Sulphur Extended Asphalt Field Trials

MH 153, Brazos County, Texas

Final Report

(Report No. 536-13F)

TTI Project 2536

FCIP Study No. 1-10-78-536

by

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and

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I. HISTORICAL REVIEW OF SEA AND SULPHUR PAVING

The cementing properties of sulphur have been known for many years. Recorded experiments in which sulphur has been added to asphalt cement date to as early as the late 1800's, with more investigations of this type having occurred in the 1930's (1).

In the United States, Canada and several other industrialized countries, two significant situations developed in the early 1970's which created and accelerated the trend to find more uses for sulphur and sulphur blended with asphalt, particularly in highway paving. The first situation was an increased environment emphasis which called for the elimination of sulphur oxide emissions from the burning of fossil fuels such as coal, fuel oil and natural gas which contained sulphur, thus greatly increasing the availability of involuntary sulphur. The second event was the oil embargo of 1973 which created an incentive for highway planners to look for alternative binder materials in the paving layers to replace part or all of the asphalt cement.

Therefore, at that time, several systems for using sulphur pavements to extend or replace asphalt cement appeared in research and development activities of several agencies in the highway industry. One of these systems was SEA or sulphur extended asphalt in which elemental sulphur is used to replace an equal volume of asphalt cement, with the sulphur usually comprising from 30 to 40 percent of the total binder weight.

The first SEA project in the United States was constructed in September, 1975 on U.S. Highway 69 near Lufkin, Texas for the State Department of Highways and Public Transportation (1). Other United States SEA projects, numbering about 86 as of 1984, have since been constructed in 32 different states in the United States.

Additionally, about 125 Canadian SEA projects have been constructed in Alberta, Ontario and Saskatchewan; and other SEA projects have been completed in Mexico and several European and Arabic countries through 1984.

II. OBJECTIVES AND OUTLINE OF FCIP STUDY NO. 1-10-78-536

There were three main objectives of this study of the sulphur extended asphalt field trials on MH 153 in Brazos County, Texas. These objectives are stated as follows: "(1) to compare the benefits of sulphur-asphalt binder prepared in a mill as an emulsion with sulphur-asphalt binder prepared by by-passing the mill and comingling the molten sulphur and hot asphalt in the pipeline leading directly to the pugmill, (2) to beneficiate local marginal aggregates, (3) to compare SEA binders with different sulphur/asphalt ratios and to present laboratory mixtures and pavement sections utilizing the SEA binders and the control asphalt cement binder through the planning, design, construction and post-construction evaluation phases of the MH 153 study".

Study Outline

The MH 153 SEA field trial study has been conducted according to the following phases:

1. Preconstruction project planning,
2. Preconstruction laboratory design and testing,
3. Construction of MH 153 trial sections and
4. Post construction monitoring and evaluation.

Preconstruction Project Planning

During this phase of study, the site of the Texas State Department of Highways and Public Transportation (SDHPT) highway construction project was selected for constructing the SEA field trials. Also, the exact locations and lengths of the different SEA trial sections were decided upon for the MH 153 project. Finally, the several different aggregate gradations and ranges of SEA binders were tentatively selected for laboratory design and evaluation prior to construction.

Preconstruction Laboratory Design and Testing

During this part of the study, laboratory mix designs were evaluated using the different SEA binder compositions and pure asphalt cement for the control, together with three different aggregate gradations using aggregate materials from four different sources of siliceous gravel and sand. Optimum binder contents were developed using the Marshall method of mixture design. The SEA mixture designs used in the construction of MH 153 were taken from these preliminary designs (4).

Construction of MH 153 Trial Sections

The SEA field trial sections were constructed in June of 1978. The trial consisted of 2700 feet (823 m) of roadway 26 feet (8 m) wide. Six trial sections, 2 through 7, each 450 feet (137 m) in length, were constructed using two different SEA binder blends and three different aggregate gradations blends. The control sections, 1 and 8, are on each end of the field trial sections and consist of the project job mix formula aggregate gradations using 100 percent conventional asphalt cement. Details of the construction operations are contained in the Report FHWA-TS-80-214 entitled "Sulphur-Extended-Asphalt Field Trials MH 153 Brazos County Texas - A Detailed Construction Report" (4).

Post Construction Monitoring and Evaluation

The performance of each of the MH 153 SEA binder field trial sections has been monitored and evaluated periodically since June of 1978. The results of these observations are contained in twelve progress reports that have been prepared and published. The last of these reports was Progress Report No. 12 dated April, 1985 (5). These reports have provided for the timely up-dating of the status of the condition of both the surface and paving layer materials, as determined from tests on cores taken from the passing or inside lane of these sections.

III. MH 153 DESIGN AND CONSTRUCTION PROCEDURES

A full discussion of the laboratory mix design and construction procedures for the MH 153 field trials may be found in Report FHWA-TS-80-214 "Sulphur-Extended Asphalt Field Trials MH 153 Brazos County, Texas, A Detailed Construction Report" (4). Highlights of the design and construction procedures used on MH 153 are given below.

Pavement Design Consideration

The SEA binder hot-mixed asphaltic materials used on the MH 153 field trials consists of a six-inch (152 mm) base course layer of conventional asphalt stabilized base governed by the Texas SDHPT Specification Item 292, "Asphalt Stabilized Base (Plant Mix)". The MH 153 project plan specification notes called for this plant produced asphaltic mixture to meet a minimum Hveem (Texas SDHPT method) laboratory stability of 25 percent. Prior to construction traffic loadings anticipated on four-lane undivided MH 153 facility were 8100 vehicles per day with 6.1 percent trucks. The pavement was subjected to an estimated 50,000 18-kip (8172 kg) equivalent single axle loads. It should be noted that both the estimated ADT and percent trucks were much too low.

An estimate has been made of the actual accumulated traffic loading on MH 153 as of January 1, 1985 which shows that this roadway has an ADT of about 12,000 and has been subjected to approximately 15,000 18-kip (8172 kg) equivalent single axle loads instead of the approximately 9,800 loads originally forecasted. This is a 53 percent increase in traffic loading. The reasons for this increase are threefold: (1) overall traffic has actually been increasing 7.6 percent instead of 5 percent; (2) trucks as an average percentage of total traffic have decreased somewhat; and (3) the average of the ten heaviest wheel loads ATHWLD (5) increased from 10,800 pounds (4903 kg) to 12,600 pounds (5720 kg) in 1981 and then returned to the earlier 10,800 pounds in 1985.

The designed and constructed pavement section of MH 153 consists of a 50-foot (15.2 m) overall width roadway surface with a top layer of 3/4 inch (19.1 mm) of skid-resistant SDHPT Specification Item 340, hot-mix asphaltic concrete over the six-inches (152 mm) of the previously noted Item 292 asphalt stabilized base. The Item 292 asphalt stabilized base layer in turn is supported by six-inches (152 mm) of lime stabilized silty clay subgrade. The two 11-foot (3.4 m) northbound lanes of MH 153 have a concrete curb and gutter for pavement section edge support and are separated from the southbound 11-foot (3.4 m) passing and 12-foot (4.0 m) traveled lanes by a 4-foot (1.2 m) double striped median area. The southbound lanes have no curb or even improved shoulder for pavement section edge support, and the SEA field trials are located on these lanes from project stations numbers 48+00 to 75+00.

The lack of pavement lateral support for the southbound lanes of MH 153, especially in the area of the field trial sections, is worth noting. In this area, these lanes have been built on generally what could be called a "fill" section, and therefore the Item 292 and Item 340 hot-mix asphaltic concrete layers are generally "daylighted" at the top of an approximately 3.5:1 fill slope. This exposure and lack of side support is believed to be contributing significantly to longitudinal cracking in the outside wheel path of all the trial sections, and also rutting in some sections, especially from the right wheel path out to the pavement edge of the traveled lane. The observed rutting in the outside wheel path is attributed to minimal lateral support.

Laboratory Design Procedure

Laboratory design procedures were consistent with the objectives of the MH 153 field trials. The hot-mixed SEA materials were designed to possess desirable stability, air void and other characteristics to serve adequately in the 6-inch (152 mm) layer of asphalt stabilized base under the anticipated traffic loadings during the pavement design life. With respect to laboratory design, Izatt and Gallaway in Report FHWA-TS-80-214 (4) recommended that "Any agency considering the use of SEA as a binder for paving mixtures should examine their standard materials requirements and use those design and test procedures normally used to produce paving mixtures made with pure asphalt cement, considering the associated effects of traffic and the environment". Based on their experiences, these Texas Transportation Institute (TTI) researchers chose the Marshall method of mix design supplemented with Schmidt's resilient modulus, the Texas Hveem stability test and the indirect tension test for further mixture evaluation.

With the exception of not using the SDHPT's own design job mix formula with SEA binder in the TTI design, the main considerations of the SEA laboratory mixture design procedure used by TTI for the MH 153 trials are summarized below.

1. SEA mixture designs involved two different aggregate blends and two different SEA binder compositions in addition to the pure asphalt cement used in the two aggregate blends as controls. Optimum binder contents were determined by the Marshall method. A summary of the binders, aggregate blends and test results obtained by the TTI designs is shown in Table 1 (4).

2. As shown in Table 1, the two SEA binders used were a 30/70 SEA binder which indicate 30 percent and 40 percent sulphur by weight of total binder, respectively. The two aggregate blends used were a 75:25 blend of bank run gravel and field sand and a 50:50 blend of concrete sand and field sand. (It is noted that use of these blends with SEA binders may be considered an effort at beneficiating "marginal" aggregates by using rounded, siliceous type materials from short haul local sources with ample supply.

3. Based on TTI experience, SEA binders have usually been substituted for the pure asphalt cement in design mixtures on an equal volume basis. Thus, volumes of SEA binder at or near the volume of pure asphalt cement at its optimum design percentage by weight may be used as a starting point to find the optimum SEA binder content.

4. Optimum binder contents recommended for the MH 153 field trials were determined for each of the SEA binder compositions and aggregate blend combinations based upon an overall evaluation of the results of

Table 1. Laboratory design results for MH 153 trial sections (4).

Aggregate Combinations			Binder Composition		Binder Contant	Marshall*	M_R	Air Voids	Hveem
Pit run Gravel, pct wt	Conc. Sand, pct wt	Field Sand, pct wt	Sulphur Content, pct wt	Asphalt Content, pct wt	Binder, pct wt	Stability, lbs	Resilient Modulus, x 10^6 psi	Air Voids, Pct vol	Stabilometer Value, Percent
75		25		100	5(0/100)	800	0.670	14	30
					6	1225		12	39
					7	1450	0.730	5	39
					8	1375			41
**75		25	30	70	6(30/70)	1400	0.520	10	36
					7	1650	0.500	8	40
					8	1850	0.660	5	40
**75		25	40	60	6(40/60)	1750	0.510	10	44
					7	2050	0.640	8	42
					8	2300	0.660	5	40
	50	50		100	6(0/100)	1300	0.490	16	30
					7	1500	0.290	14	28
					8	1650	0.350	9	25
**	50	50	30	70	6(30/70)	550	0.190	13	32
					7	750	0.230	9	32
					8	800	0.225	8	31
**	50	50	40	60	6(40/60)	1000	0.330	13	34
					7.5	1100	0.525	10	31
					8.5	1100	0.315	7	30

* Marshall and Hveem values taken from data plots (4).
**Denotes designs actually used in MH 153 construction.

Metric Conversion: 1 pound force = 4.448 newtons
1 psi = 6.894 kPa

Marshall stability, resilient modulus, air voids and Hveem stability values shown in Table 1. These binder contents are summarized in Table 2.

As shown in Table 2, the binder contents used for the job mix formula using a 55:30:15 blend of bank run river gravel: pea gravel: field sand were five percent by weight of mixture for both control Section 8 and the 40/60 SEA binder Section 2. As a result, the design volume in Section 8 was 11.6 percent of the total mix compared to 9.6 percent for Section 2 because of the reduction in volume from the 40 percent by weight of binder of higher specific gravity sulphur in Section 2. Average volume percents of conventional asphalt concrete mixture are in the range of 12 to 14 percent, depending primarily on the top size of the aggregate in the design. On a volume comparison basis the binder volume used in Section 2 is low by about 27 percent. The performance of Section 2 over the past about seven years has been excellent. Binder contents for SEA Sections 3 through 7 ranged from 7.0 percent by weight of mix to 8.5 percent and from 13.8 percent by volume of mix to 15.7 percent. Volume contents are calculated based on the following specific gravities of these constituent materials used in the MH 153 sections: asphalt, Exxon AC-20 = 1.03, sulphur = 2.07 and aggregate - 2.58.

The above optimum SEA binder contents reflect the design considerations in Table 1 of (1) achieving Marshall stabilities above the minimum 750 pounds (3,335 newtons) required for "Heavy" traffic category, (2) satisfying the mixture demand for binder and reducing laboratory air voids to a maximum of eight percent or less and (3) achieving a minimum Texas Hveem stability of 25 required for the six-inch (152 mm) Item 292 stabilized base layer. Considering the marginal aggregate blends employed in Sections 3 through 7, the laboratory air voids were reduced as much as possible without adversely affecting Marshall and Hveem stability values.

Table 2. Optimum design binder content chosen for each trial section (4).

MH 153 Trial Section	Design Aggregate Blend by Wt.	Design SEA Binder Ratio by Wt.	Design Binder Content of Mixture Pct. by Wt. of Mix	Design Binder Content of Mixture Pct. by Vol. of mix ***
2	55:30:15 Job Mix Formula	40/60	5	9.5
3	75:25 Bank run gravel: field sand	40/60	7.5	14.0
4	75.25 Bank run gravel: field sand	30/70	7.0	13.8
5**	75:25 Concrete sand: field sand	30/70	7.0	13.8
6	50:50 Concrete sand: field sand	40/60	8.5	15.7
7	50:50 Concrete sand: field sand	30/70	8.0	15.6
8* (Control)	55:30:15 Job Mix Formula	*0/100	5.0*	11.6

* Pure AC-20 asphalt cement was used.

** Comingling sulphur and asphalt in bypass line around colloid mill.

*** Calculated based on following specific gravities: Exxon AC-20 asphalt = 1.03, sulphur = 2.07 and combined aggregates = 2.58.

Field Construction Procedures

Six SEA test sections, Sections 2 through 7 were actually constructed on the MH 153 southbound lanes using the two SEA binders and the three different aggregate blends. Section 8 which utilized pure asphalt cement and the job mix formula aggregate blend constituted the control section. The binder types and the actual amounts used in each section as determined by extractions are summarized as shown in Table 3 from the MH 153 Construction Report (4).

The construction of the SEA test sections began on June 16, 1978 and was completed on June 23, 1978 (4). As shown in Table 3, each of the 450-foot (137 m) SEA sections was constructed in three two-inch (51 mm) lifts with the third lift being the top layer of the six-inch (15 mm) Item 292 paving layer. A total of 2,571 tons (2,332 metric tons) of the SEA binder hot-mixed Item 292 material was placed on Sections 2 through 7 with no apparent problem.

TTI researchers sought to maintain the production temperatures of the SEA binder hot-mixed Item 292 materials in the range from 235°F to 300°F (113°C to 149°C) (4). Actual temperatures measured ranged from a low of 230°F (110°C) on June 20 to a high of 300°F (149°C) obtained on June 16 and June 23 (4). Statistical analysis of temperature measurements showed that the batch plant during this production could have been expected to maintain temperatures within 225°F to 300°F (107°C to 149°C) ninety-five percent of the time (4). During the construction, any SEA binder material that was found to be outside the desired temperature range was rejected for use on the MH 153 roadway. Actually, one truck load of about twelve tons was rejected due to high temperatures.

Concerning plant aggregate gradation control, some problems were encountered. According to records (4), the percents by weight of material passing the No. 40 (0.42 mm) sieve for the 75:25 aggregate blend showed considerable variation as evidenced by large standard deviations in testing results. Also, the spread of actual ratios obtained for the nominal 50:50 concrete sand:field sand aggeregate blend ranged from 50:50

Table 3. Extraction results of produced SEA mixtures.

Test Section	Station to Station	SEA Binder Ratio	Lift Thickness Inches	Extracted Wt. Pct. of Binder
2	48+00 to 52+50	40/60	2 2 2	5.0 5.0 ---
3	52+50 to 57+00	40/60	2 2 2	7.5 7.5 6.8
4	57+00 to 61+50	30/70	2 2 2	--- 7.0 6.3
5	61+50 to 66+00	30/70	2 2 2	7.0, 7.0 7.0 6.3
6	66+00 to 70+50	40/60	2 2 2	--- 8.5 7.9
7	70+50 to 75+00	30/70	2 2 2	--- 8.0 7.4
8 Control	75+00 to End	0/100	2 2 2	5* 5* 5*

* Weight percent of pure asphalt cement based on SDHPT design.

Metric Conversion: 1 inch = 25.4 mm.

to 90:10. Normally, the 90:10 blend would be unacceptable; however, this blend was placed successfully. This attests to the forgiving nature of these fine graded SEA mixtures.

Concerning compaction, the SEA binder materials in Sections 2 through 7 were compacted at the highest possible temperature achieve the greatest density. Two methods were used to compact the three 2-inch (51 mm) layers of the SEA binder hot-mixed Item 292 material. These are described below:

1. Method 1: The first 2-inch (51 mm) lift of Sections 2 through 6 was compacted by a Rex 900 Model SPVIB vibratory roller using one breakdown pass without vibration.

2. Method 2: The first lift of test Section 7 and all other lifts on all other sections were compacted first with the Rex 900 as described in 1 above and then subjected to eight passes with an Ingram 5-4 (SPR14) 25-ton (22,765 kg) pneumatic tired roller.

The reason for changing to the second method of compaction was that undue lateral displacement occurred in the SEA binder hot-mixed Item 292 materials of lift number 1 at temperatures above 230°F (110°C); so breakdown rolling had to be delayed until, in the inspector's and roller operator's judgment, the mat had cooled sufficiently to allow rolling to begin. By the time that breakdown rolling could begin, the mat had cooled to the point that rolling with the 25-ton (22,765 kg) pneumatic roller produced compaction.

Concerning the conditions affecting rolling, Izatt and Gallaway stated that "...the internal resistance of these sulphur-asphalt mixes was so low that compaction was not critical to temperature above ambient and below that required to support the rollers. This was due to aggregate grading and particle shape and surface texture and, in part, to very hot weather" (4). Finally, field densities taken with a nuclear density gage verified the adequacy of the two methods of rolling and indicated a beneficial gain from adding the eight passes of the pneumatic roller (4). The pneumatic roller also removed most of the surface roller markers created by the Rex 900 (4). All surface irregularities in the six inches of Item 292 (black base) were obliterated by the final surface course, a dense graded asphalt concrete.

IV. DESCRIPTION OF TESTING ON MH 153

The following is a list of the laboratory and non-laboratory tests that have been conducted on Sections 2 through 8 following construction of the SEA field trials on MH 153.

A. Laboratory tests
   1. Density
   2. Marshall stability
   3. Marshall flow
   4. Hveem stability
   5. Resilient modulus, M_R
   6. Indirect tension
   7. Rice maximum specific gravity
   8. Extracted binder content

B. Non-laboratory tests
   1. Visual evaluation and PRS
   2. Mays Ride Meter and SI
   3. Dynaflect deflection
   4. Traffic data

The sections that follow briefly describe each test and give references where the actual test procedures may be found.

Laboratory Tests

Density

The term density is used herein to describe the unit weight of a material and is expressed in pounds per cubic feet or kilogram per cubic meter. The density of a specimen of a compacted bituminous mixture may be determined by multiplying its bulk specific gravity times the unit weight of water at 60°F (15.6°C). "The Standard Test Method for BULK

SPECIFIC GRAVITY OF COMPACTED BITUMINOUS MIXTURES USING SATURATED SURFACE-DRY SPECIMENS" is found in ASTM D2626-73 (6,7).

The density of a compacted bituminous mixture may be used to determine the air voids or relative percent density of a compacted specimen when both the bulk specific gravity and the theoretical maximum specific gravity are known. Density data may be used to evaluate the effectiveness of field compaction, indicate changes in aggregate gradations specific gravities and indicate changes in absorption of aggregates.

Marshall Stability

The Marshall stability test is utilized to measure the resistance to plastic flow of compacted cylindrical specimens of bituminous paving mixtures in pounds of force (4.448 newtons), of either laboratory produced specimens or field cores taken from roadway pavements, when these specimens are loaded on their lateral surfaces by means of Marshall apparatus. The "Standard Test Method for RESISTANCE TO FLOW OF BITUMINOUS MIXTURES USING MARSHALL APPARATUS" is found in ASTM D1559-76 (6,7).

The Marshall method of mixture design may be used with mixtures containing asphalt cement, asphalt cut-back or tar, provided the aggregates do not exceed one inch (25 mm) in maximum size. The method is used to design bituminous paving mixtures with sufficient stability to resist pavement distresses such as shoving, rutting and corrugations that occur due to plastic deformation under excessive traffic loads. The Marshall method may also be used to help evaluate the in-place stabilities of existing pavement layers through the testing of field cores.

Marshall Flow

This test is part of the above noted Marshall stability test. Flow measures the maximum deformation of a cylindrical specimen of bituminous mixture in 0.01 inches (0.25 mm) at the peak load of the Marshall stability test.

Under the Marshall design criteria, values for flow should fall within certain ranges for each category of traffic loading. A flow result below a certain range of values may indicate paving mixtures that are too brittle; a flow above a certain range may indicate a mixture subject to excessive plastic deformation.

Hveem Stability

The Hveem stability method of test measures the internal resistance, predominantly from the interparticle friction of the aggregates, of compacted cylindrical specimens of bituminous mixtures. The lateral pressure developed by the specimen from applying a vertical load on the specimen by means of a Hveem stabilometer results in a number called the Hveem stability. The "Standard Test Methods for RESISTANCE TO DEFORMATION AND COHESION OF BITUMINOUS MIXTURES BY MEANS OF HVEEM APPARATUS" are found in ASTM D1560-76 (6,7).

The Hveem stability method of test is used to design bituminous mixtures to serve under different categories of traffic loadings. Mixture designs with high Hveem stabilities from 35 to 45 (percent) are usually required on roadways with high volume traffic with a high percentage of heavily loaded trucks, such as interstate and municipal highways. The Hveem stability test may also be used to evaluate the stabilities of existing pavement layers from the testing of field cores.

Resilient Modulus

The Resilient modulus test measures the time-dependent modulus of elasticity of pavement cores or compacted cylindrical specimens of bituminous mixtures. Because these mixtures are viscoelastic, thus exhibiting more deformation with increasing time duration of loading, the conditions of testing, including the time of loading, must be carefully defined.

The TTI testing procedure uses a Mark III Resilient Modulus (M_R) device which applies a 0.1-second load pulse every three seconds across the vertical diameter of a cylindrical specimen (6). The resultant deformation or elongation of the horizontal diameter is sensed and recorded after 0.10 second from the beginning of the deformation, and this deformation is used in Schmidt's equation to compute the resilient modulus, (M_R), (8).

The resilient modulus test may be used both to assist in the design of a bituminous mixture as well as to evaluate the strength of cores taken from existing pavement layers. The test may be used to evaluate water susceptibility, changes in material stiffness and differences among different types of paving materials according to Schmidt (8). Because of the nature of the method, the resilient modulus test is one of the better simulators of actual traffic loading of compacted bituminous materials.

Indirect Tension

This test may be used to estimate the tensile strength, Poisson's ratio and static modulus of elasticity (as opposed to the resilient modulus of elasticity) of compacted cylindrical specimens or field cores of bituminous mixtures. Under this test, compressive loads are applied which act parallel to and along a vertical diametral plane of a cylindrical specimen. Horizontal and vertical deformations are recorded during testing and the applied load at failure is used to calculate the tensile strength. The procedure is found in a study by Gonzales, et al. (9). The test is usually run at $68^{o}F$ (20^{o}).

The indirect tension test may be used to measure the strengths of laboratory compacted specimens of bituminous mixtures or cores taken from pavement layers in the field. Data obtained from indirect tension testing or cylinders of laboratory compacted bituminous mixtures or field cores before and after water saturation procedures may be useful in predicting water susceptibility.

Rice Maximum Specific Gravity

This test method allows the determination of the theoretical maximum specific gravity of a bituminous mixture. The Rice maximum specific gravity gives a rapid check on the maximum density of a bituminous mixture being produced or on materials taken from roadway pavement layers. When used with the bulk specific gravity as determined from ASTM D2726-73 (See Density), the percent density and percent air voids in compacted laboratory specimens or field cores obtained from a roadway may be determined. This test may also be used to indicate when changes in materials in a bituminous mix are occurring such as changes in the specific gravity or the gradations of the aggregates. The "Standard Test Method for THEORETICAL MAXIMUM SPECIFIC GRAVITY OF BITUMINOUS PAVING MIXTURES" is found in ASTM D2041-78 (6,7).

Extracted Binder Content

This test method is used for the quantitative determination of the amount of bitumen in freshly produced hot-mixed paving mixtures and samples from existing pavements. In this test, the asphalt in a paving material is put into solution in a solvent such as trichloroethylene, 1,1,1-trichloroethane or benzene and is thus "extracted" from the paving mixture, leaving only the aggregate.

The extracted binder content is usually used for quality control testing for verifying asphalt content during production of bituminous mixtures at weigh-batch, drum-dryer and other plants. This test is also used to determine bitumen contents of field cores from existing paving layers or pavement slab samples. The "Standard Test Methods for QUANTITATIVE EXTRACTION OF BITUMEN FROM BITUMINOUS PAVING MIXTURES" is found in ASTM D 2172-81 (6,7).

Non-Laboratory Tests

Visual Evaluation

The visual evaluation method was developed by Epps, et al. (10). It provides a procedure whereby a trained observer may evaluate the condition of a roadway from right-of-way line to right-of-way line according to stated guidelines and derive a Pavement Rating Score, PRS, for the pavement condition and also ratings for condition of shoulders, roadsides, drainage features and traffic services.

For any highway pavement the visual evaluation method provides consistent guidelines to obtain a PRS value from deductions of points from a perfect pavement score of 100. The PRS value includes rutting, raveling, flushing, corrugations, alligator cracking, longitudinal cracking, transverse cracking, the state of cracks, sealing failures and pavement roughness as measured by a Mays Ride Meter. By closely following the visual evaluation procedure during periodic inspections, the condition of a roadway surface may be accurately tracked over a period of time.

The visual evaluation method may serve as an indicator of trends in condition for a roadway and help indicate when necessary remedial action should be taken to restore a pavement. The visual evaluation should be done on a regular, periodic basis in order to keep abreast of the needs for rehabilitative maintenance or reconstruction.

Mays Ride Meter

This test was developed by Ivan K. Mays of the Texas SDHPT for measuring the roughness of pavement surfaces (11). From the accumulated roughness measured and indicated by a reading for each 0.05-mile (0.08 km) or 0.20-mile (0.32-km) increment of a roadway lane, the Serviceability Index, SI, may be obtained.

The Mays Ride Meter may be used at periodic intervals to monitor the rate of decline in Serviceability Index of a roadway. Theoretically, SI values can range from maximum rating of 5.0 for a "perfectly smooth" roadway surface to the lowest value of 1.0 for an extremely rough surface. When the SI value for a roadway surface drops to a range from 2.5 to 2.0, the roadway surface has need of remedial measures to restore its riding quality. Also, rapid drops in SI value may serve to indicate severe structural distresses occurring in underlying pavement layers.

Dynaflect Deflection

Under this test, a pavement surface is subjected to the force of a dynamic oscillating load and the pavement deflection is measured at the maximum loading directly under this load and at four other locations outside the point of the load by geophones to establish a pavement deflection basin from the loading. From this deflection profile and the maximum deflection of the pavement structure, directly under the applied load a pavement stiffness coefficient and a subgrade stiffness coefficient are estimated.

The Dynaflect deflection test apparatus was developed by Lane Wells, an SIE corporation. A description of this test method is found in a study by Hankins of the Texas SDHPT for the Federal Highway Administration (12).

This test is useful for monitoring the structural condition of roadways. Test results may indicate adequate structural performance; or, increasing deflections and decreasing coefficients may give warning that future problems may be expected or that a pavement is deteriorating rapidly.

Traffic Data

At approximate intervals of twelve months, manual traffic counts were made on MH 153 and from these counts data such as that in Table 4 were compiled. Like so many other estimates for traffic, the total traffic and the percent and weight of truck traffic greatly exceed that used in the 20-year design of the facility. The traffic data shown in Table 4 are not unexpected for a fast growing area such as Bryan-College Station, Texas.

Table 4. Traffic analysis for MH 153.

	1977	1978	1979	1980	1981	1982*	1983*	1984	1985
1. Average Daily Traffic (ADT)	7680	8340	9060	9840	10,690	14,180	10,620	--	12,000
2. Directional Distribution Factor, percent	60-40	60-40	60-40	60-40	60-40	64-36	64-36	--	63-37
3. Design Hourly Volume (DHV), percent	11.7	11.7	11.7	11.7	11.7	11.8	9.4	--	10.0
4. Percent Trucks									
a. ADT	6.1	6.1	6.1	6.1	6.7	2.2	3.0	--	2.0
b. DHV	4.1	4.1	4.1	4.1	4.1	1.1	1.4	--	1.3
5. Anticipated Annual Rate of Growth, percent	8.6	8.6	8.6	8.6	8.6	13.0	4.8 (2.0)	--	3.4
6. Average of Ten Heaviest Wheel Loads Daily (ATHWLD)	10,800 lb (4903 kg)	10,800 lb (4903 kg)	10,800 lb (4903 kg)	10,800 lb (4903 kg)	12,600 lb (5720 kg)	9,100 lb (4131 kg)	10,250 lb (4642 kg)	-- --	10,800 lb (4,899)
7. Tandem Axles in ATHWLD, percent	60	60	60	60	60	50	80	--	75

*See discussion on pages 6 and 10.

V. RESULTS SUMMARY FOR EACH TEST CONDUCTED ON MH 153 TRIAL SECTIONS

Tables 4,5,6 and 7 show the results of all post-construction testing on MH 153 from July, 1978 through December, 1985, both on Sections 2 through 7 of the SEA trials and on the control section, Section 8. These four tables are extracted from previous Progress Reports and supplemented with data taken during the final evaluation in 1985. It is noted that no values are shown for Section 1 on these tables. Section 1 is a control section with the same paving mixture as Section 8, but no actual tests were taken and recorded for this section for use in the MH 153 study. Section 8 functioned as the control section.

Laboratory Tests

Density

As shown in Table 4, density tests for Sections 2 through 8 from 1978 through 1985 indicate that the densities or cores obtained from each section have not changed significantly over a period of seven years. It is interesting to note that the average of density tests for Section 2 and the control Section 8, both using the job mix formula, are equal at approximately 144 pcf (2311 kg/m^3) at the end of seven years. Sections 3, 4 and 5 using the two different SEA binders and the same 75:25 bank run river gravel: field sand aggregate gradation all show an average value for density of about 134 pcf (2151 kg/m^3) after seven years. Finally, Sections 6 and 7 using the two different SEA binders but the same 50:50 concrete sand: field sand aggregate gradation also show about the same density of 136 pcf (2183 kg/m^3).

The above tests reveal stable densities in the SEA field trial and control sections and would seem to indicate that no individual section is undergoing excessive decompaction deformation or consolidation deformation. This appears to provide an indication of good pavement performance since the compositions of the original Item 292 paving materials apparently have not changed appreciable since placement.

Table 5. Post construction laboratory testing of MH 153 field cores (5).

Sulphur/Asphalt Ratio (Section Number) Binder Type Aggregate Mixture	Density, pcf	Marshall Stability, lbs.	Marshall Flow 0.01 inch	Hveem Stability, Percent	Resilient Modulus, Mr. @ 68°F (20°C), 10^6 psi	Indirect Tension, psi	Rice Maximum Specific Gravity	Extracted Binder Content Percent***	Date Field Cores Taken
40/60*	139	270	14	18	0.19	40	2.46		7/17/78(1)****
(Section 2)	146	590	10	28	0.80	145			12/18/78(6)
SEA	145	690	12	30	0.77	110		6.3	7/23/79(12)
Job mix formula	144	360	12	24	1.10	205			9/15/80(26)
55:30:15 Bank river	145	430	15	21	0.97	220	2.44		11/18/81(41)
gravel: pea gravel:	143	710	11	34	0.53	110	2.45		
field sand									
									1/9/85(79)
40/60*	132	160	15	12	0.13	45	2.42		7/17/78(1)****
(Section 3)	134	330	9	32	0.76	125			12/18/78(6)
SEA	133	540	15	22	0.63	150		6.7	7/23/79(12)
75:25 Bank river	137	170	20	12	0.47	140			9/15/80(26)
gravel: field sand	134	340	20	9(?)	0.57	165	2.40		11/18/81(41)
	134	700	17	27	0.56	155	2.46		1/9/85(79)
30/70*	131	170	15	12	0.11	35	2.40		7/17/78(1)****
(Section 4)	134	330	11	21	0.74	150			12/18/78(6)
75:25 Bank river	134	350	15	20	0.46	130		7.3	7/23/79(12)
gravel: field sand	136	330	12	22	0.60	160			9/15/80(26)
	135	210	12	16	0.42	165	2.39		11/18/81(41)
	135	448	22	21	0.48	160	2.44		1/9/85(79)
30/70*	132	210	17	15	0.11	40	2.40		7/17/78(1)****
(Section 5)	134	300	11	21	0.84	160			12/18/78(6)
Sulphur-Asphalt**	134	430	17	18	0.46	135		7.2	7/23/79(12)
75:25 Bank river	136	390	18	19	0.60	150			9/15/80(26)
gravel: field sand	135	260	14	15	0.59	170	2.40		11/18/81(41)
	135	490	13	29	0.36	135	2.42		1/9/85(79)

Table 5. (Continued).

Sulphur/Asphalt Ratio (Section Number) Binder Type Aggregate Mixture	Density, pcf	Marshall Stability, lbs.	Marshall Flow, 0.01 inch	Hveem Stability, Percent	Resilient Modulus, Mr, @ 68°F (20°C), 10^6 psi	Indirect Tension, psi	Rice Maximum Specific Gravity	Extracted Binder Content Percent***	Date Field Cores Taken
40/60*	135	100	14	16	0.19	40	2.36		7/17/78(1)****
(Section 6)	135	190	12	20	0.51	180			12/18/78(6)
50:50 Concrete	136	260	12	20	0.38	130		8.8	7/23/79(12)
sand: field sand	138	560	19	20	0.72	145			9/15/80(26)
	137	160	11	20	0.56	190	2.39		11/18/81(41)
	135	320	10	28	0.32	145	2.42		1/9/85(79)
30/70*	135	130	10	15	0.19	40	2.37		7/17/78(1)****
(Section 7)	136	140	11	19	0.42	150			12/18/78(6)
50:50 Concrete	136	150	15	19	0.22	110		9.2	7/23/79(12)
sand: field sand	-(1)	-(1)	-(1)	-(1)	-(1)	-(1)			9/15/80(26)
	137	100	10	17	0.39	115	2.38		11/18/81(41)
	137	360	9	24	0.28	160	2.40		1/9/85(79)
0/100	-(2)	-(2)	-(2)	-(2)	-(2)	-(2)	-(2)		7/17/78(1)****
(Section 8)	143	310	11	26	0.89	140	2.44		12/18/78(6)
AC Control	141	190	13	14	0.49	155		5.0	7/23/79(12)
Job Mix Formula	146	680	14	23	0.94	130			9/15/80(26)
55:30:15 Bank River	146	460	14	26	1.11	205	2.41		11/18/81(41)
gravel: pea gravel:	141	540	14	26	0.42	140	2.47		1/9/85(79)
Field sand									

*Weight percentages of sulphur-asphalt binder.

**Sulphur-asphalt binder prepared by bypassing the colloid mix.

***Weight percent based on total mixture.

****Age of pavement in months.

(1) Field cores tested and reported in 9/15/80 did not belong to Section 7 so test results are not reported here.
(2) No tests were taken for Section 8 on 7/17/78.
Metric conversion: 1 pound force, lbs, = 4.448 newtons
1 inch = 25.4 mm
1 psi = 6.894 kPa

Table 6. Binder contents determined for MH 153 trial sections.

Trial Section No.	SEA Binder Composition	Design Binder Content	Construction Binder Contents (4)	July, 1979 Binder Content (5)
2	40/60	5.0	5.0, 5.0	6.3
3	40/60	7.5	7.5, 6.8	6.7
4	30/70	7.0	6.3, 7.0	7.3
5	30/70	7.0	7.0, 7.0 7.0, 6.3	7.2
6	40/60	8.5	7.9, 8.5	8.8
7	30/70	8.0	8.0, 7.4	9.2
8	0/100	5.0	---*	5.0

*No record of SDHPT extraction results were obtained for Section 8.

Table 7. Pavement rating scores (PRS) for MH 153.

Binder and Aggregate Type	PRS	Date
40/60 SEA Job Mix, Section 2	100	12/18/78
	100	6/29/79
	83	12/12/80
	83	12/01/81
	80*	6/30/83
	84	11/29/83
	84	6/27/84
	82	2/21/85
40/60 SEA 75:25 Bank Run Gravel: Field Sand Section 3	100	12/18/78
	98	6/29/79
	88	12/12/80
	85	12/01/81
	75*	6/30/83
	81	11/29/83
	80	6/27/84
	80	2/21/85
30/70 SEA 75:25 Bank Run Gravel: Field Sand Section 4	100	12/18/78
	97	6/29/79
	93	12/12/80
	85	12/01/81
	80	6/30/83
	80	11/29/83
	78	6/27/84
	80	2/21/85

*These values are questionable.

(Continued)

Table 7. Continued.

Binder and Aggregate Type	PRS	Date
30/70 SEA 75:25 Bank Run * Gravel:Field Sand Section 5	100	12/18/78
	98	6/29/79
	93	12/12/80
	85	12/01/82
	85	6/30/83
	88	11/29/83
	87	6/27/84
	87	2/21/85
40/60 SEA 50:50 Concrete Sand:Field Sand Section 6	100	12/18/78
	100	6/29/79
	93	12/12/80
	88	12/01/81
	87	6/30/83
	92	11/29/83
	90	6/27/84
	91	2/21/85
30/70 SEA 50:50 Concrete Sand:Field Sand Section 7	100	12/18/78
	100	6/29/79
	88	12/12/80
	80 **	12/01/81
	85	6/30/83
	87	11/29/83
	84	6/27/84
	85	2/21/85
0/100 AC Control Section 8	100	12/18/78
	100	6/29/79
	93	12/12/80
	83	12/01/81
	90	6/30/83
	83	11/29/83
	88	6/27/84
	80	2/21/85

*Sulfur-asphalt binder was prepared by bypassing the colloid mill.

**This value is questionable.

Marshall Stability

As shown in Table 5, Marshall stability values for each section from 1978 to 1985 show considerable variablity from test date to test date. Section 2 with the 40/60 SEA binder and the job mix formula has the highest average stabilitiy of 520 pounds (2,310 newtons). Sections 3, 4 and 5 are grouped closely together with their average stabilities ranging from 300 to 380 pounds (1,340 to 1,700 newtons). Section 6 ranging from 300 to 380 pounds (1,340 to 1,700 newtons). Sections 6 and 7 are the lowest with average Marshall stabilities of 270 and 180 pounds (1,200 and 800 newtons), respectively.

Based on the limited Marshall data available, as shown in Table 4, the apparent trend is for Marshall stabilities to be declining for Sections 4, 5, 6 and 7 which may indicate increased pavement distress in the near future for these sections. Concerning overall magnitudes, Marshall stabilities for all sections (with exception of one or two values obtained for Sections 2, 3, 6 and 8 from 1978 to 1985) have remained below the "Light" traffic category requirement of 500 pounds (2,220 newtons) called for in the Marshall method of design.

Marshall Flow

For all sections and the control, Table 5 shows that flow values have been variable but have not changed significantly for the seven years of testing. For all sections, the average values would meet the allowable maximum and minimum flow values for all the traffic categories of the Marshall method of design (14).

Individual flow values for all trial sections, with the possible exception of Section 3 with the 40:60 SEA binder and the 75:25 bank run river gravel:field sand aggregate blend, indicate adequate service at the present and also appear to predict satisfactory future performance. Flow values for Section 3 pavement cores measured 20 in 1980 and 1981 and thus show a weakening trend. This was offset in 1985 with a flow of 17. High values of flow were also measured for Sections 3 and 4 in 1985.

Hveem Stability

Table 5 indicates that Hveem stability values have been variable for all sections except Section 6 with the 40/60 binder and 50:50 concrete sand:field sand aggregate blend. Section 2 and Section 8, the control with the job mix formula aggregate blend, may be grouped together with the highest average Hveem stabilities of 26 and 23, respectively. Average stabilities for Sections 3 through 7 can all be grouped closely together and range from 19 to 21.

Trends in Hveem stability are difficult to determine; although, stability values for Sections 2 and 3 appear to be in apparent decline. Stability values determined from Section 6 field cores show a constant value of 20 for the four testing periods from December, 1978 through November, 1981. Based upon the most recent stability tests obtained in 1985, all sections show Hveem stability values that are higher than expected; however, cores taken at this sampling period were of excellent quality. This has not been the case for some of the cores taken in previous years.

Resilient Modulus

Resilient modulus, M_R, values for all sections are quite variable over the seven years of testing. The highest average M_R of 858,000 psi (5.92×10^6 kPa) is found for Section 8, the control with the asphalt cement binder; Section 2 also had a value almost as high. The lowest average is found for Section 7 with a M_R of 305,000 psi (2.10×10^6 kPa). Of the SEA binder sections, Section 2 has the highest average M_R, and Sections 3, 4, 5 and 6 have average M_R values ranging from 465,000 to 570,000 psi (3.20×10^6 x 3.93×10^6 kPa) at 68°F (20°C).

Values of M_R for all sections indicate satisfactory performance at present and signal good performance for the future. Although the values of M_R for Section 7 are significantly lower than any other section, they are not declining and appear to be at least holding in the region of 300,000 psi (2.07×10^6 kPa).

Of interest to note for each of the SEA pavement sections is the consistently low initial value of M_R obtained in July of 1978. This value is probably reflective of the effect from some supercooled sulphur still remaining in the SEA binder and causing the mixture to stay tender with less initial strength at the early age of less than one month.

Indirect Tension

Indirect tension values obtained for all sections since 1978 have been variable for each section. The highest average indirect tension value of 155 psi (1,078 kPa) was obtained for Section 8, the control section, using the job mix formula. The highest average indirect tension for the SEA sections was for Section 2 using the 40/60 SEA binder and the job mix formula aggregate blend. The next highest indirect tension average was found for Section 6 using the 50:50 concrete sand:field sand aggregate blend. Sections 3, 4 and 5 are close together in values with indirect tension averages ranging from 130 to 133 psi (900 to 915 kPa). The lowest average indirect tension of 115 psi (790 kPa) was obtained for Section 7; using the 50:50 concrete sand:field sand aggregate blend with 30/70 SEA binder.

Indirect tension values for all sections except Section 7 indicate satisfactory performance at the present and probably in the future. Indirect tension values for Section 7 have dropped below 125 psi (860 kPa) since late 1978; however, this section continues to perform satisfactorily.

Rice Maximum Specific Gravity

Values for Rice maximum specific gravity were obtained three times, in July, 1978, November, 1981, and January, 1985. The results of these tests, based on minimal data, indicate no significant changes in specific gravity with time. This is as it should be.

Extracted Binder Content

Extracted binder contents were taken only on one occasion, in July, 1979, on cores from the field trial sections. These values do not in themselves constitute enough data upon which to draw conclusions. Table 6 shows the MH 153 design SEA binder contents, construction measured SEA binder contents and the July, 1979 measured binder content for each trial section.

Table 6 does show that most of the binder contents determined from the produced SEA binder materials, either from sampling during construction or from the field cores taken in 1979 were approximately equal to the desired design binder contents. Exceptions are for Section 7, with a measured binder content of 9.2 percent, which was much higher than that called for by the design; whereas, for Section 2 with an SEA binder content of 5.0 percent, which was lower than the 6.3 percent design value.

Non-Laboratory Tests

Visual Evaluation

The results of eight visual evaluation tests since 1978 have yielded Pavement Rating Scores, PRS, for each of Sections 2 through 8, which are shown in Table 7. For every section, the PRS has declined from 100 to a value in the 80's. As of January, 1985, Section 6 had the highest PRS of 91 and Sections 3, 4 and 8 had the lowest of 80.

As seen in the decreasing PRS's, each trial section is experiencing some increasing pavement distress with the passage of time, particularly the distresses of longitudinal cracking, within and near the wheel paths, and some slight rutting in the outside wheel path of the travel lane for two sections. The longitudinal cracking distresses are particularly evident from the right wheel path of the outside lane out to the pavement edge and are due, at least in part, to the lack of lateral support for the six-inch (152 mm) Item 292 hot-mixed asphaltic layer caused from the absence of either a paved or unpaved shoulder at the vertical edge of the pavement layer.

Mays Ride Meter

Mays Ride Meter readings for Sections 2 through 8 have been taken on five occasions in the southbound travelled lane of MH 153 since completion of the field trials as shown in Table 8. With the exception of Serviceability Indices, SI's, determined in the Section 7 area, SI values for the other trial sections show normally expected declines from 1978 through 1985. The SI data for the year 1980 were unrealistically high, due most likely to the Mays Meter being out of calibration, and for this reason the 1980 data are omitted.

Significant declines in the Serviceability Index readings have occurred for the surfaces of Sections 5 and 6 from May, 1979 to December, 1985 indicating increased surface roughness for these sections and possibly indicating pavement layer distresses such as stability related problems. SI values for Section 7 show anomalous behavior, with values going up from May, 1979 to December, 1981. Inadequate machine calibration may explan these numbers.

Sections 5 and 6 show 1985 Serviceability Index readings that are approaching the point where consideration should be given to the future scheduling of remedial pavement maintenance to restore the riding quality. The other sections show SI values for 1985 that range from 4.3 to 2.7, and the lowest value does not call for surface rehabilitation at this time.

Finally, it should be noted that check tests taken on the southbound passing lanes at the same time as the above tests have consistently shown a better ride in the passing lane. This confirms the effect of no shoulder on the cracking evident in the outermost wheel path. Ride quality in the outside lane is still acceptable as of January, 1985, but is generally lower in quality than that of the passing lane.

Table 8. Results of Mays Ride Meter tests expressed in SI's for MH 153 trial sections (5)**.

Section No.		2	3		4	5	6		7	8	
Station No.	48+00		52+50	57+00		61+50	66+00	70+50		75+00	Date
1. **Reading** at 0.05 miles	4.2	3.9	4.0	4.1	4.5	4.4	4.1	3.9	1.8	3.2	12/18/78
Reading at 0.20 miles				4.1				4.2			12/18/78
2. Reading at 0.05 miles	3.6	3.9	3.7	3.9	3.8	4.4	4.5	3.9	2.1	3.2	5/18/79
Reading at 0.20 miles				3.8				3.7			5/18/79
3. Reading at 0.05 miles	---	---	---	---	---	---	---	---	---	---	9/8/80*
Reading at 0.20 miles				---				---			9/8/80*
4. Reading at 0.05 miles	3.2	3.3	3.1	3.1	3.3	3.0	2.7	2.8	2.9	3.7	12/8/81
Reading at 0.20 miles				3.3				3.0			12/8/81
5. Reading at 0.05 miles		4.3		4.2	4.0	3.9	2.7		2.7	3.4	5/30/85***

*SI readings taken from this data are incorrect, due most likely to the Mays Meter being out of calibration, and are therefore omitted.

**Results are shown for tests in the travelled lane (outside) only.

***Results are shown for tests in the inside lane only (average of two runs).

Metric conversion: 1 mile = 1.609 km.

Dynaflect Deflection

For all sections, as shown in Table 9, the range of individual Dynaflect deflection measurements taken from July, 1978 to January, 1985 is from 0.85×10^{-3} to 1.95×10^{-3} inches (0.021 to 0.050 mm). Averages of Dynaflect deflections for all the sections range from a low of 1.00×10^{-3} inches (0.025 mm) for Section 2 with the 40/60 SEA binder and the job mix formula to a high for Section 6 of 1.41×10^{-3} inches (0.036 mm) with the 40/60 SEA binder and the 50:50 concrete sand:field sand aggregate blend. Based on average Dynaflect deflection, pavement sections listed in order of decreasing dynamic stiffness would be Sections 2, 8, 3, 5, 7, 4 and 6.

Another interesting feature about the Dynaflect deflections for each of the SEA sections is that maximum Dynaflect deflections measured occurred at the first measurement of less than one month after construction. This would appear to be another indication of the effect of delayed structuring of sulphur causing the binder system and hence Item 292 layer, to be a little less "stiff" at the beginning of the section life. Construction compaction of gap graded mixtures such as those used in all but Sections 2 and 8 made it difficult to get good field densities during construction. Even after two summers of traffic, these sand mixes had high air voids and little additional compaction has occurred since 1980.

VI. SUMMARY FOR ALL TESTING BY TRIAL SECTION

Section 2

As illustrated in Table 5, results of laboratory tests on field cores for this trial section pavement, composed of 40/60 SEA binder and the job mix formula aggregate blend, show no detrimental trends or declines in values for any of the test categories including Marshall stability, Marshall flow, Hveem stability, resilient modulus or indirect tension from the start of testing in July, 1978 to the most recently completed

Table 9. Maximum Dynaflect deflections for MH 153 (5).

Section Number Binder Type Aggregate Type	Maximum Dynaflect Deflection X10^{-3} in.	Date Test Performed
Section 2	1.28	7/14/78
40/60 SEA	0.91	12/18/78
Job mix formula	0.96	6/28/79
55:30:15 Bank	1.29	9/3/80
river gravel: pea	0.85	12/1/81
gravel: field sand	0.74	2/19/85
Section 3	1.41	7/14/78
40/60 SEA	1.02	12/18/78
75:25 Bank	1.04	6/28/79
river gravel: field	1.25	9/3/80
sand	1.11	12/1/81
	1.07	2/19/85
Section 4	1.65	7/14/78
30/70 SEA	1.21	12/18/78
75:25 Bank	1.25	6/28/79
river gravel: field	----	9/3/80
sand	1.29	12/1/81
	1.29	2/19/85
Section 5	1.44	7/14/78
30/70 Sulphur-	1.09	12/18/78
Asphalt*	1.14	6/28/79
75:25 Bank river	1.17	9/3/80
gravel field sand	1.23	12/1/81
	1.29	2/19/85
Section 6	1.95	7/14/78
40/60 SEA	1.22	12/18/78
50:50 Concrete	1.20	6/28/79
sand: field sand	1.27	9/3/80
	1.50	12/1/81
	1.32	2/19/85
Section 7	1.83	7/14/78
30/70 SEA	1.18	12/18/78
50:50 Concrete	1.22	6/28/79
sand: field sand	1.02	9/3/80
	1.26	12/1/81
	1.45	2/19/85

(continued)

Table 9. (continued)

Section Number Binder Type Aggregate Type	Maximum Dynaflect Deflection $X10^{-3}$	Date Test Performed
Section 8 (Control)	----	7/14/78
0/100 AC	1.00	12/18/78
Job mix formula	1.15	6/28/79
55:30:15 Bank	1.12	9/3/80
river gravel: pea	1.14	12/1/81
gravel: field sand	1.35	2/1/81

*Sulphur-asphalt binder was prepared by bypassing the colloid mill.

Metric conversion: 1 inch = 25.4 mm

tests in 1985. Except for the M_R and the indirect tension test, this section is equal or higher in serviceability than Section 8, the control. Thus, laboratory tests indicate satisfactory pavement performance for Section 2 after seven years of service and this is with a mix with about 20 percent less binder than optimum!

The non-laboratory tests of visual evaluations, Mays Ride Meter roughness testing and Dynaflect deflection testing indicate that this pavement has (1) declined at about the same rate as most of the other sections in Pavement Rating Scores; however, there is some cracking and some rutting in the outside wheel path of the traveled lane; (2) declined in Serviceability Index at about the same rate as Sections 3 and 4, and (3) maintained the lowest average value of pavement maximum deflection. Based on these tests, performance to the present has been satisfactory except for the slight rutting, some longitudinal cracking and edge cracking which are caused in large part by absence of support from either a paved or unimproved shoulder.

Future performance of Section 2 should be satisfactory if the extent and magnitude of cracking in the travelled lane do not increase significantly and allow increased moisture penetration and damage to the SEA pavement layer and the underlying six-inch (152 mm) limed subgrade.

<u>Section 3</u>

Performance to date of this pavement section with the 40/60 SEA binder has been satisfactory. The apparent possible detrimental trends observed from laboratory data are (1) that Marshall flow values are a little higher than other sections and (2) that Hveem stabilities appear erratic. Non-laboratory test results expressed in PRS values from visual evaluation, SI's from Mays Rider Meter testing and data from Dynaflect deflection testing show Section 3 to be holding its own in comparison with the other trial sections.

Section 4

Performance to date of this section containing the 30/70 SEA binder is termed satisfactory. Based on both laboratory and non-laboratory test results, the performance of this section could be grouped with Sections 3 and 5 which contain the same 75:25 bank run river gravel:field sand aggregate blend.

Slight detrimental trends are detectable from the fact that laboratory results indicate that both Marshall and Hveem stabilitites declined, as shown by results in 1981 but rose in 1985. This indicates that field core quality may be a primary variable in evaluations of this type. Section 4 has the lowest average Marshall stability of Sections 3, 4 and 5, also the second highest average maximum pavement Dynaflect deflection of all sections (only Section 6 is higher) with 1.41×10^{-3} inch (0.36 mm) of maximum deflection.

Performance of Section 4 continues to be satisfactory in spite of longitudinal cracking. Apparently this has not significantly lessened pavement section performance.

Section 5

The performance to date of Section 5 containing the 30/70 SEA binder produced from 75:25 bank run gravel:field sand aggregate blend is satisfactory and roughly equivalent to that of Sections 3 and 4. Possible detrimental trends are that both Hveem and Marshall stabilities declined significantly for a while as shown in Table 4 but went up in 1985. Again, core quality is most likely the reason for this.

Performance of Section 5 is satisfactory and it compares favorably with the control. Also, longitudinal cracking from the outside wheel path to the pavement edge in the travelled lane has not adversely affected performance through 1985.

Section 6

Laboratory tests, with the exception of Marshall stability, show Section 6 containing the 40/60 SEA binder and 50:50 concrete sand:field sand to be performing quite well. Marshall stability shows considerable variation over the test period 1978 through 1985. The average Marshall stability is lower than all other sections except Section 7; however, it is an all sand mix. Hveem stabilities, on the other hand, have remained stable at about 20.

Future performance of this section will depend on whether the structural condition of the pavement layer does not weaken as would be indicated by higher Dynaflect deflection values and lower SI values. Longitudinal cracking exists in the travelled lane of Section 6 as in the other sections and may affect future performance of this section.

Section 7

Pavement Section 7 performance to date could be termed adequate but this trial section containing the 30/70 SEA binder and the 50:50 concrete sand:field sand aggregate blend can be labelled the one section with the most apparent detrimental trends in test results. To begin, averages of Marshall stability, M_R and indirect tension results are lower for this section than any other. Discarding 1980 data which was erroneous, the average Marshall stability is only 180 pounds (800 newtons) which is comparatively low; and indirect tension at 115 psi (790 x 10^3 kPa) is considerably lower than any of the other sections and lower than desired for a structural layer. Finally, Marshall flow for Section 7 is essentially stable as shown in Table 5.

Dynaflect deflections for Section 7 are the third highest of all sections with an average of 1.33 x 10^{-3} inch (0.034 mm). The PRS from visual evaluation was 80 in 1981 but this turned out to be a questionable reading. Actually, the PRS for this section compares favorable with the control.

The performance of Section 7 continues to be satisfactory. Both the laboratory test results and non-laboratory testing indicate that some deterioration is present and may cause performance qualities that would warrant maintenance in 1987 or 1988. As of 1985, this section had both longitudinal cracking and the beginning of some alligator cracking, especially in the travelled lane from right wheel path to the pavement edge, which may seriously affect future performance. This latter distress is attributed to lack of lateral support at the shoulder.

Section 8

This is the control section at the south end of the SEA field trial sections which has the 100 percent asphalt cement binder and the job mix formula aggregate blend of 55:30:15 bank run gravel:pea gravel:field sand. Laboratory testing on this section since 1978 shows it to have the second lowest average values of M_R, and the highest indirect tension and lower values of Marshall stability and Hveem stability than its comparison 40/60 SEA section, Section 2. The above differences in test results between Sections 8 and 2 may show the effects of the sulphur adding to the internal friction in the stability tests and not contributing as much to the tensile strength, the M_R and indirect tension tests.

Non-laboratory testing including PRS scores from visual evaluations and Dynaflect deflection testing shows Section 8 to be performing roughly equal to Section 2. As of 1985 performance of Section 8 based on testing to date is fairly good; but Section 2 is in better shape and has better ride quality. By mistake Section 2 mix contains only 5 percent of SEA binder the same as the control and that is about 20 percent less than the optimum of 6.3 percent.

VII. EVALUATIONS OF STUDY OBJECTIVES

Objective One

Under Objective One it was desired to compare the performance of trial sections of MH 153 whose SEA binders was formulated and mixed in a weigh-batch plant pugmill with sections whose SEA binder was formulated in a colloid mill. In the first instance, Section 5 was the trial section in which the asphalt cement and the molten sulphur were commingled in a bypass line around the colloid mill and sent on for final mixing with the aggregates in the plant pugmill (4). All other SEA binder trial sections had binders prepared by going through mixing in the special colloid mill (4).

Sections 3 and 4 are closest in composition to Section 5 with Section 3 having the 40/60 SEA binder and Section 4 have the same 30/70 SEA binder composition as Section 5. All three sections have the same aggregate blend of 75:25 bank run river gravel:field sand and approximately the same volumes of binder as illustrated in Table 6.

Laboratory test results, on the average, are equal or slightly better for Section 5 than for either Section 3 or Section 4; and it is apparent that these three trial sections could be naturally grouped together on the basis of laboratory test results. The non-laboratory test of visual evaluation, Mays Ride Meter and Dynaflect deflection also show Sections 3, 4 and 5 to be roughly equal in performance.

Therefore, based on all testing from 1978 to 1985, it may be concluded that Section 5, whose SEA binder was formulated and mixed in a bypass line prior to entry into the pugmill, is performing at least equally as well as Sections 3 and 4 whose SEA binders were prepared in the colloid mill. Numerous field tests carried out since 1978 verify this finding.

Objective Two

Under this objective it was desired to determine the extent to which marginal aggregates or low quality aggregates may be used with SEA binders prepared in a colloid mill as added directly in the pugmill. Sections 3 through 7 contain these marginal aggregates combined with such SEA binders, and the performance of these sections from 1978 through late 1985 should help provide answers to the question of this objective.

The aggregates used in Sections 3 through 7 were all siliceous generally rounded materials including bank run river gravel, and washed concrete sand from sources close to the Brazos River and a field sand from high ground about four miles (6.4 km) east of the river. The "marginal" aspect of aggregates blending contained in Sections 3 through 7 is grading of these materials. The blend of these natural rounded siliceous aggregates creates a gapped grading.

Based upon performance to date, Sections 3 through 5 have performed satisfactorily but with lower average test results in the categories of Marshall and Hveem stability, M_R and indirect tension when compared to Sections 2 and 8. Finally, Sections 6 and 7 containing SEA binders and the concrete sand:field sand aggregate blend can be termed as performing adequately but as showing lower values of Marshall stability (and also M_R and indirect tension for Section 7) than Sections 2 and 8 show. It was anticipated that Section 7, and possibly Section 6, would show an increased rate of pavement strength decline and thus lowered performance; however, after about seven years of service the effect of using marginal aggregates is hardly detectable.

To summarize, all SEA sections have performed satisfactorily to date but at lower levels compared to Sections 2 and 8. This illustrates that marginal aggregates can be upgraded with SEA binders to provide highly acceptable pavements. Sections 2 and 8 have also performed adequately but trends in results of field data and testing to date indicate that, as of 1985, these sections and possibly the control (Section 8) may be the first ones to require maintenance.

Objective Three

Under Objective Three it is desired to find the effect on pavement performance from increasing the sulphur content in an SEA binder. Under this objective trial sections constructed with 30/70 SEA binders are to be compared to trial sections constructed from 40/60 SEA binders.

Laboratory test results for Sections 3, 4 and 5 show Section 5 to be slightly better than either Section 3 or Section 4 based on averages of test results. Section 3 is performing better than Section 4 in Marshall stability and M_R but worse in Marshall flow and indirect tension. Thus, a real trend of whether one SEA binder content is better than another is difficult to find in comparing these three sections.

The other comparison that should be made is between Sections 6 and 7. Section 6 has the 40/60 SEA binder and Section 7 has the 30/170 SEA binder. Both sections have the 50:50 aggregate blend of concrete sand:field sand. Results from laboratory testing show Section 6 to have significantly higher average values for Marshall stability, M_R and indirect tension than Section 7, with average values for Marshall flow and Hveem stability being roughly equal. Section 6 has a higher Pavement Rating Score, PRS, than Section 7 in 1985, but Section 7 has a lower average Dynaflect deflection. SSI values for both sections are about the same as of 1985 at 2.7 to 2.8 for Sections 6 and 7, respectively.

In summary, based on laboratory test results to date, Section 6 with a higher percentage of sulphur in its binder appears to be performing better than Section 7 which has less sulphur in the SEA binder. No definite trend can be established for Sections 3, 4 and 5 concerning the merits of 30/70 SEA binders versus 40/60 binders; although Section 3 generally rated higher in 1985 than did Sections 4 and 5.

Objective Four

Objective Four has as its purpose to compare the performance of an SEA pavement with a conventional pavement, with both paving sections having identical binder volumes and aggregate compositions. To satisfy Objective Four, the performance of Sections 2 and 8 must be compared.

Section 2 has the 40/60 SEA binder or 40 percent by weight of sulphur in the binder. Section 8, the control, has the pure asphalt cement binder. As shown in Table 6, the percent by weight of the 40/60 SEA binder found in Section 2 by extraction analysis results is approximately 5.0 or about 9.5 percent by volume of mixture; and the percent by weight of mixture of pure asphalt cement in Section 8 of 5 percent represents a volume percent of about 11.6 of the total mixture. Therefore, the volume of binder used for Section 8 is about 22 percent greater than used for Section 2. Both Sections 2 and 8 have the same 55:30:15 job mix formula aggregate blend of bank run gravel:pea gravel:field sand. The original plan was to have the same volume of binder in each of these mixtures.

Based on laboratory tests, SEA Section 2 has performed better than Section 8 for average Marshall and Hveem stabilities, and Section 8 has performed better for average M_R and indirect tension values. Both sections have similar average maximum Dynaflect deflections and PRS's, Pavement Rating Scores.

Therefore, it can be concluded based on testing and field observations to date that Section 2 is performing slightly better than Section 8. Both sections are equally affected with longitudinal cracking in the travelled lane, and future performance of each section will be dependent, in part, on how rapidly this cracking increases in the future. There is currently (1985), two areas of structural distress in Section 8, one which has been patched. No patching has been required for Section 2 or any of the other sections.

VIII. SUMMARY

Overall performance to date of the SEA binder field trial sections on MH 153 may be described as ranging from satisfactory to very good. It should be noted that these pavement sections have been subjected to increased stresses resulting from increasing numbers and magnitudes of truck loadings to the point that as of February, 1985 an estimated 15,000

18-kip (5,720 kg) equivalent single axle loads have been experienced as of 1985 as opposed to 9,800 such loadings orginally anticipated by the end of 1985.

The SEA binder sections have generally stood up well under the noted increased truck loading, although some distresses of slight rutting of less than one quarter inch and longitudinal cracking have occurred. Truck traffic volumes and magnitudes of loadings will probably continue experiencing rapid growth in the foreseeable future.

The current truck traffic indicates an average annual growth rate in equivalent 18 kip axle loads much higher than the anticipated 5 percent rate of increase that was estimated in 1978.

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