BENEFICIAL USE OF SULFUR

IN

SULFUR-ASPHALT PAVEMENTS

Final Report

Texas A&M Research Project RF 3644

Post Construction Evaluation

of

Sand-Asphalt-Sulfur

Test Section, Kenedy County, Texas

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Final Report
FCIP Study No. 1-9-76-519
DOT-FH-11-8608

Prepared for

U. S. Department of the Interior - Bureau of Mines

and

The Sulphur Institute

by

Texas A&M Research Foundation

College Station, Texas 77843

November 1980

ABSTRACT

The use of sulfur as a means of upgrading poorly graded mineral aggregates for use in asphaltic concrete mixes has been under study by Shell Canada under the trade name of Thermopave[R] for approximately fifteen years. Laboratory work has been extensive and numerous field trials have been completed in Canada. The Texas Transportation Institute under the co-sponsorship of The Sulphur Institute, and The Bureau of Mines instituted a program to introduce this concept to the United States. Following a 4-year laboratory effort a 3,000 lineal foot (914 m), sand-asphalt-sulfur experimental test section was placed along a portion of U.S. 77 in Kenedy County, Texas. The 3,000-foot section was divided into six subsections of various thicknesses with two sections purposely underdesigned to show distress in two to three years. This was the first demonstration of the Shell concept on a Federal Highway in this country.

Following this demonstration a 36-month post-construction evaluation program was undertaken involving a series of laboratory tests of cored samples, Dynaflect deflections, Mays ridability and on-site visual inspections. At the end of this 3-year period the sulfur subsections are performing as well as the control sections. In the case of rutting, the sulfur sections are actually performing better than the control.

Recommendations have been made to continue this post-construction evaluation for an additional 36 months.

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INTRODUCTION

1.0 Introduction

Sulfur is unique among our nation's mineral resources in that it is one of the few materials which will probably be in abundant supply in the near future. For this reason, various industry, government and university groups have initiated efforts to develop new uses for sulfur.

One of the most promising outlets for sulfur is highway construction in which interest is currently being stimulated by two factors: (a) the decreasing availability or total absence of quality aggregates in a number of regions around the country, and (b) the current increase in cost and projected demands for asphalt. Sulfur's unique properties permit it to be utilized either as a structuring agent (i.e. playing the role of the aggregate) or as an integral part of the binder or both (1).

1.1 Background

The project described in this report addresses itself specifically to the use of sulfur in sand-asphalt-sulfur (SAS) paving mixtures. This concept was developed and patented by Shell Canada, Ltd. under the name Thermopave and involves the use of sulfur as a structuring agent with poorly graded sands as found in many areas of the United States and specifically along the beaches and inland regions of the Gulf Coast States. Through efforts initiated by The Sulfur Institute and co-sponsored by the U.S. Bureau of Mines, the Texas Transportation Institute has, during the 4 years prior to the Kenedy County test section, done considerable laboratory verification studies of the sand-asphalt-sulfur (SAS) technology developed in Canada. One of the prime objectives of this effort was to introduce to the United States and adapt to her conditions the utilization of sulfur in asphaltic concrete mixes for base courses.

This program culminated during April, 1977 with the successful placement of a 3,000 lineal foot (915 m) SAS test section on U.S. 77 in Kenedy County, Texas. Construction details including materials, mix designs, equipment, materials handling, quality control and evolved gas analyses are described fully in the Construction Report prepared by the Texas Transportation Institute and is available upon request (1).

1.2 Objectives

The objective of this report is to present the results of a 3-year post construction evaluation of the in-service performance of a sand-asphalt-sulfur pavement placed in Kenedy County, Texas. This sand-asphalt-sulfur pavement is compared to with a conventional hot-mix asphalt concrete pavement using both laboratory and in-situ test results.

1.3 Scope

This report encompasses a brief description of the construction of U.S. 77 including materials and suppliers, construction equipment, the methods used to meet specifications, and quality control. For a more detailed presentation see the Construction Report issued April, 1977 by the Texas Transportation Institute (TTI) (1). The laboratory portion of the testing is briefly described for the following tests performed:

1) bulk density and bulk specific gravity,
2) resilient modulus,

3) Marshall stability and flow,

4) Hveem stability, and

5) indirect (splitting) tensile strength.

The condition and performance testing of the pavement sections are also discussed and include:

1) Mays Ride Meter (and corresponding serviceability index),

2) Dynaflect deflection, and

3) visual survey (including a cracking survey).

A discussion of the test results is presented as a comparison of the six subsections placed. This comparison of the subsections should give an indication as to which materials exhibit relatively superior qualities. Further comparison is made regarding the thicknesses of the subsections with each other.

2.0 Construction of U.S. 77

The location of the SAS pavement section on U.S. 77 is 5 miles (8 km) south of Sarita and 46 miles (74 km) north of Raymondville in Kenedy County, Texas. This area is under the jurisdiction of District 21 of the Texas State Department of Highways and Public Transportation. The experimental section as shown in Figure 1 is two traffic lanes wide (26 ft = 8 m) and contains six test items, each 500 ft (153 m) in length. From south to north there are three subsections of SAS base in thicknesses of 10, 7, and 4 in (25.4, 17.8, 10.2 cm) respectively. These are followed by three sections of asphalt concrete base in thicknesses of 4, 7, and 10 in (10.2, 17.8, 25.4 cm) respectively. The arrangement of the subsections is shown in Figure 1. All the base courses were surfaced with a 1 in (2.5 cm) wearing course of conven-

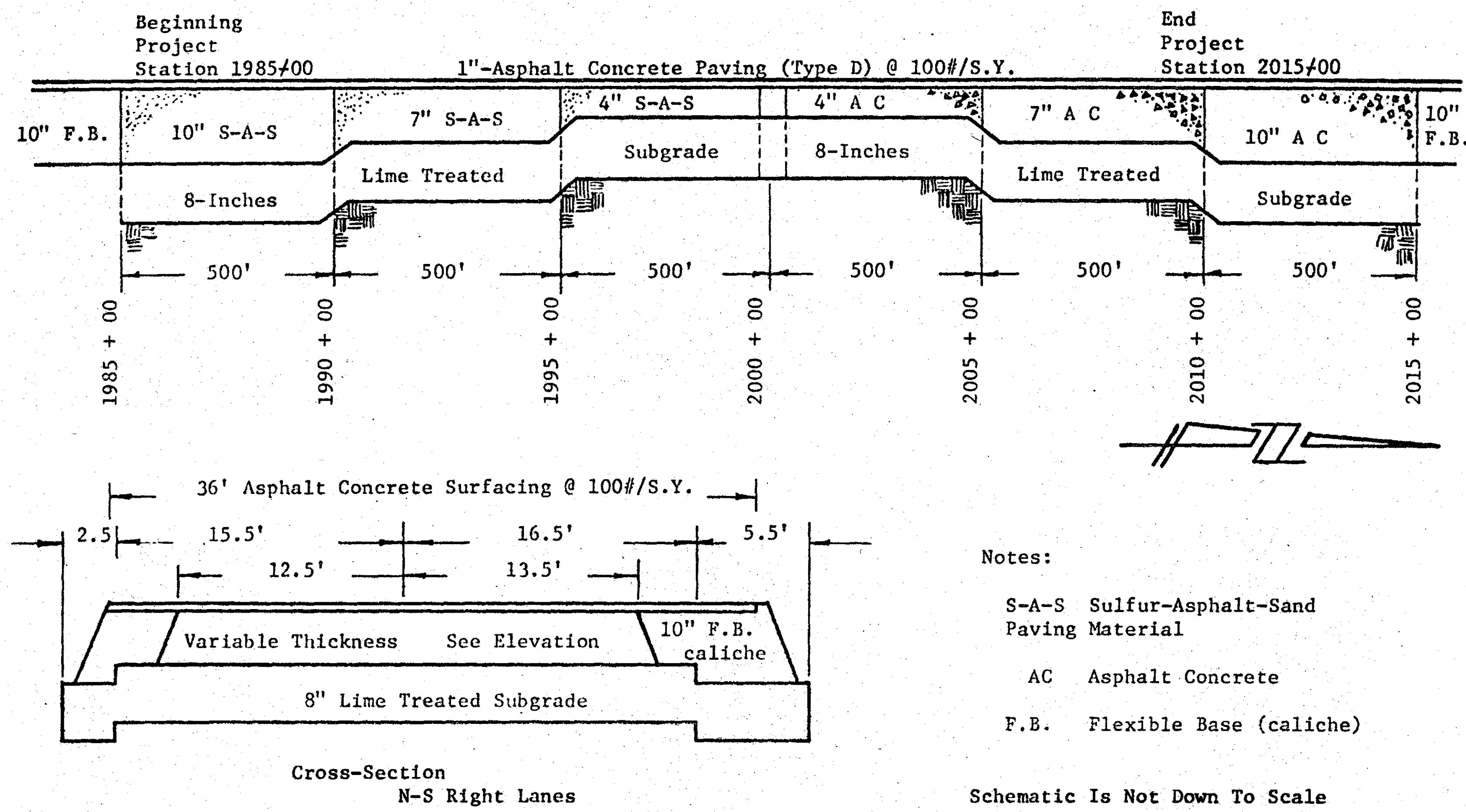


Figure 1. Layout of Sand-Asphalt-Sulfur Pavement Field Trial

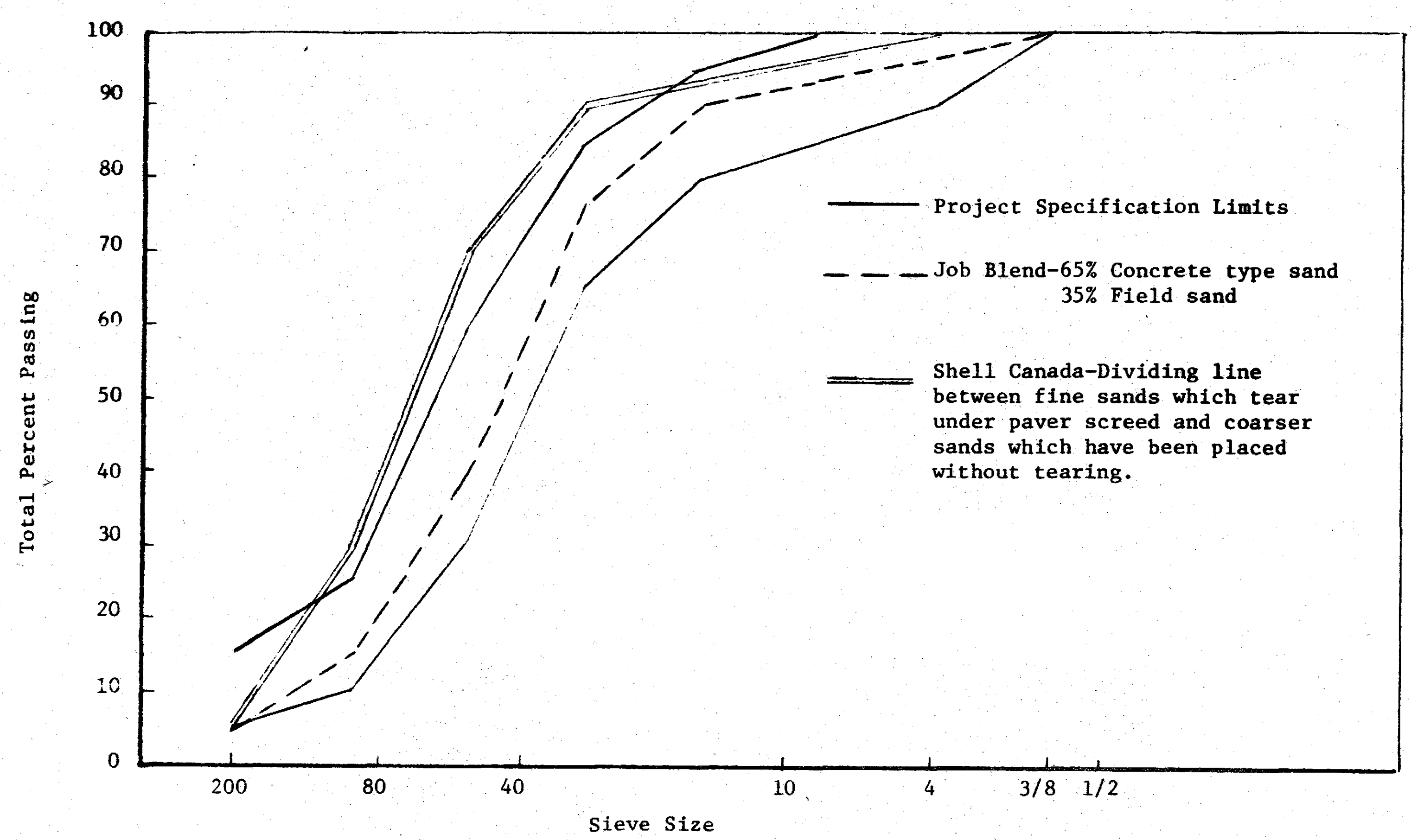
tional Type D hot-mix. This field trial was designed by TTI to compare the relative performance of an SAS pavement and a deep asphalt concrete pavement (1).

2.1 Materials

The asphalt was supplied from Gulf States Asphalt Company, Houston, Texas. The asphalt was a paving grade complying with the Texas State Department of Highways and Public Transportation (SDHPT) for Viscosity Grade AC-20. The same AC-20 was used in both the SAS and conventional asphalt concrete subsections of the pavement.

The sulfur was supplied from two sources: Warren Petroleum, a division of Gulf Oil, and Texasgulf, Inc. Delivery from both sources was made by Oil Transport Co., Abilene, Texas, and Robertson Tank Lines, Houston, Texas. Sulfur transports were tractor-trailer units of about 3,400 gallon (12.9 kl) capacities. Each unit was equipped with heating coils and steam jacketed discharge valves.

The aggregate requirements for the project were based on recommendations from Shell Canada Limited, Oakville Research Centre. The project specifications were prepared to describe sands which Shell Canada had successfully placed without appreciable imperfections in the mat. In their experiences, fine sands of near single-size have been difficult, if not impossible, to place without 'tearing' under the paver screed. Most of the sands in the vicinity of the project were either dune sands of near single-size or silty sands with appreciable plasticity. At the same time, the project sponsors were interested in using as much local sand as possible. Shell Canada's recommendation on gradation together with the grading limits selected for the project are shown in Figure 2.



U. S. Standard Sieves - ASTM Designation E 11-39

Figure 2. Master Gradation Chart Showing U.S. 77 Job Blend Relative to Specified and Recommended Limits.

The mineral aggregate selected by the contractor, Foremost Paving, Inc., consisted of a blend of two sands: 1) a concrete type sand from Wright Materials Co., 'Bluntzer' pit on the Nueces River near Corpus Christi, approximately 55 miles (89 km) north of the project, and 2) a field sand located about 500 ft (153 m) east of the project right-of-way at the hot mix plant site, station 2030. The aggregate requirements for the conventional hot-mix sections were those for a Type D aggregate as specified by SDHPT (1).

2.2 Equipment and Methods

The SAS pavement mixtures were prepared in a conventional stack-up type hot mix batch plant which was equipped with auxiliary systems for handling the liquid sulfur. The hot asphalt and liquid sulfur were transferred from separate storage into the weigh buckets by approved pumps. The dried and heated mineral aggregate was then weighed into the pug mixer and required amounts of hot asphalt and liquid sulfur in that sequence were then introduced into the mixer. Mixing was continued until a uniform paving material was prepared as required.

The emission control system consisted of an 8 ft (2.4 m) diameter cone precipitator and a wet washer supplement. Water for the wet washer, or scrubber, was truck-hauled to the site and discharged into a membrane lined pond. The pond doubled for sludge disposal and water storage. A small pump returned water from the surface of the pond to the washer in a continuous circulating system. A schematic of this emission control system is shown in Figure 3.

The coarse sand and fine sand were stored in separate stockpiles on the site. A caterpillar front-end loader was used to transport the

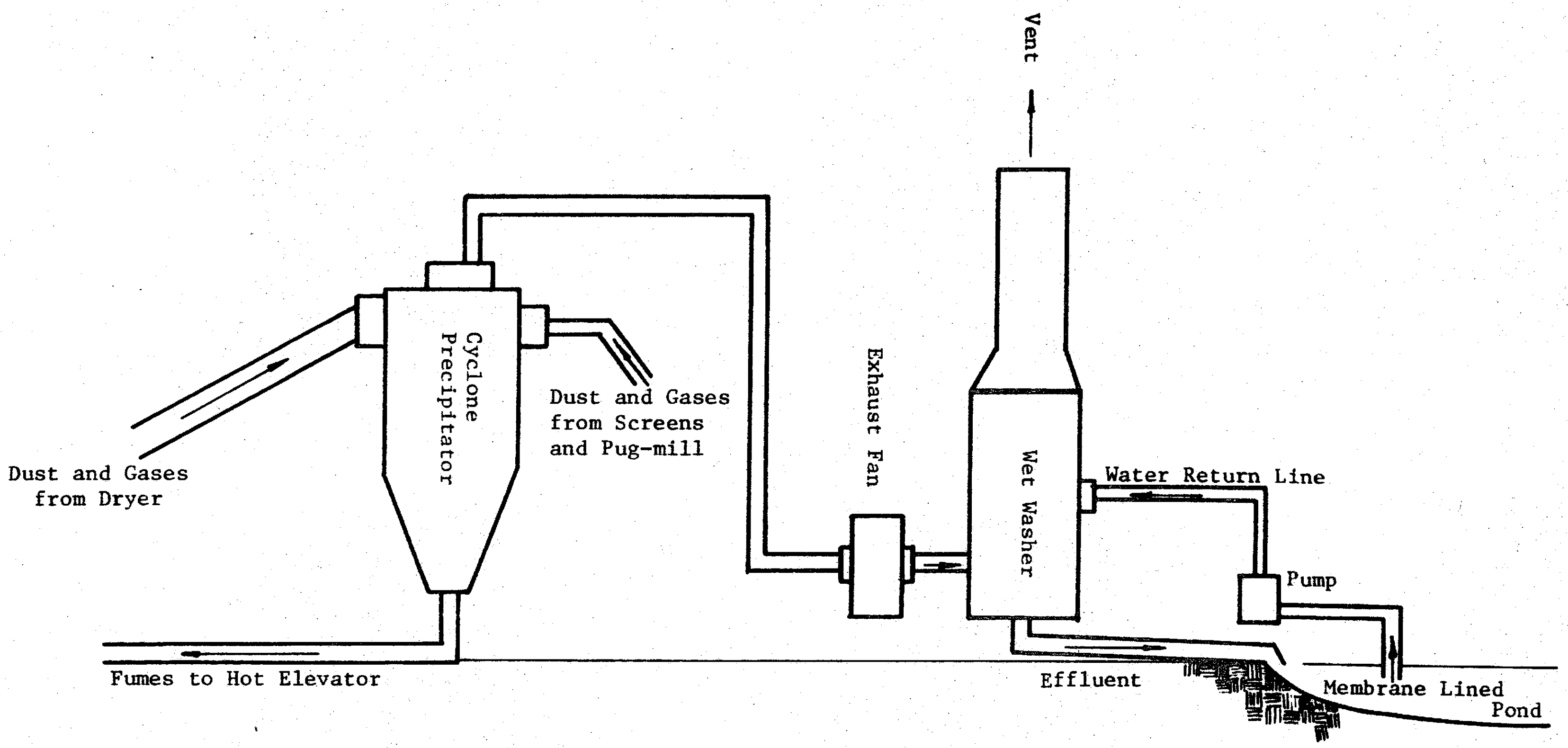


Figure 3. Schematic of the Emission Control System.

sands to a portable steel bin of which one-half was used for coarse and one-half for fine sand. The aggregate feeder system consisted of a conveyor belt that discharged into a funnel leading to the dryer.

The asphalt was stored in a salvaged horizontal railroad tanker which was equipped with heating coils and recording thermometer. Hot oil was provided by a Childress Oil Heater and an electric driven centrifugal pump was used for circulating the oil. The oil temperature was maintained at about 400°F (204°C) which kept the asphalt in storage at 290-300°F (143-149°C).

The sulfur system was designed by Mr. W. H. Richardson, Sr. Engineer of Texasgulf, Inc., Newgulf, Texas, and constructed by Mr. Parker New, Superintendent. Texasgulf provided much of the basic sulfur handling equipment and transported it to the construction site. The sulfur storage facility consisted of a used, horizontal 10 ft (3.1 m) diameter by 30 ft (9.2 m) long insulated tank heated with hot oil. The sulfur pump, attached pipe, valves and fittings, receiving hopper, and sulfur transports were steam heated. A schematic of the sulfur system is shown in Figure 4.

Dump trucks with special heated bodies were used to transport the SAS mixture from the hot-mix plant to the roadway. These bodies were developed by Shell Canada Limited, Oakville Research Centre to prevent the formation of cold lumps in the SAS mixture which may produce regions of weakness within the finished pavement. Figure 5 shows a schematic of the heated truck bodies used. The body, which was aluminum, had a tub-shaped inner shell and an outer shell insulated on the inside. The body was heated with propane burners, one on each side at the front

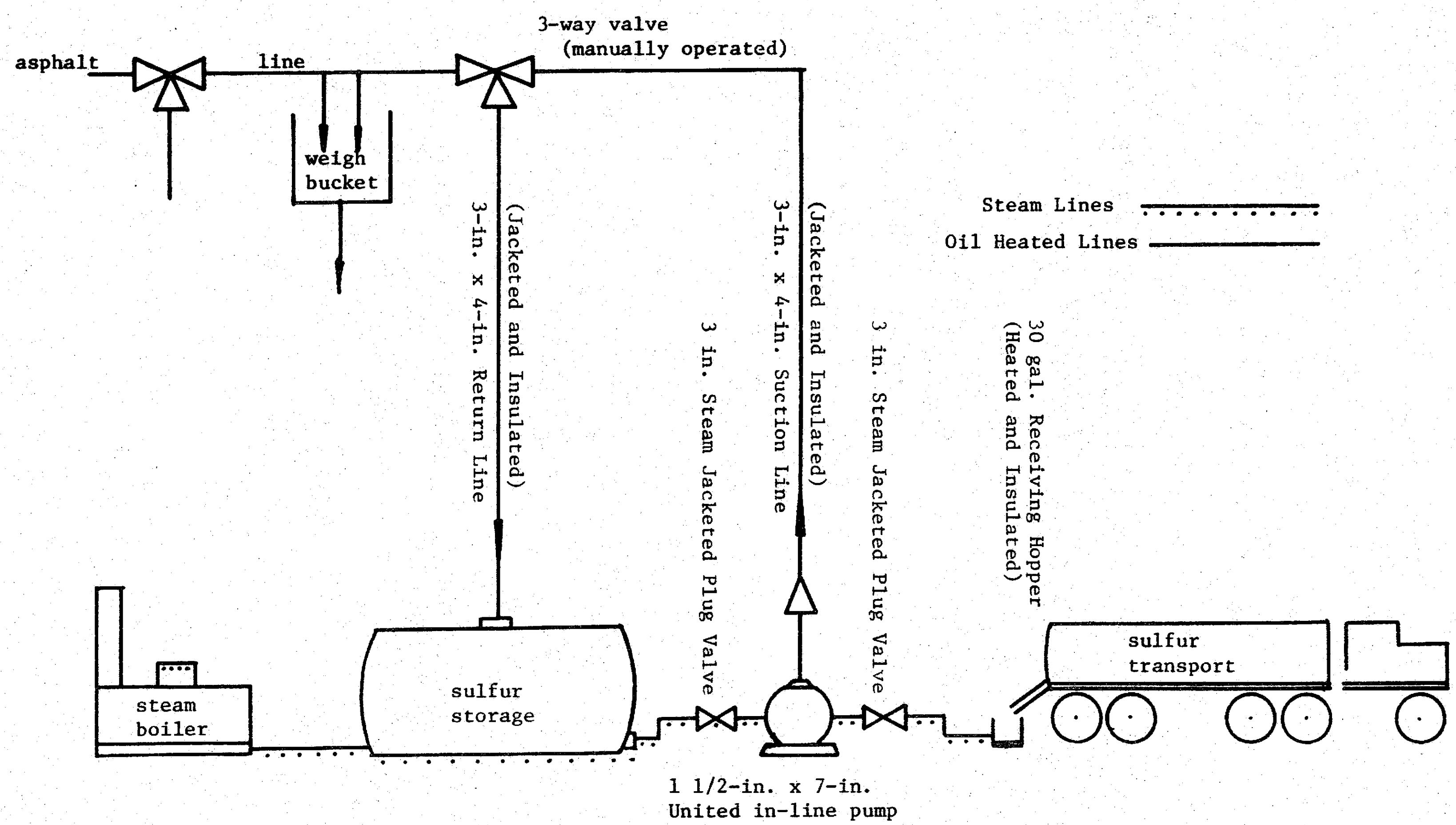


Figure 4. Schematic of the Sulfur System.

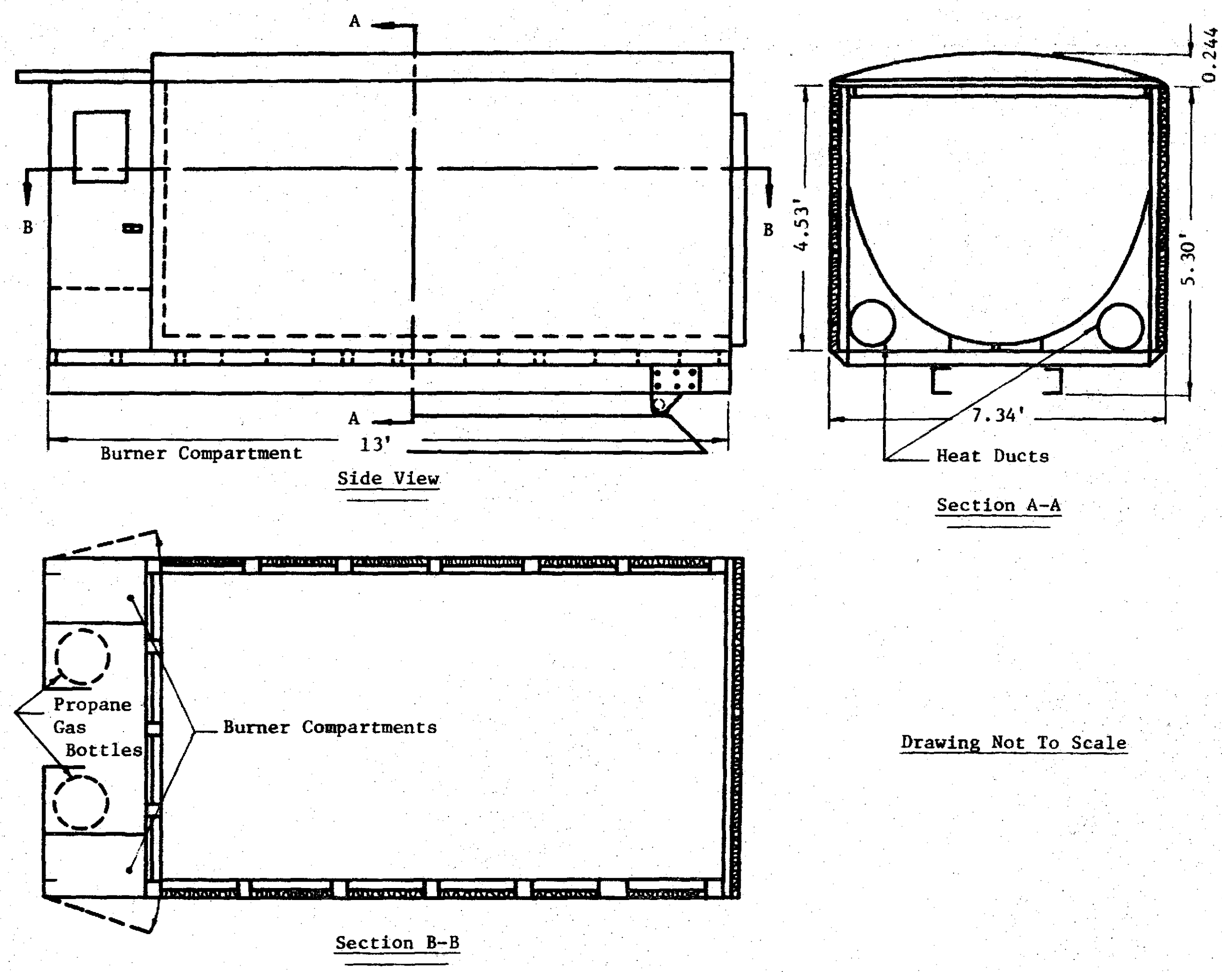


Figure 5. Heated Truck Bodies as Developed by Shell Canada Limited.

end. Cold air was forced into pipes above the flames by means of a fan. The mixture of hot burner exhaust gas and cold air was lead by ducts and baffles through the body and tail gate, then above the mix. The burners and fan were electrically operated from the truck batteries and controlled by on-off switches in the control panel on the driver's side of the cab.

The hot SAS pavement mixtures are very soft and plastic at the time of placement. They will not usually support the weight of the floating screed assembly on the conventional paver. For this reason the Barber Greene Company, Aurora, Illinois, in cooperation with Shell Canada Limited, Oakville Research Centre, developed a modified screed which was fully supported for strike-off, smoothing, and consolidation. No further compaction was needed for the SAS mixture since this would have destroyed the crystalline structure of the sulfur thereby reducing the quality of the pavement (1).

The subgrade for all test sections of the pavement was 8" (20.3 cm) lime-treated soil. In order to preserve the grade lines during construction traffic, the subgrade was covered with several inches of flexible base caliche. This cover was removed before placement of the pavement mixtures and the exposed surface was tacked with emulsified asphalt. The tack was entirely inadequate, effecting little or no bond of the paving mixture to the subbase. Some irregularities in the subgrade profile were noted in the 10 inch (25.4 cm) SAS section near station 1988. The construction of the SAS pavement was not a smooth operation. Placment was carried out in lifts of 3 inches (7.6 cm) maximum thickness. Some pavement layers, one lane wide and hundreds of feet in length, were placed true to line and grade without noticeable imperfections. Some segments were ragged with widespread imperfections of checks and tears. One possible reason for the tearing and checking may have been due to an excess of No. 50 to No. 100 mesh sand in the mixture. Another reason may have been due to lumps of cold material passing under the screed. In some stretches the grade undulated due to a "galloping screed". Two segments of the SAS test section were rejected by the engineers and removed with a blade grader.

A high batch-to-batch variability of the pavement material properties was noted during construction. It was suspected that this was due in part to the age and inefficiency of the hot-mix plant. The contractor did not have previous experience with SAS mixtures or with the special equipment involved in the operation. The laydown machine stopped and started frequently. This inconsistency coupled with the galloping screed tended to produce a poor riding surface.

2.3 Quality Control

The State Department of Highways and Public Transportation, District 21, provided a mobile laboratory for field control of the pavement construction.

Inspection personnel complied with the following schedule:

1. The temperature of each truck load of mix was checked and recorded,

2. Slump tests were made as required to monitor the consistency of the mixtures,

3. The binder content of mixtures--asphalt and sulfur, was determined,

4. A set of 9 Marshall test specimens was made at least twice daily with three specimens tested in field at 24 hrs and six specimens taken for TTI,

5. Forty pounds of mixture were taken each one-half day of operation and placed in containers for TTI, and

6. When plant was lined out one 200 lb sample of mixture was taken for study by TTI (1).

The mix design was selected well in advance of the project commencement. Selection was based on a minimum Marshall stability of 2000 lbs. (9,000 N) and a slump of 2 to 6 in (5.1 to 15.2 cm).

Variability in the aggregate proportioning device is the norm on most highway construction jobs. This variability may have been responsible for the excessive amount of No. 50 to No. 100 mesh sand that caused the tearing in the pavement as the mixture was being placed. For the most part, the operation was such as to result in a fairly uniform product.

Temperature control of the SAS mixtures was generally in the 272 to 290°F (133 to 148°C) range at the hot-mix plant. This range would drop to 266-282°F (130 to 144°C) in the field. These temperatures correspond to the working range considered acceptable for SAS systems. Temperatures were measured in the top of the loaded truck at the hot plant and in the paver hopper in the field. Between these two points a rather consistent loss of about 6°F (-14.4°C) was experienced (1).

Bitumen and sulfur content in the SAS mixture was determined by the following procedure:

1. The asphalt content was determined using a TROXLER Nuclear Asphalt Density Gauge.

2. The asphalt and some sulfur were extracted using a rotary extractor.

3. A representative portion or the entire sample from Step 2 was heated in a crucible to burn off the remaining sulfur.

4. The sulfur content was determined by subtracting the asphalt in Step 1 from total asphalt plus sulfur, Step 2 and 3.

This procedure gave the mix design 6.2 w/o asphalt, 13 w/o sulfur, and 80.8 w/o sand (1).

Specific results of the construction of the project that may have consequential effects in the post construction evaluation are summarized below:

1. Limitations in the design of the paver required that a 65/35 weight percent ratio of coarse to fine (local) sand be used to prevent tearing.

2. Because of the age and efficiency of the hot-mix plant used in the project, batch-to-batch variability was not as consistent as would be expected in a more modern plant. Part of the problem in maintaining a consistent quality paved surface can be attributed to variables in the hot-mix plant operations.

3. The heated dump truck bodies furnished by Shell performed well throughout the project. However, this job reconfirmed the need for heated dump bodies as an input to the success of this type of operation.

4. The need for good temperature control of the mix at the hot-mix plant and at the paver was demonstrated. At one point excessive temperature at the paver screed produced high emissions of H_2S and SO_2 gases. When the temperature was reduced to its proper range, the emissions also dropped to their normal, safe levels (1).

3.0 Testing Schemes for Post Construction Evaluation

Upon completion of the test sections, cores were obtained by District 21 personnel and a series of tests was run (2). Data were processed and a report was prepared. This testing period was designated as initial, (I). At 6-month intervals following construction, TTI personnel took cores and performed a series of tests on these cores. During the same 6-month intervals, SDHPT personnel collected field data in the form of Dynaflect deflections, Mays Ride Meter roughness measurements, and visual distress evaluations. Both in-situ testing and core testing have been performed in accordance with the Test Matrix presented in Figure 6.

3.1 Field Core Testing

3.1.1 Rice Maximum Specific Gravity

The maximum specific gravity of each paving material was found in accordance with ASTM Test Designation D 2041-71 (3). In this test, the pavement mixture is broken into fragments which are placed in a flask and covered with water. After this is done, as much air as possible is driven out of the mixture by a vacuum pump. The maximum specific gravity is then calculated by the following equation.

$$\text{maximum specific gravity} = \frac{A}{A+D-E}$$

where: A = weight of dry sample in air,
D = weight of flask filled with water at 77°F (25°C)
E = weight of flask filled with water and sample at 77°F (25°C) (3).

3.1.2 Bulk Specific Gravity

The pavement samples were taken with a core drill and cut into thicknesses ranging from 2 to 2 1/2 in (5.1 to 6.4 cm). Each prepared sample was tested for its bulk specific gravity according to ASTM Desig-

Test Description	Initial* I	6 mo.	Time Intervals 12 mo.	18 mo.	36 mo.
1. Traffic Analysis					
a. Average Daily Traffic Count	←		continuous		→
b. Truck and Axle Weight Distribution	○				○
2. Visual Evaluation	△	△	△	△	△
3. Mays Meter (PSI)	△	△	△	△	△
4. Dynaflect Deflections	△	△	△	△	△
5. Core Samples**					
a. Field Density and Rice Specific Gravity	△	△	△	△	△
b. Stability, Marshall	△	△	△	△	△
c. Stability, Hveem	△	△	△	△	△
d. Resilient Modulus	△	△	△	△	△
e. Indirect Tension	△	△	△	△	△
6. Interim Reports	△	△	△	△	△

○ Loadometer Survey, 1-Week Duration
△ Evaluations on Both Sand-Asphalt-Sulphur Mixes and Conventional Asphaltic Concrete Sections
* Initial Testing Performed One Week After Pavement Opened To Traffic
** Set of 3 Cores (minimum) at Each Test Section Per Sampling Period (Each Lane)

Figure 6. Testing matrix for SAS Trial , US 77, Kenedy County, Texas

nation D 2726-72 (4). In this test, the weights of the sample are taken in air, water, and in the saturated surface-dry condition. The equation for calculating this property is:

$$\text{bulk specific gravity} = \frac{A}{B-C}$$

where: A = weight of dry sample in air,
B = weight of the saturated surface-dry specimen in air, and
C = weight of the specimen in water (4).

The bulk density of the paving mixture can then be calculated by multiplying the bulk specific gravity by 62.4 pcf (999 kg/m^3), the unit weight of water at 77°F (25°C).

3.1.3 Marshall Stability and Flow

The portion of the field cores that were subjected to the Marshall stability test was tested according to the procedure established by ASTM Designation D 1559-76 (5). This method measures the resistance of a paving mixture to plastic flow. The measurement, in turn, gives an indication of a pavement's ability to withstand traffic loads and permanent deformation.

In the test procedure, samples of 4 in (10.2 cm) in diameter and about 2 to 2 1/2 in (5.1 to 6.4 cm) thick are heated to 140°F (60°C). These specimens are then placed in the loading head of the testing mechanism and the load is applied at a rate of 2 in/min (5.1 cm/min). A strip chart recorder measures the load and plots it against time. The load required to produce failure is termed the Marshall stability. The deformation of the specimen from the beginning of the test to the point of failure is termed the Marshall flow and is expressed in 1/100 in (2.5 cm) (6).

3.1.4 Hveem Stability

Although the Marshall method is the most commonly accepted means of measuring the stability of a pavement, some state and municipal agencies have adopted the Hveem concept which was set forth by F. N. Hveem, formerly of the California Division of Highways. This test has been standardized and may be found under ASTM Designation D 1560-76 (7).

In this procedure, a specimen of the same dimension as that used in the Marshall procedure is placed in a Hveem stabilometer. The specimen is subjected to a gradually increasing vertical load. As this load is applied the sample attempts to deform laterally. This lateral spread results in pressure being exerted on an annular oil cell which is separated from the specimen by a rubber diaphragm. The lateral pressure is read from a hydraulic gauge at selected vertical loads and a reading of the final displacement of the sample is taken. The Hveem stability is calculated by the equation:

$$S = \frac{22.2}{[(P_h \times D)/(P_v - P_h)] + 0.22}$$

where: S = stabilometer value
P_h = horizontal pressure, for a corresponding P_v,
D = displacement on specimen, and
P_v = vertical pressure (typically 400 psi or 2,800 kPa) (7).

3.1.5 Resilient Modulus

Young's modulus or the elastic modulus of an elastic material is defined as the ratio of stress to strain. This same relationship applies to viscoelastic materials except that the conditions of testing must be specified. This specification of test conditions is necessary because of the tendency of a viscoelastic material to creep the longer it is loaded. The result of increased creep will be a lower modulus value. Time-dependent moduli are termed "resilient moduli" (8).

In Schmidt's resilient modulus procedure, a sample of the size used in both the Marshall and Hveem procedures is used. A pulsating load is applied across the vertical diameter of the specimen which causes an elastic deformation across the horizontal diameter. This deformation is measured with transducers which require very little activating force (9). The load duration on the materials from the field trials was 0.1 second applied every 3 seconds (8). All of the samples from the field trials were tested at 68°F (20°C).

The equation used for calculating resilient modulus is:

$$M_r = \frac{P(\mu + 0.2734)}{\Delta t}$$

where: M_r = resilient modulus,
P = load,
μ = Poisson's ratio,
Δ = total sample deformation, and
t = sample thickness (8).

For elastic materials, this equation should apply to loadings in either the static or dynamic state. This equation applies reasonably well to viscoelastic materials provided that the loading time is short enough to minimize the viscous effects (9). The 0.1 second load duration is considered to meet this requirement.

Schmidt recommends a value of 0.35 for Poisson's ratio. His recommendation is based on experiments which have shown that Poisson's ratio varies from 0.2 to 0.5 for asphaltic materials. Using a value of 0.35 for μ in the resilient modulus equation would result in an error of no greater than $\pm$ 25% (9). Gallaway and Saylak have suggested a value of 0.30 for Poisson's ratio of sulfur-asphalt materials (10).

3.1.6 Indirect Tensile Strength

Typically this type of test has been used for concrete and mortar. Recently, however, this test has been applied to asphalt-stabilized materials (11). In this test, a cylindrical specimen is subjected to a compressive load from opposite sides of its diametral (diameter) plane. The result of this loading is a tensile failure which generally occurs along the diametral plane (12).

The indirect tensile test method is standardized for portland cement concrete in ASTM Designation C 496-71 (12). There are some significant differences between the way this test is performed on portland cement concrete and the way it is applied to asphaltic concrete. For the purpose of this field trial, the samples were the same size as those used in the Hveem and Marshall test procedures. The loading rate throughout the course of testing was 2 in/min (5.1 cm/min). All of the samples were tested at 68°F (20°C). The equation used to calculate the indirect or splitting tensile strength is:

$$S_t = \frac{P\,max}{\Pi\ td}$$

where: S_t = tensile strength (psi or kPa),
$P\,max$ = maximum load, lbs
t = thickness of the sample, inches, and
d = diameter of the sample (13).

3.2 Condition and Performance Data

At approximately the same intervals during which cores were taken from the field trial. condition and performance data were taken from the road site by the SDHPT, District 21 personnel. These data were in the form of Mays Ride Meter values, Dynaflect deflection, and visual inspection.

3.2.1 Mays Ride Meter (MRM)

The Mays Ride Meter was developed in 1967 by Ivan Mays, Senior Design Engineer of the Texas Highway Department. The idea was to provide a simple and useful means for measuring the ride quality of roads (14). Studies conducted at TTI concluded that the MRM was the most appropriate device for general field use when compared to three other popular roughness measuring devices (15). Two of the greatest advantages of the MRM are its ease of operation and its permanent roughness recording with respect to location (14).

The two main components of the MRM are the transmitter and the recorder. The transmitter is located in the trunk of the car directly over the center of the differential housing. A cable extends from the transmitter to the center of the differential housing. This cable gives the transmitter a solid drive mechanism for detecting roughness. The recorder is self-contained and may be either strip chart or digital read-out. As the vehicle travels on the road, the transmitter detects relative vertical motion between the automobile and the differential housing with a 0.1 in (2.54 mm) resolution. The recorder uses the electrically transmitted data to provide a continuous indication of the road roughness (16).

Measurements taken by the MRM are converted to values of serviceability index (SI). This index was developed during the American Association of State Highways and Transportation Officials Road Test (AASHTO) in an attempt to standardize a performance measurement procedure (17). In this road test, panels of people were asked to rate different conditions of pavement from 1 (very poor) to 5 (very good). Their evaluations were termed the present serviceability rating. The SI is an estimate of the present serviceability rating (18).

3.2.2 Dynaflect Deflection

The Dynaflect is a pavement deflection measuring device which was developed by the Lane-Wells Division of Dresser Industries, Inc. This system is composed of a small two wheel trailer which contains a dynamic force generator and a set of motion-sensing devices. Deflections of the material underneath the trailer, caused by a cyclic downward force, are measured while the trailer is halted at the test location (19).

The cyclic force is generated by a pair of counter-rotating unbalanced fly wheels which produce a 1,000 lb (4,450 N) vertical dynamic load. The vertical displacement caused by this load is sensed by geophones which are lowered into contact with the surface. The geophones are spaced at 1 ft (30.5 cm) intervals for a distance of 5 ft (152.5 cm) from the center of the loading wheels. A lifting device places the force generator in and out of contact with the surface of the road (19).

Since this test is in-situ, exposure to the elements must be considered. A study conducted on the seasonal variations of Dynaflect measurements has shown that weather does play an important role in the deflection of pavement materials (20). For this reason, the deflection measurements were corrected to 60°F (15°C) according to a temperature adjustment procedure established by a study conducted at the Utah

Department of Transportation (21). This procedure takes into account the pavement surface temperature and the average of the previous 5-day ambient temperature in order to arrive at an average full-depth pavement temperature. Since the pavement surface temperature was rarely recorded during the course of the field trial, the average full-depth pavement temperature was estimated according to a procedure set forth by Witczak (22).

Figure 7 shows the results of the Utah Department of Transportation study. This graph may be used for conventional asphaltic concrete materials. It is used by entering the graph at a particular pavement temperature and extending a line to the correlation curve. Next a line extended downward vertically to the proper Dynaflect correction factor. The measured deflection is multiplied by the correction factor to adjust the measurement to 60°F (16°C).

Since the temperature dependency of sulfur-asphalt materials is somewhat different than that of conventional asphaltic concrete, a modified approach to temperature correction was needed. This modification was effected by first plotting the curve of resilient modulus versus temperature for SEA, SAS, and asphaltic concrete (Figure 8). These curves give an indication of the stiffness of the materials at different temperatures. From Figures 7 and 8, a curve was made of resilient modulus versus Dynaflect correction factor for conventional asphaltic concrete (Figure 9). To use the modified approach, the pavement temperature is entered on the temperature axis on Figure 8, a line is projected to the curve of the pavement material being evaluated, and a horizontal line is extended to the resilient modulus axis. Next the resilient modulus is

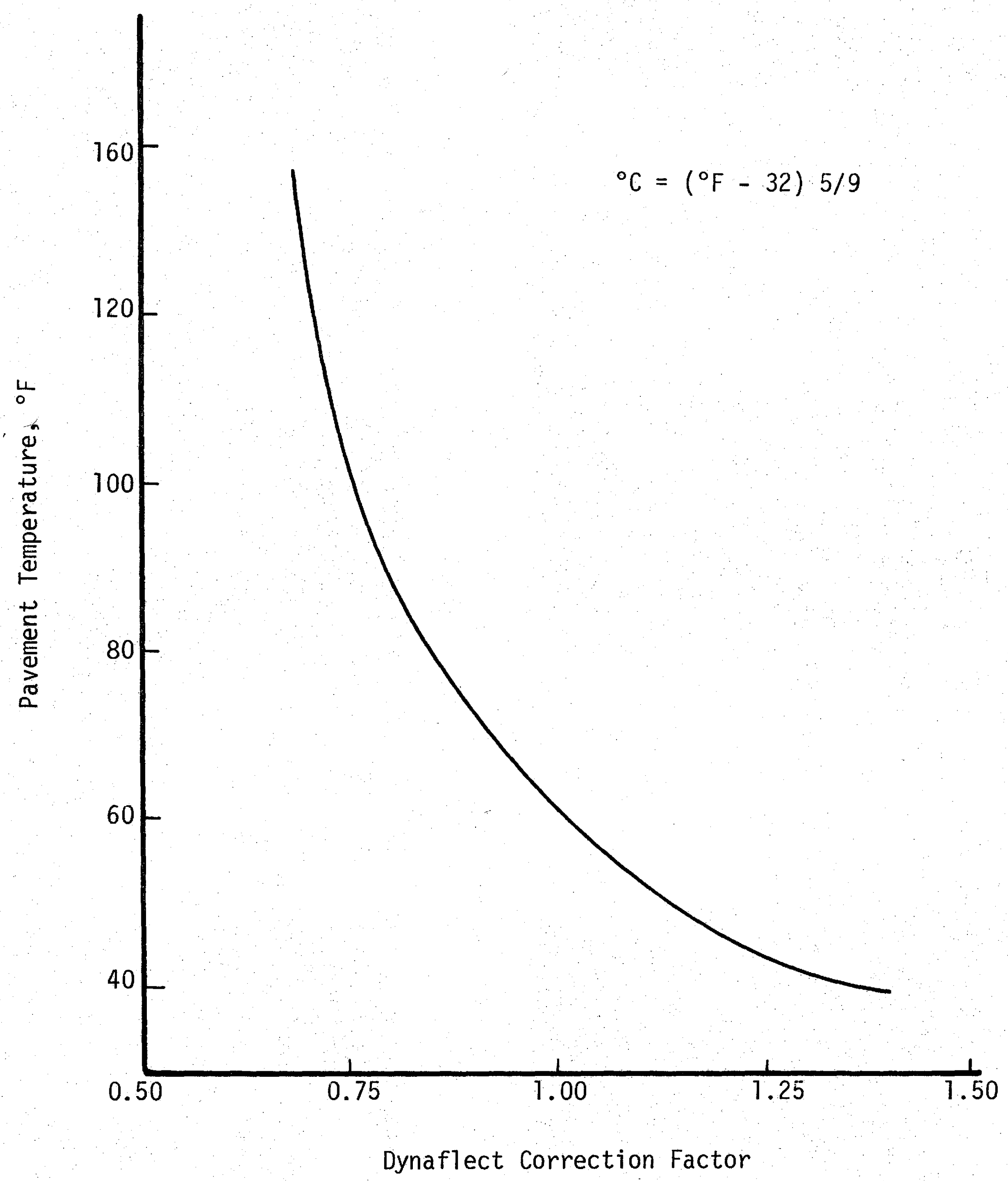


Figure 7. Pavement Temperature versus Dynaflect Temperature Correction (21).

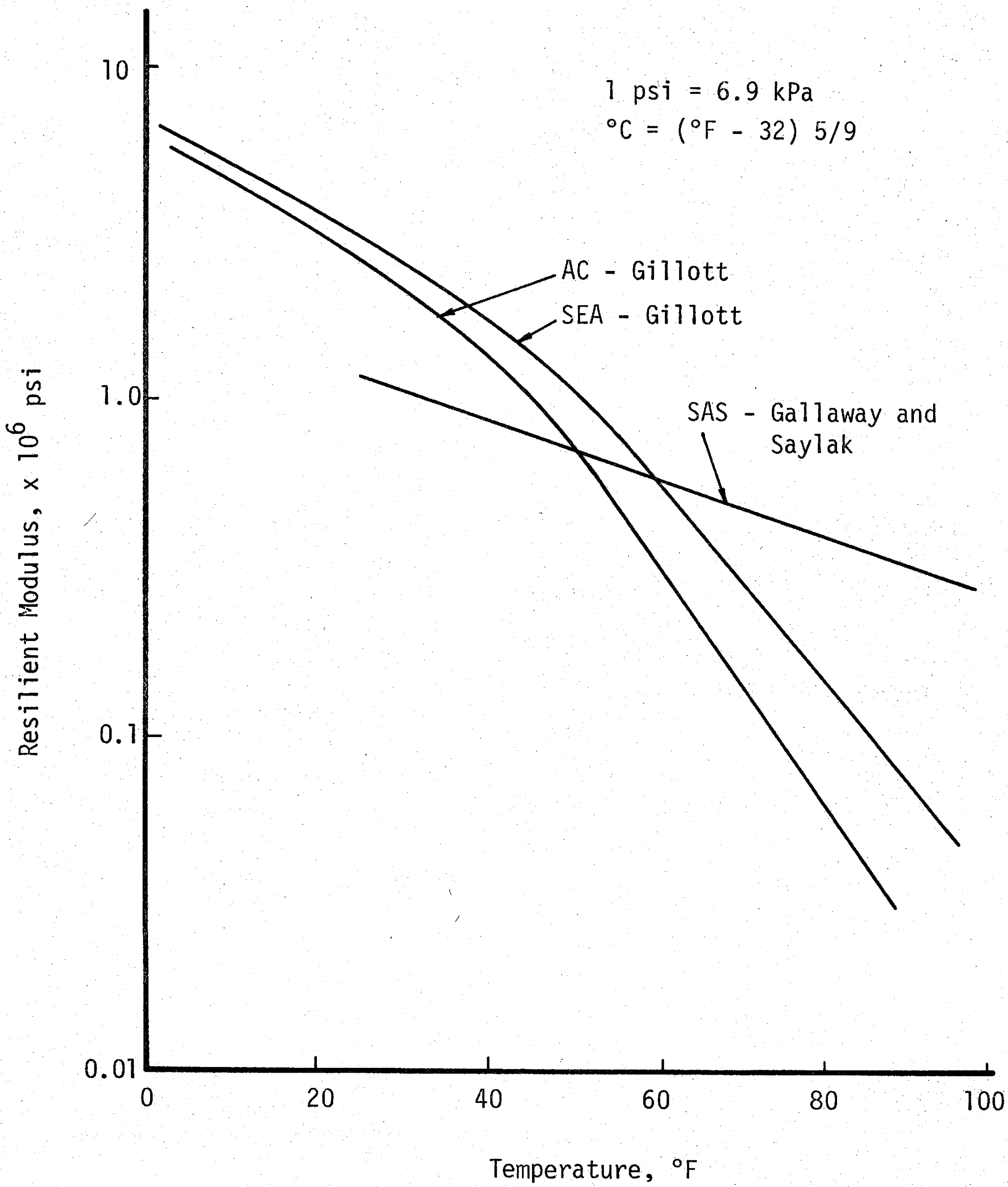


Figure 8. Resilient Modulus versus Temperature

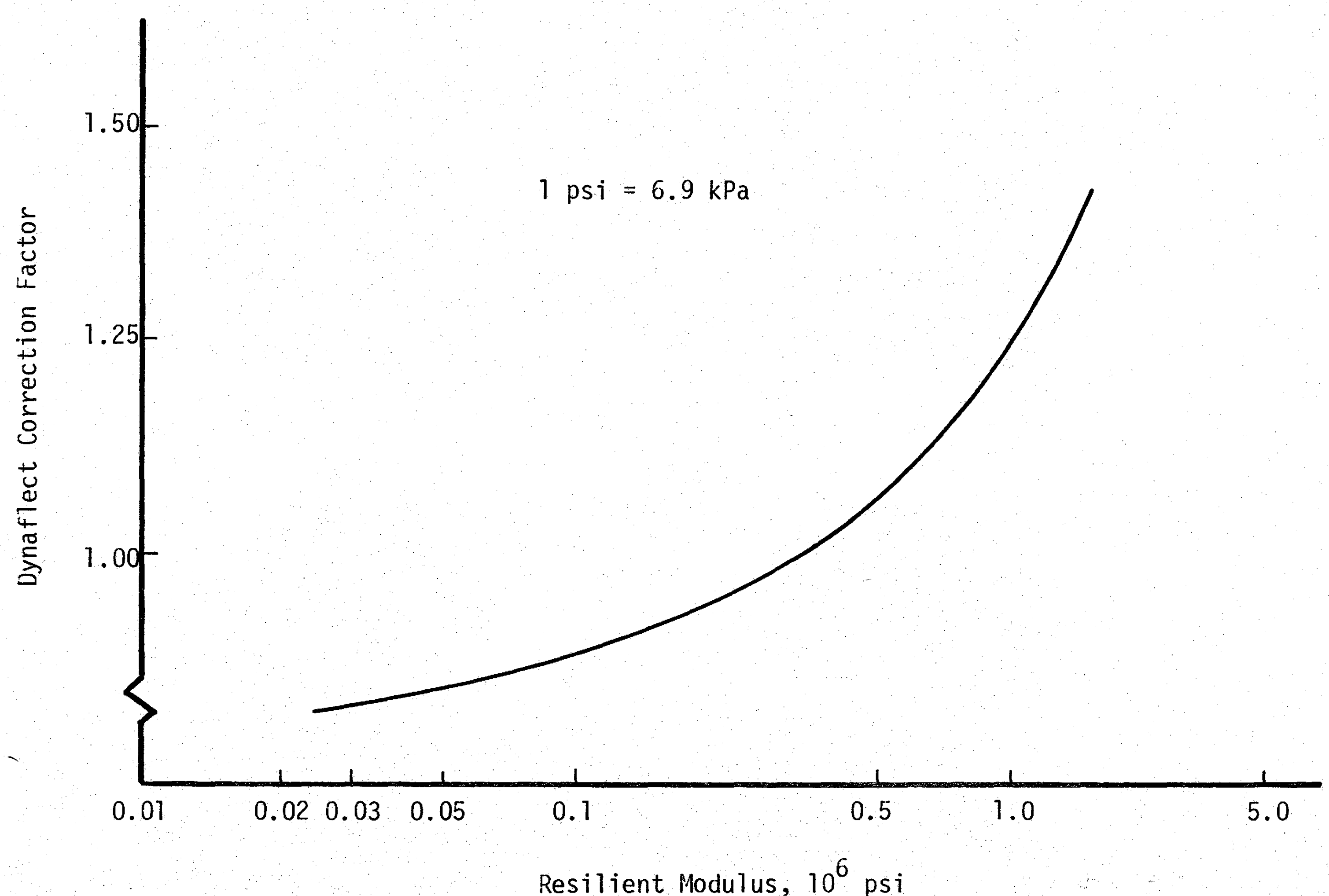


Figure 9. Dynaflect Correction Factor versus Resilient Modulus,

entered on the proper axis of Figure 9 and a line is extended to the curve. From this point a line is projected to the proper Dynaflect correction factor for the particular material.

3.2.3 Visual Inspection

Visual inspection of pavements has been considered one of the most reliable indications of pavement performance (22). In Texas, visual inspection of roadways has been standardized by the SDHPT for both rigid and flexible pavements. Items which enter into this rating include the location and type of pavement, amount and degree of distress, conditions of the shoulder, roadside and drainage characteristics, and traffic service features. The Mays Ride Meter is used in conjunction with this inspection to establish the riding quality of the pavement.

Visually distinguishable pavement distress was presented literally in the Kenedy County project. In this case, sketches of the road were made indicating the location of the pavement distress.

4.0 Data Analyses for U.S. 77

The Kenedy County field trial has now been in place for three years. The last period for collecting data was scheduled for the thirty-eighth month. Including the final testing, there have been six testing periods. There were three thicknesses of two different materials used in this test section. The materials were a sand-asphalt-sulfur (SAS) mixture with a 6.2:13 weight percent asphalt to sulfur binder and a conventional hot-mixed asphaltic concrete (HMAC) with a binder content of 5.8 weight percent of binder. Of each of these materials, there was a 4 in (10.2 cm), 7 in (17.8 cm), and 10 in (25.4 cm) base course thicknesses used.

Direct comparisons of these materials have been made in this study.

4.1 Laboratory Results

Samples were obtained from cores that were taken from U.S. 77 at specified points in time for laboratory testing. The data from these tests include bulk specific gravity, Marshall stability and flow, Hveem stability, resilient modulus at 68°F (20°C), and splitting tensile strength at 68°F (20°C). These data are presented in Table 1. It may be noted that in all the graphs of laboratory testing, the initial data point for the 10-inch (25.4 cm) HMAC section is missing. This omission was due to difficulty encountered in the initial coring of that section.

4.1.1 Bulk Specific Gravity

Traffic compaction has not had much effect on the bulk specific gravity of the materials as can be seen in Figure 10. All of the SAS sections have specific gravities of about 2.05. This has remained relatively constant throughout the study with the exception of one outlier for the 10 in (25.4 cm) section at the eighth month. A bulk specific gravity of 2.05 translates to a bulk density of 128 pcf (2,048 kg/m^3). The maximum specific gravity of the SAS material is 2.28. Therefore, the air voids in this material are about 10 percent of the total volume of the mixture. This is 2 percent above The Asphalt Institute's recommended upper limit of 8 percent. These voids are not considered to be connected and do not contribute to the water susceptibility of the material.

The bulk specific gravity of the HMAC material has leveled-out to around 2.29 after 20 months as seen in Figure 10. This is the equivalent of a bulk density of 143 pcf (2,290 kg/m^3). The maximum specific gravity

Table 1. Field Core Test Results for U. S. 77.

Base Type	Sulfur/ Asphalt Ratio	Specific Gravity	Marshall Stability lbf	Marshall Flow 0.01 in.	Hveem Stability, percent	Resilient Modulus at 68°F, psi x 10^6	Splitting Tensile, psi	Sampling Ratio (Age)**	Rice Max. Specific Gravity
		2.02	1350	17	25	0.46	155	4/77(0)	
		2.20	1445	8	31	0.70	160	12/77(8)	
10" SAS*	13/6.2	2.04	2070	10	42	0.48	200	6/78(14)	2.29
		2.02	1725	9	30	0.73	178	12/78(20)	
		2.04	1535	9	38	0.57	169	6/79(26)	
		2.02	1500	11	24	0.67	158	6/80(38)	
		2.01	1885	15	34	0.44	145	4/77(0)	
		2.04	1740	9	30	0.64	150	12/77(8)	
7" SAS*	13/6.2	1.99	1210	10	28	0.48	205	6/78(14)	2.24
		2.04	1975	9	36	0.77	168	12/78(20)	
		2.02	1430	9	29	0.52	160	6/79(26)	
		2.04	1991	11	30	0.68	166	6/80(38)	
		2.01	1890	14	32	0.45	155	4/77(0)	
		2.05	1875	10	38	0.77	185	12/77(8)	
4" SAS*	13/6.2	2.05	1450	9	30	0.55	235	6/78(14)	2.31
		2.05	1785	10	30	0.91	183	12/78(20)	
		2.05	1190	10	33	0.56	184	6/79(26)	
		2.03	1408	14	27	0.87	188	6/80(38)	
		2.13	340	11	36	0.73	215	4/77(0)	
		2.25	580	13	26	1.28	290	12/77(8)	
4" AC	0/6.2	2.25	930	14	27	1.16	325	6/78(14)	2.38
		2.29	660	13	25	1.52	291	12/78(20)	
		2.29	730	18	31	1.10	278	6/79(26)	
		2.26	475	10	27	1.64	218	6/80(38)	
		2.26	675	18	***	0.81	240	4/77(0)	
		2.26	665	11	27	1.23	255	12/77(8)	
7" AC	0/6.2	2.25	685	14	26	0.99	273	6/78(14)	2.38
		2.29	520	11	28	1.41	279	12/78(20)	
		2.31	500	9	29	0.74	247	6/79(26)	
		2.29	***	***	28	0.98	207	6/80(38)	
		***	***	***	***	***	***	4/77(0)	
		2.24	705	12	29	1.12	255	12/77(8)	
10" AC	0/6.2	2.27	420	12	24	1.02	310	6/78(14)	2.40
		2.29	645	11	29	1.54	262	12/78(20)	
		2.32	730	12	22	0.75	256	6/79(26)	
		2.28	522	8	32	1.36	215	6/80(38)	

*The mix design established for these systems was 6.2 weight percent asphalt and 13 weight percent sulfur

**Pavement age in months

***Difficulty collecting sample

1 lbf = 4.45N

1 in = 2.54 cm

1 psi = 6.89 kPa

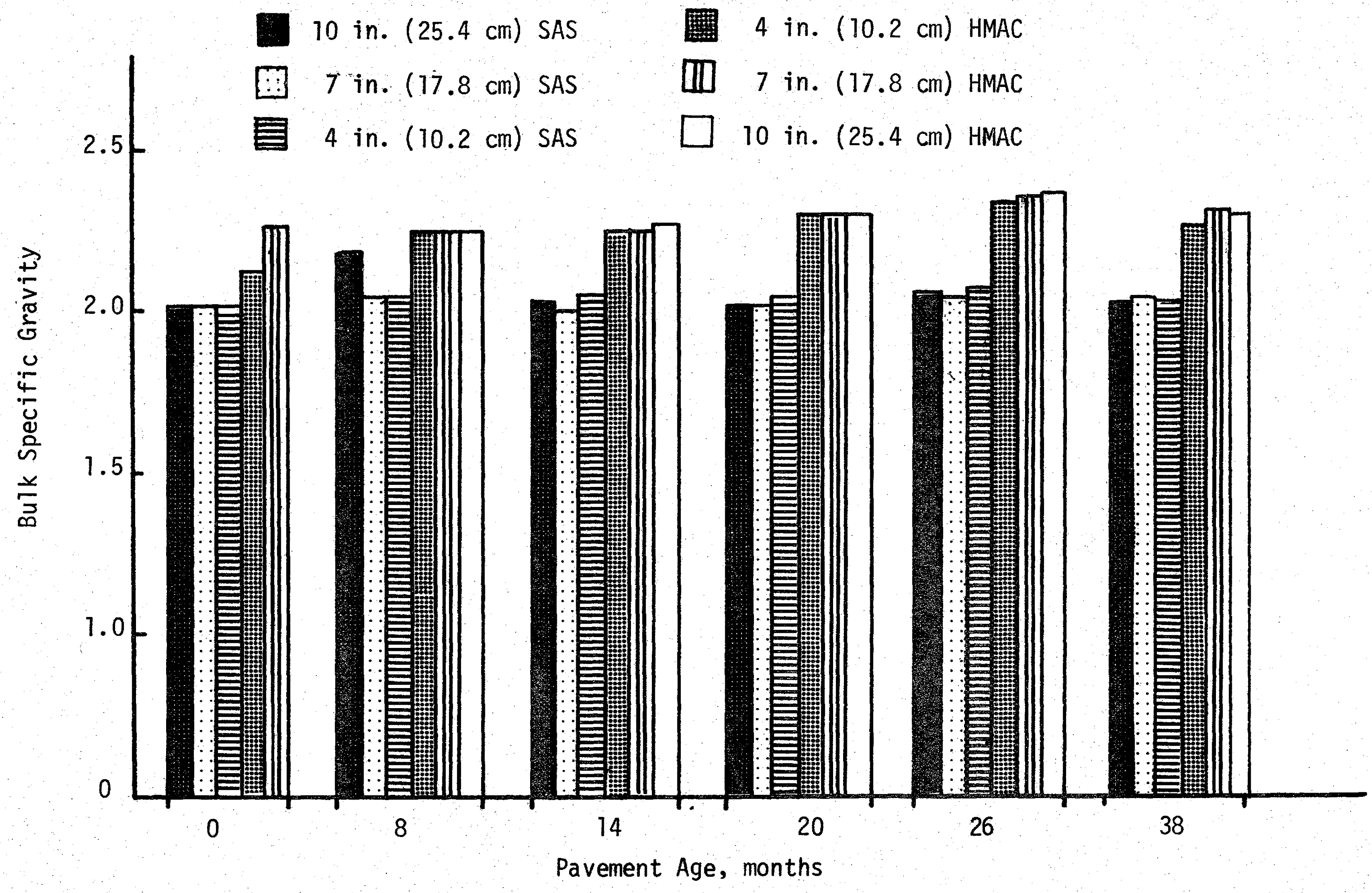


Figure 10. Bulk specific gravity versus pavement age for U. S. 77.

of this material is about 2.38. Therefore, the air voids in the mixture are about 4 percent. This is 1 percent above The Asphalt Institute's recommended value of 3 percent.

4.1.2 Marshall Stability and Flow

The Marshall stability and flow characteristics of the SAS and AC materials are shown in Figures 11 and 12, respectively. In Figure 11, it is shown that the stability for the SAS mixture has been consistently higher than that of the conventional HMAC material. Pavement thickness does not seem to be a factor in either of the materials. The variability of the data for either material is not outside of that which is normally expected for this test. Neither of the materials show a clear trend of stiffening with time. This stiffening is normally expected in asphaltic materials due to traffic compaction and the increase in the mass viscosity of the asphalt from thermal cycling.

Figure 12 shows the Marshall flow characteristics of the pavement materials. In this figure, it may be noticed that the flow is greater for the AC mixture than the SAS mixture for any of the testing periods after the initial period with the exception of the 4" SAS section at the 38th month. For the SAS mixture, the higher stability and lower flow values are indicative of a greater mass viscosity for this material. Figure 12 also reveals that after the initial testing period, the flow values decreased for both materials until the 26th month. The reader may note that this decrease was of a greater magnitude for the SAS material. Once again pavement thickness was not a factor in the data trends.

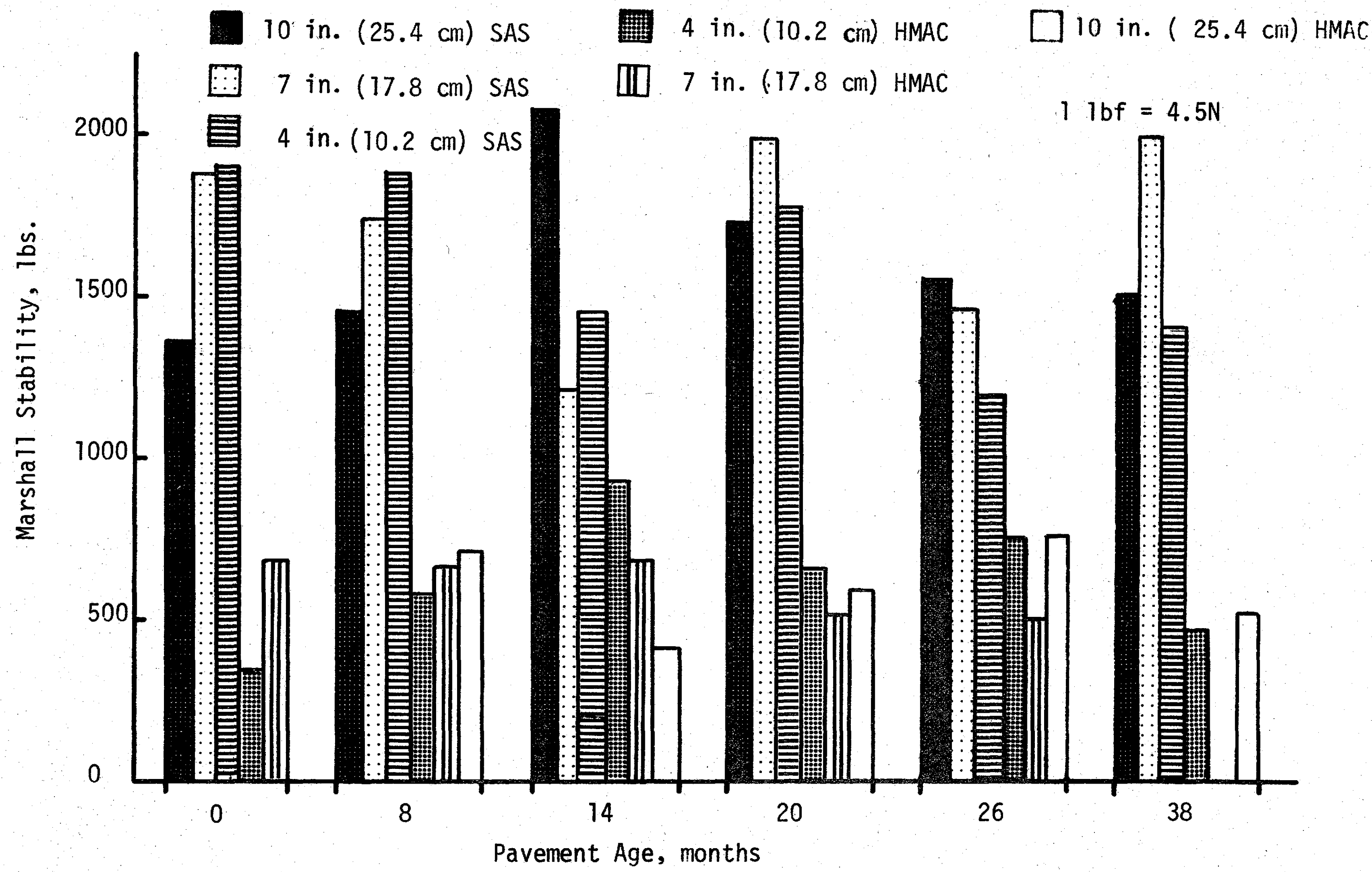


Figure 11. Marshall stability versus pavement age for U. S. 77.

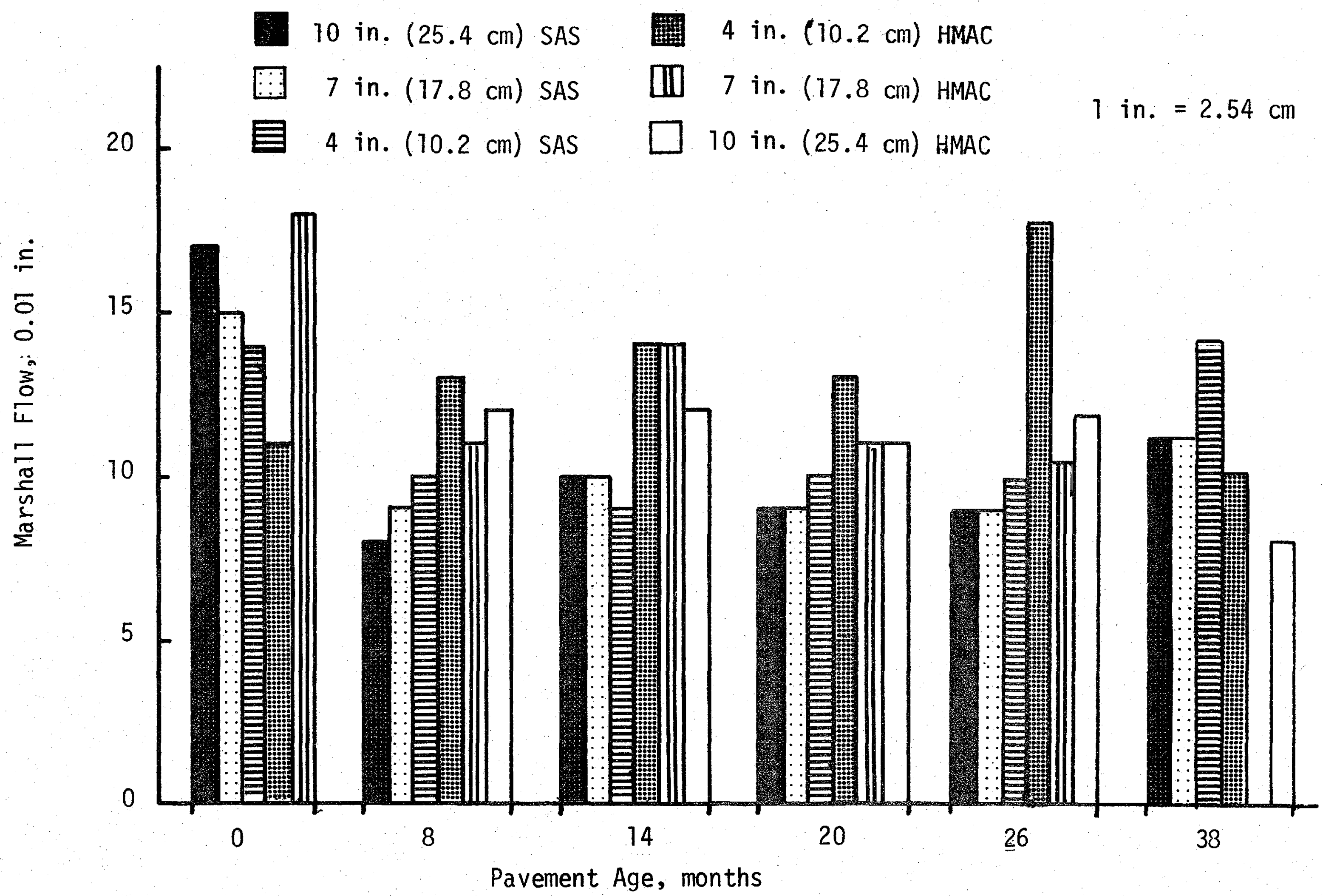


Figure 12. Marshall flow versus pavement age for U S. 77.

4.1.3 Hveem Stability

The Hveem stabilities for the base materials with respect to pavement age are shown in Figure 13. At the initial testing point, no data was available for the 7 in (178 mm) or the 10 in (254 mm) HMAC base courses. However, initially, the HMAC material seems to have a higher Hveem stability, 35 percent, than the SAS material which had an average value of 30 percent. After this point in time, the stability of the SAS mixture seems to average approximately 33 percent and the HMAC stability at about 27 percent. This means that the SAS material has a greater resistance to flow than HMAC mixture.

4.1.4 Resilient Modulus

The HMAC mixture has a consistently greater resilient modulus at 68°F (20°C) than the SAS material as demonstrated in Figure 14. Both materials seem to be gaining stiffness with age until the 26th month. The SAS mixture had an initial resilient modulus of 0.40×10^6 psi (2.76×10^6 kPa) and a maximum resilient modulus of about 0.80×10^6 psi (5.51×10^6 kPa) after 20 months. The AC material started with a value of 0.75×10^6 psi (5.18×10^6 kPa) after construction which rose to a value of 1.50×10^6 psi (10.34×10^6 kPa) at 20 months. Here it is believed that the greater shear capacity of the well-graded, type D aggregate in the HMAC mixture produced a material which has a greater mechanical interlock than the sand and asphalt in the SAS mixture.

4.1.5 Splitting Tensile Strength

This same reasoning is applied to the pattern in Figure 15 where the splitting tensile strength of the HMAC material is consistently

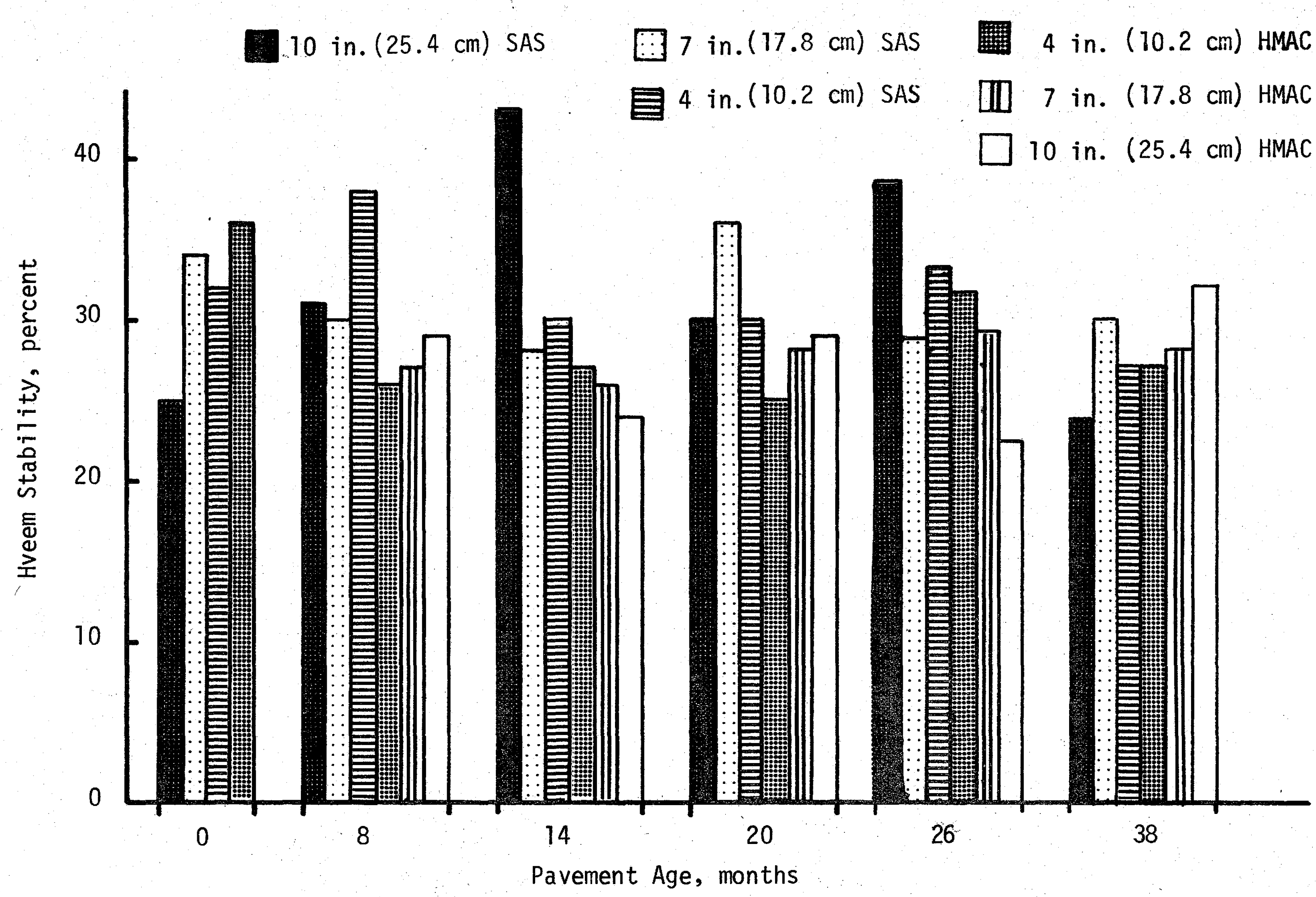


Figure 13. Hveem stability versus pavement age for U. S. 77.

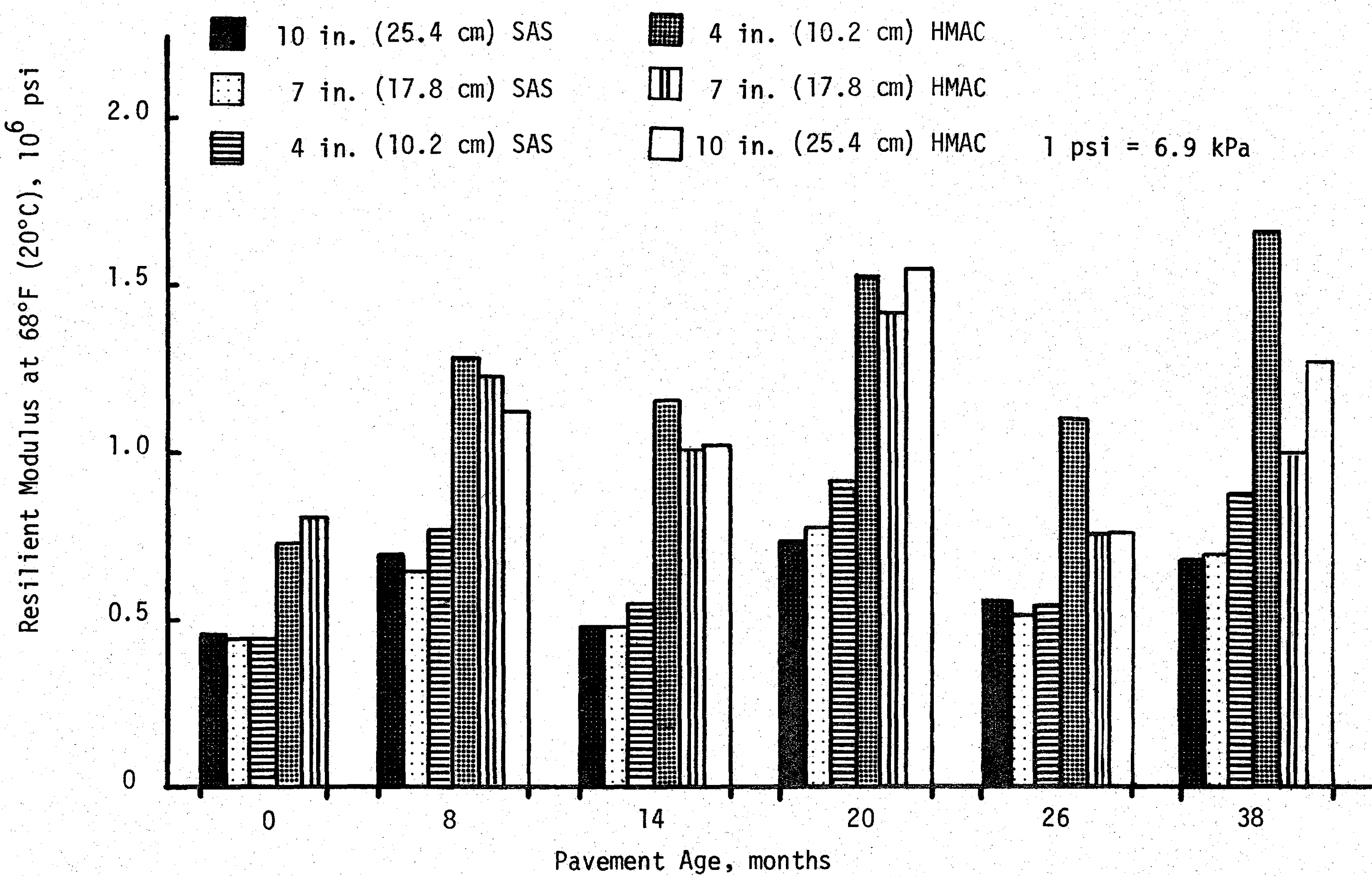


Figure 14. Resilient modulus at 68°F (20°C) versus pavement age for U. S. 77.

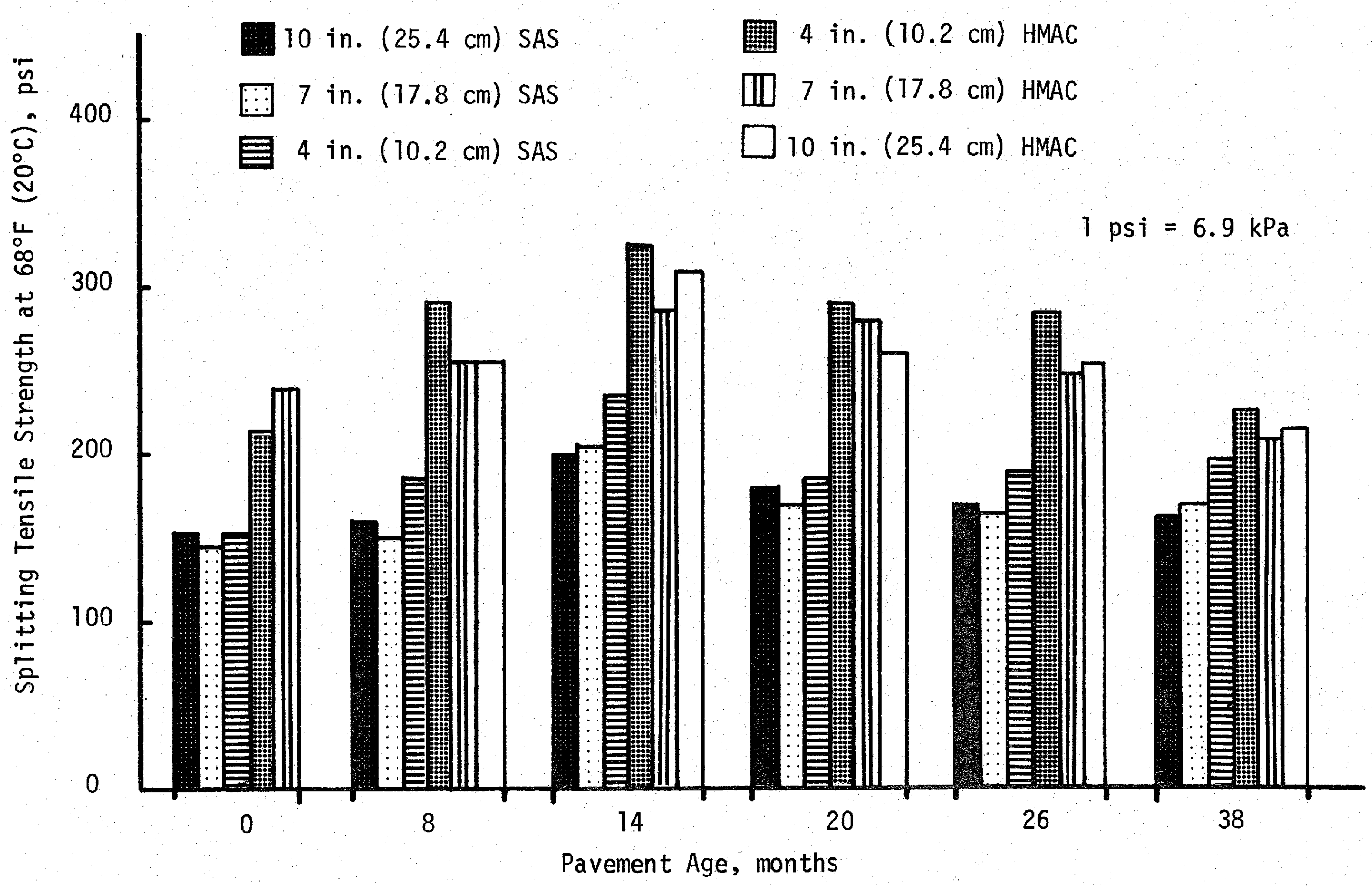


Figure 15. Splitting tensile strength at 68°F (20°C) versus pavement age for U. S. 77.

greater than that of SAS. Both the SAS and HMAC materials seem to have started on a generally upward trend until the 20th month when they both began to decline. The SAS material seems to have increased in strength from approximately 150 psi (1,035 kPa) to about 200 psi (1,389 kPa). The splitting tensile strength of the HMAC material was initially 220 psi (1,520 kPa) and increased to about 290 psi (2,000 kPa).

4.2 Condition and Performance Results

At approximately the same time that cores were taken from U.S. 77, in-situ testing was conducted by SDHPT personnel. These tests include Dynaflect deflections, visual inspection, cracking survey, and Mays Ride Meter readings.

4.2.1 Maximum Dynaflect Deflection

Figure 16 shows the maximum Dynaflect deflections for the different thicknesses of the two materials. These data are summarized in Table 2. For the two 10 in (254 mm) base courses, the deflections are equal at 8 months, 0.44×10^{-3} in (11×10^{-3} mm). After this point, the 10 in (254 mm) HMAC section had higher deflections than the equivalent SAS section. In the 7 in (17.8 cm) base sections, the HMAC had a consistently higher deflection than the SAS material. This trend was reversed in the 4 in (10.2 cm) sections, with the 4 in SAS sections having the highest deflections of all the sections. There may be indications of some of the pavement sections stiffening with time, although it is too early to state conclusively.

Table 2. Maximum Dynaflect Deflections for U. S. 77.

S/A Ratio** and Mix Type	Station	Pavement Thickness, in.*	Maximum Dynaflect Deflection, 10-3 in.	Sampling Date (Age)***
6.2/13 SAS	1985+00		0.44	12/13/77(8)
	to	11	0.48	6/6/78(14)
	1990+00		0.40	12/4/78(20)
			0.37	6/5/79(26)
			0.40	6/25/80(38)
6.2/13 SAS	1990+00		0.56	12/13/77(8)
	to	8	0.61	6/6/78(14)
	1995+00		0.53	12/4/78(20)
			0.46	6/5/79(26)
			0.52	6/25/80(38)
6.2/13 SAS	1995+00		0.88	12/13/77(8)
	to	5	0.90	6/6/78(14)
	2000+00		0.86	12/4/78(20)
			0.67	6/5/79(26)
			0.79	6/25/80(38)
0/6.2	2000+00		0.72	12/13/77(8)
	to	5	0.73	6/6/78(14)
	2005+00		0.74	12/4/78(20)
			0.55	6/5/79(26)
			0.60	6/25/80(38)
0/6.2	2005+00		0.68	12/13/77(8)
	to	8	0.78	6/6/78(14)
	2010+00		0.75	12/4/78(20)
			0.59	6/5/79(26)
			0.70	6/25/80(38)

(Continued)

Table 2. (Continued).

S/A Ratio** and Mix Type	Station	Pavement Thickness, in.*	Maximum Dynaflect Deflection, 10-3 in.	Sampling Date (Age)***
0/6.2	2010+00		0.44	12/12/77(8)
	to	11	0.60	6/6/78(14)
	2015+00		0.44	12/4/78(20)
			0.40	6/5/79(26)
			0.44	6/25/80(38)

*All sections have 1 inch asphaltic concrete wear course and 8 inch lime treated subgrade.

**6.2/13 = weight percent of asphalt and sulfur in the paving mixture.

***Pavement age in months. 1 in. = 2.54 cm

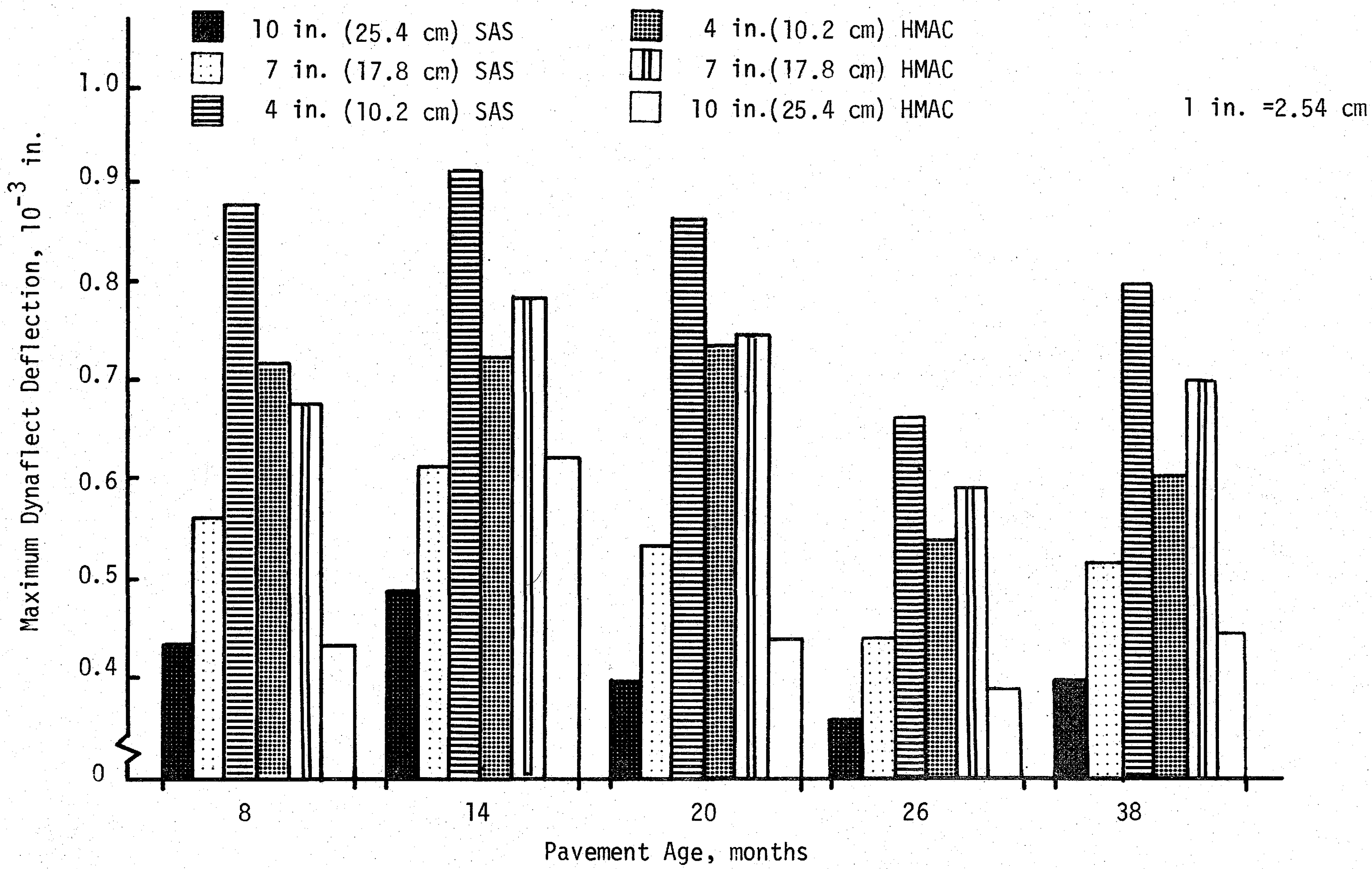


Figure 16. Maximum Dynaflect deflection versus pavement age for U. S. 77.

4.2.2 Visual Inspection

The pavement rating scores for the subsections are shown in Table 3. It must be emphasized that this evaluation is independent of the serviceability index and raveling. The serviceability index will be considered separately. Raveling is an indication of surface course performance. The surface course is not under consideration in this study. To date, very little visual distress has been noted in the test pavement.

The SDHPT had decided to use a hot-mix asphalt concrete base as the control in the segment where SAS was used as a trial section. The outside lane of this flexible base section began cracking so severely that in the summer of 1979, a chip seal was applied to the entire job including the HMAC and SAS sections. The covering of these sections was mutually agreed to by both the study supervisor and the SDHPT. Neither the HMAC or SAS sections needed a seal at the time as neither section showed any significant distress.

The SDHPT noted that there seems to be less cracking or distress as compared to the 1979 visual rating. The chip seal applied prior to June 5, 1979, seems to be very effective. In addition to some cracking, however, there is some slight rutting in the 7 and 10 in (17.8 and 25.4 cm) HMAC sections.

4.2.3 Cracking Survey

Figures 17 through 22 show the results of cracking surveys conducted on the subsections after the application of the chip seal in June, 1979. Of the cracking that has taken place in the 7 in (17.8 cm)

Table 3. Pavement Rating Scores Exclusive of Serviceability Index and Raveling for U.S. 77.

Station No.	Base Thickness and Type	Pavement Rating Score, Percent		
		12/78	6/79	6/80
1985+00 - 1990+00	10 in. (25.4 cm) SAS	100	100	100
1990+00 - 1995+00	7 in. (17.8 cm) SAS	91	95	83
1995+00 - 2000+00	4 in. (10.2 cm) SAS	91	100	95
2000+00 - 2005+00	4 in. (10.2 cm) HMAC	100	97	100
2005+00 - 2010+00	7 in. (17.8 cm) HMAC	91	93	92
2010+00 - 2015+00	10 in. (25.4 cm) HMAC	100	97	93

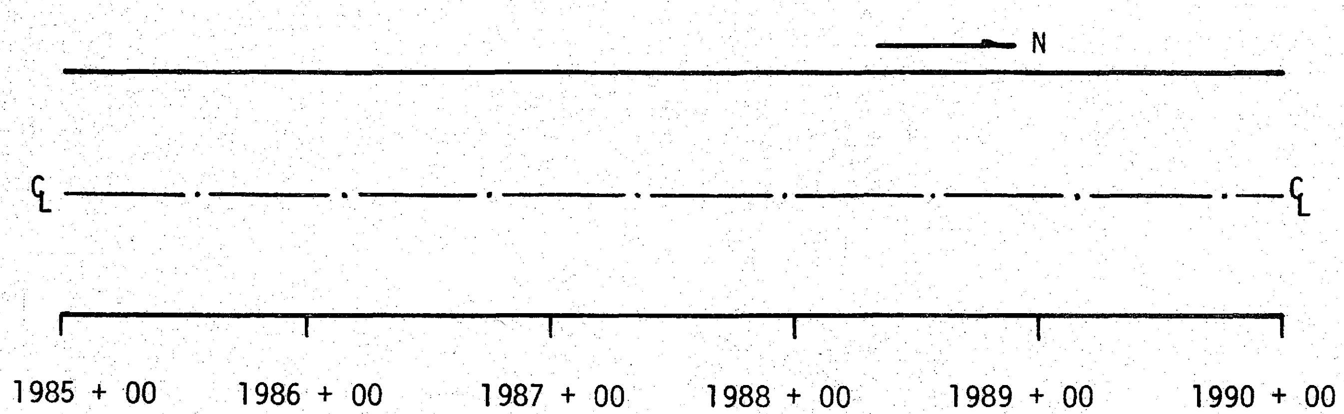


Figure 17. Results of cracking survey for 10 in. (254 cm) SAS base.

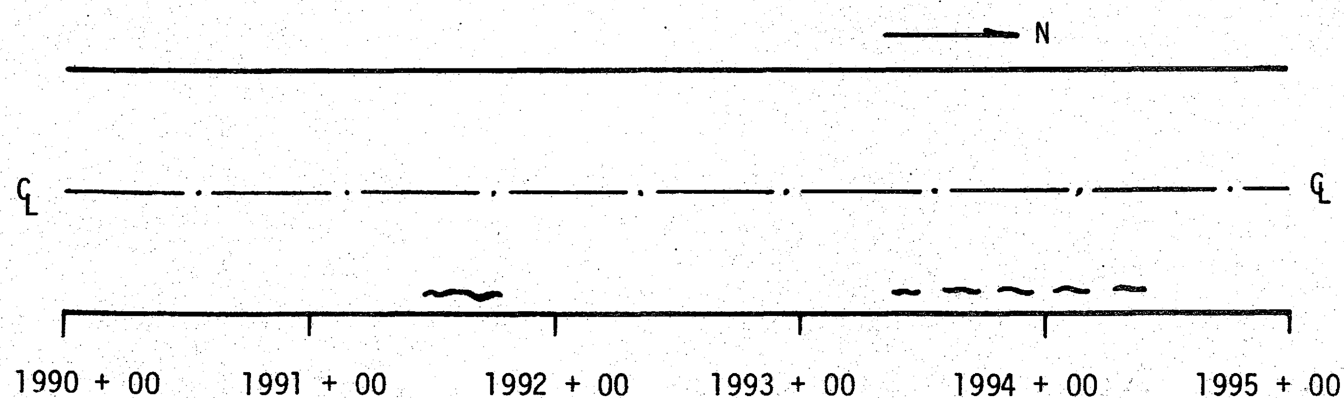


Figure 18. Results of cracking survey for 7 in. (17.8 cm) SAS base.

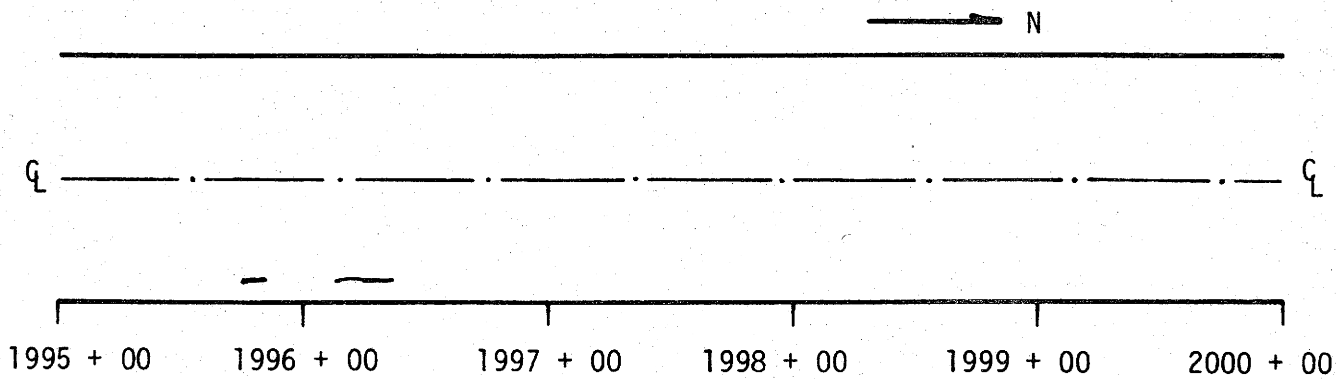


Figure 19. Results of Cracking Survey for 4 in. (10.2 cm) SAS Base.

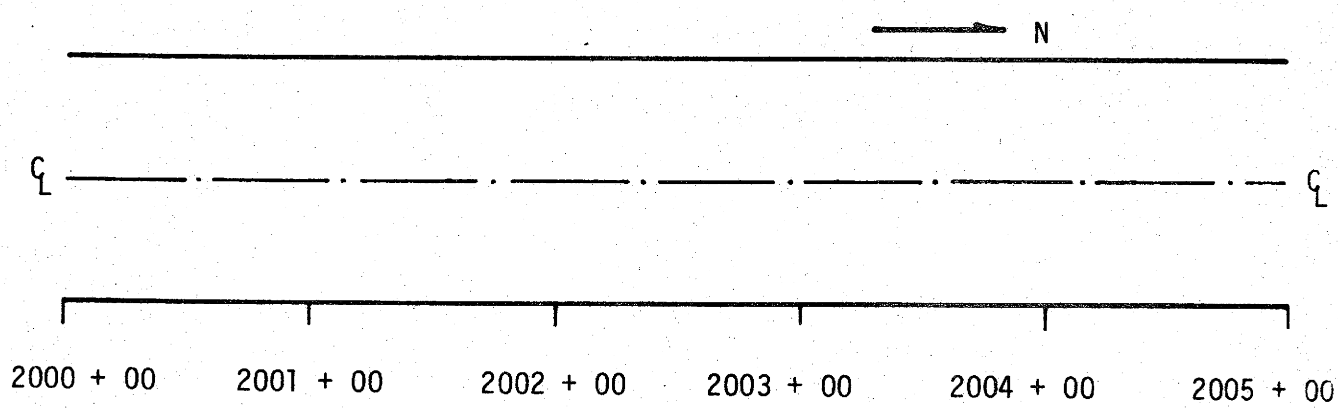


Figure 20. Results of Cracking Survey for 4 in. (10.2 cm) HMAC Base.

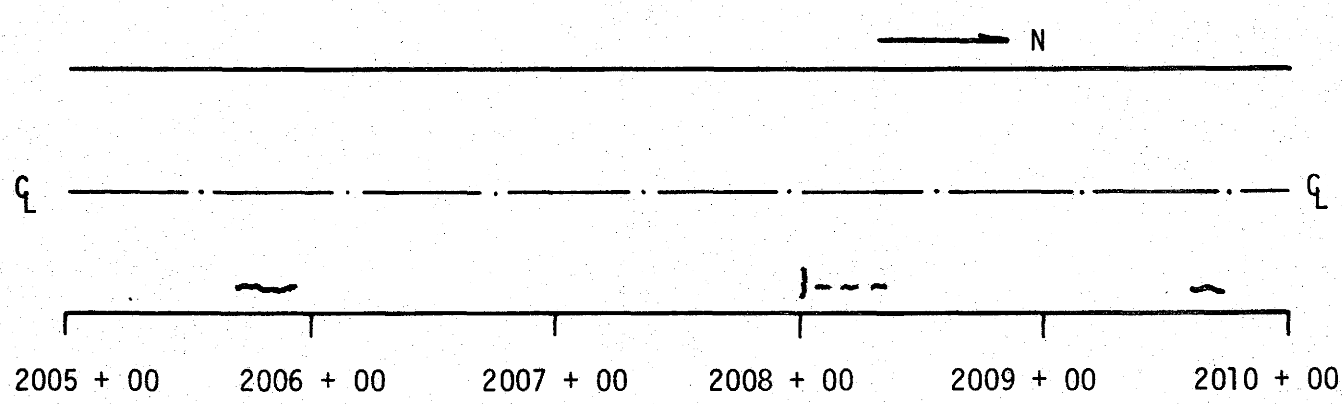


Figure 21. Results of cracking survey for 7 in. (17.8 cm) HMAC base.

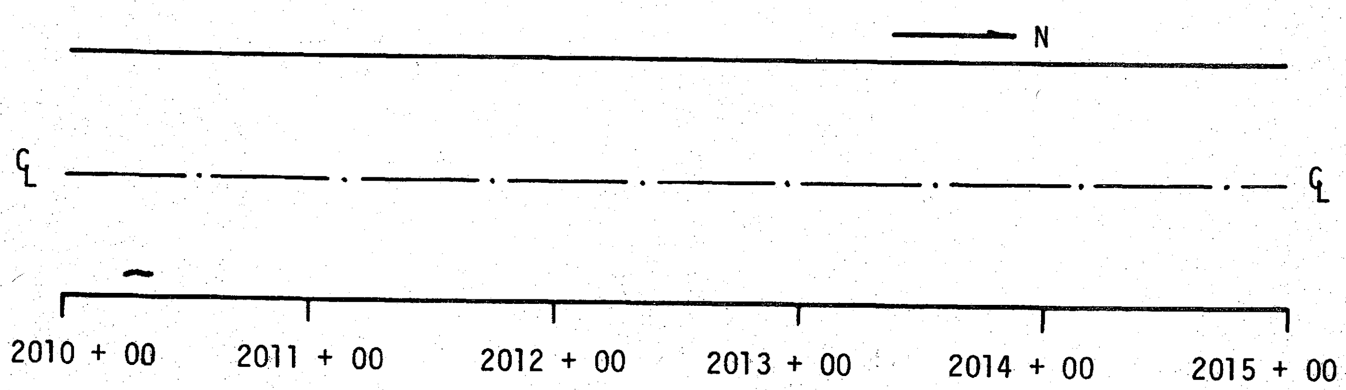


Figure 22. Results of cracking survey for 10 in. (25.4 cm) HMAC base.

and 4 in (10.2 cm) SAS test subsections, sample corings taken from these cracks have shown that the cracking has occurred only in the surface course. They are not reflection cracks from the base course material. Figure 23 shows this cracking in the surface course of the core sample taken in the 7 in (17.8 cm) SAS section.

4.2.4 Serviceability Index

The serviceability indices for the SAS and HMAC portions of the test section appear in Table 4. Readings were taken with the Mays Ride Meter vehicle straddling the wheel paths during all the testing periods. Beginning with the 18th month, readings were also taken with the wheels of the vehicle in the wheel paths. The readings which were taken with the vehicle straddling the wheel paths are questionable, since these are not truly indicative of the riding quality of the pavement. The majority of the road user vehicles stay in the wheel paths.

The readings which were taken in the wheel paths show that both the SAS and HMAC portions have a higher rating than those which were taken from straddling the wheel paths; however, both sections have decreased in riding quality at the 38th month. It can be noted that for all of the readings, the SAS portion generally has the lower serviceability indices than the HMAC portion. For the 38th month with the Mays Ride Meter vehicle wheels in the wheel paths, the SAS portion had an average serviceability index of 2.9 and the HMAC portion had an average of 3.9. The reasons for the poorer riding quality of the SAS sections are believed to be due to inadequate construction control. Specifically, the temperature control was marginal throughout the course of construction, the pug mill was out of adjustment during the period

Table 4. Mays Ride Meter Test Results for Road Serviceability Index.

Wheel Path No. 1*	Mays Ride Meter Readings Taken at 264 ft. (80.5 in.) Intervals										
6/15/77(0)**	3.1	3.8	2.8	3.2	2.8	2.9	3.1	3.3	3.2	3.7	3.7
11/15/77(6)	3.6	3.8	3.8	3.8	3.3	3.2	3.6	3.9	3.8	4.2	3.9
6/16/78(12)	3.6	3.9	3.8	3.9	3.6	3.8	4.0	4.1	4.1	4.4	4.4
12/28/78(18)	3.4	3.9	3.8	3.6	2.9	3.3	3.5	4.1	4.0	4.2	4.1
6/01/79(24)	3.4	4.0	3.8	3.7	3.4	3.4	3.8	4.4	4.1	4.1	4.3
6/26/80(36)	3.1	3.5	3.3	3.2	2.7	2.9	3.7	3.6	3.8	4.0	4.0
12/28/78(18)***	3.4	4.0	3.8	3.9	3.6	3.5	3.9	4.4	4.3	4.4	4.3
6/01/79(24)	3.7	4.0	3.9	3.9	3.8	3.6	3.9	4.2	4.3	4.4	4.3
6/26/80(36)	3.1	3.5	3.3	3.2	2.7	2.9	3.7	3.6	3.8	4.0	4.0
Wheel Path No. 2*											
6/15/77(0)	3.1	4.1	2.6	3.2	3.7	2.9	3.9	4.2	3.3	4.1	3.7
11/15/77(6)	3.3	4.1	3.6	4.2	4.1	3.8	4.5	4.4	4.1	4.5	4.5
6/16/78(12)	3.4	3.9	3.8	4.1	4.1	3.4	4.3	4.4	4.2	4.6	4.4
12/28/78(18)	2.8	3.6	3.9	4.0	3.9	3.6	4.1	3.9	3.4	4.0	3.9
6/01/79(24)	2.8	3.9	3.3	3.4	3.3	3.3	3.9	3.9	3.6	3.9	3.9
6/26/80(36)	2.7	4.0	3.7	3.7	3.4	2.7	3.5	3.2	2.9	3.9	4.0

(Continued)

Table 4. (Continued).

Wheel Path No. 3*	Mays Ride Meter Readings Taken at 264 ft. (80.5 in.) Intervals										
6/15/77(0)**	2.5	3.2	2.7	2.9	3.1	2.3	3.9	3.4	2.8	3.3	3.7
11/15/77(6)	3.0	3.9	3.5	4.1	3.8	3.0	4.5	4.0	4.0	4.2	4.5
6/16/78(12)	3.0	3.2	3.5	3.6	3.3	2.7	4.4	4.1	4.2	4.1	3.9
12/28/78(18)	2.6	3.7	3.4	3.3	3.4	3.0	3.8	3.7	4.0	3.9	3.8
6/01/79(24)	3.3	3.9	3.8	3.9	3.8	3.3	4.4	3.9	3.9	3.9	4.3
6/26/80(36)	2.6	2.7	2.9	2.9	3.0	2.7	4.1	3.6	3.8	3.7	3.4
12/28/78(18)***	3.0	3.4	3.8	3.0	3.1	2.8	4.5	4.2	4.1	4.3	4.1
6/01/79(24)	2.7	2.7	3.4	3.6	3.4	3.3	4.2	4.2	3.9	3.9	3.9
6/26/80(36)	2.3	2.6	3.2	2.7	3.0	2.3	4.1	3.5	3.9	3.7	3.5
Wheel Path No. 4*											
6/15/77(0)	2.5	2.9	2.9	2.5	2.9	2.1	3.6	3.9	3.7	3.1	-
11/15/77(6)	3.8	3.7	3.9	2.9	3.0	2.9	4.1	3.9	4.2	3.1	3.9
6/16/78(12)	3.6	3.6	3.6	2.8	2.9	2.7	3.9	3.9	4.1	4.1	3.9
12/28/78(18)	3.5	3.0	3.8	3.1	3.3	2.9	4.1	3.9	4.3	3.9	3.8
6/01/79(24)	3.5	3.0	3.6	2.8	2.8	2.6	3.9	3.6	3.7	3.5	3.6
6/26/80(36)	2.7	2.9	3.0	1.9	2.5	2.1	3.9	3.5	4.1	3.8	4.0

*Mays Ride Meter readings taken with vehicle straddling wheel paths.

**Pavement age in months.

***Mays Ride Meter readings taken with wheels in wheel paths.

Figure 23. Photograph of core taken from cracked area of 7 in. (17.8 cm) SAS section. Crack stops at SAS base material.

when the SAS mixture was being produced, and the laydown machine stopped and started frequently in the SAS portion causing a rippling effect in the surface.

5.0 Conclusions

At the close of the three year post construction evaluation period the SAS base sections have demonstrated consistently higher Marshall and Hveem stabilities than the HMAC base sections. On the other hand, the HMAC base sections have shown consistently greater resilient modulus and splitting tensile strengths than the SAS base sections. It appears that, generally speaking, the SAS material has demonstrated superior stability properties, yet lower strength and stiffness characteristics than the HMAC mixture.

Deflection measurements have increased with decreasing pavement thicknesses as would be expected. Outside of dynaflect deflection measurements, pavement thickness has not shown any influence on the laboratory test data or other performance test observations. The 4 in (10.7 cm) pavement sections designed to show distress in a two to three year period have not done so. The majority of cracking and rutting that has been observed has not occurred in the 4 in (10.2 cm) sections.

It can be stated that the SAS materials are performing comparably to the HMAC materials at the end of three years in service. In some instances (particularly with respect to rutting), it is superior to HMAC.

6.0 Recommendations

The results of the 3-year post-construction evaluation presented in this report do not permit any predictions to be made about projected service life of the test sections. Performance to date of the sand-asphalt-sulfur pavements have actually been better than originally anticipated as reflected in the excellent condition of the section which was designed to fail in two years.

Therefore, the recommendation has been made to the sponsors to continue this evaluation but, with less frequent samplings, for an additional 3-year period.

7.0 References

1. Izatt, J. O., Gallaway, B. M., Saylak, D., "Sand-Asphalt-Sulfur Pavement Field Trial, Highway U.S. 77, Kenedy County, Texas," A Construction Report, Federal Highway Administration Report FHWA-TS-78-204, April, 1977.

2. Saylak, D., Gallaway, B. M., "Post Construction Evaluation of U.S. 77 Kenedy County Sulfur-Asphalt-Sand Pavement Test Sections," Interim Report No. 1, October, 1977.

3. ASTM Designation: D 2041-71, 1977 Annual Book of ASTM Standards, Part 15, American Society for Testing and Materials.

4. ASTM Designation: D 2726-73, 1977 Annual Book of ASTM Standards, Part 15, American Society for Testing and Materials.

5. ASTM Designation: D 1559-76, 1977 Annual Book of ASTM Standards, Part 15, American Society for Testing and Materials.

6. "Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types," Asphalt Institute Manual Series, No. 2, The Asphalt Institute, March, 1974.

7. ASTM Designation: D 1560-76, 1977 Annual Book of ASTM Standards, Part 15, American Society for Testing and Materials.

8. "Operating Instructions, Mark III Resilient Modulus Device," Instruction Manual for Mark III Resilient Modulus Device, Retsina Company, El Cerrito, California.

9. Schmidt, R. J., "A Practical Method for Measuring the Resilient Modulus of Asphalt-Treated Mixes," Highway Research Record, No. 404, Highway Research Board, 1972.

10. Gallaway, B. M., and Saylak, D., "Evaluation of Sand-Asphalt-Sulfur Mixtures Under Repetitive Loading Conditions," Final Report on Texas A & M Research Foundation Project RF 983-1B, August, 1974.

11. Hadley, W. O., Hudson, R. W., and Kennedy, T. W., "Evaluation and Prediction of the Tensile Properties of Asphalt-Treated Materials," Highway Research Record, No. 351, Highway Research Board, 1971.

12. ASTM Designation: C 496-71, 1976 Annual Book of ASTM Standards, Part 14, American Society for Testing and Materials.

13. Yoder, E. J., and Witczok, M. W., Principles of Pavement Design, Second Edition, John Wiley and Sons, Inc., New York, 1975.

14. Walker, R. S., and Hudson, R. W., "A Correlation Study of the Mays Road Meter with the Surface Dynamics Profilometer," Research Report 156-1, Center for Highway Research, University of Texas at Austin, February, 1973.

15. Phillips, M. B., and Swift, G., "A Comparison of Four Roughness Measuring Systems," Research Report 32-10, Texas Transportation Institute, Texas A&M University, August, 1968.

16. Epps, J. A., Shaw, C. W., Harvey, G. G., Mahoney, J. P., and Scott, W. W., Jr., "Operational Characteristics of Mays Ride Meter," Research Report 151-3, Texas Transportation Institute, Texas A&M University, September, 1976.

17. Carey, W. N., Jr., and Irick, P. E., "The Pavement Serviceability-Performance Concept," Highway Research Record, No. 250, Highway Research Board, 1960.

18. Williamson, H. J., and Hudson, R. W., "A Study of the Relationships between Various Classes of Road-Surface Roughness and Human Ratings of Riding Quality," Research Report 156-5, Center for Highway Research, University of Texas at Austin, August, 1975.

19. Scrivner, F. H., Swift, G., and Moore, W. M., "A New Research Tool for Measuring Pavement Deflection," Highway Research Record, No. 129, Highway Research Board, 1966.

20. Poehl, R., and Scrivner, F. H., "Seasonal Variations of Pavement Deflections in Texas," Research Report No. 136-1, Texas Transportation Institute, Texas A&M University, January, 1971.

21. Anderson, D. I., "Improvement of Utah's Flexible Pavement Performance System," Report No. UDOT-MR-76-1, Utah Department of Transportation Materials and Research Section, March, 1976.

22. Gillott, J. E., Jordan, I. J., Loov, R. E., Skrive, N. G., and Ward, M. A., "Characteristics of Some Sulfur-Bonded Civil Engineering Materials," New Uses of Sulfur-II, Advances in Chemistry Series, No. 165, American Chemical Society, Washington, D.C., 1978.

23. Moore, W. H. and Swift, G., "A Technique for Measuring the Displacement Vector Throughout the Body of a Pavement Structure Subjected to Cyclic Loading," Texas Transportation Institute, Texas A&M University, August, 1971.