Evaluation of Using Different Stabilizers in the U.S. Route 15 (Maryland) Stone Matrix Asphalt

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Stone matrix asphalt (SMA) is a gap-graded hot mix that maximizes the binder and coarse-aggregate contents. A stabilizing additive, such as cellulose fiber, rock wool fiber, or polymer, is added to prevent the binder and aggregate dust from draining when the mixture is hot. Stabilizers should reduce the amount of draindown without decreasing the performance of the mixture. The effects of using different stabilizers in an SMA on draindown, rutting, low temperature cracking, and aging are evaluated. Stabilizers had no significant effect on rutting or low temperature properties. The two polymers were not as effective as the four fibers for preventing draindown, although they were better at reducing age hardening.

Stone matrix asphalt (SMA) is a gap-graded hot mix that maximizes the binder and coarse-aggregate contents. SMA technology was developed in Europe more than 20 years ago, although the characteristics of the mixture have been refined over the years. On the basis of European experience, SMAs perform better under heavy traffic loads and are more cost-effective than dense-graded mixtures. They are primarily used as surface-course mixtures in Europe (*1*).

The higher percentage of coarse aggregate in an SMA, as compared with that in a dense-graded mixture, the more contact points between coarse aggregate particles (*1*). Such contacts provide a high resistance to rutting and should reduce the influence of the type and amount of binder on rutting. An SMA has a high proportion of high quality, manufactured, coarse aggregate; a high proportion of binder and mineral filler; and a low proportion of middle-sized aggregate as compared with dense-graded mixtures.

More than 20 SMA pavements have been built in the United States, and additional SMA projects have been proposed. Construction aspects and the performances of these pavements are being evaluated. Laboratory studies also are needed to improve the definitions of the components of an SMA, and to develop tests for their design and analysis.

Gap gradation of an SMA may allow the binder and the aggregate dust to drain when the mixture is hot during storage, hauling, and placement. Hence, a stabilizing additive is used to prevent draindown. Cellulose and mineral fibers are used as stabilizers in Europe; polymers are used to a lesser degree (*1*). Fibers stiffen a binder through absorption and the resulting fiber network. Some suppliers claim there are other benefits, such as the mixture's improved resistance cracking.

Polymers stiffen a binder through various mechanisms that depend on the polymer and the method of addition. These mechanisms are not sufficiently documented in the literature. One of the more common mechanisms is to form a rubbery network, which may also reduce the temperature susceptibility of the binder and increase the elastic strain.

Stabilizers should reduce draindown without decreasing the performance of the mixture. Different stabilizers have provided different optimum binder contents in a given mixture at equal design air voids (*1*). This could indicate that stabilizers can affect how a mixture compacts. If a stabilizer excessively stiffens a binder to the point that the SMA is more difficult to compact, the mixture will require more asphalt to meet the target air-void level. This could decrease the degree of contact between aggregate particles.

OBJECTIVE

The objective of this study was to evaluate the effects of using different stabilizers in an SMA, in terms of draindown and the resistance of the SMA to rutting, low temperature cracking, aging, and moisture susceptibility.

MATERIALS

Six primary SMAs were tested in this study. These contained the same aggregates and gradation, but they used different stabilizers. The aggregate, asphalt, and three of the stabilizers were used in an SMA pavement built in 1992 as part of a resurfacing project on U.S. Route 15, south of Frederick, Maryland (*2*).

One additional SMA was also tested. The gradation used in the six SMAs was altered to reduce the gap. This gradation was added to the study when we learned that the polymer stabilizers did not prevent draindown. The objective was to determine the effects of the change in gradation on draindown and other mixture properties and thereby provide preliminary information for a planned FHWA study under which the effects of gradation on SMA properties would be examined.

Stabilizers

The stabilizers included two loose cellulose fibers named Custom Fiber CF 31500 and Interfibe 230; a pelletized cellulose fiber named Arbocel BG 50/50; a loose-rock wool fiber named Inorphil; and two polymers named Vestoplast-S and Styrelf 1-D. Arbocel, Vestoplast, and Styrelf were used in the Route 15 pavement (*2*). Custom Fiber and Interfibe were included in our study in order to investigate the

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use of domestic cellulose fibers. (The other two fibers, Arbocel and Inorphil, have European origins. Inorphil was included in the study because it has been used in other SMA pavements built in the United States.)

Stabilizers were added based on those quantities used in the pavement sections and according to the recommendations of the suppliers (*2*). Custom Fiber and Interfibe were added at 0.3 percent by mixture weight. The Arbocel pellets, which are 50-percent binder, were added at 0.6 percent by mixture weight; Inorphil was added at 0.5 percent by mixture weight; and the Vestoplast pellets were added at 7.0 percent by asphalt weight. Styrelf is an asphalt, modified with polymer by the supplier, and is received in bulk form. The amount of polymer in the binder is not reported by the supplier.

Asphalt

Properties of the AC-20 asphalt and the two modified binders are given in Table 1. The base asphalt for the Vestoplast binder was the AC-20 asphalt. A different AC-20 asphalt was used in the Styrelf binder. Both polymer-modified binders were significantly stiffer than the AC-20 asphalt, according to the capillary viscosities and penetrations. The data for the Vestoplast binder assume that the pellets and the asphalt homogeneously blend; that may not be true. When performing a mixture design, the pellets are mixed with the hot aggregate and melted before the asphalt is added.

Aggregates

The aggregates were a blend of No. 68 and No. 8 diabase from Leesburg, Virginia; a No. 10 limestone called "Bird Eye" from Frederick, Maryland; and a mineral filler called "Aglime" from Texas, Maryland. Properties of the aggregates, target gradation based on the average gradation of the pavement sections, and actual gradation of the blend used in this study are shown in Table 2.

The Los Angeles Abrasions of the coarse fraction of each aggregate were below the maximum allowable loss of 30 percent for aggregates used in SMAs (*3*).

The coarse fractions of the No. 68 diabase and Bird Eye aggregates passed the German test for flat and elongated particles, and the data for the No. 8 diabase aggregate was right at the specification. SMA specifications reject aggregates in which more than 20 percent of the particles by mass have a length-to-thickness ratio greater than 3 to 1 (*3*).

The altered gradation is also shown in Table 2. The 41-percent aggregate passing the 4.75-mm sieve is estimated to be the maximum allowable according to German gradation specifications (*1*). Styrelf was used with this gradation.

MIXTURE TESTING PROGRAM

The following mixture tests were performed:

TABLE 1 Physical Properties of the Binders

Physical Properties	Virgin Binder	Binder After Thin-Film Oven Test
AC-20		
Thin-Film Oven Test, percent loss		0.02
Penetration, 25 °C (100 g, 5 s), 0.1 mm	72	49
Absolute Viscosity, 60 °C, dPa-s	2 420	6 382
Kinematic Viscosity, 135 °C, mm²/s	478	746
Specific Gravity, 25/25 °C	1.032	
Solubility in Trichloroethylene, percent	100.00	
Inorganic Material or Ash, percent	0.00	
Flash Point, COC, °C	322	
AC-20 with 7-Percent Vestoplast by Weight		
Thin-Film Oven Test, percent loss		0.06
Penetration, 25 °C (100 g, 5 s), 0.1 mm	54	48
Absolute Viscosity, 60 °C, dPa-s	10 619	14 493
Kinematic Viscosity, 135 °C, mm²/s	1 179	1 687
Specific Gravity, 25/25 °C	1.020	
Solubility in Trichloroethylene, percent	99.96	
Inorganic Material or Ash, percent	0.02	
Flash Point, COC, °C	346	
Styrelf 1-D		
Thin-Film Oven Test, percent loss		0.26
Penetration, 25 °C (100 g, 5 s), 0.1 mm	59	43
Absolute Viscosity, 60 °C, dPa-s	33 122	90 995
Kinematic Viscosity, 135 °C, mm²/s	2 345	3 178
Specific Gravity, 25/25 °C	1.026	
Solubility in Trichloroethylene, percent	99.61	
Inorganic Material or Ash, percent	0.02	
Flash Point, COC, °C	316	

TABLE 2 Aggregate Properties

Aggregate Gradations, Percent Passing:

Sieve Size (mm)	Leesburg, Virginia Traprock No. 68	Leesburg, Virginia Traprock No. 8	Frederick, Maryland Bird Eye Limestone No. 10	Texas, Maryland Aglime Mineral Filler	40 % No. 68, 34 % No. 8, 12 % Bird Eye, 14 % Aglime: Target Blend	Actual Blend	Altered Blend
19.0	100.0				100.0	100.0	100.0
12.5	75.0	100.0		100.0	90.0	91.8	92.0
9.5	41.4	87.9	100.0	99.7	72.4	74.2	74.0
4.75	7.3	22.1	62.1	99.4	31.8	33.8	41.0
2.36	2.0	6.2	14.8	98.1	18.8	18.9	23.0
1.18	1.6	3.6	4.3	96.4	15.6	15.6	15.9
0.600	1.4	2.8	3.2	95.7	14.6	14.7	15.3
0.300	1.2	2.3	2.9	91.6	14.0	14.0	14.4
0.150	0.9	1.9	2.6	85.0	12.9	13.0	13.2
0.075	0.6	1.4	2.2	65.0	10.1	10.1	10.1

Specific Gravities and Percent Absorption:

	No. 68	No. 8	No. 10	Mineral Filler	Actual Blend	Altered Blend
Bulk Dry	2.946	2.964	2.692		2.899	2.895
Bulk SSD	2.964	2.980	2.707		2.914	2.912
Apparent	2.998	3.012	2.734	2.813	2.941	2.941
Abs (%)	0.59	0.54	0.28		0.55	0.56

Los Angeles Abrasion of the Particles in the Predominant Size Fractions above the 2.36-mm Sieve, Percent Loss by Mass:

No. 68	No. 8	No. 10
14.6	20.4	27.3

Flat and Elongated Particles in Size Fractions above the 4.75-mm Sieve, Percent by Mass:

No. 68	No. 8	No. 10
17.5	20.3	14.2

Bulk Dry = Bulk-Dry Specific Gravity.
Bulk SSD = Bulk-Saturated-Surface-Dry Specific Gravity.
Apparent = Apparent Specific Gravity.
Abs (%) = Percent Absorption.

- Marshall mixture design at 60°C, using 50 blows per side.
- Draindown:
  - –German,
  - –FHWA 2.36-mm sieve test, and
  - –Open-graded friction course (pie plate test).
- Resistance to rutting
  - –Georgia loaded-wheel tester (GLWT) at 40.6°C
  - –French pavement rutting tester at 60°C
  - –Gyratory testing machine (GTM) at 60°C
- Resistance to low temperature cracking
  - –Diametral modulus (M_d) at −32, −24, −16, −8, and 0°C
  - –Indirect tensile test at −16, −8, and 0°C
- Resistance to Aging (Strategic Highway Research Program Method M-007)
  - –Short-term Oven Aging with Diametral Modulus (M_d) at 25°C
  - –Long-term Oven Aging with M_d and Indirect Tensile Strength at 25°C
- Resistance to Moisture Damage
  - –Tensile Strength Ratio (TSR)
  - –Diametral Modulus Ratio (M_dR)
  - –Percent Visual Stripping

MIXTURE TEST DESCRIPTIONS

Mixture Design

Mixtures were designed by the 50-blow Marshall method, using binder contents of 5.5, 6.0, 6.5, and 7.0 percent by mixture weight. Optimum binder contents were based on a 3.5-percent air-void level. The target mixing and compaction temperatures were 154°C and 143°C, respectively.

Draindown

A German test, an FHWA test developed at the Turner-Fairbank Highway Research Center (TFHRC), and an FHWA test for open-graded friction courses (OGFC) were used to determine the efficiency of the stabilizers in preventing draindown in loose mixtures.

German Test (Schellenberg Institute, Germany)

Approximately 1 kg of mixture is placed into a dried, tared, 800-ml glass beaker immediately after mixing and weighed to the nearest

0.1 g (*1,3*). The beaker is then covered with aluminum foil and stored for 60 ± 1 min at the compaction temperature. (In Germany, the typical hot-mix plant discharge temperature of 170°C is used.) After storage, the mixture is placed into a tared bowl by quickly turning the beaker upside down without shaking. The final mass of the mixture is then recorded. The loss resulting from draindown is calculated as

$$\text{Loss, percent} = \frac{100(\text{original mass of mixture} - \text{final mass of mixture})}{\text{original mass of mixture}}$$

Losses up to 0.3 percent are allowable. Losses greater than 0.3 percent indicate that draindown may be a problem.

FHWA 2.36-mm Sieve Test

A dried circular bowl is weighed to the nearest 0.1 g. A dried 2.36-mm sieve of similar diameter is then placed on top of this bowl. The sieve and bowl are tared. Approximately 1 kg of asphalt mixture is placed on the sieve immediately after mixing and weighed to the nearest 0.1 g. The bowl, sieve, and mixture are covered with aluminum foil and stored for 60 ± 1 min at the compaction temperature. After storage, the sieve is removed and the final mass of the bowl is obtained, which includes the mass of the drained binder and fines. The percent loss resulting from draindown is calculated as

$$\text{Loss, percent} = \frac{100(\text{final bowl mass} - \text{initial bowl mass})}{\text{original mass of mixture}}$$

Because this test has not been correlated to problems in the field, there is no pass-fail criteria for it. It was developed as a potential replacement for the German test. Several suppliers of polymers have indicated that the German test is biased against polymers because the increase in tackiness provided by many polymers can cause more aggregate particles to stick to the bottom of the beaker.

FHWA Open-Graded Friction Course (OGFC)

The OGFC draindown test uses a 203.2- to 228.6-mm diameter clear pyrex pie plate to determine draindown (*4*). The quantity of mixture, storage time, and storage temperature used in the previous two draindown tests were again used. The loose mixture is spread across the pie plate and placed in the oven. After removal from the oven, the mixture is allowed to cool. The pie plate with mixture is then turned upside down. The degree of draindown is determined visually by comparing the amount of drained binder to five standard pictures of draindown, representing, on a scale of 1 to 5, no draindown to excessive draindown. For OGFC, draindown levels of 3 or 4 are desirable because the amount of drained binder that these levels provide is used to tack the mixture to the underlying layer. For SMAs, it was assumed that the degree of draindown should be less than or equal to a level of 3.

Resistance to Rutting

Georgia Loaded-Wheel Tester (GLWT)

The GLWT tests a beam for permanent deformation at 40.6°C. Each beam is 76.2 mm in width and thickness, and 381 mm in length. Beams are compacted by compression. The mixtures were tested at the 3.5-percent design air-void level.

The sides of a beam are confined by steel plates during testing except for the top 12.7 mm. A 690-kPa pressurized, stiff rubber hose is positioned across the top of the beam, and a loaded steel wheel runs back and forth on top of this hose for 8,000 cycles to create a rut. One cycle is defined as two passes of the wheel.

The load on the beam changes with the direction of travel. When the wheel is moving from right to left, the load is approximately 740 N at the center of the beam, while it is 630 N when moving left to right. Across the central region of the beam where the deformations are recorded, each of these loads has a variation of less then 5 percent.

Deformations are measured at the center of the beam, 51 mm left of center, and 51 mm right of center. If the average rut depth for three replicate beams exceeds 7.6 mm, the mixture is considered susceptible to rutting.

French Pavement Rutting Tester

The French (Laboratoires des Ponts et Chaussees) Pavement Rutting Tester tests a slab for permanent deformation at 60°C. Each slab is 500 mm in length, 180 mm in width, and 50 mm in thickness. The French Plate Compactor was used to fabricate the slabs. Mixtures are compacted in a steel mold using a smooth, reciprocating, pneumatic rubber tire having a diameter of 415 mm and a width of 109 mm.

Various sequences of different compaction efforts, tire pressures, and positions of the tire relative to the width of a slab are used to compact a slab. These parameters depend on the thickness of the slab and the required air-void level. The manufacturer verbally stated that the sequence used in this study should provide an air-void level close to the low end of the air-void range that can be obtained in the field after compaction by the rollers. This range varies with the type of mixture. Air voids in this study were approximately 3.5 percent, except for the Styrelf specimens that were at a 6-percent air-void level.

The French Pavement Rutting Tester tests two slabs at a time using two reciprocating tires. The tires are always at a fixed elevation. Hydraulic jacks underneath the slabs push them upward to create the load, normally 5000 ± 50 N. Each tire is inflated to 600 ± 30 kPa. The same type of tire used by the Plate Compactor is also used by this tester. Approximately 67 cycles are applied per minute. One cycle is defined as two passes of the tire. Slabs are tested in their molds.

Initially, 1,000 cycles are applied at 25°C to densify the mixture and to provide a smoother surface. The height of each slab is then calculated by averaging measurements taken at 15 positions using a depth gauge with a resolution of 0.1 mm. This average height is considered the initial height, or point-of-zero rut depth.

The slabs are then heated to 60°C for 6 hr, and the average rut depths are measured at 30, 100, 300, 1,000, and 3,000 cycles. A mixture is acceptable if the average rut depths at 1,000 and 3,000 cycles are less than or equal to 10 and 20 percent of the thickness of the slab, respectively. Slopes for different mixtures taken from log rut depth versus log cycles plots can also be compared. Rut-susceptible mixtures generally have higher slopes.

Gyratory Testing Machine (GTM)

Shear susceptibility was measured using the static shear strength (Sg), gyratory stability index (GSI), and gyratory elasto-plastic index (GEPI) provided by the U.S. Corps of Engineers GTM, Model 8A6B4C. The GTM is a combination compaction and plane strain-shear testing machine that applies stresses simulating pavement conditions. The FHWA GTM is completely automated; it monitors and calculates all parameters.

The GTM was operated in accordance with the 1993 proposed version of American Society for Testing and Materials (ASTM) D 3387, "Compaction and Shear Properties of Bituminous Mixtures by Means of the U.S. Corps of Engineers Gyratory Testing Machine (GTM)." A vertical pressure of 0.827 MPa and a 0.014-radian gyratory angle were used. Specimens were compacted to their refusal density, which is defined as the point where the change in density becomes less than 16 kg/m^3 per 100 revolutions. It required 260 to 300 revolutions to reach this density.

Mixtures were tested at binder contents of 5.6, 6.2, and 6.8 percent, which bracketed the Marshall optimum-binder contents. By doing this, the GTM data at the optimum binder contents could be evaluated and whether the GTM provided different optimum binder contents determined. Compacted specimens had a diameter of 101.6 mm and a height of 63.5 mm.

The 0.014-radian angle of compaction is set by the operator using two rollers that circle around a flange that is part of the mold chuck. The two rollers, which are 3.14159-radians apart, are offset in vertical elevation, causing the mold chuck and specimen mold to tilt. A kneading action is applied to the mixture as the rollers circle around the mold chuck. One roller compresses an oil-filled pressure gauge that is used to calculate the Sg. The GTM continuously monitors the Sg of the mixture with revolutions, but the current procedure only evaluates the Sg at the refusal density.

A mixture can shear in the mold, causing the mold and mold chuck to tilt sideways to an angle that is larger than the angle on the axis of the rollers. This angle is a measure of the shear strain in the mixture. The GTM continuously monitors this angle to the nearest 0.0017 radian. The GSI is the ratio of the maximum angle that occurs at the end of the test to the minimum intermediate angle. It is a measure of shear susceptibility at the refusal density. The minimum intermediate angle is the smallest angle that occurs after the compaction process has started. It can be greater than the angle set by the operator. The GSI at 300 revolutions is close to 1.0 for a stable mixture and is significantly above 1.1 for an unstable mixture (*5*). (A more definitive criteria for the GSI has not been established.) When designing a mixture, the manufacturer states that the optimum binder content should be less than the binder content when the GSI begins to exceed 1.0.

The GEPI is the ratio of the minimum intermediate angle to the initial angle. A GEPI of 1.0 indicates high internal friction. A GEPI significantly above 1.0 indicates lower internal friction, from the use of rounded aggregates or moisture damage. A GEPI below 1.0 indicates that the aggregate is deteriorating. (More definitive criteria for the GEPI have not been established. The manufacturer suggests using an acceptable range of 1.0 to 1.5.)

Resistance to Low-Temperature Cracking

The majority of low-temperature cracking studies use stiffness to compare the performances of binders or mixtures. High stiffnesses at cold temperatures are equated to low flexibility and to increased susceptibility to cracking. In this study, the diametral modulus test and the indirect tensile strength test were used to evaluate low-temperature cracking. Specimens were tested at the design air-void level of 3.5 percent.

Diametral Modulus, M_d

The M_d's were measured at −32, −24, −16, −8, and 0 °C using an apparatus manufactured by the Retsina Company of Oakland, California. The apparatus provides a total diametral modulus at a loading time of 0.1 s by applying a load on the vertical diameter of a specimen and measuring the total, horizontal, tensile deformation. It is marketed for measuring the resilient (mainly elastic) modulus of a specimen, but it actually measures a total modulus that consists of elastic, viscoelastic, and permanent deformations. The equation used by ASTM D 4123 to calculate M_d is as follows:

$$M_d = \frac{P\,(u + 0.2734)}{t\,H_t}$$

where

M_d = diametral modulus (MPa),
P = load (N),
u = Poisson's ratio (assumed as 0.35),
t = specimen thickness (mm), and
H_t = total horizontal deformation (mm).

The deformations were maintained within a range of 76 to 200 E-05 mm by varying the load. The test is virtually nondestructive in this range, and the same specimens can be tested at all temperatures. Specimens were tested at the lowest temperature first, and specimens were maintained at each temperature for 24 hr. A lower M_d generally means better resistance to low-temperature cracking, although this generalization may not be true for all modified binders.

Indirect Tensile Test

Indirect tensile tests were performed at −16, −8, and 0°C with a loading rate of 1.27 mm/min. Indirect tensile strength, tensile strain at failure (horizontal strain), and the amount of work needed to cause tensile failure were evaluated. The work is the area under the stress-strain curve from the beginning of the test until failure. Higher strains at failure and higher amounts of work are associated with increased resistance to low-temperature cracking. These increases are usually accompanied by lower tensile strengths, although this may not be true for all modified binders.

The equation used by ASTM D 4123 to compute the indirect tensile strength of a 101.6-mm diameter specimen is as follows:

$$S_t = \frac{6.27\,P}{t}$$

where

S_t = indirect tensile strength (kPa),
P = load (N), and
t = thickness (mm).

The equation used to compute the strain at failure, assuming a Poisson's ratio of 0.35, is

$$e_t = 0.0205\,H_t$$

where e_t is the indirect tensile strain at failure and H_t is the total horizontal deformation (mm).

Resistance to Aging

The mixtures were oven-aged according to Strategic Highway Research Program (SHRP) Method M-007 (*6*). The method stated that short-term aging produces the average amount of aging that occurs during production and placement of the paving mixture. Long-term aging produces the average amount of aging that occurs between placement and approximately 10 years of service life.

Standard practice for preparing specimens at TFHRC did not include ovenaging when this study was performed. Thus, all mixtures tested under the other parts of this study were not subjected to the SHRP aging methods. The properties of mixtures not subjected to these methods are termed "unaged" properties in this section of the study, although some aging does occur during mixing and compaction.

To simulate short-term aging, the loose mixture is placed in a forced draft oven for 4 h at 135°C. The mixture is then compacted to an air-void level typically obtained after construction. A 5- to 6-percent level is typical for SMA, and 5.5 percent was used in this study. To simulate long-term aging, the compacted specimens are placed in a forced draft oven for 5 days at 85°C.

M_d and indirect tensile strengths were measured at 25°C on the unaged and long-term aged specimens. The specimens subjected to short-term aging were only tested for M_d so that they could also be used for long-term aging. Surface-course mixtures with lower M_d and tensile strengths are more flexible and therefore more resistant to fatigue cracking, although these generalizations may not be true for all modified binders.

Resistance to Moisture Damage

Both the AC-20 asphalt and the Styrelf binder had been treated with liquid antistripping additives before being used in the pavement. Therefore, it was decided to determine the moisture susceptibilities of the Arbocel and Styrelf mixtures using ASTM D 4867 "Effect of Moisture on Asphalt Concrete Paving Mixtures," and to test the other mixtures later if appropriate. Specimens were compacted to a 5- to 6-percent air-void level. Both the diametral modulus-retained ratio (M_dR) and TSR ratio were computed in terms of percentages.

RESULTS AND DISCUSSION

An analysis of variance and Fisher's Least Significant Difference statistical procedures at a 95-percent confidence level were used to analyze the test results. Fisher's analysis places sample averages into groups by determining which averages are statistically equal. Groups are designated by letters starting with A. Averages in group A are statistically equal and higher than those in group B. Averages in group B are statistically equal and higher than those in group C, and so forth. Averages can fall into more than one group. For example, a designation of AB shows that the average can be put into group A or B.

Mixture Design

Properties at optimum binder contents are given in Table 3. The optimum binder contents were 6.5 percent for the mixtures with Interfibe, Vestoplast, and Styrelf; 6.4 percent for Arbocel; 6.2 percent for Inorphil; 5.9 percent for Custom Fiber; and 5.8 percent for the Styrelf mixture with altered gradation. The average binder contents used in the pavement sections based on quality-control testing were 6.5 ± 0.2 percent for the Styrelf and Arbocel sections, and 6.2 ± 0.3 percent for the Vestoplast sections.

It was expected that the mixture with Custom Fiber would have an optimum binder content closer to those for the other cellulose fibers. Also, a representative for Vestoplast stated that the optimum binder content for the Vestoplast mixture should have been lower than those for fibers. These mixtures were retested, but the same data were obtained.

Table 3 shows that the Styrelf mixtures had the highest Marshall stabilities and flows. The other mixtures had statistically equal stabilities and flows; one reason for this equality was that the data were highly variable. Altering the gradation lowered the void in the mineral aggregate (VMA) by approximately 1.6 percent.

The VMA versus binder-content relationships (not given in this paper) showed that the optimum binder contents for the Custom Fiber mixture and Styrelf mixture with altered gradation were at the point of minimum VMA. The VMAs were starting to increase at the optimum binder contents for the other stabilizers, indicating they

TABLE 3 Marshall Mixture Design Properties

Type of Stabilizer	Optimum Binder Content (%)	MSG	Density (kg/m^3)	Stability (N)	Flow (0.25-mm)	Air Voids (%)	VMA (%)	VFA (%)
Custom Fibers	5.9	2.623	2527	6 623	14	3.5	18.2	87.3
Inorphil	6.2	2.628	2531	6 303	12	3.5	18.2	79.9
Arbocel	6.4	2.615	2519	7 422	13	3.5	18.6	84.0
Interfibe	6.5	2.600	2500	7 406	10	3.5	19.3	80.5
Vestoplast	6.5	2.606	2520	7 504	16	3.5	18.7	82.8
Styrelf	6.5	2.606	2514	9 933	27	3.5	18.9	81.7
Styrelf with Altered Gradation	5.8	2.638	2545	10 721	30	3.5	17.3	79.2

Marshall Design Blows = 50
Mixing Temperature = 154 °C
Compaction Temperature = 143 °C

MSG = Maximum Specific Gravity of the Mixture.
VMA = Voids in the Mineral Aggregate.
VFA = Voids Filled with Asphalt.

may have been slightly high for obtaining maximum resistance to rutting.

Draindown

Table 4 shows that the mixtures with Vestoplast and Styrelf and an AC-20 control mixture had the highest amounts of draindown for all three methods, and all failed the German and OGFC tests. The four mixtures with fibers had low amounts of draindown and passed these tests. (An AC-20 control mixture with no stabilizer at a binder content of 6.5 percent was included for comparative purposes. This mixture was not included in the other parts of this study because the asphalt drained during compaction. This did not occur when using Vestoplast or Styrelf.)

Altering the gradation significantly reduced the amount of draindown. The mixture passed the OGFC test; the loss of 0.05 percent in the 2.36-mm sieve method is very low compared to the binder content of 5.8 percent. (This is a low amount of draindown even if it is assumed that all of the drained material was binder.) The mixture did fail the German test.

The German test provided one discrepancy. The loss of 3.86 percent for the Styrelf mixture was greater than the loss of 2.88 percent for the AC-20 control mixture. Middle-sized and coarse aggregate particles had stuck to the bottom of the beakers when testing these two mixtures, as they did when testing the mixture with Vestoplast. Therefore, the discrepancy may have been related to differing amounts of aggregate stuck to the beaker. When aggregates other than dust are left in the beaker, the test does not measure draindown accurately.

The 2.36-mm sieve method provided a similar discrepancy. The losses of 0.89 and 0.84 percent for the Styrelf and Vestoplast mixtures, respectively, were greater than the loss of 0.19 percent for the AC-20 control mixture. A reason for this discrepancy was not apparent. It was hypothesized that the control mixture had a higher loose density, which provided fewer channels for draindown.

Because the Marshall design and draindown results were similar for the mixtures with fibers, it was decided not to test the Interfibe and Inorphil fibers any further unless the data for the other stabilizers provided a reason for testing them. For example, if the mixtures with the other four stabilizers showed significant differences in rutting potential, then the mixtures with the Interfibe and Inorphil fibers would be tested for rutting.

Resistance to Rutting

GLWT

The GLWT results are given in Table 5. All rut depths were statistically equal and less than the specification level of 7.6 mm.

French Pavement Rutting Tester

The French Pavement Rutting Tester results are presented in Table 5. All rut depths and slopes were statistically equal, and none of the rut depths exceeded the maximum allowable levels.

GTM

The GTM data at the optimum binder contents provided by the Marshall designs are indicated in Table 5. The static shear strengths, GSIs, GEPIs, and refusal air-void levels were statistically equal from mixture to mixture; all were in the acceptable range. (This testing was added at the end of the study. Only three stabilizers were tested because of insufficient aggregate.)

The static shear strengths, GSIs, and GEPIs at all three binder contents tested, namely, 5.6, 6.2, and 6.8 percent, were equal. These data indicate that binder contents higher than the Marshall optimums could be used without detrimentally affecting rutting resistance.

Extractions were performed to determine whether the GTM breaks coarse aggregate. Table 6 shows that aggregate did break even though the Los Angeles Abrasions were within the specification. The cumulative percent passing the 4.75-mm sieve increased more than 5 percent. Extractions were also performed on Custom Fiber specimens compacted with the Marshall hammer. The two compaction methods broke an equal amount of aggregate. The Mar-

TABLE 4 Draindown Test Results

Type of Stabilizer	Binder Content, Percent by Mix Weight	German Method, Percent Weight Loss	2.36-mm Sieve Method, Percent Weight Loss	Open-Graded Friction Course Drainage Test
Custom Fibers	5.9	0.07	0.00	2.5
Inorphil	6.2	0.11	0.00	2.5
Arbocel	6.4	0.27	0.00	2.5
Interfibe	6.5	0.25	0.00	2.5
Vestoplast	6.5	1.58	0.84	5
Styrelf	6.5	3.86	0.89	5
AC-20 Control	6.5	2.88	0.19	5
Styrelf with Altered Gradation	5.8	0.40	0.05	2.5
Specification, maximum		0.30	Not Established	3

TABLE 5 GLWT, French Pavement Rutting Tester (PRT), and GTM Results

Type of Stabilizer	GWLT Rut Depth (mm)	French PRT Percent Rut Depth at 3000 Cycles	French PRT Slope	Gyratory Testing Machine Static Shear Strength, Sg (kPa)	GSI	GEPI	Refusal Air Voids (%)
Custom Fibers	5.8	4.5	0.12	430	0.89	1.24	2.7
Arbocel	6.5	6.5	0.11	430	0.93	1.21	2.4
Vestoplast	4.0	6.1	0.17	NT	NT	NT	NT
Styrelf	3.2	7.7	0.14	480	0.92	1.23	2.4
Styrelf with Altered Gradation	4.3	4.3	0.16	NT	NT	NT	NT

NT = Not Tested.

TABLE 6 Aggregate Gradations (Percent Passing) Before and After Compaction

Sieve Size (mm)	Before Compaction	After Gyratory Compaction: Custom Fiber	Arbocel	Styrelf	After Marshall Hammer Compaction: Custom Fibers
19.0	100.0	100.0	100.0	100.0	100.0
12.5	91.8	91.2	91.3	91.9	90.6
9.5	74.2	74.3	73.4	73.6	74.0
4.75	33.8	39.2	39.8	39.6	39.6
2.36	18.9	23.8	23.6	24.0	23.2
1.18	15.6	19.2	19.1	19.2	18.2
0.600	14.7	17.7	17.3	17.5	16.5
0.300	14.0	16.4	16.1	16.3	15.2
0.150	13.0	14.9	14.7	14.9	13.7
0.075	10.1	11.7	11.7	11.7	10.5

shall hammer produced less material passing the 0.075-mm sieve, but in other concurrent FHWA studies, the two compaction methods have provided equal amounts of broken aggregate and equal increases in dust. Additional extractions showed that the aggregate broke during compaction and not during mixing.

Resistance to Low-Temperature Cracking

Diametral Modulus, M_d

The M_d in Table 7 do not show conclusively that any SMA was better than another. The data were close at a given temperature. Mixtures with better low-temperature properties have a designation of *B*.

Indirect Tensile Test

The tensile test data in Table 7 also, do not indicate that any SMA was better than another. Mixtures with better low-temperature properties have a designation of A for strain at failure and for work. All mixtures had statistically equal strains at the two lowest temperatures. One reason for this was that the data were highly variable. This also caused the works to be highly variable, especially at the lowest temperature. The two mixtures with Styrelf had higher strains and works at 0°C, possibly indicating improved resistance to fatigue cracking at higher temperatures. Mixtures with the highest strains or works did not necessarily have the lowest tensile strengths, as it is often assumed.

Resistance to Aging

Table 8 shows that how the mixtures grouped together varied with the test and the degree of aging. The Styrelf mixture had the best (lowest) properties after long-term aging. This binder did contain a different base asphalt.

The unknown consequence of having different rankings for unaged properties complicated the analyses; thus, the resistance to aging was related to changes in the moduli and strengths. These changes, also shown in Table 8, were computed from the averages for each mixture. Therefore, replicate measurements needed for statistical analyses were not available. (Averages had to be used because different specimens had to be tested to obtain the unaged and aged properties.) The two polymer-modified mixtures had the lowest changes; therefore, these mixtures aged less. Altering the gradation had a negative effect on aging.

Resistance to Moisture Damage

All retained ratios were above 90 percent, and visual stripping was less than 10 percent. These data indicated little susceptibility to

TABLE 7 Low Temperature Test Results

Diametral Modulus, MPa

Type of Stabilizer	Temperature (°C) 0	-8	-16	-24	-32
Custom Fiber	22 505 (B)	34 475 (A)	43 273 (A)	46 376 (A)	57 670 (A)
Arbocel	26 580 (A)	34 382 (A)	42 749 (A)	48 272 (A)	53 202 (AB)
Vestoplast	20 092 (B)	32 062 (A)	41 860 (A)	44 500 (A)	51 540 (B)
Styrelf	21 340 (B)	32 710 (A)	37 474 (B)	48 555 (A)	53 154 (AB)
Styrelf with Altered Gradation	26 049 (A)	33 937 (A)	41 949 (A)	50 389 (A)	52 754 (AB)

Indirect Tensile Test

Type of Stabilizer	Tensile Strength (kPa)			Failure Strain ($x10^{-6}$ mm/mm)			Work for Failure, (Pa or J/m^3)		
Temperature (°C)	0	-8	-16	0	-8	-16	0	-8	-16
Custom Fiber	1 769 (B)	2 310 (B)	3 314 (AB)	389 (B)	328 (A)	196 (A)	4 777 (B)	5 659 (AB)	4 882 (A)
Arbocel	2 054 (AB)	2 353 (B)	3 294 (AB)	363 (B)	273 (A)	284 (A)	5 610 (B)	4 767 (B)	4 925 (A)
Vestoplast	2 186 (A)	2 722 (AB)	3 070 (B)	393 (B)	268 (A)	240 (A)	6 147 (B)	4 668 (B)	5 104 (A)
Styrelf	2 297 (A)	2 648 (AB)	3 594 (AB)	715 (A)	425 (A)	238 (A)	12 597 (A)	7 742 (A)	5 383 (A)
Styrelf with Altered Gradation	2 282 (A)	2 936 (A)	3 779 (A)	793 (A)	303 (A)	304 (A)	14 423 (A)	6 350 (A)	6 297 (A)

moisture damage. The effects of the stabilizers on moisture damage, if any, could not be determined because antistripping additives had been used.

CONCLUSIONS

- The stabilizers had no significant effect on rutting susceptibility or low-temperature properties, even though the optimum binder contents varied from 5.9 to 6.5 percent. Therefore, small changes in the composition of the mastic also had no effect. All mixtures had a low susceptibility to rutting.
- The two mixtures containing the polymer stabilizers were better at controlling age hardening than the mixtures with fibers. This means they should provide more resistance to cracking after a pavement ages, including resistance to low-temperature cracking.
- The two polymers did not control draindown in the 1-hr draindown tests; whereas, the four fibers reduced the amount of drained material to acceptable levels. Fibers retain their structure with changes in temperature. The two polymers are thermoplastic.
- Reducing the optimum binder content by altering the gradation decreased the amount of draindown, but it also increased the susceptibility to age hardening. It had no effect on rutting susceptibility and low-temperature properties.
- The Marshall hammer and the GTM broke equal amounts of coarse aggregate.

RECOMMENDATIONS

- Further studies of draindown tests are needed. The current tests produce discrepancies that are difficult to explain. Draindown tests need to be performed when pavements are constructed to evaluate the tests' usefulness. No draindown problems were reported during construction on U.S. Route 15, although the tests only provide a propensity for draindown.

TABLE 8 Aging Test Results

Type of Stabilizer	Dynamic Modulus, M_d at 25 °C (MPa)			Change in Dynamic Modulus[a]		
	Unaged	Short-Term Aging	Long-Term Aging	Short-Term to Unaged	Long-Term to Short-Term	Total Aging
Custom Fiber	1518 (A)	3410 (A)	5157 (AB)	1892	1747	3639
Arbocel	1095 (B)	3814 (A)	5701 (A)	2719	1887	4607
Vestoplast	1545 (A)	3644 (A)	4405 (B)	2099	761	2860
Styrelf	814 (C)	1975 (B)	2435 (C)	1161	460	1621
Styrelf with Altered Gradation	1170 (B)	3778 (A)	4979 (AB)	2608	1201	3809

Type of Stabilizer	Tensile Strength at 25 °C (kPa)		Change in Tensile Strength[a]
	Unaged	Long-Term Aging	Total Aging
Custom Fiber	478 (C)	982 (AB)	504
Arbocel	519 (B)	1001 (A)	482
Vestoplast	594 (AB)	905 (AB)	311
Styrelf	582 (AB)	835 (B)	253
Styrelf with Altered Gradation	661 (A)	1022 (A)	361

[a]Obtained by subtraction.

- Additional mixtures need to be tested to verify the conclusions of this study.
- The consequences of aggregate being broken during mixture design need to be determined.
- The results from this study need to be compared to field performance. After 1.5 years, all three SMA pavement sections have no distress.

REFERENCES

1. Stuart, K. D. *Stone Mastic Asphalt (SMA) Mixture Design,* FHWA-RD-92-006. FHWA, U.S. Department of Transportation, March 1992.
2. Harmon, T. P., C. A. Gordon, K. M. Headrick, C. Harris Jr., J. R. Bukowski, and J. A. D'Angelo. *Mix Design Support & Field Production Testing.* Demonstration Project #90, Project #9202..FHWA, U.S. Department of Transportation, June 1992.
3. SMA Technical Working Group. *SMA Material and Construction Model Specification,* FHWA, U.S. Department of Transportation, Oct. 9, 1992.
4. Smith, R. W., J. M. Rice, and S. R. Spelman. *Design of Open-Graded Asphalt Friction Courses.* FHWA-RD-74-2. FHWA, U.S. Department of Transportation, January 1974 and July 1975 (supplement).
5. Roberts, F. L., P. S. Kandhal, E. R. Brown, D. Y. Lee, and T. W. Kennedy. *Hot Mix Asphalt Materials, Mixture Design, and Construction.* National Asphalt Pavement Association (NAPA) Education Foundation, Lanham, Md., 1991.
6. *Short and Long-Term Aging,* Method M-007. Strategic Highway Research Program, National Research Council, Washington, D.C., Sept. 25, 1992.

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