42.3	0.923	7.5	
Avg.					2.346	2.410	80.5	17.8	146.4	2.6	19.5	86	39.3	42.3	0.930	7.5	
1	7.3	4843.2	2785.3	4848.0	2.348	2.428	81.0	16.7	146.5	3.3	19.0	83	38.9	42.3	0.921	7.0	
2	7.3	4826.0	2768.0	4833.9	2.336	2.428	80.6	16.6	145.8	3.8	19.4	81	39.2	42.3	0.928	7.0	
3	7.3	4838.3	2780.2	4847.2	2.341	2.428	80.7	16.6	146.1	3.6	19.3	81	39.1	42.3	0.925	7.0	
Avg.					2.342	2.428	80.8	16.6	146.1	3.6	19.2	82	39.1	42.3	0.925	7.0	

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
7.8	2.6	19.5	86	39.3	0.930	146.4	7.5
7.3	3.6	19.2	82	39.1	0.925	146.1	7.0
7.6	3.0	19.4	85	39.2	0.928	146.3	7.3
7.1	4.0	19.1	79	39.0	0.922	146.0	6.8

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.678	0
7's	2.691	84
89's	2.667	0
Mineral Filler	2.718	9
M-10's	2.673	6
Lime	2.350	1
		100

Comb. Gsb = 2.688

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.720	0
7's	2.741	84
89's	2.713	0
Mineral Filler	2.718	9
M-10's	2.681	6
Lime	2.350	1
		100

Comb. Gsa = 2.731

TABLE A8. Columbus Granite SMA Mix Design, 65 Gyrations

Project:	AAPTP 04-04 SMA for Airfields	Aggregate Type:	Granite	Percent Retained on #4 sieve:	75.5
AC Sp. Gr. (Gb) = 1.028	App. Sp. Gr. (Gsa) 2.731	Trial Blend:	1	Bulk Sp. Gravity of CA:	2.691
Eff. Sp. Gr. (Gse): 2.709	Bulk Sp. Gr. (Gsb): 2.688	NMAS:	12.5	Density of CA in DRC (kg/m ^{3}):	1550
Date 8/25/2008		% Passing #4 Sieve:	24.5	Compactive Effort:	65 Gyrations

Specimen Number	Asphalt Content	Masses: In Air (gms)	Masses: In Water (gms)	Masses: SSD (gms)	Specific Gravities: Bulk (Gmb)	Specific Gravities: TMD (Gmm)	Volumes at Ndes: Aggregate Volume cc	Volumes at Ndes: AC by Volume %	Unit Weight, pcf	Voids: VTM, %	Voids: VMA, %	Voids: VFA, %	Voids: VCAmix, %	Voids: VCAdrc, %	Voids: VCAmix / VCAdrc	Eff. AC Content %
1	5.5	4768.1	2759.2	4784.5	2.354	2.486	82.8	12.6	146.9	5.3	17.2	69	37.6	42.3	0.889	5.2
2	5.5	4740.9	2741.0	4761.4	2.347	2.486	82.5	12.6	146.4	5.6	17.5	68	37.8	42.3	0.894	5.2
3	5.5	4758.2	2753.4	4778.2	2.350	2.486	82.6	12.6	146.6	5.5	17.4	69	37.7	42.3	0.891	5.2
Avg.					2.350	2.486	82.6	12.6	146.7	5.5	17.4	69	37.7	42.3	0.891	5.2
1	6.0	4764.5	2755.6	4774.2	2.360	2.468	82.5	13.8	147.3	4.4	17.5	75	37.8	42.3	0.893	5.7
2	6.0	4777.3	2766.1	4784.2	2.367	2.468	82.8	13.8	147.7	4.1	17.2	76	37.6	42.3	0.888	5.7
3	6.0	4778.7	2761.1	4792.9	2.352	2.468	82.2	13.7	146.8	4.7	17.8	74	38.0	42.3	0.898	5.7
Avg.					2.360	2.468	82.5	13.8	147.3	4.4	17.5	75	37.7	42.3	0.891	5.7
1	7.3	4837.4	2788.4	4845.3	2.352	2.428	81.1	16.7	146.8	3.1	18.9	83	38.8	42.3	0.918	7.0
2	7.3	4834.1	2790.0	4840.2	2.358	2.428	81.3	16.7	147.1	2.9	18.7	85	38.7	42.3	0.915	7.0
3	7.3	4841.3	2783.2	4846.0	2.347	2.428	80.9	16.7	146.5	3.3	19.1	82	39.0	42.3	0.921	7.0
Avg.					2.352	2.428	81.1	16.7	146.8	3.1	18.9	83	38.8	42.3	0.917	7.0

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
5.5	5.5	17.4	68.6	37.7	0.891	146.7	5.2
6.0	4.4	17.5	74.9	37.7	0.891	147.3	5.7
7.3	3.1	18.9	83.5	38.8	0.917	146.8	7.0
6.5	4.0	18.1	77.6	38.2	0.903	146.9	6.2

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.678	0
7's	2.691	84
89's	2.667	0
Mineral Filler	2.718	9
M-10's	2.673	6
Lime	2.350	1
		100

Comb. Gsb = 2.688

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.720	0
7's	2.741	84
89's	2.713	0
Mineral Filler	2.718	9
M-10's	2.681	6
Lime	2.350	1
		100

Comb. Gsa = 2.731

TABLE A9. Columbus Granite SMA Mix Design, 80 Gyrations

Project:	AAPTP 04-04 SMA for Airfields			Aggregate Type:	Granite	Percent Retained on #4 sieve:	75.5	Date
AC Sp. Gr. (Gb) = 1.028	App. Sp. Gr. (Gsa) 2.731	Eff. Sp. Gr. (Gse): 2.709	Bulk Sp. Gr. (Gsb): 2.688	Trial Blend:	1	Bulk Sp. Gravity of CA:	2.691	8/25/2008
				NMAS:	12.5	Density of CA in DRC (kg/m ^{3}):	1550	
				% Passing #4 Sieve:	24.5	Compactive Effort:	80 Gyrations	

		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS						
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %
1	6.3	4789.5	2753.9	4811.3	2.328	2.466	81.1	14.3	145.3	5.6	18.9	70	38.8	42.3	0.918	6.0
2	6.3	4791.4	2766.7	4812.0	2.343	2.466	81.7	14.4	146.2	5.0	18.3	73	38.4	42.3	0.908	6.0
3	6.3	4797.3	2766.8	4817.9	2.339	2.466	81.5	14.3	145.9	5.2	18.5	72	38.5	42.3	0.911	6.0
Avg.					2.336	2.466	81.4	14.3	145.8	5.3	18.6	72	38.6	42.3	0.913	6.0
1	6.8	4770.9	2755.0	4780.4	2.356	2.447	81.7	15.6	147.0	3.7	18.3	80	38.4	42.3	0.908	6.5
2	6.8	4793.7	2777.7	4801.2	2.369	2.447	82.1	15.7	147.8	3.2	17.9	82	38.1	42.3	0.900	6.5
3	6.8	4789.7	2774.9	4798.4	2.367	2.447	82.1	15.7	147.7	3.3	17.9	82	38.1	42.3	0.901	6.5
Avg.					2.364	2.447	82.0	15.6	147.5	3.4	18.0	81	38.2	42.3	0.904	6.5
1	7.3	4823.5	2791.1	4829.0	2.367	2.428	81.6	16.8	147.7	2.5	18.4	86	38.4	42.3	0.909	7.0
2	7.3	4847.4	2796.6	4854.3	2.356	2.428	81.2	16.7	147.0	3.0	18.8	84	38.7	42.3	0.916	7.0
3	7.3	4835.4	2787.2	4843.9	2.351	2.428	81.1	16.7	146.7	3.2	18.9	83	38.9	42.3	0.919	7.0
Avg.					2.358	2.428	81.3	16.7	147.1	2.9	18.7	85	38.6	42.3	0.913	7.0

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
6.3	5.3	18.6	72	38.6	0.913	145.8	6.0
6.8	3.4	18.0	81	38.2	0.904	147.5	6.5
7.3	2.9	18.7	85	38.6	0.913	147.1	7.0
6.7	4.0	18.4	78	38.5	0.910	146.7	6.5
7.1	3.0	18.5	83	38.5	0.910	147.2	6.8

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.678	0
7's	2.691	84
89's	2.667	0
Mineral Filler	2.718	9
M-10's	2.673	6
Lime	2.350	1
		100

Comb. Gsb = 2.688

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.720	0
7's	2.741	84
89's	2.713	0
Mineral Filler	2.718	9
M-10's	2.681	6
Lime	2.350	1
		100

Comb. Gsa = 2.731

TABLE A10. Columbus Granite SMA Mix Design, 100 Gyrations

Project:	AAPTP 04-04 SMA for Airfields								Aggregate Type:	Granite		Percent Retained on #4 sieve:		75.5		Date
AC Sp. Gr. (Gb) = 1.028		App. Sp. Gr. (Gsa) 2.731		Eff. Sp. Gr. (Gse): 2.709		Bulk Sp. Gr. (Gsb): 2.688			Trial Blend:	1		Bulk Sp. Gravity of CA:		2.691		8/25/2008
									NMAS:	12.5		Density of CA in DRC (kg/m ^{3}):		1550		
									% Passing #4 Sieve:	24.5		Compactive Effort:		100 Gyrations		
		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS						
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %
1	6.3	4795.0	2772.8	4809.4	2.354	2.466	82.1	14.4	146.9	4.5	17.9	75	38.1	42.3	0.901	6.0
2	6.3	4805.3	2778.4	4818.9	2.355	2.466	82.1	14.4	146.9	4.5	17.9	75	38.1	42.3	0.901	6.0
3	6.3	4806.3	2781.0	4816.4	2.361	2.466	82.3	14.5	147.3	4.2	17.7	76	37.9	42.3	0.897	6.0
Avg.					2.357	2.466	82.2	14.4	147.1	4.4	17.8	75	38.1	42.3	0.901	6.0
1	6.8	4773.3	2764.7	4777.3	2.372	2.447	82.2	15.7	148.0	3.1	17.8	83	38.0	42.3	0.898	6.5
2	6.8	4710.5	2734.3	4715.2	2.378	2.447	82.5	15.7	148.4	2.8	17.5	84	37.8	42.3	0.894	6.5
3	6.8	4852.5	2819.1	4856.4	2.382	2.447	82.6	15.8	148.6	2.7	17.4	85	37.7	42.3	0.892	6.5
Avg.					2.377	2.447	82.4	15.7	148.3	2.9	17.6	84	37.9	42.3	0.896	6.5

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
6.3	4.4	17.8	75	38.1	0.901	147.1	6.0
6.8	2.9	17.6	84	37.9	0.896	148.3	6.5
6.4	4.0	17.8	78	38.0	0.900	147.4	6.2
6.8	3.0	17.6	83	37.9	0.897	148.2	6.5

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.678	0
7's	2.691	84
89's	2.667	0
Mineral Filler	2.718	9
M-10's	2.673	6
Lime	2.350	1
		100

Comb. Gsb = 2.688

Combined Gsa of Aggregates

	GsA	% Blend
67's	2.720	0
7's	2.741	84
89's	2.713	0
Mineral Filler	2.718	9
M-10's	2.681	6
Lime	2.350	1
		100

Comb. Gsa = 2.731

TABLE A11. Ruby Granite SMA Mix Design Summary, 50 Blow Marshall

Project:	AAPTP 04-04 SMA for Airfields			Date: 8/28/2008
AC Sp. Gr. (Gb) = 1.028	App. Sp. Gr. (Gsa) 2.763	Eff. Sp. Gr. (Gse): 2.747	Bulk Sp. Gr. (Gsb): 2.713	
Aggregate Type: Ruby	Trial Blend: 8-B	NMAS: 12.5	% Passing #4 Sieve: 25.7	
Percent Retained on #4 sieve: 74.3	Bulk Sp. Gravity of CA: 2.728	Density of CA in DRC (kg/m^3): 1616.4	Compactive Effort: 50 blow Marshall	

		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS									
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %	Height, in	Stability, lbs	Flow, 0.01 in
1	6.0	1094.2	632.2	1099.0	2.344	2.497	81.2	13.7	146.3	6.1	18.8	67	40.0	40.6	0.984	5.6	2.43	1950	35
2	6.0	1103.5	638.6	1107.4	2.354	2.497	81.6	13.7	146.9	5.7	18.4	69	39.7	40.6	0.978	5.6	2.41	2200	26
3	6.0	1106.5	636.7	1113.2	2.322	2.497	80.5	13.6	144.9	7.0	19.5	64	40.5	40.6	0.998	5.6	2.49	2500	32
Avg.					2.340	2.497	81.1	13.7	146.0	6.3	18.9	67	40.1	40.6	0.987	5.6	2.44	2217	31
1	6.5	1110.7	644.1	1113.5	2.366	2.478	81.5	15.0	147.7	4.5	18.5	76	39.7	40.6	0.978	6.1	2.41	1750	23
2	6.5	1101.9	633.9	1106.3	2.333	2.478	80.4	14.7	145.6	5.9	19.6	70	40.6	40.6	0.999	6.1	2.47	1750	28
3	6.5	1117.2	640.9	1121.0	2.327	2.478	80.2	14.7	145.2	6.1	19.8	69	40.7	40.6	1.003	6.1	2.49	1850	35
Avg.					2.342	2.478	80.7	14.8	146.1	5.5	19.3	72	40.2	40.6	0.989	6.1	2.44	1750	25
1	7.0	1097.5	641.3	1098.9	2.398	2.459	82.2	16.3	149.7	2.5	17.8	86	39.3	40.6	0.966	6.6	2.35	1800	20
2	7.0	1102.5	639.1	1104.0	2.371	2.459	81.3	16.1	148.0	3.6	18.7	81	39.9	40.6	0.983	6.6	2.40	1925	27
3	7.0	1107.6	644.7	1108.8	2.387	2.459	81.8	16.3	148.9	2.9	18.2	84	39.5	40.6	0.973	6.6	2.39	2200	22
Avg.					2.385	2.459	81.8	16.2	148.9	3.0	18.2	84	39.6	40.6	0.974	6.6	2.38	1975	23
1	7.0	1097.6	631.2	1099.8	2.342	2.459	80.3	15.9	146.2	4.7	19.7	76	40.7	40.6	1.001	6.6			
2	7.0	1105.6	639.2	1107.5	2.361	2.459	80.9	16.1	147.3	4.0	19.1	79	40.2	40.6	0.989	6.6			
Avg.					2.352	2.459	80.6	16.0	146.7	4.4	19.4	77	40.4	40.6	0.995	6.6			
1	8.0	1116.9	645.5	1118.8	2.360	2.422	80.0	18.4	147.3	2.6	20.0	87	40.9	40.6	1.006	7.6			
2	8.0	1107.7	638.7	1109.5	2.353	2.422	79.8	18.3	146.8	2.9	20.2	86	41.0	40.6	1.010	7.6			
Avg.					2.356	2.422	79.9	18.3	147.0	2.7	20.1	87	41.0	40.6	1.008	7.6			

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
6.0	6.3	18.9	67	40.1	0.987	146.0	5.6
6.5	5.5	19.3	72	40.2	0.989	146.1	6.1
7.0	4.4	19.4	77	40.4	0.995	146.7	6.6
8.0	2.7	20.1	87	41.0	1.008	147.0	7.6
7.3	4.0	19.6	80	40.6	0.999	146.7	6.8
7.8	3.0	20.0	85	40.8	1.005	147.0	7.4

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
007	2.731	70.5
89	2.705	8
M-10's	2.722	12
Fly Ash	2.615	8.5
Lime	2.350	1
		100

Comb. Gsb = 2.713

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
007	2.778	70.5
89	2.769	8
M-10's	2.741	12
Fly Ash	2.718	8.5
Lime	2.350	1
		100

Comb. Gsa = 2.763

TABLE A12. Ruby Granite SMA Mix Design Summary, 50 Gyrations

Project:	AAPTP 04-04 SMA for Airfields		Aggregate Type:	Ruby	Percent Retained on #4 sieve:	74.3
AC Sp. Gr. (Gb) = 1.028	App. Sp. Gr. (Gsa) 2.763	Eff. Sp. Gr. (Gse): 2.747	Trial Blend:	8-B	Bulk Sp. Gravity of CA:	2.728
		Bulk Sp. Gr. (Gsb): 2.713	NMAS:	12.5	Density of CA in DRC (kg/m ^{3}):	1616.4
			% Passing #4 Sieve:	25.7	Compactive Effort:	50 gyrations
Date	8/28/2008					

		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS						
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %
1	6.5	4762.5	2772.6	4775.0	2.378	2.478	82.0	15.0	148.4	4.0	18.0	78	39.4	40.6	0.971	6.1
2	6.5	4727.3	2744.8	4745.0	2.363	2.478	81.5	14.9	147.5	4.6	18.5	75	39.8	40.6	0.980	6.1
3	6.5	4736.6	2743.5	4750.5	2.360	2.478	81.3	14.9	147.3	4.8	18.7	74	39.9	40.6	0.982	6.1
Avg.					2.367	2.478	81.6	15.0	147.7	4.5	18.4	76	39.7	40.6	0.978	6.1
1	7.0	4757.6	2761.9	4770.1	2.369	2.459	81.2	16.1	147.8	3.7	18.8	81	40.0	40.6	0.984	6.6
2	7.0	4728.0	2718.3	4744.5	2.333	2.459	80.0	15.9	145.6	5.1	20.0	74	40.9	40.6	1.007	6.6
3	7.0	4801.3	2767.0	4811.0	2.349	2.459	80.5	16.0	146.6	4.5	19.5	77	40.5	40.6	0.997	6.3
Avg.					2.350	2.459	80.6	16.0	146.7	4.4	19.4	77	40.5	40.6	0.996	6.5
1	8.0	4823.7	2762.4	4836.8	2.325	2.422	78.9	18.1	145.1	4.0	21.1	81	41.7	40.6	1.027	7.6
2	8.0	4823.9	2772.7	4833.8	2.340	2.422	79.4	18.2	146.0	3.4	20.6	84	41.4	40.6	1.018	7.6
3	8.0	4805.3	2743.8	4817.7	2.317	2.422	78.6	18.0	144.6	4.3	21.4	80	41.9	40.6	1.032	7.6
Avg.					2.328	2.422	78.9	18.1	145.2	3.9	21.1	82	41.7	40.6	1.026	7.6
1	6.0	4772.9	2761.7	4788.6	2.355	2.497	81.6	13.7	146.9	5.7	18.4	69	39.7	40.6	0.977	5.6
2	6.0	4763.8	2748.7	4780.4	2.345	2.497	81.2	13.7	146.3	6.1	18.8	67	40.0	40.6	0.984	5.6
3	6.0	4788.8	2770.1	4804.2	2.354	2.497	81.6	13.7	146.9	5.7	18.4	69	39.7	40.6	0.978	5.6
Avg.					2.351	2.497	81.5	13.7	146.7	5.8	18.5	69	39.8	40.6	0.981	5.6

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
6.0	5.8	18.5	68.5	39.8	0.981	146.7	5.6
6.5	4.5	18.4	75.8	39.7	0.978	147.7	6.1
7.0	4.4	19.4	77.4	40.5	0.996	146.7	6.5
8.0	3.9	21.1	81.5	41.7	1.026	145.2	7.6
8.4	3.0	21.4	84.6	41.9	1.031	145.2	7.9
7.5	4.0	20.2	79.3	41.0	1.009	146.0	7.0

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
007	2.731	70.5
89	2.705	8
M-10's	2.722	12
Fly Ash	2.615	8.5
Lime	2.350	1
		100

Comb. Gsb = 2.713

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
007	2.778	70.5
89	2.769	8
M-10's	2.741	12
Fly Ash	2.718	8.5
Lime	2.350	1
		100

Comb. Gsa = 2.763

TABLE A13. Ruby Granite SMA Mix Design Summary, 65 Gyrations

Project:	AAPTP 04-04 SMA for Airfields		Aggregate Type:	Ruby	Percent Retained on #4 sieve:	74.3
AC Sp. Gr. (Gb) = 1.028	App. Sp. Gr. (Gsa) 2.763	Eff. Sp. Gr. (Gse): 2.747	Trial Blend:	8-B	Bulk Sp. Gravity of CA:	2.728
		Bulk Sp. Gr. (Gsb): 2.713	NMAS:	12.5	Density of CA in DRC (kg/m^3):	1616.4
			% Passing #4 Sieve:	25.7	Compactive Effort:	65 Gyrations
Date	8/28/2008					

Specimen Number	Asphalt Content	Masses: In Air (gms)	Masses: In Water (gms)	Masses: SSD (gms)	Specific Gravities: Bulk (Gmb)	Specific Gravities: TMD (Gmm)	Volumes at Ndes: Aggregate Volume cc	Volumes at Ndes: AC by Volume %	Unit Weight, pcf	Voids: VTM, %	Voids: VMA, %	Voids: VFA, %	Voids: VCAmix, %	Voids: VCAdrc, %	Voids: VCAmix / VCAdrc	Voids: Eff. AC Content %
1	7.0	4798.3	2767.1	4805.5	2.354	2.459	80.7	16.0	146.9	4.3	19.3	78	40.4	40.6	0.994	6.6
2	7.0	4812.3	2766.4	4820.1	2.343	2.459	80.3	16.0	146.2	4.7	19.7	76	40.6	40.6	1.000	6.6
Avg.					2.349	2.459	80.5	16.0	146.6	4.5	19.5	77	40.5	40.6	0.997	6.6
1	7.8	4811.9	2765.6	4819.3	2.343	2.429	79.6	17.8	146.2	3.5	20.4	83	41.2	40.6	1.013	7.4
2	7.8	4813.6	2769.7	4818.6	2.349	2.429	79.8	17.8	146.6	3.3	20.2	84	41.0	40.6	1.009	7.4
Avg.					2.346	2.429	79.7	17.8	146.4	3.4	20.3	83	41.1	40.6	1.011	7.4

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
7.0	4.5	19.5	77	40.5	0.997	146.6	6.6
7.8	3.4	20.3	83	41.1	1.011	146.4	7.4
8.1	3.0	20.6	86	41.3	1.016	146.3	7.7
7.4	4.0	19.8	80	40.8	1.003	146.5	6.9

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
007	2.731	70.5
89	2.705	8
M-10's	2.722	12
Fly Ash	2.615	8.5
Lime	2.350	1
		100
Comb. Gsb =		2.713

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
007	2.778	70.5
89	2.769	8
M-10's	2.741	12
Fly Ash	2.718	8.5
Lime	2.350	1
		100
Comb. Gsa =		2.763

TABLE A14. Ruby Granite SMA Mix Design Summary, 80 Gyrations

Project:	AAPTP 04-04 SMA for Airfields			Aggregate Type:	Ruby	Percent Retained on #4 sieve:	74.3	Date
AC Sp. Gr. (Gb) =	App. Sp. Gr. (Gsa)	Eff. Sp. Gr. (Gse):	Bulk Sp. Gr. (Gsb):	Trial Blend:	8-B	Bulk Sp. Gravity of CA:	2.728	8/28/2008
	2.763	2.747	2.713	NMAS:	12.5	Density of CA in DRC (kg/m ^{3}):	1616.4	
1.028				% Passing #4 Sieve:	25.7	Compactive Effort:	80 gyrations	

		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS						
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %
1	6.25	4767.1	2782.0	4777.0	2.390	2.488	82.6	14.5	149.1	4.0	17.4	77	39.0	40.6	0.960	5.8
2	6.25	4756.3	2775.6	4765.7	2.390	2.488	82.6	14.5	149.1	3.9	17.4	77	39.0	40.6	0.959	5.8
Avg.					2.390	2.488	82.6	14.5	149.1	3.9	17.4	77	39.0	40.6	0.959	5.8
1	6.5	4756.8	2767.3	4766.6	2.379	2.478	82.0	15.0	148.5	4.0	18.0	78	39.4	40.6	0.970	6.1
2	6.5	4752.6	2766.5	4757.8	2.387	2.478	82.3	15.1	148.9	3.7	17.7	79	39.2	40.6	0.965	6.1
3	6.5	4758.5	2766.5	4770.1	2.375	2.478	81.9	15.0	148.2	4.2	18.1	77	39.5	40.6	0.973	6.1
Avg.					2.380	2.478	82.0	15.1	148.5	3.9	18.0	78	39.4	40.6	0.969	6.1
1	7.0	4789.8	2763.8	4798.8	2.354	2.459	80.7	16.0	146.9	4.3	19.3	78	40.4	40.6	0.994	6.6
2	7.0	4795.5	2786.5	4804.7	2.376	2.459	81.5	16.2	148.3	3.4	18.5	82	39.8	40.6	0.980	6.6
3	7.0	4789.6	2762.2	4801.0	2.349	2.459	80.5	16.0	146.6	4.5	19.5	77	40.5	40.6	0.997	6.6
Avg.					2.360	2.459	80.9	16.1	147.2	4.0	19.1	79	40.2	40.6	0.990	6.6
1	7.5	4791.7	2766.1	4797.9	2.358	2.440	80.4	17.2	147.2	3.3	19.6	83	40.6	40.6	0.999	7.1
2	7.5	4786.0	2737.5	4800.9	2.319	2.440	79.1	16.9	144.7	4.9	20.9	76	41.6	40.6	1.023	7.1
3	7.5	4801.6	2787.3	4808.5	2.376	2.440	81.0	17.3	148.2	2.6	19.0	86	40.2	40.6	0.988	7.1
Avg.					2.351	2.440	80.2	17.2	146.7	3.6	19.8	82	40.8	40.6	1.003	7.1

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
7.0	4.0	19.1	79	40.2	0.990	147.2	6.6
7.5	3.6	19.8	82	40.8	1.003	146.7	7.1
8.3	3.0	21.0	86	41.6	1.025	145.9	7.9

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
007	2.731	70.5
89	2.705	8
M-10's	2.722	12
Fly Ash	2.615	8.5
Lime	2.350	1
		100
Comb. Gsb =		2.713

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
007	2.778	70.5
89	2.769	8
M-10's	2.741	12
Fly Ash	2.718	8.5
Lime	2.350	1
		100
Comb. Gsa =		2.763

TABLE A15. Ruby Granite SMA Mix Design Summary, 100 Gyrations

Project:	AAPTP 04-04 SMA for Airfields			
AC Sp. Gr. (Gb) = 1.028	App. Sp. Gr. (Gsa) 2.763	Eff. Sp. Gr. (Gse): 2.747	Bulk Sp. Gr. (Gsb): 2.713	
Aggregate Type: Ruby	Trial Blend: 8-B	NMAS: 12.5	% Passing #4 Sieve: 25.7	
Percent Retained on #4 sieve: 74.3	Bulk Sp. Gravity of CA: 2.728	Density of CA in DRC (kg/m ^{3}): 1616.4	Compactive Effort: 100 Gyrations	Date 8/28/2008

		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS						
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %
1	5.5	4672.0	2709.0	4686.5	2.363	2.516	82.3	12.6	147.4	6.1	17.7	66	39.2	40.6	0.965	5.1
2	5.5	4709.9	2730.7	4728.7	2.357	2.516	82.1	12.6	147.1	6.3	17.9	65	39.3	40.6	0.968	5.1
Avg.					2.360	2.516	82.2	12.6	147.3	6.2	17.8	65	39.3	40.6	0.966	5.1
1	6.0	4694.6	2724.7	4708.2	2.367	2.497	82.0	13.8	147.7	5.2	18.0	71	39.4	40.6	0.970	5.6
2	6.0	4756.1	2770.4	4762.9	2.387	2.497	82.7	13.9	148.9	4.4	17.3	75	38.9	40.6	0.957	5.6
Avg.					2.377	2.497	82.4	13.9	148.3	4.8	17.6	73	39.1	40.6	0.964	5.6
1	6.5	4770.6	2778.3	4777.7	2.386	2.478	82.2	15.1	148.9	3.7	17.8	79	39.2	40.6	0.966	6.1
2	6.5	4786.1	2767.4	4794.2	2.361	2.478	81.4	14.9	147.4	4.7	18.6	75	39.9	40.6	0.981	6.1
Avg.					2.374	2.478	81.8	15.0	148.1	4.2	18.2	77	39.6	40.6	0.973	6.1
1	7.0	4800.8	2781.6	4803.6	2.374	2.459	81.4	16.2	148.2	3.4	18.6	81	39.9	40.6	0.981	6.6
2	7.0	4818.4	2792.7	4820.7	2.376	2.459	81.4	16.2	148.3	3.4	18.6	82	39.8	40.6	0.980	6.6
Avg.					2.375	2.459	81.4	16.2	148.2	3.4	18.6	82	39.8	40.6	0.981	6.6

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
5.5	6.2	17.8	65	39.3	0.966	147.3	5.1
6.0	4.8	17.6	73	39.1	0.964	148.3	5.6
6.5	4.2	18.2	77	39.6	0.973	148.1	6.1
7.0	3.4	18.6	82	39.8	0.981	148.2	6.6
7.2	3.0	18.6	84	39.8	0.981	148.5	6.7
6.6	4.0	18.3	78	39.6	0.975	148.2	6.2

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
007	2.731	70.5
89	2.705	8
M-10's	2.722	12
Fly Ash	2.615	8.5
Lime	2.350	1
		100

Comb. Gsb = 2.713

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
007	2.778	70.5
89	2.769	8
M-10's	2.741	12
Fly Ash	2.718	8.5
Lime	2.350	1
		100

Comb. Gsa = 2.763

TABLE A16. Gravel SMA Mix Design Summary, 50 Blow Marshall

Project:	AAPTP 04-04 SMA for Airfields			Aggregate Type:	Gravel	Percent Retained on #4 sieve: 71.8	Date
AC Sp. Gr. (Gb) = 1.028	App. Sp. Gr. (Gsa) 2.630	Eff. Sp. Gr. (Gse): 2.630	Bulk Sp. Gr. (Gsb): 2.589	Trial Blend:	1	Bulk Sp. Gravity of CA: 2.598	8/25/2008
				NMAS:	12.5	Density of CA in DRC (kg/m³): 1498.9	
				% Passing #4 Sieve:	28.2	Compactive Effort: 50 blow Marshall	

Specimen Number	Asphalt Content	Masses In Air (gms)	Masses In Water (gms)	Masses SSD (gms)	Specific Gravities Bulk (Gmb)	Specific Gravities TMD (Gmm)	Volumes at Ndes Aggregate Volume cc	Volumes at Ndes AC by Volume %	Unit Weight, pcf	Voids VTM, %	Voids VMA, %	Voids VFA, %	Voids VCAmix, %	Voids VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %	Height, in	Stability, lbs	Flow, 0.01 in
1	7.0	1218.0	681.3	1224.1	2.244	2.372	80.6	15.3	140.0	5.4	19.4	72	42.3	42.2	1.003	6.4	2.74	2100	34.0
2	7.0	1221.1	683.6	1228.3	2.242	2.372	80.5	15.3	139.9	5.5	19.5	72	42.4	42.2	1.005	6.4	2.76	1750	33.0
3	7.0	1224.1	681.7	1235.2	2.212	2.372	79.4	15.1	138.0	6.8	20.6	67	43.2	42.2	1.023	6.4	2.82	1850	35.0
Avg.					2.232	2.372	80.2	15.2	139.3	5.9	19.8	70	42.4	42.2	1.004	6.4	2.77	1900	34.0
1	7.5	1172.0	658.2	1176.2	2.263	2.355	80.8	16.5	141.2	3.9	19.2	80	42.2	42.2	0.999	6.9	2.62	2050	30.5
2	7.5	1166.2	655.8	1171.5	2.261	2.355	80.8	16.5	141.1	4.0	19.2	79	42.2	42.2	1.000	6.9	2.56	2150	26.0
3	7.5	1171.0	655.1	1176.2	2.247	2.355	80.3	16.4	140.2	4.6	19.7	77	42.6	42.2	1.009	6.9	2.61	2100	29.0
Avg.					2.257	2.355	80.6	16.5	140.8	4.2	19.4	79	42.2	42.2	1.000	6.9	2.60	2100	28.5
1	8.0	1182.5	664.9	1186.0	2.269	2.339	80.6	17.7	141.6	3.0	19.4	85	42.3	42.2	1.003	7.4	2.62	1500	28.5
2	8.0	1180.8	666.7	1183.3	2.286	2.339	81.2	17.8	142.6	2.3	18.8	88	41.9	42.2	0.993	7.4	2.60	1300	25.5
3	8.0	1172.9	655.0	1176.6	2.249	2.339	79.9	17.5	140.3	3.9	20.1	81	42.8	42.2	1.015	7.4	2.66	1800	28.0
Avg.					2.268	2.339	80.6	17.6	141.5	3.0	19.4	84	42.1	42.2	0.998	7.4	2.63	1533	27.3

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
7.0	5.9	19.8	70	42.4	1.004	139.3	6.4
7.5	4.2	19.4	79	42.2	1.000	140.8	6.9
8.0	3.0	19.4	84	42.1	0.998	141.5	7.4
7.6	4.0	19.5	79	42.2	1.000	140.7	7.0

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.598	79.6
Fines	2.518	16.4
Fly Ash	2.615	0
Mineral Filler	2.718	4
Lime	2.350	0
		100

Comb. Gsb = 2.589

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.641	79.6
Fines	2.555	16.4
Fly Ash	2.718	0
Mineral Filler	2.718	4
Lime	2.350	0
		100

Comb. Gsa = 2.6295

TABLE A17. Gravel SMA Mix Design Summary, 50 Gyrations

Project:	AAPTP 04-04 SMA for Airfields			
AC Sp. Gr. (Gb) = 1.028	App. Sp. Gr. (Gsa) 2.645	Eff. Sp. Gr. (Gse): 2.630	Bulk Sp. Gr. (Gsb): 2.603	
Aggregate Type: Gravel	Trial Blend: 1	NMAS: 12.5	% Passing #4 Sieve: 28.2	
Percent Retained on #4 sieve: 71.8	Bulk Sp. Gravity of CA: 2.598	Density of CA in DRC (kg/m^3): 1498.9	Compactive Effort: 50 Gyrations	Date 8/25/2008

		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS						
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %
1	6.7	4709.0	2662.8	4725.5	2.283	2.383	81.8	14.9	142.5	4.2	18.2	77	41.1	42.2	0.975	6.3
2	6.7	4685.6	2648.3	4702.3	2.281	2.383	81.8	14.9	142.3	4.3	18.2	77	41.2	42.2	0.976	6.3
Avg.					2.282	2.383	81.8	14.9	142.4	4.2	18.2	77	41.2	42.2	0.976	6.3
1	7.2	4697.0	2658.0	4706.5	2.293	2.365	81.7	16.1	143.1	3.0	18.3	83	41.2	42.2	0.976	6.8
2	7.2	4724.3	2673.2	4733.6	2.293	2.365	81.7	16.1	143.1	3.0	18.3	83	41.2	42.2	0.976	6.8
Avg.					2.293	2.365	81.7	16.1	143.1	3.0	18.3	83	41.2	42.2	0.976	6.8
1	7.5	4827.8	2725.3	4834.4	2.289	2.354	81.3	16.7	142.8	2.8	18.7	85	41.5	42.2	0.983	7.1
2	7.5	4536.4	2565.6	4544.3	2.293	2.354	81.5	16.7	143.1	2.6	18.5	86	41.4	42.2	0.981	7.1
Avg.					2.291	2.354	81.4	16.7	142.9	2.7	18.6	86	41.4	42.2	0.982	7.1
1	7.8	4725.5	2671.9	4732.7	2.293	2.342	81.2	17.4	143.1	2.1	18.8	89	41.6	42.2	0.985	7.4
2	7.8	4719.4	2669.1	4725.7	2.295	2.342	81.3	17.4	143.2	2.0	18.7	89	41.5	42.2	0.984	7.4
Avg.					2.294	2.342	81.3	17.4	143.1	2.1	18.7	89	41.5	42.2	0.985	7.4

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
6.7	4.2	18.2	77	41.2	0.976	142.4	6.3
7.5	2.7	18.6	86	41.4	0.982	142.9	7.1
7.8	2.1	18.7	89	41.5	0.985	143.1	7.4
7.2	3.0	18.3	83	41.2	0.976	143.1	6.8
6.8	4.0	18.2	78	41.2	0.976	142.5	6.4

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.598	79.6
Fines	2.603	16.4
Fly Ash	2.615	0
Mineral Filler	2.718	4
Lime	2.350	0
		100

Comb. Gsb = 2.603

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.641	79.6
Fines	2.645	16.4
Fly Ash	2.718	0
Mineral Filler	2.718	4
Lime	2.350	0
		100

Comb. Gsa = 2.645

TABLE A18. Gravel SMA Mix Design Summary, 65 Gyrations

Project:	AAPTP 04-04 SMA for Airfields									Aggregate Type:	Gravel		Percent Retained on #4 sieve:		71.8		Date
AC Sp. Gr. (Gb) = 1.028		App. Sp. Gr. (Gsa) 2.645		Eff. Sp. Gr. (Gse): 2.630		Bulk Sp. Gr. (Gsb): 2.603				Trial Blend:	1		Bulk Sp. Gravity of CA:		2.598		8/25/2008
										NMAS:	12.5		Density of CA in DRC (kg/m ^{3}):		1498.9		
										% Passing #4 Sieve:	28.2		Compactive Effort:		65 Gyrations		

Specimen Number	Asphalt Content	Masses: In Air (gms)	Masses: In Water (gms)	Masses: SSD (gms)	Specific Gravities: Bulk (Gmb)	Specific Gravities: TMD (Gmm)	Volumes at Ndes: Aggregate Volume cc	Volumes at Ndes: AC by Volume %	Unit Weight, pcf	Voids: VTM, %	Voids: VMA, %	Voids: VFA, %	Voids: VCAmix, %	Voids: VCAdrc, %	Voids: VCAmix/VCAdrc	Eff. AC Content %
1	6.5	4701.9	2667.0	4715.1	2.296	2.390	82.5	14.5	143.3	3.9	17.5	78	40.7	42.2	0.964	6.1
2	6.5	4702.7	2672.2	4714.4	2.303	2.390	82.7	14.6	143.7	3.7	17.3	79	40.5	42.2	0.960	6.1
Avg.					2.299	2.390	82.6	14.5	143.5	3.8	17.4	78	40.6	42.2	0.962	6.1
1	7.0	4674.9	2654.9	4684.2	2.304	2.372	82.3	15.7	143.8	2.9	17.7	84	40.8	42.2	0.967	6.6
3	7.0	4690.8	2659.6	4700.5	2.298	2.372	82.1	15.7	143.4	3.1	17.9	83	40.9	42.2	0.970	6.6
Avg.					2.301	2.372	82.2	15.7	143.6	3.0	17.8	83	40.9	42.2	0.968	6.6
1	7.2	4670.2	2659.3	4681.9	2.309	2.367	82.3	16.2	144.1	2.5	17.7	86	40.8	42.2	0.967	6.8
2	7.2	4676.1	2651.1	4683.2	2.301	2.367	82.0	16.1	143.6	2.8	18.0	85	41.0	42.2	0.971	6.8
Avg.					2.305	2.367	82.2	16.1	143.8	2.6	17.8	85	40.9	42.2	0.969	6.8
1	7.5	4720.5	2680.5	4726.8	2.307	2.354	82.0	16.8	143.9	2.0	18.0	89	41.0	42.2	0.972	7.1
2	7.5	4684.5	2658.9	4691.0	2.305	2.354	81.9	16.8	143.8	2.1	18.1	89	41.1	42.2	0.973	7.1
3	7.5	4765.4	2699.7	4771.5	2.300	2.354	81.7	16.8	143.5	2.3	18.3	87	41.2	42.2	0.977	7.1
4	7.5	4737.9	2687.6	4742.2	2.306	2.354	81.9	16.8	143.9	2.0	18.1	89	41.0	42.2	0.973	7.1
Avg.					2.305	2.354	81.9	16.8	143.8	2.1	18.1	88	41.1	42.2	0.974	7.1

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
6.5	3.8	17.4	78	40.6	0.962	143.5	6.1
7.0	3.0	17.8	83	40.9	0.968	143.6	6.6
7.2	2.6	17.8	85	40.9	0.969	143.8	6.8
7.5	2.1	18.1	88	41.1	0.974	143.8	7.1
6.4	4.0	17.3	77	40.5	0.960	143.4	6.0

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.598	79.6
Fines	2.603	16.4
Fly Ash	2.615	0
Mineral Filler	2.718	4
Lime	2.350	0
		100
Comb. Gsb =		2.603

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.641	79.6
Fines	2.645	16.4
Fly Ash	2.718	0
Mineral Filler	2.718	4
Lime	2.350	0
		100
Comb. Gsa =		2.6447

TABLE A19. Gravel SMA Mix Design Summary, 80 Gyrations

Project:	AAPTP 04-04 SMA for Airfields			Aggregate Type:	Gravel	Percent Retained on #4 sieve:	71.8
	App. Sp. Gr. (Gsa)	Eff. Sp. Gr. (Gse):	Bulk Sp. Gr. (Gsb):	Trial Blend:	1	Bulk Sp. Gravity of CA:	2.598
AC Sp. Gr. (Gb) =	2.645	2.630	2.603	NMAS:	12.5	Density of CA in DRC (kg/m ^{3}):	1498.9
1.028				% Passing #4 Sieve:	28.2	Compactive Effort:	80 Gyrations

Date: 8/25/2008

		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS						
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %
1	6.3	4679.7	2661.8	4689.2	2.308	2.399	83.1	14.1	144.0	3.8	16.9	78	40.2	42.2	0.953	5.9
2	6.3	4679.8	2664.0	4688.4	2.312	2.399	83.2	14.2	144.2	3.6	16.8	78	40.1	42.2	0.951	5.9
Avg.					2.310	2.399	83.2	14.2	144.1	3.7	16.8	78	40.2	42.2	0.952	5.9
1	6.5	4802.2	2731.1	4818.5	2.301	2.390	82.6	14.5	143.6	3.7	17.4	78	40.6	42.2	0.961	6.1
2	6.5	4812.3	2743.7	4821.6	2.316	2.390	83.2	14.6	144.5	3.1	16.8	82	40.2	42.2	0.952	6.1
3	6.5	4689.5	2673.0	4698.2	2.316	2.390	83.2	14.6	144.5	3.1	16.8	81	40.2	42.2	0.952	6.1
4	6.5	4672.5	2657.7	4687.2	2.302	2.390	82.7	14.6	143.7	3.7	17.3	79	40.5	42.2	0.960	6.1
Avg.					2.309	2.390	82.9	14.6	144.1	3.4	17.1	80	40.3	42.2	0.956	6.1
1	6.7	4670.2	2659.3	4681.9	2.309	2.382	82.8	15.0	144.1	3.1	17.2	82	40.5	42.2	0.959	6.3
2	6.7	4676.1	2651.1	4683.2	2.301	2.382	82.5	15.0	143.6	3.4	17.5	81	40.7	42.2	0.964	6.3
Avg.					2.305	2.382	82.6	15.0	143.8	3.2	17.4	81	40.6	42.2	0.961	6.3
1	7.0	4815.0	2742.3	4822.2	2.315	2.372	82.7	15.8	144.5	2.4	17.3	86	40.5	42.2	0.960	6.6
2	7.0	4810.6	2742.9	4817.2	2.319	2.372	82.9	15.8	144.7	2.2	17.1	87	40.4	42.2	0.957	6.6
Avg.					2.317	2.372	82.8	15.8	144.6	2.3	17.2	87	40.4	42.2	0.959	6.6

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
6.3	3.7	16.8	78	40.2	0.952	144.1	5.9
6.5	3.4	17.1	80	40.3	0.956	144.1	6.1
6.7	3.2	17.4	81	40.6	0.961	143.8	6.3
7.0	2.3	17.2	87	40.4	0.959	144.6	6.6
6.8	3.0	17.3	83	40.5	0.961	144.0	6.4
6.1	4.0	16.6	76	40.0	0.949	144.2	5.7

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.598	79.6
Fines	2.603	16.4
Fly Ash	2.615	0
Mineral Filler	2.718	4
Lime	2.350	0
		100

Comb. Gsb = 2.603

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.641	79.6
Fines	2.645	16.4
Fly Ash	2.718	0
Mineral Filler	2.718	4
Lime	2.350	0
		100

Comb. Gsa = 2.6447

TABLE A20. Gravel SMA Mix Design Summary, 100 Gyrations

Project:	AAPTP 04-04 SMA for Airfields									Aggregate Type:	Gravel		Percent Retained on #4 sieve:		71.8		Date
AC Sp. Gr. (Gb) = 1.028		App. Sp. Gr. (Gsa) 2.645		Eff. Sp. Gr. (Gse): 2.630		Bulk Sp. Gr. (Gsb): 2.603				Trial Blend:	1		Bulk Sp. Gravity of CA:		2.598		8/25/2008
										NMAS:	12.5		Density of CA in DRC (kg/m ^{3}):		1498.9		
										% Passing #4 Sieve:	28.2		Compactive Effort:		100 Gyrations		
		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes				VOIDS						
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %	
1	6.0	4664.9	2662.5	4675.5	2.317	2.406	83.7	13.5	144.6	3.7	16.3	77	39.8	42.2	0.943	5.6	
3	6.0	4681.8	2665.8	4696.1	2.306	2.406	83.3	13.5	143.9	4.2	16.7	75	40.1	42.2	0.950	5.6	
2	6.0	4756.6	2717.7	4767.8	2.320	2.406	83.8	13.5	144.8	3.6	16.2	78	39.7	42.2	0.942	5.6	
Avg.	Avg.				2.312	2.406	83.5	13.5	144.2	3.9	16.5	76	39.9	42.2	0.945	5.6	
1	6.5	4796.8	2737.4	4806.8	2.318	2.389	83.3	14.7	144.6	3.0	16.7	82	40.1	42.2	0.951	6.1	
2	6.5	4779.8	2730.4	4786.0	2.325	2.389	83.5	14.7	145.1	2.7	16.5	84	39.9	42.2	0.946	6.1	
Avg.	Avg.				2.322	2.389	83.4	14.7	144.9	2.8	16.6	83	40.0	42.2	0.948	6.1	

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
6.0	3.9	16.5	76	39.9	0.945	144.2	5.6
6.5	2.8	16.6	83	40.0	0.948	144.9	6.1
6.4	3.0	16.6	82	40.0	0.948	144.8	6.0

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.598	79.6
Fines	2.603	16.4
Fly Ash	2.615	0
Mineral Filler	2.718	4
Lime	2.350	0
		100
Comb. Gsb =		2.603

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.641	79.6
Fines	2.645	16.4
Fly Ash	2.718	0
Mineral Filler	2.718	4
Lime	2.350	0
		100
Comb. Gsa =		2.6447

TABLE A21. Limestone SMA Mix Design Summary, PG 76-22, 50 Blow Marshall

Project:	AAPTP 04-04 SMA for Airfields		Aggregate Type:	Limestone	Percent Retained on #4 sieve:	77.1
AC Sp. Gr. (Gb) = 1.028	App. Sp. Gr. (Gsa) 2.749	Eff. Sp. Gr. (Gse): 2.739	Bulk Sp. Gr. (Gsb): 2.720	Trial Blend: 4	Bulk Sp. Gravity of CA:	2.731
			NMAS:	12.5	Density of CA in DRC (kg/m^3):	1548.6
			% Passing #4 Sieve:	22.9	Compactive Effort:	50 blow Marshall
Date	8/25/2008					

Specimen Number	Asphalt Content	Masses: In Air (gms)	Masses: In Water (gms)	Masses: SSD (gms)	Specific Gravities: Bulk (Gmb)	Specific Gravities: TMD (Gmm)	Volumes at Ndes: Aggregate Volume cc	Volumes at Ndes: AC by Volume %	Unit Weight, pcf	Voids: VTM, %	Voids: VMA, %	Voids: VFA, %	Voids: VCAmix, %	Voids: VCAdrc, %	Voids: VCAmix / VCAdrc	Eff. AC Content %	Height, in	Stability, lbs	Flow, 0.01 in
1	6.5	1123.2	649.2	1127.2	2.350	2.472	80.8	14.9	146.6	4.9	19.2	74	38.0	43.2	0.879	6.3	2.44	1400	26.5
2	6.5	1132.1	655.8	1136.6	2.355	2.472	80.9	14.9	146.9	4.7	19.1	75	37.8	43.2	0.876	6.3	2.46	1350	26.5
3	6.5	1134.6	655.9	1138.3	2.352	2.472	80.8	14.9	146.8	4.9	19.2	75	37.9	43.2	0.878	6.3	2.46	1400	26.5
Avg.					2.352	2.472	80.9	14.9	146.8	4.8	19.1	75	37.9	43.2	0.878	6.3	2.45	1383	26.5
1	7.5	1127.5	654.9	1129.0	2.378	2.435	80.9	17.4	148.4	2.3	19.1	88	37.9	43.2	0.878	7.3	2.47	1650	20.5
2	7.5	1121.1	647.0	1122.0	2.360	2.435	80.3	17.2	147.3	3.1	19.7	84	38.4	43.2	0.888	7.3	2.45	1600	21.5
3	7.5	1132.6	654.0	1134.4	2.358	2.435	80.2	17.2	147.1	3.2	19.8	84	38.4	43.2	0.890	7.3	2.46	1600	19.0
Avg.					2.365	2.435	80.4	17.3	147.6	2.9	19.6	85	38.2	43.2	0.885	7.3	2.46	1617	20.3

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
6.5	4.8	19.1	75	37.9	0.878	146.8	6.3
7.5	2.9	19.6	85	38.2	0.885	147.6	7.3
7.4	3.0	19.5	85	38.2	0.885	147.5	7.2
6.9	4.0	19.3	79	38.0	0.881	147.1	6.7

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.735	65
7's	2.719	25
89's	2.714	0
820's	2.602	4
Mineral Filler	2.718	5
Lime	2.350	1
		100

Comb. Gsb = 2.720

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.759	65
7's	2.748	25
89's	2.752	0
820's	2.747	4
Mineral Filler	2.718	5
Lime	2.350	1
		100

Comb. Gsa = 2.749

TABLE A22. Limestone SMA Mix Design Summary, PG 76-22, 50 Gyrations

Project:	AAPTP 04-04 SMA for Airfields								Aggregate Type:	Limestone		Percent Retained on #4 sieve:		77.1		Date
AC Sp. Gr. (Gb) = 1.028		App. Sp. Gr. (Gsa) 2.749		Eff. Sp. Gr. (Gse): 2.739		Bulk Sp. Gr. (Gsb): 2.720			Trial Blend:	4		Bulk Sp. Gravity of CA:		2.731		8/25/2008
									NMAS:	12.5		Density of CA in DRC (kg/m ^{3}):		1548.6		
									% Passing #4 Sieve:	22.9		Compactive Effort:		50 Gyrations		
		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS						
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %
1	7.5	4771.2	2742.7	4776.7	2.346	2.435	79.8	17.1	146.4	3.7	20.2	82	38.7	43.2	0.897	7.3
2	7.5	4742.8	2725.4	4754.0	2.338	2.435	79.5	17.1	145.9	4.0	20.5	81	38.9	43.2	0.902	7.3
3	7.5	4773.0	2757.3	4782.5	2.357	2.435	80.1	17.2	147.1	3.2	19.9	84	38.5	43.2	0.891	7.3
Avg.					2.347	2.435	79.8	17.1	146.4	3.6	20.2	82	38.7	43.2	0.897	7.3
1	7.8	4815.8	2776.8	4823.9	2.352	2.424	79.7	17.8	146.8	2.9	20.3	85	38.8	43.2	0.898	7.6
2	7.8	4831.0	2788.3	4837.5	2.358	2.424	79.9	17.9	147.1	2.7	20.1	86	38.6	43.2	0.895	7.6
3	7.8	4821.9	2773.8	4828.1	2.347	2.424	79.6	17.8	146.5	3.2	20.4	85	38.9	43.2	0.901	7.6
Avg.					2.352	2.424	79.7	17.8	146.8	3.0	20.3	85	38.8	43.2	0.898	7.6

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
7.5	3.6	20.2	82	38.7	0.897	146.4	7.3
7.8	3.0	20.3	85	38.8	0.898	146.8	7.6
7.3	4.0	20.2	80	38.7	0.896	146.2	7.1

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.735	65
7's	2.719	25
89's	2.714	0
820's	2.602	4
Mineral Filler	2.718	5
Lime	2.350	1
		100
Comb. Gsb =		2.720

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.759	65
7's	2.748	25
89's	2.752	0
820's	2.747	4
Mineral Filler	2.718	5
Lime	2.350	1
		100
Comb. Gsa =		2.749

TABLE A23. Limestone SMA Mix Design Summary, PG 76-22, 65 Gyrations

Project:	AAPTP 04-04 SMA for Airfields	Aggregate Type: Limestone	Percent Retained on #4 sieve: 77.1	Date	8/25/2008
AC Sp. Gr. (Gb) = 1.028	App. Sp. Gr. (Gsa) 2.749	Eff. Sp. Gr. (Gse): 2.739	Bulk Sp. Gr. (Gsb): 2.720		
		Trial Blend: 4	Bulk Sp. Gravity of CA: 2.731		
		NMAS: 12.5	Density of CA in DRC (kg/m ^{3}): 1548.6		
		% Passing #4 Sieve: 22.9	Compactive Effort: 65 Gyrations		

		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS						
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %
1	5.5	4692.4	2739.9	4704.8	2.388	2.510	83.0	12.8	149.0	4.9	17.0	71	36.3	43.2	0.840	5.3
2	5.5	4760.7	2781.9	4773.2	2.391	2.510	83.1	12.8	149.2	4.8	16.9	72	36.2	43.2	0.839	5.3
3	5.5	4717.6	2762.4	4732.5	2.395	2.510	83.2	12.8	149.4	4.6	16.8	73	36.1	43.2	0.836	5.3
Avg.					2.391	2.510	83.1	12.8	149.2	4.7	16.9	72	36.2	43.2	0.838	5.3
1	6.0	4749.5	2779.1	4756.4	2.402	2.491	83.0	14.0	149.9	3.6	17.0	79	36.3	43.2	0.840	5.8
2	6.0	4676.4	2735.0	4686.2	2.397	2.491	82.8	14.0	149.6	3.8	17.2	78	36.4	43.2	0.843	5.8
3	6.0	4740.5	2780.6	4747.8	2.410	2.491	83.3	14.1	150.4	3.3	16.7	80	36.1	43.2	0.835	5.8
Avg.					2.403	2.491	83.0	14.0	149.9	3.5	17.0	79	36.2	43.2	0.839	5.8
1	7.2	4782.4	2771.8	4792.2	2.367	2.446	80.8	16.6	147.7	3.2	19.2	83	38.0	43.2	0.880	7.0
2	7.2	4795.5	2780.0	4803.3	2.370	2.446	80.9	16.6	147.9	3.1	19.1	84	37.9	43.2	0.878	7.0
3	7.2	4802.9	2790.1	4809.6	2.378	2.446	81.1	16.7	148.4	2.8	18.9	85	37.7	43.2	0.873	7.0
Avg.					2.372	2.446	80.9	16.6	148.0	3.0	19.1	84	37.9	43.2	0.879	7.0

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
5.5	4.7	16.9	72.0	36.2	0.838	149.2	5.3
6.0	3.5	17.0	79.1	36.2	0.839	149.9	5.8
7.2	3.0	19.1	84.1	37.9	0.879	148.0	7.0
5.8	4.0	16.9	76.4	36.2	0.839	149.7	5.6

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.735	65
7's	2.719	25
89's	2.714	0
820's	2.602	4
Mineral Filler	2.718	5
Lime	2.350	1
		100

Comb. Gsb = 2.720

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.759	65
7's	2.748	25
89's	2.752	0
820's	2.747	4
Mineral Filler	2.718	5
Lime	2.350	1
		100

Comb. Gsa = 2.749

TABLE A24. Limestone SMA Mix Design Summary, PG 76-22, 80 Gyrations

Project:	AAPTP 04-04 SMA for Airfields			Aggregate Type:	Limestone	Percent Retained on #4 sieve:	77.1
AC Sp. Gr. (Gb) = 1.028	App. Sp. Gr. (Gsa) 2.749	Eff. Sp. Gr. (Gse): 2.739	Bulk Sp. Gr. (Gsb): 2.720	Trial Blend:	4	Bulk Sp. Gravity of CA:	2.731
				NMAS:	12.5	Density of CA in DRC (kg/m ^{3}):	1548.6
				% Passing #4 Sieve:	22.9	Compactive Effort:	80 Gyrations

Date: 8/25/2008

Specimen Number	Asphalt Content	Masses: In Air (gms)	Masses: In Water (gms)	Masses: SSD (gms)	Specific Gravities: Bulk (Gmb)	Specific Gravities: TMD (Gmm)	Volumes at Ndes: Aggregate Volume cc	Volumes at Ndes: AC by Volume %	Unit Weight, pcf	Voids: VTM, %	Voids: VMA, %	Voids: VFA, %	Voids: VCAmix, %	Voids: VCAdrc, %	Voids: VCAmix / VCAdrc	Eff. AC Content %
1	6.8	4775.1	2774.3	4780.0	2.381	2.462	81.6	15.7	148.6	3.3	18.4	82	37.4	43.2	0.865	6.6
2	6.8	4769.9	2771.8	4776.1	2.380	2.462	81.5	15.7	148.5	3.3	18.5	82	37.4	43.2	0.866	6.6
3	6.8	4769.5	2766.7	4776.6	2.373	2.462	81.3	15.7	148.1	3.6	18.7	81	37.6	43.2	0.870	6.6
Avg.					2.378	2.462	81.5	15.7	148.4	3.4	18.5	82	37.4	43.2	0.867	6.6
1	7.0	4812.6	2799.9	4818.7	2.384	2.454	81.5	16.2	148.8	2.9	18.5	85	37.4	43.2	0.866	6.8
2	7.0	4812.9	2796.4	4818.2	2.381	2.454	81.4	16.2	148.5	3.0	18.6	84	37.5	43.2	0.868	6.8
Avg.					2.382	2.454	81.5	16.2	148.6	2.9	18.5	84	37.5	43.2	0.867	6.8
1	7.2	4776.1	2790.1	4780.9	2.399	2.446	81.9	16.8	149.7	1.9	18.1	89	37.1	43.2	0.860	7.0
2	7.2	4768.0	2772.7	4774.5	2.382	2.446	81.3	16.7	148.6	2.6	18.7	86	37.6	43.2	0.871	7.0
3	7.2	4750.8	2758.2	4755.2	2.379	2.446	81.2	16.7	148.4	2.7	18.8	85	37.7	43.2	0.872	7.0
Avg.					2.387	2.446	81.4	16.7	148.9	2.4	18.6	87	37.5	43.2	0.868	7.0
1	7.5	4745.0	2756.3	4749.3	2.381	2.435	81.0	17.4	148.6	2.2	19.0	88	37.8	43.2	0.876	7.3
2	7.5	4782.5	2782.1	4787.6	2.385	2.435	81.1	17.4	148.8	2.1	18.9	89	37.7	43.2	0.874	7.3
3	7.5	4762.1	2764.0	4767.5	2.377	2.435	80.8	17.3	148.3	2.4	19.2	88	37.9	43.2	0.878	7.3
Avg.					2.381	2.435	81.0	17.4	148.6	2.2	19.0	88	37.8	43.2	0.876	7.3

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
6.8	3.4	18.5	82	37.4	0.867	148.4	6.6
7.0	2.9	18.5	84	37.5	0.867	148.6	6.8
7.2	2.4	18.6	87	37.5	0.868	148.9	7.0
7.5	2.2	19.0	88	37.8	0.876	148.6	7.3
7.0	3.0	18.6	84	37.5	0.868	148.6	6.7
6.6	4.0	18.5	78	37.4	0.866	148.1	6.3

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.735	65
7's	2.719	25
89's	2.714	0
820's	2.602	4
Mineral Filler	2.718	5
Lime	2.350	1
		100

Comb. Gsb = 2.720

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.759	65
7's	2.748	25
89's	2.752	0
820's	2.747	4
Mineral Filler	2.718	5
Lime	2.350	1
		100

Comb. Gsa = 2.749

TABLE A25. Limestone SMA Mix Design Summary, PG 76-22, 100 Gyrations

Project:	AAPTP 04-04 SMA for Airfields			Aggregate Type: Limestone	Percent Retained on #4 sieve: 77.1	Date
AC Sp. Gr. (Gb) = 1.028	App. Sp. Gr. (Gsa) 2.749	Eff. Sp. Gr. (Gse): 2.739	Bulk Sp. Gr. (Gsb): 2.720	Trial Blend: 4	Bulk Sp. Gravity of CA: 2.731	8/25/2008
				NMAS: 12.5	Density of CA in DRC (kg/m ^{3}): 1548.6	
				% Passing #4 Sieve: 22.9	Compactive Effort: 100 Gyrations	

Specimen Number	Asphalt Content	Masses: In Air (gms)	Masses: In Water (gms)	Masses: SSD (gms)	Specific Gravities: Bulk (Gmb)	Specific Gravities: TMD (Gmm)	Volumes at Ndes: Aggregate Volume cc	Volumes at Ndes: AC by Volume %	Unit Weight, pcf	Voids: VTM, %	Voids: VMA, %	Voids: VFA, %	Voids: VCAmix, %	Voids: VCAdrc, %	Voids: VCAmix/VCAdrc	Eff. AC Content %
1	5.5	4789.5	2836.6	4795.0	2.446	2.510	85.0	13.1	152.6	2.6	15.0	83	34.8	43.2	0.805	5.3
2	5.5	4714.8	2771.8	4721.0	2.419	2.510	84.0	12.9	150.9	3.6	16.0	77	35.5	43.2	0.821	5.3
3	5.5	4746.8	2807.7	4753.4	2.440	2.510	84.8	13.1	152.2	2.8	15.2	82	34.9	43.2	0.809	5.3
Avg.					2.435	2.510	84.6	13.0	151.9	3.0	15.4	81	35.0	43.2	0.812	5.3
1	6.0	4757.7	2804.6	4762.7	2.430	2.491	84.0	14.2	151.6	2.5	16.0	85	35.5	43.2	0.823	5.8
2	6.0	4753.0	2804.8	4760.0	2.431	2.491	84.0	14.2	151.7	2.4	16.0	85	35.5	43.2	0.822	5.8
Avg.					2.430	2.491	84.0	14.2	151.7	2.4	16.0	85	35.5	43.2	0.822	5.8
1	6.5	4764.8	2786.1	4775.8	2.395	2.472	82.3	15.1	149.4	3.1	17.7	82	36.8	43.2	0.852	6.3
2	6.5	4768.9	2791.8	4778.9	2.400	2.472	82.5	15.2	149.8	2.9	17.5	83	36.7	43.2	0.849	6.3
3	6.5	4786.9	2797.0	4795.0	2.396	2.472	82.4	15.1	149.5	3.1	17.6	83	36.8	43.2	0.851	6.3
Avg.					2.397	2.472	82.4	15.2	149.6	3.0	17.6	83	36.7	43.2	0.850	6.3

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
5.5	3.0	15.4	81	35.0	0.812	151.9	5.3
6.0	2.4	16.0	85	35.5	0.822	151.7	5.8
6.5	3.0	17.6	83	35.0	0.812	151.9	5.3
4.6	4.0	14.4	73	34.2	0.793	152.4	4.4

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.735	65
7's	2.719	25
89's	2.714	0
820's	2.602	4
Mineral Filler	2.718	5
Lime	2.350	1
		100
Comb. Gsb =		2.720

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.759	65
7's	2.748	25
89's	2.752	0
820's	2.747	4
Mineral Filler	2.718	5
Lime	2.350	1
		100
Comb. Gsa =		2.749

TABLE A26. Limestone SMA Mix Design Summary, PG 64-22, 50 Blow Marshall

Project:	AAPTP 04-04 SMA for Airfields								Aggregate Type:	Limestone			Percent Retained on #4 sieve:	77.1		Date			
AC Sp. Gr. (Gb) = 1.028		App. Sp. Gr. (Gsa) 2.749		Eff. Sp. Gr. (Gse): 2.739		Bulk Sp. Gr. (Gsb): 2.720			Trial Blend:	4			Bulk Sp. Gravity of CA:	2.731		8/25/2008			
									NMAS:	12.5			Density of CA in DRC (kg/m^3):	1548.6					
									% Passing #4 Sieve:	22.9			Compactive Effort:	50 blow Marshall					
		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS									
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %	Height, in	Stability, lbs	Flow, 0.01 in
1	7.4	1226.5	711.7	1229.1	2.371	2.440	80.7	17.1	147.9	2.8	19.3	85	38.0	43.2	0.881	7.2	2.48	1700	18.0
2	7.4	1223.1	707.6	1227.5	2.353	2.440	80.1	16.9	146.8	3.6	19.9	82	38.5	43.2	0.892	7.2	2.49	1600	16.0
3	7.4	1224.8	709.8	1228.8	2.360	2.440	80.3	17.0	147.3	3.3	19.7	83	38.3	43.2	0.887	7.2	2.48	1700	19.5
Avg.					2.361	2.440	80.4	17.0	147.3	3.2	19.6	84	38.3	43.2	0.886	7.2	2.48	1667	17.8

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
7.4	3.2	19.6	83.5	38.3	0.886	147.3	7.2

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.735	65
7's	2.719	25
89's	2.714	0
820's	2.602	4
Mineral Filler	2.718	5
Lime	2.350	1
		100

Comb. Gsb = 2.720

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.759	65
7's	2.748	25
89's	2.752	0
820's	2.747	4
Mineral Filler	2.718	5
Lime	2.350	1
		100

Comb. Gsa = 2.749

TABLE A27. Limestone SMA Mix Design Summary, PG 64-22, 50 Gyrations

Project:	AAPTP 04-04 SMA for Airfields			Aggregate Type:	Limestone	Percent Retained on #4 sieve:	77.1	Date
AC Sp. Gr. (Gb) = 1.028	App. Sp. Gr. (Gsa) 2.749	Eff. Sp. Gr. (Gse): 2.739	Bulk Sp. Gr. (Gsb): 2.720	Trial Blend:	4	Bulk Sp. Gravity of CA:	2.731	8/25/2008
				NMAS:	12.5	Density of CA in DRC (kg/m^3):	1548.6	
				% Passing #4 Sieve:	22.9	Compactive Effort:	50 Gyration	

		Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS						
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %
1	7.6	4812.1	2782.7	4821.1	2.361	2.432	80.2	17.5	147.3	2.9	19.8	85	38.4	43.2	0.890	7.4
2	7.6	4842.2	2796.0	4853.3	2.354	2.432	80.0	17.4	146.9	3.2	20.0	84	38.6	43.2	0.894	7.4
Avg.					2.357	2.432	80.1	17.4	147.1	3.1	19.9	85	38.5	43.2	0.892	7.4
1	7.8	4833.2	2795.7	4837.0	2.368	2.424	80.3	18.0	147.7	2.3	19.7	88	38.4	43.2	0.889	7.6
2	7.8	4809.8	2763.9	4815.6	2.344	2.424	79.5	17.8	146.3	3.3	20.5	84	39.0	43.2	0.903	7.6
3	7.8	4818.4	2789.4	4822.9	2.370	2.424	80.3	18.0	147.9	2.2	19.7	89	38.3	43.2	0.887	7.6
Avg.					2.361	2.424	80.0	17.9	147.3	2.6	20.0	87	38.6	43.2	0.893	7.6
1	8.0	4801.8	2775.2	4814.1	2.355	2.416	79.7	18.3	147.0	2.5	20.3	88	38.8	43.2	0.899	7.8
2	8.0	4820.8	2795.5	4828.6	2.371	2.416	80.2	18.5	148.0	1.9	19.8	91	38.4	43.2	0.890	7.8
Avg.					2.363	2.416	79.9	18.4	147.5	2.2	20.1	89	38.6	43.2	0.894	7.8

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
7.6	3.1	19.9	84.6	38.5	0.892	147.1	7.4
7.8	2.6	20.0	86.9	38.6	0.893	147.3	7.6
8.0	2.2	20.1	89.1	38.6	0.894	147.5	7.8
7.2	4.0	19.8	79.8	38.4	0.89	146.7	6.9
7.6	3.0	19.9			0.89		

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.735	65
7's	2.719	25
89's	2.714	0
820's	2.602	4
Mineral Filler	2.718	5
Lime	2.350	1
		100

Comb. Gsb = 2.720

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.759	65
7's	2.748	25
89's	2.752	0
820's	2.747	4
Mineral Filler	2.718	5
Lime	2.350	1
		100

Comb. Gsa = 2.749

TABLE A28. Limestone SMA Mix Design Summary, PG 64-22, 64 Gyrations

Project:	AAPTP 04-04 SMA for Airfields			Aggregate Type:	Limestone	Percent Retained on #4 sieve:	77.1	Date
AC Sp. Gr. (Gb) = 1.028		App. Sp. Gr. (Gsa) 2.749	Eff. Sp. Gr. (Gse): 2.739	Bulk Sp. Gr. (Gsb): 2.720	Trial Blend: 4	Bulk Sp. Gravity of CA:	2.731	8/25/2008
				NMAS:	12.5	Density of CA in DRC (kg/m^3):	1548.6	
				% Passing #4 Sieve:	22.9	Compactive Effort:	65 Gyration	

Specimen Number	Asphalt Content	Masses			SPECIFIC GRAVITIES		VOLUMES AT Ndes			VOIDS						
		In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	Aggregate Volume cc	AC by Volume %	Unit Weight, pcf	VTM, %	VMA, %	VFA, %	VCAmix, %	VCAdrc, %	VCAmix / VCAdrc	Eff. AC Content %
1	7.2	4747.0	2756.8	4759.9	2.370	2.449	80.9	16.6	147.9	3.2	19.1	83	37.9	43.2	0.878	7.0
2	7.2	4766.1	2768.6	4776.5	2.374	2.449	81.0	16.6	148.1	3.1	19.0	84	37.8	43.2	0.876	7.0
Avg.					2.372	2.449	80.9	16.6	148.0	3.2	19.1	83	37.9	43.2	0.877	7.0

% AC	VTM	VMA	VFA	VCAmix	VCAratio	Unit Weight	Eff AC
7.2	3.2	19.1	83.5	37.9	0.877	148.0	7.0

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.735	65
7's	2.719	25
89's	2.714	0
820's	2.602	4
Mineral Filler	2.718	5
Lime	2.350	1
		100

Comb. Gsb = 2.720

Combined Gsa of Aggregates

	GsA	% Blend
67's	2.759	65
7's	2.748	25
89's	2.752	0
820's	2.747	4
Mineral Filler	2.718	5
Lime	2.350	1
		100

Comb. Gsa = 2.749

TABLE A29. Measured Aggregate Gradations

Sieve Size	Percent Passing						Control Points
	Diabase	Columbus Granite (Gyratory)	Columbus Granite (Marshall)	Ruby Granite	Gravel	Limestone	
3/4"	100.0	100.0	100.0	100.0	100.0	100.0	100
1/2"	100.0	93.6	96.9	99.5	95.2	89.8	90-99
3/8"	94.5	61.9	68.0	68.7	65.0	64.2	50-85
#4	32.1	24.5	29.4	25.7	28.2	22.9	20-40
#8	22.0	18.2	24.0	20.1	22.3	12.0	16-28
#16	19.6	16.5	21.3	16.7	20.1	9.6	
#30	17.7	12.7	19.0	14.7	16.4	8.9	
#50	15.7	10.8	16.9	13.3	14.5	8.5	
#100	13.3	9.7	14.5	11.9	13.3	8.2	
#200	9.8	8.7	12.5	11.0	9.4	7.8	8-11

APPENDIX B – P401 MIX DESIGNS

TABLE B1. Diabase P401 Mix Design Summary

Project:	AAPTP 04-04, P401 Mix Designs							Aggregate Type:	Diabase	Compactive Effort:		75 blows
AC Sp. Gr. (Gb) = 1.028		App. Sp. Gr. (Gsa)		Eff. Sp. Gr. (Gse): 2.977		Bulk Sp. Gr. (Gsb): 2.960		Trial Blend:	1	Binder Grade:		PG 76-22
								NMAS:	12.5	Compaction Temp:		320-330 °F
		Masses			SPECIFIC GRAVITIES		VOIDS					
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	VTM, %	VMA, %	VFA, %	Stability, lbs	Flow, 0.01 in	Eff. AC Content %
1	5.0					2.720						4.8
2	5.0	1214.7	752.6	1216.3	2.620	2.720	3.7	15.9	77	5119	12	4.8
3	5.0	1222.7	756.6	1224.2	2.615	2.720	3.9	16.1	76	4606	10	4.8
Avg.					2.617	2.720	3.8	16.0	76	4863	11	4.8
1	5.5	1219.2	755.6	1219.6	2.628	2.696	2.5	16.1	84	4798	13	5.3
2	5.5	1215.3	755.3	1215.6	2.640	2.696	2.1	15.7	87	4996	11	5.3
3	5.5	1217.9	757.1	1218.2	2.641	2.696	2.0	15.7	87	4522	12	5.3
Avg.					2.636	2.696	2.2	15.8	86	4772	12	5.3

% AC	VTM	VMA	VFA	Stability	Flow	Eff AC
5.0	3.8	16.0	76	4863	11	4.8
5.5	2.2	15.8	86	4772	12	5.3
5.1	3.5	16.0	78	4846	11	4.9

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
68's	3.036	35
8's	2.990	15
10's	3.030	35
Natural Sand	2.638	15
		100

Comb. Gsb = 2.960

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
68's		35
8's		15
10's		35
Natural Sand		15
		100

Comb. Gsa =

TABLE B2. Columbus Granite P401 Mix Design Summary

Project:	**AAPTP 04-04, P401 Mix Designs**							**Aggregate Type:**	Granite	**Compactive Effort:**	75 blows	
AC Sp. Gr. (Gb) = 1.028		App. Sp. Gr. (Gsa) 2.745		Eff. Sp. Gr. (Gse): 2.717		Bulk Sp. Gr. (Gsb): 2.701		**Trial Blend:**	1	**Binder Grade:**	PG 76-22	
								NMAS:	12.5	**Compaction Temp:**	320-330 °F	
		Masses			SPECIFIC GRAVITIES		VOIDS					
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	VTM, %	VMA, %	VFA, %	Stability, lbs	Flow, 0.01 in	Eff. AC, %
1	5.0	1221.5	712.7	1225.8	2.381	2.511	5.2	16.3	68	5108	12	4.8
2	5.0	1218.6	710.8	1219.8	2.394	2.511	4.7	15.8	71	4768	11	4.8
3	5.0	1223.5	714.9	1224.6	2.400	2.511	4.4	15.6	72	5341	11	4.8
Avg.					2.392	2.511	4.8	15.9	70	5072	12	4.8
1	5.5	1220.2	717.2	1220.8	2.423	2.492	2.8	15.2	82	5183	12	5.3
2	5.5	1220.0	718.1	1220.6	2.428	2.492	2.6	15.1	83	5723	14	5.3
3	5.5	1221.9	719.1	1222.8	2.426	2.492	2.7	15.1	82	4918	15	5.3
Avg.					2.426	2.492	2.7	15.1	82	5275	14	5.3
1	6.0	1207.5	711.3	1207.7	2.433	2.474	1.7	15.3	89	5112	18	5.8
2	6.0	1212.6	715.6	1212.9	2.438	2.474	1.4	15.1	90	5895	22	5.8
3	6.0	1209.9	712.6	1210.2	2.431	2.474	1.7	15.4	89	5047	16	5.8
Avg.					2.434	2.474	1.6	15.3	89	5351	19	5.8

% AC	VTM	VMA	VFA	Stability	Flow	Eff AC
5.0	4.8	15.9	70	5072	12	4.8
5.5	2.7	15.1	82	5275	14	5.3
6.0	1.6	15.3	89	5351	19	5.8
5.3	3.5	15.5	78	5191	13	5.1

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
007's	2.720	40
89's	2.673	15
M-10's	2.722	30
Natural Sand	2.638	15
		100

Comb. Gsb = 2.701

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
007's	2.773	40
89's	2.761	15
M-10's	2.741	30
Natural Sand	2.664	15
		100

Comb. Gsa = 2.745

TABLE B3. Ruby Granite P401 Mix Design Summary

Project:	**AAPTP 04-04, P401 Mix Designs**							**Aggregate Type:**	Ruby Granite	**Compactive Effort:**		75 blows
AC Sp. Gr. (Gb) = 1.028		App. Sp. Gr. (Gsa) 2.760		Eff. Sp. Gr. (Gse): 2.729		Bulk Sp. Gr. (Gsb): 2.693		**Trial Blend:**	4	**Binder Grade:**		PG 76-22
								NMAS:	12.5	**Compaction Temp:**		320-330 °F
		Masses			SPECIFIC GRAVITIES		VOIDS					
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	VTM, %	VMA, %	VFA, %	Stability, lbs	Flow, 0.01 in	Eff. AC Content %
1	5.0	1221.7	714.2	1222.3	2.404	2.521	4.6	15.2	70	5088	12	4.5
2	5.0	1219.8	713.2	1220.5	2.404	2.521	4.6	15.2	70	4598	10	4.5
3	5.0					2.521						4.5
Avg.					2.404	2.521	4.6	15.2	70	4843	11	4.5
1	5.5	1220.0	720.9	1221.9	2.435	2.502	2.7	14.6	82	4285	12	5.0
2	5.5	1222.3	721.0	1222.9	2.435	2.502	2.7	14.6	82	4779	13	5.0
3	5.5	1218.3	717.9	1218.8	2.432	2.502	2.8	14.7	81	4531	13	5.0
Avg.					2.434	2.502	2.7	14.6	81	4532	13	5.0

% AC	VTM	VMA	VFA	Stability	Flow	Eff AC
5.0	4.6	15.2	70	4843	11	4.5
5.5	2.7	14.6	81	4532	13	5.0
5.3	3.5	14.8	77	4660	12	4.8

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
7's	2.763	37
89's	2.697	17
M-10's	2.639	31
Natural Sand	2.638	15
		100

Comb. Gsb = 2.693

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
7's	2.819	37
89's	2.760	17
M-10's	2.740	31
Natural Sand	2.661	15
		100

Comb. Gsa = 2.760

TABLE B4. Gravel P401 Mix Design Summary

Project:	**AAPTP 04-04, P401 Mix Designs**							**Aggregate Type:**	Gravel	**Compactive Effort:**		75 blows
AC Sp. Gr. (Gb) = 1.028		App. Sp. Gr. (Gsa) 2.644		Eff. Sp. Gr. (Gse): 2.634		Bulk Sp. Gr. (Gsb): 2.604		**Trial Blend:**	1	**Binder Grade:**		PG 76-22
								NMAS:	12.5	**Compaction Temp:**		320-330 °F
		Masses			SPECIFIC GRAVITIES		VOIDS					
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	VTM, %	VMA, %	VFA, %	Stability, lbs	Flow, 0.01 in	Eff. AC Content %
1	5.0	1210.8	692.3	1212.7	2.327	2.443	4.8	15.1	68	4415	13	4.6
2	5.0	1217.5	693.8	1220.1	2.313	2.443	5.3	15.6	66	3826	13	4.6
3	5.0	1223.3	694.7	1224.2	2.310	2.443	5.4	15.7	65	4117	13	4.6
Avg.					2.317	2.443	5.2	15.5	67	4119	13	4.6
1	5.5	1222.3	707.2	1223.2	2.369	2.426	2.4	14.0	83	3576	11	5.1
2	5.5	1222.8	704.3	1223.7	2.354	2.426	3.0	14.6	80	3681	10	5.1
3	5.5	1219.4	702.5	1220.6	2.354	2.426	3.0	14.6	80	3719	11	5.1
Avg.					2.359	2.426	2.8	14.4	81	3659	10	5.1

% AC	VTM	VMA	VFA	Stability	Flow	Eff AC
5.0	5.2	15.5	67	4119	13	4.6
5.5	2.8	14.4	81	3659	10	5.1
5.3	3.5	14.7	76	3799	11	4.9

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.598	68
Fines	2.603	20
Natural Sand	2.638	12
		100

Comb. Gsb = 2.604

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.641	68
Fines	2.645	20
Natural Sand	2.661	12
		100

Comb. Gsa = 2.644

TABLE B5. Limestone P401 Mix Design Summary, PG 76-22

Project:	**AAPTP 04-04, P401 Mix Designs**							**Aggregate Type:**	Limestone		**Compactive Effort:**	75 blows
AC Sp. Gr. (Gb) = 1.028		App. Sp. Gr. (Gsa) 2.746		Eff. Sp. Gr. (Gse): 2.729		Bulk Sp. Gr. (Gsb): 2.705		**Trial Blend:**	4		**Binder Grade:**	PG 76-22
								NMAS:	12.5		**Compaction Temp:**	320-330 °F
		Masses			SPECIFIC GRAVITIES		VOIDS					
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	VTM, %	VMA, %	VFA, %	Stability, lbs	Flow, 0.01 in	Eff. AC, %
1	5.0	1222.1	714.5	1223.1	2.403	2.521	4.7	15.6	70	3747	12	4.7
2	5.0	1226.4	714.7	1227.1	2.393	2.521	5.1	15.9	68	3638	14	4.7
3	5.0	1222.4	714.5	1223.2	2.403	2.521	4.7	15.6	70	3825	14	4.7
Avg.					2.400	2.521	4.8	15.7	69	3737	14	4.7
1	5.5	1215.2	715.8	1216.0	2.429	2.502	2.9	15.1	81	4083	13	5.2
2	5.5	1208.4	709.2	1208.9	2.418	2.502	3.3	15.5	78	3908	12	5.2
3	5.5	1216.7	716.1	1217.2	2.428	2.502	3.0	15.2	81	4036	10	5.2
Avg.					2.425	2.502	3.1	15.3	80	4009	11	5.2

% AC	VTM	VMA	VFA	Stability	Flow	Eff AC
5.0	4.8	15.7	69	3737	14	4.7
5.5	3.1	15.3	80	4009	11	5.2
5.4	3.5	15.4	77	3941	12	5.1

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.730	28
89's	2.722	10
820's	2.708	47
Natural Sand	2.638	15
		100

Comb. Gsb = 2.705

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.756	28
89's	2.752	10
820's	2.766	47
Natural Sand	2.661	15
		100

Comb. Gsa = 2.746

TABLE B6. Limestone P401 Mix Design Summary, PG 64-22

Project:	**AAPTP 04-04, P401 Mix Designs**							**Aggregate Type:**	Limestone		**Compactive Effort:**	75 blows
AC Sp. Gr. (Gb) = 1.028		App. Sp. Gr. (Gsa) 2.746		Eff. Sp. Gr. (Gse): 2.744		Bulk Sp. Gr. (Gsb): 2.705		**Trial Blend:**	1		**Binder Grade:**	PG 64-22
								NMAS:	12.5		**Compaction Temp:**	300-310 °F
		Masses			SPECIFIC GRAVITIES		VOIDS					
Specimen Number	Asphalt Content	In Air (gms)	In Water (gms)	SSD (gms)	Bulk (Gmb)	TMD (Gmm)	VTM, %	VMA, %	VFA, %	Stability, lbs	Flow, 0.01 in	Eff. AC Content %
1	5.0	1209.2	710.9	1210.1	2.422	2.533	4.4	14.9	71	3501	9	4.5
2	5.0	1210.4	711.4	1211.2	2.422	2.533	4.4	14.9	71	2940	9	4.5
3	5.0	1207.4	706.5	1207.8	2.409	2.533	4.9	15.4	68	2981	7	4.5
Avg.					2.418	2.533	4.6	15.1	70	3141	8	4.5
1	5.5					2.514						5.0
2	5.5	1217.7	716.4	1218.2	2.427	2.514	3.5	15.2	77	3096	9	5.0
3	5.5	1216.6	715.7	1217.1	2.426	2.514	3.5	15.2	77	3125	7	5.0
Avg.					2.427	2.514	3.5	15.2	77	3111	8	5.0

% AC	VTM	VMA	VFA	Stability	Flow	Eff AC
5.0	4.6	15.1	70	3141	8	4.5
5.5	3.5	15.2	77	3111	8	5.0

Combined Gsb of Aggregates

Stockpile	Gsb	% Blend
67's	2.730	28
89's	2.722	10
820's	2.708	47
Natural Sand	2.638	15
		100

Comb. Gsb = 2.705

Combined Gsa of Aggregates

Stockpile	Gsa	% Blend
67's	2.756	28
89's	2.752	10
820's	2.766	47
Natural Sand	2.661	15
		100

Comb. Gsa = 2.746

TABLE B7. Measured Aggregate Gradations

Sieve Size	Percent Passing					Control Points
	Diabase	Columbus Granite	Ruby Granite	Gravel	Limestone	
3/4"	98.8	100.0	100.0	100.0	100.0	100.0
1/2"	80.7	97.1	98.0	89.5	93.7	79-99
3/8"	70.3	76.9	83.9	81.2	85.5	68-88
#4	49.0	51.0	52.0	63.3	67.2	48-68
#8	38.2	41.2	36.8	49.4	44.0	33-53
#16	31.2	33.9	27.2	37.2	29.5	20-40
#30	23.1	25.2	18.7	25.9	18.8	14-30
#50	13.3	14.7	10.8	15.4	10.2	9-21
#100	7.6	8.6	7.2	9.2	6.6	6-16
#200	4.6	5.2	4.9	5.7	5.0	3-6

APPENDIX C – DRAFT ADVISORY CIRCULAR ON SMA

PART V – FLEXIBLE SURFACE COURSES
ITEM P-XXX PLANT MIX BITUMINOUS STONE MATRIX ASPHALT (SMA) PAVEMENTS

DESCRIPTION

XXX-1.1 This item shall consist of Stone Matrix Asphalt (SMA) pavement surface course composed of mineral aggregate and bituminous material mixed in a central mixing plant and placed on a prepared course in accordance with these specifications and shall conform to the lines, grades, thicknesses, and typical cross sections shown on the plans. Each course shall be constructed to the depth, typical section, and elevation required by the plans and shall be rolled, finished, and approved before the placement of the next course.

*** ***

This specification is suitable for use in constructing the following SMA pavement mixtures:

1. **All runway, taxiway and apron pavements.**
2. **All other airfield pavements such as shoulders, blast pads, or overruns.**

The dimensions and depth of the "surface course" for which this specification applies shall be that as is defined by the Engineer's pavement design as performed in accordance with FAA Advisory Circular 150/5320-6, current edition.

SMA should be considered a "premium" surface mix. SMA mixes will have higher asphalt contents than conventional P-401 mixes. The increased asphalt content greatly enhances the durability of SMA mixes, particularly in terms of cracking. The increased coarse aggregate content of SMA mixes produces a stone skeleton to carry the load. SMA mixes also tend to be more resistant to fuel spills than conventional P-401 mixes. Although SMA has been shown to reduce reflective cracking, SMA should not be used on pavements with unsound bases.

State highway department SMA specifications may be used for shoulders, access roads, perimeter roads, stabilized base courses under Item P-501, and other pavements not subject to aircraft loading. When state highway specification are approved, include all applicable/approved state specifications in the contract documents.

*** ***

MATERIALS

XXX-2.1 AGGREGATE. Aggregates shall consist of crushed stone, crushed gravel, or crushed slag or other inert finely divided mineral aggregate. The portion of combined materials retained on the No. 4 (4.75 mm) sieve is coarse aggregate. The portion of combined materials passing the No. 4 (4.75 mm) sieve and retained on the No. 200 (0.075 mm) sieve is fine aggregate, and the portion passing the No. 200 (0.075 mm) sieve is mineral filler.

a. Coarse Aggregate. Coarse aggregate shall consist of sound, tough, durable particles, free from adherent films of matter that would prevent thorough coating and bonding with the bituminous material and be free from organic matter and other deleterious substances. The percentage of wear shall not be greater than 30 percent when tested in accordance with ASTM C 131. The sodium sulfate soundness loss shall not exceed 10 percent, or the magnesium sulfate soundness loss shall not exceed 18 percent, after five cycles, when tested in accordance with ASTM C 88.

* *

Aggregates with a higher percentage loss of wear or soundness may be specified in lieu of those above, provided a satisfactory service record under similar conditions of service and exposure has been demonstrated.

* *

Aggregate shall contain at least **[ ]** percent by weight of individual pieces having two or more fractured faces and **[ ]** percent by weight having at least one fractured face. The area of each face shall be equal to at least 75 percent of the smallest midsectional area of the piece. When two fractured faces are contiguous, the angle between the planes of fractures shall be at least 30 degrees to count as two fractured faces. Fractured faces shall be obtained by crushing.

* *

For pavements designed for aircraft gross weights of 60,000 pounds (27 200 kg) or more, the Engineer shall specify 90 percent for two fractured faces and 100 percent for one fractured face. For pavements designed for aircraft gross weights less than 60,000 pounds (27 200 kg), the Engineer shall specify 85 percent for two fractured faces and 95 percent for one fractured face.

In areas where slag is not available or desired, the references to it should be deleted from all aggregate paragraphs.

* *

The aggregate shall not contain more than a total of 20 percent by weight of flat particles, elongated particles, and flat and elongated particles with a 3:1 value, and shall not contain more than a total of 5 percent by weight of flat particles, elongated particles, and flat and elongated particles with a 5:1 value when tested in accordance with ASTM D4791.

* *

Acceptable stone matrix asphalt mixes have been successfully designed and produced in the lab having higher percentages of flat and elongated particles, for both 3:1 and 5:1 ratios. However, the field compactability of these mixes has not been verified. If aggregate sources having a higher percentage of flat and elongated particles are proposed to be used, replace the above paragraph as follows: "The aggregate shall not contain more than a total of [ ] percent and [ ] percent by weight of flat particles, elongated particles, and flat and elongated particles when tested in accordance with ASTM D4791 with a value of 3:1 and 5:1, respectively."

* *

Slag shall be air-cooled, blast furnace slag, and shall have a compacted weight of not less than 70 pounds per cubic foot (1.12 mg/cubic meter) when tested in accordance with ASTM C 29.

b. Fine Aggregate. Fine aggregate shall consist of clean, sound, durable, angular shaped particles produced by crushing stone, slag, or gravel that meets the requirements for wear and soundness specified for coarse aggregate. The aggregate particles shall be free from coatings of clay, silt, or other objectionable matter and shall contain no clay balls. The fine aggregate, including any blended material for the fine aggregate, shall have a plasticity index of not more than 6 and a liquid limit of not more than 25 when tested in accordance with ASTM D 4318.

Natural (non-manufactured) sand may not be used in SMA. The aggregate shall have sand equivalent values of **[ ]** or greater when tested in accordance with ASTM D 2419.

* *

Typically the sand equivalent value should be 45, unless local conditions require lower value.

* *

c. Sampling. ASTM D 75 shall be used in sampling coarse and fine aggregate, and ASTM C 183 shall be used in sampling mineral filler.

XXX-2.2 MINERAL FILLER. Mineral filler shall meet the requirements of ASTM D 242.

XXX-2.3 BITUMINOUS MATERIAL. Bituminous material shall conform to the following requirements: **[ ]**.

* *

Asphalt cement binder shall conform to [AASHTO M320 Performance Grade (PG) [_____]] [ASTM D 3381 Table 1, 2, or 3 Viscosity Grade][ASTM D 946 Penetration Grade [_____]]. Test data indicating grade certification shall be provided by the supplier at the time of delivery of each load to the mix plant. Copies of these certifications shall be submitted to the Engineer. The Engineer shall specify the grade of bituminous material, based on geographical location and climatic conditions. Asphalt Institute Superpave Series No. 1 (SP-1) provides guidance on the selection of performance graded binders. Table VI-1, Selecting Asphalt Grade, contained in the Asphalt Institute's Manual Series-1 (MS-1) provides guidance on the selection of asphalt type. For cold climates, Table 2 of ASTM D 3381 may be specified to minimize the susceptibility for thermal cracking. The Engineer should be aware that PG asphalt binders may contain modifiers that require elevated mixing and compaction temperatures that exceed the temperatures specified in Item P-XXX.

Grades of some materials are listed below:

NOTE: Performance Graded (PG) asphalt binders should be specified wherever available. The same grade PG binder used by the state highway department in the area should be considered as the base grade for the project (e.g. the grade typically specified in that specific location for dense graded mixes on highways with design Equivalent Standard Axle Loads (ESALS) less than 10 million). The exception would be that grades with a low temperature higher than PG XX-22 should not be used (e.g. PG XX-16 or PG XX-10), unless the Engineer has had successful experience with them. Typically, rutting is not a problem on airport runways. However, at airports with a history of stacking on end of runways and taxiway areas, rutting has accrued due to the slow speed of loading on the pavement. If there has been rutting on the project or it is anticipated that stacking may accrue during the design life of the project, then the following grade "bumping" should be applied for the top 125 mm (5 inches) of paving in the end of runway and taxiway areas: for aircraft tire pressure between 100 and 200 psi, increase the high temperature two grades; for aircraft tire pressure greater than 200 psi, increase the high temperature two grades. Each grade adjustment is 6 degrees C. Polymer Modified Asphalt, PMA, has shown to perform very well in these areas. The low temperature grade should remain the same.

Additional grade bumping and grade selection information is given in Table A.

Table A. Binder Grade Selection and Grade Bumping Based on Gross Aircraft Weight.

Expected Loading	Recommended PG Binder Grade
Airfield pavements with gross aircraft weights < 60,000 lbs (27, 200 kg) and with tire pressures < 100 psi (689 kPa)	The same grade PG binder used by the state highway department in the area should be considered as the base grade for the project (e.g. the grade typically specified in that specific location for dense graded mixes on highways with design Equivalent Standard Axle Loads (ESALS) less than 10 million). The exception would be that grades with a low temperature higher than PG XX-22 should not be used (e.g. PG XX-16 or PG XX-10), unless the Engineer has had successful experience with them.
Airfield pavements with gross aircraft weights > 60,000 lbs (27, 200 kg) and with tire pressures > 100 psi (689 kPa) but < 200 psi (1,378 kPa)	Increase (bump) high temperature by two grades, e.g. PG 76-22 instead of PG 64-22, if PG 64-22 is the base climatic grade.
Taxiways or ends of runways subject to stacking for Airfield pavements with gross aircraft weights > 60,000 lbs (27, 200 kg) and with tire pressures > 100 psi (689 kPa) but < 200 psi (1,378 kPa) or for airfield pavements with design aircraft tire pressures ≥ 200 psi (1,378 kPa).	Increase (bump) high temperature by two grades, e.g. PG 76-22 instead of PG 64-22, if PG 64-22 is the base climatic grade.

Various highway agencies are currently evaluating the multiple stress creep and recovery (MSCR) test for use in the PG binder grading system. This test will better address the unique characteristics of modified binder than the current DSR tests at high temperature. Once the MSCR is implemented in the PG binder grading system, the grade adjustments given in the table above will need to be modified to reflect the changes in the PG binder grading system.

* *

The Contractor shall furnish vendor's certified test reports for each lot of bituminous material shipped to the project. The vendor's certified test report for the bituminous material can be used for acceptance or tested independently by the Engineer.

XXX-2.4 Fibers. Fibers are typically added at 0.3 % by total weight of mix to prevent draindown of the binder during construction. Fibers may be either cellulose or mineral, conforming to Table B or C, respectively.

Table B. Cellulose Fiber Requirements.

Properties	Requirement
Sieve Analysis - Method A Alpine Sieve[1] Analysis Fiber length Passing No. 100 (0.150 mm) sieve	 0.25 in (6 mm) (max) 70 ± 10 percent
Ash Content[2]	18 ± 5 percent non-volatiles
pH[3]	7.5 ± 1.0
Oil Absorption[4]	5.0 ± 1.0 (times fiber weight)
Moisture Content[5]	< 5 percent (by weight)

[1]This test is performed using an Alpine Air Jet Sieve (Type 200 LS). A representative 5 gram sample of fiber is sieved for 14 minutes at a controlled vacuum of 11 psi (75 kPa). The portion remaining on the screen is weighed.

[2]A representative 2-3 gram sample of fiber is placed in a tared crucible and heated between 1100 degrees and 1200 degrees F (595 degrees and 650 degrees C) for not less than 2 hours. The crucible and ash are cooled in a desiccator and reweighed.

[3]Five grams of fiber is added to 100 ml of distilled water, stirred, and let sit for 30 minutes. The pH is determined with a probe calibrated with pH 7.0 buffer.

[4]Five grams of fiber is accurately weighed and suspended in an excess of mineral spirits for not less than 5 minutes to ensure total saturation. It is then placed in a screen mesh strainer (approximately 0.5 square millimeter hole size) and shaken on a wrist–action shaker for 10 minutes (approximately 1-1/4 inch (31-3/4 mm) motion at 240 shakes/minute). The shaken mass is then transferred without touching, to a tared container and weighed. Results are reported as the amount (number of times its own weight) the fibers are able to absorb.

[5]Ten grams of fiber is weighed and placed in a 250 degrees F (121 degree C) forced-air oven for 2 hours. The sample is then reweighed immediately upon removal from the oven.

Table C. Mineral Fiber Requirements.

Properties	Requirement
Sieve Analysis Fiber length[1]	 0.25 inch (6 mm) max mean test value
Thickness[2]	0.0002 inch (0.005 mm) max mean test value
Shot content[3] No. 60 (250 micron) sieve No. 230 (63 micron) sieve	 95 percent passing (min) 65 percent passing (max)

[1]The fiber length is determined according to the Bauer McNett fractionation.

[2]The fiber diameter is determined by measuring at least 200 fibers in a phase contract microscope.

[3]Shot content is a measure of non-fibrous material. The shot content is determined on vibrating sieves. Two sieves, No. 60 and No. 230, are typically utilized; for additional information see ASTM C 612.

XXX-2.4 PRELIMINARY MATERIAL ACCEPTANCE. Prior to delivery of materials to the job site, the Contractor shall submit certified test reports to the Engineer for the following materials:

a. Coarse Aggregate.

(1) Percent of wear.
(2) Soundness.
(3) Voids in Coarse Aggregate (VCA) in Dry Rodded Condition.
(4) Unit Weight of Slag.
(5) Percent Fractured Faces.
(6) Percent Flat, Elongated, and Flat and Elongated Particles.

b. Fine Aggregate.

(1) Liquid limit.
(2) Plasticity index.
(3) Sand equivalent.
(4) Uncompacted voids in fine aggregate

c. Mineral Filler.

d. Bituminous Material. Test results for bituminous material shall include temperature/viscosity charts for mixing and compaction temperatures.

e. Fiber.

The certification(s) shall show the appropriate ASTM test(s) for each material, the test results, and a statement that the material meets the specification requirement.

The Engineer may request samples for testing, prior to and during production, to verify the quality of the materials and to ensure conformance with the applicable specifications.

XXX-2.5 ANTI-STRIPPING AGENT. Any anti-stripping agent or additive if required shall be heat stable, shall not change the asphalt cement viscosity beyond specifications, shall contain no harmful ingredients, shall be added in recommended proportion by approved method, and shall be a material approved by the Department of Transportation of the State in which the project is located.

COMPOSITION

XXX-3.1 COMPOSITION OF MIXTURE. The bituminous plant mix shall be composed of a mixture of gap-graded aggregate, filler and anti-strip agent if required, and bituminous material. The several aggregate fractions shall be sized, handled in separate size groups, and combined in such proportions that the resulting mixture meets the grading requirements of the job mix formula (JMF).

XXX-3.2 JOB MIX FORMULA. No bituminous mixture for payment shall be produced until a job mix formula has been approved in writing by the Engineer. The bituminous mixture shall be designed using procedures contained in Chapter 5, MARSHALL METHOD OF MIX DESIGN, of the Asphalt Institute's Manual Series No. 2 (MS-2), Mix Design Methods for Asphalt Concrete, sixth edition or Chapter 5 SUPERPAVE MIX DESIGN, of the Asphalt Institute's Superpave Series No. 2 (SP-2).

The design criteria in Table 1 are target values necessary to meet the acceptance requirements contained in paragraph XXX-5.2b. The criteria is based on a production process which has a material variability with the following standard deviations:

Tensile Strength Ratio (TSR) of the composite mixture, as determined by ASTM D 4867 with one freeze-thaw cycle, shall not be less than 0.75. Hydrated lime or other anti-stripping agent shall be added to the asphalt, as necessary, to produce a TSR of not less than 0.75. If an anti-strip agent is required, it will be provided by the Contractor at no additional cost to the Owner.

* *

Engineer may specify a TSR of not less than 0.80 in areas that are prone to stripping at a TSR of 0.75. Engineer may specify more than one freeze-thaw conditioning cycles in areas that are prone to stripping.

* *

The job mix formula shall be submitted in writing by the Contractor to the Engineer at least **[ ]** days prior to the start of paving operations and shall include as a minimum:

Percent passing each sieve size for total combined gradation, individual gradation of all aggregate stockpiles and percent by weight of each stockpile used in the job mix formula.

a. Percent of asphalt cement.

b. Asphalt performance and type of modifier if used.

c. Percent and type of fiber.

d. Method of compaction (50 Blow Marshall or 65 design gyrations).

e. Mixing temperature.

f. Compaction temperature.

g. Temperature of mix when discharged from the mixer.

h. Temperature-viscosity relationship of the asphalt cement.

i. Plot of the combined gradation on the Federal Highway Administration (FHWA) 45 power gradation curve.

j. Graphical plots of stability (if applicable), flow (if applicable), air voids, voids in the mineral aggregate, and unit weight versus asphalt content.

k. Voids in Coarse Aggregate Ratio.

l. Percent natural sand.

m. Percent fractured faces.

n. Percent by weight of flat particles**,** elongated particles, and flat and elongated particles (and criteria).

o. Tensile Strength Ratio (TSR).

p. Antistrip agent (if required).

q. Date the job mix formula was developed.

The Contractor shall submit to the Engineer the results of verification testing of three (3) asphalt samples prepared at the optimum asphalt content. The average of the results of this testing shall indicate conformance with the job mix formula requirements specified in Tables 1and 2.

When the project requires asphalt mixtures of differing aggregate gradations, a separate job mix formula and the results of job mix formula verification testing must be submitted for each mix.

The job mix formula for each mixture shall be in effect until a modification is approved in writing by the Engineer. Should a change in sources of materials be made, a new job mix formula must be submitted within [ ] days and approved by the Engineer in writing before the new material is used. After the initial production job mix formula(s) has/have been approved by the Engineer and a new or modified job mix formula is required for whatever reason, the subsequent cost of the Engineer's approval of the new or modified job mix formula will be borne by the Contractor. There will be no time extension given or considerations for extra costs associated with the stoppage of production paving or restart of production paving due to the time needed for the Engineer to approve the initial, new or modified job mix formula.

* *

The Engineer shall specify the number of days. A minimum of 10 days is recommended.

Job mix formula not developed within the previous 90 days are not recommended.

The SMA Mix Design Criteria laboratory compactive effort applicable to the project shall be specified by the Engineer.

* *

TABLE 1. SMA MIX DESIGN CRITERIA

Property	Requirement
Cellulose or mineral fiber	Required
Draindown, ASTM D 6390, at 25 degrees F (13.9 degrees C) above anticipated production temperature	< 0.3 percent
Laboratory compaction effort	**[ ]**
Minimum VMA	17.0 percent
VCA Ratio	< 1.0, VCA_{DRC} determined according to ASTM C 29
Air voids for optimum asphalt content selection	4.0 percent
Acceptance air void range	2.8 to 4.2 percent

* *

The Engineer shall specify a laboratory compaction effort of either 50-Blow Marshall or 65 N_{design} gyrations.

* *

The mineral aggregate shall be of such size that the percentage composition by weight, as determined by laboratory sieves, will conform to the gradation specified in Table 2 when tested in accordance with ASTM C 136 and C 117.

The gradation in Table 2 represent the limits that shall determine the suitability of aggregate for use from the sources of supply. The aggregate, as selected (and used in the JMF), shall have a gradation within the limits designated in Table 2 and shall not vary from the low limit on one sieve to the high limit on the adjacent sieve, or vice versa, but shall be well graded from coarse to fine.

Deviations from the final approved mix design for bitumen content and gradation of aggregates shall be within the action limits for individual measurements as specified in paragraph XXX-6.5a. The limits still will apply if they fall outside the master grading band in Table 2.

The VCA Ratio is the ratio of the VCA_{drc} and the VCA_{mix}. In order to ensure that the SMA mixture maintains stone-on-stone contact, this ratio must be less than one (1.0). The calculations for VCA_{drc} and VCA_{mix} are as follows:

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

where,

G_{CA} = dry bulk specific gravity of the coarse aggregate fraction determined according to ASTM C127,
γ_w = density of water (999 kg/m^3), and
γ_s = Unit weight of coarse aggregate in the dry-rodded condition (kg/m^3).

$$VCA_{mix} = 100 - \left(G_{mb} \div G_{CA}\right)P_{CA}$$

where,

G_{mb} = bulk specific gravity of the compacted SMA sample measured according to AASHTO T166 (ASTM D 2726), and
P_{CA} = percent coarse aggregate (percent retained on the No. 4 (4.75 mm) sieve for SMA mixtures with NMAS greater than ½ inch (12.5 mm)). For SMA mixtures with NMAS of 3/8 inch (9.5 mm), this is the percent retained on the No. 8 (2.36 mm) sieve. Nominal maximum aggregate size (NMAS) is defined as one sieve size larger than the first sieve to cumulatively retain 10 percent. Design gradation ranges are shown in Table 2 for three NMAS.

TABLE 2. Design Gradation

Sieve Size, in (mm)	Percent Passing by Mass		
	3/8 in (9.5 mm) NMAS	½ in (12.5 mm) NMAS	¾ in (19.0 mm) NMAS[1]
3/4 (19.0)	100	100	90-100
1/2 (12.5)	100	90-100	50-88
3/8 (9.5)	70-95	50-85	25-60
No. 4 (4.75)	26-40	20-32	20-28
No. 8 (2.36)	20-28	16-24	16-24
No. 200 (0.075)	8-12	8-12	8-11

[1]Not recommended for use on the wearing surface of the pavement. ¾ in (19.0 mm) SMA has been successfully used on airfields in China and highways in Virginia and Maryland below the pavement surface. Larger NMAS SMA mixtures will tend to have greater macrotexture. Larger NMAS SMA mixtures are more likely to be permeable to water if not properly compacted.

The aggregate gradations shown are based on aggregates of uniform specific gravity. The percentages passing the various sieves shall be corrected when aggregates of varying specific gravities are used, as indicated in the Asphalt Institute Manual Series No. 2 (MS-2), Chapter 3.

*** ***

Where locally available aggregates cannot be economically blended to meet the grading requirements of the gradations shown, the gradations may be modified to fit the characteristics of such local aggregates with approval of the FAA. The modified gradation must produce a paving mixture that satisfies the mix design requirements.

*** ***

XXX-3.3 RECYCLED ASPHALT CONCRETE. Recycled HMA shall not be used in SMA mixtures without the Engineer's approval.
* *

There is limited experience using Reclaimed Asphalt Pavement (RAP) in SMA. A research study on the use of RAP in SMA is reported in:

Watson, D. E., A. Vargas-Nordcbeck, J. Moore, D. Jared, and P. Wu. "Evaluation of the Use of Reclaimed Asphalt Pavement in Stone Matrix Asphalt Mixtures." In Transportation

Research Record No. 2051 Transportation Research Board, National Academies, Washington, DC, 2008, Pp 64-70.

Fractionated RAP could add a beneficial stockpile to allow better gradation control. RAP containing coal tars may require additional precautions during production and may be excluded.

* *

XXX-3.4 TEST SECTION. Prior to full production, the Contractor shall prepare and place a quantity of bituminous mixture according to the job mix formula. The amount of mixture shall be sufficient to construct a test section **[ ]** long and **[ ]** wide**,** placed in two lanes, with a longitudinal cold joint, and shall be of the same depth specified for the construction of the course which it represents. A cold joint is an exposed construction joint at least 4 hours old or whose mat has cooled to less than 160° F. The underlying grade or pavement structure upon which the test section is to be constructed shall be the same as the remainder of the course represented by the test section. The equipment used in construction of the test section shall be the same type and weight to be used on the remainder of the course represented by the test section.

THE TEST SECTION SHALL BE EVALUATED FOR ACCEPTANCE AS A SINGLE LOT IN ACCORDANCE WITH THE ACCEPTANCE CRITERIA IN PARAGRAPH XXX-5.1 AND XXX-6.3. THE TEST SECTION SHALL BE DIVIDED INTO EQUAL SUBLOTS. AS A MINIMUM THE TEST SECTION SHALL CONSIST OF 3 SUBLOTS.

The test section shall be considered acceptable if; 1) mat density, air voids, and joint density are 90 percent or more within limits, 2) gradation, and asphalt content are within the action limits specified in paragraphs XXX-6.5a and 5b, and 3) the voids in the mineral aggregate and VCA ratio are within the limits of Table 1.

If the initial test section should prove to be unacceptable, the necessary adjustments to the job mix formula, plant operation, placing procedures, and/or rolling procedures shall be made. A second test section shall then be placed. If the second test section also does not meet specification requirements, both sections shall be removed at the Contractor's expense. Additional test sections, as required, shall be constructed and evaluated for conformance to the specifications. Any additional sections that are not acceptable shall be removed at the Contractor's expense. Full production shall not begin until an acceptable section has been constructed and accepted in writing by the Engineer. Once an acceptable test section has been placed, payment for the initial test section and the section that meets specification requirements shall be made in accordance with paragraph XXX-8.1.

Job mix control testing shall be performed by the Contractor at the start of plant production and in conjunction with the calibration of the plant for the job mix formula. If aggregates produced by the plant do not satisfy the gradation requirements or produce a mix that meets the JMF. It will be necessary to reevaluate and redesign the mix using plant-produced aggregates. Specimens shall be prepared and the optimum bitumen content determined in the same manner as for the original design tests.

* *

The test section should be a minimum of 300 feet (90 m) long and 20 to 30 feet (6 to 9 m) wide. The test section affords the Contractor and the Engineer an opportunity to determine the quality of the mixture in place, as well as performance of the plant and laydown equipment.

* *

Contractor will not be allowed to place the test section until the Contractor Quality Control Program, showing conformance with the requirements of Paragraph XXX-6.1, has been approved, in writing, by the Engineer.

XXX-3.5 TESTING LABORATORY. The Contractor's laboratory used to develop the job mix formula shall meet the requirements of ASTM D 3666 including the requirement to be accredited by a national authority such as the National Voluntary Laboratory Accreditation Program (NVLAP), the American Association for Laboratory Accreditation (AALA), or AASHTO Accreditation Program (AAP). Laboratory personnel shall meet the requirements of Section 100 of the General Provisions. A certification signed by the manager of the laboratory stating that it meets these requirements shall be submitted to the Engineer prior to the start of construction. The certification shall contain as a minimum:

a. Qualifications of personnel; laboratory manager, supervising technician, and testing technicians.

b. A listing of equipment to be used in developing the job mix.

c. A copy of the laboratory's quality control system.

d. Evidence of participation in the AASHTO Materials Reference Laboratory (AMRL) program.

e. ASTM D 3666 certification of accreditation by a nationally recognized accreditation program.

CONSTRUCTION METHODS

XXX-4.1 WEATHER LIMITATIONS. The bituminous mixture shall not be placed upon a wet surface or when the surface temperature of the underlying course is less than specified in Table 3. The temperature requirements may be waived by the Engineer, if requested; however, all other requirements including compaction shall be met.

TABLE 3. BASE TEMPERATURE LIMITATIONS

Mat Thickness	Base Temperature (Minimum)	
	Deg. F	Deg. C
3 in. (7.5 cm) or greater	40	4
Greater than 1 in. (2.5 cm) but less than 3 in. (7.5 cm)	45	7
1 in. (2.5 cm) or less	50	10

XXX-4.2 BITUMINOUS MIXING PLANT. Plants used for the preparation of bituminous mixtures shall conform to the requirements of ASTM D 995 with the following changes:

a. Requirements for All Plants.

(1) Truck Scales. The bituminous mixture shall be weighed on approved scales furnished by the Contractor, or on certified public scales at the Contractor's expense. Scales shall be inspected and sealed as often as the Engineer deems necessary to assure their accuracy. Scales shall conform to the requirements of the General Provisions, Section 90-01.

In lieu of scales, and as approved by the Engineer, asphalt mixture weights may be determined by the use of an electronic weighing system equipped with an automatic printer that weighs the total paving mixture. Contractor must furnish calibration certification of the weighing system prior to mix production and as often thereafter as requested by the Engineer.

(2) Testing Facilities. The Contractor shall provide laboratory facilities at the plant for the use of the Engineer's acceptance testing and the Contractor's quality control testing. The Engineer will always have priority in the use of the laboratory. The lab shall have sufficient space and equipment so that both testing representatives (Engineer’s and Contractor’s) can operate efficiently. The lab shall also meet the requirements of ASTM D 3666.

The plant testing laboratory shall have a floor space area of not less than 150 square feet, with a ceiling height of not less than 7-½ feet. The laboratory shall be weather tight, sufficiently heated in cold weather, air-conditioned in hot weather to maintain temperatures for testing purposes of 70 degrees F +/- 5 degrees F. The plant testing laboratory shall be located on the plant site to provide an unobstructed view, from one of its windows, of the trucks being loaded with the plant mix materials.

Laboratory facilities shall be kept clean, and all equipment shall be maintained in proper working condition. The Engineer shall be permitted unrestricted access to inspect the Contractor's laboratory facility and witness quality control activities. The Engineer will advise the Contractor in writing of any noted deficiencies concerning the laboratory facility, equipment, supplies, or testing personnel and procedures. When the deficiencies are serious enough to be adversely affecting the test results, the incorporation of the materials into the work shall be suspended immediately and will not be permitted to resume until the deficiencies are satisfactorily corrected.

As a minimum, the plant testing laboratory shall have:

(a) Adequate artificial lighting
(b) Electrical outlets sufficient in number and capacity for operating the required testing equipment and drying samples.
(c) Fire extinguishers (2), Underwriter's Laboratories approved
(d) Work benches for testing, minimum 2-½ feet by 10 feet.
(e) Desk with 2 chairs
(f) Sanitary facilities convenient to testing laboratory
(g) Exhaust fan to outside air, minimum 12 inch blade diameter
(h) A direct telephone line and telephone including a FAX machine operating 24 hours per day, seven days per week
(i) File cabinet with lock for Engineer
(j) Sink with running water, attached drain board and drain capable of handling separate material
(k) Metal stand for holding washing sieves
(l) Two element hot plate or other comparable heating device, with dial type thermostatic controls for drying aggregates
(m) Mechanical shaker and appropriate sieves (listed in JMF, Table 3) meeting the requirements of ASTM E-11 for determining the gradation of coarse and fine aggregates in accordance with ASTM C 136
(n) Marshall testing equipment meeting ASTM D 6926, ASTM D 6927, automatic compaction equipment capable of compacting three specimens at once and other apparatus as specified in ASTM C 127, D 2172, D 2726, and D 2041
(o) Oven, thermostatically controlled, inside minimum 1 cubic foot
(p) Two volumetric specific gravity flasks, 500 cc
(q) Other necessary hand tools required for sampling and testing
(r) Library containing contract specifications, latest ASTM volumes 4.01, 4.02, 4.03 and 4.09, AASHTO standard specification parts I and II, and Asphalt Institute Publication MS-2.
(s) Equipment for Theoretical Specific Gravity testing including a 4,000 cc pycnometer, vacuum pump capable of maintaining 30 ml mercury pressure and a balance, 16-20 kilograms with accuracy of 0.5 grams
(t) Extraction equipment, centrifuge and reflux types and ROTOflex equipment
(u) A masonry saw with diamond blade for trimming pavement cores and samples
(v) Telephone

Approval of the plant and testing laboratory by the Engineer requires all facilities and equipment to be in good working order during production, sampling and testing. Failure to provide the specified facilities shall be sufficient cause for disapproving bituminous plant operations.

The Owner shall have access to the lab and the plant whenever Contractor is in production.

(3) Inspection of Plant. The Engineer, or Engineer's authorized representative, shall have access, at all times, to all areas of the plant for checking adequacy of equipment; inspecting operation of the plant: verifying weights, proportions, and material properties; and checking the temperatures maintained in the preparation of the mixtures.

(4) Storage Bins and Surge Bins. Use of surge and storage bins for temporary storage of hot bituminous mixtures will be permitted as follows:

(a) The bituminous mixture may be stored in surge bins for ***a*** period of time not to exceed 3 hours.

(b) The bituminous mixture may be stored in insulated storage bins for a period of time not to exceed 24 hours**.**

The bins shall be such that mix drawn from them meets the same requirements as mix loaded directly into trucks.

If the Engineer determines that there is an excessive amount of heat loss, segregation**,** or oxidation of the mixture due to temporary storage, no temporary storage will be allowed.

XXX-4.3 HAULING EQUIPMENT. Trucks used for hauling bituminous mixtures shall have tight, clean, and smooth metal beds. To prevent the mixture from adhering to them, the truck beds shall be lightly coated with a minimum amount of paraffin oil, lime solution, or other approved material. Petroleum products shall not be used for coating truck beds. Each truck shall have a suitable cover to protect the mixture from adverse weather. When necessary, to ensure that the mixture will be delivered to the site at the specified temperature, truck beds shall be insulated or heated and covers shall be securely fastened.

XXX-4.4 MATERIAL TRANSFER DEVICE. A self-propelled materials transfer vehicle with a minimum of 15 tons storage capability shall be used to feed the paver. The materials transfer vehicle shall have integral remixing capability. A paver insert with a minimum capacity of 12 tons shall be used in the paver.

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The use of a materials transfer vehicle is strongly encouraged. However, SMA pavements have been successfully constructed without a transfer vehicle. The use of a transfer vehicle and paver insert may be waived by the Engineer.

****

XXX-4.5 BITUMINOUS PAVERS. Bituminous pavers shall be self-propelled with an activated heated screed, capable of spreading and finishing courses of bituminous plant mix material that will meet the specified thickness, smoothness, and grade. The paver shall have sufficient power to propel itself and the hauling equipment without adversely affecting the finished surface.

The paver shall have a receiving hopper of sufficient capacity to permit a uniform spreading operation. The hopper shall be equipped with a distribution system to place the mixture uniformly in front of the screed without segregation. The screed shall effectively produce a finished surface of the required evenness and texture without tearing, shoving, or gouging the mixture.

The paver shall be equipped with a control system capable of automatically maintaining the specified screed elevation. The control system shall be automatically actuated from either a reference line and/or through a system of mechanical sensors or sensor-directed mechanisms or devices that will maintain the paver screed at a predetermined transverse slope and at the proper elevation to obtain the required surface. The transverse slope controller shall be capable of maintaining the screed at the desired slope within plus or minus 0.1 percent.

The controls shall be capable of working in conjunction with any of the following attachments:

a. Ski-type device of not less than 30 feet (9.14 m) in length.

b. Taut stringline (wire) set to grade.

c. Short ski or shoe.

d. Laser control.

If, during construction, it is found that the spreading and finishing equipment in use leaves tracks or indented areas, or produces other blemishes in the pavement that are not satisfactorily corrected by the scheduled operations, the use of such equipment shall be discontinued and satisfactory equipment shall be provided by the Contractor.

XXX-4.6 ROLLERS. Rollers of the vibratory, steel wheel, and pneumatic-tired type shall be used. They shall be in good condition, capable of operating at slow speeds to avoid displacement of the bituminous mixture. The number, type, and weight of rollers shall be sufficient to compact the mixture to the required density while it is still in a workable condition.

All rollers shall be specifically designed and suitable for compacting hot mix bituminous concrete and shall be properly used. Rollers that impair the stability of any layer of a pavement structure or underlying soils shall not be used. Depressions in pavement surfaces caused by rollers shall be repaired by the Contractor at its own expense.

The use of equipment that causes crushing of the aggregate will not be permitted.

a. Nuclear Densometer. The Contractor shall have on site a nuclear densometer during all paving operations in order to assist in the determination of the optimum rolling pattern, type of roller and frequencies, as well as to monitor the effect of the rolling operations during production paving. The Contractor shall also supply a qualified technician during all paving operations to calibrate the nuclear densometer and obtain accurate density readings for all new bituminous concrete. These densities shall be supplied to the Engineer upon request at any time during construction. No separate payment will be made for supplying the density gauge and technician.

XXX-4.7 PREPARATION OF BITUMINOUS MATERIAL. The bituminous material shall be heated in a manner that will avoid local overheating and provide a continuous supply of the bituminous material to the mixer at a uniform temperature. The temperature of the bituminous material delivered to the mixer shall be sufficient to provide a suitable viscosity for adequate coating of the aggregate particles, but shall not exceed 325 degrees F (160 degrees C), unless otherwise required by the manufacturer.

XXX-4.8 PREPARATION OF MINERAL AGGREGATE. The aggregate for the mixture shall be heated and dried prior to introduction into the mixer. The maximum temperature and rate of heating shall be such that no damage occurs to the aggregates. The temperature of the aggregate and mineral filler shall not exceed 350 degrees F (175 degrees C) when the asphalt is added. Particular care shall be taken that aggregates high in calcium or magnesium content are not damaged by overheating. The temperature shall not be lower than is required to obtain complete coating and uniform distribution on the aggregate particles and to provide a mixture of satisfactory workability.

XXX-4.9 PREPARATION OF BITUMINOUS MIXTURE. The aggregates and the bituminous material shall be weighed or metered and introduced into the mixer in the amount specified by the job mix formula.

The combined materials shall be mixed until the aggregate obtains a uniform coating of bitumen and is thoroughly distributed throughout the mixture. Wet mixing time shall be the shortest time that will produce a satisfactory mixture, but not less than 25 seconds for batch plants. The wet mixing time for all plants shall be established by the Contractor, based on the procedure for determining the percentage of coated particles described in ASTM D 2489, for each individual plant and for each type of aggregate used. The wet mixing time will be set to achieve 95 percent of coated particles. For continuous mix plants, the minimum mixing time shall be determined by dividing the weight of its contents at operating level by the weight of the mixture delivered per second by the mixer. The moisture content of all bituminous mixtures upon discharge shall not exceed 0.5 percent.

* *

For batch plants, wet mixing time begins with the introduction of bituminous material into the mixer and ends with the opening of the mixer discharge gate. Distribution of aggregate

and bituminous material as they enter the pugmill, speed of mixer shafts, and arrangement and pitch of paddles are factors governing efficiency of mixing. Prolonged exposure to air and heat in the pugmill harden the asphalt film on the aggregate. Mixing time, therefore, should be the shortest time required to obtain uniform distribution of aggregate sizes and thorough coating of aggregate particles with bituminous material.

*** ***

XXX-4.10 PREPARATION OF THE UNDERLYING SURFACE. Immediately before placing the bituminous mixture, the underlying course shall be cleaned of all dust and debris. A prime coat or tack coat shall be applied in accordance with Item P-602 or P-603, if shown on the plans.

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Engineer should evaluate the presence of paint and/or rubber deposits on the existing pavement and, if needed, may specify milling, grinding or other suitable means to remove same prior to placement of new bituminous material.

*** ***

XXX-4.11 LAYDOWN PLAN, TRANSPORTING, PLACING, AND FINISHING. Prior to the placement of the bituminous mixture, the Contractor shall prepare a laydown plan for approval by the Engineer. This is to minimize the number of cold joints in the pavement. The laydown plan shall include the sequence of paving laydown by stations, width of lanes, temporary ramp location(s), and laydown temperature. The laydown plan shall also include estimated time of completion for each portion of the work (i.e. milling, paving, rolling, cooling, etc.). Modifications to the laydown plan shall be approved by the Engineer.

The bituminous mixture shall be transported from the mixing plant to the site in vehicles conforming to the requirements of paragraph XXX-4.3. Deliveries shall be scheduled so that placing and compacting of mixture is uniform with minimum stopping and starting of the paver. Hauling over freshly placed material shall not be permitted until the material has been compacted, as specified, and allowed to cool to atmospheric temperature.

*** ***

Engineer may, at his option, add the following language:

"For all runway, taxiway and apron pavements, Contractor shall use a stringline to place each lane of each lift of bituminous surface course. However, at the Contractor's option, Contractor shall use stringline for first lift of bituminous surface course and then survey the grade of that lift. Provided grades of that lift of bituminous surface course meet the tolerances of paragraphs XXX-5.2b(6), then Contractor may place successive lifts of bituminous surface course using a long ski, or laser control per paragraph XXX-4.5. However, Contractor shall survey each lift of bituminous surface course and certify to Engineer that every lot of each lift meets the grade tolerances of paragraph XXX-5.2b(6) before the next lift can be placed without a stringline. If the grades of a single lot do not meet the tolerances of XXX-5.2b(6), then the Contractor shall use a stringline for each entire lift. Corrective action in paragraph XXX-5.2b(6) applies to the final lift of surface course; however, for multiple lift construction, the Contractor shall correct to ensure the final lift of surface course is a minimum of [ ] inches and a maximum of [ ] inches." (Engineer to specify minimum and maximum tolerances for final lift of surface course)

Paving during nighttime construction shall require the following:

a. All paving machines, rollers, distribution trucks and other vehicles required by the Contractor for his operations shall be equipped with artificial illumination sufficient to safely complete the work.

b. Minimum illumination level shall be twenty (20) horizontal foot candles and maintained in the following areas:

(1) An area of 30 feet wide by 30 feet long immediately behind the paving machines during the operations of the machines.

(2) An area 15 feet wide by 30 feet long immediately in front and back of all rolling equipment, during operation of the equipment.

(3) An area 15 feet wide by 15 feet long at any point where an area is being tack coated prior to the placement of pavement.

c. As partial fulfillment of the above requirements, the Contractor shall furnish and use, complete artificial lighting units with a minimum capacity of 3,000 watt electric beam lights, affixed to all equipment in such a way to direct illumination on the area under construction.

d. In addition, the Contractor shall furnish [ ] portable floodlight units similar or equal to [ ].

*** ***

Engineer to specify the minimum number of floodlighting units and may elect to specify a particular manufacturer's lighting unit "or equal".

If nighttime paving requires the critical re-opening of airfield facilities, the following additional language should be added:

> **"If the Contractor places any out of specification mix in the project work area, the Contractor is required to remove it at its own expense, to the satisfaction of the Engineer. If the Contractor has to continue placing non-payment bituminous concrete, as directed by the Engineer, to make the surfaces safe for aircraft operations, the Contractor shall do so to the satisfaction of the Engineer. It is the Contractor's responsibility to leave the facilities to be paved in a safe condition ready for aircraft operations. No consideration for extended closure time of the area being paved will be given. As a first order of work for the next paving shift, the Contractor shall remove all out of specification material and replace with approved material to the satisfaction of the Engineer. When the above situations occur, there will be no consideration given for additional construction time or payment for extra costs."**

*** ***

The initial placement and compaction of the mixture shall occur at a temperature suitable for obtaining density, surface smoothness, and other specified requirements but not less than 250 degrees F (121 degrees C).
Edges of existing bituminous pavement abutting the new work shall be saw cut and carefully removed as shown on the drawings and painted with bituminous tack coat before new material is placed against it.

Upon arrival, the mixture shall first be introduced to a material transfer device, then be placed to the full width by a bituminous paver. It shall be struck off in a uniform layer of such depth that, when the work is completed, it shall have the required thickness and conform to the grade and contour indicated. The speed of the paver shall be regulated to eliminate pulling and tearing of the bituminous mat. Unless otherwise permitted, placement of the mixture shall begin along the centerline of a crowned section or on the high side of areas with a one-way slope. The mixture shall be placed in consecutive adjacent strips having a minimum width of **[ ]** except where edge lanes require less width to complete the area. Additional screed sections shall not be attached to widen paver to meet the minimum lane width requirements specified above unless additional auger sections are added to match. The longitudinal joint in one course shall offset the longitudinal joint in the course immediately below by at least 1 foot

(30 cm); however, the joint in the surface top course shall be at the centerline of crowned pavements. Transverse joints in one course shall be offset by at least 10 feet (3 m) from transverse joints in the previous course.

Transverse joints in adjacent lanes shall be offset a minimum of 10 feet (3 m).

On areas where irregularities or unavoidable obstacles make the use of mechanical spreading and finishing equipment impractical, the mixture may be spread and luted by hand tools. Areas of segregation in the surface course, as determined by the Engineer, shall be removed and replaced at the Contractor's expense. The area shall be removed by saw cutting and milling a minimum of 2 inches deep. The area to be removed and replaced shall be a minimum width of the paver and a minimum of 10 feet long.

* *

The Engineer should add more detail as appropriate to areas that require removal and replacements. The Engineer should specify the widest paving lane practicable in an effort to hold the number of longitudinal joints to a minimum.

* *

XXX-4.12 COMPACTION OF MIXTURE. After placing, the mixture shall be thoroughly and uniformly compacted by power rollers. The surface shall be compacted as soon as possible when the mixture has attained sufficient stability so that the rolling does not cause undue displacement, cracking or shoving. The sequence of rolling operations and the type of rollers used shall be at the discretion of the Contractor. The speed of the roller shall, at all times, be sufficiently slow to avoid displacement of the hot mixture and be effective in compaction. Any displacement occurring as a result of reversing the direction of the roller, or from any other cause, shall be corrected at once.

Sufficient rollers shall be furnished to handle the output of the plant. Rolling shall continue until the surface is of uniform texture, true to grade and cross section, and the required field density is obtained.

To prevent adhesion of the mixture to the roller, the wheels shall be equipped with a scraper and kept properly moistened but excessive water will not be permitted.

In areas not accessible to the roller, the mixture shall be thoroughly compacted with approved power driven tampers. Tampers shall weigh not less than 275 pounds, have a tamping plate width not less than 15 inches, be rated at not less than 4,200 vibrations per minute, and be suitably equipped with a standard tamping plate wetting device.

Any mixture that becomes loose and broken, mixed with dirt, contains check-cracking, or in any way defective shall be removed and replaced with fresh hot mixture and immediately compacted to conform to the surrounding area. This work shall be done at the Contractor's expense. Skin patching shall not be allowed.

XXX-4.13 JOINTS. The formation of all joints shall be made in such a manner as to ensure a continuous bond between the courses and obtain the required density. All joints shall have the same texture as other sections of the course and meet the requirements for smoothness and grade.

The roller shall not pass over the unprotected end of the freshly laid mixture except when necessary to form a transverse joint. When necessary to form a transverse joint, it shall be made by means of placing a bulkhead or by tapering the course. The tapered edge shall be cut back to its full depth and width on a straight line to expose a vertical face prior to placing the adjacent lane. In both methods, all contact surfaces shall be given a tack coat of bituminous material before placing any fresh mixture against the joint.

Longitudinal joints which are irregular, damaged, uncompacted, or otherwise defective **[**or which have been left exposed for more than 4 hours, or whose surface temperature has cooled to less than 160° F**]** shall be cut back [specify cutback] to expose a clean, sound surface for the full depth of the course. All contact surfaces shall be cleaned and dry prior and given a tack coat of bituminous material prior to placing any fresh mixture against the joint. The cost of this work and tack coat shall be considered incidental to the cost of the bituminous course.

* *

Engineer may retain the bracketed language regarding the treatment of "cold joints" when considered necessary. The cutback should be no more than 6 inches.

* *

XXX-4.14 SKID RESISTANT SURFACES/SAW-CUT GROOVING. If shown on the plans, skid resistant surfaces for asphalt pavements shall be provided by construction of saw-cut grooves. Pavement shall be sufficiently cooled prior to grooving.

Transverse grooves shall be saw-cut in the pavement forming a ¼ inch wide by ¼ inch deep by 1-½ inches center to center configuration. The grooves shall be continuous for the entire length of the pavement. They shall be saw-cut transversely in the pavement to within 10 feet of the pavement edge to allow adequate space for equipment operation. The tolerances for saw-cut grooves shall meet the following:

a. Alignment tolerance – Plus or minus 1-½ inches in alignment for 75 feet.

b. Groove tolerance – Minimum depth 3/16 inch, except that not more than 60 percent of the grooves shall be less than ¼ inch. Maximum depth 5/16 inch. Minimum width ¼ inch. Maximum width 5/16 inch.

c. Center-to-center spacing – Minimum spacing 1-⅜ inches. Maximum spacing 1-½ inches.

Grooves shall not be less than 6 inches and not more than 18 inches from in-pavement light fixtures. Cleanup of waste material shall be continuous during the grooving operation. All waste material shall be removed from the pavement surface and disposed of off-site in accordance with governing laws and regulations. All arrangements for disposal of waste material shall be made prior to the start of grooving. Waste material shall not be allowed to enter the airport storm or sanitary sewer system.

MATERIAL ACCEPTANCE

XXX-5.1 ACCEPTANCE SAMPLING AND TESTING. Unless otherwise specified, all acceptance sampling and testing necessary to determine conformance with the requirements specified in this section will be performed by the Engineer at no cost to the Contractor except that coring **[**and profilograph testing**]** as required in this section shall be completed and paid for by the Contractor. Testing organizations performing these tests **[**except profilograph**]** shall meet the requirements of ASTM D 3666. All equipment in Contractor furnished laboratories shall be calibrated by an independent testing organization prior to the start of operations at the Contractor's expense.

* *

See note to Engineer in section XXX-5.2b(5) regarding the use of profilograph testing. If this testing is specified, it is performed and paid for by the Contractor.

* *

a. Plant-Produced Material. Plant-produced material shall be tested for air voids and VCA ratio on a lot basis. Sampling shall be from material deposited into trucks at the plant or from trucks at the job site. Samples shall be taken in accordance with ASTM D 979. A lot will consist of:

- one day or shift's production not to exceed 2,000 tons (1 814 000 kg), or
- a half day or shift's production where a day's production is expected to consist of between 2,000 and 4,000 tons (1 814 000 and 3 628 000 kg), or
- similar subdivisions for tonnages over 4,000 tons (3 628 000 kg).

Where more than one plant is simultaneously producing material for the job, the lot sizes shall apply separately for each plant.

(1) Sampling. Each lot will consist of four equal sublots. Sufficient material for preparation of test specimens for all testing will be sampled by the Engineer on a random basis, in accordance with the procedures contained in ASTM D 3665. One set of laboratory compacted specimens will be prepared for each sublot in accordance with ASTM D 6926, at the number of blows required by paragraph XXX-3.2, Table 1. Each set of laboratory compacted specimens will consist of three test portions prepared from the same sample increment.

The sample of bituminous mixture may be put in a covered metal tin and placed in an oven for not less than 30 minutes nor more than 60 minutes to stabilize to compaction temperature. The compaction temperature of the specimens shall be as specified in the job mix formula.

* *

Engineer should increase hold times to not less than 60 minutes and not more than 90 minutes when absorptive aggregates are used,

* *

(2) Testing. Air voids will be determined in accordance with ASTM D 3203. VCA ratio shall be determined as described previously.

Prior to testing, the bulk specific gravity of each test specimen shall be measured by the Engineer in accordance with ASTM D 2726 using the procedure for laboratory-prepared thoroughly dry specimens, or ASTM D 1188, whichever is applicable, for use in computing air voids and pavement density.

For air voids determination, the theoretical maximum specific gravity of the mixture shall be measured one time for each sublot in accordance with ASTM D 2041, Type C, D or E container. The value used in the air voids computation for each sublot shall be based on theoretical maximum specific gravity measurement for the sublot.

(3) Acceptance. Acceptance of plant produced material for air voids and VCA ratio shall be determined by the Engineer in accordance with the requirements of paragraph XXX-5.2b.

b. Field Placed Material. Material placed in the field shall be tested for mat and joint density on a lot basis.

(1) Mat Density. The lot size shall be the same as that indicated in paragraph XXX-5.1a and shall be divided into four equal sublots. One core of finished, compacted materials shall be taken by the Contractor from each sublot. Core locations will be determined by the Engineer on a random basis in accordance with procedures contained in ASTM D 3665. Cores shall not be taken closer than one foot from a transverse or longitudinal joint.

(2) Joint Density. The lot size shall be the total length of longitudinal joints constructed by a lot of material as defined in paragraph XXX-5.1a. The lot shall be divided into four equal sublots. One core of finished, compacted materials shall be taken by the Contractor from each sublot. Core locations will be determined by the Engineer on a random basis in accordance with procedures contained in ASTM D 3665. ALL CORING SHALL BE CENTERED ON THE JOINT. THE MINIMUM CORE DIAMETER FOR JOINT DENSITY DETERMINATION SHALL BE 5 INCHES (125 mm).

(3) Sampling. Samples shall be neatly cut with a core drill. The cutting edge of the core drill bit shall be of hardened steel or other suitable material with diamond chips embedded in the metal cutting edge. The minimum diameter of the sample shall be five inches. Samples that are clearly defective, as a result of sampling, shall be discarded and another sample taken. The Contractor shall furnish all tools, labor, and materials for cutting samples, cleaning, and filling the cored pavement. Cored pavement shall be cleaned and core holes shall be filled in a manner acceptable to the Engineer and within one day after sampling.

(4) Testing. The bulk specific gravity of each cored sample will be measured by the Engineer in accordance with ASTM D 2726 or ASTM D 1188, whichever is applicable. The percent compaction (density) of each sample will be determined by dividing the bulk specific gravity of each sublot sample by the average bulk specific gravity of all laboratory prepared specimens for the lot, as determined in paragraph XXX-5.1a(2). The bulk specific gravity used to determine the joint density at joints formed between different lots shall be the lowest of the bulk specific gravity values from the two different lots.

(5) Acceptance. Acceptance of field placed material for mat density will be determined by the Engineer in accordance with the requirements of paragraph XXX-5.2b(1). Acceptance for joint density will be determined in accordance with the requirements of paragraph XXX-5.2b(3).

c. Partial Lots — Plant-Produced Material. When operational conditions cause a lot to be terminated before the specified number of tests have been made for the lot, or when the Contractor and Engineer agree in writing to allow overages or other minor tonnage placements to be considered as partial lots, the following procedure will be used to adjust the lot size and the number of tests for the lot.

The last batch produced where production is halted will be sampled, and its properties shall be considered as representative of the particular sublot from which it was taken. In addition, an agreed to minor placement will be sampled, and its properties shall be considered as representative of the particular sublot from which it was taken. Where three sublots are produced, they shall constitute a lot. Where one or two sublots are produced, they shall be incorporated into the next lot, and the total number of sublots shall be used in the acceptance plan calculation, i.e., n = 5 or n = 6, for example. Partial lots at the end of asphalt production on the project shall be included with the previous lot.

d. Partial Lots — Field Placed Material. The lot size for field placed material shall correspond to that of the plant material, except that, in no cases, shall less than three (3) cored samples be obtained, i.e., n = 3.

XXX-5.2 ACCEPTANCE CRITERIA.

a. General. Acceptance will be based on the following characteristics of the bituminous mixture and completed pavement as well as the implementation of the Contractor Quality Control Program and test results:

(1) Air voids
(2) VCA Ratio
(3) Mat density
(4) Joint density
(5) Thickness
(6) Smoothness
(7) Grade

Mat density and air voids will be evaluated for acceptance in accordance with paragraph XXX-5.2b(1). VCA Ratio will be evaluated for acceptance in accordance Table 1. Joint density will be evaluated for acceptance in accordance with paragraph XXX-5.2b(3).

Thickness will be evaluated by the Engineer for compliance in accordance with paragraph XXX-5.2b(4). Acceptance for smoothness will be based on the criteria contained in paragraph XXX-5.2b(5). Acceptance for grade will be based on the criteria contained in paragraph XXX-5.2b(6).

The Engineer may at any time, notwithstanding previous plant acceptance, reject and require the Contractor to dispose of any batch of bituminous mixture which is rendered unfit for use due to contamination, segregation, incomplete coating of aggregate, or improper mix temperature. Such rejection may be based on only visual inspection or temperature measurements. In the event of such rejection, the Contractor may take a representative sample of the rejected material in the presence of the Engineer, and if it can be demonstrated in the laboratory, in the presence of the Engineer, that such material was erroneously rejected, payment will be made for the material at the contract unit price.

b. Acceptance Criteria.

(1) Mat Density and Air Voids. Acceptance of each lot of plant produced material for mat density and air voids shall be based on the percentage of material within specification limits (PWL). If the PWL of the lot equals or exceeds 90 percent, the lot shall be acceptable. Acceptance and payment shall be determined in accordance with paragraph XXX-8.1.

(2) VCA Ratio. VCA Ratio < 1.0.

(3) Joint Density. Acceptance of each lot of plant produced material for joint density shall be based on the percentage of material within specification limits (PWL). If the PWL of the lot is equal to or exceeds 90 percent, the lot shall be considered acceptable. If the PWL is less than 90 percent, the Contractor shall evaluate the reason and act accordingly. If the PWL is less than 80 percent, the Contractor shall cease operations and until the reason for poor compaction has been determined. IF THE PWL IS LESS THAN 71 PERCENT, THE PAY FACTOR FOR THE LOT USED TO COMPLETE THE JOINT SHALL BE REDUCED BY 5 PERCENTAGE POINTS. This lot pay factor reduction shall be incorporated and evaluated in accordance with paragraph XXX-8.1.

(4) Thickness. Thickness of each lift of surface course shall be evaluated by the Engineer for compliance to the requirements shown on the plans. Measurements of thickness shall be made by the Engineer using the cores extracted for each sublot for density measurement. The maximum allowable deficiency at any point shall not be more than ¼ inch less than the thickness indicated for the lift. Average thickness of lift, or combined lifts, shall not be less than the indicated thickness. Where the thickness tolerances are not met, the lot or sublot shall be corrected by the Contractor at his expense by removing the deficient area and replacing with new pavement. The Contractor, at his expense, may take additional cores as approved by the Engineer to circumscribe the deficient area.

(5) Smoothness. The final surface shall be free from roller marks. The finished surfaces of each course of the pavement, except the finished surface of the final course, shall not vary more than ⅜ inch when evaluated with a 16 foot straightedge. The finished surface of the final course of pavement shall not vary more than ¼ inch when evaluated with a 16 foot straightedge. The lot size shall be [ ] square yards (square meters). Smoothness measurements shall be made at 50 foot intervals and as determined by the Engineer. In the longitudinal direction, a smoothness reading shall be made at the center of each paving lane. In the transverse direction, smoothness readings shall be made continuously across the full width of the pavement. However, transverse smoothness readings shall not be made across designed grade changes. At warped transition areas, straightedge position shall be adjusted to measure surface smoothness and not design grade transitions. When more than 15 percent of all measurements within a lot exceed the specified tolerance, the Contractor shall remove the deficient area to the depth of the final course of pavement and replace with new material. Skin patching shall not be permitted. Isolated high points may be ground off providing the course thickness complies with the thickness specified on the plans. High point grinding will be limited to 15 square yards. Areas in excess of 15 square yards will require removal and replacement of the pavement in accordance with the limitations noted above.

* *

The Engineer shall specify the lot size. A minimum of 2,000 square yards (1 650 square meters) is recommended.

Use of a profilograph can be included in the specifications for surface smoothness for runways and taxiways on a case by case basis provided it is approved by the FAA. Use of a profilograph may not be practical for all asphalt construction. Thin lift overlays and other minimum resurfacing may not allow for removal of existing pavement roughness. However, the use of the profilograph is recommended for new construction or overlays designed to correct grade and smoothness deficiencies. If the profilograph is to be included, straightedge requirements need only apply to the perpendicular direction. To include profilograph requirements, add ASTM E 1274 to the referenced testing list and add the following:

(a) Profilograph. The Contractor shall furnish a 25 foot wheel base California type profilograph and competent operator to measure pavement surface deviations. The profilograph shall be operated in accordance with the manufacturer's instructions and at a speed no greater than 3 mph. Original profilograms for the appropriate locations interpreted in accordance with ASTM E 1274 shall be furnished to the Engineer. The profilograms shall be recorded on a scale of one inch equal to 25 feet longitudinally and one inch equal to one inch (or full scale) vertically. Profilographs shall be calibrated prior to testing.

The surface of the runway and/or taxiway pavements of continuous placement of 50 feet or more shall be tested and evaluated as described herein. One pass along the centerline shall be required for each paving lane. Runs shall be continuous through a day's production. Each trace shall be completely labeled to show paving lane and stationing.

The Contractor shall furnish paving equipment and employ methods that produce a riding surface for each section of pavement having an average profile index meeting the requirements of Table 7. A typical section will be considered to be the width of the paving lane and 1/10 of a mile long. The profile index will be determined in accordance with ASTM E 1274. A blanking band of 0.2 inches shall be used. Within each 1/10 mile section, all areas represented by high points having a deviation in excess of 0.4 inches in 25 feet or less shall be removed by the Contractor using an approved method. After removing all individual deviations in excess of 0.4 inches, additional corrective work shall be performed if necessary to achieve the required ride quality. All corrective work shall be completed prior to determination of pavement thickness.

On pavement sections where corrections were necessary, second profilograph runs shall be performed to verify that the corrections have produced an average profile index of 15 inches per mile or less. If the initial average profile index was less than 15, only those areas representing greater than 0.4 inch deviation will be re-profiled for correction verification.

Individual sections shorter than 50 feet and the last 15 feet of any section where the Contractor is not responsible for the adjoining section shall be straightedged in accordance with paragraph XXX-5.2b(5).

If there is a section of 250 feet or less, the profilogram for the section shall be included in the evaluation of the previous section. If there is an independently placed section of 50 to 250 feet in length, a profilogram shall be made for that section and the pay adjustment factors for short section of Table 7 shall apply.

All costs necessary to provide the profilograph and related to furnishing the appropriate profilograms as required in this provision are incidental to pavement construction and no direct compensation will be made therefore.

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(6) Grade. The finished surface of the pavement shall not vary from the gradeline elevations and cross sections shown on the plans by more than ½ inch (12.70 mm). The finished grade of each lot will be determined by running levels at intervals of 50 feet (15.2 m) or less longitudinally and all breaks in grade transversely (not to exceed 50 feet) to determine the elevation of the completed pavement. The Contractor shall pay the cost of surveying of the level runs that shall be performed by a licensed surveyor. The documentation, stamped and signed by a licensed surveyor, shall be provided by the Contractor to the Engineer. The lot size shall be **[ ]** square yards (square meters). When more than 15 percent of all the measurements within a lot are outside the specified tolerance, or if any one shot within the lot deviates ¾ inch or more from planned grade, the Contractor shall remove the deficient area to the depth of the final course of pavement and replace with new material. Skin patching shall not be permitted. Isolated high points may be ground off providing the course thickness complies with the thickness specified on the plans. The surface of the ground pavement shall have a texture consisting of grooves between 0.090 and 0.130 inches wide. The peaks and ridges shall be approximately 1/32 inch higher than the bottom of the grooves. The pavement shall be left in a clean condition. The removal of all of the slurry resulting form the grinding operation shall be continuous The grinding operation should be controlled so the residue from the operation does not flow across other lanes of pavement. High point grinding will be limited to 15 square yards. Areas in excess of 15 square yards will require removal and replacement of the pavement in accordance with the limitations noted above.

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A minimum of 2,000 square yards (1 650 square meters) is recommended.

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c. Percentage of Material Within Specification Limits (PWL). The percentage of material within specification limits (PWL) shall be determined in accordance with procedures specified in Section 110 of the General Provisions. The specification tolerance limits (L) for lower and (U) for upper are contained in Table 5.

d. Outliers. All individual tests for mat density and air voids shall be checked for outliers (test criterion) in accordance with ASTM E 178, at a significance level of 5 percent. Outliers shall be discarded, and the PWL shall be determined using the remaining test values.

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The specification tolerance limits applicable to the project, based on design criteria specified in Table 1, shall be specified by the Engineer from the information shown below and inserted into Table 5. Asterisks denote insert points.

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TABLE 5. ACCEPTANCE LIMITS FOR AIR VOIDS AND MAT DENSITY

Property	Requirement
Acceptance air void range	2.8 to 4.2
In-place density Specification Tolerance Limit for Mat Density, L	**[***]**
In-place density Specification Tolerance Limit for Joint Density, L	**[***]**

NOTE TO ENGINEER. The Engineer may specify both upper and lower PWL acceptance criteria (two-sided) for density. Use 101.3 as the Upper tolerance limits when two-sided density acceptance criteria is specified AND insert edit paragraph XXX-8.1. See Notes to Engineer following paragraph XXX-8.1. The Engineer may specify a lower limit for in-place mat density specification of 96.8 percent of G_{mb} or 93.5 percent of G_{mm}. Similarly, the Engineer may specify a lower limit for in-place joint density of 93.9 percent of G_{mb} or 90.5 percent of G_{mm}.

*** ***

A lot is the quantity of material to be controlled and may represent a specified tonnage or a specified number of truckloads. The lot size, to be determined by the Engineer, should, for the most part, depend on the operational capacity of the plant, but shall in no case exceed 2,000 tons (1 814 000 kg) in accordance with paragraph XXX-5.1a.

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XXX-5.3 RESAMPLING PAVEMENT FOR MAT DENSITY.

a. General. Resampling of a lot of pavement will only be allowed for mat density**,** and then, only if the Contractor requests same, in writing, within 48 hours after receiving the written test results from the Engineer. A retest will consist of all the sampling and testing procedures contained in paragraphs XXX-5.1b and XXX-5.2b(1). Only one resampling per lot will be permitted.

(1) A redefined PWL shall be calculated for the resampled lot. The number of tests used to calculate the redefined PWL shall include the initial tests made for that lot plus the retests.

(2) The cost for resampling and retesting shall be borne by the Contractor.

b. Payment for Resampled Lots. The redefined PWL for a resampled lot shall be used to calculate the payment for that lot in accordance with Table 6.

c. Outliers. Check for outliers in accordance with ASTM E 178, at a significance level of 5 percent.

[XXX-5.4 LEVELING COURSE. Any course used for truing and leveling shall meet the requirements of paragraph XXX-3.2, XXX-5.2b(1) for air voids and XXX-5.2b(2), but shall not be subject to the density requirements of paragraph XXX-5.2b(1) for mat density and XXX-5.2b(3). The leveling course shall be compacted with the same effort used to achieve density of the test section. The truing and leveling course shall not exceed a nominal thickness of 1-½ inches (37.5 mm). The leveling course is the first variable thickness lift of an overlay placed prior to subsequent courses.**]**

*** ***

Use this paragraph only when there is a need to restore proper cross-section prior to overlaying. Areas of the pavement requiring a leveling course shall be shown on the plans.

*** ***

CONTRACTOR QUALITY CONTROL

XXX-6.1 GENERAL. The Contractor shall develop a Quality Control Program in accordance with Section 100 of the General Provisions. The program shall address all elements that affect the quality of the pavement including, but not limited to:

a. Mix Design

b. Aggregate Grading
c. Quality of Materials
d. Stockpile Management
e. Proportioning
f. Mixing and Transportation
g. Placing and Finishing
h. Joints
i. Compaction
j. Surface Smoothness
k. Personnel
l. Laydown Plan

The Contractor shall perform quality control sampling, testing, and inspection during all phases of the work and shall perform them at a rate sufficient to ensure that the work conforms to the contract requirements, and at minimum test frequencies required by paragraph XXX-6.3 and Section 100 of the General Provisions. As a part of the process for approving the Contractor's plan, the Engineer may require the Contractor's technician to perform testing of samples to demonstrate an acceptable level of performance.

No partial payment will be made for materials that are subject to specific quality control requirements without an approved plan.

XXX-6.2 TESTING LABORATORY. The Contractor shall provide a fully equipped asphalt laboratory meeting the requirements of paragraph XXX-3.5 and XXX-4.2a(2) located at the plant or job site. The Contractor shall provide the Engineer with certification stating that all of the testing equipment to be used is properly calibrated and will meet the specifications applicable for the specified test procedures.

XXX-6.3 QUALITY CONTROL TESTING. The Contractor shall perform all quality control tests necessary to control the production and construction processes applicable to these specifications and as set forth in the approved Quality Control Program. The testing program shall include, but not necessarily be limited to, tests for the control of asphalt content, aggregate gradation, temperatures, aggregate moisture, field compaction, and surface smoothness. A Quality Control Testing Plan shall be developed as part of the Quality Control Program.

a. Asphalt Content. A minimum of two tests shall be performed per lot in accordance with ASTM D 6307 or ASTM D 2172 for determination of asphalt content. The weight of ash portion of the test, as described in ASTM D 2172, shall be determined as part of the first test performed at the beginning of plant production; and as part of every tenth test performed thereafter, for the duration of plan production. The last weight of ash value obtained shall be used in the calculation of the asphalt content for the mixture. The asphalt content for the lot will be determined by averaging the test results.

The use of the nuclear method for determining asphalt content in accordance with ASTM D 4125 is permitted, provided that it is calibrated for the specific mix being used.

b. Gradation. Aggregate gradations shall be determined a minimum of twice per lot from mechanical analysis of extracted aggregate in accordance with ASTM D 5444 and ASTM C 136 (Dry Sieve). When asphalt content is determined by the nuclear method, aggregate gradation shall be determined from hot bin samples on batch plants, or from the cold feed on drum mix or continuous mix plants, and tested in accordance with ASTM C 136 (dry sieve) using actual batch weights to determine the combined aggregate gradation of the mixture.

c. Moisture Content of Aggregate. The moisture content of aggregate used for production shall be determined a minimum of once per lot in accordance with ASTM C 566.

d. Moisture Content of Mixture. The moisture content of the mixture shall be determined once per lot in accordance with ASTM D 1461 **[**or AASHTO T110**]**.

* *

ASTM D 1461 may be replaced with AASHTO T110 moisture content testing procedure using a conventional oven or microwave.

* *

e. Temperatures. Temperatures shall be checked, at least four times per lot, at necessary locations to determine the temperatures of the dryer, the bitumen in the storage tank, the mixture at the plant, and the mixture at the job site.

f. In-Place Density Monitoring. The Contractor shall conduct any necessary testing to ensure that the specified density is being achieved. A nuclear gauge may be used to monitor the pavement density in accordance with ASTM D 2950.

g. Additional Testing. Any additional testing that the Contractor deems necessary to control the process may be performed at the Contractor's option.

h. Monitoring. The Engineer reserves the right to monitor any or all of the above testing.

XXX-6.4 SAMPLING. When directed by the Engineer, the Contractor shall sample and test any material that appears inconsistent with similar material being sampled, unless such material is voluntarily removed and replaced or deficiencies corrected by the Contractor. All sampling shall be in accordance with standard procedures specified.

XXX-6.5 CONTROL CHARTS. The Contractor shall maintain linear control charts both for individual measurements and range (i.e., difference between highest and lowest measurements) for aggregate gradation and asphalt content.

Control charts shall be posted in a location satisfactory to the Engineer and shall be kept current. As a minimum, the control charts shall identify the project number, the contract item number, the test number, each test parameter, the Action and Suspension Limits applicable to each test parameter, and the Contractor's test results. The Contractor shall use the control charts as part of a process control system for identifying potential problems and assignable causes before they occur. If the Contractor's projected data during production indicates a problem and the Contractor is not taking satisfactory corrective action, the Engineer may suspend production or acceptance of the material.

a. Individual Measurements. Control charts for individual measurements shall be established to maintain process control within tolerance for aggregate gradation and asphalt content. The control charts shall use the job mix formula target values as indicators of central tendency for the following test parameters with associated Action and Suspension Limits:

CONTROL CHART LIMITS FOR INDIVIDUAL MEASUREMENTS		
Sieve	Action Limit	Suspension Limit
¾ inch (19.0 mm)	0%	0%
½ inch (12.5 mm)	+/-6%	+/-9%
⅜ inch (9.5 mm)	+/-6%	+/-9%
No. 4 (4.75 mm)	+/-6%	+/-9%
No. 16 (1.18 mm)	+/-5%	+/-7.5%
No. 50 (0.30 mm)	+/-3%	+/-4.5%
No. 200 (0.075 mm)	+/-2%	+/-3%
Asphalt Content	+/-0.45%	+/-0.70%

b. Range. Control charts for range shall be established to control process variability for the test parameters and Suspension Limits listed below. The range shall be computed for each lot as the difference between the two test results for each control parameter. The Suspension Limits specified below are based on a sample size of n = 2. Should the Contractor elect to perform more than two tests per lot, the Suspension Limits shall be adjusted by multiplying the Suspension Limit by 1.18 for n = 3 and by 1.27 for n = 4.

CONTROL CHART LIMITS BASED ON RANGE (Based on n = 2)	
Sieve	Suspension Limit
½ inch (12.5 mm)	11 percent
⅜ inch (9.5 mm)	11 percent
No. 4 (4.75 mm)	11 percent
No. 16 (1.18 mm)	9 percent
No. 50 (0.30 mm)	6 percent
No. 200 (0.075 mm)	3.5 percent
Asphalt Content	0.8 percent

c. Corrective Action. The Contractor Quality Control Program shall indicate that appropriate action shall be taken when the process is believed to be out of tolerance. The Plan shall contain sets of rules to gauge when a process is out of control and detail what action will be taken to bring the process into control. As a minimum, a process shall be deemed out of control and production stopped and corrective action taken, if:

(1) One point falls outside the Suspension Limit line for individual measurements or range; or

(2) Two points in a row fall outside the Action Limit line for individual measurements.

* *

The aggregate control chart parameters and Suspension and Action Limits contained in the above paragraphs are based on ¾ inch (19.0 mm) maximum size aggregate gradation. When 1-inch (25.0 mm) or 1-½ inch (37.5 mm) maximum size aggregate is specified, the Individual Measurements Chart requirements should be amended as follows:

Sieve	**Action Limit**	**Suspension Limit**
1 inch or 1-½ inch	**0%**	**0%**
¾ inch	**6%**	**11%**

When ½-inch (12.5 mm) maximum size aggregate is specified, the ¾-inch (19.0 mm) and 1-inch (25.0 mm) sieves should be deleted from the Individual Measurements Chart and the ½-inch (12.5 mm) sieve Action and Suspension Limits should be changed to 0%. For the ½-inch (12.5 mm) gradation, the ½-inch sieve should be deleted from the Range Chart.

* *

XXX-6.6 QUALITY CONTROL REPORTS. The Contractor shall maintain records and shall submit reports of quality control activities daily, in accordance with the Contractor Quality Control Program described in General Provisions, Section 100.

METHOD OF MEASUREMENT

XXX-7.1 MEASUREMENT. Plant mix bituminous concrete pavement shall be measured by the number of tons (kg) of bituminous mixture used in the accepted work. Recorded batch weights or truck scale weights will be used to determine the basis for the tonnage.

Saw-cut grooving of bituminous pavement shall be measured by the number of square yards of saw-cut grooving as specified in-place, completed and accepted.

BASIS OF PAYMENT

XXX-8.1 PAYMENT. Payment for an accepted lot of bituminous concrete pavement shall be made at the contract unit price per ton (kg) for bituminous mixture adjusted according to paragraph XXX-8.1a, subject to the limitation that:

> The total project payment for plant mix bituminous concrete pavement shall not exceed **[ ]** percent of the product of the contract unit price and the total number of tons (kg) of bituminous mixture used in the accepted work (See Note 2 under Table 6).

Payment for accepted saw-cut grooving shall be made at the contract unit price per square yard.

The price shall be compensation for furnishing all materials, for all preparation, mixing, and placing of these materials, and for all labor, equipment, tools, and incidentals necessary to complete the item.

* *

> **The Engineer shall specify a value ranging from 100 to the maximum lot pay factor amount. (106 percent for single-sided density or 103 percent when double-sided density is specified.) When the total project payment for Item P-XXX pavement exceeds the contract unit price, any AIP or PFC funds used to pay the excess may require an amendment to the AIP grant or PFC application for the project.**

* *

a. Basis of Adjusted Payment. The pay factor for each individual lot shall be calculated in accordance with Table 6. A pay factor shall be calculated for both mat density and air voids. The lot pay factor shall be the higher of the two values when calculations for both mat density and air voids are 100 percent or higher. The lot pay factor shall be the product of the two values when only one of the calculations for either mat density or air voids is 100 percent or higher. The lot pay factor shall be the lower of the two values when calculations for both mat density and air voids are less than 100 percent.

TABLE 6. PRICE ADJUSTMENT SCHEDULE [1]

Percentage of Material Within Specification Limits (PWL)	Lot Pay Factor (Percent of Contract Unit Price)
96 – 100	106
90 – 95	PWL + 10
75 – 89	0.5 PWL + 55
55 – 74	1.4PWL – 12
Below 55	Reject [2]

[1] ALTHOUGH IT IS THEORETICALLY POSSIBLE TO ACHIEVE A PAY FACTOR OF 106 PERCENT FOR EACH LOT, ACTUAL PAYMENT ABOVE 100 PERCENT SHALL BE SUBJECT TO THE TOTAL PROJECT PAYMENT LIMITATION SPECIFIED IN PARAGRAPH XXX-8.1.

[2] The lot shall be removed and replaced. However, the Engineer may decide to allow the rejected lot to remain. In that case, if the Engineer and Contractor agree in writing that the lot shall not be removed, it shall be paid for at 50 percent of the contract unit price and the total project payment shall be reduced by the amount withheld for the rejected lot.

For each lot accepted, the adjusted contract unit price shall be the product of the lot pay factor for the lot and the contract unit price. Payment shall be subject to the total project payment limitation specified in paragraph XXX-8.1. Payment in excess of 100 percent for accepted lots of bituminous concrete pavement shall be used to offset payment for accepted lots of bituminous concrete pavement that achieve a lot pay factor less than 100 percent.

* *

NOTE TO ENGINEER. The Engineer may specify both upper and lower PWL acceptance criteria (two-sided) for density. Use the following pay adjustment schedule when two-sided acceptance criteria for density is specified edit Table 5 to include the Upper tolerance limits and edit paragraph XXX-8.1.

TABLE 6. PRICE ADJUSTMENT SCHEDULE [1]

Percentage of Material Within Specification Limits (PWL)	Lot Pay Factor (Percent of Contract Unit Price)
93 – 100	103
90 – 93	PWL + 10
70 – 89	0.125PWL + 88.75
40 – 69	0.75PWL +45
Below 40	Reject [2]

[1] ALTHOUGH IT IS THEORETICALLY POSSIBLE TO ACHIEVE A PAY FACTOR OF 103 PERCENT FOR EACH LOT, ACTUAL PAYMENT ABOVE 100 PERCENT SHALL BE SUBJECT TO THE TOTAL PROJECT PAYMENT LIMITATION SPECIFIED IN PARAGRAPH XXX-8.1.

[2] The lot shall be removed and replaced. However, the Engineer may decide to allow the rejected lot to remain. In that case, if the Engineer and Contractor agree in writing that the lot shall not be removed, it shall be paid for at 50 percent of the contract unit price AND THE TOTAL PROJECT PAYMENT LIMITATION SHALL BE REDUCED BY THE AMOUNT WITHHELD FOR THE REJECTED LOT.

* *

* *

If a profilograph is used, add the following paragraphs and change existing paragraph XXX-8.1b to XXX-8.1d (The pay adjustment in Table 7 is optional to the Owner and Engineer when using the profilograph):

b. Profilograph Smoothness. When the final average profile index (subsequent to any required corrective action) does not exceed 7 inches per mile, payment will be made for that section at the contract unit price for the completed pavement. If the final average profile index (subsequent to any required corrective action) exceeds 7 inches per mile, but does not exceed 15 inches per mile, the Contractor may elect to accept a contract unit price adjustment in lieu of reducing the profile index.

c. Basis of Adjusted Payment for Smoothness. Price adjustment for pavement smoothness will be made in accordance with Table 7. The adjustment will apply to the total tonnage of asphalt concrete within a lot of pavement and shall be applied with the following equation:

(Tons of asphalt concrete in lot) x (lot pay factor) x (unit price per ton) x (smoothness pay factor) = payment for lot

TABLE 7. AVERAGE PROFILE INDEX SMOOTHNESS PAY FACTOR

(Inches per mile per 1/10 mile)	Short Sections	Pay Factor
00.0 - 7	00.0 - 15.0	100%
7.1 - 9	15.1 - 16	98%
9.1 - 11	16.1 - 17	96%
11.1 - 13	17.1 - 18	94%
13.1 - 14	18.1 - 20	92%
14.1 - 15	20.1 - 22	90%
15.1 & up	22.1& up	corrective work required[1]

1The Contractor shall correct pavement areas not meeting these tolerances by removing and replacing the defective work. If the Contractor elects to construct an overlay to correct deficiencies, the minimum thickness of the overlay shall not be less than twice the size of the maximum size aggregate. The corrective overlay shall not violate grade criteria and butt joints shall be constructed by sawing and removing the original pavement in compliance with the thickness/maximum aggregate size ratio. Skin patching shall not be permitted.

Unit bid price adjustment will apply to total bituminous mixture and asphalt cement quantities within the 1/10 mile segment of pavement. Deductions will be applied to recorded project quantities. Any pavement section less than 1/10 mile will be accepted on a pro-rated basis.

Material used in building the pavement above the specified grade shall not be included in the quantities for payment.

****

b. Payment. Payment will be made under:

Item P-XXX-8.1a Bituminous **[Surface] [Base] [Binder] [Leveling]** Course—per ton (kg)

Item P-XXX-8.1b Saw-Cut Grooving—per square yard

TESTING REQUIREMENTS

ASTM C 29	Bulk Density (“Unit Weight”) and Voids in Aggregate
ASTM C 88	Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate
ASTM C 117	Materials Finer than 75µm (No.200) Sieve in Mineral Aggregates by Washing
ASTM C 127	Specific Gravity and Absorption of Coarse Aggregate
ASTM C 131	Resistance to Degradation of Small Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
ASTM C 136	Sieve Analysis of Fine and Coarse Aggregates

ASTM C 183	Sampling and the Amount of Testing of Hydraulic Cement
ASTM C 566	Total Evaporable Moisture Content of Aggregate by Drying
ASTM D 75	Sampling Aggregates
ASTM D 979	Sampling Bituminous Paving Mixtures
ASTM D 995	Mixing Plants for Hot-Mixed Hot-Laid Bituminous Paving Mixtures
ASTM D 1073	Fine Aggregate for Bituminous Paving Mixtures
ASTM D 1188	Bulk Specific Gravity and Density of Compacted Bituminous Mixtures Using Paraffin-Coated Specimens
ASTM D 1461	Moisture or Volatile Distillates in Bituminous Paving Mixtures
ASTM D 2041	Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures
ASTM D 2172	Quantitative Extraction of Bitumen from Bituminous Paving Mixtures
ASTM D 2419	Sand Equivalent Value of Soils and Fine Aggregate
ASTM D 2489	Estimating Degree of Particle Coating of Bituminous-Aggregate Mixtures
ASTM D 2726	Bulk Specific Gravity and Density of Non-Absorptive Compacted Bituminous Mixtures
ASTM D 2950	Density of Bituminous Concrete in Place by Nuclear Methods
ASTM D 3203	Percent Air Voids in Compacted Dense and Open Bituminous Paving Mixtures
ASTM D 3665	Random Sampling of Construction Materials
ASTM D 3666	Minimum Requirements for Agencies Testing and Inspecting Road and Paving Materials
ASTM D 4125	Asphalt Content of Bituminous Mixtures by the Nuclear Method
ASTM D 4318	Liquid Limit, Plastic Limit, and Plasticity Index of Soils
ASTM D 4791	Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate
ASTM D 4867	Effect of Moisture on Asphalt Concrete Paving Mixtures
ASTM D 5444	Mechanical Size Analysis of Extracted Aggregate
ASTM D 6926	Preparation of Bituminous Specimens Using MARSHALL Apparatus
ASTM D 6927	MARSHALL Stability and Flow of Bituminous Mixtures
ASTM E 11	Wire-Cloth Sieves for Testing Purposes

ASTM E 178	Dealing with Outlying Observations
ASTM E 1274	Measuring Pavement Roughness Using a Profilograph
AASHTO T 30	Mechanical Analysis of Extracted Aggregate
[AASHTO T 110	Moisture or Volatile Distillates in Bituminous Paving Mixtures**]**
The Asphalt Institute's Manual No. 2 (MS-2)	Mix Design Methods for Asphalt Concrete

MATERIAL REQUIREMENTS

ASTM D 242	Mineral Filler for Bituminous Paving Mixtures
ASTM D 946	Penetration Graded Asphalt Cement for Use in Pavement Construction
ASTM D 3381	Viscosity-Graded Asphalt Cement for Use in Pavement Construction
ASTM D 4552	Classifying Hot-Mix Recycling Agents
AASHTO M320	Performance Graded Asphalt Binder

END OF ITEM P-XXX