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16. Abstract

This study evaluates the basic mixture properties, fatigue potential, rutting or deformation potential of several river sands and blow sands from sources throughout Texas. The mixture characterization evaluates (1) resistance to lateral flow under vertical load by means of the resistance or R value; (2) moisture damage susceptibility as indicated by the percent moisture pick-up following soaking and as indicated by stability loss following vacuum saturation; and (3) air voids content. The Herrin (2) and Chevron (1) criteria for accepting aggregates as candidates for bituminous stabilization were evaluated based on the results of the mixture tests mentioned above as well as resilient modulus, fatigue, and permanent deformation testing.

The resilient moduli of the selected sands were determined versus temperature. These data were compared with resilient moduli data from other bituminous stabilized aggregates as well as other traditional base materials. Laboratory resilient moduli versus temperature were coupled with field determined moduli (using nondestructive deflection measuring techniques) from several locations to evaluate structural coefficients of the in situ bituminous stabilized marginal sands.

Two sands representing the range of properties evaluated were subjected to permanent deformation testing using the Shell Method (23). The factors of gradation, percent fines, binder content, weighted average annual air temperature, and traffic were considered.

Based on results of the testing discussed above, recommendations are made concerning the use of asphalt stabilized marginal sands.

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EVALUATION OF BITUMINOUS STABILIZED BLOW SANDS AND RIVER SANDS IN TEXAS

by

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and

Joe W. Button

Research Report No. 235-2F
Research Study Number 2-9-78-235
Economic Asphalt Treated Bases

Sponsored by

Texas State Department of Highways and Public Transportation
in cooperation with
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February 1986

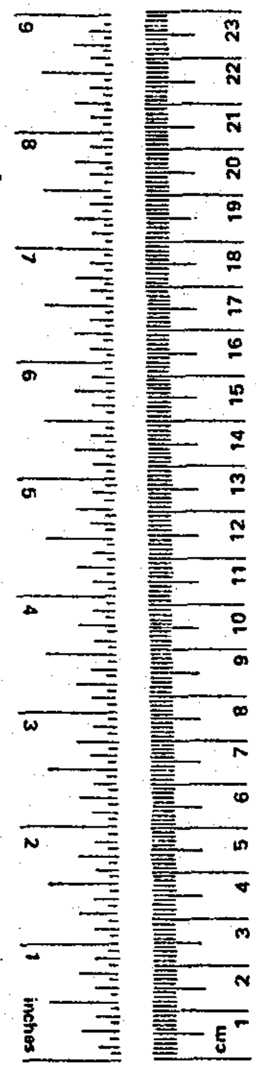
TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, Texas

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

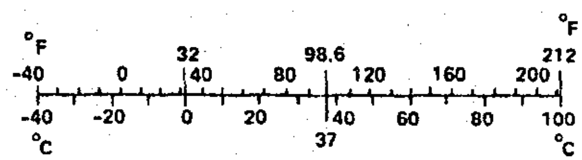
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The comments do not necessarily reflect the views or policies of the Federal Highway Administration. This report does not constitute a standard, specification or regulation.

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INTRODUCTION

The shortage of high quality aggregates together with increased traffic has created a need for treating local materials for use as base courses. In the southwest and midwest, sands transported by water or wind or both often are the only substantial aggregate source available. Asphalt has become a common base stabilizer for these marginal materials in the last few years. However, the criteria developed for materials selection and design and construction techniques have been based primarily on requirements developed for asphalt concrete surface courses. Thus, because of these "strict" requirements, materials evaluation and pavement design techniques are being used that significantly increase cost and provide a stabilized material whose properties are in excess of those required by traffic and the environment.

To provide an economical material to satisfy the particular requirements of asphalt base courses, current materials selection criteria and pavement design methods should be investigated and altered, if necessary. A first step in this direction is to define a number of interacting mixture properties. These include:

1. rheological characteristics,
2. fracture strength,
3. fatigue resistance, and
4. durability.

This report specifically treats emulsion stabilized sand bases and hot sand bases (and full-depth hot sand asphalt) in Texas.

Materials selection criteria and pavement design criteria to provide satisfactory performance in a given environment will be suggested based on:

1. characterization of laboratory molded mixtures,
2. characterization of field core samples, and
3. in situ structural evaluation of the asphalt stabilized sands.

The purpose of this study is to evaluate the potential of Texas sands stabilized with asphalt cement or asphalt emulsion to serve as a base course. Specific objectives are to evaluate the asphalt stabilized sands with respect to:

1. elastic or resilient deformation potential,
2. resistance to lateral flow,
3. permanent deformation
4. moisture susceptibility, and
5. fatigue potential.

Data have been collected from laboratory compacted sand asphalt mixtures, field cores of sand asphalts and in situ pavements. In order to evaluate the potential of these materials as a base course; the following properties were measured:

1. resistance value (following exposure to moisture),
2. resilient modulus,
3. air void content,
4. creep deformation, and
5. flexural fatigue.

MATERIALS

The aggregates listed in Table 1 were used in the laboratory molded specimens. Generally, these aggregates represent either silicious wind blown sands or silicious river deposited sands. Most of these sands are rather poorly graded.

Two criteria were used to evaluate the potential of these sands for bituminous stabilization: The Chevron (1)* U.S.A. criteria and the Herrin (2) criteria. These criteria are presented in Appendix C.

The Chevron criteria judge the acceptability of the sand for stabilization based on gradation, plasticity, sand equivalence, and resistance of the untreated material to lateral flow. Results of the evaluation with respect to these criteria are summarized in Table 2.

The Herrin criteria identifies an aggregate as being suitable for stabilization with asphalt as either a soil bitumen, sand bitumen, or sand-gravel bitumen mixture. Expected performance within each category is based on percent fines, plasticity index, and liquid limit. These results are also summarized in Table 2.

A few sand asphalt cores were tested in the laboratory for data comparison with results from laboratory compacted specimens. Aggregates used in these pavements are listed in Table 3. Because these are field cores, virgin aggregates were not available for laboratory testing. However, based on the locations of the cored specimens and the general aggregate descriptions, it was assumed that the aggregates comprising the cores are quite similar to the aggregates used in the laboratory molded specimens.

* The numbers in parentheses refer to the list of references to this paper.

Table 1. Aggregates used in the laboratory molded specimens.

Asphalt Type	Aggregate Designation	Gradation	Sand Equivalent	% Minus 200 Sieve	Liquid Limit	Plasticity Index	Description
Asphalt cement	District 5 (FM 168)[a]	poorly graded	41	2.8	21.0	0	high plains blow sand
	District 20 (U.S. 96)[b]	silty sand	48	13.8	22.8	5.8	East Texas sand
	District 21 (Beck Pit)	well graded	47	8.8	24.5	0	West Texas river sand
	District 25 (FM 3182)	poorly graded	41	3.9	20.3	0	West Texas river sand, gravel
Emulsified asphalt	District 5 (FM 168)	poorly graded	41	2.8	21.0	0	high plains blow sand
	District 11 (Gibson sand)	poorly graded	58	2.9	23.8	0	East Texas sand
	District 11 (FM 3736)	poorly graded	72	12.0	13.2	0	East Texas sand
	District 11 (Daniels Sand)	silty sand	20	28.6	22.0	0	East Texas sand
	District 16 (Padre Island)	poorly graded	98	1.8	24.0	0	Texas gulf beach sand
	District 20 (FM 255)	silty sand	32	15.3	18.3	0	East Texas Sand
	District 20 (SH 87)[c]	silty sand	89	26.4	22.5	2	East Texas sand
	District 25 (FM 3182)	poorly graded	41	3.9	20.3	0	West Texas river sand, gravel

[a] FM = farm to market road.
[b] US = U. S. highway.
[c] SH = state highway.

Table 2. Evaluation of potential for sands for use as asphalt stabilized base course.

Aggregate	Chevron Method				Herrin Method		
	Criteria				Criteria		
	R-Value	Sand Equivalent	Plasticity	Gradation	Soil Type[f]	Fines Content	Plasticity
District 5 (FM 168)[a]	no[d]	yes	yes	poor	soil bitumen	fair	fair
District 20 (U.S. 96)[b]	marginal	yes	marginal	silty sand	soil bitumen	good	fair
District 21 (Beck Pit)	good	yes	yes	well	sand gravel bitumen	. . .	. . .
District 25 (FM 3182)	marginal	yes	yes	poor	soil bitumen	good	fair
District 11 (Gibson)	yes[e]	yes	yes	poor	soil bitumen	good	fair
District 11 (FM 3736)	yes	yes	yes	poor	sand bitumen	. . .	. . .
District 11 (Daniels)	marginal	no	yes	silty sand	soil bitumen	poor	fair
District 16 (Padre Island)	no	yes	yes	poor	soil bitumen	fair	fair
District 20 (FM 255)	marginal	yes	yes	silty sand	soil bitumen	good	good
District 20 (SH 87)[c]	good	yes	yes	silty sand	soil bitumen	poor	fair

[a] FM indicates farm to market road.

[b] U.S. indicates U.S. highway.

[c] SH indicates state highway.

[d] No = fails to meet criteria.

[e] Yes = meets criteria.

[f] Soil Type: Soil bitumen is approximately a silty sand or poorly graded sand in the Chevron System. Sand gravel bitumen is generally a well graded soil in the Chevron System.

Table 3. Sand asphalt field cores

Designation	Aggregate
District 11 (SH 103)[a]	mixture of East Texas sands (poorly graded)
District 11 (Loop 287)	mixture of East Texas sands (poorly graded)
District 11 (Loop 287)	Gibson sand (poorly graded)
District 11 (Loop 207)	Bradley Pit sand (poorly graded)
District 20 (SH 87)	poorly graded subgrade sand
District 20 (SH 96)	sand used at hot mix plant site
District 20 (FM 255)[b]	subgrade sand (poorly graded) Layer B
District 20 (FM 255)	subgrade sand (poorly graded) Layer C
District 20 (FM 255)	poorly graded 100% sand

[a]SH indicates state highway

[b]FM indicates farm to market road

Laboratory Molded Specimens

Asphalt emulsion stabilized mixtures were tested according to the sequence shown in Figure 1a. The Chevron U.S.A. procedure (1) was followed throughout the testing sequence. Detailed procedures are given in Appendix C. Each aggregate was mixed with a cationic slow setting emulsion (CSS-1) conforming to criteria in ASTM Specification for Cationic Emulsified Asphalt (D 2397-79).

Sand asphalt mixtures were tested according to the sequence shown in Figure 1b. The asphalt used was an AC-10 supplied by the Exxon refinery in Baytown, Texas. All asphalt cement conformed to criteria in ASTM Specification for Viscosity-Graded Asphalt Cement for Use in Pavement Construction (D 3381-76).

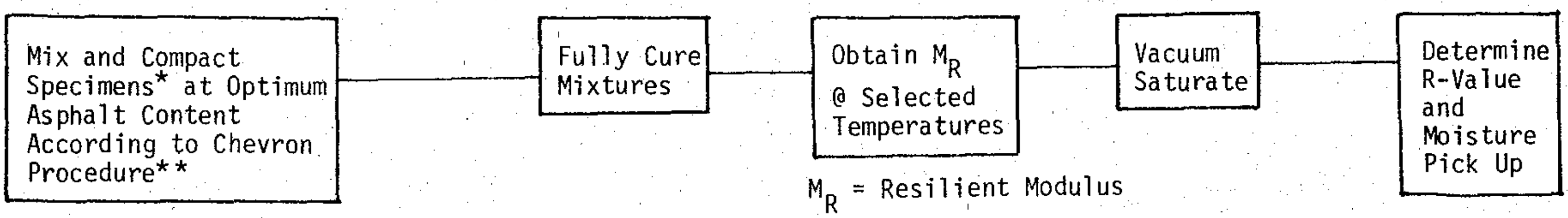
Field Cores

Field cores were obtained for testing on selected projects. Tests on these cores were performed primarily to verify or to compare with tests performed on laboratory molded specimens.

The following tests were performed on field cores:

1. diametral resilient modulus,
2. resistance value (R-value) following vacuum saturation, and
3. air voids content.

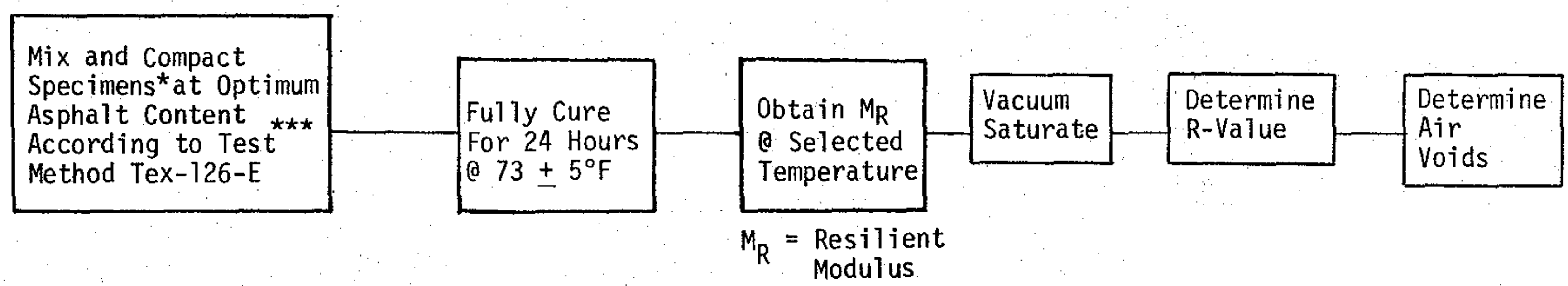
The diametral resilient modulus, R-value, and vacuum saturation testing followed procedures outlined in Appendix A of Reference 1.



*3 specimens were molded at each of 3 asphalt contents: Optimum, Opt. + 1% and Opt - 1%.

**The Chevron U.S.A. procedure was followed throughout this test sequence.

Figure 1a. Asphaltic Emulsion Mixture Test Sequence.



***Manual of testing procedures, Texas State Department of Highways and Public Transportation, Volume 1, 1974.

Figure 1b. Asphalt Cement Mixture Test Sequence.

TESTS RESULTS ON LABORATORY MOLDED SPECIMENS

Asphalt Cement Stabilized Sands

Laboratory molded hot sand-asphalt mixtures exhibited comparatively low R-values. This is particularly true of the District 5 and District 25 sands. Gradations of these sands are shown in Table 4. Laboratory test results reflect the poor gradation of these two blow sands. The District 20 silty sand is also a poor candidate for a high stability mix due to its poor gradation (Table 4).

The Chevron and Asphalt Institute emulsion mix criteria were used to evaluate the sand-asphalt mixes. Although several states and agencies have adopted sand-asphalt criteria, these criteria are generally derived from surface hot-mix specifications and thus require low air void contents (less than 6 percent). Based on the sand-asphalt air voids criteria, the sands evaluated in this study would not be suitable.

Table 5 shows that only the District 21 material meets the Chevron (1) and Asphalt Institute (3) criteria established for emulsion stabilized mixtures that requires a minimum R-value of 78 after vacuum saturation and a maximum of 5 percent moisture pick-up. Moisture pick up is the increase in weight of the specimen as a percentage of the dry weight of the specimen (1). The District 20 silty sand is marginal, and the poorly graded blow sands from Districts 5 and 25 are clearly substandard based on the Chevron or Asphalt Institute criteria.

Excessive moisture pick-up and low stabilities following vacuum saturation is in part a result of high air void content of the poorly graded sands.

Table 4. Properties of aggregates used in laboratory mixes.

	District 5 (FM 168)	District 20 (U.S. 96)	District 21 (Beck Pit)	District 25 (FM 3182)	District 11 (Gibson)	District 11 (FM 3736)	District 11 (Daniels)	District 16 (Padre Island)	District 20 (FM 255)	District 20 (U.S. 87)
Sieve Sizes	Percent retained									
1 in.[a]	. . .	. . .	16.8	. . .	. . .	. . .	. . .	. . .	. . .	. . .
3/4 in.	. . .	. . .	28.0	. . .	. . .	. . .	. . .	. . .	. . .	2.7
3/8 in.	. . .	0.6	35.0	. . .	. . .	. . .	. . .	. . .	. . .	3.7
No. 4	. . .	1.4	50.3	. . .	. . .	. . .	0.3	. . .	. . .	5.0
No. 8	. . .	2.5	61.3	. . .	0.1	0.2	0.7	. . .	0.1	12.5
No. 10	. . .	2.7	63.3	0.1	0.1	0.7	0.7	. . .	0.1	19.3
No. 16	0.01	5.5	67.8	0.3	0.1	11.5	0.8	0.1	0.1	20.4
No. 30	0.1	17.8	71.9	3.4	0.8	56.2	1.8	0.2	2.2	24.5
No. 40	0.3	33.6	73.6	11.6	11.7	67.0	6.1	0.4	7.0	29.1
No. 50	2.8	58.4	76.2	44.3	49.3	73.6	19.5	0.5	27.4	30.9
No. 60	26.0	71.5	79.2	59.4	73.1	77.8	33.8	0.8	41.6	33.0
No. 80	73.7	81.8	84.7	67.9	91.6	83.2	48.6	26.7	69.7	37.1
No. 100	86.8	83.8	87.0	76.8	94.0	84.9	54.0	58.9	75.0	63.7
No. 200	97.2	86.2	91.8	96.1	97.1	88.0	71.4	98.2	84.7	73.6
Sand equivalent	41	48	47	41	58	72	20	98	32	88.6
Fines modulus	0.90	1.69	4.14	1.25	1.44	2.26	0.77	0.60	1.05	39
Plastic index	0	5.8	0	0	0	0	0	0	0	1.92
Liquid limit	21	22.8	24.5	20.3	23.8	13.2	22.0	24.8	18.3	22.5
Plastic limit	NP	17.0	NP	NP	NP	NP	NP	NP	NP	NP

[a]1 in. = 25.4 mm.

[b]NP = non plastic.

Table 5. Summary of stabilities and moisture pick-up of laboratory molded asphalt cement stabilized sands.

Aggregate	Optimum[a] % AC Used	R-Value After Vacuum Saturation	Air Voids, percent	Moisture Pick-up, percent
District 5 (FM 168)	6	too weak to test	19	11
District 20 (U.S. 96)	6	80	15	6
District 21 (Beck Pit)	6	80	3	4
District 25 (FM 3182)	6	70	18	10

[a]All tests were performed on mixtures containing 4, 5 and 6 percent AC-10. The optimum AC percentage is based on the highest R-value and lowest moisture pick-up.

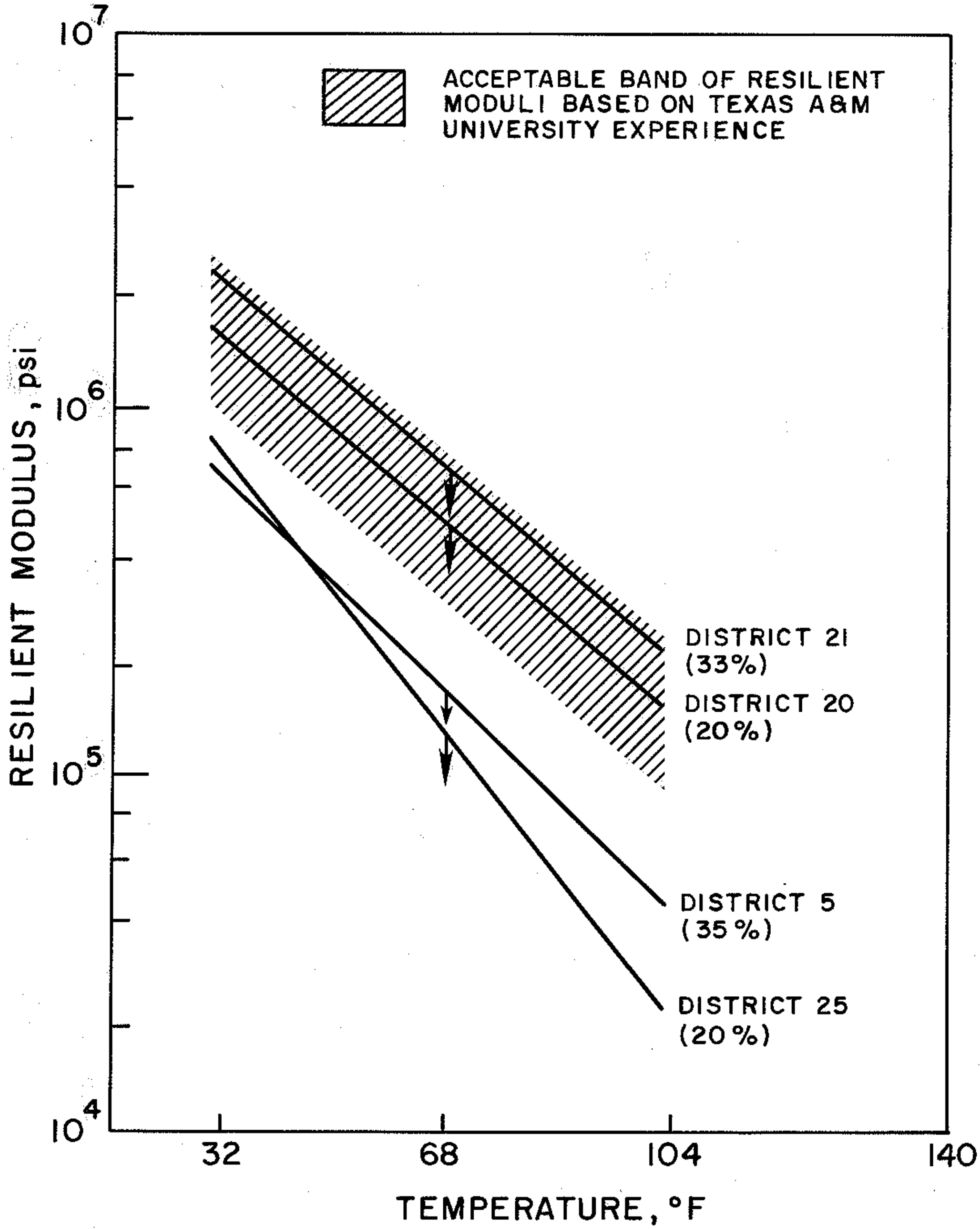
A criteria for mix design based on resilient modulus versus temperature relationships does not exist. However, researchers at Texas A&M have been collecting diametral resilient modulus versus temperatures data for several years on various types of asphalt mixtures (both laboratory molded and field cores). These data have been compared to field performance data.

Figure 2 illustrates the band of resilient moduli versus temperature selected as an acceptable range for performance of asphalt treated bases and full-depth asphalt sections in Texas. Resilient modulus versus temperature for the asphalt cement stabilized fine sands is shown by the solid lines. The length of the arrows represents the magnitude of decrease in resilient modulus at 68°F after vacuum saturation.

Resilient moduli of Districts 20 and 21 asphalt stabilized materials are within the acceptable band. Those of Districts 5 and 25 mixtures are below this band.

Based on R-values before and after moisture conditioning and resilient modulus testing, these previous data indicate that hot asphalt stabilized materials from District 20 and 21 have the potential to perform as satisfactory base materials. Poorly graded blow sands with a very small fines fraction exhibited very low stabilities, high air voids, excessive moisture pick-up, and low resilient properties.

Quality of paving mixtures seems to depend on gradation. Poorly graded sands with low minus 200 sieve fraction should be used with caution.



*Indicates percentage drop in resilient modulus following vacuum saturation (at 68^{0}F). This is depicted graphically by the arrow.

Figure 2. Resilient modulus - temperature relationship for asphalt cement stabilized sands.

Emulsion Stabilized Sands

All of the emulsion stabilized sands evaluated are poorly graded (Table 1). The results of the poor gradation is a high air void content resulting in a relatively high moisture pick-up.

Table 6 summarizes the R-value stability and moisture pick-up data from the sand-emulsion mixes. The R-values of the wind blown sand (District 5) and the Padre Island beach sand (District 16) were unacceptable according to Chevron criteria (1) (lower than 78). Excessive moisture pick-up is evident for the mixtures using the District 5 blow sand, the District 25 blow sand, the District 16 near beach sand, and the District 11 (Gibson) sand. All other mixtures recorded moisture increases only slightly greater than the criterion of 5 percent moisture pick-up by weight of the dry sample.

Generally, the poorly graded East Texas river sands from Districts 11 and 20 are acceptable in terms of R-value and marginal in terms of moisture pick-up.

Figure 3 is a plot of resilient modulus data at 73°F. These resilient moduli compare favorably with the band of resilient moduli values plotted on Figure 3. This band represents a range of resilient moduli at 73°F for mixtures that have performed acceptably in the field as bases or full-depth asphalt pavements (4-6).

Based on Hveem (R-value) stability, moisture susceptibility, and resilient modulus test results, emulsion stabilized sands studied are potentially suitable for base course layers.

Table 6. Summary of stability and moisture pick-up of laboratory molded emulsion stabilized specimens.

Aggregate	Optimum % Asphaltic Emulsion	R-Value After Vacuum Saturation	Air Voids, %	Moisture Pick-Up, %
District 5 (FM 168)	10	61	25	15
District 11 (Gibson)	11	78	24	16
District 11 (FM 3736)	11	80	10	3
District 11 (Daniels)	11	81	11	5
District 16 (Padre Island)	12	68	27	15
District 20 (FM 255)	12	89	12	7
District 20 (SH 87)	10	86	12	5
District 25 (FM 3182)	8	78	19	11

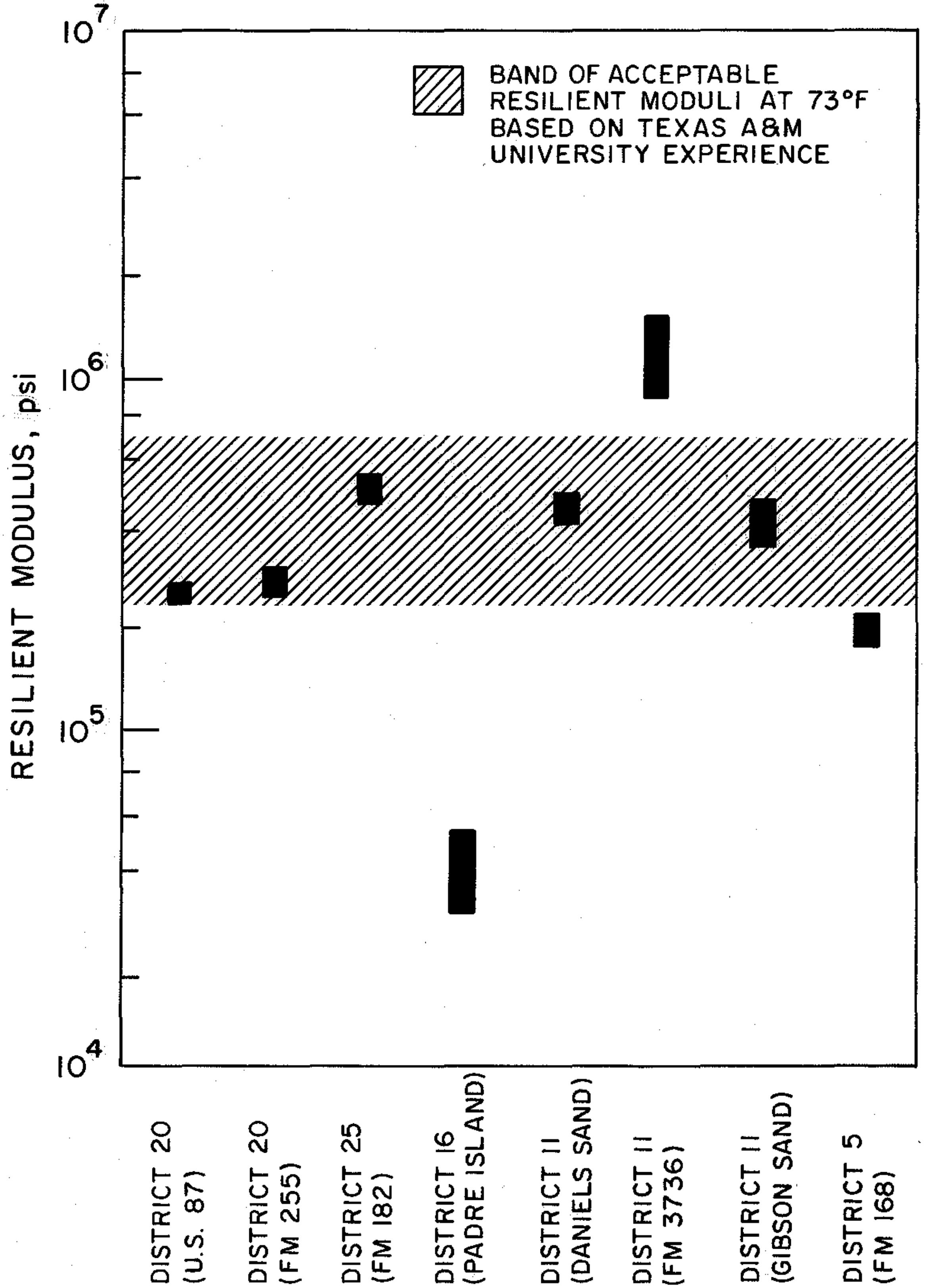


Figure 3. Resilient moduli at 73^{O}F for asphaltic emulsion stabilized sands.

TEST RESULTS ON FIELD CORES

Field cores from Districts 11 and 20 were tested for resistance to lateral flow (R-value) and moisture pick-up following vacuum saturation. These data are summarized in Table 7. Except for those from District 20 (U. S. Highway 96), all cores meet the minimum requirements of 78 for the R-value following vacuum saturation. A description of the aggregates in each core is in Table 3.

The fact that the air void contents for both laboratory specimens and field cores are of the same order of magnitude indicates that the laboratory compactive effort is comparable to that used in the field.

As in the laboratory molded cores, the percent moisture pick-up is above the recommended maximum of 5 percent. This once again indicates that this criterion may be too stringent for sand mixtures because of the inherent high air void content. One suggestion for a moisture pick-up criterion for sand mixtures is to establish an acceptable level based on a relationship to the existing air voids in a compacted mixture.

The resilient moduli of the field cores at 73ºF is compared to the resilient moduli of other base materials typically used in Texas in Figure 4. All of the cores exhibited a modulus in the range of other quality base materials. This indicates their ability to function satisfactorily in a pavement system under short duration dynamic loading.

Table 7. Summary of stabilities and moisture pick-up of field cores.

Aggregate	Asphalt	R-Value After Vacuum Saturation	Air Voids, percent	Moisture Pick Up, percent
District 11 (SH 103)	AC-10	85	16	6
District 11 (LP 287)	AC-10	81	22	11
District 11 (LP 287)	AC-10	90	21	9
District 11 (Loop 207)	AC-10	90	18	7
District 20 (SH 87)	asphalt emulsion	78	16	7
District 20 (SH 96)	AC-10	60	16	6
District 20 (FM 255)	asphalt emulsion	80	20	8
District 20 (FM 255)	asphalt emulsion	79	21	9
District 20 (FM 255)	asphalt emulsion	78	20	10

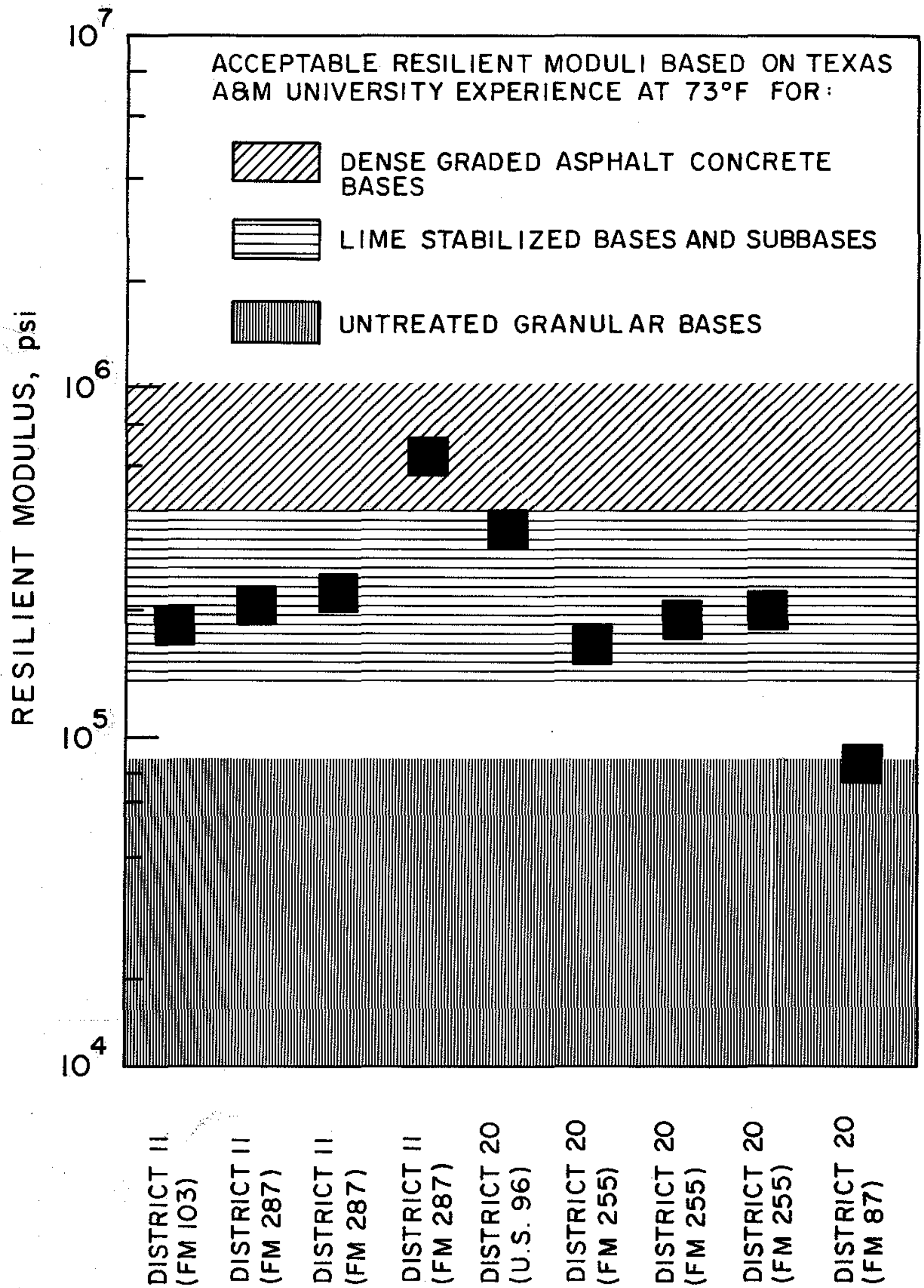


Figure 4. Resilient moduli at 73^0F for field cored asphalt stabilized sands.

STRUCTURAL EVALUATION

Laboratory derived resilient modulus versus temperature data were used together with layered elastic computer modeling to evaluate the structural potential of selected sand asphalt and asphalt emulsion stabilized sand bases.

Accurate determination of the resilient modulus is important in evaluating pavement structures by layered elastic modeling. In addition, temperature susceptibility of asphalt bound materials makes it necessary to evaluate the relationship between resilient modulus and temperature.

In order to establish a credible relationship between resilient modulus and temperature, both laboratory testing and in situ testing were used. Diametral resilient moduli were determined at 40, 68, 77, and 104°F to establish the resilient modulus-temperature relationship. In addition, the Falling Weight Deflectometer and the Dyanflect were used to evaluate the in situ dynamic modulus of the pavement layer in question. Thus, laboratory and in situ readings were used concurrently to establish a resilient modulus versus temperature relationship.

In Situ Modulus Determination

Discussion of the in situ resilient modulus determination is divided into two parts: (1) Falling Weight Deflectometer (FWD) and (2) Dynaflect.

The FWD is a nondestructive pavement tester that simulates the effects of a fast moving wheel load. It is capable of a load range of 3 to 12 kips, an associated loading time of 26 ms (milliseconds) and vertical deflections that may be taken at any desired position from the center of the loading plate outward along the deflection basin. Several European research projects (7, 8) have compared the deflection, stresses, and strains of the FWD with those caused by moving wheel loads. Correspondence of the two was remarkably good (within 10 percent) (5).

Assuming that the elastic moduli of materials may be derived from deflection tests, it is believed that the FWD stands a better chance of representing actual wheel loads than most other available steady-state loading systems. Using the concept of establishing pavement layer moduli by deflection basin matching, the modified linear elastic program ELSYM 5, was used to match the FWD basin by an iterative procedure (9).

The ELSYM 5 program was modified so that variable stiffnesses depending on the state of stress under each corresponding deflection sensor could be used for calculating the total deflection. The individual solution is valid for that particular deflection position only.

The ELSYM 5 program, using principals of the methods of equivalent thicknesses as well as Bouissinesq's equation, has been streamlined into a iterative procedure through which unique solutions can be obtained quickly. The resulting program is called ISSEM 4 (9). The obvious advantage of the FWD, together with the analysis package, is that it allows one to vary load or stress level and to evaluate the in situ elastic response to each layer in the pavement system as a function of stress level.

The Dynaflect was also used to evaluate the in situ elastic moduli of several sand asphalt cement and asphaltic emulsion stabilized materials. The Dynaflect applies a sinusoidal load of 1000 pounds amplitude at a frequency of 8 Hz on two steel wheels 20 inches apart in contact with the pavement. This is relatively low when compared to the FWD and is a problem in evaluating layers whose elastic response is highly stress dependent. A second noteworthy limitation of the Dynaflect is that the magnitude of the load is fixed and therefore stress sensitivity cannot be suitably evaluated within layers of the pavement structure.

However, like the FWD, the Dynaflect has the capacity to measure the deflection basin developed by the 1000 pound sinusoidal load. This is done by geophones located at 10, 15.6, 26, 37.4 and 49 inches from the two load points. Accelerometers record acceleration and integrate it twice to determine surface deflection.

Previous research at Texas Transportation Institute (TTI) (10-12) indicates several ways to use the Dynaflect deflection basin to evaluate in situ elastic moduli. All of these methods are based on deflection basin matching by means of layered elastic analysis. The method selected for this analysis has been used at TTI in comparative evaluations of recycled pavements, sulfur extended asphalt pavements, and conventional pavement sections. This method is based on a modification of Vaswani's procedure (13) in which the dual parameters of maximum Dynaflect deflection, d_{max}, and spreadability, S, (the ratio of the average deflection within the basin to d_{max}) are evaluated to graphically determine an effective thickness of pavement above the subgrade.

Dual parametric charts were developed specifically for the Dynaflect load configuration using the layered elastic computer program

BISTRO (14). Once the loci of d_{max} and S are plotted on dual parametric charts for a given pavement, the subgrade elastic modulus and effective thickness for a selected composite modulus of the structural pavement (above the semi-infinite subgrade) can be determined. If several charts are developed reflecting different composite pavement moduli, the composite (or weighted average) modulus of the pavement in question may be evaluated by knowning the pavement's cross-sectional thickness. Such a procedure was first developed by Vaswani (13).

Since the dual parametric procedure gives only a composite or weighted average modulus of all layers above the subgrade, it may be ineffective in selecting a specific layer in a multilayered structure. Fortunately, in this analysis the layer in question was either the only layer of structural significance or one of only two structurally significant layers. Thus, the in situ elastic response could be effectively evaluated for the sand-asphalt cement and asphalt emulsion stabilized sands.

Matching Laboratory and In Situ Data

The diametral resilient modulus device developed by Schmidt (15) was used to evaluate the resilient modulus versus temperature relationship for laboratory tested field cores. The in situ measured modulus at the pavement temperature at the time of evaluation was plotted on the same chart, and the laboratory curve was shifted maintaining the same slope (indicating the modulus sensitivity to temperature) to pass through the in situ measurement. Pavement temperature at the time of in situ evaluation was computed by The Asphalt Institute procedure (16). Figure 5 illustrates this procedure for U. S. Highway 96. The in situ modulus closely approximated the laboratory curve in each case.

The procedure just discussed was used for each pavement evaluated in situ to develop average annual elastic moduli for the respective climatic conditions of each pavement. These moduli were used, as will be explained in the following section, to develop layer structural coefficients for hot sands and emulsion stabilized sand bases evaluated.

STRUCTURAL COEFFICIENTS

The AASHTO Interim Guide for Flexible Pavement Design is based on the experience of a factorial road test experiment. The concept of structural layer coefficients, a_i, is familiar to most pavement engineers as a relative measure of the performance of a given material in a given position within the pavement sturcture.

Structural coefficients are actually regression coefficients that describe the contribution of the material and layer in question to the total pavement structure. As one might expect, these coefficients are highly sensitive to the interactions within the pavement structure.

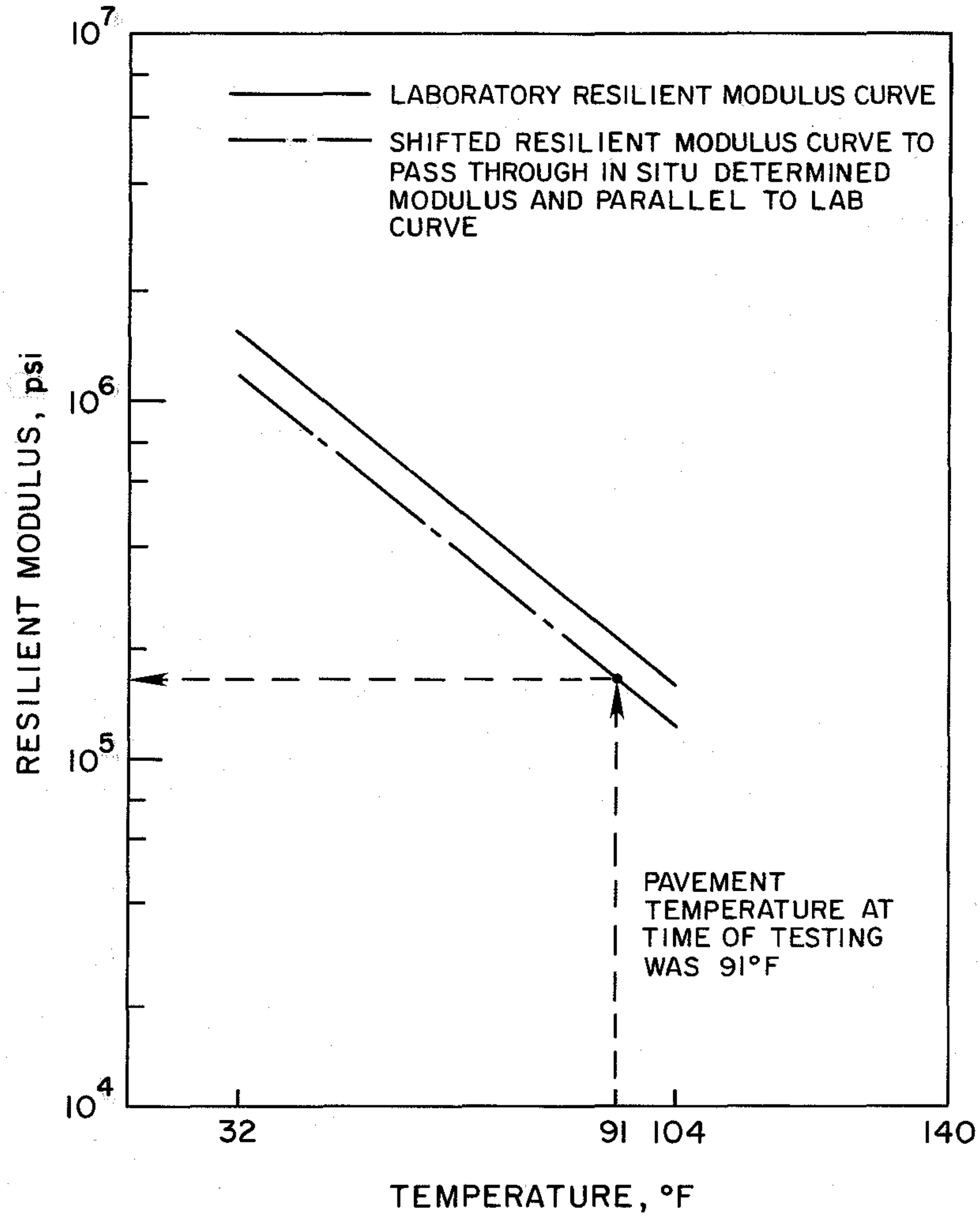


Figure 5. Procedure for shifting the laboratory resilient modulus versus temperature curve to reflect in situ data.

Indeed these coefficients are not unique material properties but are a function of temperature, pavement structural geometry, interdependency of structural layers, load intensity, etc.

Clearly, any design parameter that is so sensitive to so many variables is quite limited as a design parameter. Despite the limitations, the structural coefficients can be effectively used as a comparative performance index. It is presented as such in this paper.

Previous research (11, 17, 18) has shown that the structural coefficient for base course material is highly correlated to the temperature versus stiffness relationships of base material and the design layer thickness of the base. Furthermore, previous research has also shown that the single most significant parameter associated with the fundamental AASHTO flexible pavement performance equation is subgrade deformation, W_s. This was verified by extensive regression analyses using the original AASHTO data and by evaluating other mechanistic parameters such as tensile strain at the bottom of the asphalt bound layers and vertical compressive strains within layers and at the top of the subgrade (11, 17, 18). It is therefore possible to compute an AASHTO structural coefficient for a given material by:

1. evaluating the stiffness versus temperature relationship of the material,

2. evaluating the performance parameter of subgrade deformation by layered elastic analyses, and

3. selecting a structural coefficient based on the relationship between performance life and subgrade deformation.

Figure 6 presents a chart by which the structural layer coefficient

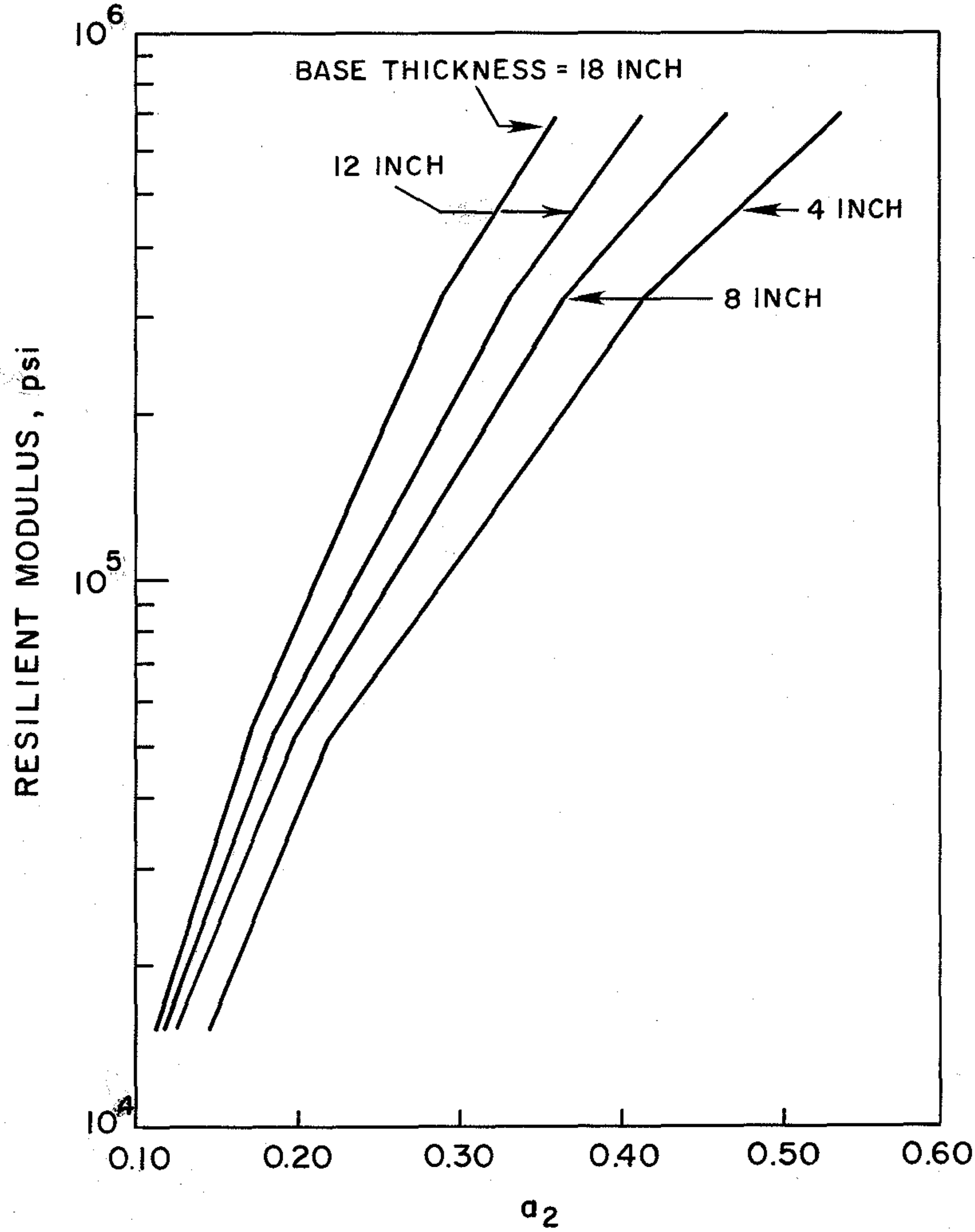


Figure 6. Relationship between structural coefficient, a_2, and average annual resilient modulus.

of a base course can be determined based on temperature versus elastic modulus, resilient modulus, or dynamic modulus data. Note that the structural coefficient, a_2, is dependent not only on the modulus (in this case resilient modulus, M_R) but also the base course thickness, h_2.

To develop Figure 6, the stress sensitive layered elastic computer program PSAD2A (19) was used to model 27 pavement sections from Loop 4 of the AASHTO Road Test. Materials comprising these sections had been previously characterized. Asphalt concrete layers were characterized in terms of dynamic modulus versus temperature while the untreated base, subbase, and subgrade materials were characterized in terms of resilient modulus as a function of stress. A W_s computed from PSAD2A for each section was then regressed against the number of design loads (18 kip single axle) to a selected terminal serviceability (P_t = 2.5) for each of the 27 sections.

Once a relationship between pavement life, in terms of design load applications to a selected serviceability level, and W_s was established, structural coefficients of bases of different resilient moduli from those used in Loop 4 of the Road Test could be determined. This was done by substituting the sand asphalt moduli versus temperature relationships for those of the unbound bases used at the Road Test and computing W_s. A performance life, N_t, in terms of number of design load applications was determined from the regression equation. Finally, with the N_t known, a_2 could be calculated from the fundamental flexible AASHTO design equation

$$\log N_t = \log \rho + G_t/\beta$$

where

N_t = number of design load applications,

ρ = a function of design load and the structural coefficients,

β = a function of design load and the structural coefficients, and

G_t = a function of the selected terminal serviceability.

This evaluation is based on the potential of the subgrade to distribute stresses in the elastic range. The asphalt stabilized sands appear to be well suited to adequately distribute these stresses and thus to protect the subgrade. However, their resistance to all important permanent deformation and thermal cracking is not a part of this analysis.

The potential of these asphalt stabilized sands to resist permanent deformation will be discussed subsequently.

Pavements and asphalt stabilized sand bases evaluated by laboratory resilient modulus testing and in situ deflection testing are given in Table 8.

FM 842 and FM 2680 contain hot sand bases that were evaluated in situ by the FWD. SH 6 and U. S. Highway 84 contain emulsion stabilized limestone bases and were similarly evaluated. SH 6 and U. S. Highway 84 are included for comparative purposes as they represent asphalt stabilized aggregate bases of accepted good quality in Texas.

Structural coefficients and elastic moduli from which these coefficients were evaluated are listed in Table 9. The elastic moduli represent the weighted annual average value for each respective location based on the procedure described in the preceeding section.

The sands stabilized with asphalt cement as well as those stabilized

Table 8. Pavement bases tested by in situ deflection and laboratory resilient modulus.

Pavement	Type Base	Pavement Description
FM 842[a] Lufkin, Tex.	AC-10 stabilized sand	6-7 inches of hot sand base over 6 inches lime stabilized subbase
FM 2680 Lufkin, Tex.	AC-10 stabilized sand	seal coat over 6.5 inches of hot sand base and 12 inches of select materials
SH 110[b] Smith Co., Tex.	AC-20 stabilized sand	1.5 inches HMAC over 8 inches asphalt stabilized sand base
U.S. 96[c] Jasper Co., Tex.	emulsified asphalt stabilized sand (plant mix)	seal coat over 6-10 inches emulsion stabilized sand and 6 inches of lime stabilized subbase
FM 255 Jasper co., Tex.	emulsified asphalt stabilized sand (plant mix)	seal coat over 8 inches of emulsion stabilzed sand base and 12 inches of select material
FM 1632 Tyler Co., Tex.	emulsified asphalt (road mix)	seal coat over 8 inches of emulsion stabilized sand base
SH 6 Waco, Tex.	emulsified asphalt stabilized limestone	1.5 inches HMAC over 4 inches emulsion stabilized limestone base and 12 inches of select material
U.S. 84 Waco, Tex.	emulsified asphalt stabilized limestone	1.5 inches HMAC over 2 inches emulsion base and 8 inches of gravel base

[a]FM indicates farm to market road.

[b]SH indicates state highway.

[c]U.S. indicates U.S. highway.

Table 9. Average annual elastic moduli and structural coefficients.

Pavement	Stabilizer	Aggregate	E_{avg}, psi	a_2[a]
FM 842[b]	AC-10	poorly graded sand	120,000	0.26
FM 2680	AC-10	poorly graded sand	240,000	0.33
SH 110[c]	AC-20	well graded sand	160,000	0.29
U.S. 96[d]	emulsion	silty gravel	220,000	0.32
FM 255	emulsion	silty sand	180,000	0.26
FM 1632	emulsion	poorly graded sand	140,000	0.28
SH 6	emulsion	limestone	250,000	0.34
U.S. 84	emulsion	limestone	180,000	0.30

[a]Based on 8 to 10 in. base thickness.

[b]FM indicates farm to market road.

[c]SH indicates state highway.

[d]U.S. indicates U.S. highway.

Note: Structural coefficients from AASHTO Road Test data for bituminous treated bases are:

0.34 - coarse graded
0.30 - sand asphalt

with asphalt emulsions possess structural coefficients, a_2, in the range of 0.26 to 0.33. This compares favorably with structural coefficient established at the AASHTO Road Test of 0.30 for asphalt stabilized sands. In addition, the coefficients of the emulsion stabilized limestone for SH 6 and U.S. Highway 84 serve as a comparative standard by which to evaluate the asphalt stabilized sands as well as the analytical procedure.

If one compares the structural coefficients in Table 9 to those developed at the AASHTO Road Test for structural base layers, the Texas asphalt stabilized sands rank as viable alternatives based on the criteria of subgrade protection. For example, the structural coefficients in Table 9 are generally significantly superior to those for the lime, cement, and untreated bases studied at the AASHTO Road Test.

PERMANENT DEFORMATION EVALUATION

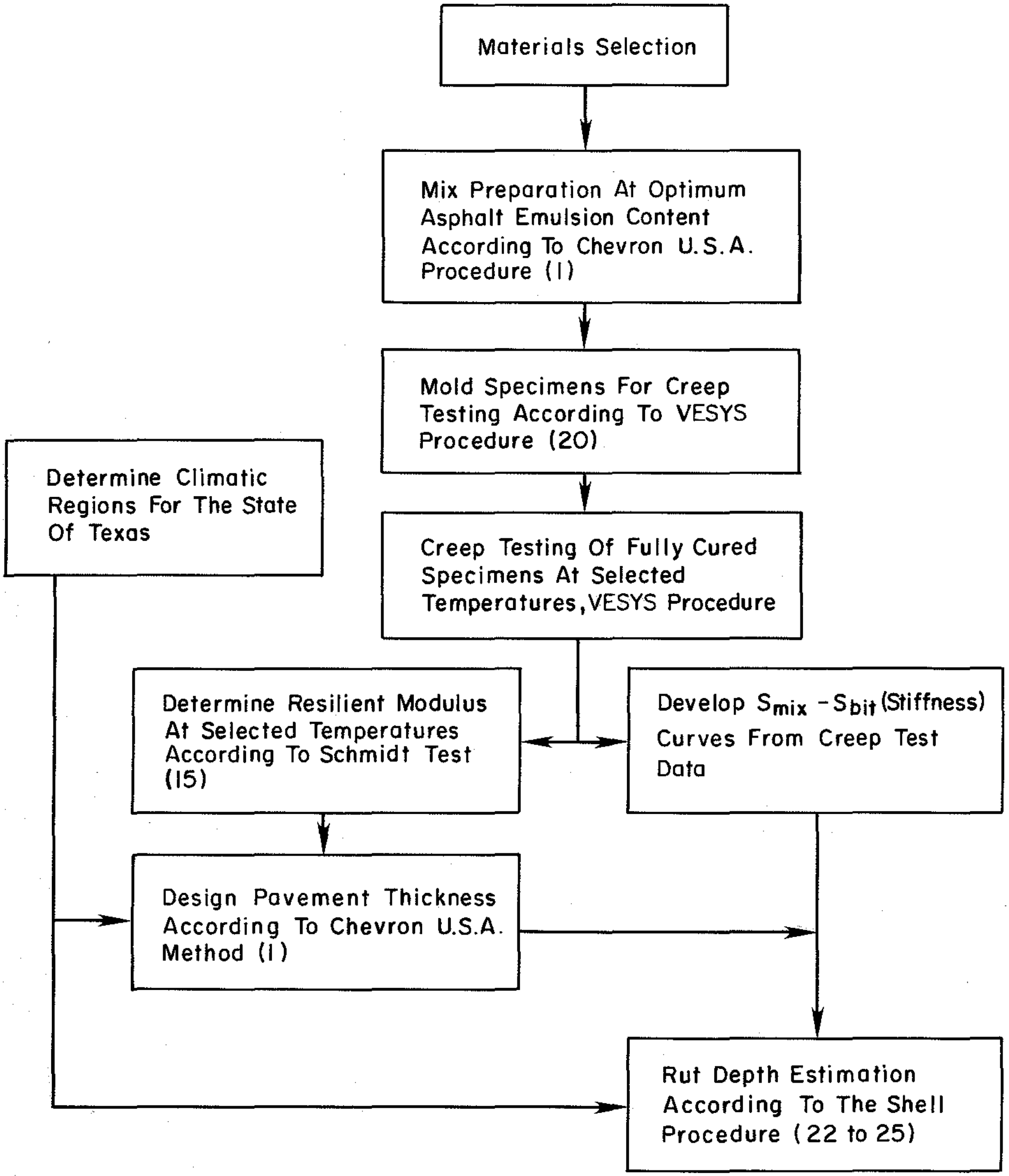
The structural coefficients discussed in the preceeding section were based on the material characterization parameter of resilient modulus determined by dynamic field and laboratory testing. The response of the asphalt (asphalt cement or emulsified asphalt) stabilized material has been satisfactorily shown to be essentially elastic for these load durations (approximately a tenth of a second). A question remains, however, regarding the ability of these stabilized marginal aggregates to resist plastic deformation due to repeated loadings during the pavement life. The purpose of this section is to evaluate the permanent deformation or rutting potential of selected sand-asphalt materials.

The flow chart in Figure 7 shows the sequence followed for this investigation.

Materials Selection

The resistance to lateral flow (R-value) and diametral resilient modulus (M_R) of laboratory molded as well as field cored paving mixtures containing poorly graded Texas sands bound with asphalt cement and emulsified asphalt were discussed previously in this report. Some of these materials were found to have desirable properties when used as part of pavement structures.

Representative samples, from the previous work, of a potentially marginal aggregate as well as a potentially poor sand were selected for analysis in this phase of the investigation. A poorly graded siliceous river deposited sand from District 11 (Lufkin, Texas) containing an appreciable amount of fines (aggregate passing the #200 sieve) was selected as a potentially marginal aggregate. Conversely, a poorly graded siliceous beach sand from District 16 (Padre Island), but devoid of fines, was chosen as a poor candidate for use as a base in a full depth pavement. The contrasting selection of these two sands was made hoping they would cover a typical gradation spectrum of common wind and water deposited sands in Texas. For convenience, the gradation characteristics and other important properties of these materials are summarized in Table 10 together with a limestone and basalt aggregate graded to provide a high quality asphalt mixture. The limestone aggregate was used as the control mix for this investigation.



Note: Numbers in parenthese indicate the reference of the cited procedure.

Figure 7. Sequence for the evaluation of rutting potential of pavements constructed with asphalt emulsified Texas sands.

Table 10. Aggregate characteristics.

Sieve Size	Aggregate Gradation (percent retained)		
	Control Aggregate (Limestone-Basalt)	District 11 Trinity Co. (FM 3736)	District 16 Padre Island (Beach Sand)
3/4 inch	-	-	-
1/2 inch	-	-	-
3/8 inch	-	-	-
#4	40.0	-	-
#8	15.0	0.2	-
#10	-	0.7	-
#16	5.0	11.5	0.1
#30	5.0	56.2	0.2
#40	-	67.0	0.4
#50	25.0	73.6	0.5
#60	-	77.8	0.8
#80	-	83.2	26.7
#100	4.0	84.9	58.9
#200	6.0	88.0	98.2
Sand Equivalent	-	72	98
Fineness Modulus	-	2.6	0.60
Plastic Index	-	0	0
Liquid Limit	-	13.2	24.8
Plastic Limit	-	N.P.	N.P.

It has been shown that except for the R-value of the District 16 sand-asphalt mix, both marginal materials were considered suitable for asphalt stabilization according to both the Chevron U.S.A. criteria (1) and the Asphalt Institute criteria (3).

Mix Preparation and Laboratory Molding

Asphalt emulsion stabilized mixtures were designed according to the Chevron U.S.A. procedure (1). An anionic slow setting emulsion (SS) was used in the mixes with District 11 sand, and a cationic slow setting emulsion (CSS) was found suitable for District 16 sand. The emulsion residue (binder) was an AC-10 asphalt cement characterized by a softening point, equal to 113^{o}F and a penetration index, PI, equal to zero.

The specimens were molded for creep testing in accordance with the procedure described in the VESYS User's Manual (20). The molded specimens were allowed to cure for a period of eight days at a temperature of 77^{o}F. At the end of this period, the specimens were examined and determined to be fully cured. Table 11 lists the characteristics of the different mixes studied. Because the District 11 mix (mix 2) was considered to be a somewhat lean mix, a second set of specimens was molded using a higher emulsion content (mix 3). This allowed comparison of two different quantities of binder in that particular mix.

Testing Procedure

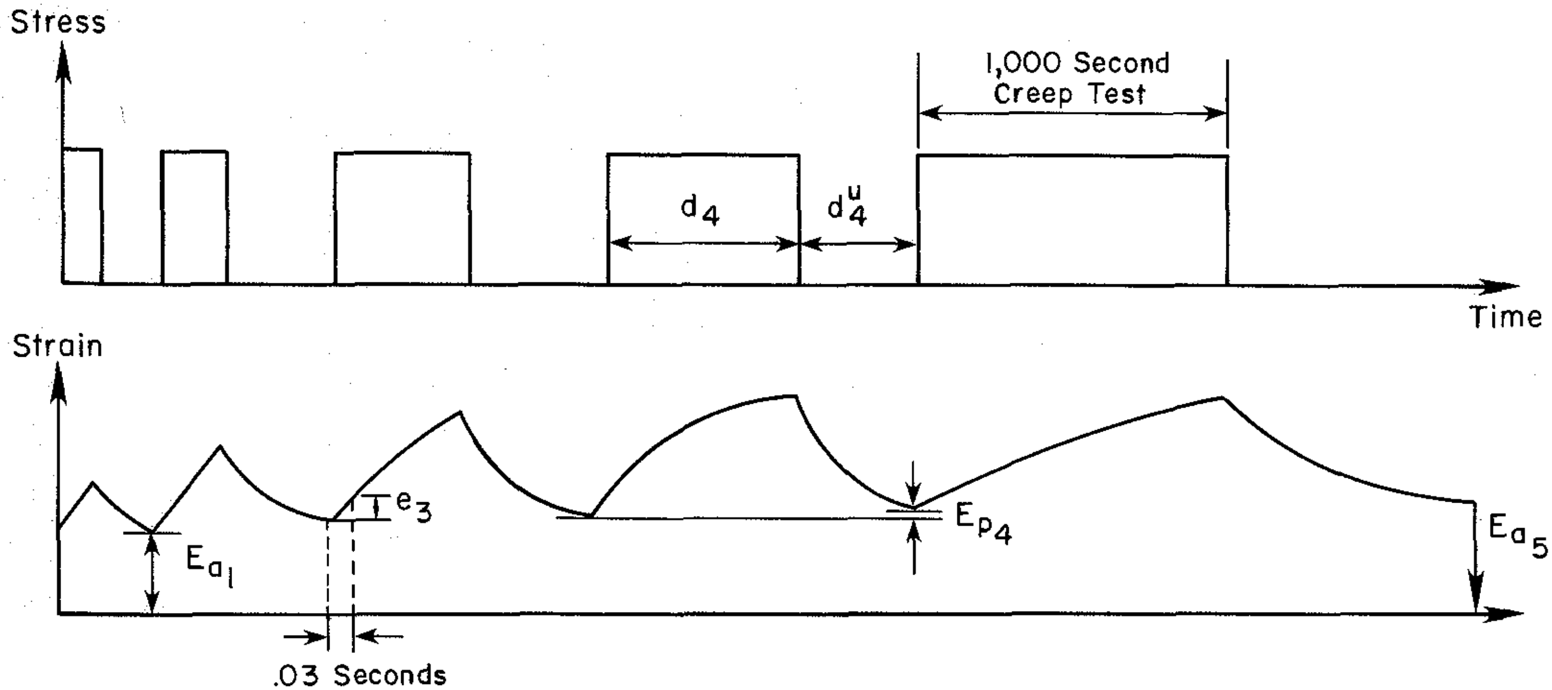
Unconfined constant compressive stress (creep) tests were pre-

Table 11. Mix description.

Characteristics	Mix 1	Mix 2	Mix 3	Mix 4
Aggregate Source	Control	District 11 (FM 3736)	District 11 (FM 3736)	District 16 (Padre Island)
Type of Stabilizer	Asphalt Cement	SS Asphalt Emulsion	SS Asphalt Emulsion	CSS Asphalt Emulsion
Content, weight percent	5.0	9.5	11.0	12.0
Binder Residue	AC-10	AC-10	AC-10	AC-10
Content, percent	5.0	6.6	7.6	8.3
Content, percent	10.9	12.3	14.2	12.9
Voids, volume percent	5.0	16.7	13.8	26.6
Remarks	High density	Dry mix. Under-asphalted	Optimum emulsion content (Chevron criteria)	Optimum emulsion content (Chevron criteria)

formed for the different mixes at temperatures of 32°F, 77°F and 104°F according to the VESYS procedure (20). This procedure consists of an incremental cycle of application and removal of a constant load (incremental static-dynamic test). Each application was followed by a rest period; Figure 8 gives a description of the loading function. The 1000 seconds load application cycle was used for the calculation of a creep compliance and ultimately, the stiffness of the mix. The shorter load applications were considered to have a preconditioning effect on the specimen. This was assumed to simulate pavement consolidation due to traffic load during the early life of the pavement. The magnitude of the specimen deformation was measured at various times during the 1000 seconds loading: 0.03, 0.1, 1.0 seconds and so on. Mix stiffness values (S_{mix}) were calculated as the ratio of the stress level (σ) at which the test was conducted to strain (e) produced at the time of the measurements. Shell researchers (27) recommend that the creep test be performed within the linear visco-elastic stress range. Therefore, care was taken to adjust the magnitude of the load to insure that the specimens responded in the linear range during the creep test.

Diametral resilient moduli (M_R) were measured at 77°F and 104°F for the different mixes according to the Schmidt method (15). The creep test specimens were cut to the required size so that the Schmidt apparatus could be used. Table 12 shows the moduli values for the different mixes.



d_i = Duration Of The ith Load Pulse

d_i^u = Rest Period After The ith Load Pulse

E_{p_i} = Increment In Permanent Strain Due To The ith Load Pulse

E_{a_n} = Total Accumulated Permanent Strain Due To n Pulse $E_{a_n} = \sum_{i=1} E_{p_i}$

e_i = Strain Amplitude Measured At .03 Seconds Due To The ith Load Pulse

Figure 8. Stress application and strain function of incremental test series. (From reference 20).

Table 12. Resilient moduli of the different mixes.

Mix I.D.	M_R @ 77°F (psi)	M_R @ 104°F (psi)
1	406,000	110,000
2	493,000	275,500
3	174,000	108,750
4	43,500	-

Legend:

Mix 1 (Control Mix).

Mix 2 (District 11 sand).

Mix 3 (District 11 sand with higher binder content).

Mix 4 (District 16 sand).

Pavement Thickness Design

A necessary step for the evaluation of rutting potential was the determination (design) of full depth pavement thicknesses to account for the following variables:

1. traffic volume,
2. subgrade strength, and
3. ambient temperature.

The effect that traffic produces on pavement performance was analyzed by designing for a different number of repetitions of the equivalent axle load (EAL), e.g. the number of passes of an 18,000-pound single axle load with dual wheels on each side of the axle producing a contact stress of 85 psi per wheel.

The influence of subgrade strength on both, thickness design and the estimation of permanent deformation was considered by assuming two types of subgrades over which pavements made of these materials could be constructed. A weak subgrade and a strong one, exhibiting elastic moduli of 3,600 psi and 29,000 psi, respectively, were considered representative extremes of untreated subgrades found in Texas. The effect of subgrade modulus on pavement thickness design is presented in Figures 15 through 18.

Finally, the effects of temperature on pavement performance was accounted for by designing pavements for different climatic regions occurring in Texas.

A suitable method for the determination of the required pavement thickness was adopted. Criteria for thickness design are discussed in the section entitled Analysis of Design Methods.

Climatic Regions

The moduli of unbound materials are generally not affected by variations in ambient temperature. However, the properties of asphalt treated materials are strongly dependent on such factors. For this reason any rational pavement design method should account for such variations in temperature. The Shell design procedure (22) incorporates a method to relate the mean annual or monthly air temperature to an effective asphalt temperature depending on the thickness of the asphalt layers.

It should be recognized that in order to evaluate the rutting potential of "marginal" asphalt pavements for the state of Texas, a division of the state according to annual climatic variations is necessary. For this purpose, the mean monthly air temperatures (MMAT) as well as the mean annual air temperatures (MAAT), recorded for twenty-one cities (Table 13) throughout the state, were taken as a representative sample of the mean temperatures occuring in Texas. Each MMAT was assigned a weighting factor that would take into account the relative effect of ambient temperature on pavement properties with respect to permanent deformation. Edwards and Valkering (23) produced a graph showing the relation between temperature and the weighting factors. From the arithmetic mean of these factors, weighted mean annual air temperature (w. MAAT) were derived for each city. A detailed description of this approach can be found in references 3, and 22. Four distinct climatic regions are shown on Figure 9 along with (MAAT) and the (w. MAAT) values for each region.

Table 13. Mean annual air temperatures for various cities. (from reference 29).

City	Region	MAAT (°F)	W. MAAT (°F)
Abilene	2	64.6	71.1
Amarillo	3	57.4	64.7
Austin	2	70.5	73.8
Brownsville	1	73.8	76.6
College Station	2	60.0	73.2
Corpus Christi	1	72.0	75.9
Del Rio	1	70.0	75.2
Dallas/Ft. Worth	2	66.2	72.5
El Paso	4	63.3	70.7
Ft. Stockton	4	66.0	71.4
Houston	2	67.5	73.6
Laredo	1	74.0	77.9
Lubbock	3	59.7	65.3
Lufkin	2	66.7	72.0
Midland/Odessa	4	64.6	70.5
Presidio	1	70.2	76.1
San Angelo	2	66.2	72.4
San Antonio	1	68.7	73.6
Tyler	2	65.7	71.2
Waco	2	65.8	72.2
Wichita Falls	2	64.0	72.0

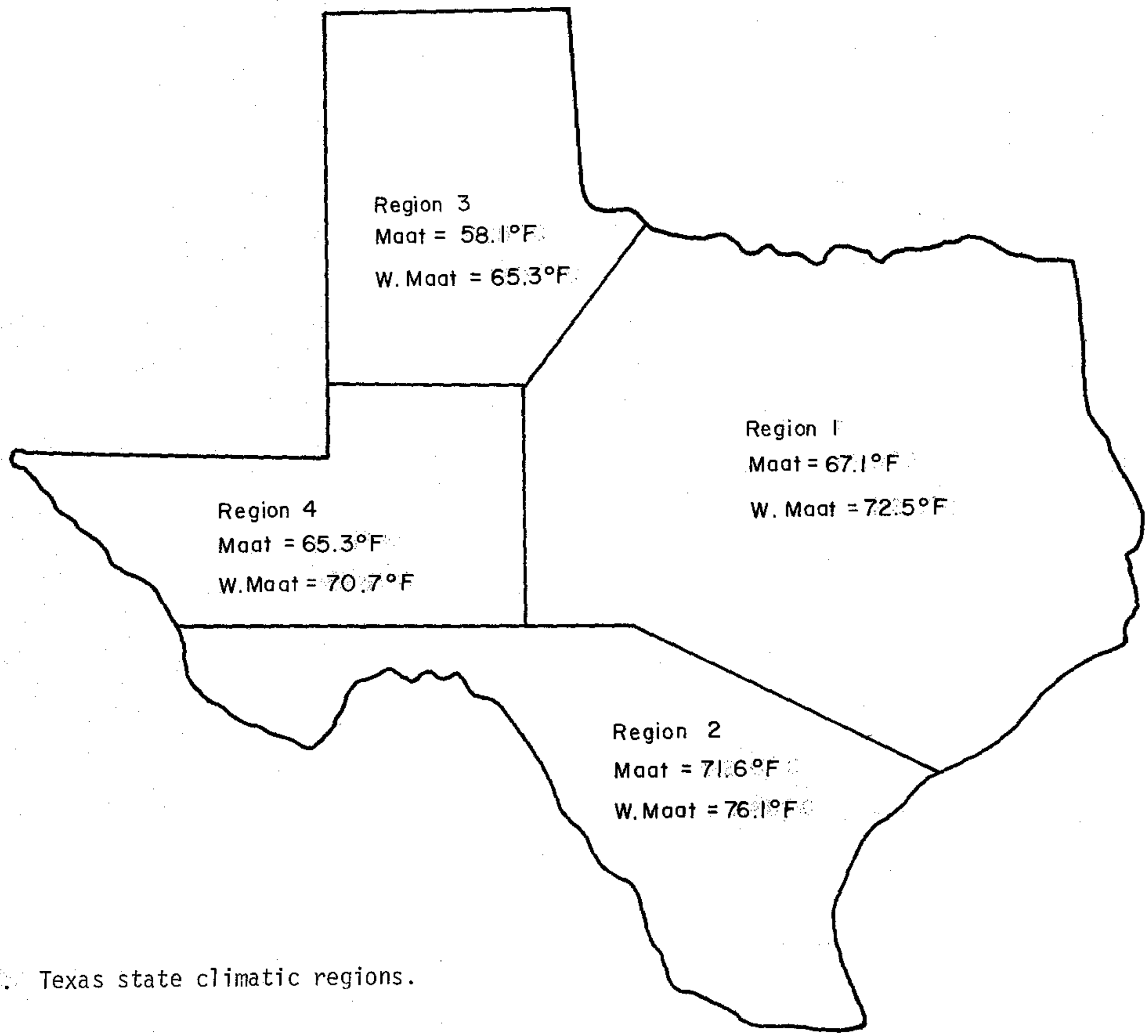


Figure 9. Texas state climatic regions.

Once the climatic regions were established, the next step was to derive representative MMAT's for the different regions from the arithmetic mean of the MMAT's of the cities within each region (see Table 14). These regional MMAT's were used later to calculate annual effective viscosities (using the Shell procedure) for different thicknesses of the asphalt treated layers. This process will be discussed later.

Review of the Theory

Permanent deformation is the result of two different mechanisms, densification (volume change) and repetitive shear deformation (plastic flow with no volume change). Two broad categorical approaches to the problem are currently being used. One such approach consists of design procedures based on empirical correlations of excessive deformations to some predefined failure criteria, usually elastic stresses and/or strains. Its major disadvantages is that the degree of deformation cannot be predicted after a given number of strain repetitions. Procedures in the second category provide a method for the prediction of accumulated deformations in pavement systems. For such an approach, limiting the vertical subgrade strain is not the only criteria since this does not insure that permanent deformation in the upper pavement layers will not be present.

The Shell pavement design procedure incorporates the positive factors previously mentioned and presents an additional advantage. It uses a relatively simple test to evaluate permanent deformation properties of the pavement materials.

Table 14. Mean monthly air temperature for Texas climatic regions.

Month	Average MMAT, (°F)			
	Region 1	Region 2	Region 3	Region 4
January	55.4	47.3	37.4	43.7
February	59.0	50.0	41.0	49.1
March	65.3	57.2	47.3	55.4
April	73.4	66.2	59.0	64.4
May	79.7	74.2	65.3	72.5
June	84.2	81.5	77.0	81.5
July	86.0	84.2	80.6	84.2
August	86.0	84.2	77.0	82.4
September	81.5	77.9	69.8	75.2
October	73.4	68.0	59.9	65.3
November	62.6	57.2	46.4	54.5
December	56.3	50.0	40.1	46.4
MAAT, (°F)	71.6	67.1	58.1	65.3

Creep is the continuous time-dependent deformation of materials under constant stress or load. The parameters of concern are the stress level, the temperature and the loading time. For asphalt mixes, the deformation under the applied load is the result of a change in the (the binder between the mineral aggregate particles is squeezed into the voids) as affected by the rheological properties of the binder (asphalt) at the given temperature and load duration (27). As a result of this process, gradually more particle-to-particle contacts are formed, and the transfer of force is progressively absorbed more through the aggregate structure and less through the binder. In 1973, J. F. Hills (28) developed a theoretical physical model which he used to justify this mode of deformation. Hills went a step further in the development of his model by taking account of the gradation of the mineral aggregate by observing that, for well graded mixes, the binder layer between aggregate particles consists of a mixture of smaller sized particles (filler) and bitumen. Given this situation, the deformation process can be compared to assembling a set of "chinese boxes" (as Hills called it). The behavior (sliding) of a particle pair is influenced by the binder layer which contains at the same time smaller pairs of particles sliding one against the other as a result of the stress application (27, 28). Figure 10 helps to visualize the process. Taking this into consideration, Hills was able to characterize the deformation behavior of asphalt mixes during the creep test by means of a set of curves for which the following basic equation applies:

$$\frac{e_{mix}}{Fy} = \left\{ \left(1 + \frac{e^{1}_{bit}}{Fx}\right)^{0.5q} - 1 \right\}$$

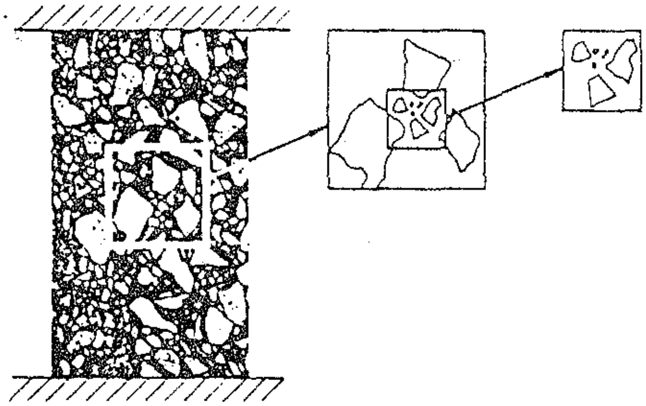


Figure 10. Concept of the "Chinese Boxes". (Reference 28).

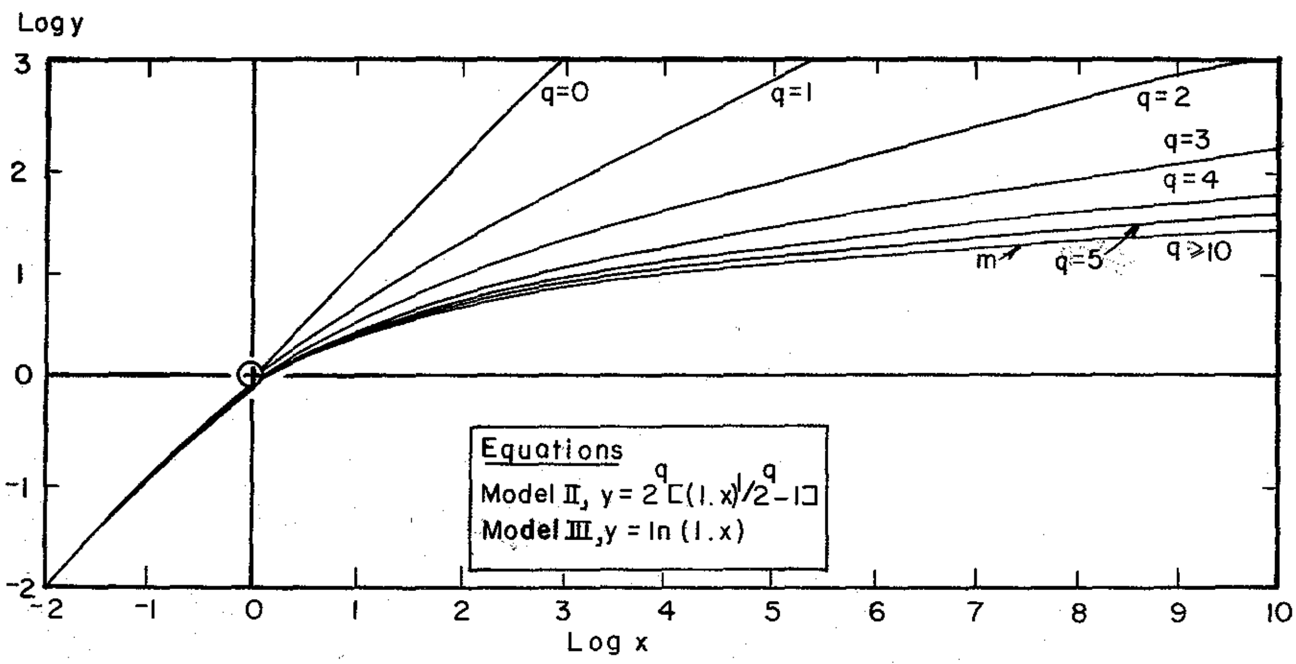


Figure 11. Characteristic curves for the deformation of asphalt mixes derived from the theoretical models (Reference 28).

where:

$$e^{1}_{bit} = \frac{\sigma t}{3},$$

then,

$$\frac{e_{mix}}{Fy} = 2^{q} \left\{ \left(1 + \frac{\sigma t}{3\eta Fx}\right)^{0.5q} - 1 \right\}$$

where:

e_{mix} = axial strain in the mix,

$1/Fy$, $1/Fx$ = factors which are constant for one particular creep test and are dependent on the internal structure of the asphalt mix at the start of the test,

q = integer > 1 corresponding to the number of "Chinese boxes" used in the model and depends on the grading of the mineral aggregate,

e^{1}_{bit} = viscous component of the calculated strain in the bitumen,

σ = constant stress applied to the specimen,

t = loading time, and

η = dynamic viscosity of the binder.

Figure 11 represents this equation graphically. Yet, a graphical representation of this form is not suitable for the characterization of the deformation behavior since plotting the mix strain (e_{mix}) as a function of the strain of the bitumen (e^{1}_{bit}) will show a dependence on the stress level .

As an alternative, e_{mix} can be plotted as a function of loading time (t), but again, this relation shows dependence on test temperature, bitumen grade, and stress level. It has been shown by Hills and others (28, 29) that the effect of the previously mentioned parameters can be eliminated by plotting the stiffness of the mix (S_{mix}) as a function of the bitumen stiffness (S_{bit}) (see Figures 12a and 12b for illustration). Stiffness modulus is defined as the visco-elastic equivalent of the modulus of elasticity (E) (27). In principle it is possible to perform separate tests on the bitumen residue to measure S_{bit} for the same parameters (duration of load application and temperature) as in the creep test (29). However, an easier way to evaluate S_{bit} is by means of the Van der Poel nomograph (30). To enter this nomograph, the same temperature and loading time for the creep test are used. Also required are the "Ring and Ball" softening point (T_{800}) and the Penetration Index (PI) of the bitumen recovered from the mix.

Presentation of creep data in the form of stiffness modulus of the mix as a function of stiffness modulus of the bitumen is very valuable because the materials are characterized independently of test variables and can be adapted to design calculations (28).

The real advantage of the creep test, besides being relatively simple to perform, is that it enables one to correlate the creep behavior of asphalt mixes with the depth of rutting that will occur when these mixes are used on pavement structures subjected to specific traffic loading and climatic conditions (29). This subject will be treated in the next section.

"Rutting tests" performed on a tracking machine have been conducted

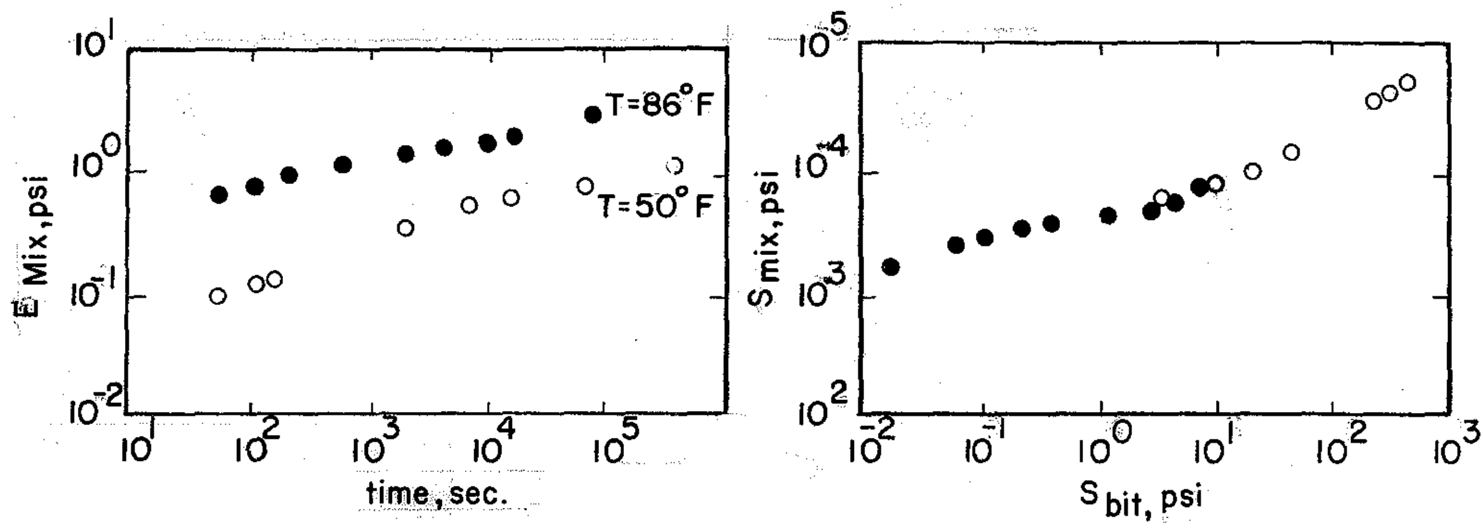


Figure 12a. Creep test results at two different temperatures. (Tested at a stress level of 20 psi). (From reference 29).

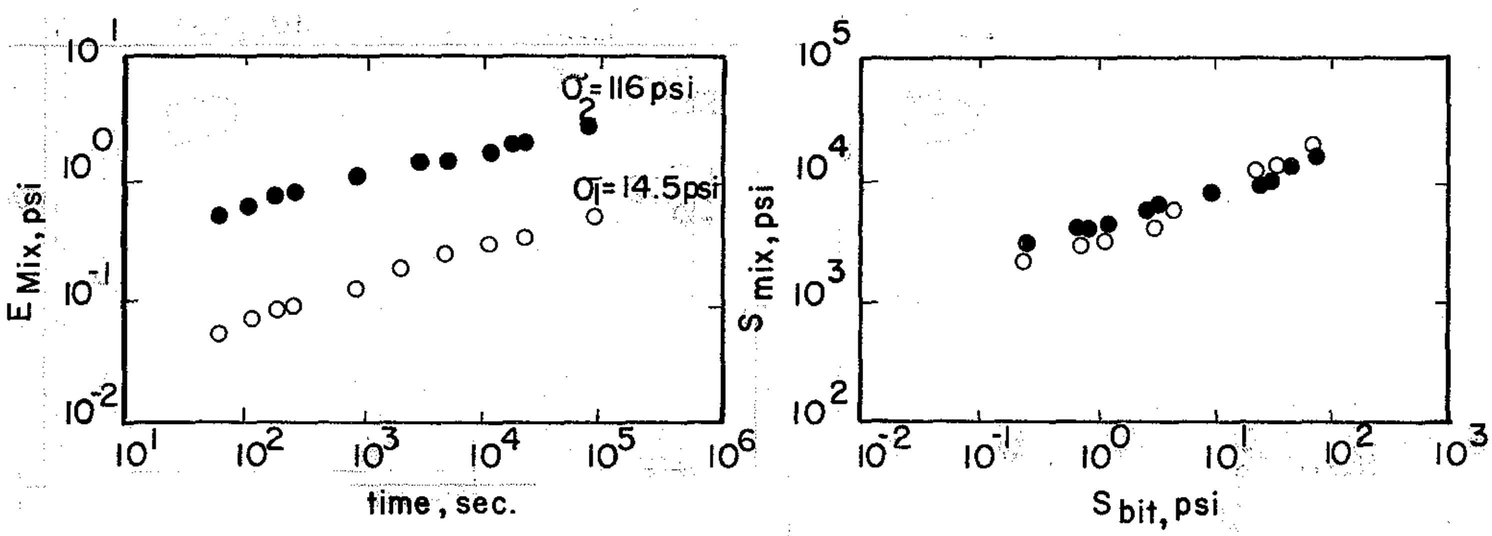


Figure 12b. Creep test results at two different stresses. (Test temperature, 68^{o}F). (From reference 29).

by Shell researchers (28) hoping to simulate actual road conditions and hence to study the correlation between creep tests and the rutting observed in service.

In order to correlate the results of both tests (creep and rutting), it is necessary to express them in terms of the same parameters: S_{mix} and S_{bit}. A description of the rutting test can be found in references 21 and 31.

The calculation of S_{mix} for the rutting test involves the use of the following equation, which is based on the pavement cross-section in Figure 13.

$$S_{mix} = \frac{\sigma_{avg}}{(H_o - H)/\ H_o}$$

where,

σ_{avg} = average stress level in layer H_o. (The product of a proportionality factor (Z) times the contact stress (σ_o)),

H_o = thickness of asphalt layer, and

H_o - H = reduction in thickness of the asphalt layer in the middle of the wheel path.

The calculation of S_{bit} for the rutting test is based on the assumption that only the viscous (irreversible) component of the stiffness modulus of the bitumen contributes to permanent deformation.

It has been shown (22, 29) that the strain of a bitumen is the result of the summation of three components, elastic (e_E), delayed or visco-elastic (e_D), and viscous (e_{visc}). According to this, S_{bit}

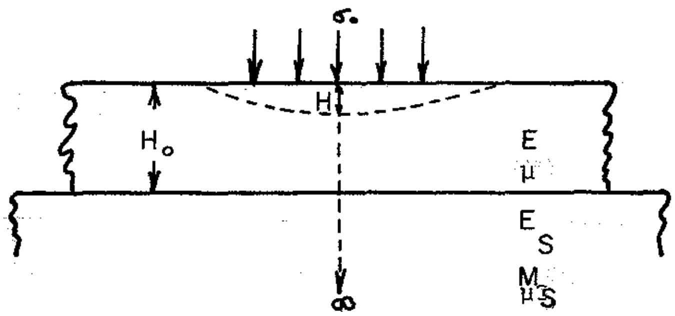


Figure 13. Elastic layer model for two pavement layers. (Reference 29).

of a mix can be expressed as,

$$\frac{1}{S_{bit}} = \frac{e_{bit}}{\sigma_0} = \frac{1}{S_{bitE}} + \frac{1}{S_{bitD}} + \frac{1}{S_{bit,\ visc}}$$

where σ_0 is the contact stress.

The viscous component of the bitumen stiffness modulus is given by,

$$\frac{1}{S_{bit,\ visc}} = \frac{1}{3\eta/t} = \frac{t}{3\eta} \quad .$$

For long loading times, the viscous component predominates, thus, the stiffness modulus of the binder (S_{bit}) can be represented by $S_{bit,\ visc}$ (see Appendix B).

In order to account for the dynamic effect of the wheel passes, the loading times are allowed to be superimposed. The total cumulative loading time in the rutting test is:

$$t = N \times t_w.$$

where N is the total number of wheel passes and t_w is the loading time per pass. Thus

$$S_{bit,\ visc} = \frac{3\eta}{N\ t_w}$$

The bitumen viscosity (η) is the only variable affected by temperature. In order to provide for changes in this parameter during the test (as indeed occurs in service conditions), the total contribution of different temperatures to the deformation need to be considered. Thus, the above equation will take the form:

$$S_{bit,\ visc} = \frac{3}{t_w} \left(\frac{\eta_{T1}}{N_{T_1}} + \frac{\eta_{T2}}{N_{T_2}} + \cdots\cdots\cdots + \frac{\eta_{Tn}}{N_{Tn}} \right) \text{ or}$$

$$S_{bit,\ visc} = \frac{3}{t_w \sum_{i=1}^{n} \left(\frac{\eta}{N}\right)_{T_i}} \quad .$$

Notice that t_w is held constant. It has been suggested in the Shell Method that t_w equals 0.02 seconds which is equivalent to a wheel at 30 to 60 mph. At this point, the results of the rutting test can be represented by S_{mix} as a function of $S_{bit,\ visc}$. Research by Hills and Van de Loo (29) indicates that data from the two types of test (creep and rutting) show satisfactory agreement, particularly when S_{bit} approaches $S_{bit,\ visc}$. However, to insure that a higher degree of correlation is attained, creep testing must be performed in the linear visco-elastic range (27). For a given temperature and duration of load, the stiffness of a certain type of asphalt is a constant. If asphalt mix specimens are tested under identical conditions of stress and temperature and if the stress levels are such that the specimens do not deform outside the linear visco-elastic range, unique values of S_{mix} will be obtained for each of the samples. Consequently, different S_{mix} -S_{bit} curves are obtained.

Analysis of the Design Methods

Usually a question arises as to which distress mode is critical for the pavement being designed. For the purpose of this study, we may limit these distress modes to the following:

1. fracture from repeated loading (fatigue), and
2. permanent deformation (rutting).

To deal with this problem, design procedures (21, 20, 32) currently incorporate a limitation of the vertical compressive strain at the top of the subgrade (permanent deformation criteria) and the horizontal tensile strain on the underside of the asphalt treated layer (fatigue criteria). If we are to evaluate the potential of asphalt treated sands to withstand permanent deformation, it is necessary to first determine suitable pavement thicknesses. These pavements must be designed to satisfy the criteria mentioned previously. Fatigue testing was conducted for the emulsion stabilized District 11 and District 16 sands. Results of these flexural beam tests are presented in Figure 14. These fatigue criteria were incorporated into the "Chevron U.S.A. Thickness Design Procedure for Asphalt and Emulsified-Asphalt Mixes" (33, 34). Based on the Chevron procedure, the fatigue criteria in Figure 15, shifted for field conditions (33), and the vertical strain criteria incorporated in the Chevron Method, design thicknesses were determined for asphalt emulsion stabilized bases for various climatic and traffic conditions.

Figures 15 through 18 show the required pavement thickness for each pavement material as a function of traffic volume, expressed as the number of equivalent axle loads (EAL), and the appropriate subgrade strength and climatic region. The Shell method can now be used to evaluate the rutting potential of these pavements.

The Shell design method is based on a model in which the pavement is regarded as a linear elastic multilayered system. The design criteria is similar to that of the Chevron procedure. In addition, the visco-elastic nature of the bituminous materials is taken into account by using stiffness values appropriate for the temperatures and times of loading occuring in pavement structures. Creep test data for the emulsion stabilized sands were analyzed as follows.

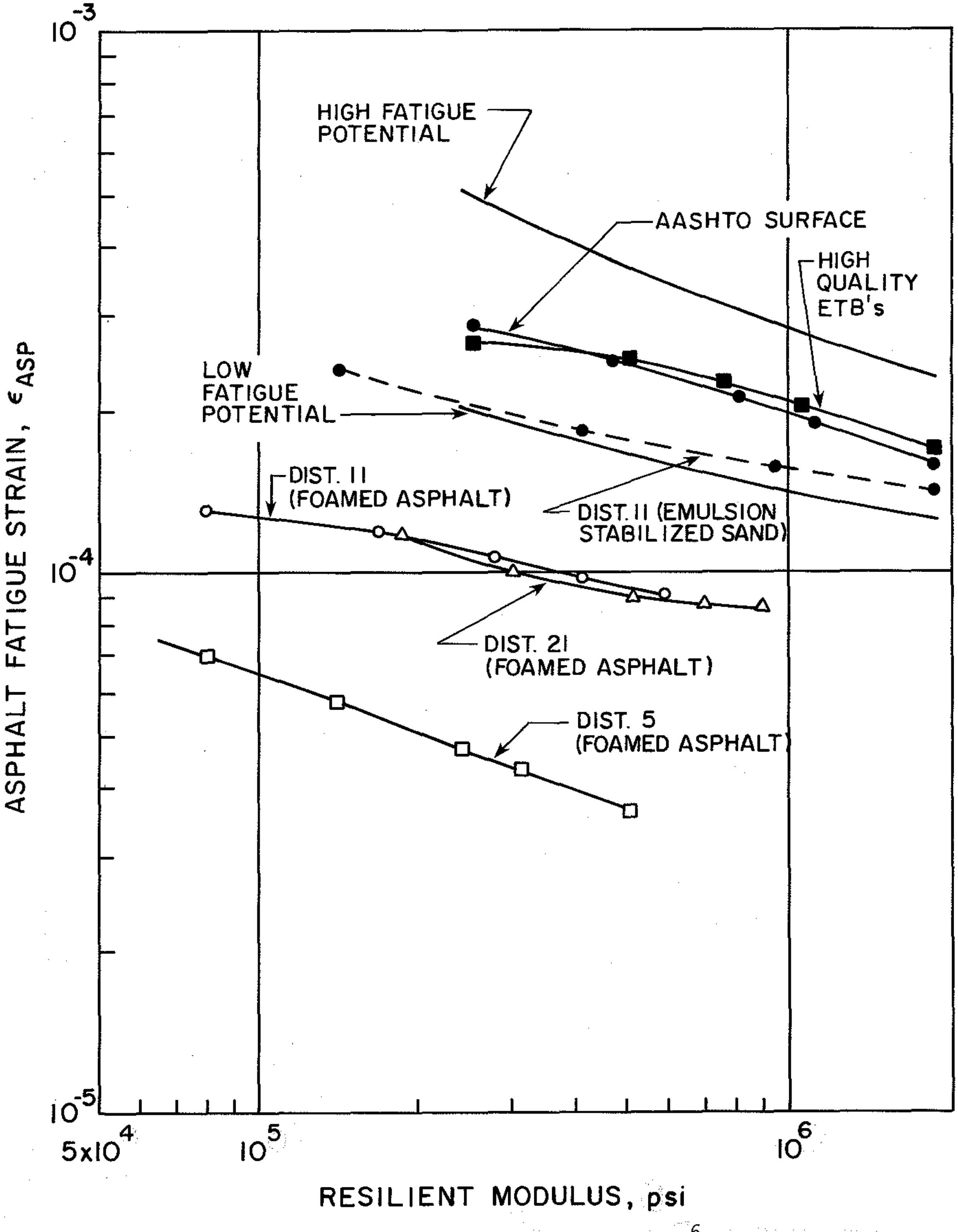


Figure 14. Comparison of fatigue curves for 10^6 18 Kip axle load applications (after reference 35).

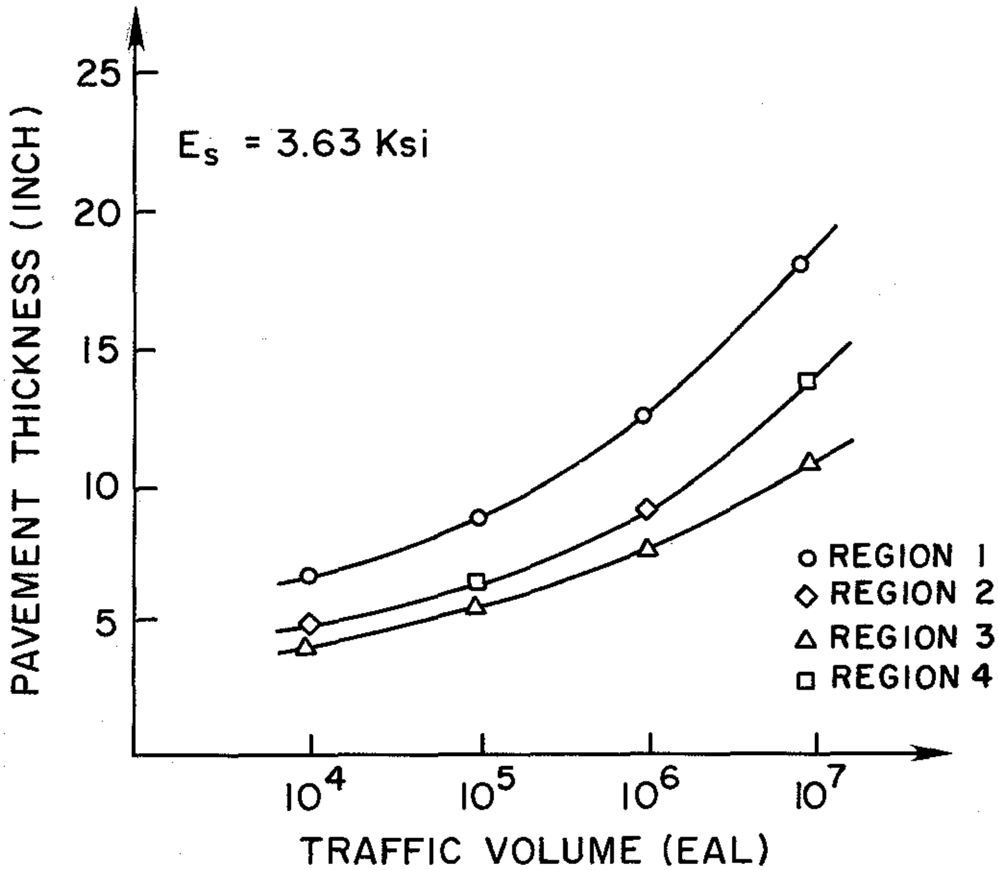


Figure 15a. Pavement thickness (inches), Mix 1

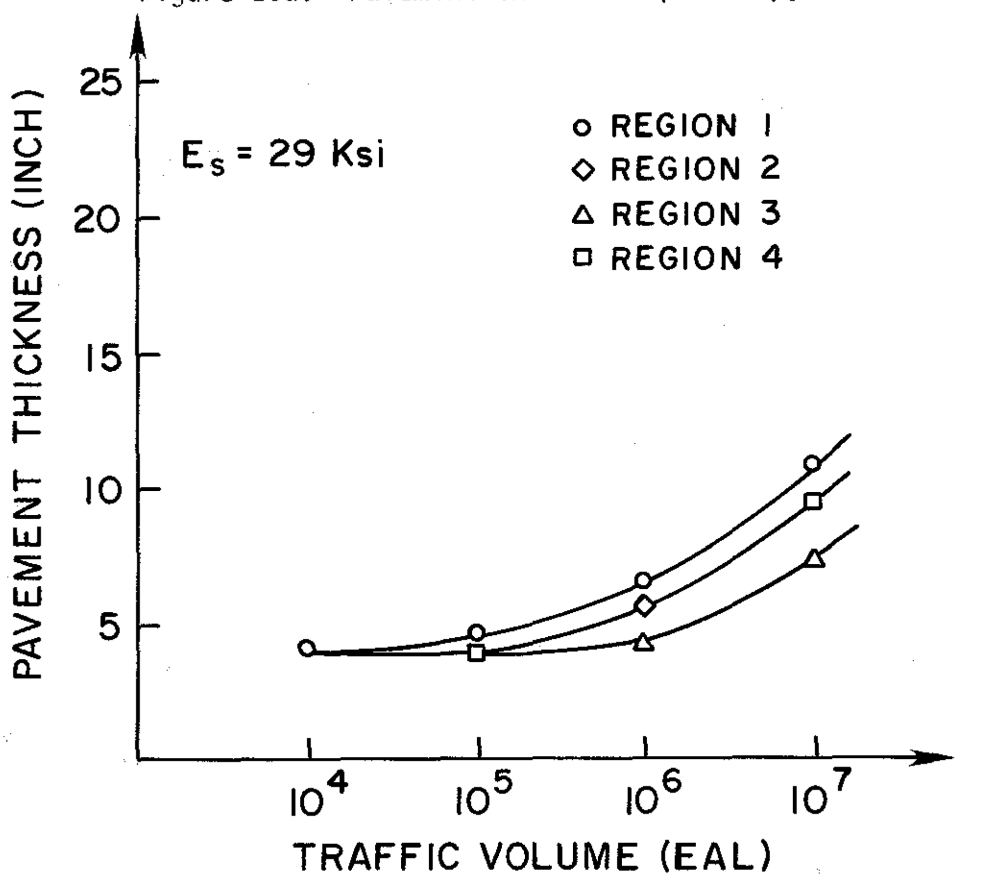


Figure 15b. Pavement thickness (inches), Mix 1

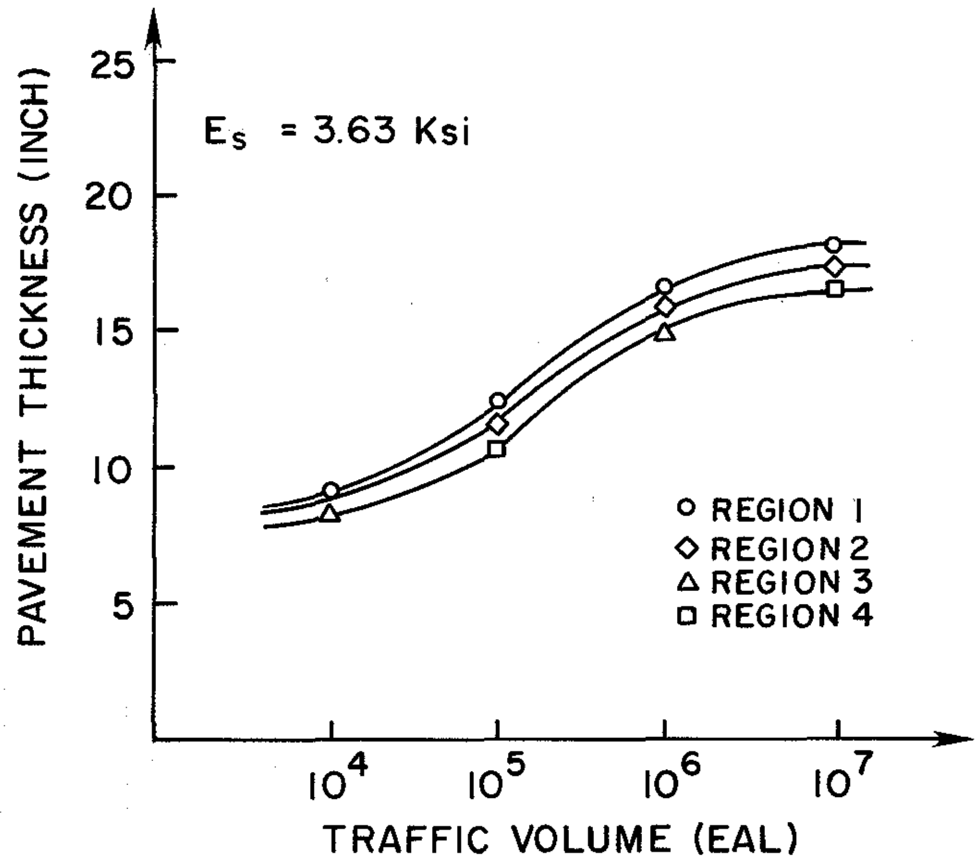


Figure 16a. Pavement thickness (inches), Mix 2

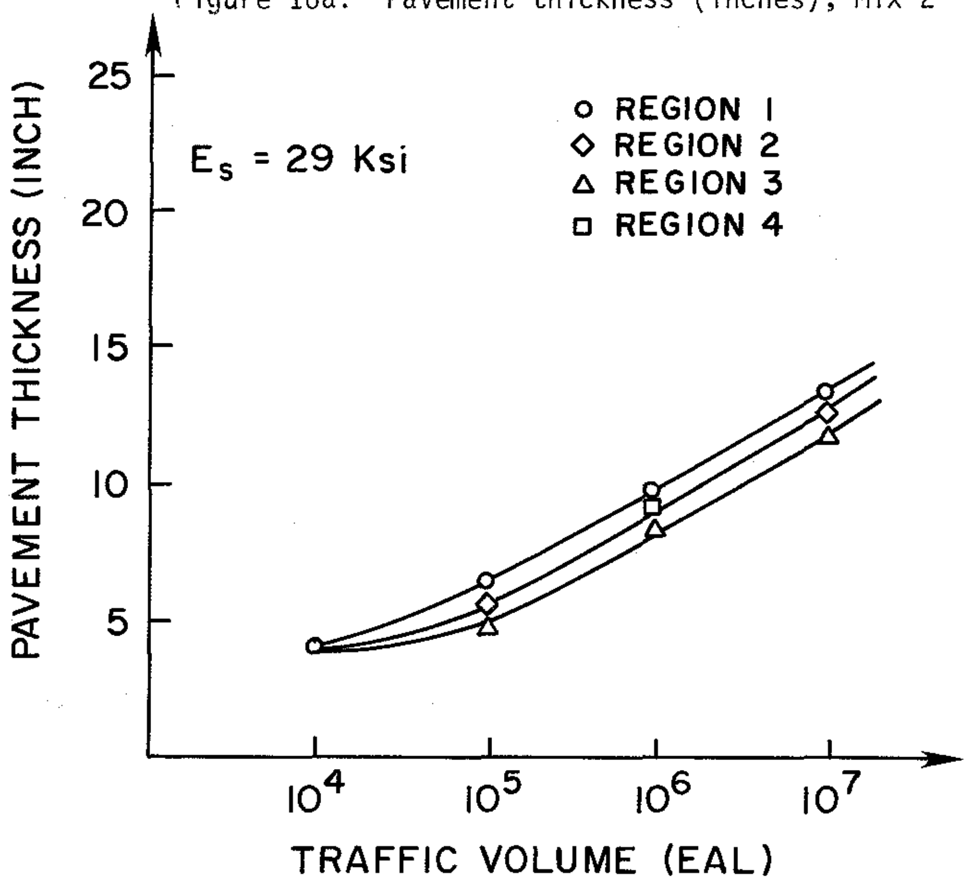


Figure 16b. Pavement thickness (inches), Mix 2

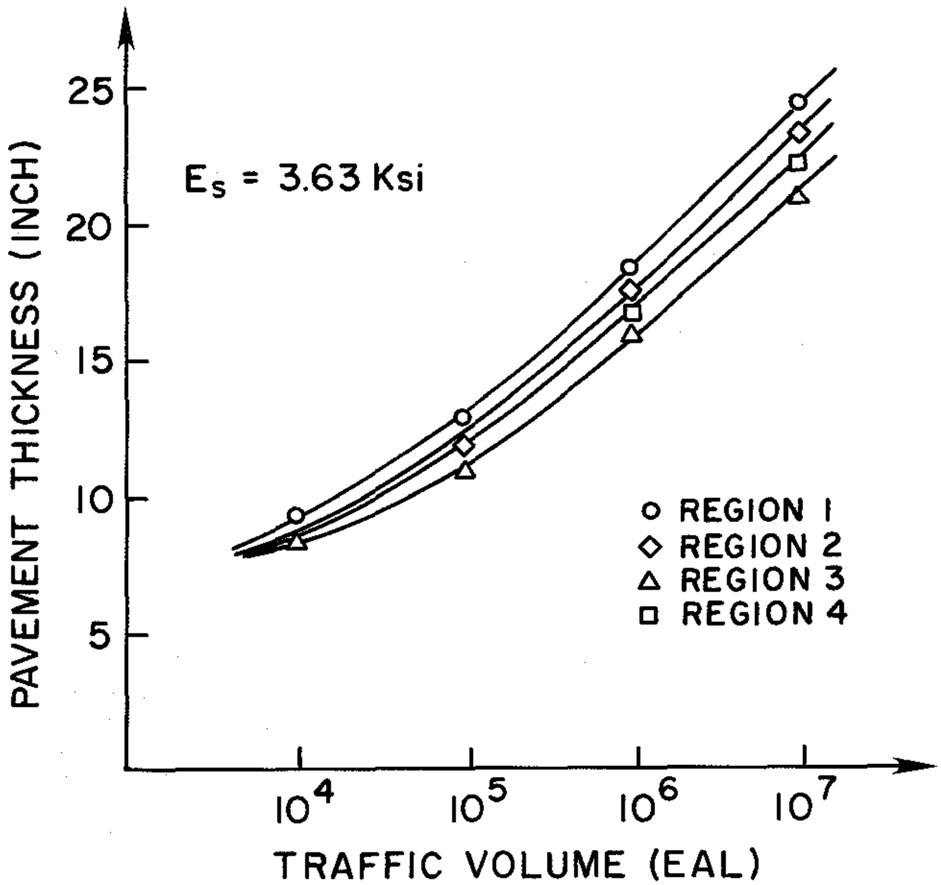


Figure 17a. Pavement thickness (inches), Mix 3

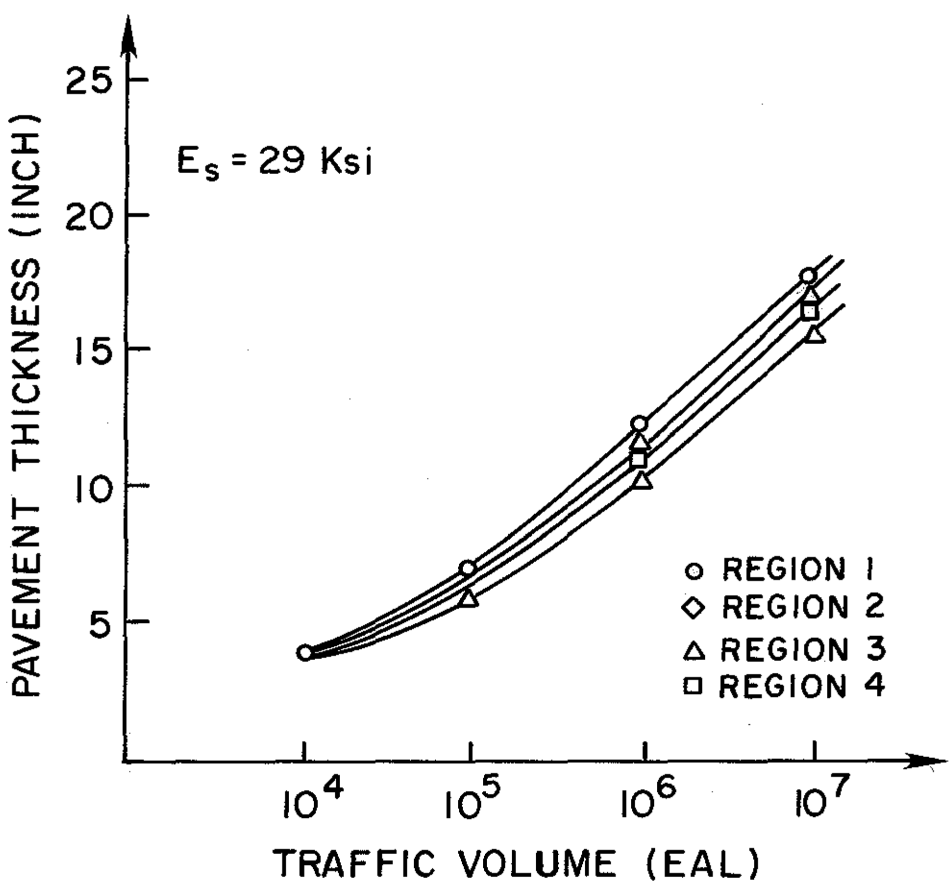


Figure 17b. Pavement thickness (inches), Mix 3.

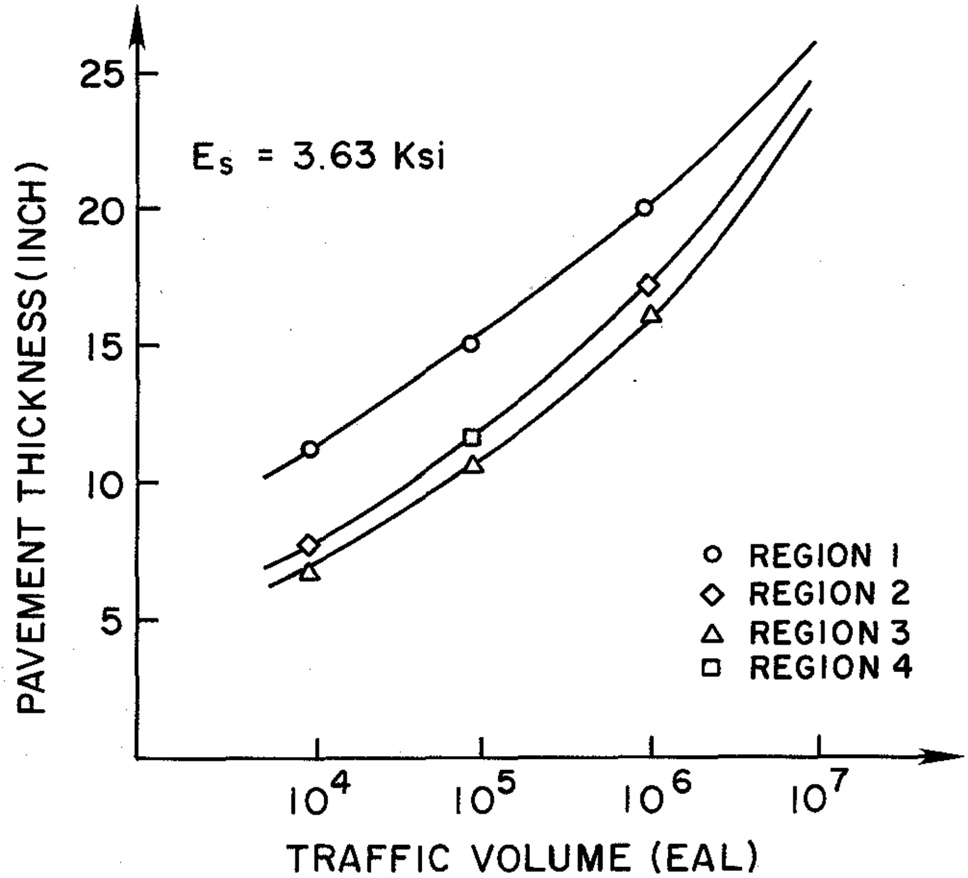


Figure 18a. Pavement thickness (inches), Mix 4 (top 3 inch - Mix 1).

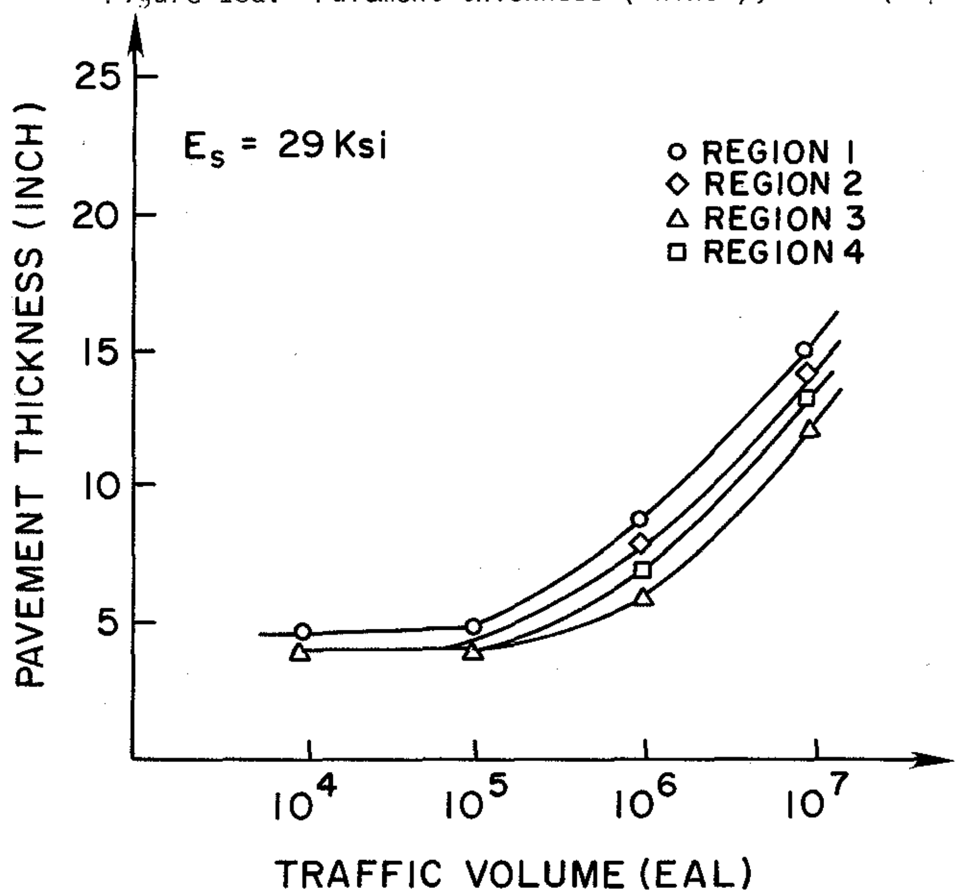


Figure 18b. Pavement thickness (inches), Mix 4 (top 3 inch - Mix 1).

1. Pavement thicknesses required to prevent development of either critical subgrade strain of critical bituminous layer tensile strain were established by the Chevron procedure (see Figures 15 through 18). These asphalt thicknesses were then subdivided so that the difference in temperature and hence the difference in the binder viscosity throughout the asphalt layer could be taken into account. The suggested subdivision, (h_{1-1} = 1.6 inches, h_{1-2} = 1.6 inches, and h_{1-3} =3.2 inches, where h_1 is the total thickness of the full depth layer being considered and h_{1-1} represents the surface layer) was based on detailed studies (23) which indicate that the uppermost layers are subject to the greatest temperature changes.

2. For each sub-layer thickness, the annual effective viscosity ~~of the binder~~ (V_{yeff-i}) was calculated. An effective pavement temperature was then associated with the viscosities. Details on how these values are computed can be found on references 23 and 24. Table 15 shows V_{yeff-i} and T_{yeff-i} values calculated for different thicknesses which relate to the AC-10 base asphalt used. For each condition and for different numbers of applications of the standard wheel load (87 psi of contact pressure), $S_{bit,\ visc-i}$ was calculated as follows:

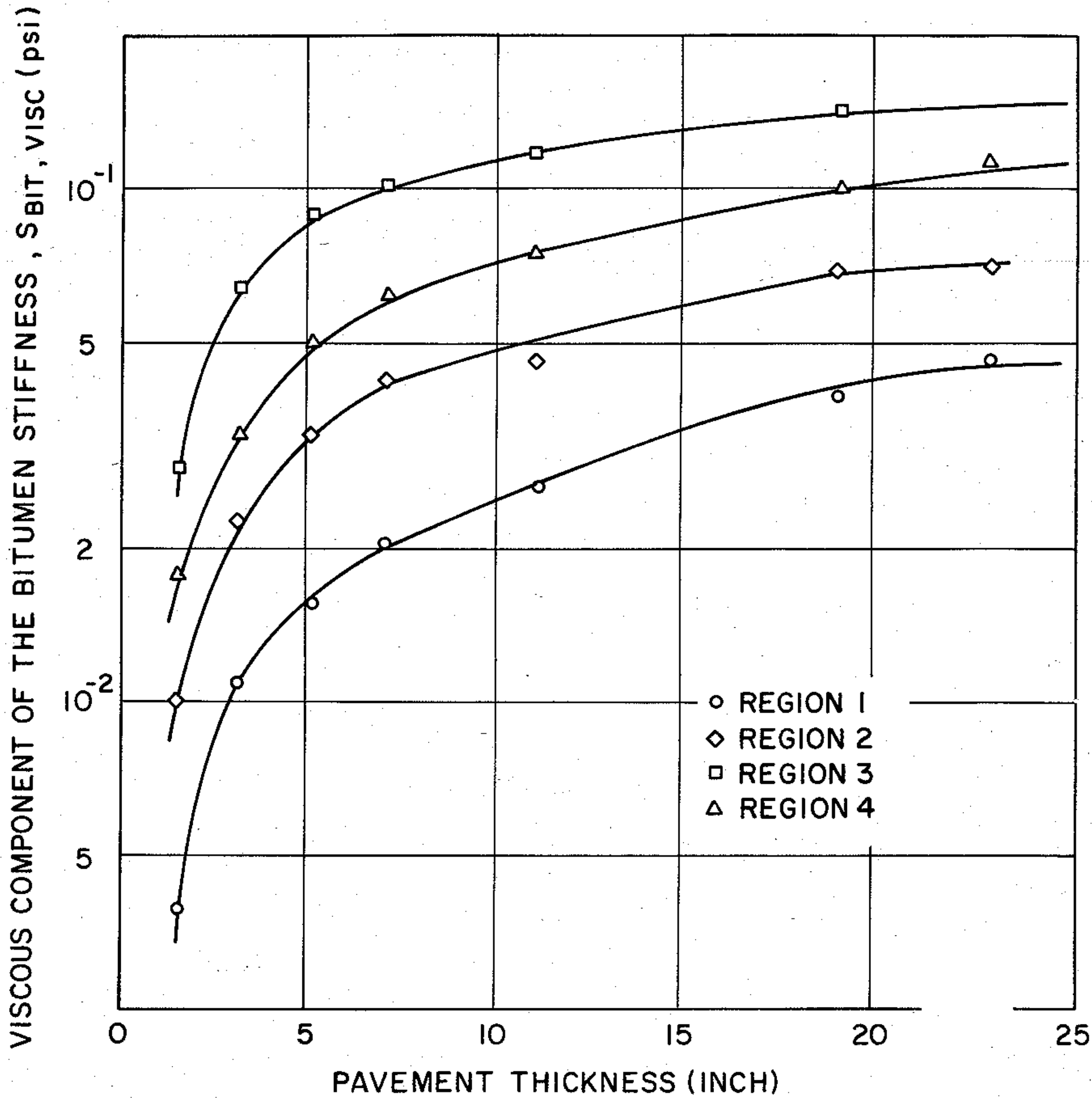
$$S_{bit,\ visc-i} = \frac{3\ V_{yeff-i}}{t_w \times W_{eq}}$$

where W_{eq} is the number of applications of the standard wheel.

Figure 19 can be used to determine the values of $S_{bit,\ visc-i}$ as a function of the <u>total</u> pavement thickness. These curves were plotted using the data in Table 15 and the above equation.

Table 15. Annual Effective Viscosities and Temperatures for AC-10 Asphalt.

Sublayer Index	Thickness (Inches)	Region 1		Region 2		Region 3		Region 4	
		T_{yeff-i} (^{o}F)	V_{yeff-i} (Stokes)	T_{yeff-i} (^{o}F)	V_{yeff-i} (Stokes)	T_{yeff-i} (^{o}F)	V_{yeff-i} (Stokes)	T_{yeff-i} (^{o}F)	V_{yeff-i} (Stokes)
1-1	1.6	108.5	3.3×10^{3}	102.2	6.2×10^{3}	93.6	2.0×10^{4}	98.6	1.0×10^{4}
1-2	1.6	104.0	5.3×10^{3}	96.8	1.2×10^{4}	87.8	4.0×10^{4}	94.1	1.8×10^{4}
1-3	4.0	96.8	1.2×10^{4}	91.4	2.4×10^{4}	83.3	6.6×10^{4}	88.7	3.4×10^{4}
	8.0	95.0	1.7×10^{4}	89.6	3.0×10^{4}	82.4	7.7×10^{4}	86.9	4.4×10^{4}
	16.0	91.4	2.4×10^{4}	86.9	4.4×10^{4}	79.7	1.0×10^{5}	84.2	6.0×10^{4}
	20.0	90.5	3.0×10^{4}	86.0	5.0×10^{4}	78.8	1.1×10^{5}	83.3	7.0×10^{4}



NOTE:

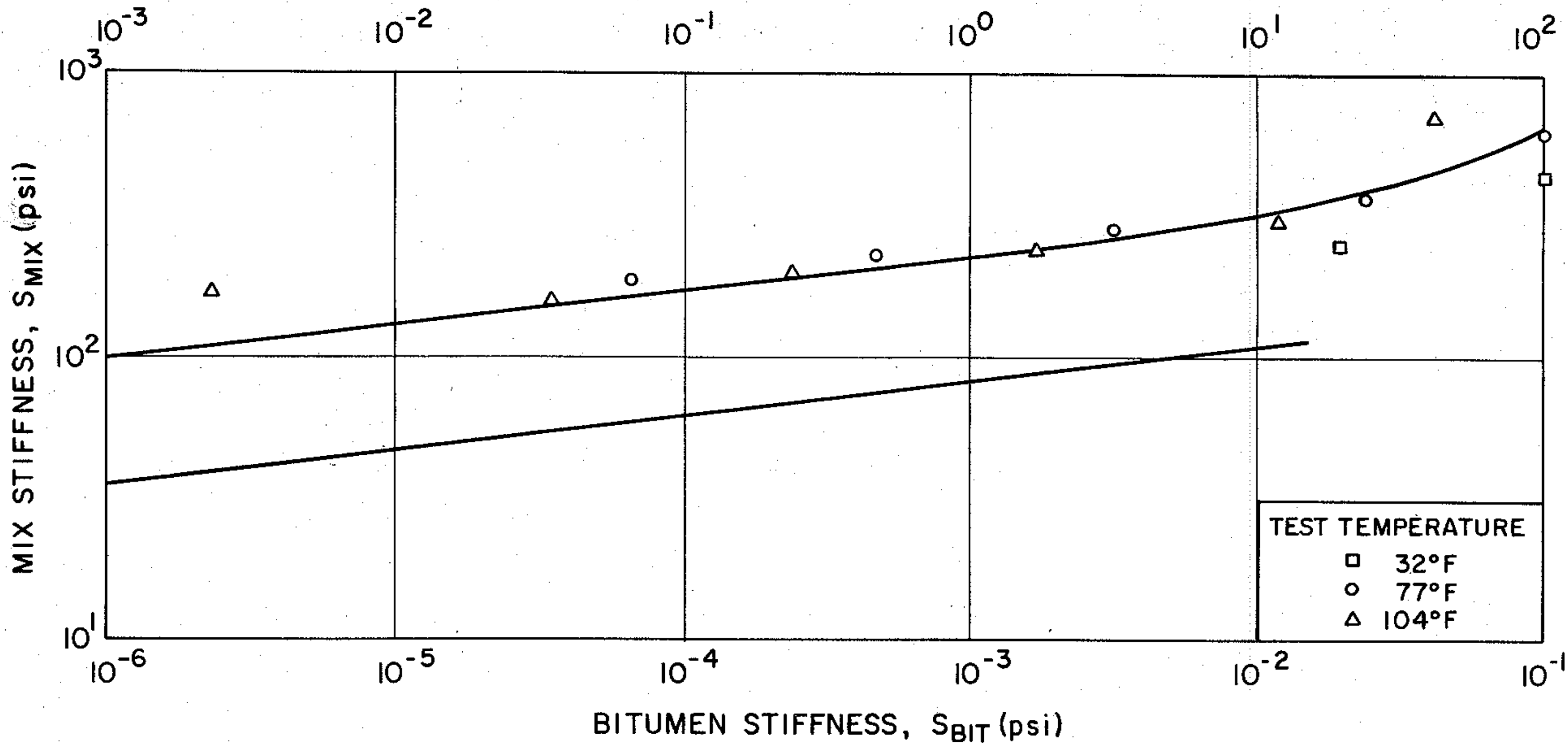
$S_{bit,\ visc}$ values shown in this graph were calculated for 10^4 repetitions of the EAL. For different numbers of EAL repetitions (N), multiply the $S_{bit,\ visc}$ value obtained for a given payment thickness and climatic region, by a factor of $10^4/N$.

Figure 19. Relationship between the viscous component of asphalt stiffness and pavement thickness.

3. For the values of $S_{bit,\ visc-i}$ derived from Figure 19 at different levels of strain repetitions, S_{mix-i} values were determined from the relevant stiffness curves (see Figures 20 through 26) obtained from creep test data. In order to do so, $S_{bit,\ visc-i}$ was assumed to be equal to S_{bit} values obtained from the Van der Poel nomograph (values on the abscissa of the stiffness graphs). Notice the stiffness curves have been extrapolated for low values of S_{bit}.

4. Permanent deformation is dependent on the stress level which results from the application of loads. It would be unrealistic to assume that the distribution is the same throughout the thickness of the asphalt layers. Thus, it is very important to estimate the average stresses for the different sub-layers. These have been found to be dependent on the tire contact pressure (σ_0), the ratio of the radius of contact and sub-layer thickness, Poisson's ratio, and the ratio between the moduli of the various layers or sub-layers. In addition to these, the average stress (σ_{avg}) has proven to be dependent on pavement temperature (23). The average stress can be expressed as the product of a proportionality factor and the contact stress (σ_0). This factor has been calculated (24) by running the elastic layer program "BISAR" along with the relevant input parameters. Based on linear elasticity, upon application of load, the reduction in layer thickness was found to be proportional to σ_{avg}. Thus, the relation between σ_{avg} and σ_0 can be expressed as:

$$Z_i = \frac{\delta_i/h_{1-i}}{\sigma_0/E_{1-i}} = \frac{\delta_i/h_{1-i} \times E_{1-i}}{\sigma_0} = \frac{\sigma_{avg_{1-i}}}{\sigma_0}$$



NOTE: The top scale on the graph corresponds to the top curve. Likewise, the bottom scale corresponds to the extrapolated curve on the bottom of the figure.

Figure 21. Stiffness curve for District 11 sand mix, (Mix 2).

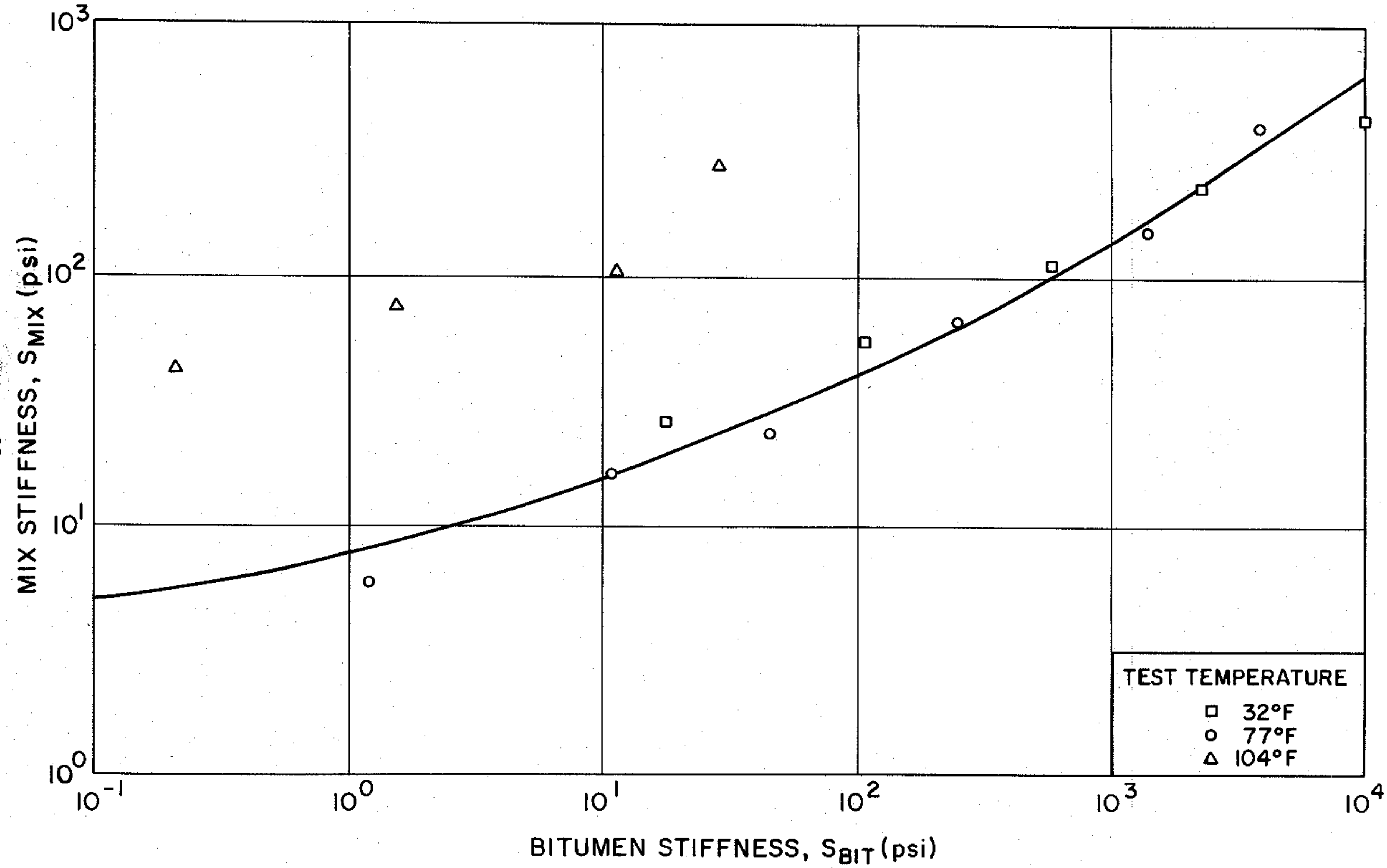


Figure 22. Stiffness curve for higher binder content District 11 sand mix, (Mix 3).

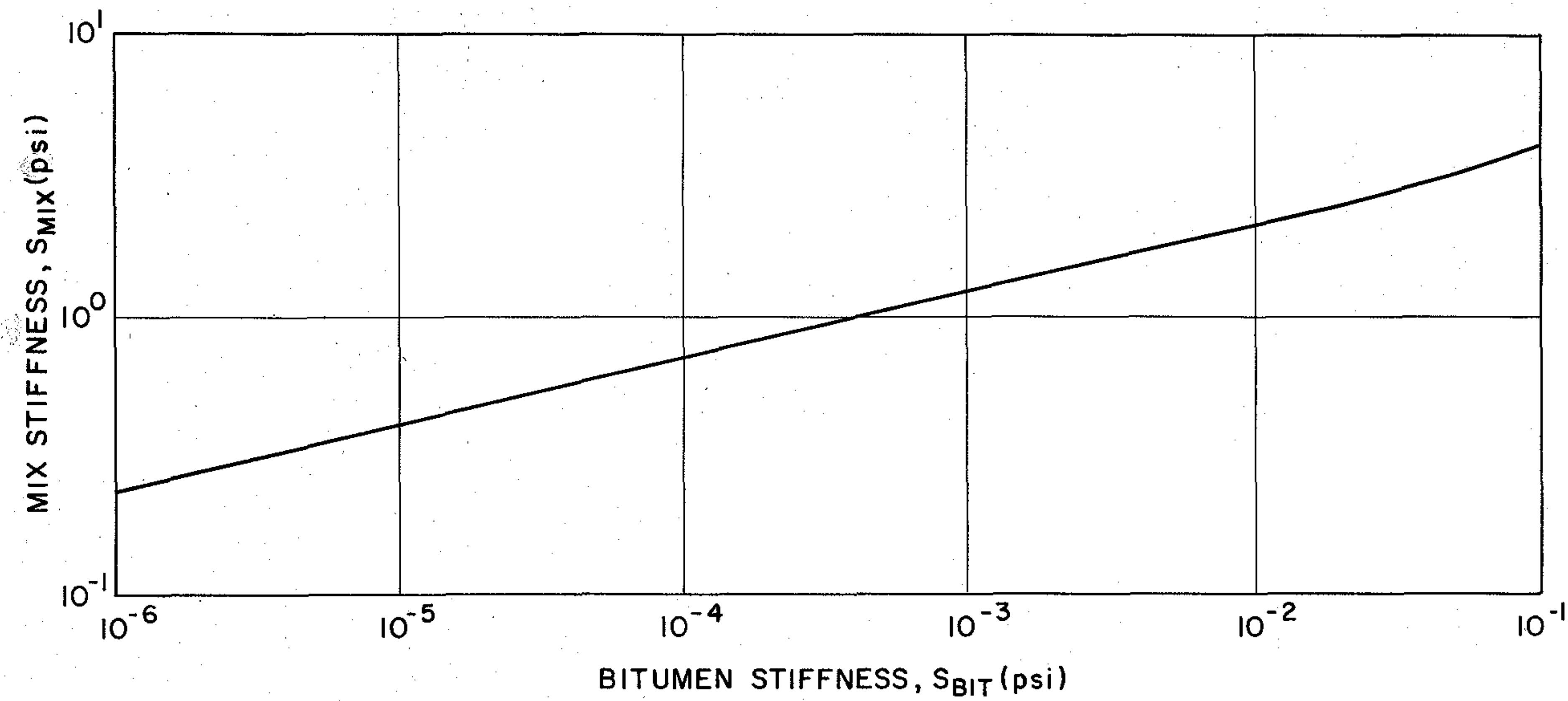


Figure 23. Extrapolated stiffness curve for Mix 3.

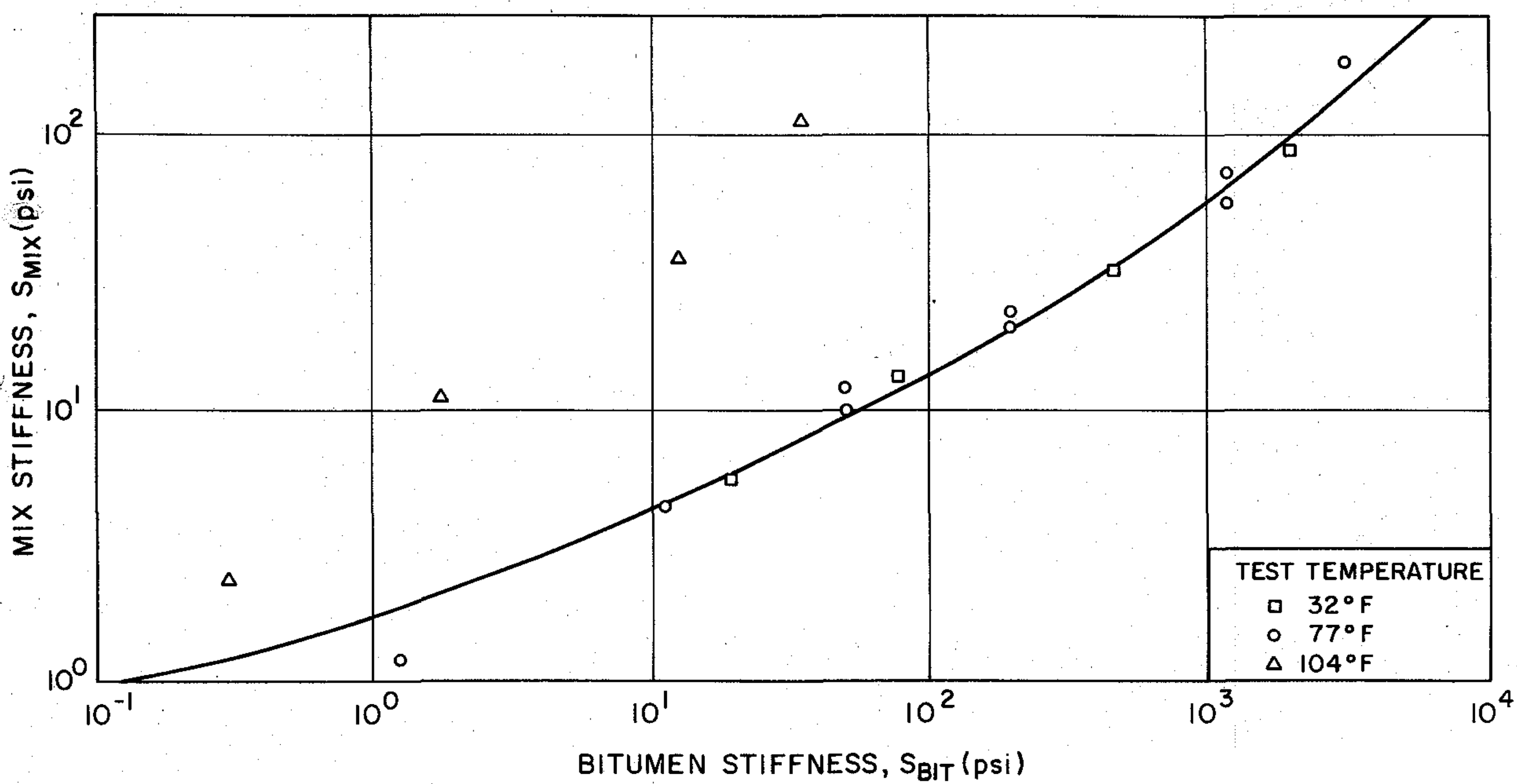


Figure 24. Stiffness curve for District 16 sand mix, (Mix 4).

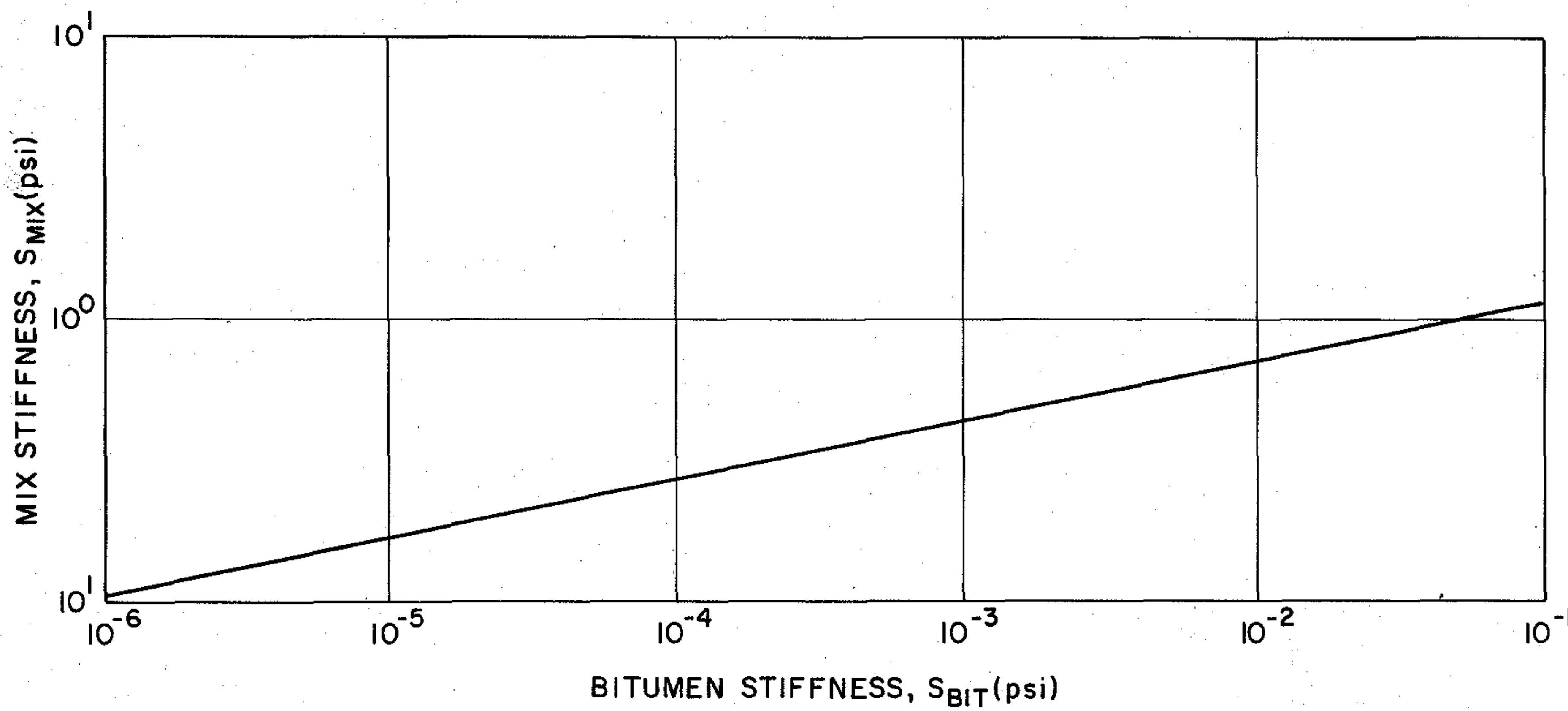


Figure 25. Extrapolated stiffness curve for Mix 4.

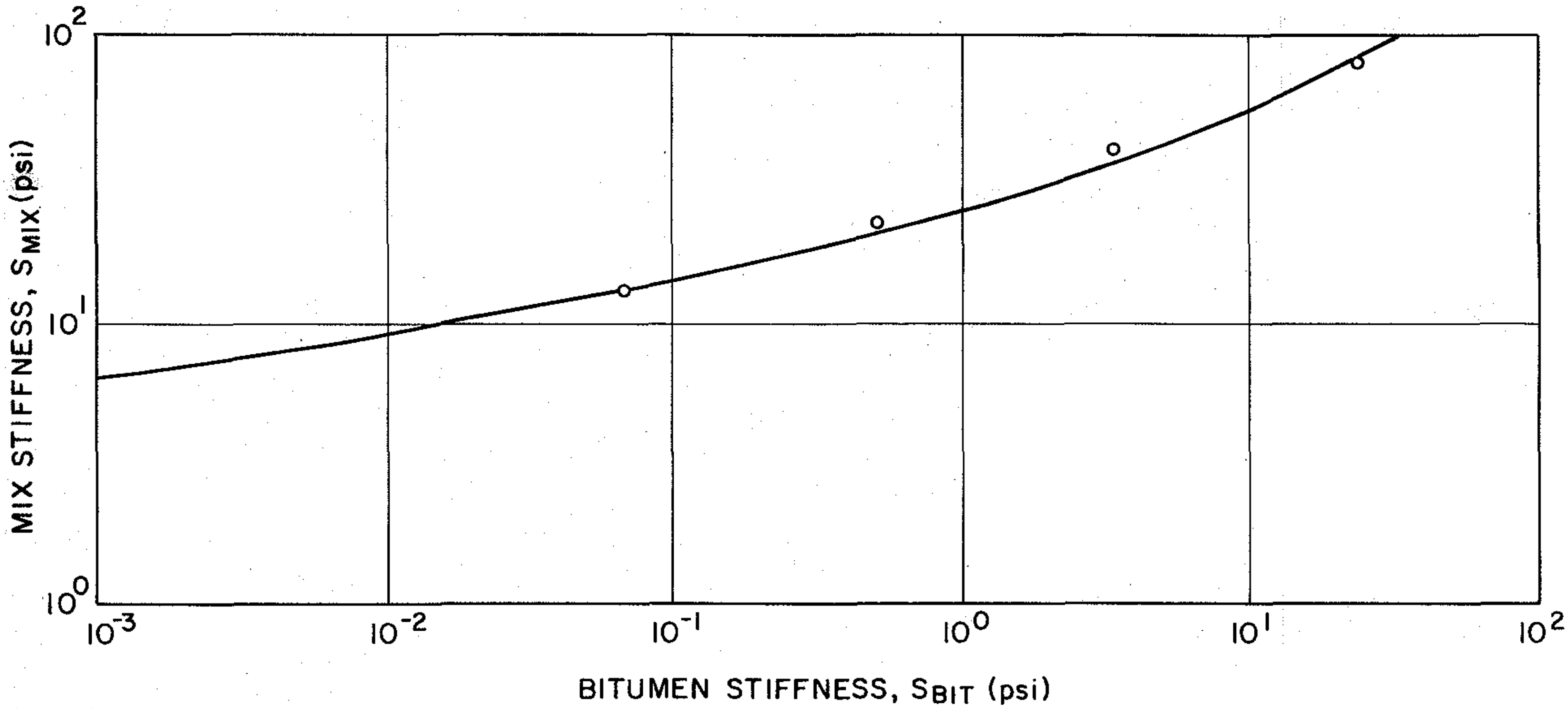


Figure 26. Stiffness curve for saturated District 11 sand mix, (Mix 2).

where,

Z_i = proportionality factor between $\sigma_{avg_{1-i}}$ and σ_0,

h_{1-i} = thickness of the i-th asphalt sub-layer,

E_1-i = modulus of sub-layer, and

δ_i = deformation of sub-layer i (different between the vertical displacements at the top and bottom of sub-layer i).

The necessary Z_i factors can be determined from tables found in the "Shell Design Manual" (23) for the values of the relevant parameters. It must be emphasized that since Z_i is directly proportional to the rut depth its proper selection is of great importance. Figure 27 is used to determine the input data required for the selection of Z_i factors

5. The final step for the calculation of permanent deformation was the estimation of the rut depth itself. This was accomplished by means of the following equation (23):

$$\Delta h_{1-i} = Cm_i \times h_{1-i} \times \frac{Z_i \ \sigma_0}{S_{mix-i}}$$

where,

σ_0 = contact pressure of the standard wheel (87 psi),

Cm_i = correction factor dependent on mix type which takes account of the differences between static (creep) and dynamic (rutting) behavior (for asphalt-sand mixes, Cm_i is about 1.6), and

Δh_{1-i} = reduction in sub-layer thickness.

The total deformation of each pavement structure being considered is,

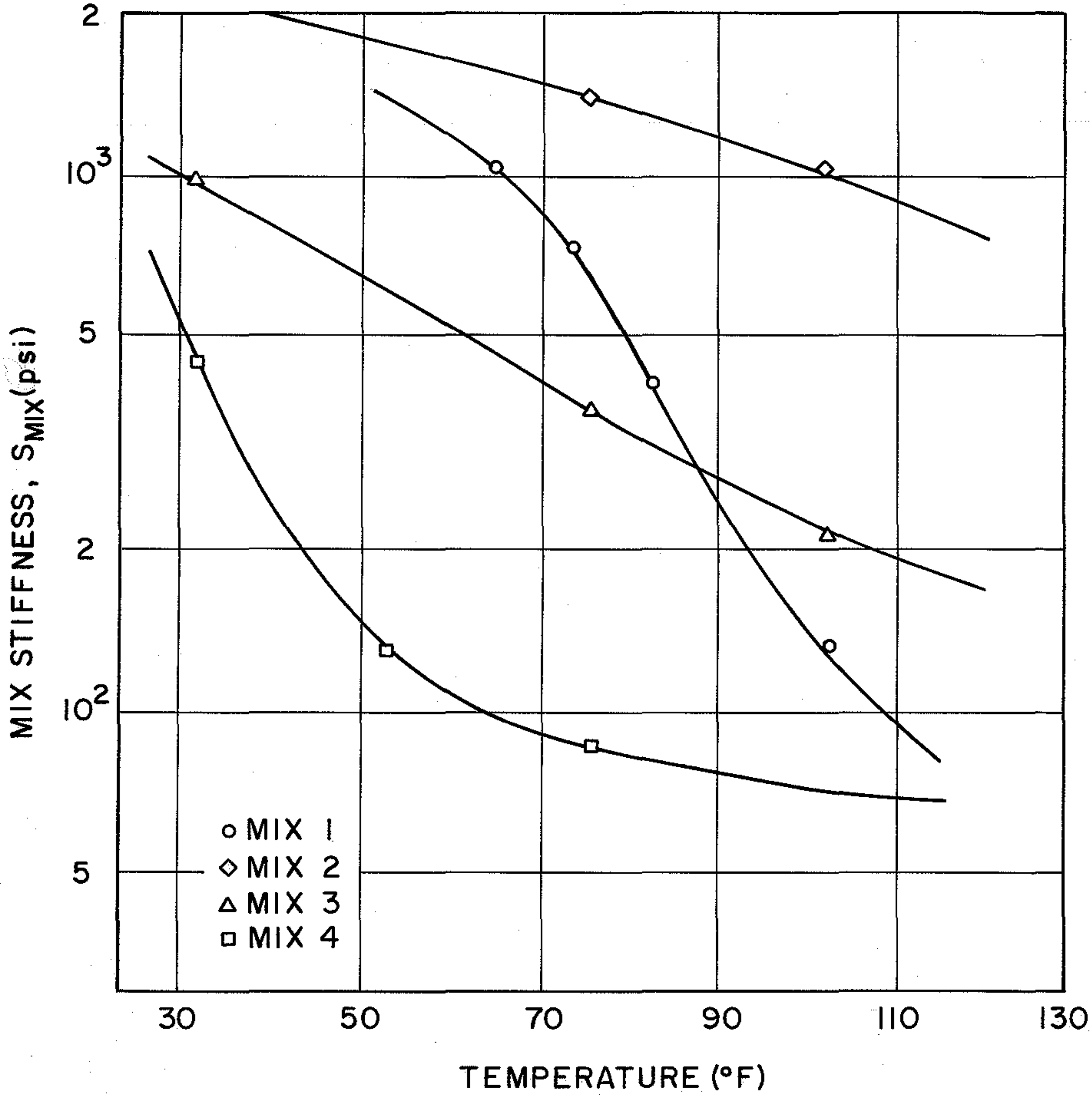


Figure 27. Relationship between mix stiffness and mix temperature for a loading time of 0.03 secs.

$$\Delta h_{tot} = \sum_{i=1}^{3} \Delta h_{1-i} \quad .$$

Some kind of performance criteria in terms of maximum acceptable rut depth needs to be established in order to reject pavements not complying with such requirements. There are no absolute standards under which excessive rutting can be considered as a safety hazard. Vehicle speed, tire wear, porosity of the surface, and wet or dry pavement are among the variables that need to be considered. Nevertheless, some generalizations can be made. Rut depths of less than 0.33 inches normally do not pose any serious problems, but rut depths over 0.67 inches can be serious (36). The following rating was then adopted for this investigation:

- less than 0.1 inches - negligible
- from 0.1 to 0.33 inches - acceptable
- from 0.33 to 0.5 inches - marginal and
- greater than 0.5 inches - unacceptable.

Results

Thickness Design. The effect of different climates on the design thickness can be observed from Figures 15 to 18. As was anticipated, thicker pavements are required for regions having warmer climates, all other conditions being constant. In most instances, subgrade strain criteria was the controlling design factor. However, for a high numbers of strain repetitions (traffic), fatigue criteria controlled the design thickness.

It was observed from the beginning that the District 16 sand would not provide a strong full depth pavement. When measuring the resilient modulus (M_R), the Schmidt apparatus could not be used since the deformations induced in the specimens were too high. In order to evaluate the mix, it was necessary to approximate the value of the resilient modulus by using the creep compliance data. The material proved to be "weak" with a resilient modulus of about 43,500 psi at 77^oF, only 14,500 psi higher than the modulus of the strong subgrade considered for this investigation. The approach was then to determine a full depth pavement thickness using mix 4 (Padre Island) as a base and the control mix as a surface course.
Using the concept of substitution ratios (reference 21), Figures 20 and 24 were used to develop a full depth thickness design where both mixes 1 and 4 were included as surface and base, respectively. The void content of this sand-asphalt mix was so high (27%) that only subgrade strain criteria were considered for pavement design.

Creep Behavior. Comparing the stiffness curves for mix 2 and mix 3, (Figures 21 and 22), shows that mix 2 is considerably stiffer. The only difference between the two materials is the binder content and the air voids. Both were tested at similar stress levels (20 psi) which insured that the specimens responded within the linear visco-elastic range. Yet another difference between the stiffness curves is that, in the case of mix 2, the curve tends to flatten at low values of S_{mix}. In the case of mix 3, the curve continues to decrease and retains a relatively steep slope. It also shows a tendency to flatten but at smaller values of S_{mix} and S_{bit}. To explain these differences in creep behavior, it is necessary to go back to the theoretical model developed by Hills (28). This behavior can be attributed to the increasing contribution of the mineral skeleton to the stiffness of the mix. That is mix 2, a dryer one, tends to develop more contacts at an early stage during the creep test and thus it has a tendency to be stiffer.

Attention was directed toward the fact that mix 2 exhibited stiffness values which are higher than those of the control mix (compare Figures 20 and 21). Higher stiffness does not necessarily mean greater strength. Because District 11 sand contains a high percentage of mineral filler (see Table 10) and the samples appeared to be under-asphalted*, negative capillary tension could be contributing to the high cohesion within the material (37). To evaluate this, any capillary tension that could have existed in the mix was destroyed by vacuum saturating the specimens. Mix 2 specimens were tested following vacuum saturation at lower stress levels to avoid excessive deformation. However, the material still exhibited acceptable creep behavior as indicated by the stiffness curve (Figure 26). Vacuum saturation testing of mix 2 was limited. Therefore, rutting potential was not evaluated for the saturated condition. The results from this experiment suggest that mix 2, if used as part of pavement structures, should be protected from water penetration. The lower stress levels at which the saturated samples were tested also suggest

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*In Figure 22, mix 2 shows a very low temperature susceptibility which can be attributed to the low asphalt content.

that the strength of this mix will be considerably reduced under field conditions, regardless of the high stiffness values obtained. The lower asphalt content of mix 2 may render it more susceptible to fatigue failure.

In the analysis of Padre Island beach sand (mix 4) lower stress levels, in the order of 5 psi, had to be applied for two inter-related reasons: to insure that the test was conducted in the linear visco-elastic range and to avoid specimen failure. The stiffness curve on Figure 24 shows this material is in fact a "soft" one. Compared to the others, mix 4 contains the highest binder content and the highest air voids (about 27%). The rounded nature of the mineral particles may also be responsible for low stiffness values. However, some data (27) indicate that materials showing low stiffness moduli during creep testing, may not necessarily perform unacceptably under field conditions. In the creep test there are no lateral forces acting on the specimen (unconfined). A mix with such high void content and poor gradation will only be able to resist small shear forces during creep testing and will therefore undergo early failure. In a pavement layer, lateral pressure is applied by the surrounding material, thus, as a system it will exhibit higher strength.

It has been stated (27) that most asphalt paving materials respond within the linear visco-elastic range when the creep test is conducted applying a stress of 15 psi for 60 minutes at a temperature of $104^{o}F$. Clearly, this was not the case for mix 4 because creep tests had to be performed at lower temperatures in order to avoid specimen failure. In addition, as the test temperature was increased, the stress levels had to be reduced to insure the specimens responded in the linear range. In Figure 27, the anomalous shape of the curve corresponding to mix 4 indicates that at some point the specimens may not have responded in the linear visco-elastic range as they were undergoing creep testing. This was also reflected in the stiffness curve (Figure 25) where some points plotted at higher S_{mix} values for equal values of S_{bit}.

Rutting Potential. Two extremes were encountered, in the case of the under-asphalted District 11 sand (mix 2), the calculated rutting was negligible (on the order of tenths of a millimeter) independent of the climatic region or the traffic volume associated with each particular pavement. The small magnitude of the rut depth is directly related to the high S_{mix} values used to calculate the reduction in layer thickness.

The estimated rut depth associated with the control mix was also found to be very small. But this time little rutting was expected due to the high density and strength of the mix.

By contrast with the previous mix, District 16 sand (mix 4) when used as a base course, produced unacceptable rutting at high traffic volumes (over 10^5 EAL passes). This was true regardless of the type of subgrade underneath the pavement. District 11 sand at higher binder content (mix 3), when used for a full depth pavement and not just as a base, also showed excessive rutting. Figure 28 presents the results of the rut depth calculations in the form of a bar graph. The rut depth rating established in the preceeding chapter describes the severity of the rutting. The height of the bar does not represent the exact magnitude of the deformation, but indicates the relative performance of a particular mix. Figure 28 represents the results of rutting in Region 1 which is the most deleterious region (see Figure 9). The relative effects of climatic region (weighted annual temperature) on the various mixes can be gleaned from the tabulated rut depth data in Appendix B. It is clear from these data that Mixes 1 and 2 will not rut significantly even in Region 1. Mix 3 will perform unacceptably in Region 1 even under moderate traffic and will also perform unacceptably in Region 3 (the coolest) at high traffic levels. Mix 4 is marginal to unacceptable even when used only as a base in Region 3, the coolest region.

It appears from these data that the District 11 sand can be used successfully throughout Texas in a full depth pavement. The exercise does point out, however, the sensitivity to binder content, traffic and temperature.

Standardization of Creep Tests. To obtain adequate data from the creep test, it is necessary to insure that the factors affecting measured results, and which cannot be eliminated by theoretical means, are secured or controlled by means of standardization.

Two standard features for the creep test were overlooked in the course of this investigation. These were as follows:

1. The end faces of the specimens need to be plane and parallel.

Moreover, they should be able to expand laterally without the fractional restraint, so as to enable a uniform stress distribution to be maintained during the test. Polishing the end faces and the use of a lubricating system such as two rubber membranes with grease between them has proven to give better results (27). A uniform stress distribution is essential so that the Z_i factors used in the calculation of rut depth correspond to the actual stress distribution induced in the pavement layers upon application of wheel loads. However, if the specimens are not polished, and rough contacts exist between the loading plattens and the sample, a short specimen rather than a tall one, will result in greater data scatter (27). In our case, the specimens were tall (8 ± 0.2 in), so that the probable effect of not polishing them is small.

2. Research conducted at the Koninklijke/Shell Laboratory, Amsterdam (27) indicates that the differences in stiffness measurements performed on different test apparatuses can be neglected if the same preloading procedure is adopted. In accordance with this, a preloading stress of about 10 percent of the standard principal stress (15 psi) for a duration of 2 minutes has been recommended. The preloading conditions used in this investigation exceeded the previous recommendation. The use of the full stress level for preconditioning the specimens appears to have influenced the mixes in different ways. In the case of mix 1 and mix 2, a consolidation effect apparently occurred thus increasing the stiffness of the material as the stress was absorbed more and more by the mineral structure. Mix 4,

on the contrary, might have been changed to a state closer to failure and thus, the stiffness of the material was somehow reduced. This explains, in part, the extremes in the rut depth calculated for the different mixes and pavement cross-sections.

ECONOMIC CONSIDERATIONS

This report provides laboratory and field determinations of resilient moduli of several sand asphalt mixtures typical of those used in Texas. Either these representative moduli or measured moduli for a specific mix may be used in Figure 6 to predict structural coefficients for asphalt stabilized sands used as base courses. These structural coefficients may in turn be used in the AASHTO flexible pavement design equation (38) to predict acceptable pavement layer thicknesses under various conditions of subgrade support, traffic, environment and pavement type.

Base course thickness requirements determined from the AASHTO procedure may then be used together with site specific cost data to compare costs among base course material alternatives.

Legend: Mix 1 (Control Mix)
Mix 2 (District 11 Asphalt Emulsion-Sand Mix)
Mix 3 (District 11 Asphalt Emulsion-Sand Mix with Higher Binder Content)
Mix 4 (District 16 Asphalt Emulsion-Sand Mix)

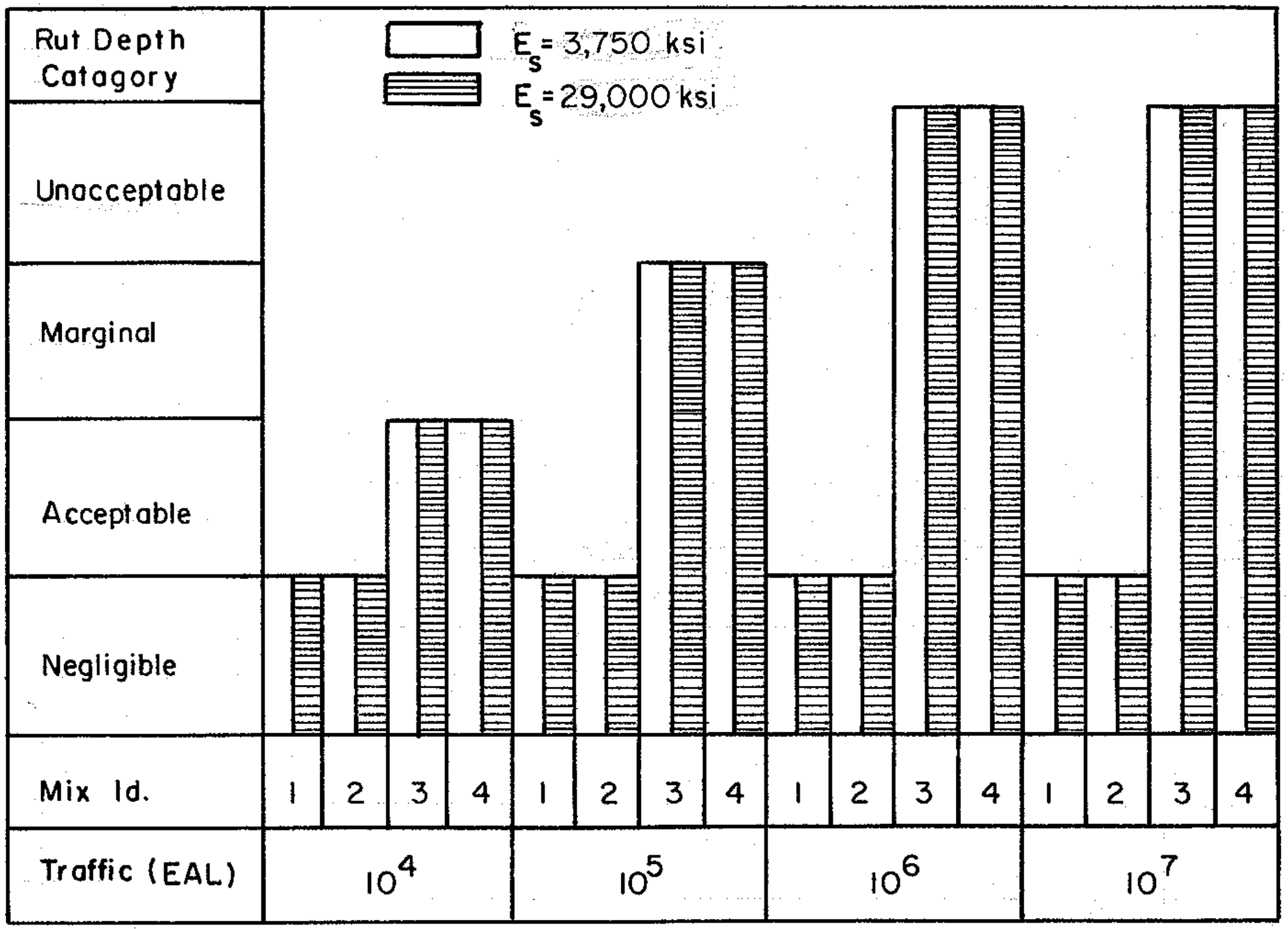

Figure 28. Expected rut depth for marginal mixes tested.

CONCLUSIONS AND RECOMMENDATIONS

Selected Texas blow sands and river sands are suitable aggregate sources for asphalt stabilized bases. Many of these sands produce mixtures with high air void contents and high water absorption; however, suitable resilient moduli, resistance to flow (stability), and resistance to loss of strength in the presence of water may be obtained.

The aggregate selection criteria established by Chevron U.S.A. (1) and Herrin (2) are generally suitable. However, these criteria certainly do not assure a suitable mixture, and thus mixture testing is essential.

The Chevron (1) and Asphalt Institute (3) mixture design methods for bases stabilized with asphalt emulsions appear to have suitable criteria in terms of resistance value following vacuum saturation. However, their criteria for moisture pick-up in asphalt stabilized sands may be unduly restrictive as many of the mixtures evaluated in the study had adequate resilient moduli, resistance to flow, and water-susceptibility behavior but excessive moisture pick-up as determined by the established criteria.

The authors suggest that the diametral resilient modulus test be used to evaluate the structural contribution of the asphalt stabilized sand base to the pavement system. The structural contribution of asphalt stabilized sands can be evaluated by layered elastic analysis. Resilient modulus versus temperature relationships should be developed in the laboratory and used in combination with the Santucci procedure (33) to design the pavement structure.

The in situ derived elastic modulus of asphalt stabilized sands under working stress levels are comparable to elastic moduli of base materials which have demonstrated the ability to effectively protect the subgrade from high contact stresses applied at the surface. The relative contribution of the sand asphalt bases to the structural function of the pavement system is illustrated by the structural layer coefficients. The coefficients are competitive with bases stabilized with cement, lime, and untreated high-quality aggregates such as crushed stone.

Permanent deformation does not appear to be a significant problem for asphalt stabilized sands containing a substantial percentage of fines. The District 11 sand stabilized with cationic emulsion produced a very stiff mix with low rutting potential. On the other hand, the District 16 sand which contained a low percentage of fines (< 2%) had obvious stability and permanent deformation problems. A mineral filler such as cement or fly ash may be required here.

Figure 29 summarizes S_{mix} vs. S_{bit} curves for a variety of asphalt-aggregate systems. As can be seen, the District 11 mix appears acceptable while the District 16 mix is unacceptable.

Although it is evident that vacuum saturation has some deleterious effect on permanent deformation, the effect on the District 11 sand was not catastrophic. Further research should evaluate the effect of saturation on the susceptibility of a sand asphalt mixture to permanent deformation. Of course, proper drainage design is absolutely essential when asphalt stabilized sand bases are used in pavement structures in order to minimize water entry.

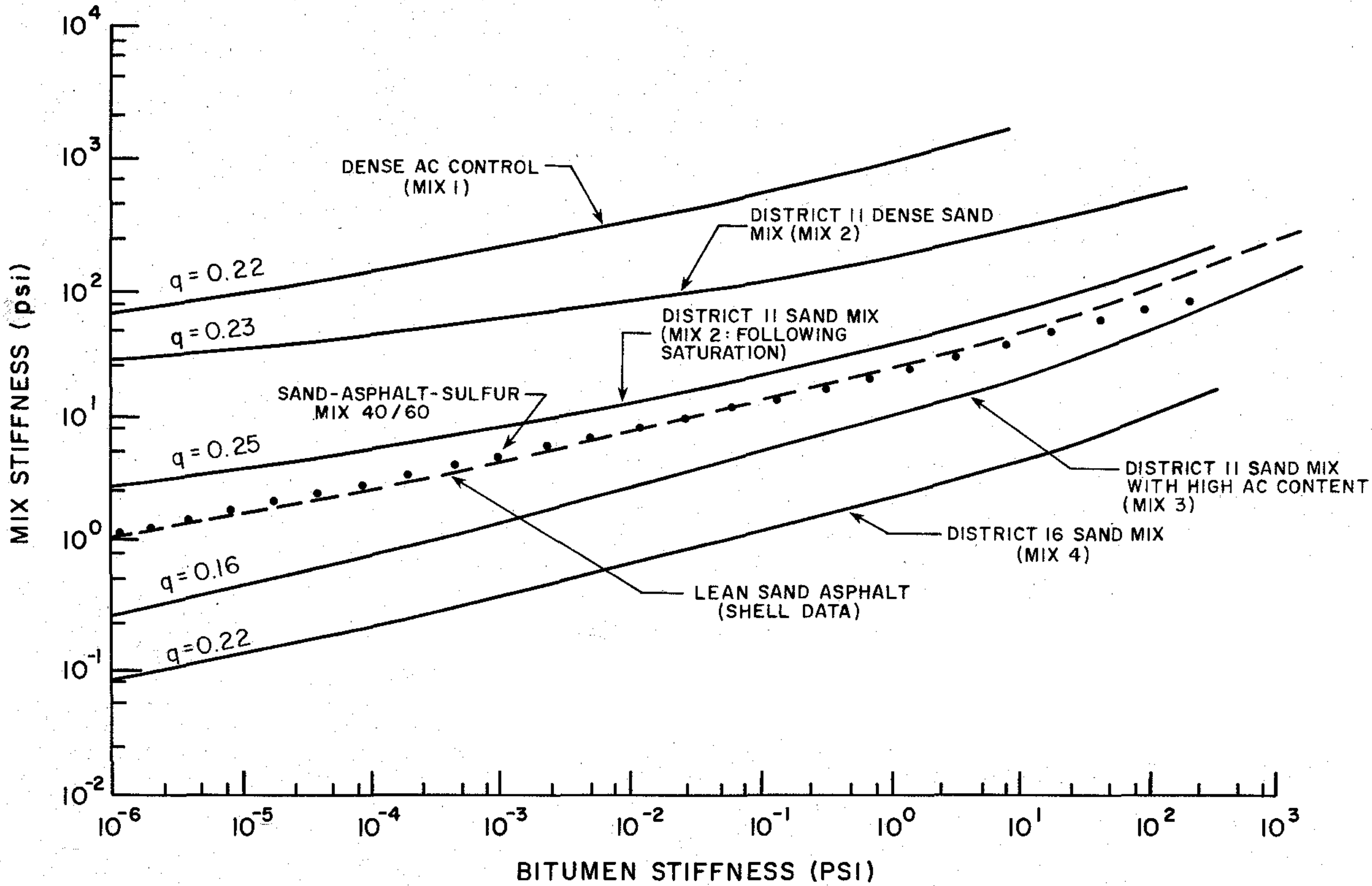

Figure 29. Mixture - Bitumen Stiffness Summary Curves for Emulsion Stabilized and Asphalt Stabilized Mixtures.

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Appendix - A

Derivation of $S_{bit\ visc}$

The relationship between stiffness and time (duration of load) and temperature can be divided into three components. The elastic $S_{bit\ E} = \frac{\sigma}{\varepsilon}$; the time dependent on delayed elastic, $S_{bit\ D} = \frac{\sigma}{\varepsilon(t)}$; and the viscous (long loading times or high temperature), $S_{bit\ visc} = \frac{3\eta}{t}$. The definition of $S_{bit\ E}$ and $S_{bit\ D}$ are rather self explanatory but the definition of $S_{bit,\ visc}$ requires explanation.

Recalling that the shear modulus, G, is defined as:

$G = \tau/\gamma$ and that

$G = \frac{E}{2(1+\mu)}$ ($G = \frac{E}{3}$ for $\mu = 0.5$) and that viscosity, η, is defined as:

$\eta = \tau/d\gamma/dt$.

we may write the following expression:

$$\tau \int dt = \eta \int d\gamma$$

$$\tau t = \eta\gamma \text{ and since } G = \frac{\tau}{\gamma} \sim \frac{E}{3}$$

$$G\gamma t = \eta\gamma \text{ and}$$

$$E = \frac{3\eta}{t}$$

where $E = S$.

Appendix - B

Example Tabulated Rut Depth Calculations

The units of the different quantities appearing in the sample calculation sheets are as follows:

1. H_{1-i} = millimeters (in).
2. $S_{bit,\ visc\ 1-i}$ = psi.
3. $S_{mix\ 1-i}$ = psi.
4. $T_{yeff\ 1-i}$ = °F.
5. E_{1-i} = psi.
6. Z_{1-i} = has no units.
7. Δh_{1-i} = inches.
8. V_{yeff-i} = psi - sec.
9. tw = second.
10. Weq = number of 18,000 axle load equivalents.
11. Cm_i = has no units.
12. σo = psi.

The following equations are referenced in the tabulated calculations sheets.

$$S_{bit,\ visc-i} = \frac{3\ V_{yeff-i}}{tw \times Weq}. \qquad \text{(A-1)}$$

$$\Delta h_{1-i} = Cm_i \times h_{1-i} \times \frac{Zi\ \sigma o}{S_{mix-i}}. \qquad \text{(A-2)}$$

Mix 1, (Control Mix)

Climatic Region 1

MAAT = 71.6^{o}F

Reference	Subgrade	E_s = 3625 psi			
	Traffic (EAL)	10^4	10^5	10^6	10^7
Asphalt Layer Subdivision	h_{1-1}	1.57	1.57	1.57	1.57
	h_{1-2} (in.)	1.57	1.57	1.57	1.57
	h_{1-3}	3.54	5.11	8.85	14.56
Equation (A-1), Figure 19	$S_{bit, visc. 1-1}$ (psi)	0.005	.00005	.00005	.000005 x .000145
	$S_{bit, visc 1-2}$	0.008	.0008	.00008	.000008
	$S_{bit, visc 1-3}$	0.0181	.002	.0003	.00004
Figure 20	$S_{mix 1-1}$	31900	21750	14500	10150
	$S_{mix 1-2}$ (psi)	34800	21750	15950	11020
	$S_{mix 1-3}$	40600	27550	20300	14210
Table 15	$T_{yeff 1-1}$	108.5	108.5	108.5	108.5
	$T_{yeff 1-2}$ (^{o}F)	104.0	104.0	104.0	104.0
	$T_{yeff 1-3}$	96.8	96.8	95.0	92.3
Figure 27	E_{1-1}	105850	105850	105850	105850
	E_{1-2} (psi)	129050	129050	129050	129050
	E_{1-3}	188500	188500	188500	188500
Shell Design Manual	Z_{1-1}	0.25	0.29	0.45	0.50
	Z_{1-2}	0.45	0.46	0.55	0.60
	Z_{1-3}	0.60	0.55	0.40	0.30

Mix 1, (Control Mix) (Continued)

Reference	Subgrade	E_s = 3625 psi			
	Traffic (EAL)	10^4	10^5	10^6	10^7
Equation (A-2)	Δh_{1-1}	.012	.0024	.005	.008
	Δh_{1-2} (in.)	.002	.003	.006	.009
	Δh_{1-3}	.006	.011	.019	.032
-	Rut Depth (in.)	0.009	0.016	0.030	0.050
	Rating	Negl.	Negl.	Negl.	Negl.

Mix 2, (District 11 Sand)

Climatic Region 1
MAAT = 71.6°F

Reference	Subgrade	E_s = 3625 psi			
	Traffic (EAL)	10^4	10^5	10^6	10^7
Asphalt Layer Subdivision	h_{1-1}	1.57	1.57	1.57	1.57
	h_{1-2} (in.)	1.57	1.57	1.57	1.57
	h_{1-3}	6.10	8.85	13.38	14.76
Equation (A-1), Figure 19	$S_{bit,visc\ 1-1}$	0.005	0.0005	0.00005	0.000005
	$S_{bit,visc\ 1-2}$ (psi)	0.008	0.0008	0.00008	0.000008
	$S_{bit,visc\ 1-3}$	0.0232	0.0028	0.0003	0.00003
Figure 21	$S_{mix\ 1-1}$	101500	75400	53650	37700
	$S_{mix\ 1-2}$ (psi)	110200	81200	58000	40600
	$S_{mix\ 1-3}$	127600	95700	72500	50750
Table 15	$T_{yeff\ 1-1}$	108.5	108.5	108.5	108.5
	$T_{yeff\ 1-2}$ (°F)	104.0	104.0	104.0	104.0
	$T_{yeff\ 1-3}$	95.0	95.0	95.0	95.0
Figure 27	E_{1-1}	957000	957000	957000	957000
	E_{1-2} (psi)	1029500	1029500	1029500	1029500
	E_{1-3}	1131000	1160000	1203500	1232500
Shell Design Manual	Z_{1-1}	0.20	0.40	0.50	0.53
	Z_{1-2}	0.35	0.50	0.57	0.60
	Z_{1-3}	0.55	0.44	0.30	0.28

Mix 2, (District 11 Sand) (Continued)

Reference	Subgrade	E_s = 3625 psi			
	Traffic (EAL)	10^4	10^5	10^6	10^7
Equation	Δh_{1-1}	.001	.0012	.0024	.003
	Δh_{1-2} (in.)	.001	.0016	.0024	0.09
	Δh_{1-3}	.005	.006	.008	.008
-	Rut Depth (in.)	0.007	0.008	0.013	0.019
	Rating	Negl.	Negl.	Negl.	Negl.

Mix 3, (District 11 Sand at Higher Asphalt Content) Climatic Region 1

MAAT = 71.6°F

Reference	Subgrade	E_s = 3625 psi			
	Traffic (EAL)	10^4	10^5	10^6	10^7
Asphalt Layer Subdivision	h_{1-1} (in.)	1.57	1.57	1.57	1.57
	h_{1-2}	1.57	1.57	1.57	1.57
	h_{1-3}	6.10	9.84	14.76	20.86
Equation (A-1), Figure 19	$S_{bit,visc\ 1-1}$ (psi)	0.005	0.0005	0.00005	0.000005
	$S_{bit,visc\ 1-2}$	0.008	0.0008	0.00008	0.000008
	$S_{bit,visc\ 1-3}$	0.0232	0.0029	0.0004	0.00005
Figures 22 & 23	$S_{mix\ 1-1}$ (psi)	1740	870	478	290
	$S_{mix\ 1-2}$	1885	1015	536.5	333.5
	$S_{mix\ 1-3}$	261000	1450	797.5	464
Table 15	$T_{yeff\ 1-1}$ (°F)	108.5	108.5	108.5	108.5
	$T_{yeff\ 1-2}$	104.0	104.0	104.0	104.0
	$T_{yeff\ 1-3}$	95.9	95.0	91.4	89.6
Figure 27	E_{1-1}	203000	203000	203000	203000
	E_{1-2}	217500	217500	217500	217500
	E_{1-3}	261000	261000	275500	290000
Shell Design Manual	Z_{1-1}	0.25	0.40	0.52	0.60
	Z_{1-2}	0.50	0.55	0.60	0.65
	Z_{1-3}	0.50	0.40	0.30	0.20

Mix 3, (Continued)

Reference	Subgrade	E_s = 3625 psi			
	Traffic (EAL)	10^4	10^5	10^6	10^7
Equation (A-2)	Δh_{1-1}	.031	0.098	.236	0.455
	Δh_{1-2} (in.)	.055	0.118	.244	.425
	Δh_{1-3}	0.161	0.378	.772	1.252
-	Rut Depth (in.)	0.248	0.594	1.252	2.091
	Rating	Accept	Marg.	Unaccpt.	Unaccpt.

Mix 3, (District 11 Sand at Higher Asphalt Content) Climatic Region 1
MAAT = $58.1^{o}F$

Reference	Subgrade	E_s = 3625 psi			
	Traffic (EAL)	10^4	10^5	10^6	10^7
Asphalt Layer Subdivision	h_{1-1}	1.57	1.57	1.57	1.57
	h_{1-2} (in.)	1.57	1.57	1.57	1.57
	h_{1-3}	5.51	7.87	12.99	17.71
Equation (A-1), Figure 19	$S_{bit,visc\ 1-1}$	0.031	0.0031	0.00031	0.000031
	$S_{bit,visc\ 1-2}$ (psi)	0.062	0.0062	0.00062	0.000062
	$S_{bit,visc\ 1-3}$	0.1116	0.0123	0.0014	0.00016
Figures 22 & 23	$S_{mix\ 1-1}$	2900	1450	768.5	435
	$S_{mix\ 1-2}$ (psi)	3480	1885	942.5	507.5
	$S_{mix\ 1-3}$	4060	2175	7189	609
Table 15	$T_{yeff\ 1-1}$	93.2	93.2	93.2	93.2
	$T_{yeff\ 1-2}$ (^{o}F)	87.8	87.8	87.8	87.8
	$T_{yeff\ 1-3}$	83.3	82.4	80.6	78.8
Figure 27	E_{1-1}	261000	261000	261000	261000
	E_{1-2} (psi)	290000	290000	290000	290000
	E_{1-3}	319000	319000	333500	348000
Shell Design Manual	Z_{1-1}	0.20	0.30	0.50	0.50
	Z_{1-2}	0.45	0.50	0.60	0.65
	Z_{1-3}	0.50	0.40	0.30	0.25

Mix 3 (Continued)

Reference	Subgrade	E_s = 3625 psi			
	Traffic (EAL)	10^4	10^5	10^6	10^7
Equation (A-2)	Δh_{1-1}	.016	.043	0.142	.252
	Δh_{1-2} (in.)	.028	.059	.138	.260
	Δh_{1-3}	.094	.201	.455	1.01
-	Rut Depth (in.)	0.138	0.303	0.732	1.524
	Rating	Negl.	Accept	Unaccpt	Unaccpt

Mix 4, (District 16 sand)

Top 80 mm control mix (except where indicated by *)

Climatic Region 3

MAAT = 58.1 ^{o}F

Reference	Subgrade	E_s = 3625 psi			
	Traffic (EAL)	10^4	10^5	10^6	10^7
Asphalt Layer Subdivision	h_{1-1}	1.57	1.57	1.57	>610 Not
	h_{1-2} (in.)	1.57	1.57	1.57	Economical
	h_{1-3}	3.94	7.87	12.99	↓
Equation (A-1), Figure 19	$S_{bit,visc\ 1-1}$	0.031	0.0031	0.00031	↓
	$S_{bit,visc\ 1-2}$ (psi)	0.062	0.0062	0.00062	↓
	$S_{bit,visc\ 1-3}$	0.1015	0.012	0.0014	↓
Figures 24 & 25	$S_{mix\ 1-1}$	43500	30450	20300	↓
	$S_{mix\ 1-2}$	50750	33350	23200	↓
	$S_{mix\ 1-3}$	1029.5	696	420.5	↓
Table 15	$T_{yeff\ 1-1}$	93.2	93.2	93.2	↓
	$T_{yeff\ 1-2}$ (^{o}F)	87.8	87.8	87.8	↓
	$T_{yeff\ 1-3}$	85.1	82.4	80.6	↓
Figure 27	E_{1-1}	246500	246500	246500	↓
	E_{1-2} (psi)	319000	319000	319000	↓
	E_{1-3}	58000	65250	65250	↓
Shell Design Manual	Z_{1-1}	0.0	0.20	0.30	↓
	Z_{1-2}	0.60	0.60	0.70	↓
	Z_{1-3}	0.50	0.30	0.20	↓

Mix 4 (Continued)

Reference	Subgrade	E_s = 3625 psi			
	Traffic (EAL)	10^4	10^5	10^6	10^7
Equation (A-3)	Δh_{1-1}	0.0	.001	.0024	↓
	Δh_{1-2} (in.)	.002	.003	.005	↓
	Δh_{1-3}	.299	.531	.969	↓
-	Rut Depth (in.)	0.303	0.547	0.976	-
	Rating	Accept	Marg.	Unaccpt.	-

Appendix C

Aggregate Evaluation Criteria

a. Herrin

b. Chevron

Table C1. Properties of Materials Suitable for Bituminous Stabilization (After Herrin - 2)

% Passing Sieve	Sand-Bitumen	Soil-Bitumen	Sand-Gravel Bitumen
1-1/2"			100
1"	100		
3/4"			60-100
No. 4	50-100	50-100	35-100
No. 10	40-100		
No. 40		35-100	13-50
No. 100			8-35
No. 200	5-12	Good - 3-20 Fair - 0-3 & 20-30 Poor - >30	0-12
Liquid Limit		Good - <20 Fair - 20-30 Poor - 30-40 Unusable - >40	
Plasticity Index	10	Good - <5 Fair - 5-9 Poor - 9-15 Unsuable - >12-15	10

Chevron

Construction Specification B-5-A
Central Plant Mix Bitumuls Base Treatment

1.0 DESCRIPTION

Central plant mix Bitumuls Base Treatment is a cold mixed, cold laid base course mixture of mixing Type Bitumuls and suitable aggregate used for highways, streets, roads, airport runways, parking areas, storage yards and similar paved areas. The aggregate may be any non-cohesive inert material meeting the specified gradation and test criteria. The base course materials are mixed in a central mix plant and hauled to the project and laid by acceptable spreaders, conventional pavers, or base pavers.

2.0 MATERIALS

2.1 Aggregate may be any suitable sand, blast furnace, slag, coral, volcanic cinder, gravel, ore tailings, crushed ledge stone or rock, or other inert mineral which will meet the gradation, stability and test criteria as outlined in Table I.

TABLE I

AGGREGATES SUITABLE FOR TREATMENT WITH BITUMULS EMULSIFIED ASPHALTS

Category	AASHO or ASTM Test Method	Processed* Dense Graded Aggregates	SANDS			Semi-Processed Crushed, Pit or Bank Run Aggregates
			Poorly Graded	Well Graded	Silty Sands	
Gradation: 1-1/2"	C-136	100				100
% Passing 1"		90-100				80-100
3/4"		65-90				-
1/2"		-	100	100	100	-
# 4		30-60	75-100	75-100	75-100	25-85
16		15-30	-	35-75	-	-
50		7-25	-	15-30	-	-
100		5-18	-	-	15-65	-
200		4-12	0-12	5-12	12-25	3-15
Sand Equivalent, %	D-2419	30 Min.	30 Min.	30 Min.	30 Min.	30 Min.
Plasticity Index	D-424	-	NP	NP	NP	-
Untreated Resistance R Value	T-190	78 Min.	60 Min.	60 Min.	60 Min.	60 Min.
Loss, L.A. Rattler (500 Revs.)	T-97	50 Max.	-	-	-	60 Max.

*Must have at least 65% by weight Crushed Particles

2.2 Bitumuls Emulsified Asphalt

The class, type, and grade of Bitumuls Emulsified Asphalt selected shall meet the requirement as specified in Table II.

Construction Specification B-5-B
Travel Plant Mix Bitumuls Base Treatment

1.0 DESCRIPTION

Travel plant mix Bitumuls Base Treatment is a cold mixed, cold laid base course mixture of mixing Type Bitumuls and suitable aggregate used for highways, streets, roads, airport runways, parking areas, storage yards and similar paved areas. The aggregate may be any non-cohesive inert material meeting the specified gradation and test criteria. These base course aggregates are mixed by the travel plant and are then either laid down in a continuous windrow for spreading or are continuously spread out mechanically into a uniform, level mat. The travel plant meters and proportions the aggregate and the emulsion in a confined pug mill mixer. The travel plant may be either of two general types: one type mechanically picks up the aggregate from a prepared windrow; the other type is fed by dumping the aggregate (by dump truck) directly into the receiving hopper of the travel plant.

From an air pollution and environmental point of view, Bitumuls travel plant mixes have been very satisfactory. There is a minimum of noise, dust, smoke or fumes generated because the paving mixture is produced from aggregates which are damp or moist and, therefore, almost dustless. The Bitumuls Emulsified Asphalt for the cold travel plant paving mixtures is not hot enough to create any objectionable odors, fumes, or smoke. For small rural or scattered projects, travel plants, such as the Moto-Paver, or the Midland Mixer-Paver, seem well adapted.

2.0 MATERIALS

2.1 Aggregate may be any suitable sand, blast furnace slag, coral, volcanic cinder, gravel, ore tailings, crushed ledge stone or rock, or other inert mineral meeting the gradation, stability and test criteria outlined in Table I.

TABLE I

AGGREGATE SUITABLE FOR
TREATMENT WITH BITUMULS EMULSIFIED ASPHALTS
IN TRAVEL PLANTS

Category	ASTM Test Method	Processed* Dense Graded Aggregates	SANDS			Semi-Processed Crusher, Pit or Bank Run Aggregates	Processed Commercial Aggregate (1)
			Poorly Graded	Well Graded	Silty Sands		
Gradation: % Passing 1½"	C-136	100				100	100
1"		90-100				80-100	80-100
3/4"		65-90				-	-
1/2"		-	100	100	100	-	-
# 4		30-60	75-100	75-100	75-100	25-85	-
16		15-30	-	35-75	-	-	-
50		7-25	-	15-30	-	-	-
100		5-18	-	-	15-65	-	0-3
200		4-12	0-12	5-12	12-25	3-15	0-1
Sand Equivalent, %	D-2419	30 Min.	30 Min.	30 Min.	30 Min.	30 Min.	-
Plasticity Index	D-424	-	NP	NP	NP	-	-
Untreated Resistance R Value	T-190	78 Min.	60 Min.	60 Min.	60 Min.	60 Min.	-
Loss L.A. Rattler (500 Revolutions)	T-96	50 Max.	-	-	-	60 Max.	40 Max.

*Must have at least 65% by weight crushed particles.

(1) Notes:

(a) The Processed Commercial Aggregate shown in Table I is normally a graded aggregate with essentially no material passing the No. 200 sieve and having a nominal top size of 3/4". (For example, ASTM sizes 6, 7, or 8 or combinations of these sizes would be a suitable grading.)

(b) Normally when the graded processed commercial aggregate with substantially no material passing the No. 200 sieve is used in the travel plant, the emulsion selected will be the coarse mixing, CM-h or CM-Kh grades.

Construction Specifications B-5-C
Mixed-In-Place
Bitumuls Base Treatment

Chevron

1.0 DESCRIPTION

1.1 Mixed-In-Place Bitumuls Base Treatment is a *base* course mixture of emulsified asphalt and in-place aggregate. Mixed-In-Place Bitumuls Base Treatments are used for streets, roads, highways, airport runways and taxiways, parking areas, playgrounds, storage yards, lagoons, athletic tracks, and many other paved areas where the native aggregate in-place is of suitable nature for improvement by bituminous base treatment. In many instances a suitable local aggregate can be economically hauled and spread to strengthen and adjust the grade, at a convenient time, before the in-place mixing operations are started. Surface or wearing courses varying from Chip Seal Coats to Thick Asphalt Concrete may be applied over these base courses.

Bitumuls Mixed-In-Place base course construction procedures provide a most economical and ecologically desirable method of construction. There is a minimum of noise, dust, smoke, or fumes generated because the in-place aggregates are premoistened before or during the mixing. The Bitumuls Emulsified Asphalt for in-place mixing is not heated hot enough to create any objectionable odors, fumes, or smoke.

2.0 MATERIALS

2.1 Aggregate may be suitable native sandy gravels, gravelly sands, sand, blast furnace slag, coral, volcanic cinder, gravel, reclaimed aggregate, ore tailings, crushed ledge stone or rock, or other inert mineral which will meet the gradation, stability, and test criteria outlined in Table I.

TABLE I
AGGREGATES SUITABLE FOR IN-PLACE BASE TREATMENT WITH BITUMULS EMULSIFIED ASPHALTS

Category		AASHO or ASTM Test Method	Hauled-In or Select or Graded Aggregates	SANDS			Semi-Processed Crushed, Pit or Bank Run, or In-Place Aggregates
				Poorly Graded	Well Graded	Silty Sands	
Gradation:	1-1/2"	C-136	100				100
% Passing	1"		90-100				80-100
	3/4"		65-90				-
	1/2"		-	100	100	100	-
	# 4		30-60	75-100	75-100	75-100	25-85
	16		15-30	-	35-75	-	-
	50		7-25	-	15-30	-	-
	100		5-18	-	-	15-65	-
	200		4-12	0-12	5-12	12-25	3-15
Sand Equivalent, %		D-2419	30 Min.	30 Min.	30 Min.	30 Min.	30 Min.
Plasticity Index		D-424	-	NP	NP	NP	-
Untreated Resistance R Value		T-190	78 Min.	60 Min.	60 Min.	60 Min.	60 Min.
Loss in L.A. Rattler (after 500 revolutions)		C-131	50 Max.	-	-	-	-

2.2 Bitumuls

The class, type, and grade of Bitumuls Emulsified Asphalt selected shall meet the requirement as specified in Table II.