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16. Abstract

In June, 1978 FCIP Study No. 1-10-78-536 was initiated in which six 450-foot (137-m) trial sections of six-inch (152-mm) depth sulphur extended asphalt (SEA)binder stabilized base were constructed on the two southbound lanes of MH (Municipal Highway) 153 in the cities of Bryan and College Station, Texas. The control section was built with a Texas State Department of Highways and Public Transportation Item 292 conventional asphalt cement stabilized base, which was also used on the bulk of the project. Two different SEA binders were utilized, a 30/70 and a 40/60 binder. Three different aggregate gradations were employed, using blends of rounded siliceous materials consisting of bank run river gravel, pea gravel, concrete sand and field sand. Since these blends of materials produced gap gradations, the aggregates used were considered to be "marginal". This report describes the significant points concerning the design, construction and all the subsequent laboratory testing and field evaluations of the MH 153 SEA trial and control sections that have occurred from June, 1978 to December, 1981. The final part of the report discusses the four objectives of the study concerning the uses of SEA binders and marginal aggregates and how well these objectives were met based on the laboratory and field testing results.

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SULPHUR EXTENDED ASPHALT FIELD TRIALS
MH 153, BRAZOS COUNTY, TEXAS

Final Report

(Report No. 536-7)

TTI Project 2536

FCIP Study No. 1-10-78-536

by

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Prepared for

The Texas State Department of Highways and

Public Transportation

and

The Sulphur Institute

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SULPHUR EXTENDED ASPHALT FIELD TRIALS

MH 153, BRAZOS COUNTY, TEXAS

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 environmental 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 in pavements to extend or replace asphalt cement appeared in the 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. Another system which came into being around 1973 was sulphlex which is elemental sulphur modified with plasticizer(s) and which is used to completely replace asphalt cement. A third system was sand-asphalt-sulphur or S-A-S in which sulphur is added to a hot sand-asphalt cement mixture in an effort to utilize poorly graded sands.

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 have since been constructed in Arizona, Delaware, Florida, Idaho, Illinois, Kansas, Louisiana, Maine, Massachusetts, Michigan, Minnesota, Mississippi, Montana, Nevada, New York, North Carolina, Ohio, Pennsylvania, Texas, Washington and Wisconsin (2). Additionally, Canadian SEA projects have been constructed in Alberta, Ontario and Saskatchewan; other SEA projects have been completed in several European countries and in Saudi Arabia (2).

Sulphlex paving projects or test sections have been completed in Arizona, Florida, Michigan, Nebraska, North Dakota, Pennsylvania and Texas. S-A-S-type projects have been utilized in Canada for approximately 20 years, and one S-A-S project was completed on U. S. Highway 77 in Kenedy County, Texas in 1977 (3).

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 mixture designs for study." See Appendices A and B of (4). These objectives have and are being achieved by continuing the evaluation of the MH 153 trial 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,
4. Post construction monitoring and evaluation.

Preconstruction Project Planning

During this phase of the 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, 1978. The approximate location of these sections was on Wellborn Road from just south of F&B and Old College Roads in Bryan to just south of Maple Street and north of State Highway 60 in College Station. Wellborn Road can be considered the extension (to the north) of SDHPT Farm to Market Road 2154 which terminates at the junction with State Highway 60 on the northwest corner of the main campus of Texas A&M University.

The trials consisted of 2700 feet (823 m) of the southbound part of the roadway which was 26 feet (8 m) wide. Six trial sections, starting with 2 at F&B Road to 7 north of Maple Street, each 450 feet

(137 m) in length, were constructed using two different SEA binders and three different aggregate gradation blends. The control sections 1 and 8 were considered to be on the north and south ends respectively of the field trials. These sections consisted of the SDHPT project job mix formula aggregate gradation using 100 percent 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 five progress reports that have been prepared and published. The last of these reports was Progress Report No. 5 dated February, 1982 (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 Considerations

The SEA binder hot-mixed asphaltic materials used on MH 153 field trials comprised a six-inch (152-mm) base course layer of conventional asphalt stabilized base governed by 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. Traffic loadings anticipated on the four-lane undivided MH 153 facility upon completion were 8100 vehicles per day with 6.1 percent trucks. The pavement was designed structurally for a 20-year life during which it would be subjected to an estimated 50,000 18-kip (8172-kg) equivalent single axle loads.

An estimate has been made of the actual accumulated traffic loading on MH 153 as of January 1, 1982 which shows that this roadway has been subjected to approximately 8000 18-kip (8172-kg) equivalent single axle loads instead of the approximately 5600 loads originally forecasted. This is a 42 percent increase in traffic loading. The reasons for this increase are threefold: (1) overall traffic has actually been increasing at 8.6 percent per year instead of 5 percent; (2) trucks as an average percentage of total traffic have increased from 6.1 to 6.7 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.

The designed and constructed pavement section for 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 mm) northbound lanes of MH 153 have a concrete curb and gutter for pavement section edge support and are seperated from the southbound 11-foot (3.4-m) passing and 13-foot (4.0-m) traveled lanes by a 4-foot (1.2-m) double striped median area. The southbound lanes have no curb, paved or even improved shoulder for pavement section edge support, and the SEA field trials are located on these lanes from project station 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 what could generally be called a "fill" section, and therefore the Item 292 and Item 340 hot-mix asphaltic concrete material layers are generally "daylighted" at the top of an approximate 3:1 to 4:1 fill slope. This exposure and lack of side support is believed to be contributing significantly to longitudinal cracking in 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.

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 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 in the TTI designs is shown in Table 1 (4).

2. As shown in Table 1, the two SEA binders used were a 30/70 and a 40/60 SEA binder which indicate 30 percent and 40 percent sulphur content 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. (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.)

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

Aggregate Combinations			Binder Composition		Binder Content	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

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. Binder contents recommended for the MH 153 field trials were determined for each of the SEA binder compositions and aggregate blend combinations based upon overall evaluation of the results of 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 chosen 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. 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 SEA binder contents reflect the design considerations from 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 asphalt 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. Design binder content chosen for each trial section (4)

MH 153 Trial Section	Design Aggregate Blend	Design SEA Binder Ratio	Design Binder Content of Mixture: Pct. by Wt. of Mix	Design Binder Content of Mixture: Pct. by Vol. of mix***
2	55:30:15 Bank run gravel:pea gravel: field sand	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 Bank run gravel:pea gravel: field sand	*0/100	5.0*	11.6

* Pure 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 below 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. 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 problems.

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.

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 aggregate blend ranged from 50:50 to 90:10. Normally, the 90:10 blend would be unacceptable.

Concerning compaction, the SEA binder materials in Sections 2 through 7 were compacted at the highest possible temperature to

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	40/60	2	5.0
	to		2	5.0
	52+50		2	---
3	52+50	40/60	2	7.5
	to		2	7.5
	57+00		2	6.8
4	57+00	30/70	2	---
	to		2	7.0
	61+50		2	6.3
5	61+50	30/70	2	7.0, 7.0
	to		2	7.0
	66+00		2	6.3
6	66+00	40/60	2	---
	to		2	8.5
	70+50		2	7.9
7	70+50	30/70	2	---
	to		2	8.0
	75+00		2	7.4
8 Control	75+00	0/100	2	5*
	to		2	5*
	End		2	5*

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

Metric Conversion: 1 inch = 25.4 mm.

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, one with vibration and four additional passes 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 (22,675-kg) pneumatic tired roller.

The reason for changing to the second method of compaction was that undue lateral displacement occured 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 (22,675-kg) pneumatic roller produced effective 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 temperatures 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 removed most of the surface roller marks created by the REX 900 (4).

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

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 kilograms 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 D2726-73 (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 field compaction efforts, indicate changes in aggregate gradations and 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 stability method 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 cored samples.

Marshall Flow

This test is a 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 loadings. 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 traffic loadings. High Hveem stabilities from 35 to 45 (percent)may be required on roadways serving significant traffic loadings from 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 cored samples.

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 .05 or 0.10 second from the beginning of the deformation, and this deformation is used in Schmidt's equation to compute the resilient modulus (8).

The resilient modulus test may be used both to design bituminous mixtures and 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 the 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 Gonzalez, et al. (9).

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 of cylinders of laboratory compacted bituminous mixtures and 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 paving 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 compacted core specimens 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 of 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 paving layers. In this test, the bitumen in a paving material is put into solution in a solvent such as tricholoroethylene; 1, 1, 1-tricholorethane or benezene and is thus "extracted" from the aggregate system.

The extracted binder content test is usually used for quality control testing for verifying bitumen contents during production of bituminous mixtures at weigh-batch, drum-dryer and other plants. This test is also used to determine bitumen contents of compacted mixtures in existing layers or crumbled pavement core samples. The "Standard Test Methods for QUANTITATIVE EXTRACTION OF BITUMEN FROM BITUMINOUS PAVING MIXTURES" is found in ASTM D2172-38 (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 the 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, a pavement stiffness coefficient and a subgrade stiffness coefficient are obtained.

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.

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 1981, both on Sections 2 through 7 of the SEA trials and on the control section, Section 8. These four tables are extracted from Progress Report No. 5 (5). 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 to be used in the MH 153 study.

Laboratory Tests

Density

As shown in Table 4, density tests for Sections 2 through 8 from 1978 through 1981 indicate that the densities of cores obtained from each section have not changed significantly over a period of three 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 three 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 three 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

Table 4. 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 gravel: pea gravel: field sand	145	430	15	21	0.97	220	2.44		11/18/81(41)
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)
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)
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		11/19/81(41)

* Weight percentages of sulphur-asphalt binder.
** Sulphur-asphalt binder prepared by by-passing the colloid mill.
*** Weight percent based on total mixture.
**** Age of pavement in months.

Table 4. (Continued.) 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*	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)
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)
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:									
field sand									

(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 = 25.4mm
1 psi = 6.894 kPa

Table 5. 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 6. Pavement rating scores, PRS, for MH 153** (5).

Section Binder Type Aggregate Type	Pavement Rating Score, PRS	Date Observed
Section 2	100	12/18/78
40/60 SEA	100	6/29/79
Job mix formula	83	12/12/81
55:30:15 Bank river	83	12/01/81
gravel: pea gravel: field		
sand		
Section 3	100	12/18/78
40/60 SEA	98	6/29/79
75:25 Bank river	88	12/12/80
gravel: field sand	85	12/01/81
Section 4	100	12/18/78
30/70 SEA	97	6/29/79
75:25 Bank river	93	12/12/80
gravel: field sand	85	12/01/81
Section 5	100	12/18/78
30/70 SEA*	98	6/29/79
75:25 Bank river	93	12/12/80
gravel: field sand	85	12/01/81
Section 6	100	12/18/78
40/60 SEA	100	6/29/79
50:50 Concrete sand:	93	12/12/80
field sand	88	12/01/81
Section 7	100	12/18/78
30/70 SEA	100	6/29/79
50:50 Concrete	88	12/12/80
sand: field sand	80	12/01/81
Section 8	100	12/18/78
0/100 AC	100	6/29/79
Job mix formula	93	12/12/80
55:30:15 Bank	85	12/01/81
river gravel: pea		
gravel: field sand		

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

**Reported for the travelled lane only.

Table 7. Results of Mays Ride Meter tests expressed in SI's for MH 153 trial sections (5)**

Section No.		2	3		4	5	6		7		
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	[illegible]	12/8/81
Reading at 0.20 miles				3.3				3.0			12/8/81

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

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

Metric Conversion: 1 mile = 1.609 km

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 SEA binder paving materials apparently have not changed appreciably since placement.

Marshall Stability

As shown in Table 4, Marshall stability values for each section from 1978 to 1981 show considerable variability from test date to test date. Section 2 with the 40/60 SEA binder and the job mix formula has the highest average stability of 470 pounds (2,090 newtons) with the control Section 8 second at 410 pounds (1,820 newtons). Sections 3, 4 and 5 are grouped closely together with their average stabilities ranging 280 to 320 pounds (1,250 to 1,420 newtons). Sections 6 and 7 are the lowest with average Marshall stabilities of 250 and 130 pounds (1,110 and 580 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 the exception of one or two values obtained for Sections 2, 3, 6 and 8 from 1978 to 1981) have remained below the "Light" traffic category requirement of 500 pounds (2,220 newtons) called for in the Marshall method of design (13).

Marshall Flow

For all sections and the control, Table 4 shows that flow values have been variable but have not changed significantly for the three years of testing. For all sections, the average value of flow ranges from a low of 12 for Section 7 to a high of 16 for Section 3. These average values would meet the allowable maximum and minimum flow values for all of the traffic categories of the Marshall method of design (13).

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 and 22 in 1980 and 1981 respectively and thus show a questionable trend. High values of flow were also measured for Sections 5 and 6 in 1980.

Hveem Stability

Table 4 indicates that Hveem stability values have been variable for all sections except Section 6 with the 40/60 SEA binder and 50:50 concrete sand:field sand aggregate blend. Section 2 and Section 8, the control with the job mix formula aggregate blend, can be grouped together with the highest average Hveem stabilities. Average stabilities for Sections 3 through 7 can all be grouped closely together and range from 17 to 19.

Trends in Hveem stability are difficult to determine, although stability values for Sections 2 and 3 appear to be declining. Stability values determined from Section 6 field cores show a constant 20 for the last four testing periods. Based upon the most recent stability tests obtained in 1981, Sections 3, 4, 5 and 7 show Hveem stability values that are lower than expected: however, cores taken at this sampling period were of poor quality.

Resilient Modulus

Resilient modulus, M_R, values for all sections are quite variable over the three 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. 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 466,000 to 520,000 psi (3.21×10^6 to 3.58×10^6 kPa).

Values of M_R for all the 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 tensile strength at the early age of 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 (1,070 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 125 to 130 psi (860 to 900 kPa). The lowest average indirect tension of 105 psi (720 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 and may indicate forthcoming distress problems for this section.

Rice Maximum Specific Gravity

Values for Rice maximum specific gravity were obtained only twice, in July 1978 and November 1981. The results of these tests, based on minimal data, indicate no significant changes in specific gravity with time.

Extracted Binder Content

Extracted binder contents were taken on only 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 5 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 5 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 binder contents. Only in one instance, for Section 7, was the measured binder content of 9.2 percent much higher than that called for by the design.

Non-Laboratory Tests

Visual Evaluation

The results of four visual evaluation tests since 1978 have yielded Pavement Rating Scores, PRS, for each of Sections 2 through 8 which are shown in Table 6. For every section the PRS has declined from 100 to a value in the 80's. As of December 1981, Section 6 had the highest PRS of 88 and Section 7 had the lowest of 80. These rating scores have been obtained from visual evaluations taken in the outside or travel lane of the southbound lanes of MH 153. Although not reported specifically in this publication, PRS values for the inside or southbound passing lanes of MH 153 are generally higher for each of the sections since the passing lane carries considerably less truck traffic than the travelled lane and is performing better.

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 rutting in the wheel paths for two sections. The longitudinal cracking distresses are particularly evident from the right wheel path 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 7. 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 1981. 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 Serviceability Index readings have occurred for the surface of Sections 5 and 6 from May, 1979 to December, 1981 indicating increased surface roughness for these sections which in turn indicates possible pavement layer distresses such as stability related problems. SI values for Section 7 show anomalous behavior, with values going up from 1979 to 1981. Inadequate machine calibration may explain why this phenomenon is occurring.

Sections 5 and 6 show 1981 Serviceability Index readings that are approaching the magnitude of 2.5 where consideration should be given to the scheduling of remedial pavement maintenance to restore the riding quality. The other sections show 1981 SI values that range from 3.0 to 3.7, even the lowest of which are yet some time away from calling for surface rehabilitation.

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 distress evident in the outermost wheel path of the travelled lane which is affecting riding quality.

Dynaflect Deflection

For all sections, as shown in Table 8, the range of individual dynaflect deflection measurements taken from July, 1978 to December,

Table 8. 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		
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
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
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
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
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

Table 8. (Continued.) Maximum dynaflect deflections for MH 153 (5).

Section Number Binder Type Aggregate Type	Maximum Dynaflect Deflection X10^{-3} in.	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 gravel: field sand	1.14	12/1/81

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

Metric Conversion: 1 inch = 25.4 mm

1981 is from 0.85×10^{-3} to 1.95×10^{-3} inches (0.021 to 0.050 mm). Averages of dynaflect deflections for all of the sections range from a low of 1.06×10^{-3} inches (0.027 mm) for Section 2 with the 40/60 SEA binder and the job mix formula to a high of 1.43×10^{-3} inches (0.036 mm) for Section 6 with the 40/60 SEA binder and the 50:50 concrete sand:field sand aggregate blend. Based on average dynaflect deflections, 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 trial sections is that maximum dynaflect deflections measured occurred at the first measurement or less than one month after construction. This would appear to be another indication of the effect of the delayed structuring of sulphur causing the binder system, and hence the Item 292 layer, to be a little less "stiff" at the beginning of the section life.

VI. SUMMARY FOR ALL TESTING BY TRIAL SECTION

Section 2

As illustrated in Table 4, 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 tests in 1981. Except for the M_R and indirect tension tests, this section is equal or higher in serviceability than Section 8, the control. Thus, laboratory tests indicate satisfactory pavement performance to date for Section 2.

The non-laboratory tests of visual evaluations, Mays Ride Meter roughness testing and dynaflect deflection testing indicate that this pavement has (1) declined a little faster than most of the other sections in Pavement Rating Scores due to a combination of cracking and some rutting in the outside wheel path of the travelled 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 rutting, longitudinal cracking and edge cracking which are believed caused in 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 layer.

Section 3

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 appear to be increasing and (2) that Hveem stabilities may be in a sharp decline. Non-laboratory test results expressed in PRS values from visual evaluation, SI's from Mays Ride Meter testing and data from dynaflect deflection testing show Section 3 to be holding its own in comparison with the other trial sections.

From the results of all testing, Section 3 could be grouped together with Sections 4 and 5 containing the same 75:25 aggregate blend of bank run river gravel:field sand. Future performance of Section 3 will probably continue to be satisfactory if longitudinal cracking in the travelled lane, especially from the outside wheel path out to the pavement edge, does not cause premature deterioration of this pavement layer from moisture penetration.

Section 4

Performance to date of this section containing 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.

Possible detrimental trends are detectable from the fact that laboratory results indicate that both Marshall and Hveem stabilities may be declining rapidly, as shown by results for 1981. Section 4 has the lowest average Marshall stability of Sections 3, 4 and 5 and also the second highest average maximum pavement dynaflect deflection of all sections (only Section 6 is higher) with 1.35×10^{-3} inch (0.034 mm) of maximum deflection.

Future performance of Section 4 will probably continue to be satisfactory if longitudinal cracking, from the right wheel path of the travelled lane to the pavement edge, does not significantly lessen pavement section performance by allowing increased moisture penetration.

Section 5

The performance to date of Section 5 containing the 30/70 SEA binder produced from 75:25 bank run river gravel:field sand aggregate blend can be called satisfactory and roughly equivalent to that of Sections 3 and 4. Possible detrimental trends are that both Hveem and Marshall stabilities may be declining significantly as shown in Table 4.

Future performance of Section 5 is expected to be satisfactory if 1981 declines in stabilities do not continue and if dynaflect deflections remain at the same level as the 1981 values. Also, longitudinal cracking from the outside wheel path to the pavement edge in the travelled lane may adversely affect future performance.

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 adequately. Marshall stability shows a considerable decline from the 1979 to the 1981 value. If the high 1980 Marshall test is discarded, the average Marshall stability is lower than all other sections except Section 7. Hveem stabilities, on the other hand, have remained stable at 20.

Section 6 also shows the highest average dynaflect deflection for all sections of 1.43×10^{-3} inch (0.036 mm) and low SI values for 1981 compared with all other sections. These two tests indicate a possible weakening of the pavement structural layer and the potential for an increase in surface distress manifested in this section.

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. Also, the same longitudinal cracking exists in the travelled lane of Section 6 as is in the other sections and may effect 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 labeled 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 130 pounds (580 newtons) which is very low; and indirect tension at 105 psi (720 x 10^3 kPa) is considerably lower than any of the other sections and lower than desired for a structural layer. Finally, Marshall stability for Section 7 is also showing considerable decline from 1979 as shown in Table 4.

Dynaflect deflections for Section 7 are the third highest of all sections with an average of 1.30 x 10^{-3} inch (0.033 mm). The PRS from visual evaluation is the lowest for all sections with a value of 80 in 1981. Finally, SI is low with a value of 2.9 as measured in 1981.

The performance of Section 7 could be at the edge of a decline. Both laboratory test results and non-laboratory testing indicate that serious deterioration could be present that may cause performance to become unsatisfactory in the near future. As of 1981, this section had both longitudinal cracking and the beginning of noticeable alligator cracking, especially in the travelled lane from right wheel path to the pavement edge, which may seriously affect future performance.

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 higher average values of M_R and indirect tension and lower values of Marshall stability and Hveem stability than its comparison 40/60 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 strengths in 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. The future performance of Section 8 based on testing to date should be good, but this is also dependent on the possible future worsening of existing longitudinal cracking in the travelled lane.

VII. EVALUATIONS OF STUDY OBJECTIVES

Objective One

Objective one is to compare the performance of the trial sections of MH 153 whose SEA binders were 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 comingled 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 having 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 Tables 2 and 3.

Laboratory test results, on the average, are equal or slightly better for Section 5 than 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 tests 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 1981, it may be concluded at this time 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.

Objective Two

The purpose of objective two is to determine the extent to which marginal aggregates or low quality aggregates can be used with SEA binders prepared in a colloid mill. Sections 3 through 7 contain these marginal aggregates combined with such SEA binders, and the performance of these sections from 1978 through late 1981 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, a 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 the aggregate blends contained in Sections 3 through 7 is the grading of these materials because blending of these natural rounded siliceous aggregates for the MH 153 project created gaped gradings.

Based upon indications 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 showing much lower values of Marshall stability (and also M_R and indirect tension for Section 7) than Sections 2 and 8 show. It is anticipated that Section 7, and possibly

Section 6, could show an increased rate of pavement strength decline and thus lowered performance in the near future.

To summarize, SEA Sections 3 through 5 have performed satisfactorily to date but at lower levels compared to Section 2 and Section 8, the control section, and thus illustrate that marginal aggregates can be upgraded with SEA binders to provide satisfactorily performing pavements. Sections 6 and 7 have performed adequately but trends in results of testing to date raise questions about the continued performance of these two sections.

Objective Three

Objective three is 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.

The first comparison should be made between Section 3 containing the 40/60 SEA binder and Sections 4 and 5 each containing 30/70 binder. All three sections have the same 75:25 bank run river gravel: field sand aggregate blend.

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 Section 6 and 7. Section 6 has the 40/60 SEA binder and Section 7 has the 30/70 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 1981, but Section 7 has a lower average dynaflect deflection. SI values for both sections are about the same as of 1981 at 2.8 to 2.9.

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 with the lower sulphur percentage 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 SEA binders.

Thus, with respect to comparing Sections 6 and 7, objective three has been achieved in that it has been shown that Section 6 with a higher percentage of sulphur in its binder is exhibiting better performance than Section 7. Since higher sulphur percentage in the case of Section 3 versus Sections 4 and 5 has apparently not produced significant increases or differences in performance, achievement of objective three in this instance is inconclusive. Based on the above results, it appears that the aggregate gradations composed of the sands are achieving higher performance as the sulphur is increased as a percentage of the SEA binder.

Objective Four

The purpose of objective four is to compare the performance of an SEA pavement with a conventional pavement, with both paving section materials 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 binder of sulphur. Section 8, the control, has the pure asphalt cement binder. As shown in Tables 2 and 3, 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 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 for Section 2. Both Sections 2 and 8 have the same 55:30:15 job mix formula aggregate blend of bank run river gravel:pea gravel:field sand.

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 about equal average maximum dynaflect deflections and PRS's, Pavement Rating Scores.

Therefore, it can be concluded based on testing to date that Sections 2 and 8 are performing roughly equally. Both sections are about equally affected with the 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.

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 the end of 1981 an estimated 8,000 18-kip (5,720-kg) equivalent single axle loads had been experienced, as opposed to 5,600 such loadings originally anticipated by the end of 1981. Thus, these sections have already been subjected to traffic loads originally estimated for March, 1983.

The current truck traffic indicates an annual average growth rate in equivalent 18-kip (5,720-kg) axle loads of about 14 percent. The originally estimated growth rate was 5 percent annually.

The SEA binder sections have generally stood up well under the noted increased truck loading, although some distresses of rutting and longitudinal cracking have occurred. Truck traffic volumes and magnitudes of loadings will probably continue to increase as the Bryan-College Station area is expected to continue experiencing rapid growth in the foreseeable future.

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