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Research performed in cooperation with DOT, FHWA.
Research Study Title: Asphalt Additives for Increased Pavement Flexibility.

16. Abstract

Research was performed to evaluate asphalt additives as economic treatments to reduce premature cracking without adversely affecting rutting.

Five asphalt additives were selected and evaluated in a comprehensive laboratory test program. Additives selected for evaluation included block copolymer rubber, SBR latex, ethylene vinyl acetate, polyethylene and carbon black. Asphalts two grades softer than that normally used in HMAC from two sources with widely differing properties were blended with each additive. Assessments of the effects of these additives were based on rheological and physicochemical properties of asphalt cement and on mixture stability, stiffness, tensile properties, and resistance to fatigue, thermal cracking, plastic deformation and moisture damage.

Although each additive tested reduced the temperature susceptibility of both asphalts and showed the potential to reduce cracking and/or rutting in asphalt concrete pavements, no additive surfaced as a cure-all.

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ASPHALT ADDITIVES

FOR

INCREASED PAVEMENT FLEXIBILITY

by

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Research Report 471-2F
Research Study 2-9-85-471

Sponsored by

State Department of Highways and Public Transportation
In cooperation with
U.S. Department of Transportation, Federal Highway Administration

November 1987

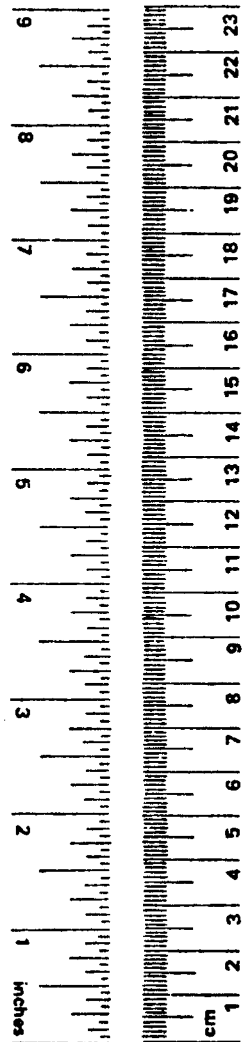
TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, Texas

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

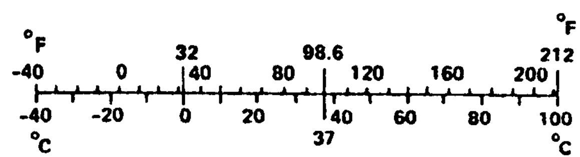
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	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



IMPLEMENTATION STATEMENT

The asphalt additives addressed in this laboratory study appear promising as cost-effective treatments to improve flexibility of hot mixed asphalt concrete pavements and overlays. Controlled field experiments and end-to-end test pavements containing the additives discussed in this report have been installed in southern and north central Texas. Evaluation of these and several other isolated test pavements containing these additives will provide essential information to assess cost-effectiveness. Pavement thickness should not be reduced when additives of these types are employed. Therefore, use of these additives will result in no cost savings during the first year. Cost savings should be realized by extended pavement service life and reduced maintenance.

Design of paving mixtures containing polymeric additives or carbon black may be performend in the usual manner. Mixing and compaction temperatures should be increased, however, to accommodate the higher than usual binder viscosity at the normal mixing and compacting temperatures. In the field, higher than usual mixing and compaction temperatures will also be required to assure adequate coating of the aggregate and densification of the paving mixture.

The use of asphalt additives is greatly simplified when the additive and asphalt are blended prior to arrival at the plant. However, one district in Texas requires that latex be added after introduction of the asphalt and initial coating of the aggregate. This is an attempt to eliminate detrimental changes in properties of latex-modified asphalt that may occur during hot storage. This, of course, requires modifications to the mixing plant. Incorporation of additives in asphalt is discussed in Chapter 6.

Presently, it appears that a generic asphalt additive specification will be impossible to develop. Rather a specification for a particular type of additive will need to be peculiar to that material since the properties of available additives vary so tremendously. Some field experience with additives is needed before significant changes in existing specifications (usually provided by additive suppliers) can be recommended. This experience is being accumulated under Study 187 (Task 5). New specifications regarding an asphalt-additive blend should address additive/asphalt ratio and viscosity temperature susceptibility. Acceptance criteria based on mixture properties should consider minimum increases in tensile strength (indirect tension), stiffness (resilient modulus at temperatures above $70^{o}F$), resistance to creep and permanent deformation and compliance at low temperatures.

Future research efforts should be directed toward development of an acceptable method for extracting modified asphalt cement from paving mixtures to facilitate determination of binder content. This problem may be solved by using the nuclear method to determine binder content. Long-term effects of additive-asphalt compatibility after aging also need to be examined.

DISCLAIMER

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CHAPTER 1

INTRODUCTION

In order to reduce the potential for cracking of asphalt concrete mixtures, at least one of two objectives must be accomplished: (1) increase mixture tensile strength or (2) increase mixture flexibility. Both of these objectives can be accomplished by simply increasing the mixture asphalt content; however, mixture stability will be adversely affected. Mixture tensile strength can also be increased by using harder asphalt, but flexibility will suffer. Softer asphalts will, of course, improve flexibility at the expense of tensile strength and stability which may fall below specified values. To date, these objectives have not been possible simultaneously. However, the advent of new asphalt additives may eliminate the requirement of this historical compromise.

The overall purpose of this research study was to evaluate asphalt additives as economic alternatives to reduce premature pavement cracking. The laboratory test program was designed to examine stiffness, brittleness and flexibility at low temperatures and high loading rates and evaluate the resistance to fatigue-type tensile loads such as those caused by vehicular loading and thermal variations. Increases in flexibility must not, however, be gained at the expense of structural stability.

The primary objective was to evaluate performance of materials added to asphalt concrete mixtures for the purpose of reducing the pavement cracking potential. In reaching this goal, it was necessary (1) to determine the types of distress these materials can economically correct, (2) to develop guidelines which can be used by the Department in the development of specifications for purchase of additives or modified asphalts as well as design, construction and quality control when additives are employed and (3) to compare performance of the additives in the laboratory and in the field.

An asphalt cement additive is defined, in this study, as a material which would normally be added to/or mixed with the asphalt before mix production, or during mix production, to improve the properties and/or performance of the resulting binder and/or mix.

At the start of this study, all known asphalt additives were considered. Funding and time constraints permitted testing of only five additives. The interest lay primarily in products that would, immediately, upon addition to asphalt concrete, alter the mechanical properties. Materials marketed as purely anti-stripping or antioxidant additives were eliminated from the study. A substantial amount of research has been performed in Texas on paving mixtures containing sulfur, hydrated lime, tire rubber, fibers and Chemkrete, therefore, these were also eliminated. The products finally selected for evaluation in the study included:

1. Latex (emulsified styrene-butadiene-rubber),
2. Block Copolymer Rubber (styrene-butadiene-styrene),
3. Ethylene Vinylacetate,
4. Finely dispersed Polyethylene, and
5. Carbon Black

The research consisted of a systematic identification of promising types of asphalt additives designed to reduce cracking (thermal, fatigue, reflective) and plastic deformation (rutting, shoving, corrugations) in asphalt concrete pavements. Asphalt cements with and without additives were tested in the laboratory to determine chemical, rheological, and thermal properties as well as sensitivity to heat and oxidation and compatibility between asphalts and additives. Asphalt concrete mixtures were tested to determine stability, compactibility and water susceptibility as well as stiffness, tensile, fatigue and creep/permanent deformation properties as functions of temperature. State-of-the-art analytical techniques were used in predicting the ability of the additives to reduce pavement distress and prolong pavement service life. Procedures were developed which can be utilized to implement the results of this research during pavement design and construction operations.

A unique opportunity was afforded this study in that a companion study by TTI sponsored by the Federal Highway Administration (FHWA) (1) evaluated the same five additives in the laboratory in a very rigorous experimental program. The overall objectives of the FHWA study were to (1) identify through laboratory testing, the most promising types of additives or admixtures for reducing rutting and cracking in hot-mixed asphalt pavements,

(2) develop guidelines showing how the additives can be incorporated into actual pavements and (3) develop procedures for evaluating additives. Some of the results from that study are summarized herein. These complementary studies were a cooperative effort between the Texas State Department of Highways and Public Transporation (SDHPT) and the FHWA. The FHWA study was essentially a comprehensive laboratory study of the physical and chemical properties of additive-modified asphalt binders. Fatigue and creep properties and resistance to crack propagation of paving mixtures were also quantified over a range of service temperatures and selected mixture properties were used in mathematical models to predict pavement performance. The SDHPT study was primarily a field study. However, certain laboratory tests on binders and paving mixtures were necessary to initiate the field work. In addition, materials identical to those tested in the FHWA research program were used to study the utility of the force ductility, investigate heat stability of modified binders and evaluate tensile properties of paving mixtures over a wide range of temperatures and loading rates. The field work will be presented in a later report.

This report presents findings of laboratory experiments on the binders and binder-aggregate mixtures. Forecasts, using these data with mathematical models and other analytical techniques to predict the effects of the additives on hot mixed asphalt concrete and determine their influence on pavement service life, are given. Methods for implementation of the findings in paving applications are discussed. Detailed data and technical discussions of theory and analytical techniques are given in the Appendices.

Findings from this study clearly show that, to date, no asphalt additive is a panacea. However, for certain conditions of traffic, pavement substrate, asphalt paving materials and climate, the data indicate that certain carefully selected and properly applied asphalt additives have the potential to provide cost-effective extensions to pavement service life.

CHAPTER 2

LITERATURE REVIEW

PURPOSE AND OBJECTIVE

A considerable amount of trade literature and some rather authoritative reports exist under the category of additives in asphalt concrete. In fact, three reports were found to be most thorough and explicit in their treatment of additives and their role in asphalt concrete. These reports are: (1) Study of Asphalt Cement Additives and Extenders, prepared by Pavement Management Systems Ltd. (ref. 2), (2) Road Binders and Energy Savings, prepared by the Organization for Economic Cooperation and Development (ref. 3) and (3) Improvement in Rolled Asphalt Surfacings by the Addition of Organic Polymers (ref. 4). An extensive array of trade literature on the various additives compliments the above listed reports. This literature often incorporates impressive amounts of data from research sponsored by the manufacturer. The bibliography of this report documents trade literature.

The asphalt additive industry is a very dynamic one. New additives and more data enter the scene on almost a daily basis. Therefore, the synthesis of additive data must be a flexible and dynamic operation. The purpose of this literature search is to concisely document the most currently available additive data and to develop a data base through which additives can be compared and evaluated as to their potential to alleviate the pavement distress mechanisms of rutting (permanent deformation) and cracking (both load and thermally induced).

This literature survey presents the most reliable additive data available in a tabular format. The additives addressed in this data base are those which effectively and primarily address the problems of rutting and cracking. The data base was used to select the five additives evaluated in this research program. Since the goal of this research was to evaluate additives which control rutting and cracking in new pavements, the following types of additives were addressed in the literature survey: elastomers, polymers, resins, carbon black, lime, and antioxidants. Primary interest lies in additives which control rutting or cracking by altering the bitumen properties in a favorable way from a chemical and/or rheological standpoint; thus, the search has been restricted to these types of additives. In addition, reclaimed crumb rubber and sulfur were not considered.

ADDITIVE SELECTION CRITERIA

Haas, et al. (2) suggest that the general format illustrated in Figure 1 be used in the evaluation of additives for specific purposes within the pavement system. They also suggest that the aspects of additive degradation at asphalt mixing temperatures and compatability of the additive with the source and grade of asphalt cement must be considered.

ADDITIVES AVAILABLE

A number of available asphalt additives have been categorized by generic name in Table 1. Typical concentrations suggested for use together with the most current information on unit costs are also listed. Obviously, concentrations vary with the source and grade of asphalt being used. This list is by no means exhaustive but is provided as a convenient source of information.

LABORATORY DATA

Data from laboratory studies of additive-modified asphalt paving materials are summerized in Tables 2 and 3. The laboratory data are divided into those derived from additive-bitumen combinations (Table 2) and from additive-asphalt concrete mixture combinations (Table 3).

Generally, the polymer-type additives are shown to reduce temperature susceptibility and binder brittleness and increase ductility, toughness and tenacity. (Toughness and tenacity are measured from areas under stress-strain plots derived from a unique tensile test (32) on the modified binder.) Carbon black in asphalt exhibited a significant increase in

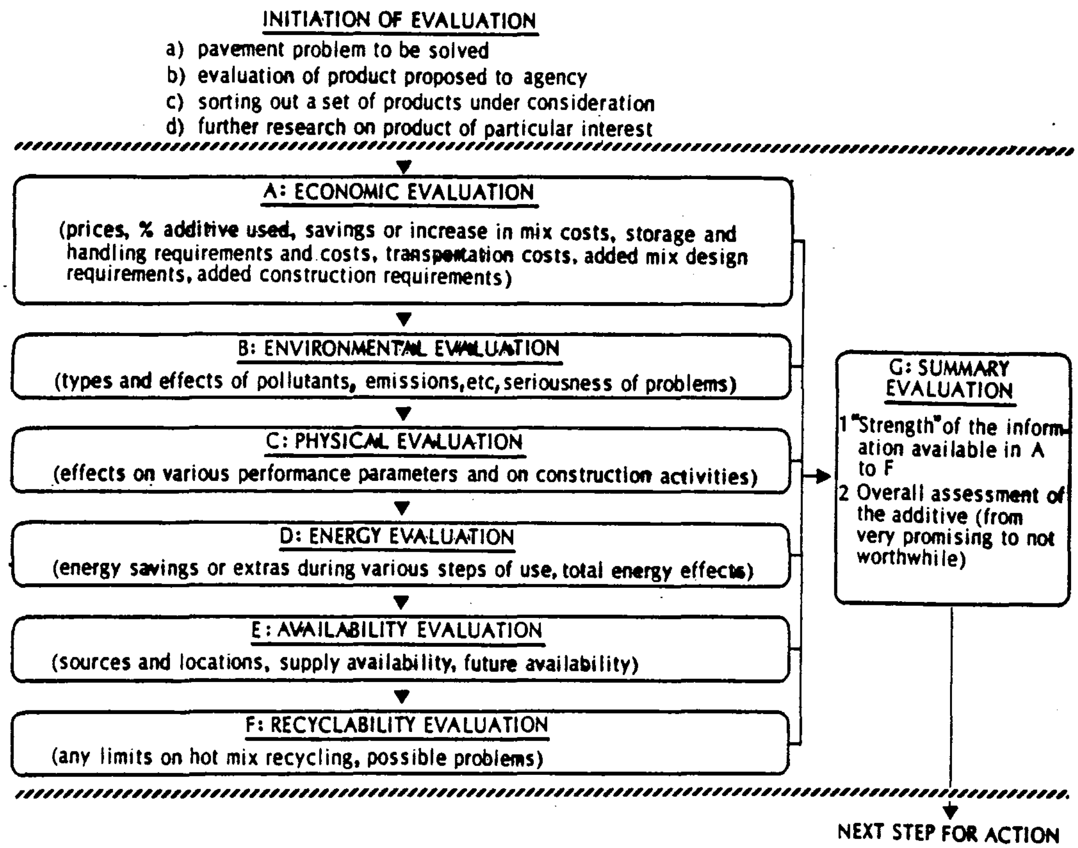

Figure 1. General Framework for Evaluation of Asphalt Cement Additives. (After Haas et al., ref. 2)

Table 1. Asphalt Additives to Control Rutting and/or Cracking in Pavements

CATEGORY	GENERIC NAME	TRADE NAME OR MANUFACTURER	HOW ADDED	SUGG. CONC. (in asphalt)	APPROX. COST INCREASE PER TON HMAC	REFERENCES
Synthetic Rubber Type Copolymers	SBR (Styrene-Butadiene-Rubber)	Dow Chemical Goodyear Polysar Ultrapave Finaprene	Added in a mix plant as a separate stream after addition of asphalt cement	3-5%	----	2,3,6,7,8
	SBS (Styrene-Butadiene-Styrene)	Shell Kraton D	Preblended with asphalt cement using high shear	3-5%	$6-10	2,3,9,10
	SBS (Vulcanized)	Styrelf	Preblended with asphalt cement using high shear	3-5%	$6-10	----
	Neoprene Latex	DuPont	-----	---	----	2
	SEBS (Styrene-Ethylene-Butylene Styrene)	Shell Kraton G	Preblended with asphalt cement using high shear	3-5%	$10-15	2,3,9
	SEPS (Styrene-Ethylene-Propylene Styrene)	None	-----	---	----	9

Table 1. (Continued)

CATEGORY	GENERIC NAME	TRADE NAME OR MANUFACTURER	HOW ADDED	SUGG. CONC. (in asphalt)	APPROX.COST PER TON HMAC	REFERENCES
Polymers	Polyolefins a. Polyethylene	Novophalt 3M-Asphadur	Preblended with asphalt cement using high shear	5%	$5	2,3,11,12, 13,15,16, 17,
	b. Polypropylene	None		----	----	2,3
	Polysulfides Polyisoprenes Polybutenes Polybutylene	None	------	----	----	2,3
	Nylon and polyner resin byproducts	Solar Laglugel	------	----	----	18,19
Copolymers	EVA (Ethylene-Vinyl Acetate)	Exxon-EX 042 DuPont-Elvax	Preblended with asphalt cement using high shear	3-5%	$3-5	4,20
	Unknown	Accorex	Preblended with asphalt shear using high shear	1% by wt. of mix	$20	21
	Polyisobutylene & Polyvinyl acetate EPDM (Ethylene-Propylene-Diene-Monomer)	None	------	----	----	----

Table 1. (Continued)

CATEGORY	GENERIC NAME	TRADE NAME OR MANUFACTURER	HOW ADDED	SUGG. CONC. (in asphalt)	APPROX.COST PER TON HMAC	REFERENCES
Dry Powder	Carbon Black	Cabot-Microfil-8	Batch Plant-Preweighed poly-ethylene bags Drum Plant-High shear blended in asphalt cement with dispersing agent	10-15%	$8-10	22,24
	Hydrated Lime	Several	Slurry on aggre-gate	1% by wt. of mix	$2	----
Organic Metallic Complex	Manganese (exact formulation propietary)	Chemkrete-Lubrisol	Preblended with asphalt cement using low shear	2-4%	$2-5	24,25
Acrylics	------	Rhom & Haas	Note: Not presently marketed as an asphalt additive but may be soon	---	----	----
Anti-Oxidants	Lead and Zinc diethyldithio-carbonate Lead diamyldititho-carbonate Lead and Zinc-dialkyl-dithiocarbonate	------	Preblended with asphalt cement using low shear	1-2%	$2-3	26,27,28, 29,30

Table 2. Summary of Effects of Additives on Asphalt Cement.

TRADE NAME	LABORATORY DATA						
	Base Bitumen	Percent Latex by wt. Asph.	Penetration @ 77°F	$T_{R\&B}$, °F	Fraas Bk. Pt., °F	Plasticity Range, °F	Penetration Index
Latex (SBR) (Ref. 3)	60/70 Bitumen	0%	64	129	3	126	+0.42
		5%	53	145	5	140	+1.76
		10%	42	156	7	149	+2.30

TRADE NAME	Base Bitumen	Percent Latex by Wt. Asph.	Penetration 39.2°F	Penetration 77°F	$T_{R\&B}$, °F	Ductility (39.2°F)
Latex "Dow Downright" (SBR) (Ref. 5)	AC-10	0	23	65	115	2
		3	24	61	129	11
		5	27	56	137	150
	AC-20	0	15	42	118	1
		3	16	37	129	8
		5	18	36	131	14

TRADE NAME	Base Bitu-men	Percent Latex by Wt. Asphalt	Penetration 32°F	Penetration 77°F	Viscosity 140°F	Viscosity 275°F	Ductility (39.2°F)	Sp. Gr. (77°F)	Flash C.O.C.	Brittleness
Latex Goodyear & Polysar (SBR) (Ref. 6)	AC-5	0	4	145	473	1.8	14	1.018	600	56
		2% of pliopave	4	133	776	4.6	141	1.014	600	45
		2% of polysar	12	131	769	4.0	110	1.014	600	39
	AC-10	0	2	95	899	2.5	6	1.023	600	55
		2% of pliopave	3	83	1709	5.3	46	1.019	600	50
		2% of polysar	3	85	1388	4.8	33	1.017	600	50

Table 2. (Continued)

TRADE NAME	LABORATORY DATA

Latex Goodyear Ultrapave (SBR) (Ref. 7)

Base	Condition	Penetration 32°F	Penetration 77°F	$T_{R\&B}$ (°F)	Ductility (39.2°F)	Toughness
85-100 pen	Untreated	25	97	117	10	15
	Treated	27	80	128	150+	90
100-120 pen	Untreated	26	106	112	-	-
	Treated	32	90	125	-	-
120-15- pen	Untreated	38	127	110	-	-
	Treated	35	109	123	-	-

SBS Rubber (Ref. 3)

Base Asphalt	% of Additive by weight	$T_{R\&B}$(°F)	T_{FRASS}(°F)	Plasticity Range	Penetration Index
40/50 Asphalt	0	129	9	120	-0.5
	5%	165	-13	178	+3.5

Shell KRATON D (SBS) (Ref. 9,10)

Base Asphalt	% of Add. by weight	Penetration (77°F)	$T_{R\&B}$(°F)	Ductility (39.2°F)	Toughness	Tenacity	Viscosity 176°F	Viscosity 212°F	Viscosity 248°F	Viscosity 275°F	P.I.	Pen-vis No.
AC-5 (Shell Wood River)	0	164	106	10	26	6	6800	1400	480	-	-	-
	3	84-124	120-161	20-53	53-153	26-117	16800-112500	3300-5250	1000-1300	-	-	-
AC-5 (Exxon)	0	128	112	31	17	10	78	-	-	250	-0.9	-0.9
	3	100	121	98	85	67	560	-	-	570	0.5	0.2
	6	78	193	91	171	141	-	-	-	1675	6.8	1.0

Table 2. (Continued)

TRADE NAME	LABORATORY DATA								
Shell KRATON G (SEBS) (Ref. 9)									

Base Asphalt	% of Add. by weight	Penetration (77°F)	$T_{R\&B}$(°F)	Ductility (39.2 °F)	Toughness	Tenacity	Viscosity		
							175°F	212°F	248°F
AC-5 (Shell Wood River)	0	164	106	10	26	6	6800	1400	480
	3	83-177	126-151	12-21	55-92	16-74	18500-22000	3900-4400	1200-1400

Shell KRATON D (SIS) (Ref. 9)

Base Asphalt	% of Add. by weight	Penetration (77°F)	$T_{R\&B}$(°F)	Ductility (39.2°F)	Toughness	Tenacity	Viscosity		
							176°F	212°F	248°F
AC-5 (Shell Wood River)	0	164	106	10	26	6	6800	1400	480
		97-106	122-135	12-45	62-127	30-96	12600-37000	2700-4100	800-1000

Novophalt (polyethylene) (Ref. 12)

Base Asphalt	% of Add. by weight	Penetration (77°F)	Viscosity				Ductility (39.2°F)	Solubility in Trichloroethylene	Flashpoint °F
			39.2°F	77°F	140°F	275°F			
Unknown	0	83	1434	1420	1362	354	4.7	99.92	615
	Unknown	68	1850	2190	3752	957	3.25	95.72	620

3M Asphadur (polyethylene) (Ref. 13)

Base Asphalt	% of Add. by weight Asp.	Penetration (77°F)	Ductility (77°F)	$T_{R\&B}$(°F)	Viscosity (140°F)
120/150 Pen.	0	68	150+	120	1192
	6	59	125	123	1998

Table 2. (Continued)

Trade Name	Laboratory Data
EVA (Ethylene Vinyl Acetate) (Ref. 2)	

Binder	Pen 77°F	R & B °F	Temperature °C For Viscosity of 2 Poise	Temperature °C For Viscosity of 50 Poise	After RTFOT Pen 77°F	After RTFOT R & B °F
Conventional Bitumen (A)	56	126	174	112	37	142
(94%A + 6% 300pen) + 5% EVA	42	154	184	115	33	165
(78%A + 22% 300pen) + 5% EVA	51	145	178	109	38	158
A + 2% EVA	52	140	181	117	32	154
A + 3.5% EVA	41	147	186	116	29	160
A + 5% EVA	35	158	195	120	26	172

Type	Pen 77°F	R & B °F	Viscosity @ 113°F (poise)
Bitumen	48	131	$9.0x10^4$
Bitumen + 5% EVA	52	147	$3.5x10^5$

Table 2. (Continued)

Trade Name	Laboratory Data
Carbon Black (Ref. 22)	

% Carbon Black Filler	Degree of Erosion After 300 Hrs. of UV and Water Spray
0	Complete erosion in three areas, metal substrate exposed
2	Complete erosion in only one area
15	No exposed metal substrate, same alligator cracks

Temperature F°	Viscosity, poises						
	100% 300-400 pen Asphalt	300-400 pen + 21.2 pha* Microfil 25	300-400 pen + 21.2 pha Microfil 8	100% 150-200 pen Asphalt	150-200 pen + 21.2 pha Microfil 25	100% 85-100 pen Asphalt	85-100 pen + 21.2 pha Microfil 8
140	2.4×10^2	5.0×10^2	3.0×10^3	6.0×10^2	1.0×10^3	1.3×10^3	1.9×10^3
77	8.6×10^4	2.8×10^5	1.6×10^6	3.4×10^5	9.4×10^5	1.8×10^6	5.2×10^6
39.2	2.0×10^7	1.9×10^7	5.5×10^7	6.7×10^7	6.0×10^7	—	—

Asphalt Grades and Blends with Carbon Black Filler	Viscosity at 140°F (poise)	Pen. at 39.2°F 200 g., 60 sec.	Pen. at 77°F 100 g., 5 sec.
300-400 pen 21.2 pha Microfil 8 21.2 pha Microfil 25	240 3030 500	71 52 89	277 163 257
150-200 pen 21.2 pha Microfil 25	600 1020	40 49	148 144
85-100 pen 21.2 pha Microfil 25	1340 1930	25 32	67 72

Table 2. (Continued)

TRADE NAME	LABORATORY DATA							
	Asphalt Grade	% Mn	Pen. 100g 5 sec 0.1mm				Viscosity	
			50°F		77°F		140°F (poises)	140°F poises x 10^3
			Unaged	Aged	Unaged	Aged	Unaged	Aged In Extended RTFOT
Chemkrete (Ref. 24)	AC-2.5	0.00	27	8	200	19	318	207
		0.08	54	8	>330	19	178	250
		0.125	76	8	>330	20	130	199
		0.20	138	7	>330	17	78	1,550
	AC-5	0.00	19	7	128	15	545	-
		0.08	36	6	252	16	303	-
		0.125	51	7	>330	16	225	-
		0.20	95	6	>330	17	120	-
	AC-20	0.00	11	5	50	12	2090	126
		0.08	19	5	98	13	932	404
		0.125	23	5	135	13	575	228
		0.20	44	5	243	13	305	894

Table 2. (Continued)

Trade Name	Laboratory Data
Chemkrete (Ref. 24) (Ref. 25)	See table below
Solar Laglugel (Nylon and Synthetic Resins) (Ref. 18)	See tables below

Chemkrete (Ref. 24) (Ref. 25)

	Arizona		California		Georgia		Illinois		Virginia	
Grade	Control	Chemkrete	Control	Chemkrete	Control	Chemkrete	Control	Chemkrete	Control	Chemkrete
	AR8000	AR4000	AR8000	AR4000	AC20	AC20	AC20	AC20	AC20	AC20
Curing	Roadway 1 month	Roadway 1 month	Unknown	Unknown	None	None	None	None	28 days @ 140°F	28 days @ 140°F
Pen. 77°F 100 g. 5 sec.	17.5	7.8	16	8	90	138	69	103	43	18
Viscosity 140°F poises 275°F CS	19,975 796	300,000+ 2,823	19,594 696	104,284 1290	1955 399	1031 295	1820 345	995 344	5092	108,638
Ductility 77°F CM	100+	1.1	100+	0					105+	6

Solar Laglugel (Nylon and Synthetic Resins) (Ref. 18)

	Viscosity	
	AC20	Treated
@ 140°F poises	1970	2075
@ 275°F CS	310	328

Penetration 77°F 100 g. 5 sec	
AC20	Treated
63	60

*pha = parts per hundred parts of asphalt

Table 3. Summary of Effects of Additives on Asphalt Paving Mixtures

TRADE NAME	LABORATORY DATA				
	Marshall Stability	Flow	Hveem Stability	Compressive Strength (Dry)	Compressive Strength (Wet)
STYRELF (SBS-Vulcanized) (Ref 8)	20% inc.*	0-7% inc.	0-2% inc.	40% inc.	50-70% inc.

TRADE NAME	Marshall Stability	Flow	Fatigue Life	Complex Modulus	Wheel Tracking Rate	Indirect Tensile Strength	Creep	Permanent Deformation, cm
NOVOPHALT (Polyethylene) (Refs. 11,12,14,15)	20-70% inc.	20% inc.	3.2 times inc.	1-3 times inc.	50 pen: >2 Novophalt: < 0.5	1.5-2 times	Significant dec.	4.35→ 0.33(40°C) 1.28→ 0.05(20°C)

TRADE NAME	Marshall Stability	Flow	Indirect Tension Strength @ 140°F	Cold Water Abrasion Test (9)
3M Asphadur (Polyethylene) (Ref 13,16,17)	7-60% inc.	12-23% dec.**	10-70% inc.	30% reduction with 4% of ASPHADUR

Table 3. (Continued)

TRADE NAME	LABORATORY DATA			
ACCOREX (Ref 31)			AC with Accorex	AC Standard
	HVEEM Stab.		27	28
	Resilient Modulus psi x 10^3	104° F	100	40
		77° F	750	540
		33° F	2050	2130
	Indirect Tension @ 77°F Ult. Stress, psi		190	150

TRADE NAME	LABORATORY DATA			
DUPONT ELVAX (EVA) (Ref 20)		Unmodified	95% AC 20 5% Elvax 360	97% AC 20 3% Elvax 360
	Initial Marshall Stabs. lbs.	1155	1175	1007
	After 16 Day Immersion at 140° F Stabs. lbs.	751	1114	980
	% of Initial Stab. after 16 Day Immersion	65.0	98.4	97.3
	% of AC Control Sample After 16 Days	100	148	131

Table 3. (Continued)

TRADE NAME	LABORATORY DATA				
CARBON BLACK (Ref 23)		15 parts/100 asphalt Microfil 8	AR 1000	AR 2000	AR 4000
	Compressive Strengths	Dry % of Control	115	98	149
		After Immersion			
		% of Control	173	181	152
		% of Dry	61	51	70

TRADE NAME		Control	Treated
SOLAR LAGLUGEL (Nylon & Synthetic Resins) (Ref 19)	Marshall Stabs. lbs.	1310	1370
	Tensile Strength, % Retained After Moisture Treatment	45	65
	Stripping Resistance % Asphalt Retained 24 hrs. at 60°C	26	47

Table 3. (Continued)

TRADE NAME	LABORATORY DATA

CHEMKRETE
Ref 24

Location	ARIZONA		COLORADO		OKLAHOMA		WYOMING	
Sample Condition	Control	Chemkrete	Control	Chemkrete	Control	Chemkrete	Control	Chemkrete
Asphalt Grade	AR 4000	AR 4000	AC-10	AC-10	85-100	85-100	AC-10	AC-10
Specimen Curing Methods	28 Days	28 Days	?	?	Road Cores 8 months	Road Cores 8 months	28 Days at 140°F	28 Days at 140°F
Marshall: Stab lbs	3380	7485	--	--	1709	453	2627	4185
Flow 0.01"	15	17	--	--	12	13	10	13
HVEEM: Stability	49	61	29	37	48	54	--	--
Cohesion	373	892	284	310	312	437	--	--
Unconfined Comp. Str. Dry psi	686	1308	451	598	--	--	396	820
Wet psi	453	626	281	478	--	--	257	668

*inc. - additive produces an increase in mixture property of the given quantity.

**dec. - additive produces a decrease in mixture property of the given quantity.

resistance to abrasion; the practical significance of this is not readily apparent.

Additives in mixtures exhibited moderate improvements in Marshall stability and tensile properties but generally no significant increase in Hveem stability. Although Hveem stability is quite sensitive to binder content, it is not very sensitive to binder properties.

As mentioned previously, most of the additives improve the temperature susceptibility of an asphalt. This change in the rheological properties of the asphalt is dependent, of course, on the type of additive and the quantity added. Generally, one can expect an increase in binder viscosity at temperatures above 40^{O}F and no appreciable change in consistency at temperatures below 40^{O}F. Therefore, by using an asphalt one or two grades softer than that normally used in hot mix paving mixtures plus an appropriate additive, one can take advantage of the original low viscosity of the asphalt in the low temperature range to increase resistance to cracking and, simultaneously, depend on the higher viscosity in the high temperature range to increase resistance to rutting.

FIELD DATA

The only disadvantage of the increased viscosity of the modified binders at high temperatures is that it extends into the temperature range at which asphalt concrete is mixed (275-325^{O}F). It is, therefore, often necessary to increase the operating temperature of the mixing plant to achieve adequate coating of aggregate and provide for satisfactory compaction of the mixture. Plant temperature increases from 0 to 70^{O}F have been reported with about 35^{O}F being most usual. Obviously, the required temperature increase will depend upon the type and quantity of additive used. This is, nevertheless, an important consideration for the paving contractor from an economic standpoint, in that more fuel will be required to operate his plant at a higher than normal temperature. It should also be considered when selecting the type and quantity of additive since economic trade-offs may present themselves.

Table 4 contains a brief summary of several field tests that have been installed at different locations in the United States and Europe. Most of the field tests that were documented well enough for inclusion in Table 4 are less than three years old. Although latex and some of the SBS rubber products have been used in asphalt for sealcoats for several years, these and other additives in this category are relatively new to the hot mixed asphalt concrete industry.

Generally, no particular problems have been associated with the placement and compaction of paving mixtures containing additives. However, when placed at the same temperature as a conventional mix, modified mixtures may be noticeably stiffer in that they may not lay as smoothly; on one job, improved workability was attributed to EVA (1). Polyethylene added using the Novophalt process and carbon black appear to resist rutting and shoving in asphalt concrete (4,5,37). Chemkrete was sometimes associated with increased brittleness as manifested by pavement cracking (7,10). In one instance, prolonged hot storage of SBS/SB rubber-modified asphalt resulted in a significant decrease in viscosity which produced a tender mixture; laboratory data indicate this could also occur with latex (SBR) modified asphalts. Additives will increase the in-place cost of hot mixed asphalt concrete by about 10 to 15 percent.

Table 4. Description of Selected Field Tests on Asphalt Additives

Location	Additives Tested	Pavement Section	Summary of Tests and Results
New Jersey Rt 41 & Rt 154	Chemkrete Latex-Dow Solar Lagulel 3M Additive 5990 (polyolefin) Plus Ride (tire rubber) Control Section	1½" top course 1½" binder course 6" stabilized base	Placed in August 1984. New construction. Sections are 1740' LF x 36' wide. After one year in service, all sections are performing well. A few cracks have appeared in the section containing 3M additive which produced relatively stiff lab mixtures. Although rideability is good on all sections, Plus Ride exhibits the worst rideability. Rutting (1/16-1/8") was noted only in the Plus Ride section. Approximately 600 tons of each mix was produced using batch plant. Chemkrete (3.3%) was preblended with AC-10. Laglugel (1.3%) was preblended with AC-20. 3M additive and Plus Ride in preweighed plastic bags were added in pug mill following AC-20 at a rate of 8.3 and 60 lb. per ton of mix, respectively. Latex (3%) was metered into pug mill following AC-20. Mixing temperature for 3M, Plus Ride and latex were increased to about 350°F, and were compacted immediately behind paver. Mix production and paving operations went well for all mixtures.
California IH80 near Monte Vista	Shell Kraton D (SBS) Microfil 8 (carbon black) Latex Ramflex (devulcanized tire rubber) Bonifibers (polyester) Hercules fibers (polyethylene)	3" HMAC Fabric 9" PC Concrete 4" Cement Trt. Base	Overlay installed in June 1985 in a mountainous region. Long haul from plant to construction site required production of extra hot mixtures (320-330°F). SBS plus asphalt at high temperatures for a long period apparently resulted in reduced viscosity of binder and tender mixture during construction. Also polypropylene fibers melted. After 3 months in service pavements are performing well. Carbon black section is exhibiting slight flushing; however, it may be about 0.4% more than the design binder content. Test sections ≈ 2000' in one 12' lane. Used batch plant. Estimated traffic @ 70,000 -- 18 kip EAL.
Bowie, Texas US 287	Chemkrete Control Section	2" HMAC A-R Sealcoat 1½" HMAC Sealcoat 11" Flexbase	Overlay placed in July 1985, 1.86 mi., 1-lane. No construction or early performance problems. Drum mix plant temperatures ranged from 255 to 290°F. Chemkrete was added to AC-5 in a tank truck with low shear blending. No difference in performance to date.
LaGrange, Texas SH 71	Chemkrete Control Section	1¼" HMAC 1½" HMAC Flexbase	Overlay placed in May 1984, 2-mile, 1-lane. Some rain occurred during construction. Asphalt content was too high (5.8% instead of 5.3%) in portions of the test section. Chemkrete was metered in-line into AC-10 prior to entering drum mix plant. Plant temperature about 300°F. Twenty-five percent exhibited excessive rutting and shoving by the middle of the second summer in service. Reconstruction is scheduled for the fall of 1985.
College Station, TX FM 2818	Shell Kraton G (SEBS) Control Section	1½" HMAC 2 Sealcoats 6" Flexbase 8" pit-run gravel 6" lime-stab. subgrade	Overlay placed in Spring 1985. Kraton G was preblended with 120-150 pen asphalt at 3% prior to shipping to plant site. One transport of modified asphalt was utilized. No construction or early performance problems. Modified mix was noticeably stiffer than control mix and did not lay as smoothly; however, no difference after 3 months.

Table 4. (Continued)

Location	Additives Tested	Pavement Section	Summary of Tests and Results
A421 South of Marston Moretaine, United Kingdom	Novophalt (Ref. 34)	Sealcoat	In August 1984, tests were performed on a trial section to assess the performance of the product with regard to rideability, rutting and surface texture after two years of heavy trafficking. The rolling straight edge results satisfied the specifications, and the rideability is good. The surface texture measurements show that there has been little loss of surface texture. The rut depth measurements show no rutting to have taken place in the wheel tracks. The bitumen used was 50 pen grade, straight run containing 4% Novophalt. The handling properties of the material appeared similar to those of "normal" asphalt. The chippings appeared well gripped by the binder. Difficulties were experienced in carrying out tests on binder and also in carrying out analyses when the polyolefins floated in the methylene chloride solution. Mixture was stable at a storage temperature of 320°F. The wheel tracking results indicate that the material is not damaged excessively under heavy traffic. March 1982.
Prater Flyover (section 1220 of the A20 Motorway), Austria	Novophalt (Ref. 35)	Wearing Course	Placed September 1977 with 8% polyethylene. Visual inspections were performed on July and September 1979. 2400 ft long and 90 ft wide. Heavy, high speed traffic. The occasional roughness of the surface is probably due to segregation during the laying process. Adhesion of the chippings to the mortar is excellent. The few cracks that occurred are largely due to the type of the bridge construction. The depth of ruts were only about one third of the rut depth of the next section. The skid resistance measurements did not indicate any significant difference. The increased viscosity of the modified binder would permit an increase of the binder content by 0.5% in absolute terms as compared to conventional asphaltic concrete without any unacceptable deformations. The Marshall values for Novophalt do not differ from the usual values, but the bearing values are approx. 40% higher than the next section. Flow values are accordingly lower and rigidity is twice as high. Because of the high viscosity of the binder, the laying temperature should be 36°F above the usual value. The test results obtained with recovered bitumen explain the high deformation resistance under the influence of heat as well as the diminished susceptibility to cracking. Tensile splitting tests show a substantial improvement of cohesion at higher temperatures. The resistance to dynamic deformation of the Novophalt surfacing is about three times as high as that of conventional asphalts.
Crowthorne, Berkshire	EVA (Ref. 3)	2" HMAC	140 tons of asphalt modified with 5% EVA and 50 tons of a conventional 50 pen bitumen were mixed and placed. Precoated chippings (20 mm) were applied to all the asphalts to provide surface texture. The control asphalt was mixed at 355°F and compacted to a thickness of 2". Seventy tons of the EVA modified 70 pen bitumen was mixed at a temperature of 355°F but attempts to roll this asphalt at 320°F failed. Additional loads of the modified asphalt were allowed to cool to between 195°F and 210°F before rolling, and at these temperatures the asphalt exhibited good handling characteristics. The remainder of the asphalt containing

Table 4. (Continued)

Location	Additives Tested	Pavement Section	Summary of Tests and Results
continued	EVA (continued)		EVA was mixed at 320°F and compacted at temperatures between 160°F and 210°F. Only when the temperature fell below about 175°F was there difficulty in obtaining sufficient embedment of the precoated chippings. Tests on asphalt mixtures taken from the surface course have shown that resistance to permanent deformation was improved by a factor of between 2 and 6, and and showed that EVA also improved the workability of rolled asphalt, allowing it to be mixed and placed at lower than normal temperatures. 1982
Arkansas I-30 Saline County	Accorex (Ref. 36)	Not Available	Placed in August 1983. Overlay. A ¼ mile section of surface course with Accorex was constructed. Approximately 150 tons of Accorex modified hot mix surface course was placed. The recommended percentage addition was 0.8% by weight of aggregate. The Accorex was added by placing plastic bags of Accorex into the aggregate filled pug mill and mixing. Then asphalt was added and mixed. The compaction temperatures of the control and test sections were approximately the same. Some clumping of the material was seen before compaction but disappeared after rolling. Three months after construction, measurements showed negligible amounts of rutting. No final conclusions can be drawn from this test. However, it has demonstrated that Accorex can be added to a hot mix in a conventional batch plant and placed on the roadway with little or no problems.
Projects constructed in 1980 Oklahoma Nevada Wyoming New Hampshire Illinois Arizona Nebraska Iowa Virginia South Carolina	Chemkrete (Ref. 10)	Variety of pavement sections	In each of these projects, with the exception of South Carolina, the sections placed with the Chemkrete modifier achieved higher strength and stability. However, the Chemkrete sections of these pavements exhibited poor low temperature properties which resulted in excessive cracking. Raveling was also noted in the Chemkrete sections of the pavements in Oklahoma and Virginia.

Table 4. (Continued)

Location	Additives Tested	Pavement Section	Summary of Tests and Results
Projects constructed in 1981 Ohio Pennsylvania California New Hampshire Maine Oregon Georgia Colorado Mississippi	Chemkrete (Ref. 7)	Various	In each of these projects, with the exception of Mississippi, the Chemkrete section achieved higher strength and stability. Chemkrete Technologies, Inc. (CTI) attributes the cracking problems that developed in the 1981 projects to production, mixing and construction irregularities. After reviewing the performance of 1981 projects, CTI recommended reducing the concentration to one part Chemkrete and 15 parts asphalt.
Projects constructed in 1982 Idaho California Seiling, Oklahoma Enid, Oklahoma West Virginia Hawaii Washington, D.C. Alaska	Chemkrete (Ref. 7)	Various	At the time of the report (May 1983), the construction of the 1982 projects had been completed for 8 to 16 months and each project was performing very well except the project in Enid, Oklahoma, where spot failures developed on the Chemkrete section and required patching immediately after construction, and subsequently the entire Chemkrete section had to be overlayed. <u>COST</u> The increase for Chemkrete modified asphalt is $3.25 per ton of mix, plus freight. This amounts to about a 15% increase.
Springdale-Big Timber, Montana	Carbon Black (15% by wt. asphalt cement) Control Sections (Ref. 37)	4.8" HMAC 2.4" HMAC 16" base	New construction on IH 90 in May 1983 (I90-7 (37) 350-U2). Treated asphalt concrete surface course was 4.8-inches thick and placed in two lifts. Carbon black (15%) was metered into a blower using a vane feeder and then pneumatically blown into a drum mix plant at the point of entry of the 200/300 pen asphalt. The control sections contained 120/150 and 85/100 pen asphalts. A specially designed device inside the drum was used to aid in mixing the carbon black with the asphalt. There was some loss of carbon black through the plant as evidenced by the deposit on the water pond from the wet scrubber. Plant temperature and compaction techniques were same for all mixtures. After two years in service there is more rutting in the 120/150 pen section and more cracking in the 85/100 pen section than in the carbon black section.

Table 4. (Continued)

Location	Additives Tested	Pavement Section	Summary of Tests and Results
Ft. Worth, Texas SH 121 (6-8 lanes)	Dow Latex Control Section (Ref. 38)	2" HMAC Fabric 8" CRCP	Overlay placed over CRCP in June 1985. Latex (3% solids) was added in drum mixed as a separate stream behind the asphalt stream. Test and control mixes contain AC-10. Plant temperatures increased about 60°F for latex mixes. Job length about 7 miles. ADT ≈ 70,000. Fabric is 6 oz/yd^2 polyester. After three months in service latex pavement performing well; control pavement showing flushing and 3/4" ruts.
Harlingen, Texas US 83	Shell Kraton D (SBS) Control Section	1½" HMAC A-R Sealcoat HMAC	Overlay placed over asphalt-rubber sealcoat in June 1985. Test pavement one mile in length (1 transport of binder). Modifier consisted of 60% Kraton 1101 and 40% Kraton 1118 in an extender oil (Dutrex 739). Polymer to oil ratio was 50/50. Modifier preblended with AC-10 prior to delivery. Control asphalt was AC-20. Plant temperature for Kraton mixture about 340°F; for control mixture about 300°F. Rained immediately upon completion of test pavement. Currently no difference in pavements.
Kern Road near South Bend, Indiana	Styrelf 13 (Ref. 39)	1" HMAC surface course 3" base course with additive prepared subgrade	Placed on July 31, 1984. 5.5% of styrelf was added. The mix on this project seemed to hold its heat for quite a long time. The design asphalt content seemed excessive and possibly the design procedure should be reviewed. The base course mix behaved as would be expected once the asphalt content was reduced. The mix exhibited the expected "stickiness" and appeared to be "tough" under the roller. No mixing problems with the batch plant operation.
Mulberry St. Des Moines Iowa	Styrelf 13 (Ref. 40)	1-2" overlay on a city street	Placed on August 12, 1984. One inch thick at the curb line and two inches thick at the center line. The mix was made in a batch plant. The mix seemed to retain heat for a longer time. The finished pavement looked excellent.
test strip near Vandenburg Air Force Base	Latex rubber (Ref. 41)	aggregate surface chipseal on the alligator cracked pavement	A year old test strip indicated that the rubber additive greatly improved low temperature flexibility of the material and drastically increased the tackiness of the emulsion. No excess chips remained on the surface. A 10-year life expectancy or greater is predicted. The present value of chipseal with latex is smaller than the conventional chip seal. Placed in 1976.
U.S. Highways 60 and 66 in Potter Co. in Texas	Polyethylene 3M-Asphadur (Ref. 42)	3" HMAC 14" flexible base	The project was originally upgraded to multi-lane in 1951. The roadway consisted of three 11 foot lanes west with a 4 foot concrete median strip. The project was overlaid in 1974 with 70 lb/sq yd of asphalt concrete pavement (type F). The concrete median strip was to be removed and the roadway would consist of one 12 foot and one 15.5 foot lane each direction with a 14 foot continuous left turn lane. Stabilized asphalt concrete pavement at the rate of 150 lbs/sq yd was placed on high traffic volume intersections. The remainder with 150 lbs/sq yd conventional asphalt concrete pavement. The additive was introduced in the pug mill after the aggregate and asphalt had been mixed. 6% by weight of the asphalt content was used. The temperature selected for the stabilized asphalt mix was 375°F. The cost per sq.

Table 4. (Continued)

Location	Additives Tested	Pavement Section	Summary of Tests and Results
	3M-Asphadur (continued)		yd. for the stabilized asphalt concrete pavement was $3.90 with the stabilizing additive being $1.57 or 40% of the cost. No shoving, rutting or movement observed 5 months after the construction.
Test bay at TRRL (pilot-scale experiment) United Kingdom	Novophalt Polyethylene (Ref. 14)	1.6" HMAC w/Novophalt prepared base	Austrian bitumen, with 7% polyethylene was used. Ten tons of hot mix containing Novophalt were used in placing a 1.6-inch pavement 9 feet wide and 100 feet long. Ten tons of similar asphalt containing 50 pen bitumen were placed as a control. Compaction temperatures ranged from 195°F to 330°F; and density, wheel-tracking rate and embedment of coated chippings during rolling, were all improved with increasing compaction temperature. To achieve the similar densities to control, Novophalt required a compaction temperature 72°F higher than the control. The resistance to permanent deformation was improved at all temperatures with the addition of Novophalt. Wheel-tracking rates were reduced by up to a factor of two compared with the factor of five found in laboratory tests. The texture-depth values for the asphalt with Novophalt were higher than those for the control at all temperatures. Novophalt had to have 284°F to achieve reasonable imbedment of chippings whereas control achieved similar results at 18°F to 36°F lower. Test performed in 1982.
Highway inside Vienna, Austria	Novophalt Polyethylene (Ref. 15)	Not Available	A 1200 foot roadway exposed to very heavy traffic. Half of the pavement is made with Novophalt, the other half with normal asphalt. Over the last five years, it has been observed that the Novophalt test pavement shows fewer indentations, ruts and deformations and practically no cracking. Placed in 1980.
Two viaducts on the Appenine and "Trafori" motorways (Autostrada)	Novophalt Polyethylene (Ref. 11)	Wearing course-dimensions unknown	80/100 pen bitumen was modified with 4% and 7% of polyethylene on the Appenine, and only 4% additive was used in the "Trafori" highway because of the colder prevailing climate. In both operations, the compaction temperature was 320°F or greater. Although the working temperatures were always higher than those specified, the test results were not always in line with those desired. From the creep tests, some sections display some tendency toward visco-plastic deformation. In some cases, the wearing course was observed to creep during the passage of the roller. In these sections there was a drop in the compound modulus and an increase in the deformability. The first achievement was that a practically waterproof pavement was obtained, this being evident from the high compaction and low residual voids observed in the core samples. The second achievement was the compounding of asphalt concretes having high mechanical strength. Finally, the bitumen containing additive succeeds in maintaining its physio-chemical properties under thermal stress. This self-protection capacity indicates that the polymer is effectively cooperating with the bitumen in the mix. (1983)

CHAPTER 3

DESCRIPTION OF MATERIALS

ASPHALT ADDITIVES

Five types of additives which appear likely to improve resistance to rutting and cracking were selected for study. The five types were:

1. Carbon black microfiller,
2. Styrene-butadiene rubber (SBR), added as latex,
3. Thermoplastic block copolymer rubber,
4. Polyethylene finely dispersed in asphalt, and
5. Copolymers of ethylene and vinyl acetate (EVA).

Only one carbon black preparation was evaluated since there is presently only one product produced particularly for asphalt modification, Microfil-8, supplied by Cabot Corporation. Microfil-8 is a mixture of approximately 92 percent high-structure HAF grade carbon black plus approximately 8 percent oil similar to the maltenes portion of asphalts, formed into soft pellets dispersible in asphalt.

Styrene-butadiene latexes are available in a wide variety of monomer proportions, molecular weight ranges, emulsifier types and other variables. Two products specifically recommended for use in hot-mixed asphalt concrete were included in the investigation, Latex XUS 40052.00 from Dow Chemical USA and Ultra Pave 70 from Textile Rubber and Chemical Co. Both are anionic and contain about 70 percent solids.

Thermoplastic block copolymer rubber was obtained from Shell Development Company in two preparations, dry crumbs of Kraton TR60-8774 (a blend of equal parts Kraton D-1101 3-block styrene-butadiene-styrene polymer and Kraton DX-1118 2-block styrene-butadiene polymer), and a rubbery solution of equal parts Kraton D-1101 and Dutrex 739 rubber extender oil. Only the TR60-8774 was used in the mixture study. The styrene-butadiene polymers do not have permanent polarization, but the presence of the aromatic rings and double bonds allow for induced polarization from the polar asphalt molecules.

Table 5. Properties of asphalts.

Asphalt source	Texaco				San Joaquin Valley			
Grade	AC-5		AC-10	AC-20	AR-1000		AR-2000	AR-4000
Serial No.	7	86	11	17	19	101	25	31
Specific gravity at 77°F	1.019	...	...	1.029	1.017	...	...	1.017
Flash point , COC, °F	...	565	595	...	...	530	595	...
Viscosity at 140°F , P	506	537	1080	2040	498	423	1100	2170
Viscosity at 275°F , cSt	224	217	332	398	128	150	185	256
Penetration at 77°F , 100 g, 5 s	194	186	118	75	146	164	86	57
Penetration at 39.2°F, 100 g, 5 s	20	17	12	8	10	12	5	4
Penetration at 39.2°F, 200 g, 60 s	63	66	41	28	46	59	25	16
Softening point , °C	40.4	41.4	46.6	51.8	41.6	41.2	47.8	51.2
Softening point, °F	104.5	106.5	116	125	107	106	118	124
Temperature susceptibility[1], 140° to 275°F	-3.42	-3.42	-3.40	-3.52	-3.94	-3.71	-3.93	-3.92
PVN[2]	-0.3	-0.4	-0.3	-0.6	-1.6	-1.2	-1.6	-1.4
P.I.[3] from penetration at 39.2°F and 77°F	-1.0	-1.4	-1.1	-1.0	-2.0	-1.9	-2.4	-2.0
P.I. from penetration at 77°F and softening point	0.0	+0.2	+0.3	+0.3	-0.7	-0.4	-0.4	-0.6
Penetration ratio[4]	32	35	35	37	32	36	29	28
After Rolling Thin Film Oven Test								
Weight change, %	...	-0.07	-0.03	...	...	-1.08	-0.39	...
Viscosity at 140°F[c], P	...	1190	2770	...	...	893	1900	...
Viscosity at 275°F[d], cSt	...	311	500	...	...	180	276	...
Penetration at 77°F[e]	...	112	71	...	...	104	57	...
% of original	...	60	60	...	...	63	66	...

[4] 100 (Pen 39.2°F, 200 g, 60 s)/ (Pen 77°F, 100 g, 5 s).

[1] Temperature susceptibility = $(\log\log \eta_2 - \log\log \eta_1)/(\log T_2 - \log T_1)$ where η = viscosity in cP, T = absolute temperature.

[2] Determined from penetration at 77°F and viscosity at 275°F (McLeod, 1976).

[3] P.I. = $(20 - 500\alpha/(1 + 50\alpha)$:

α = $[\log(pen_2) - \log(pen_1)]/(T_2 - T_1)$, or

$[\log 800 - \log(pen_{25°C})]/(T_{SP} - 25)$, where T = temperature, °C.

(After Reference 1)

Information on the Novophalt process indicated that almost any polyolefin was satisfactory for processing. Dispersions containing six polyethylene resins which varied in density, molecular weight and melt index were prepared. These included Rexene PE109, Dow 526, Dow 527, Dowlex 880, Dowlex 2045 and Dow 69065P. Polyethylene is a linear nonpolar polymer.

Four EVA resins differing in monomer ratio, solubility, softening point and melt index were studied. These included Elvax grades 40-W, 150, and 250 from DuPont Company and EX 042 from Exxon Chemical Americas. EVA has permanent polarity associated with the acetate group.

ASPHALT CEMENTS

Asphalts for this study were obtained from two sources known to produce asphalt of substantially different composition and temperature susceptibility. Three grades of paving asphalt were obtained from each source: AC-5, AC-10 and AC-20 grades from the Texaco refinery at Port Neches, Texas, which processes a blend of crude oils from East Texas, Mexico, South America and Wyoming, and AR-1000, AR-2000 and AR-4000 grades from a California refinery which processes crude oil originating in the San Joaquin Valley. Additional supplies of the AC-5 and AR-1000 grades were obtained later from the same refineries.

Table 5 presents the test results obtained on the asphalts, and several parameters calculated from them which indicate susceptibility of their physical properties to temperature change. Temperature susceptibility is greater for the San Joaquin Valley asphalts than for the Texaco asphalts. Temperature susceptibility of the three grades from each source is similar.

Component composition of the Texaco AC-5 and AC-10 and San Joaquin Valley AR-1000 and AR-2000 grade asphalts is shown in Table 6. The San Joaquin Valley asphalts have a relatively low asphaltenes content and a high content of nitrogen bases (Table 6); the latter component is a solvent for asphaltenes and makes asphaltenes compatible with the other maltenes fractions. This composition yields a sol-type asphalt with Newtonian behavior. Asphalts with higher asphaltenes content and lower content of nitrogen bases, as in the Texaco asphalts, are more likely to exhibit

Table 6. Component composition of asphalts.

Property	Texaco Asphalts AC-5	Texaco Asphalts AC-10	Texaco Asphalts AC-20	San Joaquin Valley Asphalts AR-1000	San Joaquin Valley Asphalts AR-2000	San Joaquin Valley Asphalts AR-4000
Corbett Analysis[a]						
Asphaltenes, %	14.6	-	14.8	5.0	-	6.0
Saturates, %	13.4	-	10.1	13.7	-	10.0
Naphthene Aromatics, %	41.5	-	30.3	36.1	-	33.5
Polar Aromatics, %	30.5	-	44.8	45.1	-	50.6
Rostler Analysis[b]						
Asphaltenes, %	19.1	22.4	-	9.2	10.3	-
Nitrogen bases, %	21.0	18.6	-	37.7	42.0	-
First Acidaffins, %	22.0	14.1	-	16.8	9.0	-
Second Acidaffins, %	25.0	33.5	-	22.2	28.3	-
Paraffins, %	12.9	11.4	-	14.1	10.4	-
Refractive index of paraffins, n_D^{25}	1.4812	1.4820	-	1.4862	1.4907	-
Durability rating $(N+A_1)/(P+A_2)$	1.13	0.73	-	1.50	1.32	-
Sulfur, %	-	5.08	-	-	1.34	-

[a]ASTM D4124 (Precipitates asphaltenes using n-heptane)

[b]ASTM D2006 (Discontinued) (Precipitates asphaltenes using n-pentane)

[c]Durability decreases with increasing parameter value; 0.4 - 1.0 = Group I, "superior" durability; 1.0 - 1.2 = Group II, "good" durability; 1.2 - 1.5 = Group III; "satisfactory" durability, Rostler and White (1970).

non-Newtonian behavior as the asphaltenes component is not completely solvated, so a gel structure can develop. These properties of the asphalt binders are related to the resistance of paving mixtures to deformation. They are also related to the relative compatibility with, or solvent power for, polymers such as the rubbers and resins suggested as additives. Table 6 shows only minor differences in the functional groups other than asphaltenes.

When this study was initiated, it was expected that additives would be incorporated into the medium-viscosity AC-10/AR-2000 grade asphalts which would improve the temperature susceptibility so that the viscosity at high temperatures would equal or exceed that of the higher-viscosity AC-20/AR-4000 grades while the stiffness at low temperatures would be decreased to the levels of the low-viscosity AC-5/AR-1000 grades. After it became apparent that additives of the types selected were much more effective at increasing high-temperature viscosity than in decreasing low-temperature stiffness, emphasis was shifted to incorporating the additives into the low-viscosity AC-5/AR-1000 grade asphalts, to increase their viscosity at high temperatures and improve resistance to rutting, while maintaining the cracking resistance of the low-viscosity base asphalts at low temperatures.

BLENDING OF ASPHALTS AND ADDITIVES

Compatibility

Compatibility is a term often used in reference to asphalt cements and polymeric additives. The term can be easily misunderstood. Compatibility refers to the relative solubility of the polymer in the asphalt. That is, highly compatible systems will exhibit little or no phase separation due to differences in specific gravity) under static conditions at high temperatures. It should be pointed out that polymers are not generally "soluble" in any asphalt. Certain components in asphalt will, however, soften and cause swelling in the polymers. Apparently, some highly polar polymers will react chemically with certain asphalt components and thus produce blends that appear homogeneous and do not phase-separate. Such materials would be considered compatible. Another description of compatibility involves the formation of a continuous network of microscopic strands of additive within an asphalt matrix. An incompatible system would contain discrete additive particles which either rise to the surface or settle to the bottom of the asphalt when held in a liquid state without agitation.

Asphalt Carbon Black Blends

Dispersions of carbon black in the Texaco AC-5 and AC-10 grade asphalts and the San Joaquin Valley AR-1000 and AR-2000 grade asphalts were prepared to determine the effect of the carbon black on the properties of the asphalts. Dispersions were prepared by adding preweighed pellets to preheated asphalt in a Waring Blender. The changes in temperature susceptibility are depicted graphically in Figures 1 and 2. Detailed data are given in Appendix A, Tables A1 and A2. The temperature susceptibility of all the asphalt: carbon black blends was lower than that of the base asphalt from which each was made. The principal effect of incorporation of Microfil-8 was to increase the viscosity at 140^{o}F and 275^{o}F. The penetration at 77^{o}F was decreased by addition of Microfil-8, but the penetration at 39.2^{o}F remained essentially unchanged. The 85 percent AC-5:15 percent Microfil-8 blend has approximately the same viscosity at 140^{o}F as the AC-20 grade asphalt; whereas, the 10 percent blend is equivalent to AC-10 at 140^{o}F. The 90 percent AC-10:10 percent Microfil-8 blend had approximately the same viscosity at 140^{o}F as the AC-20; whereas, the 85:15 blend containing AC-10 was equivalent to an AC-40 grade asphalt at 140^{o}F. The addition of 10 percent Microfil-8 in San Joaquin Valley AR-1000 or AR-2000 also increased the viscosity at 140^{o}F by about one grade level (Figure 2). The addition of 15 percent Microfil-8 in the San Joaquin Valley asphalts increased the 140^{o}F viscosity by almost two grade levels. The effect at the 85:15 level was not quite as great for the San Joaquin Valley asphalts as for the Texaco asphalts.

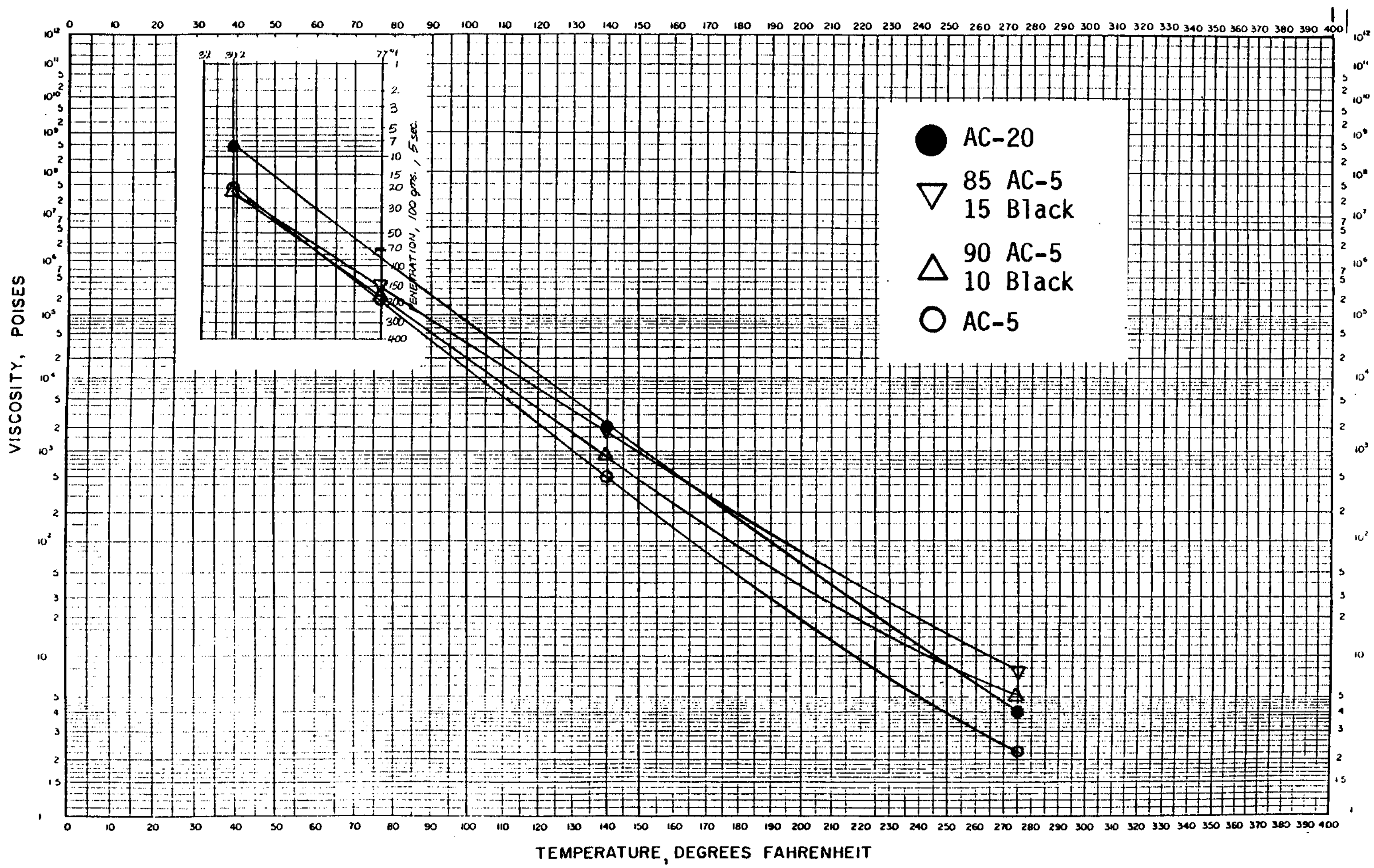

Figure 1. Blends of Texaco AC-5 and Microfil-8.
(After Reference 1)

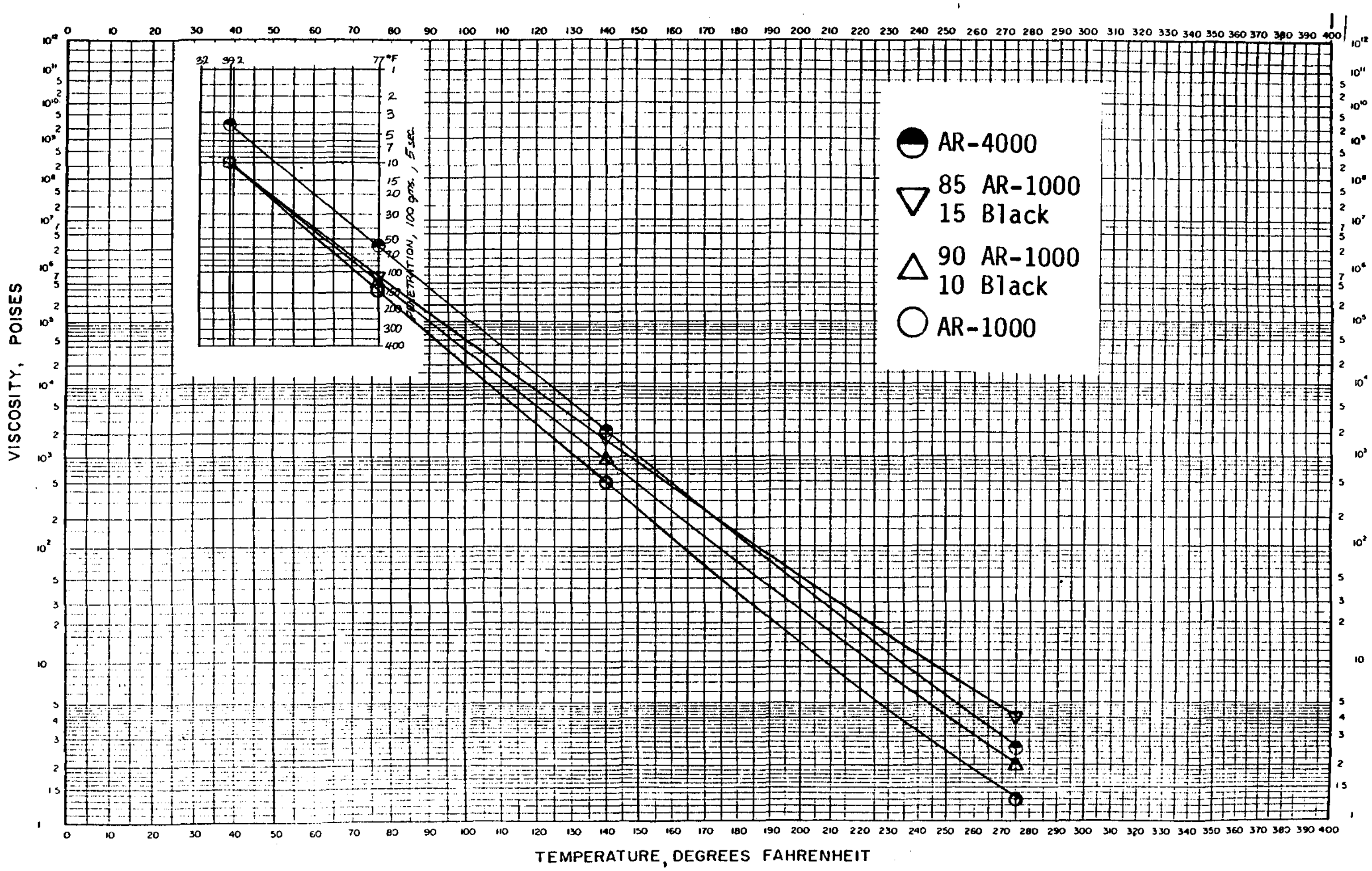

Figure 2. Blends of San Joaquin Valley AR-1000 and Microfil-8.
(After Reference 1)

Because the oils used to pelletize the carbon black in Microfil-8 might affect the asphalt properties, these oils were isolated and characterized (1). A weighed sample of Microfil-8 was placed in toluene and allowed to stand for several hours. The carbon black was filtered out and washed with additional toluene. The combined toluene fractions were filtered to clarify the solution. The toluene was evaporated to yield 5.9 weight percent of a light amber oil with a consistency similar to that of motor oil. Infrared spectra were obtained for a film of the oil on a salt plate. The ketone, phenolic OH, and sulfoxides are characteristic of a high-boiling petroleum hydrocarbon fraction that has been oxidized by atmospheric oxygen. Such oxidation might be expected since the oil has been exposed as a thin film on the carbon black surface. The recovered oil showed hydrocarbon spectra with no oxygenated chemical functionality except ketones, a trace of phenolic OH, and a low level of sulfoxides. Ketone and sulfoxide concentration are estimated at about 0.25 and 0.005 mole L^{-1}, respectively. The oil has an aromatic fingerprint region (700-900 cm^{-1}) and an aromatic C=C stretching band similar to those found in heavy petroleum distillation fractions. The evidence is strong that a high-boiling petroleum fraction is used to pelletize Microfil-8.

Dispersion of SBR in Asphalts

The recommendation of both manufacturers for incorporation into asphalt concrete is to add the latex in the plant a few seconds after the aggregate has been coated with asphalt. Dispersions of both latexes were prepared at levels of 3 percent and 5 percent solids in Texaco AC-10 and AC-5 and San Joaquin Valley AR-2000 and AR-1000 grade asphalts. Each 300 g batch was prepared by preheating the asphalt in a Waring Blender, then adding the latex slowly while blending to flash off the water and disperse the rubber.

Figures 3 and 4 show the penetration and viscosity of the blends of styrene-butadiene rubber (SBR) in Texaco AC-10 and San Joaquin Valley AR-2000 asphalts. Detailed data are given in Appendix A, Tables A3 and A4. Observations made under the microscope show that the SBR from both latexes is

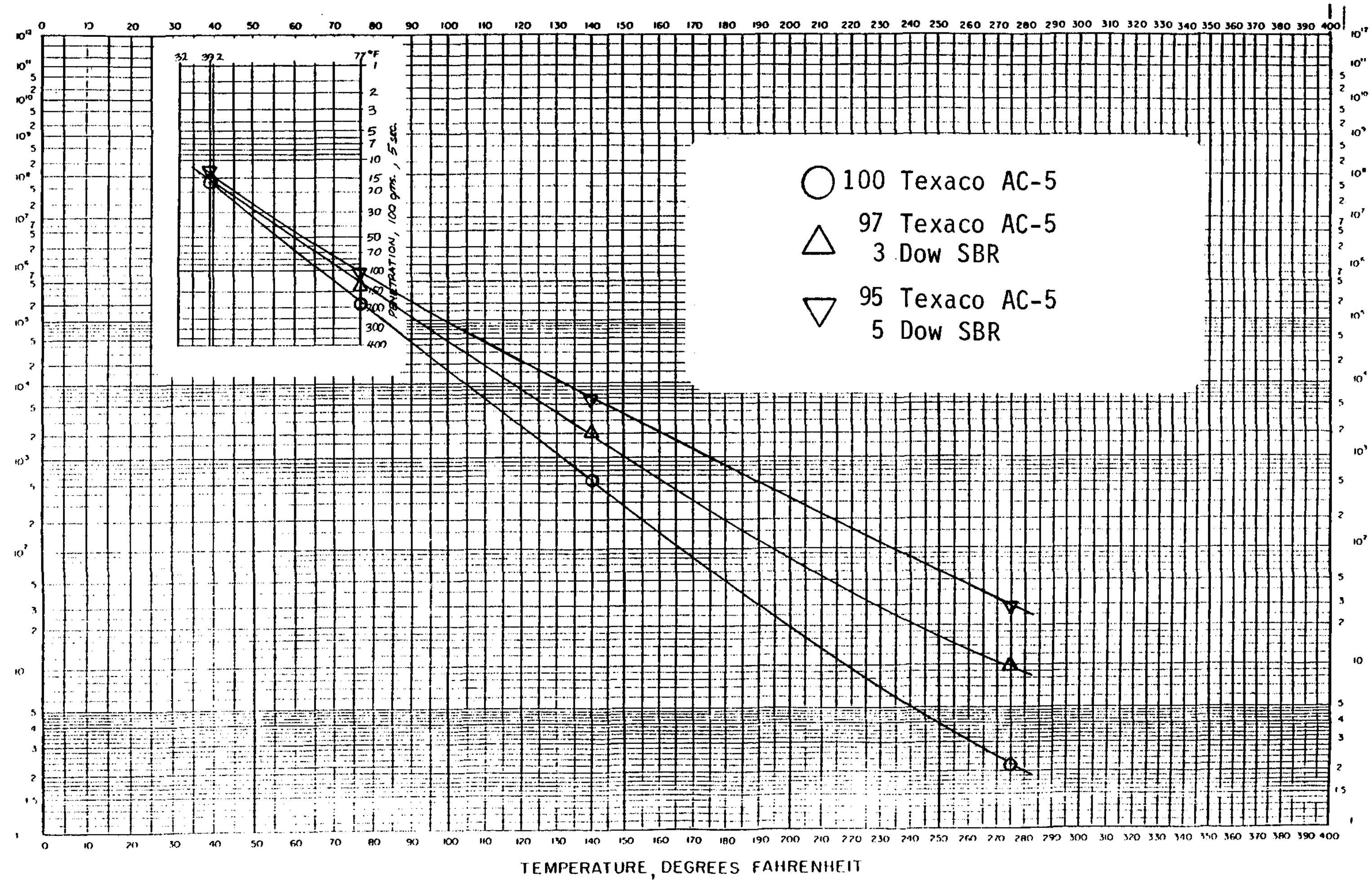

Figure 3. Penetration and viscosity versus temperature for Texaco AC-5, and dispersions of SBR latex.
(After Reference 1)

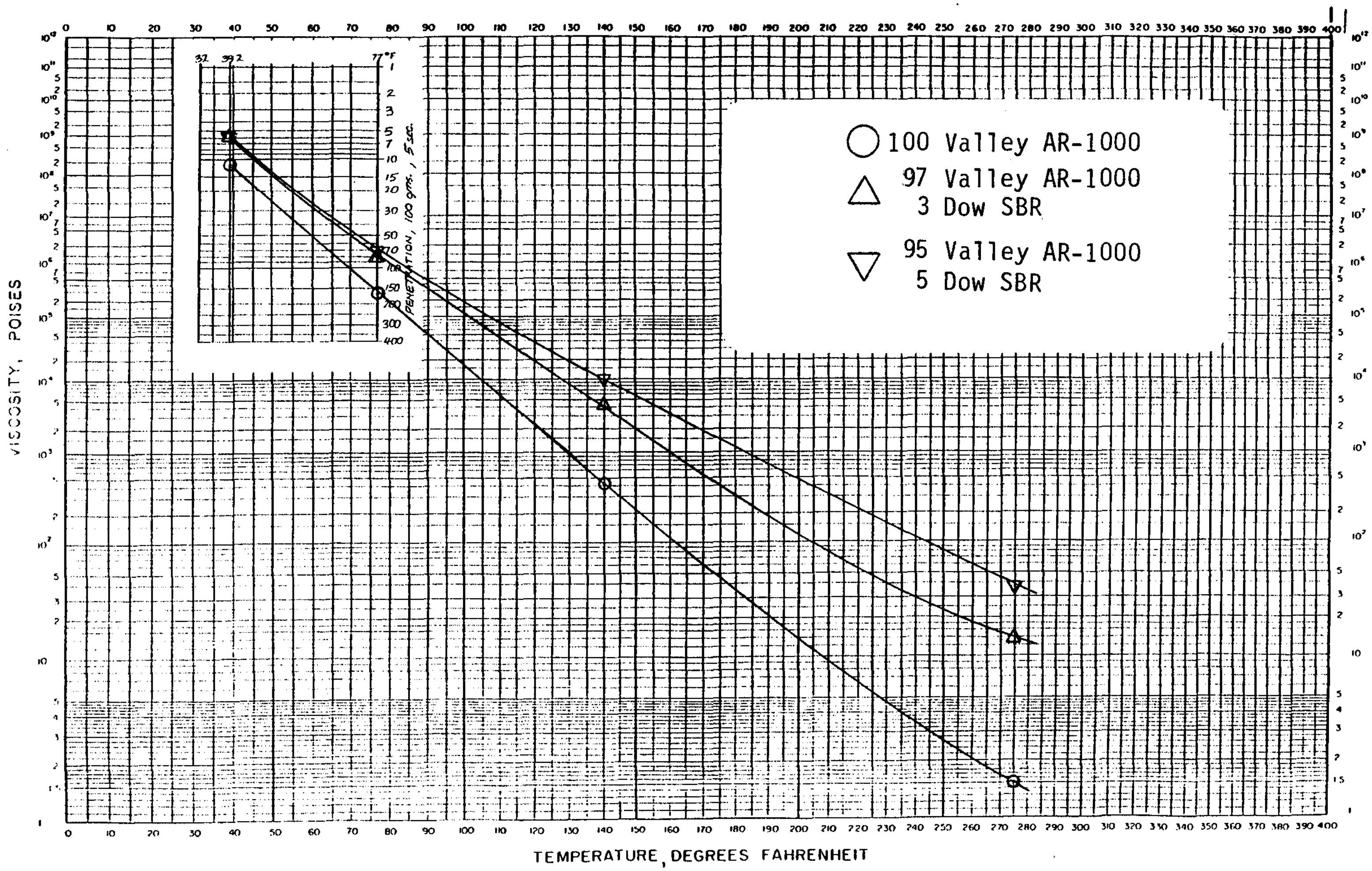

Figure 4. Penetration and viscosity versus temperature for San Joaquin Valley AR-1000, and dispersions of SBR latex.
(After Reference 1)

soluble in the San Joaquin Valley asphalt, but is not completely soluble in the Texaco asphalt.

Three percent SBR increased the 140^{O}F viscosity of the Texaco AC-10 asphalt into the AC-30 range for the Dow XUS 40052.00 latex, and the AC-40 range for the Ultra Pave 70. Five percent of either latex increased the 140^{O}F viscosity well beyond the AC-40 range. Penetration at 77^{O}F was reduced by addition of SBR, but the penetration at 39.2^{O}F was affected only slightly, except for high values obtained for penetration at 39.2^{O}F for a blend of 3 percent SBR from Ultra Pave 70 in Texaco AC-10. Repeated tests of that blend confirmed the high penetration values, but a second preparation of the same composition did not yield high values for penetration at 39.2^{O}F.

In the San Joaquin Valley AR-2000, which appeared to be the better solvent for SBR, addition of 3 percent SBR increased viscosity to the AC-20 level, while 5 percent increased the viscosity to approximately the high end of the AC-40 level. Penetration was reduced at both 77^{O}F and 39.2^{O}F. The temperature susceptibility of the San Joaquin Valley AR-2000, in the range between 39.2^{O} and 140^{O}F, was not changed significantly by addition of SBR; the values for penetration index and penetration ratio were decreased slightly from the values for the control (Appendix A).

Dispersions of Dow SBR Latex XUS 40052.00 at 3 percent and 5 percent latex solids in Texaco AC-5 grade and San Joaquin Valley AR-1000 grade asphalts were prepared (Table A4) at higher temperatures (376-390^{O}F). The higher temperatures reduced an earlier problem of stalling the Waring Blender during incorporation of the latex.

The addition of 3 percent SBR increased the 140^{O}F viscosity from about 500 P to about 2000 P (i.e. AC-20 range) for the Texaco AC-5 and to about 4000 P (i.e. AC-40 range) for the San Joaquin Valley AC-1000. Addition of 5 percent SBR increased the 140^{O}F viscosity to beyond 5000 P for the AC-5 and to 10,000 P for the AR-1000. The 275^{O}F viscosity also was increased to quite high levels. Since, in plant practice, the latex is added after about 90 percent of the aggregate surfaces are coated by asphalt, the high levels of

$275^{\circ}F$ viscosity may not cause difficulty in mixing and coating the aggregate, but asphalt concrete containing such high-viscosity binders may be difficult to place and compact. The temperature susceptibility of both base asphalts was substantially decreased by incorporation of SBR. The decrease in temperature susceptibility is also shown by increased values for penetration index, PVN, and penetration ratio.

Dispersion of SBS/SB Copolymers in Asphalts

Shell supplied dispersions of 5 percent Kraton TR60-8774 in the AC-5 and AR-1000 asphalt. The values determined for the Texaco AC-5/Kraton TR60-8774 blend did not agree with the values reported by Shell. The differences were attributed to nonhomogeneity of the blend, which appeared to have "zones" with a gelled consistency. After discussions with Shell personnel, the blend was reheated to a higher temperature $356^{\circ}F$ and the determinations repeated. The gelled zones were less evident at the higher temperature, however, there still seemed to be some variability within material poured from the same well-stirred beaker. This variability was demonstrated by an abnormal variation in softening point of two specimens tested side by side.

Subsequently, Shell recommended an additive composed of equal parts Kraton D-1101 and Dutrex 739 rubber extender oil. Since the polystyrene "domains" of the SBS block polymer are plasticized by the extender oil, the concentrate can be incorporated into asphalt without the high-shear mixing which is necessary for incorporating the block polymer directly into asphalt. Shell supplied a sample of a 50:50 blend of Kraton D-1101 and Dutrex 739, which was used to prepare four blends. Data for these blends in Texaco AC-5 and San Joaquin Valley AR-1000 are shown in Appendix A, Table A5. The data are plotted in Figures 5 and 6. Even after being dissolved in the rubber extender oil, the SBS block polymer did not readily form homogeneous blends in the Texaco asphalt, but seemed to have strings of gel throughout when melted. The $140^{\circ}F$ viscosity was increased from the 500 P level to the 1000-1200 P level by addition of 6 percent of the SBS/oil blend. The addition of 12 percent of the blend increased the $140^{\circ}F$ viscosity of the San

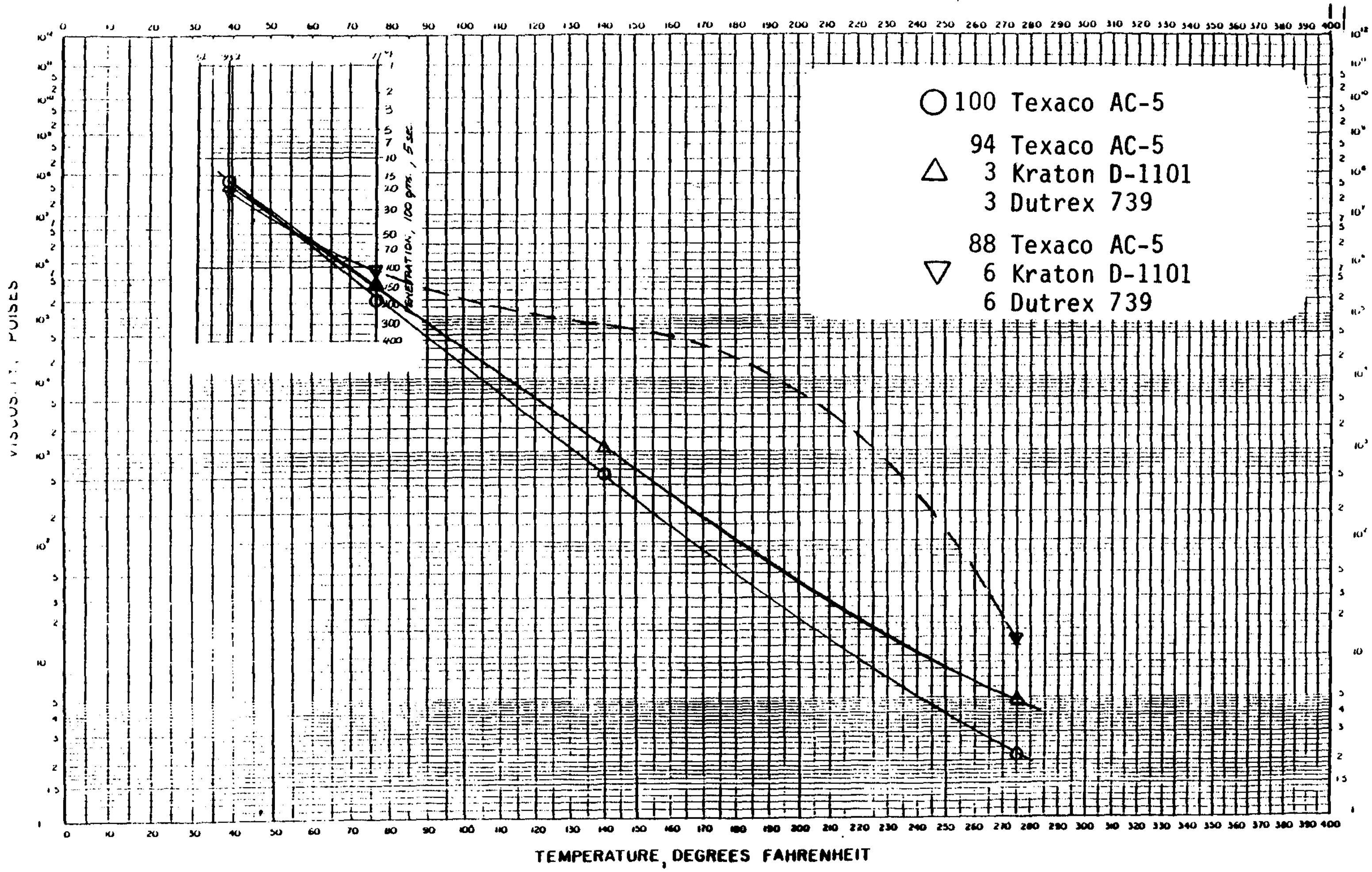

Figure 5. Penetration and viscosity versus temperature for Texaco AC-5, and solutions of S-B-S thermoplastic rubber. (After Reference 1)

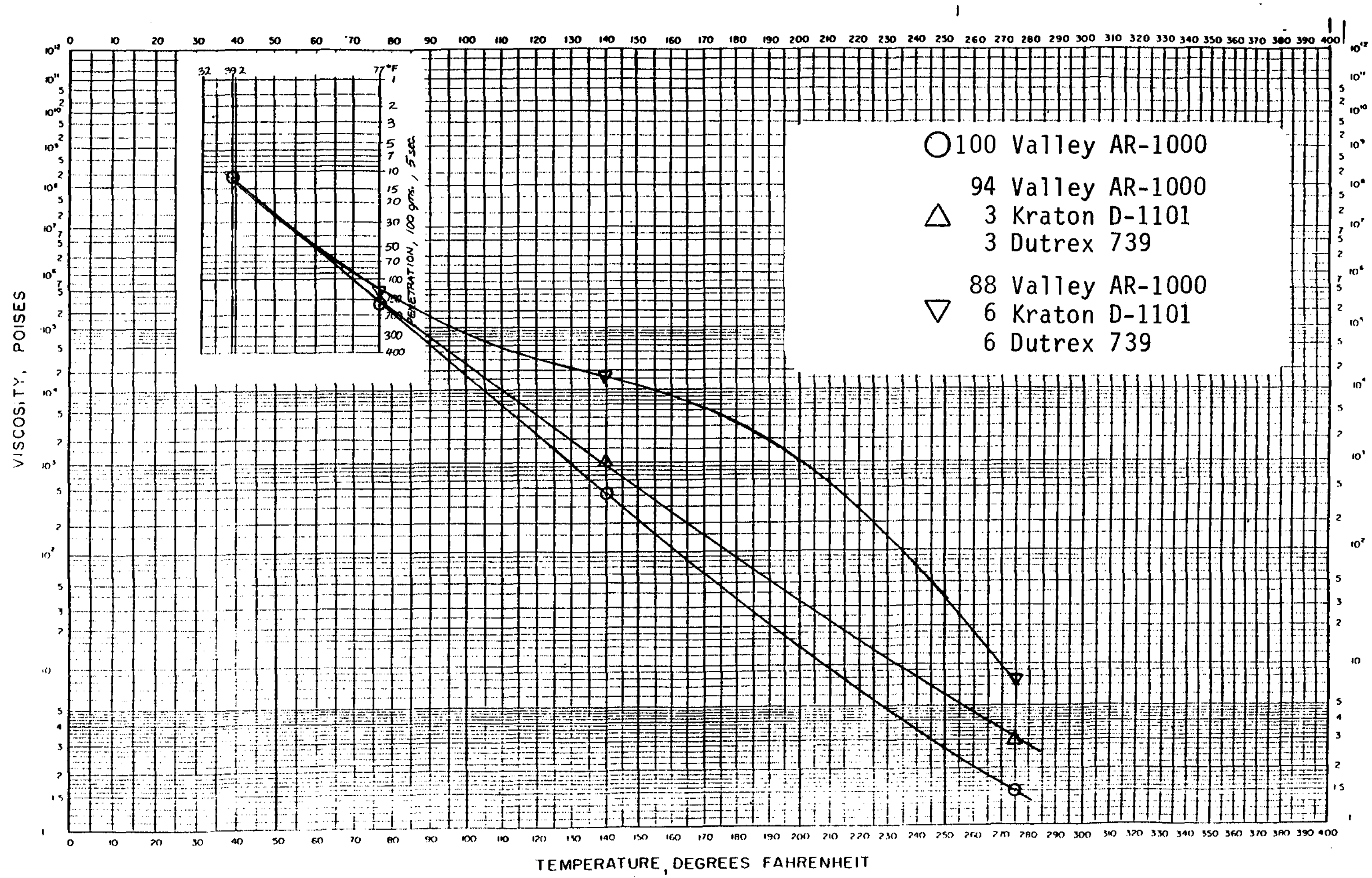

Figure 6. Penetration and viscosity versus temperature for San Joaquin Valley AR-1000, and solutions of S-B-S thermoplastic rubber.
(After Reference 1)

Joaquin Valley AR-1000 to more than 15,000 P. The Texaco AC-5 containing 12 percent of the SBS/oil blend was a gel at 140°F and could not be tested in the capillary viscometer. Penetration at 77°F was decreased, but penetration at 39.2°F was unaffected or slightly increased by addition of the SBS/oil blend.

Dispersion of Polyethylene in Asphalt

Polyethylene resins are semicrystalline polymers which are not soluble, or only slightly soluble, at temperatures below the melting point of the resin. The Novophalt process, developed by the Felsinger Company in Austria, consists of dispersing polyethylene (4 to 7 percent by weight) in asphalt at temperatures of approximately 300 to 355°F by high-speed, high-shear mixing in equipment similar to a colloid mill with very close spacing between the stationary and rotating members. The gap between rotor and stator in the laboratory equipment is 0.03 mm (0.001 in); while the gap in production equipment is 0.1 mm (0.004 in). When it is properly dispersed, the polyethylene will still separate during hot storage, i.e., float to the top of the stored asphalt, but the particles do not coalesce and can be readily redispersed by low-shear stirring, according to the information supplied by the Felsinger Company.

Scrap or recycled polyethylene is used in the production of Novophalt. While it is claimed that almost any polyethylene can be used, a desirable range of properties was suggested, i.e. a melt index between 0.7 and 1.2, and a density about 0.93. Low-molecular weight polyethylenes such as used for wax additives do not contribute much strength. Requirements for the bitumen also are either not very critical or not well defined.

All the polyethylene dispersions for this study were prepared in a Model 60 Vicosator high-speed dispersing mill supplied by the Felsinger Company, following their procedure, which requires several passes of the asphalt-polyethylene mixture through the mill. Novophalt is usually produced at the site (hot plant) and used immediately, to avoid settling during storage. In laboratory testing, it is necessary to reheat and stir thoroughly to redisperse the "creamed" polyethylene phase each time a specimen is withdrawn for testing.

Five polyethylene resins differing in density, molecular weight and melt index were dispersed in Texaco AC-10. Table A6 (Appendix A) shows data collected during the runs and the penetration and viscosity values measured. One or two passes through the mill were sufficient to disperse these LDPE resins to small spherical particles, which generally became irregular in shape, though not much smaller, with additional passes through the mill. The difficulty in determining viscosity at 140^{O}F, and examination of microscope slides of the preparations, show that three of the polyethylene resins were not dispersed to the standards recommended for Novophalt. The appearance of the particles indicates that the high density polyethylene, linear low density polethylene, and high molecular weight low density polyethylene resins probably were not liquid, but retained a strong gel structure, at the 355^{O}F temperature reached during blending.

Addition of 5 percent polyethylene increased the stiffness of the asphalt over the whole range of temperatures tested. The viscosity at 140^{O}F was increased about four-fold by the two low-density polyethylenes, and much more by the high-density, linear-low-density, and high-molecular-weight low-density polyethylenes, which produced blends having gel-like consistency and did not flow through the capillary viscometers. Since the reduction of penetration at 39.2^{O}F was less than the reduction at 77^{O}F, the overall effect of polyethylene was a reduction of temperature susceptibility.

Dispersions of one of the low-density resins, Dow LDPE 526, were also prepared in the Texaco AC-5 and in the San Joaquin Valley AR-1000 and AR-2000. The particle size of the dispersed LDPE in these asphalts was similar to that obtained in the Texaco AC-10 asphalt.

In the Texaco AC-5, the effect was similar to that for the same resin in Texaco AC-10 (see Serial No. 49 in Table A6). In the San Joaquin Valley asphalts, the effect was less dramatic than in the Texaco asphalts. Table A7 shows that fairly consistent results were obtained in replicate preparations of Novophalts using LDPE 526 in the AC-5, AC-10, AR-1000 and AR-2000 asphalts.

Five gallon lots of Novophalt needed for preparation of asphalt concrete specimens, were prepared using 5 percent LDPE 526 in Texaco AC-5 and San Joaquin Valley AR-1000. Properties of these blends are shown in Table A8, and plots of the rheological data are provided in Figures 7 and 8.

Dispersion of EVA in Asphalts

Dispersions of three ethylene-vinyl acetate copolymer resins, Elvax 40-W, 150 and 250, in San Joaquin Valley AR-2000 and Texaco AC-10 asphalts were prepared as 300 g batches in the Waring Blender. Examination under the microscope showed differences in compatibility with the two asphalts. There were also differences in compatibility between the three resins, which differed in melt index, which affects viscosity of solutions, and in ratio of the two monomers, which affects solubility.

Table A9 (Appendix A) shows the properties of the blends in San Joaquin Valley AR-2000 and Texaco AC-10. Addition of the EVA polymers at the 3 percent level increased the 140^{O}F viscosity of each asphalt from the AC-10 level to the AC-20 range. Penetration at 77^{O}F was decreased by the addition of EVA to Texaco AC-10 but was not changed much by the addition of EVA to San Joaquin Valley AR-2000. Overall, the penetration at 39.2^{O}F exhibited only a slight increase. Of the three EVA resins tested, Elvax 150 appeared to be the most compatible with the asphalts and Elvax 250 the least compatible.

Preliminary trails of a lower-melting EVA resin, Exxon EX042, indicated that this copolymer is more readily incorporated into asphalt than the Elvax resins. Dispersions of Exxon EX042 and Elvax 150 in the Texaco AC-5 and San Joaquin Valley AR-1000 asphalts were prepared. These blends were prepared by stirring with a low-shear Jiffy mixer instead of by high-shear mixing in a Waring Blender. The Exxon EX042 resin appeared to dissolve completely while being stirred at 325^{O}F. It was necessary to increase the temperature to 347^{O}F to completely dissolve the Elvax 150. The data obtained on these blends are presented in Appendix A, Table A10.

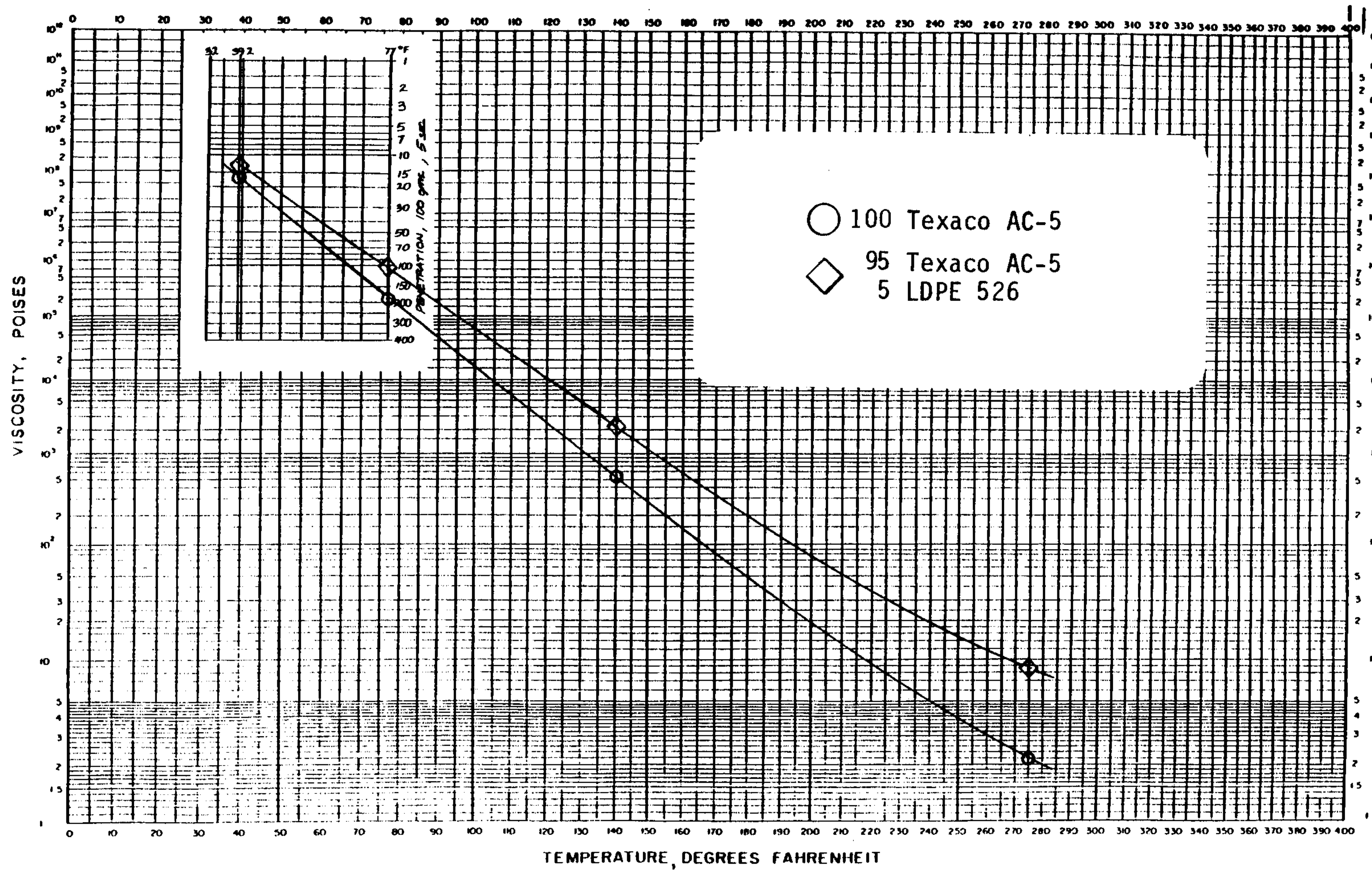

Figure 7. Penetration and viscosity versus temperature for Texaco AC-5, and dispersion containing 5% LDPE 526. (After Reference 1)

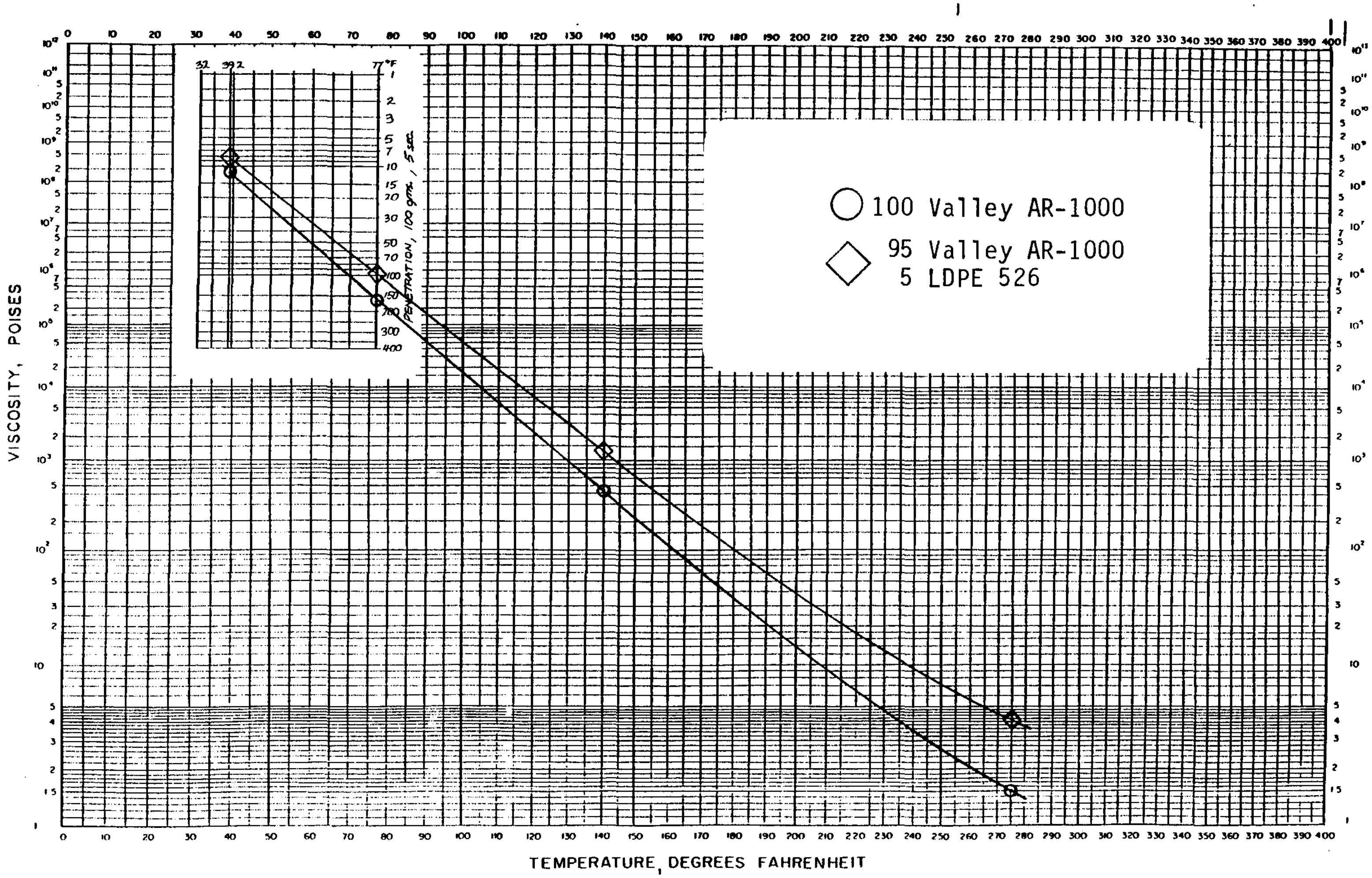

Figure 8. Penetration and viscosity versus temperature for San Joaquin Valley AR-1000, and dispersion containing 5% LDPE 526.
(After Reference 1)

The EX042 did not change the properties of the asphalt as much as an equivalent amount of Elvax 150. The viscosity at 275^{0}F was increased by addition of the EVA resins, but not to the very high levels of the SBR latex blends. EX042 at 3 percent and 5 percent increased the 140^{0}F viscosity of Texaco AC-5 slightly, but did not affect the 140^{0}F viscosity of San Joaquin Valley AR-1000. Elvax 150 at 3 percent increased the 140^{0}F viscosity to about 800 P and 5 percent to near 1200 P (i.e. AC-10 range) for both asphalts. Effect on penetration at 77^{0}F appeared inconsistent. Penetration at 39.2^{0}F was not appreciably affected by incorporation of EVA. Plots of penetration and viscosity vs temperature are given in Figures 9 through 11. Since the changes achieved with 5 percent EVA are comparatively modest, it appears appropriate to incorporate higher levels of EVA.

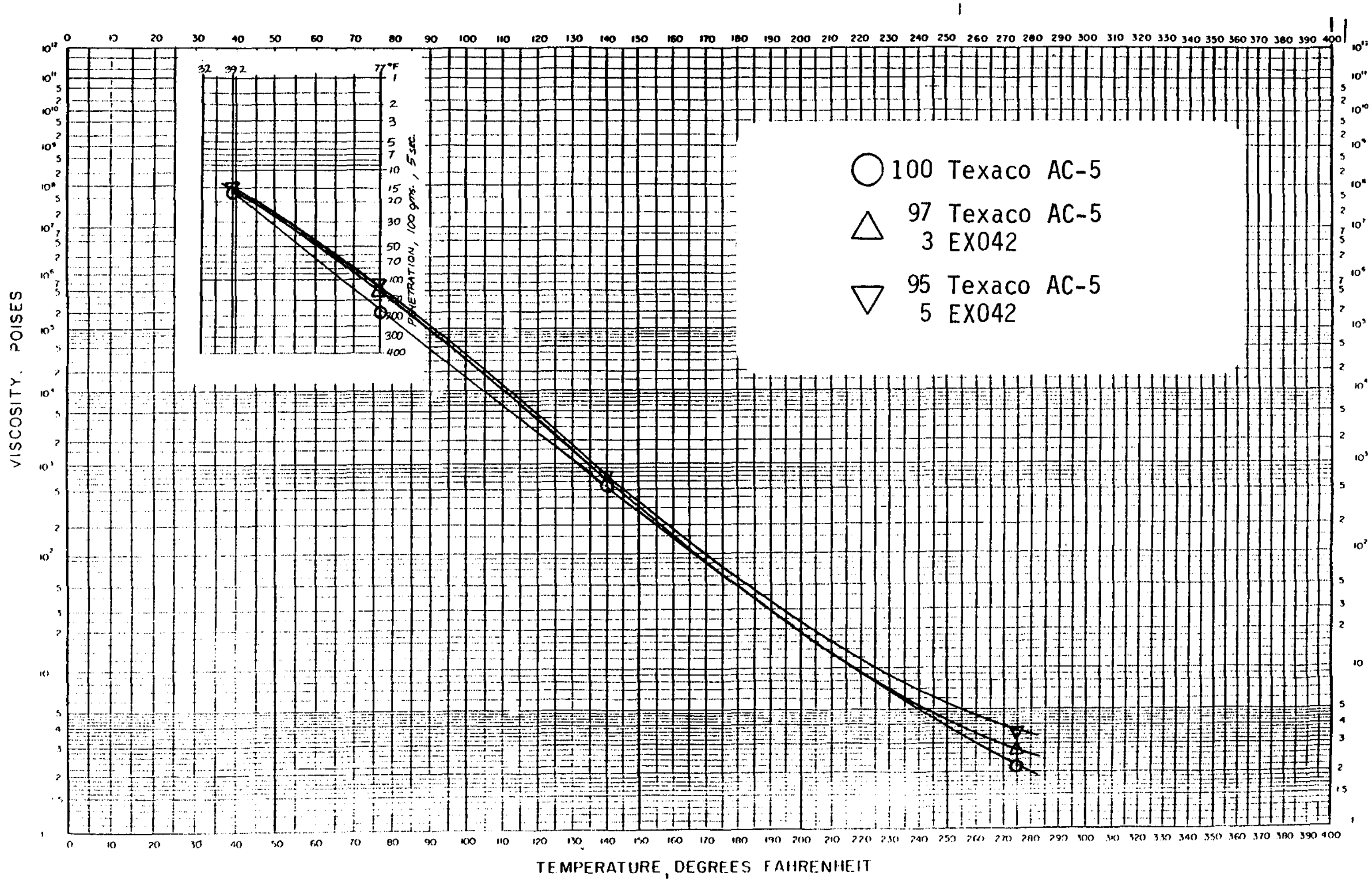

Figure 9. Penetration and viscosity versus temperature for Texaco AC-5, and blends with EVA resin EX042.
(After Reference 1)

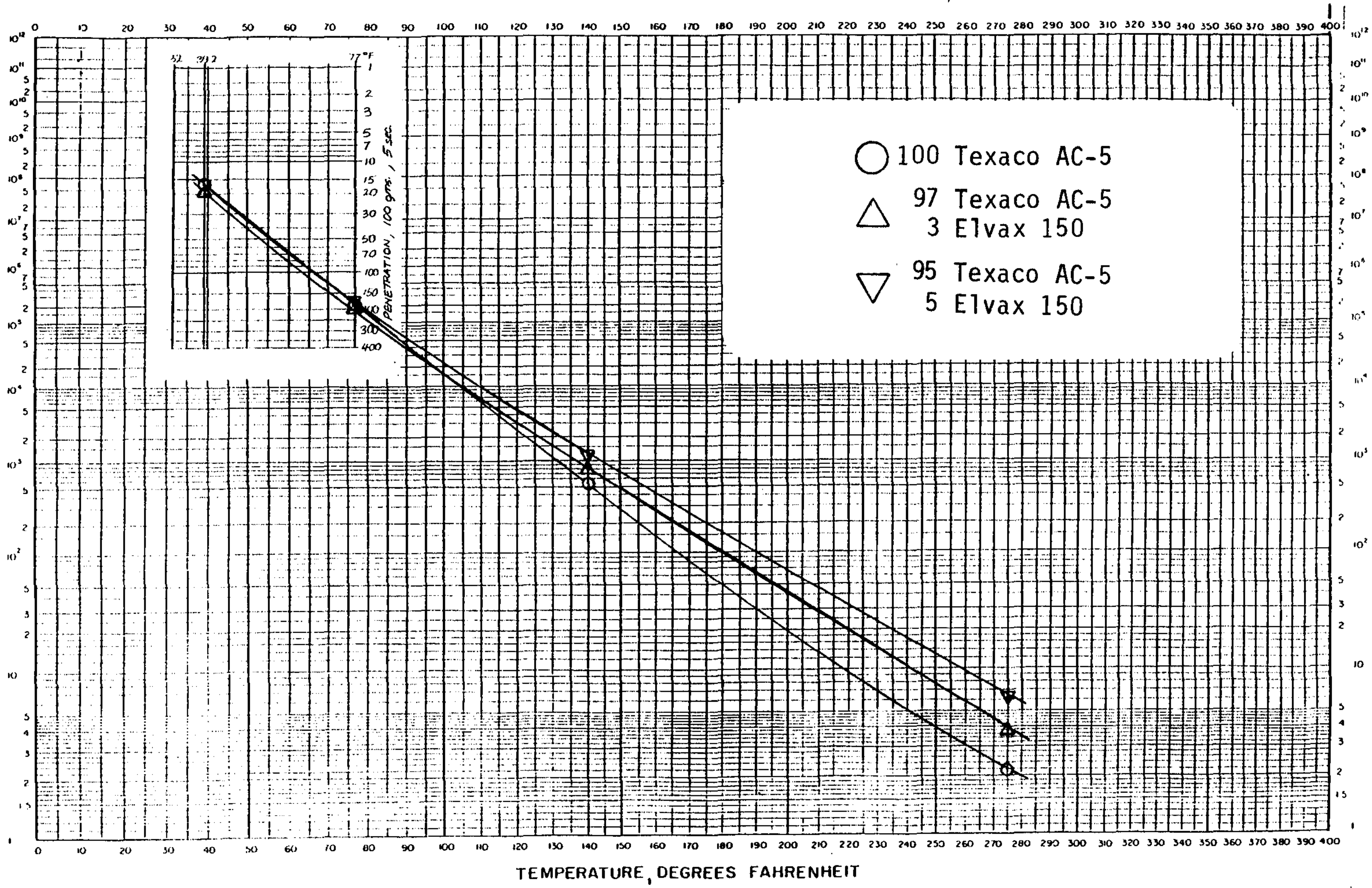

Figure 10. Penetration and viscosity versus temperature for Texaco AC-5, and blends with EVA resin ELVAX 150.
(After Reference 1)

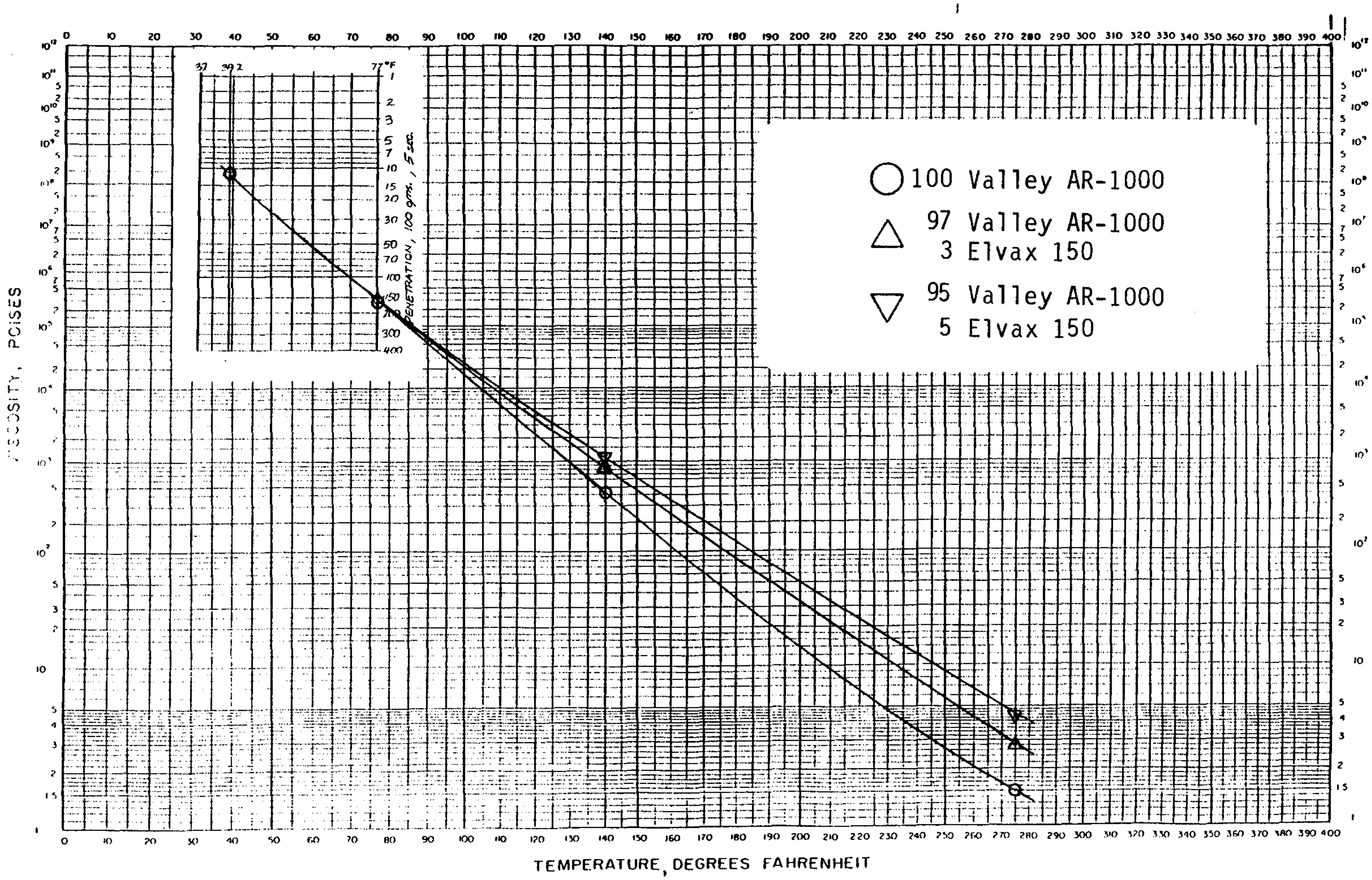

Figure 11. Penetration and viscosity versus temperature for San Joaquin Valley AR-1000, and blends with EVA resin ELVAX 150.
(After Reference 1)

CHAPTER 4

TEST RESULTS ON ADDITIVE-ASPHALT BLENDS

Flash point, ductility testing and physical property changes following rolling thin film oven aging were determined for selected additive-asphalts combinations.

ROLLING THIN FILM OVEN AGING

The thin film oven test (TFOT), normally used by Texas SDHPT, was replaced in this work by the rolling thin film oven test (RTFOT) in an attempt to produce uniformly aged modified binders for use in subsequent testing. Experience has shown that the undisturbed surface of the asphalt will sometimes form a "crust" or "scum" during the TFOT. This is a particular problem with some modified asphalts.

Table 7 presents results after exposure of the modified binders to the RTFOT. Generally, the test results are inconsistent and difficult to analyze. AASHTO specifications allow a four-fold increase in viscosity at 140^{o}F; all of the materials meet this criterion.

Viscosity at 275^{o}F for the blends containing SBR decreased during the RTFO test, probably indicating thermal degradation of the polymer during the 325^{o}F exposure. Binders containing the LDPE 526 and EVA yielded the largest increase in consistency.

FLASH POINT TESTING

Flash points were determined for one blend containing each of five types of additives (Table 8). The flash points of the blends are lower than for the base asphalts, but still well above standard specification limits.

Table 7. Change in properties of asphalts and selected blends exposed to rolling thin film oven aging.

Serial No.	86	101	102	130	131	132	133	134	135	97	113	136
Composition, %												
Texaco AC-5	100	...	95	...	97	95	...	...	95	95	...	...
San Joaquin Valley AR-1000	...	100	...	95	...	...	97	95	...	...	95	90
S-B-S/S-B block polymer[a]	...	...	5	5	...	...	...	...	...	...	...	...
SBR[b]	...	...	...	...	3	5	3	5	...	...	...	...
EVA[c]	...	...	...	...	...	...	...	...	5	...	...	...
LDPE 526[d]	...	...	...	...	...	...	...	...	...	5	5	...
Microfil-8[e]	...	...	...	...	...	...	...	...	...	...	...	10
Viscosity at 140°F[f], P	537	423	6720	1720	1960	5480	4020	10,100	1170	2200	1295	889
Viscosity at 275°F[g], cSt	217	150	873	431	1020	2780	1190	3600	634	843	399	257
Penetration at 77°F[h], 100 g, 5 s	186	164	103	134	140	114	83	72	156	105	98	121
After Rolling thin Film Oven Test[i]												
Weight change, %	-0.07	-1.08	-0.05	-0.95	-0.19	-0.19	-0.76	-0.63	-0.11	-0.05	-0.91	-0.81
Viscosity at 140°F, P	1190	983	15,900	2940	4110	9230	8250	19,600	2740	5060	4170	1890
η_a/η_o	2.22	2.11	2.37	1.71	2.10	1.68	2.05	1.94	2.34	2.30	3.22	2.13
Viscosity at 275°F, cSt	311	180	2680	486	877	2400	1170	2710	1040	1320	431	324
η_a/η_o	1.43	1.20	3.28	1.13	0.86	0.86	0.98	0.75	1.64	1.57	1.08	1.26
Penetration at 77°F	112	104	80	87	85	103	49	46	73	65	57	78
% of original penetration	60	63	78	65	61	90	59	64	47	62	58	64

[a]Kraton TR60-8774, blend of equal parts Kraton D-1101 three-block S-B-S polymer and Kraton DX-1118 two-block S-B (styrene-butadiene) polymer.

[b]Styrene-butadiene rubber from Dow XUS 40052.00 latex.

[c]Elvax 150, 32-24% vinyl acetate, softening point 230°F, specific gravity 0.957.

[d]Dow low-density polyethylene, density 0.919, melt index 1.0.

[e]Lot CS-4632, 93.3% high-structure HAF carbon black, 6.7% oil.

[f]AASHTO T202.

[g]AASHTO T201.

[h]AASHTO T49.

[i]AASHTO T240.

(Modified after Reference 1)

Table 8. Flash point and ductility of asphalts and selected blends with additives.

Serial No.	86	101	102	130	131	132	133	134	135	136	137	113	T1	T2	T3	T4
Composition, %																
Texaco AC-5	100	...	95	...	97	95	...	...	95	...	...	...	95	...	85	...
San Joaquin Valley AR-1000	...	100	...	95	...	...	97	95	...	90	95	95	...	95	...	85
S-B-S/S-B block polymer[a]	...	...	5	5	...	...	...	...	...	...	...	...	...	...	...	...
SBR[b]	...	...	...	...	3	5	3	5	...	...	5	...	...	...	...	...
EVA[c]	...	...	...	...	...	...	...	...	5	...	...	...	...	...	...	...
Microfil-8	...	...	...	...	...	...	...	...	...	10	...	...	...	...	15	15
LDPE 526 (Novophalt)	...	...	...	...	...	...	...	...	...	...	...	5	5	5	...	...
Flash Point, °F, AASHTO T48 (COC)	565	530	...	500	...	...	...	...	560	495	510	495	...	...	...	...
Ductility, cm, AASHTO T51																
At 39.2°F, 1 cm/min	>150	>150	...	...	...	...	...	...	...	...	...	...	...	...	...	...
At 39.2°F, 5 cm/min	>150	>150	68[d]	141[d]	>150	>150	36	>150	24[d]	...	...	...	...	...	...	...
At 77°F, 5 cm/min	>150	130	98[d]	83[e]	>150	>150	131[f]	144[g]	45[d]	...	...	...	35	26	11	7

[a]Kraton TR60-8774, blend of equal parts Kraton D-1101 three-block S-B-S polymer and Kraton DX-1118 two-block S-B (styrene-butadiene) polymer.

[b]Styrene-butadiene rubber from Dow XUS 40052.00 latex.

[c]Elvax 150, 32-24% vinyl acetate, softening point 230°F, specific gravity 0.957.

[d]Pulled out as much thicker threads than unmodified asphalts.

[e]Broke by "necking", i.e. one point of the thick threads pulled out to very thin threads, which then broke.

[f]One specimen >150 cm.

[g]Two specimens >150 cm.

(Modified after reference 1)

DUCTILITY TESTING

Ductilities were determined in accordance with ASTM D113 for AC-5 and AR-1000 asphalts and selected blends of them with additives.

The ductility test results are presented in Table 8. Both base asphalts had very good ductility at both 39.2°F and 77°F. Specimens of the blend containing Elvax 150 broke at relatively short elongations, probably due to the presence of undissolved resin particles. The blends containing thermoplastic block polymer (SBS) rubber formed thick threads, obviously much stronger than the threads of unmodified asphalts, with some of the material within the end holders actually pulled out into the thread.

FORCE DUCTILITY

The force ductility test is a modification of the asphalt ductility test. The test has been described (43,44) as a means to measure tensile load-deformation characteristics of asphalt and asphalt-rubber binders.

The test is performed as described by ASTM D113 with certain changes. The principal alteration of the apparatus consists of adding two force cells in the loading chain. Specimens are maintained at 39.2°F by circulating water through the ductility bath during testing.

A second major alteration of the standard ASTM procedure involves the test specimen shape. The mold is modified to produce a test specimen with a constant cross-sectional area for a distance of approximately 3 centimeters. This mold geometry produces a deformation rate of 0.74 ± 0.01 cm/min between the gage marks of the test specimen at a fixed grips test rate of 1 cm/min (45).

Force data is transferred from the load cells to analog recording equipment. Signals received by this equipment are then transferred to a microcomputer. Data are stored on magnetic computer disks for later reduction and analysis.

Raw data obtained from the force ductility machine are initially in terms of a force-time relationship. However, the constant deformation rate of 0.74 cm/min allows conversion of force-time information to force-strain data. Stress data is calculated using the initial one square centimeter cross sectional area. Engineering strain, ε_e, is obtained by dividing the change in gauge length by the original gauge length as follows:

$$\varepsilon_e = \frac{\Delta L_o}{L_o} \qquad \text{Equation 1}$$

Modulus of elasticity was determined by evaluating the initial slope of the stress-strain curve. The initial slope of the stress-strain curve in the linear region under primary loading will be referred to as the "asphalt modulus". A second slope was observed for certain blends which was characterized by secondary loading. Examples of typical stress-strain curves are shown in Figures A1 and A2 in Appendix A.

Although the data are limited, the stress-strain curves (Appendix A) may be indicating compatibility between the additives and the asphalts. The polymeric additives have been shown to be more compatible in the San Joaquin Valley asphalt than in the Texaco asphalt (1). Those polymers that are compatible, i.e., "dissolved" in the asphalt or develop a continuous network of microscopic strands, are characterized by a secondary loading which exhibits significantly more stress than the unmodified asphalt. Note that in the Texaco asphalt, only Kraton shows the second peak. In the San Joaquin Valley asphalt, Kraton, latex and Elvax show the second peak. Carbon black and polyethylene (Novophalt) are not "dissolved" in any asphalt, but exist as a discontinuous phase in the continuous asphalt phase.

Force ductility data from modified and unmodified asphalts before and after the RTFOT are given in Table 9. Each value represents an average of two tests. Incorporation of the additives in AC-5 or AR-1000 consistently results in an increase in the maximum engineering stress. Novophalt and Microfil-8 exhibit the largest increase in maximum engineering stress.

Area under the stress-strain curve could be considered total work or energy required to produce failure. AC-5 and AR-1000 containing an additive exhibit marked increases in energy required to produce failure. These data (Table 9) and the curves in Figures A1 and A2 show that the changes in stress-strain properties imparted by these additives are highly dependent upon the properties (probably physical and chemical) of the base asphalt.

Table 9. Summary of Forced Ductility Tests @ 39.2°F and 5 cm/min.

Base Asphalt & Condition	Sample Type	Maximum Engineering Stress, psi	Maximum Engineering Strain, in/in	Area Under Stress-Strain Curve	Initial Slope of True Stress-Strain Curve	Total Deformation at Specimen Rupture, cm
Original Texaco	AC-5 + 5% Kraton	2.9	23.5	19.6	10.5	65
	AC-5 + 5% Latex	2.2	4.4	4.2	8.6	-
	AC-5 + 5% Elvax 150	3.1	13.7	33.1	10.8	70
	AC-5 + 5% Novo-phalt	4.6	4.1	10.7	15.7	23
	AC-5 + 15% Microfil-8	5.2	3.3	6.7	13.3	12
	Texaco AC-5	1.9	3.7	2.9	6.2	120+
	Texaco AC-20	12.7	8.8	29.7	34.6	35
Texaco After RTFOT	AC-5 + 5% Kraton	6.9	29.4	54.9	2.5	89
	AC-5 + 5% Latex	4.3	4.9	8.3	5.5	120+
	AC-5 + 5% Elvax	5.8	6.2	14.3	2.3	20
	AC-5 + 5% Novophalt	10.8	5.7	23.1	4.1	15
	AC-5 + 15% Microfil-8	8.5	3.6	18.1	3.5	12
	Texaco AC-5	3.3	3.9	5.2	5.4	120+
	Texaco AC-20	12.0	2.1	22.3	3.5	6
San Joaquin Valley After RTFOT	AR-1000 + 5% Kraton	13.7	26.0	108.0	5.4	35
	AR-1000 + 5% Latex	10.6	40.0	46.1	3.6	100+
	AR-1000 + 5% Elvax	8.0	13.5	60.9	2.7	40
	AR-1000 + 5% Novophalt	19.4	10.1	52.8	8.4	70
	AR-1000 + 15% Microfil-8	15.8	6.31	34.2	5.4	22
	San Joaquin AR-1000	12.5	7.05	24.8	4.3	100+
	San Joaquin AR-4000	No data - samples broke on initial loading				0

Figures 12 and 13 show that a relationship exists between maximum engineering stress of the binders and tensile strength of corresponding mixtures. (Indirect tension test results is presented and discussed in detail in Chapter 4). It appears that the force ductility test may be useful in predicting changes in mixture tensile strength when asphalt additives are used.

SLIDING PLATE VISCOSITY AT 77°F

The sliding glass plate microviscometer (ASTM D3570-77) was used to determine binder viscosities at 77°F. For purposes of comparison, viscosities determined at three temperatures for selected asphalts and asphalt-additive blends are presented in Table 10.

The sliding plate test is inappropriate for binders containing granular materials with particle sizes approaching the binder film thickness. As a result, data from the binder containing LDPE 526 is questionable.

Viscosity at 77°F of the AC-5 is increased significantly by the addition of the SBS/SB block polymer and the EVA. However, only slight increases in viscosity at 77°F are exhibited upon addition of the SBR and the Microfil-8. Microfil-8 is composed of carbon black with 8 percent oil. This oil may be at least partly responsible for retention of the low viscosity at 77°F.

HEAT STABILITY OF MODIFIED ASPHALTS

Heat stabilities of unmodified and modified Texaco asphalts were evaluated in an attempt to predict any problems that might occur during prolonged hot storage. Three different test conditions were used.

Samples of each binder were placed in covered penetration tins to minimize oxidation and exposed to 350°F for 48 hours then visually examined. No visual change occurred in the unmodified Texaco AC-5, AC-10 and AC-20 asphalts. A hard, crazed crust formed on the surface of the Kraton, Novophalt and Microfil-8. In the Microfil-8 specimen, the carbon black settled to the bottom. The Ultrapave latex specimen exhibited a lumpy,

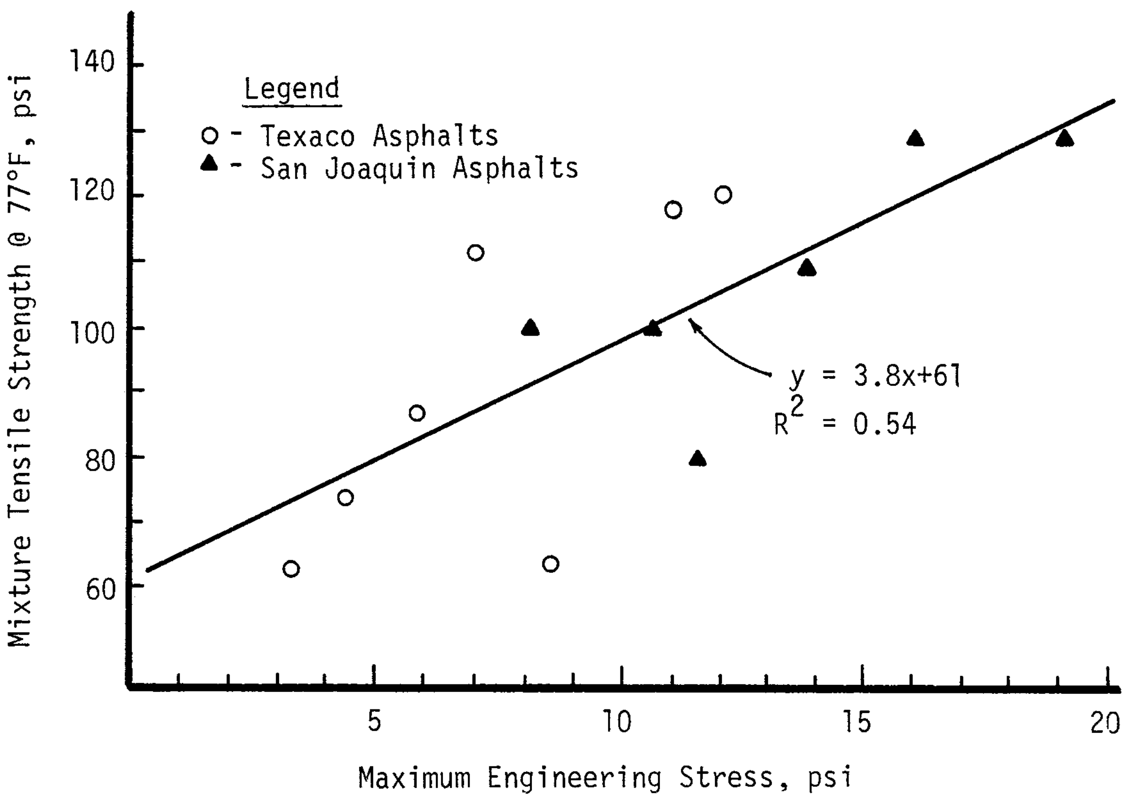

Figure 12. Mixture Tensile Strength as a Function of Maximum Engineering Stress from Force-Ductility Test. (Mixture tensile strength was measured at 77°F and 2 in/minute using the indirect tension test. Force ductility data after RTFOT were used.)

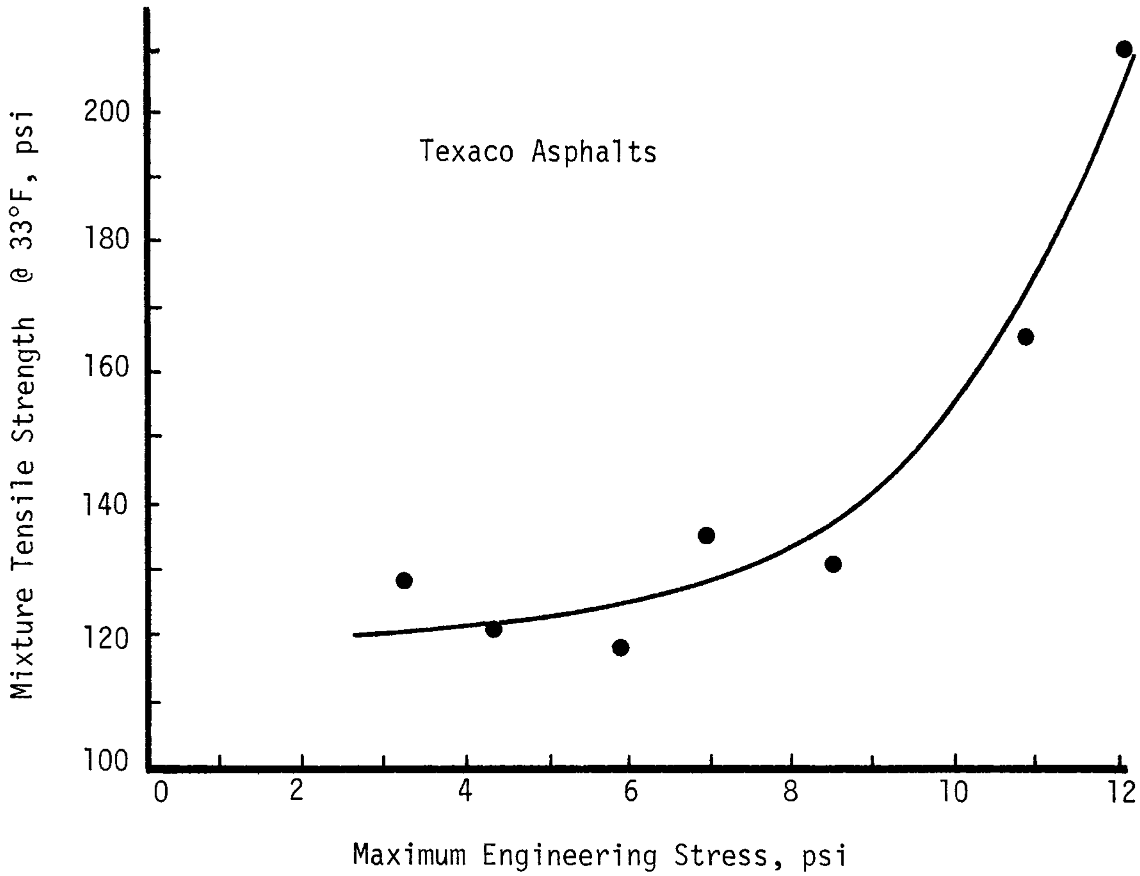

Figure 13. Mixture Tensile Strength as a Function of Maximum Engineering Stress. (Mixture tensile strength was measured at 33°F and 2 in/min using indirect tension test. Force ductility data at 39°F after RTFOT were used.)

Table 10. Viscosity data at 275, 140 and 77°F for selected asphalts and asphalt-additive blends.

Asphalt	Additive	% Asphalt: % Additive	275°F,[f] cst.	140°F,[g] P	77°F[h] P x 10^6
Texaco AC-5	...	...	224	506	0.25
Texaco AC-20	...	...	398	2040	0.31
Texaco AC-5	S-B-S/S-B Block Polymer[a]	95:5	873	1170-6720	0.42
	SBR[b]	97:3	1020	1960	0.28
	EVA[c]	95:5	618	1160	0:32
	LDPE 526[d]	95:5	843	2200	1.5
	Microfil-8[e]	85:15	740	1850	0.26

[a]Kraton TR60-8774, blend of equal parts Kraton D-1101 three-block S-B-S polymer and Kraton DX-1118 two-block S-B (styrene-butadiene) polymer.

[b]Styrene-butadiene rubber from Dow XUS 40052.00 latex.

[c]Elvax 150, 32-34% vinyl acetate, softening point 230°F, specific gravity 0.957.

[d]Dow low-density polyethylene, density 0.919, melt index 1.0.

[e]Lot CS-4632, 93.3% high-structure HAF carbon black, 6.7% oil.

[f]AASHTO T201.

[g]AASHTO T202.

[h] ASTM D-3570.

[i]Not determined.

Table 11. Summary of data from Texaco AC-5 and selected additive blends before and after heat stability testing.

Composition	Viscosity 140°F, cSt		Viscosity 275°F, P		Penetration @ 77°F	
	Before Heat Testing	After Heat Testing	Before Heat Testing	After Heat Testing	Before Heat Testing	After Heat Testing
95% Texaco AC-5 + 5% LDPE 526[a]	2200	1796	843	833	60	68
95% Texaco AC-5 + 5% SBS[b]	2100	1420	873	465	82	120
97% Texaco AC-5 + 3% EVA[c]	490	410	380	300	107	106
85% Texaco AC-5 + 15% Microfil-8[d]	1850	1900	740	*[f]	75	98
95% Texaco AC-5 + 5% SBR[e]	3900	904	2780	*[f]	116	147
Texaco AC-5	510	500	211	190	155	145

[a]Dow low-density polyethylene, density 0.919, melt index 1.0.

[b]Kraton TR60-8774, blend of equal parts Kraton D-1101 three-block S-B-S polymer and Kraton DX-1118 two-block S-B (styrene-butadiene) polymer.

[c]Elvax 150, 32-24% vinyl acetate, softening point 230°F, specific gravity 0.957.

[d]Lot CS-4632, 93.3% high-structure HAF carbon black, 6.7% oil.

[e]Styrene-butadiene rubber from Dow XUS 40052.00 latex.

[f]Unable to obtain data due to repeated clogging of viscometer.

sticky surface but no hard crust; latex is not very compatible with these Texaco asphalts.

In a second test, fresh samples in covered penetration tins were exposed to 325°F for 24 hours. A hard, crazed crust formed on the surface of the Novophalt. The specimen containing Ultrapave latex again exhibited a lumpy surface and carbon black settled out. Penetration at 77°F of AC-5, AC-10, AC-20 and AC-5 + Elvax was decreased by 15 to 20 percent. Penetration at 77°F of Microfil-8 + AC-5 was decreased by 35 percent. Penetration of AC-5 + Ultrapave latex actually increased by 20 percent while penetration of AC-5 + Kraton and Novophalt increased by about 7 percent. Data is presented in Table A 11, Appendix A.

Heat stability of these materials was also evaluated by exposure to 500°F for two hours in a covered penetration can. None of these procedures are standard tests for asphalt cements. Viscosity and penetration data were obtained after the heating procedure and compared to data that was obtained before heating (Table 11). Results show primarily that no hardening occurs in these materials when exposed to 500°F for two hours while protected from exposure to significant oxygen. Ultrapave latex and Kraton SBS/SB in Texaco AC-5, both exhibited a significant decrease in consistency. Obviously, the interpretation of results is quite subjective. Unpublished data from Shell and California DOT confirm that prolonged exposure of SBR and SBS modified asphalts to temperatures above 350°F for significant periods will cause a reduction in viscosity.

CHAPTER 5

LABORATORY TESTS ON ASPHALT CONCRETE MIXTURES

MIXTURE DESIGN

Two different aggregates were selected for use in the mixture study to provide a wide variation in mixture properties. The aggregate used in most of the mixture tests consisted of subrounded, silicious river gravel and similar sand with limestone crusher fines added to improve stability. This material was selected as the primary aggregate because it produces a relatively binder-sensitive mixture which accentuates the properties of the binders more than a high-stability mix. The secondary aggregate was composed of crushed limestone with field sand added to improve workability. This relatively absorptive, very angular material produces a high stability mix suitable for high-type roadway systems. Both of these aggregate blends are routinely used for paving construction in Texas. Details of these aggregate blends and gradations are given in Appendix B.

The asphalts used in this segment of the study include Texaco and San Joaquin (California) Valley products. Texaco AC-20 (in the control mixtures) and Texaco AC-5 modified with the five additives discussed previously were the primary binders for the mixtures. San Joaquin Valley AR-4000 in control mixtures and AR-1000 modified with additives were the secondary binders. The additives were incorporated into the mixtures using methods which simulate field conditions as closely as possible. For example, latex and carbon black were added to the hot asphalt-aggregate mixture and stirred for an extra one minute period during mixing; whereas, the other three additives were preblended in the asphalt cement before combining with the aggregate.

Optimum binder content was determined using the Marshall Method with emphasis on uniform air void content (density). The Marshall method was selected because it is much more sensitive to binder properties than the Hveem method. Results of the mix design procedures are given in Table B4,

Appendix B. Values in Table B4 may have been interpolated if tests were not actually performed at the selected optimum binder content.

Optimum binder content for most of the mixtures including river gravel and crushed limestone was about 4.5 percent. Mixtures containing carbon black require a slightly higher binder content. The primary reason for this is that the carbon black modified binder has a significantly higher specific gravity and the binder is added on an equivalent volume basis. Apparently, the carbon black reduces the lubricating effects of the binder thus producing a slightly higher air void content at a given compaction energy. On the average, mixtures containing the Texaco asphalts yielded higher Marshall stabilities than those containing the California Valley asphalts (Appendix B, Table B4). This is a direct result of the rheological properties of the binders. Mixtures made using crushed limestone, of course, gave higher stabilities than those made using river gravel.

In the mixture test program the values given for latex content are for the total weight percentage of liquid latex. Liquid latex is approximately 70 percent solids and 30 percent water. Therefore, 5 percent latex is equivalent to 3.5 percent rubber solids by weight of asphalt cement.

PREPARATION OF SPECIMENS FOR MIXTURE TESTING

Paving mixtures for the laboratory tests were produced using the river gravel and crushed limestone aggregates with the aforementioned binders. The test program is described in Figure 14. All mixtures were mixed and compacted at constant binder viscosities. That is, binder viscosity upon mixing was 170+20 cSt and upon compacting was 280+30 cSt. This was an attempt to produce specimens with approximately equivalent air void contents. Mixing and compaction temperatures for each binder are given in Appendix B. Test methods included Hveem and Marshall stability, resilient modulus at 5 temperatures, indirect tension at 3 temperatures and 3 loading rates, and an assessment of resistance to damage by moisture. River gravel specimens were prepared using 50-blow Marshall compaction; limestone specimens were prepared using 75-blow compaction. Marshall compaction was used in this portion of the laboratory study because it affords control of

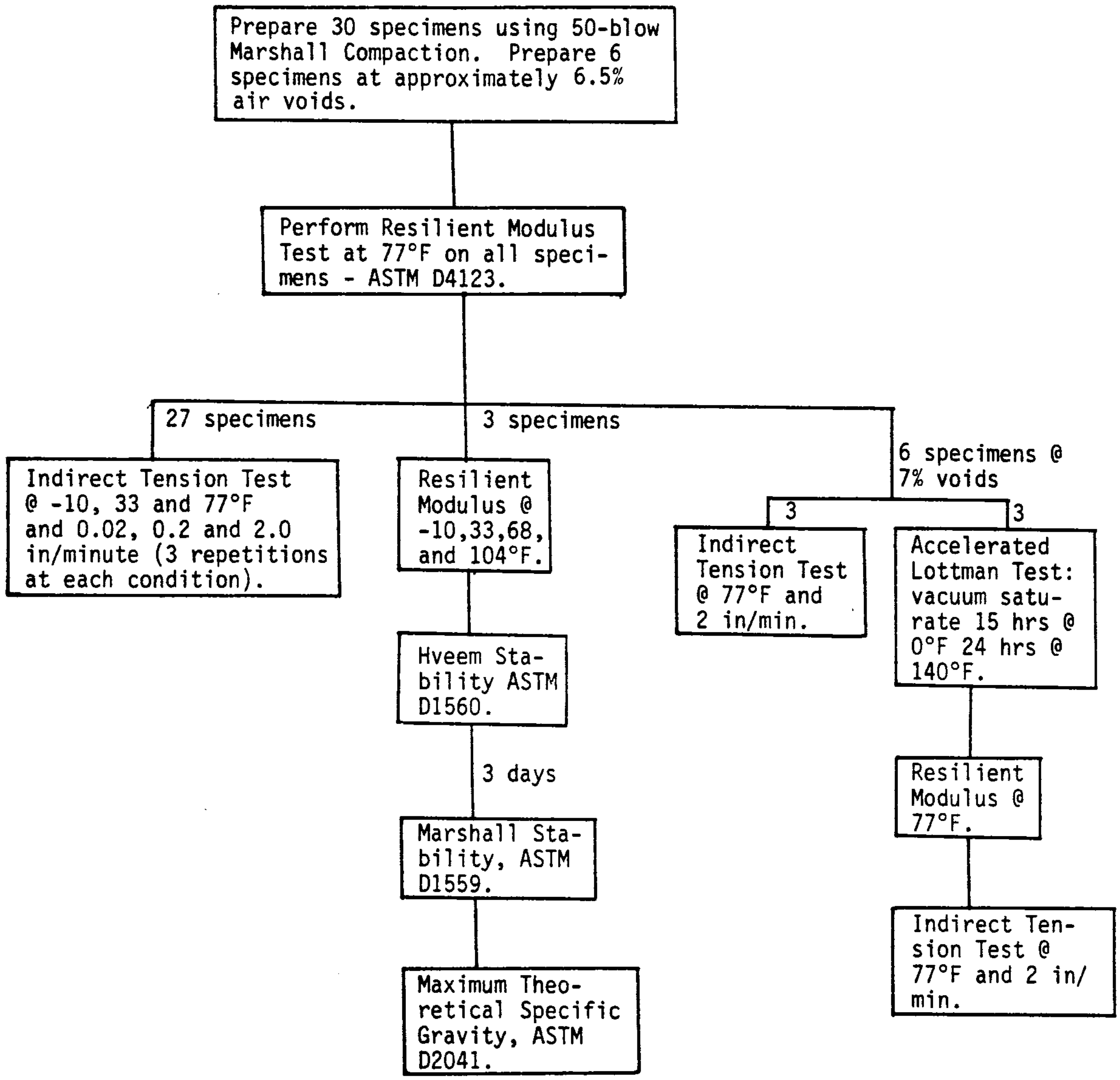

Figure 14. Test program to evaluate asphalt concrete mixtures.

compaction energy. Moisture-treated specimens are typically compacted to approximately 6.5 percent voids to allow sufficient intrusion of the water.

MARSHALL STABILITY

None of the mixtures containing modified AC-5 or AR-1000 binders exhibited Marshall stabilities greater than the AC-20 or AR-4000 controls (Tables 12 and 13). However, all of the modifiers showed the capacity to improve stability over that of the AC-5 or AR-1000 control mixtures. No single additive showed the ability to produce mixtures with consistently higher Marshall stabilities than the other additives. Kraton and Novophalt generally exhibited the greatest improvements.

Marshall flows for these laboratory mixtures were often below values specified by most highway agencies. This is the nature of this river gravel mixture which was specifically chosen because of its sensitivity to binder properties and should not be a concern.

After collection of significant data, it is surmised that the design asphalt content selected for the latex modified mixture with Texaco asphalt was slightly higher than it should have been. As a result, the latex mixture probably exhibited lower air void content, stability and stiffness than it should have.

HVEEM STABILITY

Hveem stability (Table 12 and 13) is largely dependent upon interparticle friction of the aggregate and does not correlate well with binder properties. However, the test was performed because the Texas SDHPT employs it in mix design procedures. As one might expect, there were no correlations between Hveem stability and the additives utilized for either of the two mix types. The latex plus Texaco AC-5 mixture exhibited the lowest Hveem stability; this may have been a result of excessive binder content as mentioned earlier.

No particular problems were encountered in determining Hveem stability of these modified mixtures. It appears, therefore, that the Hveem design

Table 12. Resilient modulus and stability of mixtures containing Texaco asphalt and river gravel.

Type Mixture	Air Void Content, Percent	Hveem Stability	Marshall Test		Resilient Modulus, psi x 10^3				
			Stability	Flow	0°F	33°F	68°F	77°F	104°F
Control: AC-20	5.0	43	1600	8	2200	1600	700	470	110
Control: AC-5	4.3	43	900	9	1800	1200	270	160	34
AC-5 + 15% Microfil 8	5.5	42	900	8	1700	1100	250	140	36
AC-5 + 5% Elvax 150	4.9	46	1100	9	-	-	300	220	45
AC-5 + 5% Kraton D	4.6	47	1300	7	-	-	380	290	47
AC-5 + 5% Latex	4.1	41	1000	10	1800	1500	250	150	35
AC-5 + 5% Novophalt	5.5	51	1300	8	-	-	470	370	69

Table 13. Resilient modulus and stability of mixtures containing SanJoaquin Valley asphalt and river gravel.

Type Mixture	Air Void Content, Percent	Hveem Stability	Marshall Test		Resilient Modulus, psi x 10^3				
			Stability	Flow	10°F	32°F	68°F	77°F	104°F
Control: AR-4000	4.4	49	1200	7	2000	1700	900	710	93
Control: AR-1000	4.1	48	700	6	2000	1400	250	140	25
AR-1000 + 15% Microfil 8	5.0	50	1200	7	1900	1600	430	250	40
AR-1000 + 5% Elvax 150	5.2	44	600	7	2000	1500	240	120	19
AR-1000 + 5% Kraton D	4.8	46	900	6	2000	1500	370	210	29
AR-1000 + 5% Latex	5.1	48	800	6	1900	1500	370	230	32
AR-1000 + 5% Novophalt	5.3	46	950	5	2000	1600	460	280	39

method would be suitable for application when using these types of binders but would not be sensitive to differences in binder properties.

It should be pointed out that all specimens were compacted using the Marshall hammer. However, the Hveem stability values should be valid for comparisons within this study.

RESILIENT MODULUS

Mixture stiffness was measured in accordance with ASTM D 4123-82 using the Mark III Resilient Modulus device. Typically, a diametral load of approximately 72 pounds was applied for a duration of 0.1 seconds while monitoring the diametral deformation perpendicular to the loaded plane. The load is normally reduced to about 20 pounds for tests performed at 100^{O}F or higher to prevent damage to specimens. Resilient modulus measured over a range of temperatures is used to estimate mixture temperature susceptibility. Test results are given in Tables 12 and 13 and plotted in Figures 15 and 16.

Results at the low temperatures (33^{O} to 10^{O}F) are typical; that is, resilient modulus approaches a limiting value of about 2 million psi. At the higher temperatures (above 60^{O}F), however, the additives exhibit the capacity to increase resilient modulus of the mixtures. The rheological properties of the binders strongly influence the resilient modulus values. Resilient modulus of the AC-20 or AR-4000 mixtures was consistently higher than corresponding modified mixtures. Analysis of variance using $\alpha = 0.05$ and Duncan's multiple range test showed that resilient modulus of the additive modified mixtures was significantly different from the control mixtures (AC-20 and AR-4000) at 68^{O}F and higher, but not at 33^{O}F and below. On the average, Novophalt and Kraton showed the greatest increases in mixture stiffness at the higher temperatures.

Although pavement performance data based on resilient modulus has not been established, it appears that the ideal binder should provide low mixture stiffness at low temperatures to improve flexibility and reduce cracking and/or provide higher mixture stiffness at high temperatures to reduce permanent deformation.

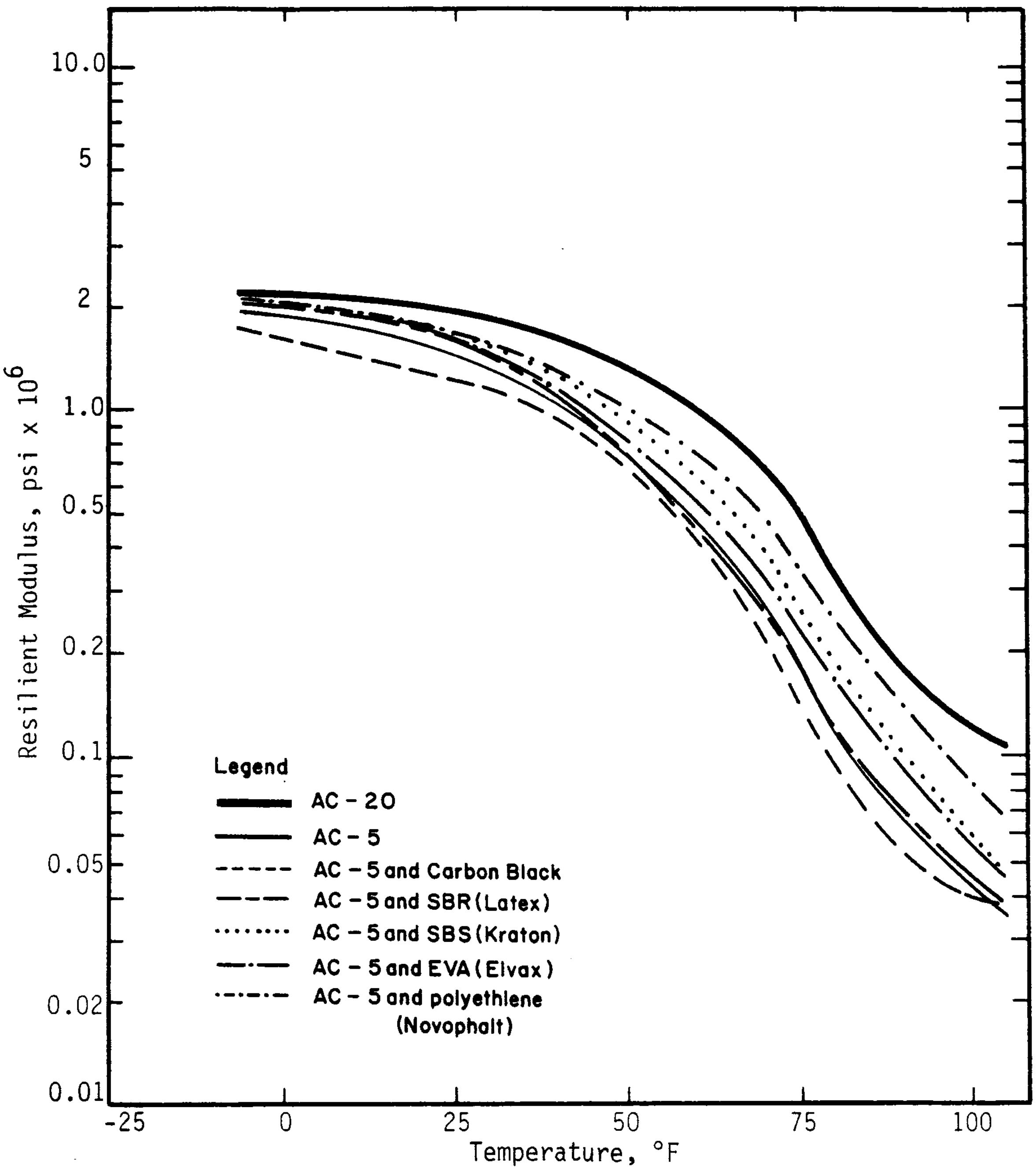

Figure15. Resilient modulus as a function of temperature for river gravel mixtures containing Texaco asphalts with and without additives.

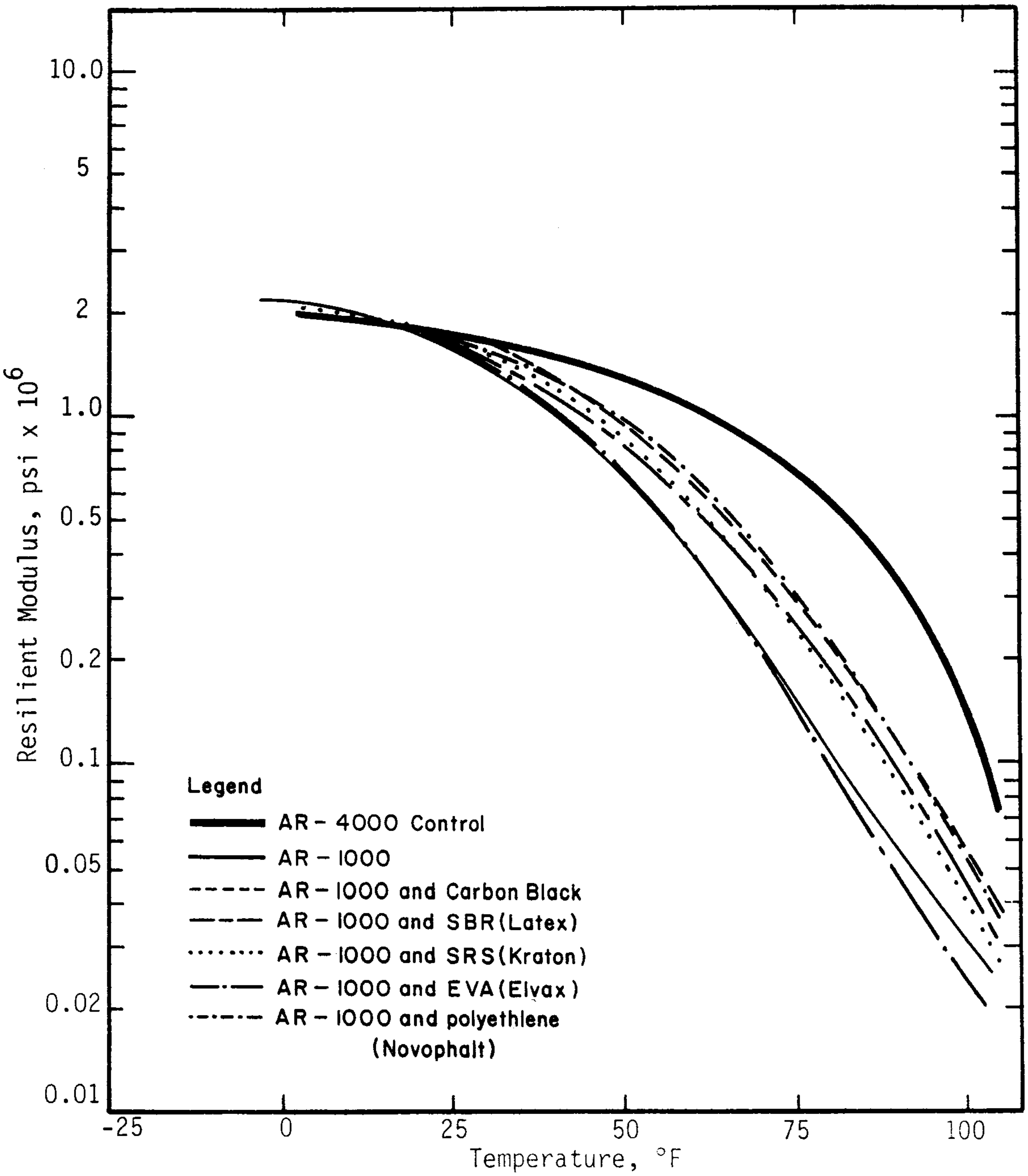

Figure 16. Resilient modulus as a function of temperature for mixtures containing San Joaquin Valley asphalts with and without additives.

INDIRECT TENSION

The indirect tension test employs the indirect method of measuring mixture tensile properties. The 2-inch high and 4-inch diameter cylindrical specimens were loaded diametrally at a constant rate of deformation until complete failure occurred. Diametral deformation perpendicular to the loaded plane was monitored in order to quantify mixture stiffness. Tests were conducted at nominal temperatures of -15, 33 and 77^{o}F and deformation rates of 0.02, 0.2 and 2-inches per minute on specimens made using the Texaco asphalts. Data are tabulated in Appendix C, Tables C1 through C3 and plotted in Figures 17 through 22. Strain at failure is the total diametral strain in the specimen at the maximum load in the plane perpendicular to the applied load. Secant modulus is the slope of the straight line on the stress strain plot from the origin to the point of maximum stress and corresponding strain, thus the term "secant".

Regarding the Texaco asphalt mixtures, the AC-20 control mixture consistently exhibited the greatest tensile strength at 77^{o}F and all loading rates. At lower temperatures, tensile strength of the AC-20 control mixture appeared to reach a maximum of about 400 pounds per square inch. Tensile strengths of the mixtures containing the AC-5 with or without an additive are shown to exceed 400 pounds per square inch by 10 to 25 percent. At low temperatures and the higher loading rates, all of the additives demonstrated the ability to increase mixture tensile strength over that of the AC-5 or AC-20 alone. Furthermore, the mixtures containing AC-5, with and without an additive, generally required significantly more strain to produce failure at the intermediate temperatures than the mixtures containing AC-20.

Tensile strengths at 77^{o}F and 2-inches per minute of the mixtures made using the San Joaquin Valley asphalt (Table 14) are generally greater than those made using the Texaco asphalts (mixtures containing Kraton are an exception). This may be due to the greater compatibility of the additives with the San Joaquin Valley material.

At very low temperatures (as those experienced in northern regions of the United States) and high loading rates (as those induced by traffic), soft asphalts modified with the additives studied herein have the potential to

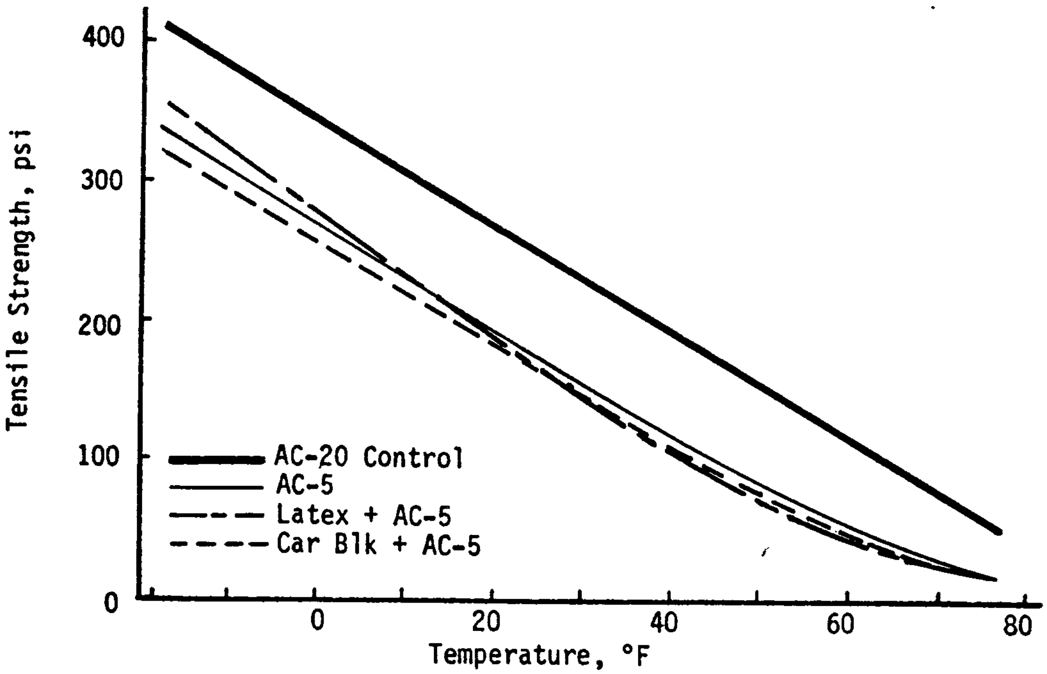

Figure 17a

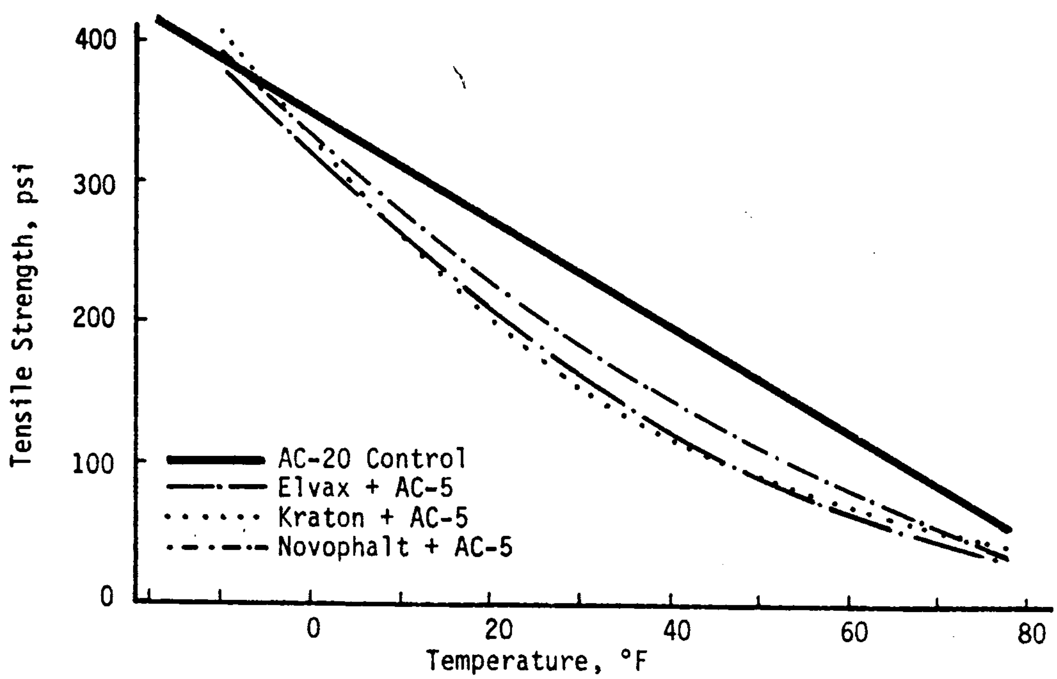

Figure 17b

Figure 17. Tensile strength as a function of temperature for displacement rate of 0.02 in/min for Texaco asphalt mixtures.

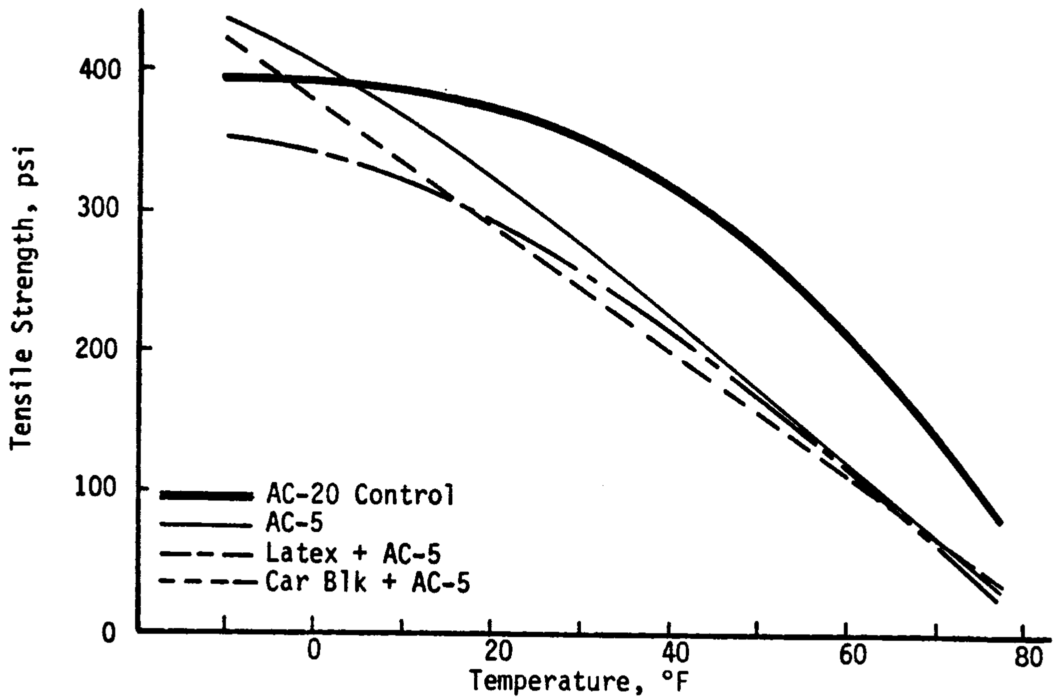

Figure 18a

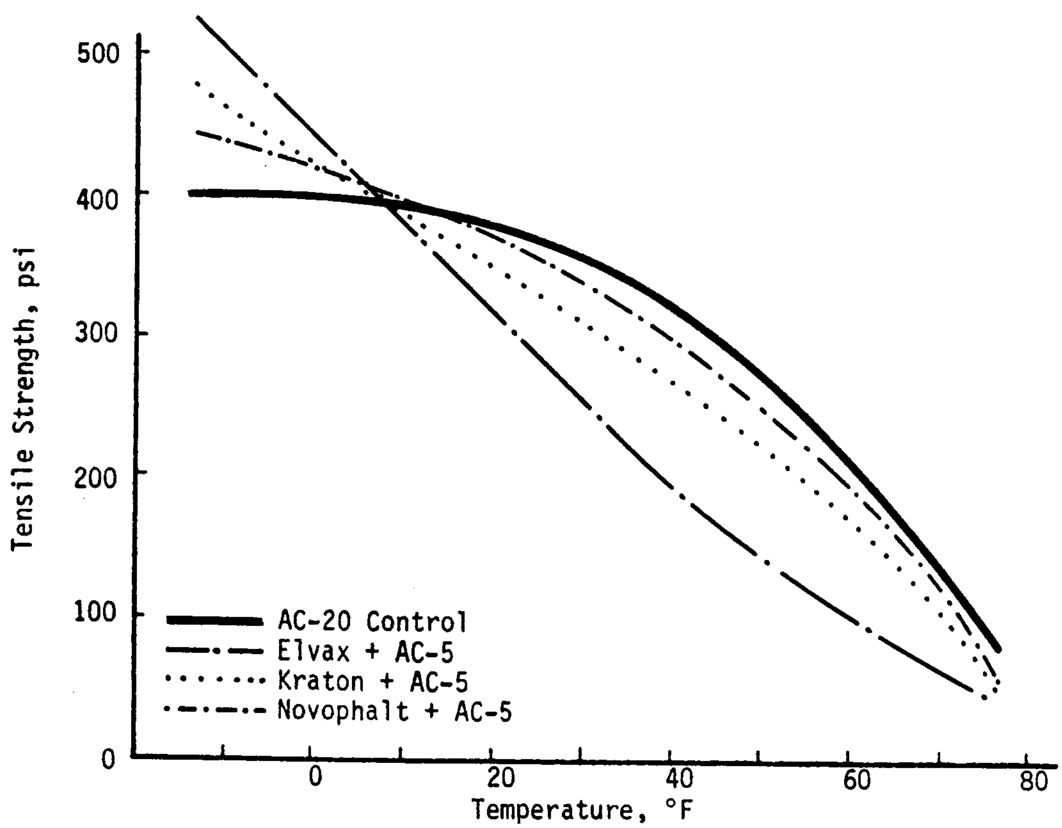

Figure 18b

Figure 18. Tensile strength as a function of temperature for displacement rate of 0.2 in/min for Texaco asphalt mixtures.

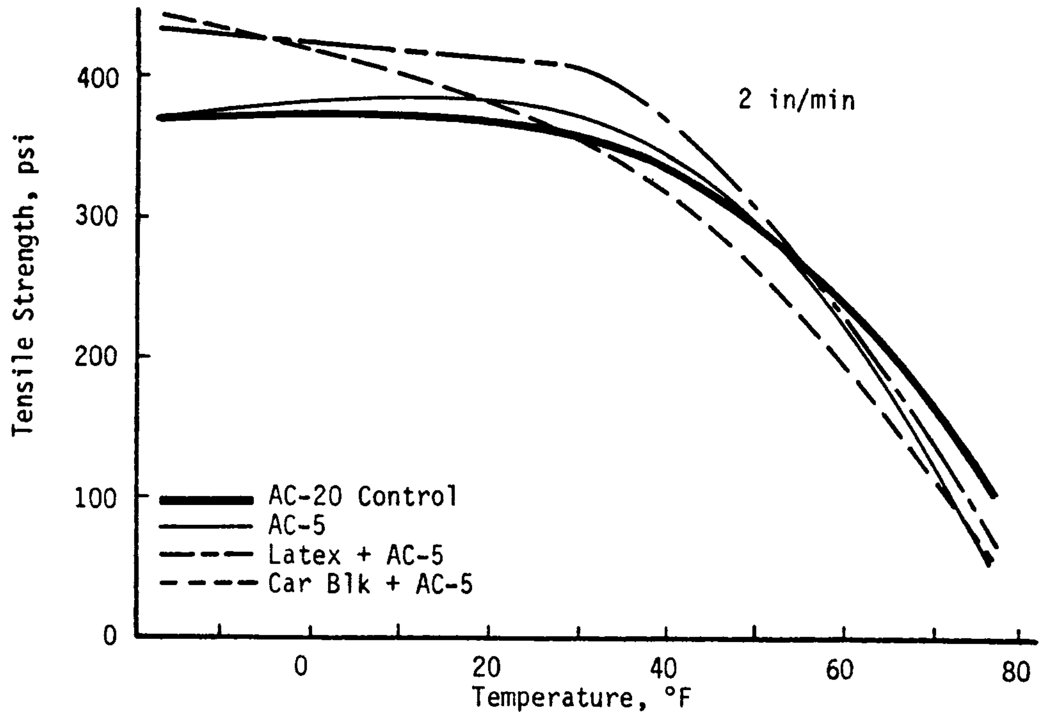

Figure 19a

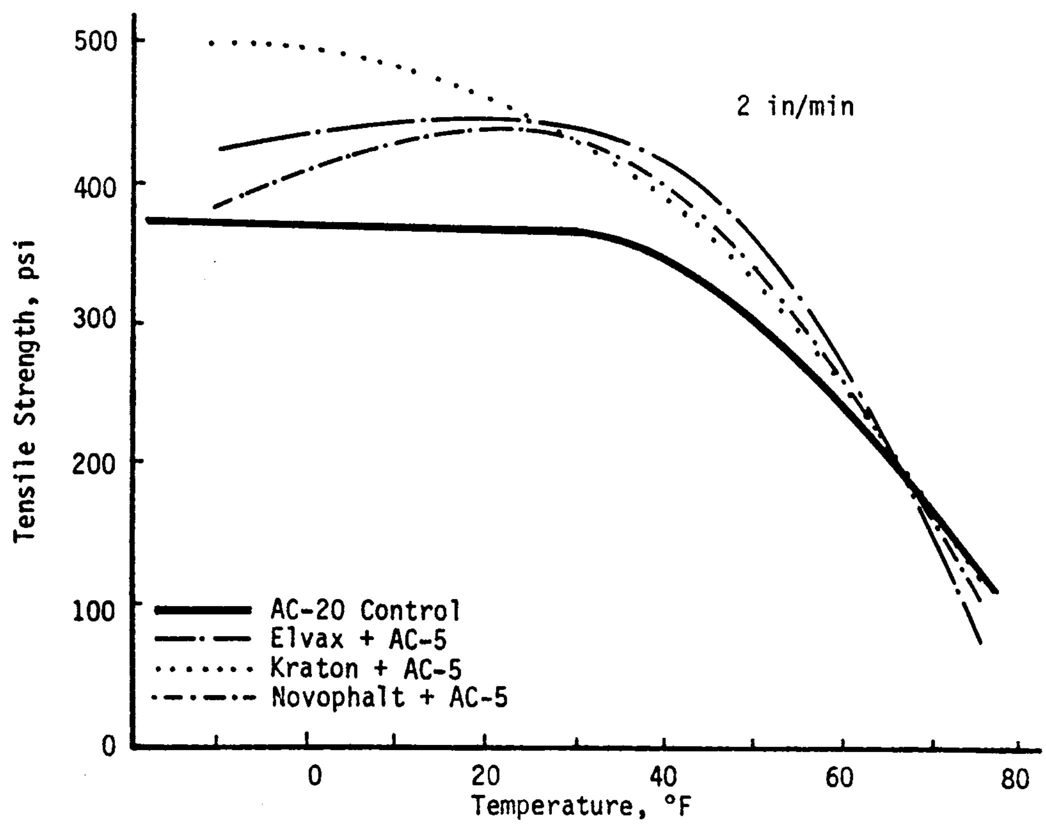

Figure 19b

Figure 19. Tensile strength as a function of temperature for displacement rate of 2 in/min for Texaco asphalt mixtures.

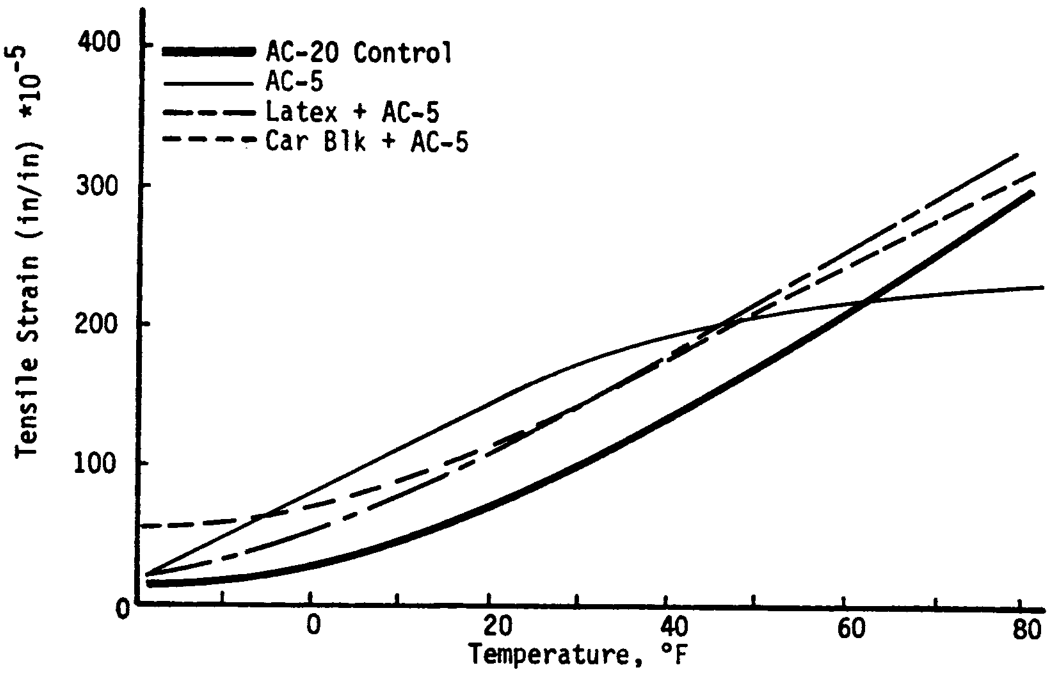

Figure 20a

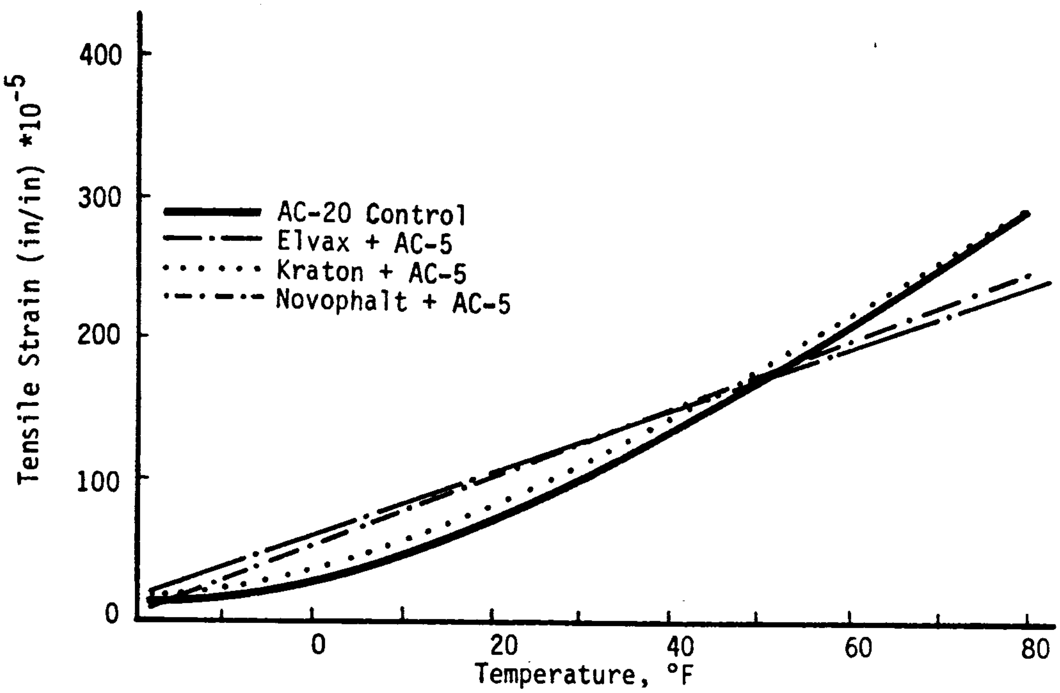

Figure 20b

Figure 20. Tensile strain at failure as a function of temperature for a displacement rate of 0.02 in/min for Texaco asphalt mixtures.

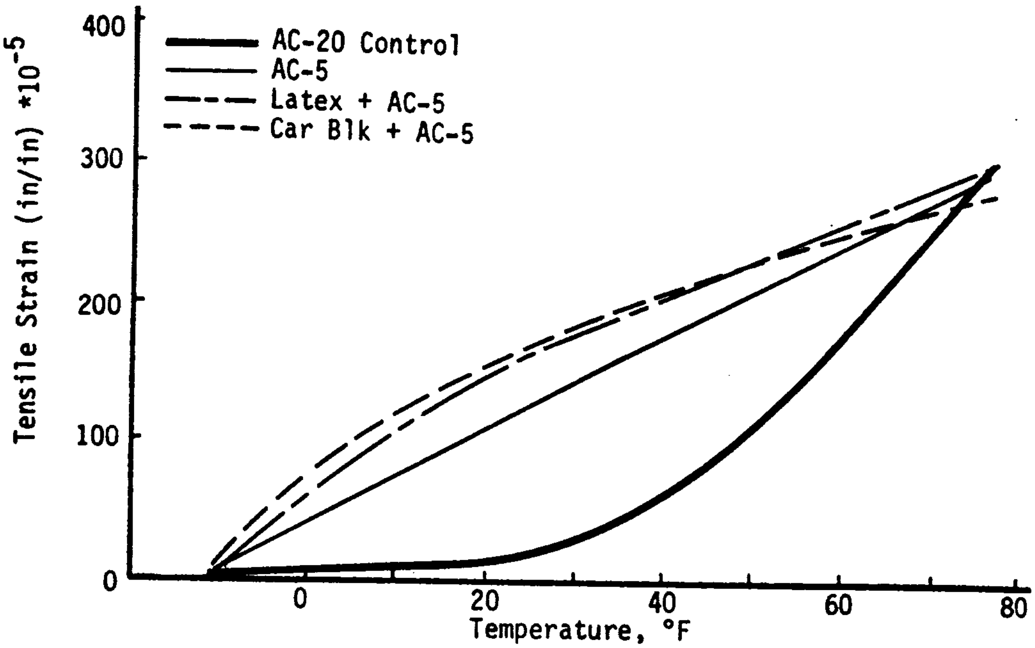

Figure 21a

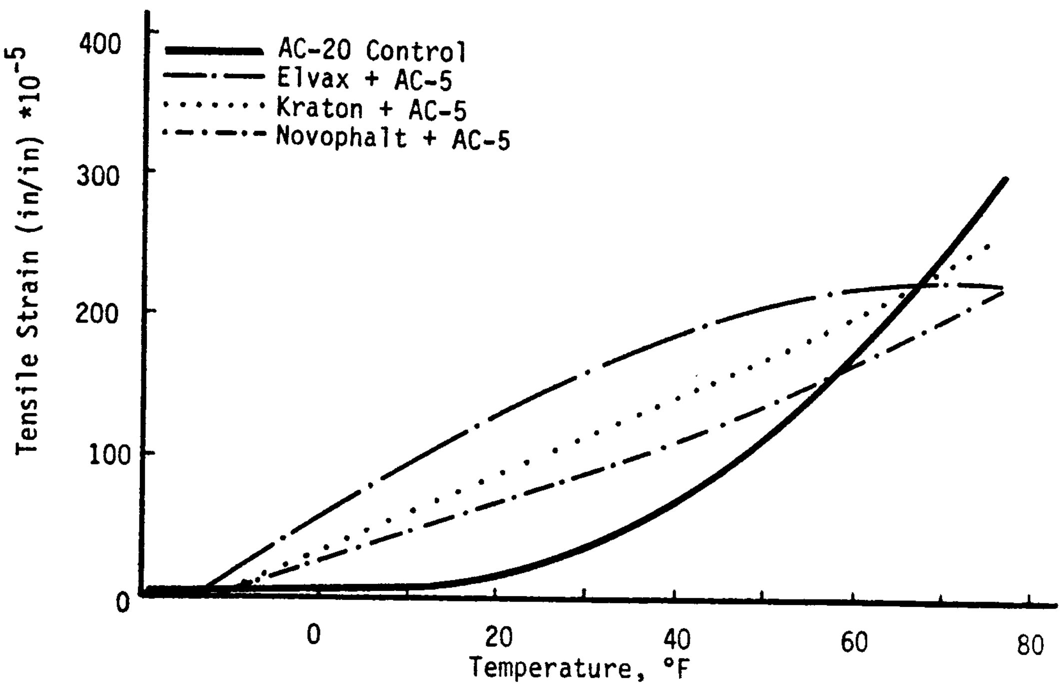

Figure 21b

Figure 21. Tensile strain at failure as a function of temperature for a displacement rate of 0.2 in/min for Texaco asphalt mixtures.

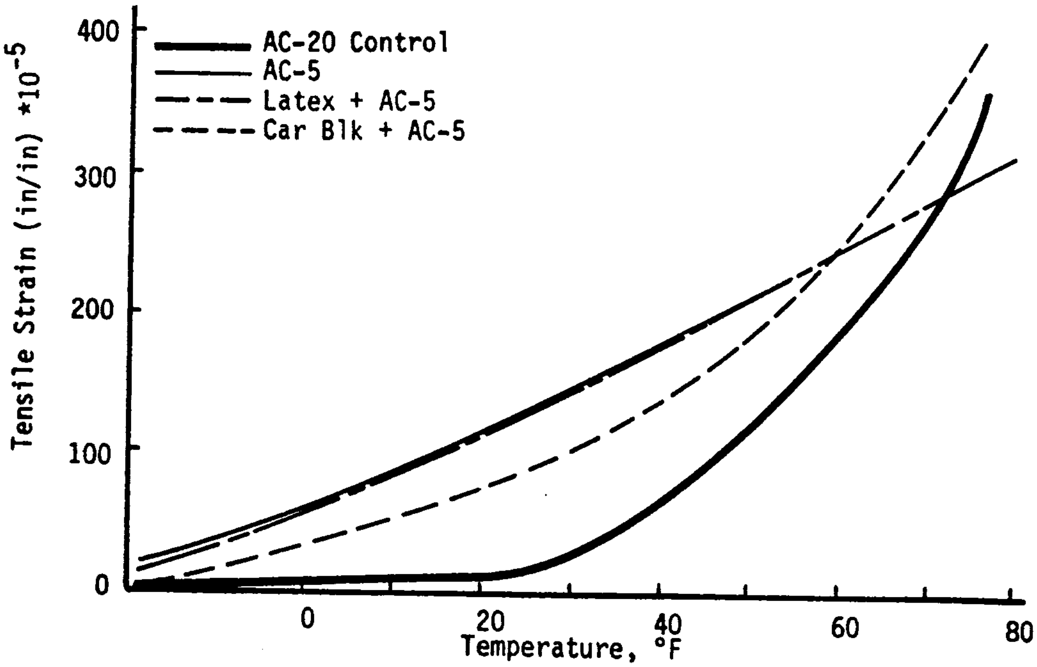

Figure 22a

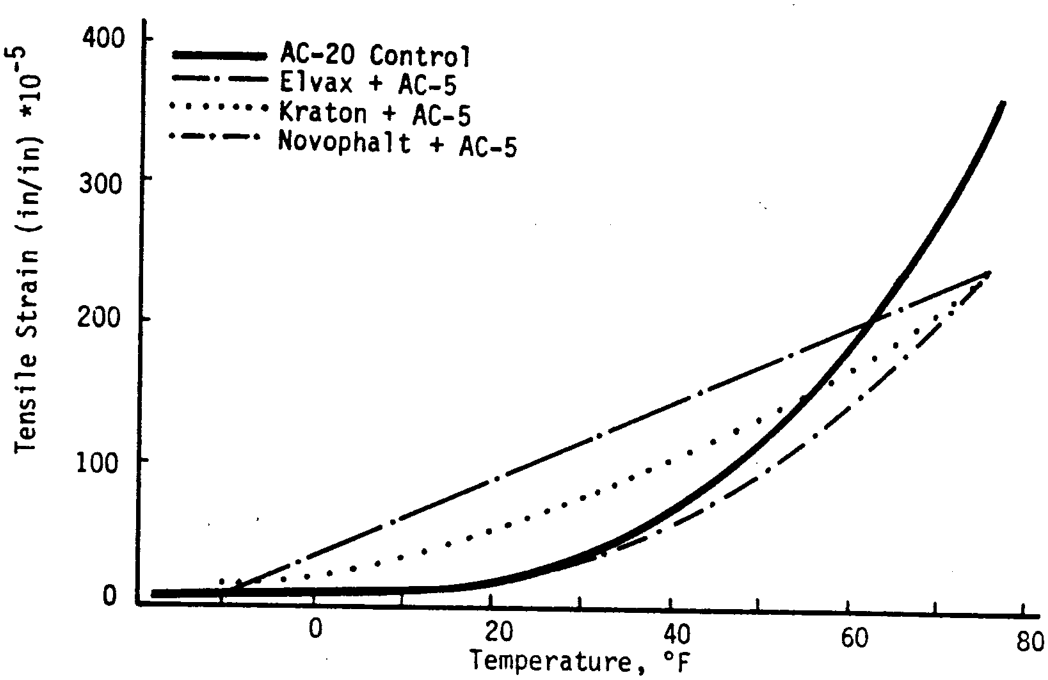

Figure 22b

Figure 22. Tensile strain at failure as a function of temperature for a displacement rate of 2 in/min for Texaco asphalt mixtures.

Table 14. Tensile properties and resilient modulus of mixtures (SanJoaquin Valley asphalt and river gravel).

Type Mixture	Air Void Content, percent	Resilient Modulus @ 77°F, psi x 10^3	Tensile Properties*		
			Tensile Strength, psi	Strain @ Failure, in/in	Secant Modulus, psi
Control: AR-4000	4.4	720	260	0.0028	95,000
Control: AR-1000	4.2	140	80	0.0032	25,000
AR-1000 + 5% Latex	5.1	220	100	0.0028	35,000
AR-1000 + 15% Microfil 8	5.2	260	130	0.0029	46,000
AR-1000 + 5% Kraton D	4.9	200	110	0.0031	34,000
AR-1000 + 5% Novophalt	5.2	310	130	0.0025	53,000
AR-1000 + 5% Elvax 150	5.0	130	100	0.0042	25,000

*Tensile tests at 2 in/min and 77°F.

increase resistance to traffic induced cracking. This is inferred as a result of the increase in tensile strength and strain at failure (flexibility). However, since neither the tensile strength nor strain at failure is increased by the additives at low loading rates, the additives may not appreciably affect thermally induced cracking. Based solely on the results of these indirect tension tests, any increase in service life would be modest and cost effectiveness would be questionable. Positive statements regarding cost effectiveness can only be made upon completion of a significant number of controlled field trials.

MOISTURE RESISTANCE

Indirect tension and resilient modulus tests before and after exposure to moisture were used to evaluate the susceptibility of the mixtures to damage by moisture. The modified accelerated Lottman (46) moisture treatment consisted of vacuum saturating the specimens at a vacuum of 4-inches of mercury below atmospheric pressure at room temperature, wrapping them in cellophane to retain the moisture and freezing them at $0^{O}F$ for 15 hours followed by a 24-hour period at $140^{O}F$. The specimens were then brought to $77^{O}F$ and tested in accordance with the program depicted in Figure 14. Test results are given in Appendix C, Tables C4 and C5 and Figure 23. Normally, samples used in moisture testing are compacted to approximately 6.5 percent air voids; however, to economize and provide direct comparison with data in Table 14, the samples containing the San Joaquin Valley asphalts (Table C5) were compacted using standard procedures and the resulting void contents were approximately 4 percent.

Ratios for resilient modulus and indirect tension were calculated by dividing measurements after moisture treatment by those obtained on untreated specimens. These tests were performed to evaluate any changes in moisture sensitivity of the paving mixture effected by the additives.

The most obvious result from these data is that the mixtures made using the San Joaquin Valley asphalts are more susceptible to moisture damage than those made using the Texaco asphalts (Figure 23). This is consistent with predictions from the infrared analysis which showed a significantly higher

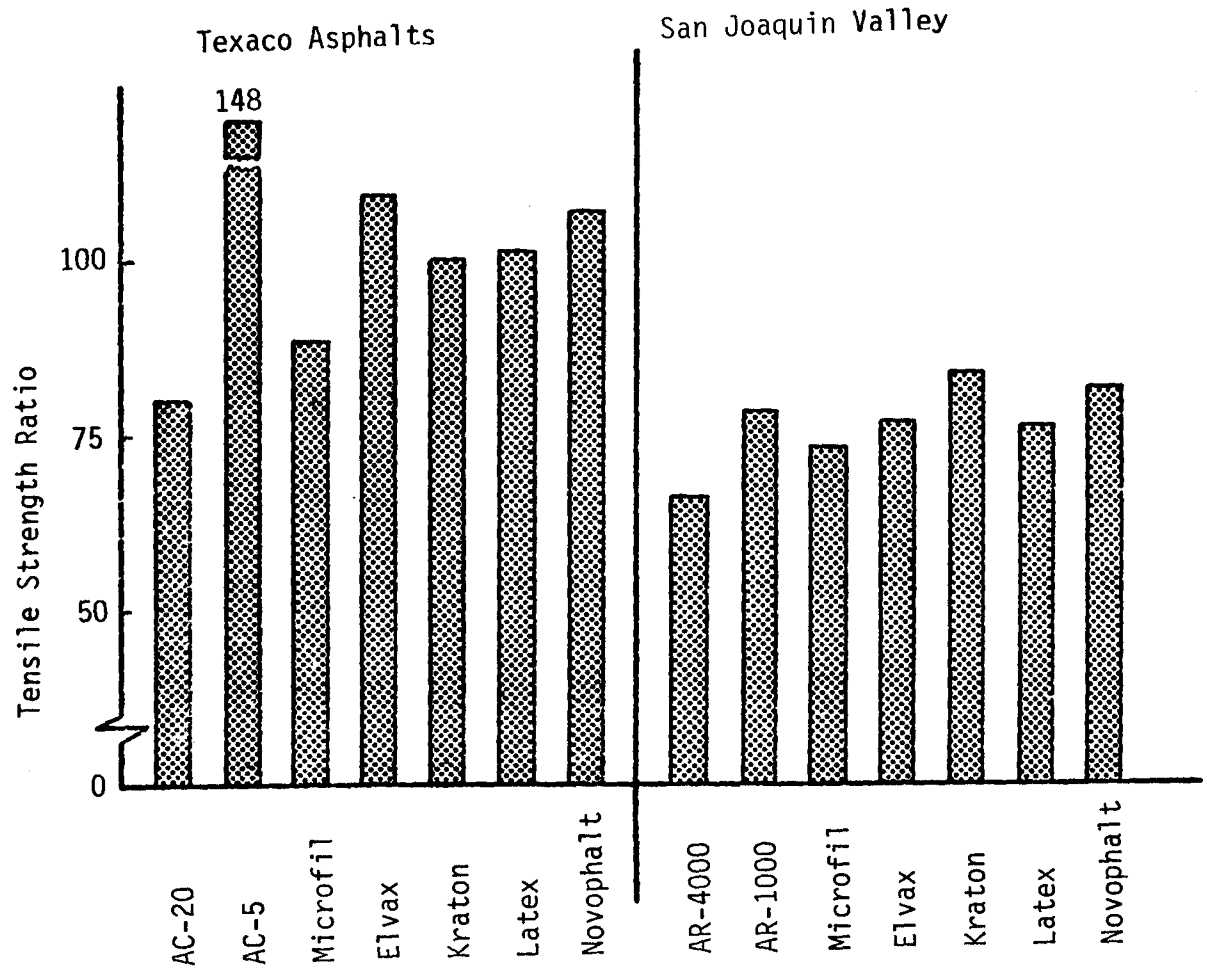

Figure 23. Tensile strength ratios for mixtures.

concentration of carboxylic acid salts in the San Joaquin Valley asphalt (1). When the Texaco and San Joaquin Valley asphalts are considered separately the mixtures containing the softer binders (AC-5 and AR-1000) with or without an additive always exhibited greater tensile strength ratios than the control mixtures containing the AC-20 and AR-4000.

It appears that, generally, the additives have little effect on moisture susceptibility of the mixtures made using the materials included in this study. Mixtures containing Microfil-8 exhibited slightly lower tensile strength ratios with both asphalts. Microfil-8 differs significantly in properties when compared to the polymers utilized. It is basically a granular material with no ability to coat an aggregate with a continuous film. In fact, surfaces of mixtures containing Microfil-8 had a "dry" appearance when compared with mixtures containing the other binders.

Resilient modulus ratios are generally supportive of the results obtained from the tensile strength ratios but showed considerably more scatter. Tensile strength ratios from this procedure are widely accepted as relatively sensitive measures of moisture susceptibility. Resilient modulus ratios were merely measured to add to the data base since the test is fast and inexpensive.

EXTRACTION AND RECOVERY WITH ADDITIVES

Asphalt concrete containing the asphalts and additives studied herein were extracted and the binders were recovered (1) (Appendix C, Tables C6 and C7). Some of the recovered binders were analyzed to determine the amount of the additives recovered. There were differences in the relative effectiveness of the hot (reflux) and cold (centrifugal) extraction methods. Some of the results with the San Joaquin Valley asphalts were contrary to those found for the Texaco asphalts. The limited number of tests did not establish whether the differences in extractability of the additives were specific to the asphalt used, or were due to other factors in the preparation and history of the asphalt concrete.

Since the conventional extraction methods do not remove all of the additives, data obtained for the amount of extracted binder and for properties of the recovered binders should be used only with the realization that a substantial fraction of the additive may remain in the extracted aggregate.

Analysis of some of the recovered binders showed that the amount of additive in the recovered binder may be determined by using an analytical method specific for the type of additive present. Content of carbon black can be determined by thermogravimetric analysis (TGA), and content of polyethylene in Novophalt can be determined by filtration of the dispersion in toluene or trichlorethylene. Determination of the content of SBR (e.g. from latex) by centrifuging and filtering of the dispersion in methyl isobutyl ketone appeared promising; but further effort would be required to develop a reliable method. The analytical methods must be standardized using the specific asphalt involved, since asphalts from different sources will yield different "blank" values for the analyses.

In summary, the paving engineer should recognize that standard extraction methods are not totally effective for extracting modified asphalts from paving mixtures.

EVALUATION OF FATIGUE CRACKING POTENTIAL

Approach

The potential of mixtures of asphalt concrete modified by asphalt additives to crack due to cyclic fatigue was evaluated by Little, et al (1) using two approaches: (1) the phenomenological beam fatigue approach and (2) a fracture mechanics based controlled displacement approach. This subsection summarizes research results presented in reference 1.

The phenomenological regression approach is the most common method used in fatigue testing or analysis of highway materials. The very familiar relationship used to represent the fatigue response is of the form:

$$N_f = K_1 \, (1/\varepsilon_t)^{K2} \qquad \text{Equation 2}$$

where N_f is the number of repetitions to failure, ε_t is the repeatedly induced tensile strain and K_1 and K_2 are regression constants. These parameters are influenced by several variables including type and rate of load, type of test, mixture properties and temperatures. Hence, K_1 and K_2 are not material properties.

The beam fatigue test may be performed either in a controlled strain or controlled stress mode. The proper test mode depends on the type of pavements being simulated. Epps and Monismith (47) reported that a controlled stress mode of loading is encountered in thick, stiff pavements typically 6-inches thick or more. Controlled strain loading is, on the other hand, typically encountered in thin pavement sections (2-inches or less).

Generally, the phenomenological approach provides a reasonably simple approach which has been almost universally adopted. However, it bears the limitation that it cannot account for both crack initiation and propagation. Such distinctions may be very important in establishing the fatigue life of a new material expected to be used for a wide range of applications. It seems reasonable that a stiff but brittle material may perform well in a controlled stress laboratory test, but fail rapidly due to immediate crack propagation if the material is used in situ where controlled strain is the mode of cyclic applications.

The fracture mechanics-based approach employs a device which applies a controlled displacement to an asphalt concrete beam. The device was developed at Texas A&M (48) and is called the overlay tester as it was initially used to simulate the controlled displacement opening and closing of a crack beneath an asphalt concrete overlay. Fracture mechanics techniques are used to evaluate the energy required to propagate the crack through the material.

In summary, two testing techniques were used to evaluate the potential of asphalt concrete mixtures to fail in fatigue. First, the controlled stress beam fatigue test was used to simulate controlled stress as induced due to repeated applications of a design load. Second, the controlled displacement (overlay) test was used to simulate the controlled cyclic strains imparted to a pavement due to movement of the underlying fractured pavement, such as joint movement in a PCC pavement. These tests should yield a thorough analysis of the fatigue potential of the materials evaluated.

Sample Fabrication

Beams 3x3x15-inches were prepared using the Cox kneading compactor for both controlled stress (flexural beam fatigue) testing and controlled displacement (overlay) testing. Mixing and compaction temperatures for the various asphalt-additive blends were determined based on viscosity versus temperature data.

The temperatures required for mixing based on the viscosity data were often quite high. For example, asphalt blends with latex required mixing temperatures of 405^{o}F and 414^{o}F, respectively, for AC-5 and AR-1000. These temperatures are not practical under field conditions and more realistic mixing temperatures of 340^{o}F and 315^{o}F, respectively, were used. The predicted compaction temperatures based on viscosity versus temperature data was adjusted downward for each asphalt-additive blend, except carbon black.

It was virtually impossible to compact mixtures at the predicted compaction temperatures due to excessive shoving under the compaction foot. Adjusted mixing and compaction temperatures are listed in Appendix D, Table D1.

All additives and asphalts were heated to 300^{o}F for 40 minutes and poured into separate cans prior to mixing. Even so, large lumps were observed in the pre-blended additives (Kraton and Elvax). These additives were heated for an additional 40 minutes to completely melt the lumps. The blending procedure should be explicitly identified when such additives are used.

A target air void contents of 6-percent was established for each beam. In order to minimize void content variability among samples, it was necessary to alter the compaction procedures specified by the VESYS User's Manual (49). This problem was magnified because of the poor degree of interlocking among the smooth, rounded river gravel particles resulting in easy shearing and shoving of the mixture.

A second problem was within-sample variation in air void content. For example, density gradients from top to bottom of the beams were identified. For beams with a 6-percent air void content, it was typical to measure 7.5 to 8-percent air voids in the top, 6-percent in the middle and 4 to 5-percent in the bottom of the beam. To minimize the problem, a stepwise increasing compaction pressure was used. Low pressure at the early stage of compaction was used to stabilize the sample, followed by high pressures to reduce the air void contents.

A trial and error method was used to determine the proper compaction procedure. The procedure resulted in a difference in air void content from top to bottom of the beam of less than 0.5 percent.

The target air voids content was achieved for all mixtures except those containing carbon black. For these beams, it was much more difficult to compact the specimens to the 6-percent air void level. Even when twice the compactive effort was applied, the air void content could only be reduced to about 7-percent. Consequently, the void contents for the samples containing carbon black are from 1/2 to 1-percent higher than for the other samples. This difference in compaction is largely due to the higher mass viscosity of the carbon black modified mixture.

Controlled Stress Flexural Fatigue

Experiment Design. Flexural beam fatigue testing was performed as shown in Figure 24. All testing was on beams fabricated with a silicious river gravel aggregate of the gradation and specifications shown in Appendix B. The production quality of each beam was controlled by assuring an air void content of between 5.5 and 6.5 percent for all beams except those containing carbon black where the range was 6.5 to 7.0 percent.

Nine beam samples were tested at each combination of variables. Three beams were tested at each of three stress levels (low, intermediate and high). The logarithm of the strain, ε_t, induced at the 200th repetition at the stress level in question was plotted versus the logarithm of the number of load cycles to failure, N_f. A least squares regression curve was fitted through the data to determine the characteristic parameters K_1 and K_2.

Aggregate		River Gravel (Silicious)					
Curing Condition	**Binder / Test Temperature, °F**	**AC-20**	**AC-5 Carb.Blk.**	**AC-5 Latex**	**AC-5 Novo.**	**AC-5 SBS**	**AC-5 EVA**
Normal	33	9*	9	9	9	9	9
Normal	68	9	9	9	9	9	9
14 days @ 140°F	33						
14 days @ 140°F	68	9	9	9	9	9	9

*9 samples were tested for each cell, 3 at each of 3 stress levels.

Figure 24. Test matrix for flexural beam fatigue testing.
°C = (°F-32)/1.8 (After Reference 1)

Test Results. A summary of the flexural beam fatigue data (controlled stress) is given in Appendix D, Table D2. In order to more easily evaluate the relative fatigue response, the 68°F data are plotted in Figure 25 and the 32°F data are plotted in Figure 26. Based on the log ε_t versus log N_f plots the following trends are apparent:

1. At 68°F, each additive blend with AC-5 produced a mixture which has statistically superior fatigue properties compared to the control mixture using AC-20 asphalt as the binder. Although the plots of fatigue results from mixtures containing AC-20, AC-5 with Novophalt and AC-5 with carbon black are closely grouped, they are statistically different (α = 0.05). Statistical difference is defined as when either the intercept or slope or both are different.

2. At 68°F, the mixtures containing blends of AC-5 and EVA (Elvax), AC-5 and SBR (Latex) and AC-5 and SBS (Kraton) performed the same for practical purposes; however, the fatigue plots are statistically different (α = 0.05). These mixtures showed significantly superior fatigue responses to the mixtures containing either blends of AC-5 and Microfil-8 or AC-5 and polyethylene (Novophalt).

3. At 32°F, the modified AC-5 asphalt blends once again provided a response superior to the control. Fatigue results among mixtures containing blends of AC-5 and polyethylene (Novophalt), SBS (Kraton), SBR (Latex) and EVA (Elvax) were not significantly different.

4. The levels of applied flexural stress, strain at the 200th load cycle and cycles to failure are documented in Tables D3 through D8 of Appendix D. Stress levels over the range of approximately 200 psi to approximately 475 psi were used for all mixtures. The 200th cycle strains were substantially different among the mixtures tested at 68°F (see Tables D3 though D8). The general trend was a substantially more flexible response for AC-5 blends containing EVA, SBS (Kraton) and SBR (Latex).

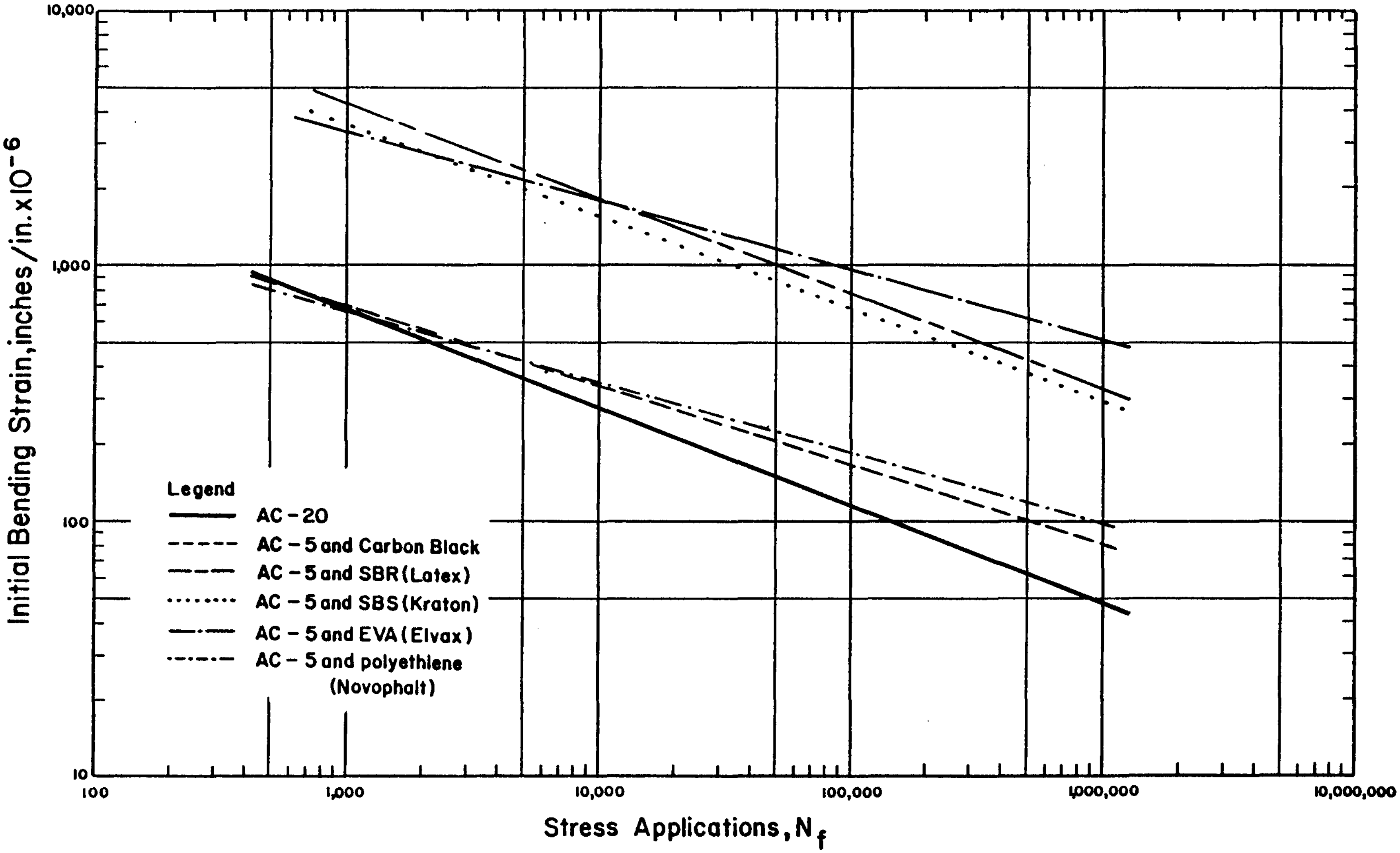

Figure 25. Controlled stress flexural beam fatigue results at 68°F (20°C). (After Reference 1)

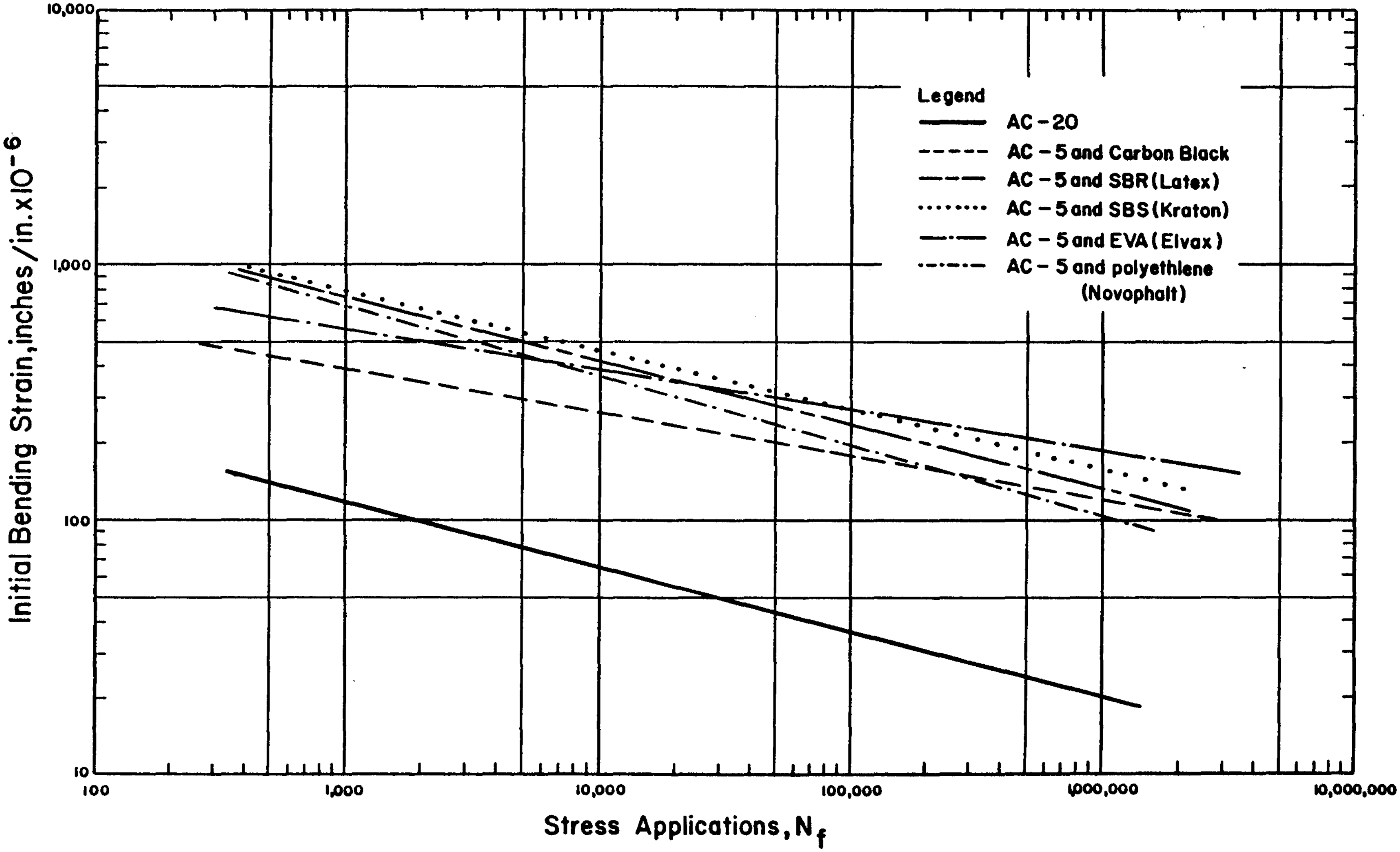

Figure 26. Controlled stress flexural beam fatigue results at 32°F (0°C). (After Reference 1)

Flexural Fatigue After Aging. Asphalt concrete mixtures are often subjected to extended periods of accelerated oxidative aging at high temperatures. The laboratory mixture fabrication procedure subjects mixtures to an environment similar to that of the hot mix plants. No standard procedure has been documented to simulate post construction oxidative aging in the field. However, laboratory testing at Texas A&M (50,51) has revealed that aging at 140°F substantially changes material properties such as resilient modulus and indirect tensile strength and, furthermore, that essentially all detectable changes in mixture properties occur within a 14-day period (52). Beams were aged for 14 days and tested in controlled stress flexural fatigue to evaluate the effects of accelerated oxidative aging (1). This should represent the effects of aging at substantially longer periods of oxidative aging in the field.

Table D9 (Appendix D) compares flexural controlled stress fatigue parameters K_1 and K_2 for aged and unaged specimens tested at 68°F. The R^2 values associated with each test indicate the degree to which the regression curves account for the variance between initial strain and cycles to failure. The K_1 and K_2 values are substantially different between the aged and unaged samples. The general trend is poorer fatigue response following aging. A more fracture-susceptible response is demonstrated by the generally higher K_1 values coupled with substantially low K_2 values which indicate a much steeper slope. Figure 27 illustrates the fatigue curves following accelerated aging.

Based on the data summarized in Table D9 and Figure 27, the following trends were identified (1):

1. The most dramatic effect of accelerated aging on fatigue life occurred in the SBS (Kraton) and EVA (Elvax) mixtures. A significant change in the slope of the fatigue curves revealed a much more rapid fatigue rate as stress level increases for those mixtures compared to their unaged counterpart mixtures.

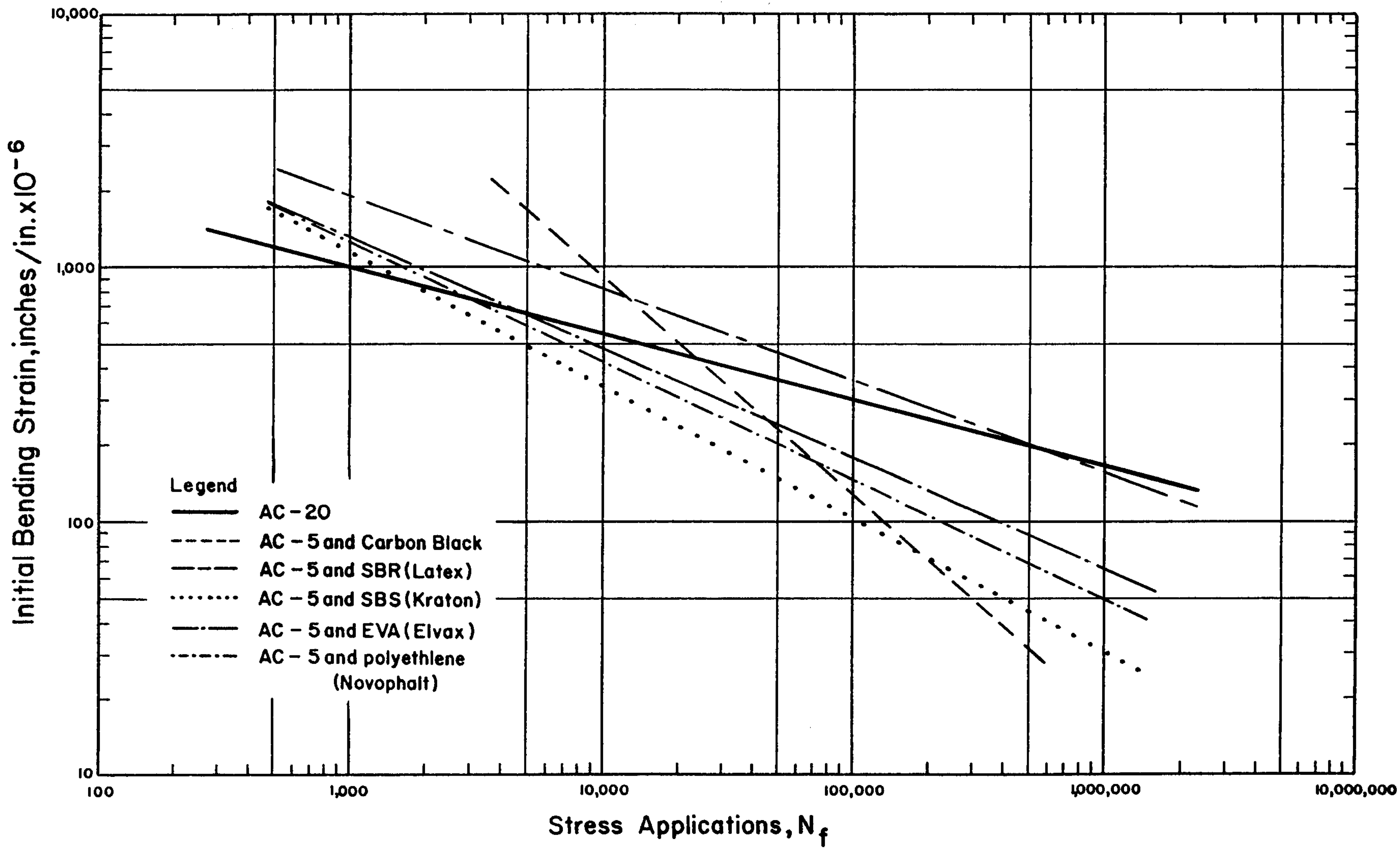

Figure 27. Controlled stress flexural fatigue results at 68°F (20°C) following 14-day aging at 140°F (60°C). (After Reference 1)

2. The aging effects on the SBR (Latex) and polyethylene (Novophalt) mixtures were less pronounced but highly significant and resulted in a substantially decreased fatigue life.

3. The accelerated aging period significantly improved the flexural fatigue response of the AC-20 mixtures in direct contrast to the effects on AC-5 asphalt-additive blends.

It is difficult to assess the results of the heat aging experiment. The complexities of the binder-additive compatibility no doubt have an effect.

General Discussion of Flexural Fatigue Results. The flexural fatigue data graphically presented in Figures 25 and 26 represents the ability of asphalt concrete samples to resist fatigue induced damage when a selected stress level is applied. The magnitude of strain produced by the stress level is, of course, dependent upon the stiffness of the mixture. This must be considered as the plots in Figures 25 and 26 and the data in Appendix Tables D3 through D8 are evaluated.

At 68^{O}F, the three stress levels used to test each mixture were approximately equal. However, the strains induced at the 200th load cycle were substantially larger for most AC-5 plus additive mixtures due to their lower stiffness values. In fact, mixtures composed of AC-5 blends with SBS and EVA produced strains which were about nine times larger than those produced in the AC-20 control mixture. The mixture containing latex resulted in strains about 15 times larger than those produced in the AC-20 control. A most interesting result is that the AC-5 mixture containing polyethylene was most similar to the AC-20 control in terms of level of induced flexural strain.

At 32^{O}F, the range of induced strains in all mixtures containing AC-5 and the polymers were quite similar. The mixtures containing polyethylene developed approximately the same level of induced strains as those containing latex, SBS and EVA. The AC-5 and carbon black mixture exhibited a significantly lower range of induced strains over a similar range of applied stresses.

In conclusion, although the mixtures containing AC-5 blends with latex, SBS and EVA exhibit superior fatigue performance at 68^{O}F based on the N_f

versus $_t$ (at 200th load cycle) criterion, the mixtures containing AC-5 blends with polyethylene and carbon black possess sufficient stiffness such that stress levels higher than for AC-20 are required to induce the critical strains. Based on the total analysis of fatigue data, mixtures containing AC-5 and polyethylene possess attractive fatigue properties as those mixtures combine good fatigue resistance based on the N_f versus $_t$ (200th load cycle) criterion, higher values of stiffness than other AC-5 and additive blends at $68^{o}F$ and similar values of stiffness and a similar N_f versus $_t$ (200th load cycle) relationship at $32^{o}F$.

Controlled Displacement Fatigue Testing

General. A mechanistic approach proposed by several researchers (43,53 and 54) considers fatigue as a process of cumulative damage and utilizes fracture mechanics to investigate this property. In this approach, fatigue life, under a given stress state, is defined as the period of time during which damage increases according to a crack propagation law from an initial state to a critical or final level. The method accounts for the changes in state of stress due to cracking, geometry and boundary conditions, material characteristics and variability. The fatigue life can be obtained from both controlled stress and controlled strain tests. The method is independent of the mode of testing.

Little, et al (1), in an FHWA sponsored study, applied this approach using controlled displacement testing to evaluate the five additives discussed previously in mixtures containing the river gravel with the Texaco and San Joaquin Valley asphalts. Results of these tests are presented below. A detailed discussion of the theory and mechanics of the test is given in Reference 41. They used the overlay tester to evaluate controlled displacement fracture resistance. The overlay tester, shown schematically in Figure 28, simulates the horizontal movement of an existing crack below an asphalt concrete surface. The existing crack may be a portland cement concrete joint or a crack in an old asphalt concrete pavement or stabilized base.

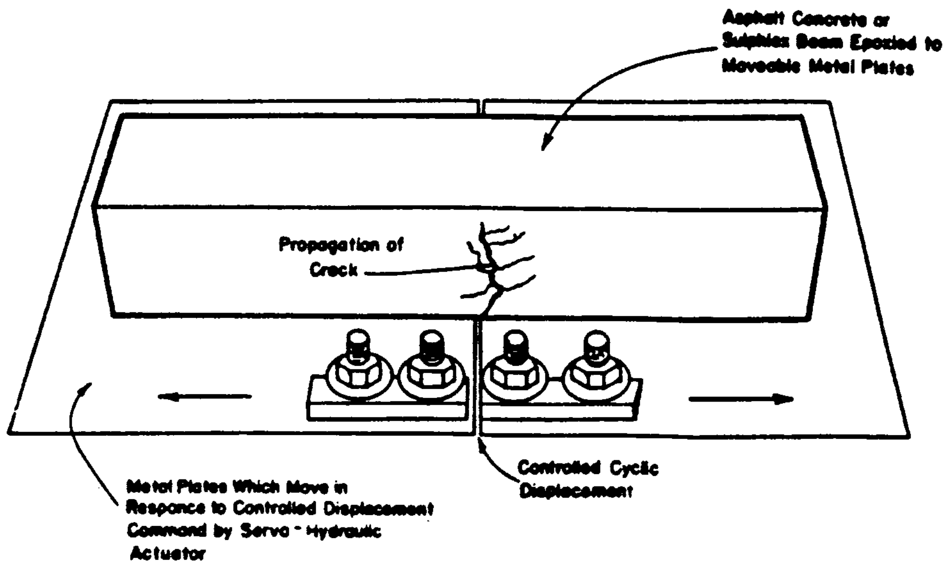

Figure 28. Schematic of overlay tester.

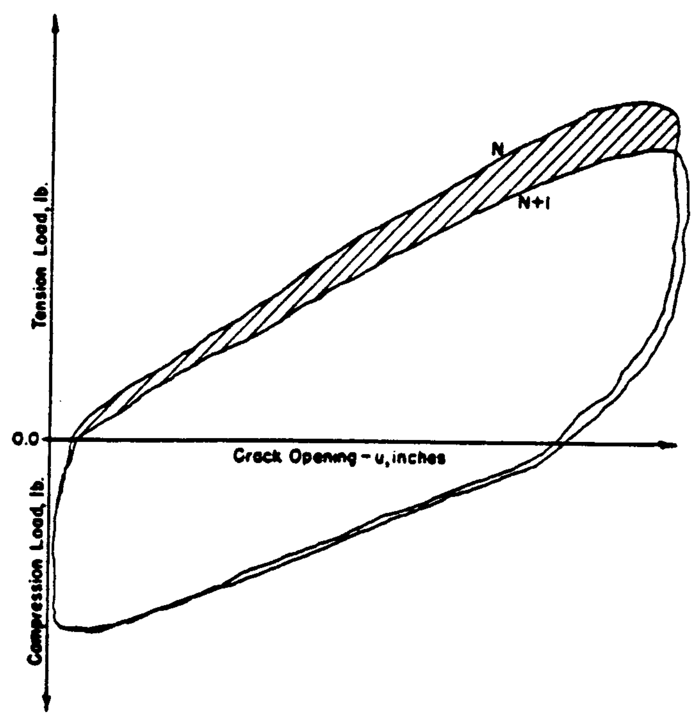

Figure 29. Displacement response in overlay tester recorded on X-Y plotter.

The fabrication procedure for the beam specimens used in this test is identical to that used in beam fatigue testing. This relatively large specimen size allows the use of typical paving mixture aggregate gradations. The overlay tester was calibrated to apply a maximum ram displacement of 0.045 in for specimens tested at 77^{o}F in a manner illustrated schematically in Figure 28. The oscillating horizontal movement was designed to simulate the opening and closing of pavement cracks produced by thermal contraction and expansion of pavement materials.

A loading rate of 6 cycles per minute was used throughout most of the test program. The load and displacement values were monitored and recorded on any X-Y plotter as illustrated in Figure 29. The change in crack length with each loading cycle was visually measured. The area within the load-displacement loops was used to measure the energy required to cause crack propagation and thus to compute the J integral. The J-integral is in essence an energy term which defines the energy per unit area of crack length required to cause the predetermined magnitude of crack tip opening displacement. Once again, a more detailed discription may be found in Reference 1. Note the shaded area in Figure 29 which represents the energy dissipation as the crack grows from cycle N to cycle N + 1.

Method of Evaluation Using Fracture Mechanics. The primary objective of controlled displacement, fracture mechanics based on testing is to evaluate the potential of modified asphalt concrete mixtures to resist fracture due to thermal cycling or other contraction induced displacement.

Figure 30 shows the typical form of a da/dN versus J* regression plot. An upward shift in this line represents a material possessing more brittle behavior and, of course, a more ductile material will plot below the control curve (Figure 31). In the displacement control mode, which was used in this study, the slope of the regression line indicates how sensitive the material is to crack growth. A steep slope is an indication of rapid reduction in crack growth rate, da/dN, as the test continues. This may be due to several effects:

1. A brittle material exhibits a rapid crack growth in the early cycles, leaving a small uncracked ligament behind. In the displacement

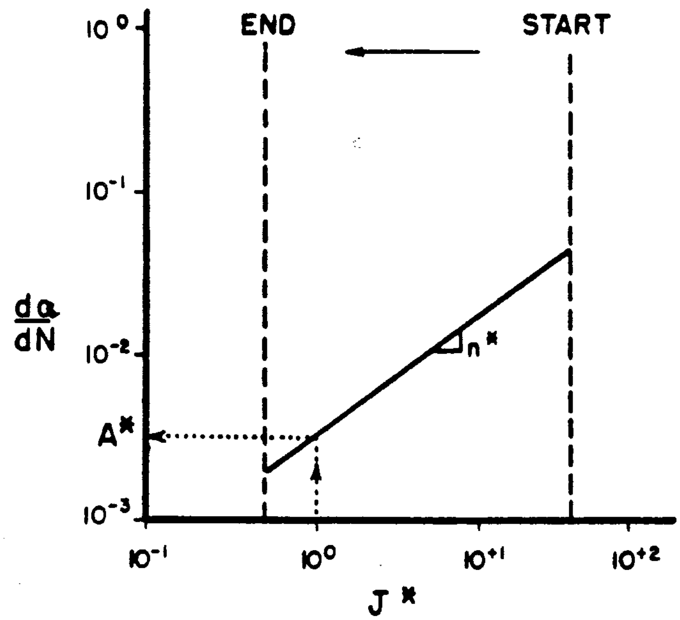

Figure 30. da/dN versus J*, general trend for a controlled displacement test. (After Reference 1)

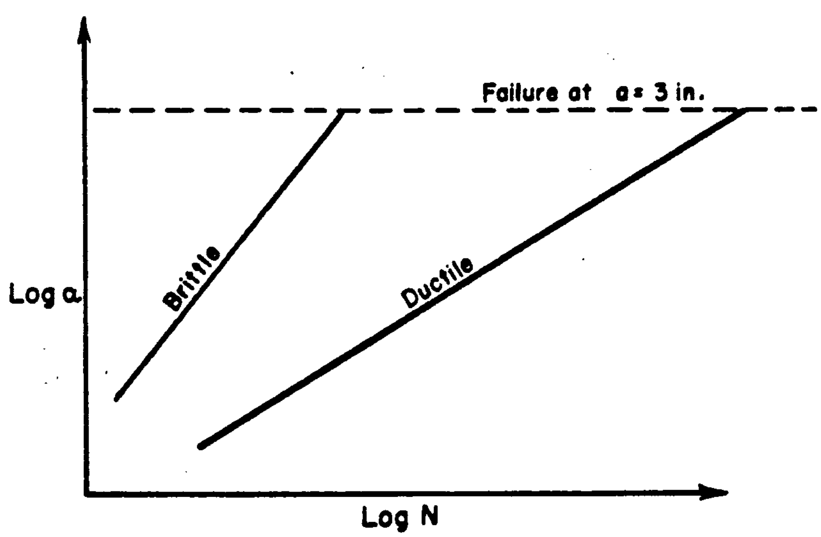

Figure 31. Crack length, a, versus displacement cycle number, N, for brittle and ductile mixtures. (After Reference 1)

control mode, the smaller the size of uncracked ligament, the slower is the crack growth rate.

2. A ductile material may exhibit some crack growth in early cycling due to low stiffness and the presence of voids. However, due to the ductile nature of the material, a significant crack blunting occurs which inhibits the crack growth rate. Generally, ductile materials exhibit relatively small n* values compared to brittle materials which means that the crack growth is insensitive to fatigue and slow throughout the test.

As a result, in the application of J* parameter, the interpretation of the fatigue-fracture behavior cannot be made solely on the basis of either the "intercept," A*, or the "slope," n*, of the Paris equation:

$$\log da/dN = \log A^* + n^* \log J^* \qquad \text{Equation 3}$$

A combined form of parameters A and n, in Paris' law accounts for the effects of both parameters in fatigue-fracture behavior. In this approach, the term (n* + log A*) is defined to be a measure of resistance to crack growth. The term (n* + log A*) is the logarithmic value of crack speed (log da/dN) when the numerical value of J* is equal to 10. This term will be referred to as "crack speed index." This parameter always will be negative; the more negative it is, the more crack resistant the material is. These indexes are recorded at crack lengths of 1-inch and 2-inch in Tables 15 and 16.

Discussion of Results. Based on a review of the results presented in Tables 15 and 16 and Figures 32 through 35, the following trends are noted:

1. At 33^{o}F all additive-soft asphalt blends demonstrated significantly superior crack propagation characteristics compared to the control mixtures which were bound with a harder asphalt without additives. The improvement in the resistance to crack propagation due to the additive-soft asphalt blends was equally dramatic for both asphalts (San Joaquin Valley and Texaco).

Table 15. Summary of controlled displacement fatigue results at 77°F. (After Reference 1)

Base Asphalt	Type	Sample No.	A*	n*	$\mathrm{Log}(\frac{da}{dN})$@ a=1	$\mathrm{Log}(\frac{da}{dN})$@ a=2	N_f	Air Void(%)
Texaco AC	AC-20	3	0.005110	1.116	-1.476	-2.202	200	5.7
		4	0.004816	0.874	-1.827	-2.375	300	6.0
					-1.652	-2.289		
	AC-5 + Carbon Black	3	0.031929	1.174	-0.413	-1.328	50	7.1
		4	0.005523	1.596	-1.504	-2.640	1000	6.3
					-0.959	-1.984		
	AC-5 + Elvax	1	0.528484	0.178	-0.101	-0.214	4	6.5
		3	0.195776	0.502	-0.184	-0.550	7	6.0
		30	0.012785	0.405	-0.610	-0.828	10	5.6
					-0.298	-0.531		
	AC-5 + Kraton	3	0.001414	1.890	-1.236	-2.466	515	5.1
		4	0.010515	1.408	-1.019	-1.971	185	6.4
					-1.128	-2.219		
	AC-5 + Latex	7	0.003667	1.763	-1.844	-2.973	980	6.9
		8	0.005577	1.059	-2.124	-2.668	500	5.4
					-1.984	-2.821		
	AC-5 + Novo-phalt	1	0.024345	0.792	-0.987	-1.554	70	6.0
		2	0.005914	1.088	-1.758	-2.451	300	6.9
					-1.373	-2.003		
San Joaquin Valley AR	AR-4000	1	0.008446	0.789	-1.235	-1.770	100	6.3
		4	0.004751	0.977	-1.184	-1.860	119	5.6
					-1.210	-1.815		
	AR-1000 + Carbon Black	2	0.002628	1.537	-1.858	-2.852	476	6.8
		3	0.002699	1.320	-2.024	-2.819	500	6.2
					-1.941	-2.836		
	AR-1000 + Elvax	2	0.001452	1.727	-2.690	-3.666	>2000	6.2
		3	0.000994	2.118	-2.766	-3.940	>2000	6.7
					-2.728	-3.803		
	AR-1000 + Kraton	2	0.002371	1.709	-2.233	-3.106	1256	6.8
		4	0.000379	2.354	-2.955	-4.285	>2000	6.8
					-2.594	-3.696		
	AR-1000 + Latex	2	0.001493	1.967	-2.268	-3.439	1900	6.8
		6	0.001126	1.738	-2.616	-3.588	>2000	6.4
					-2.442	-3.514		
	AR-1000 + Novo-phalt	1	0.001992	1.826	-1.821	-3.023	764	6.8
		2	0.000855	1.767	-2.390	-3.528	800	6.0
					-2.106	-3.276		

Table 16. Summary of controlled displacement fatigue results at 33°F. (After Reference 1)

Base Asphalt	Type	Sample No.	A*	n*	$\text{Log}(\frac{da}{dN})$@ a=1	$\text{Log}(\frac{da}{dN})$@ a=2	N_f	Air Void(%)
Texaco AC	AC-20	15	0.319806	0.471	-0.474	-0.707	9	5.8
		18	-	-	-	-	2	5.9
					-0.474	-0.707		
	AC-5 + Carbon Black	23	0.025585	0.453	-1.689	-1.894	420	6.8
		32	0.011457	0.501	-2.077	-2.322	766	7.2
					-1.883	-2.108		
	AC-5 + Elvax	12	0.012743	0.649	-1.846	-2.270	295	5.9
		13	0.007318	0.679	-2.294	-2.633	484	5.9
					-2.070	-2.452		
	AC-5 + Kraton	1	0.005222	0.792	-2.423	-2.799	922	5.5
		15	0.005701	0.854	-2.358	-2.802	800	5.7
					-2.391	-2.801		
	AC-5 + Latex	22	0.004590	0.497	-2.499	-2.727	1379	5.9
		23	0.006648	0.679	-2.318	-2.645	1000	6.0
					-2.409	-2.686		
	AC-5 + Novo-phalt	12	0.006027	1.045	-2.121	-2.759	743	6.1
		18	0.003382	0.579	-2.622	-2.896	1952	5.8
		24	0.006325	0.716	-2.296	-2.662	1000	6.0
					-2.346	-2.772		
San Joaquin Valley AR	AR-4000	5	-	-	-	-	-	6.2
		13	-	-	-	-	-	6.3
	AR-1000 + Carbon Black	4	0.086014	0.402	-1.088	-1.293	95	7.3
		11	0.012573	1.755	-0.963	-2.243	250	6.7
		5	0.009827	0.661	-2.032	-2.381	415	7.3
					-1.361	-1.972		
	AR-1000 + Elvax	4	0.009960	1.158	-1.834	-2.474	650	7.0
		9	0.005762	0.496	-2.298	-2.536	822	6.8
					-2.066	-2.505		
	AR-1000 + Kraton	7	0.016701	0.918	-1.738	-2.232	419	6.5
		8	0.013175	0.964	-1.547	-2.198	322	6.1
					-1.643	-2.215		
	AR-1000 + Latex	10	0.025178	1.225	-0.633	-1.639	131	6.3
		11	0.098267	0.625	-0.628	-1.071	50	6.7
					-0.631	-1.355		
	AR-1000 + Novo-phalt	11	0.016646	1.247	-0.627	-1.719	140	6.8
		12	0.017599	2.013	-0.709	-2.091	220	6.6
					-0.668	-1.905		

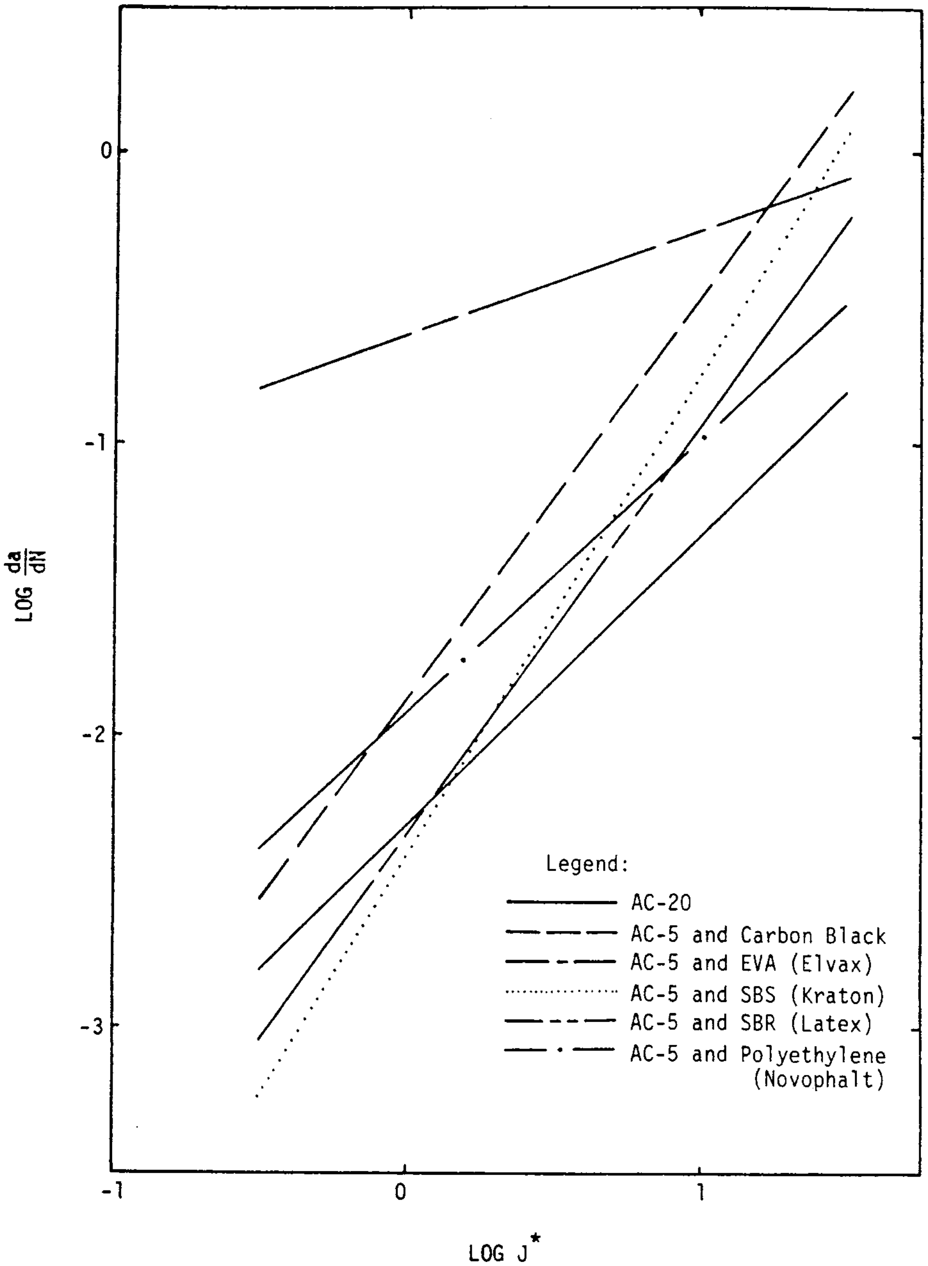

Figure 32. Log-log plot of crack speed versus J-integral at 77°F (25°C) for Texaco asphalts.
(After Reference 1)

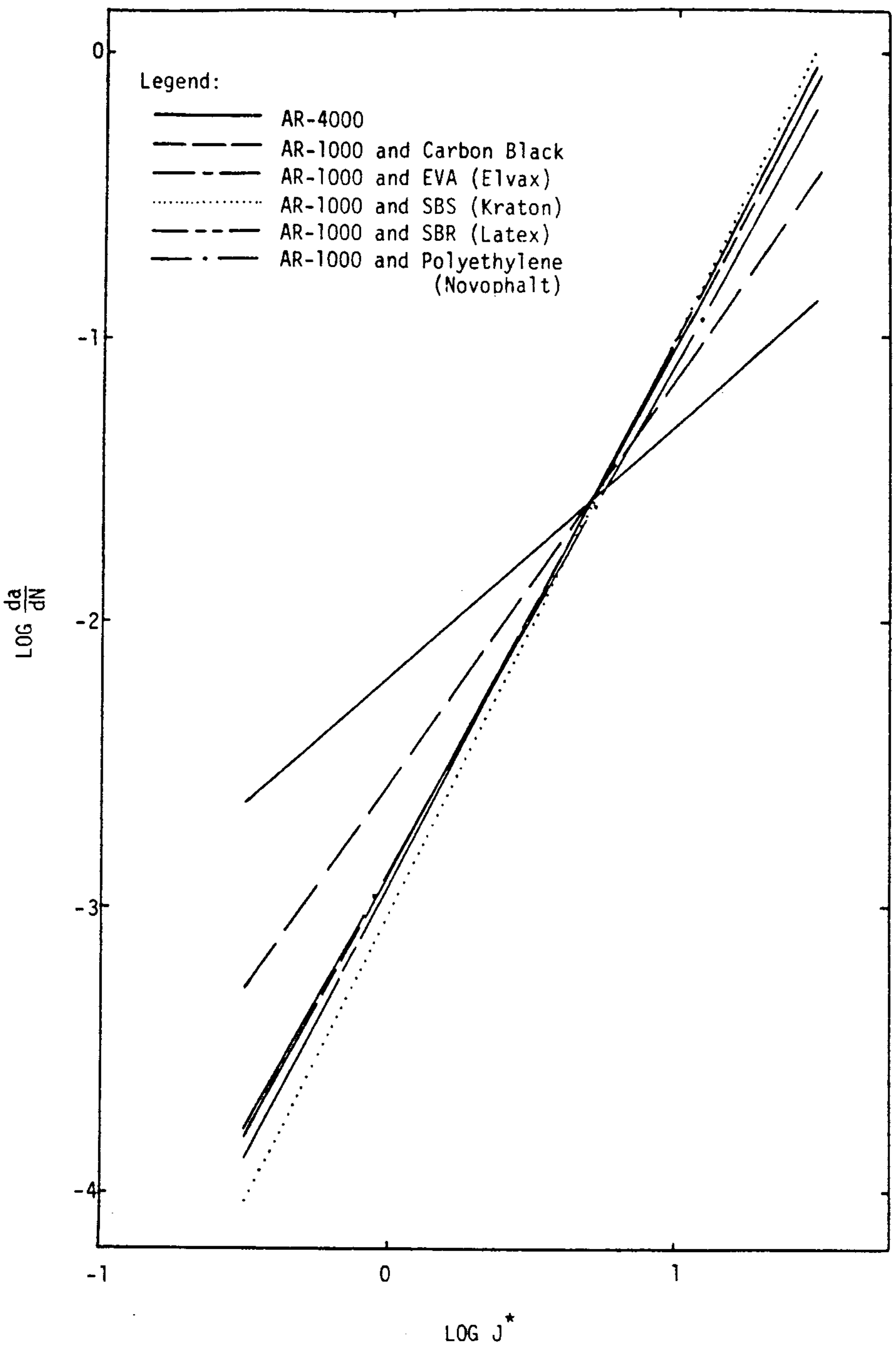

Figure 33. Log-log plot of crack speed versus J-integral at 77°F (25°C) for California Valley asphalts. (After Reference 1)

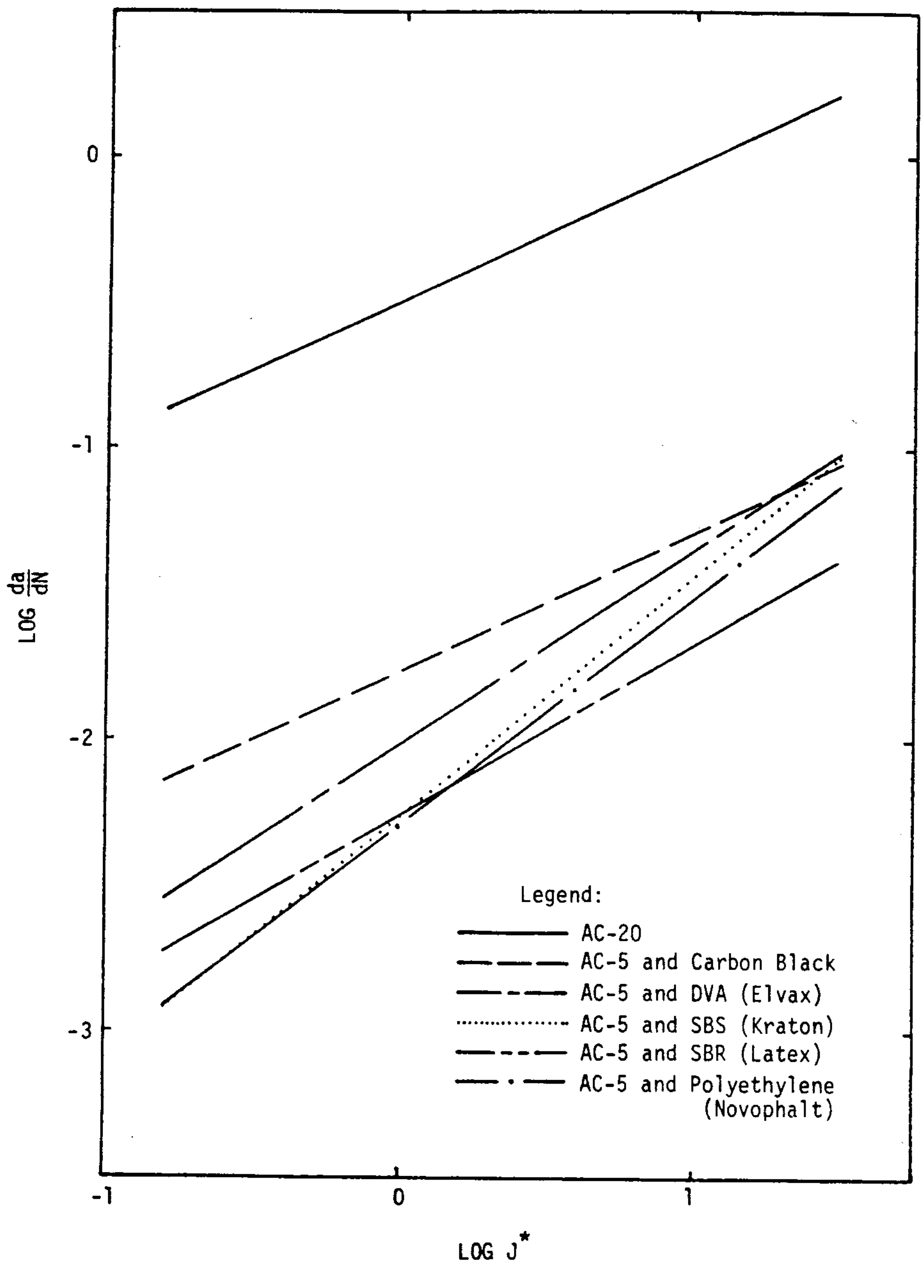

Figure 34. Log-log plot of crack speed versus J-integral at 33°F (1°C) for Texaco asphalts. (After Reference 1)

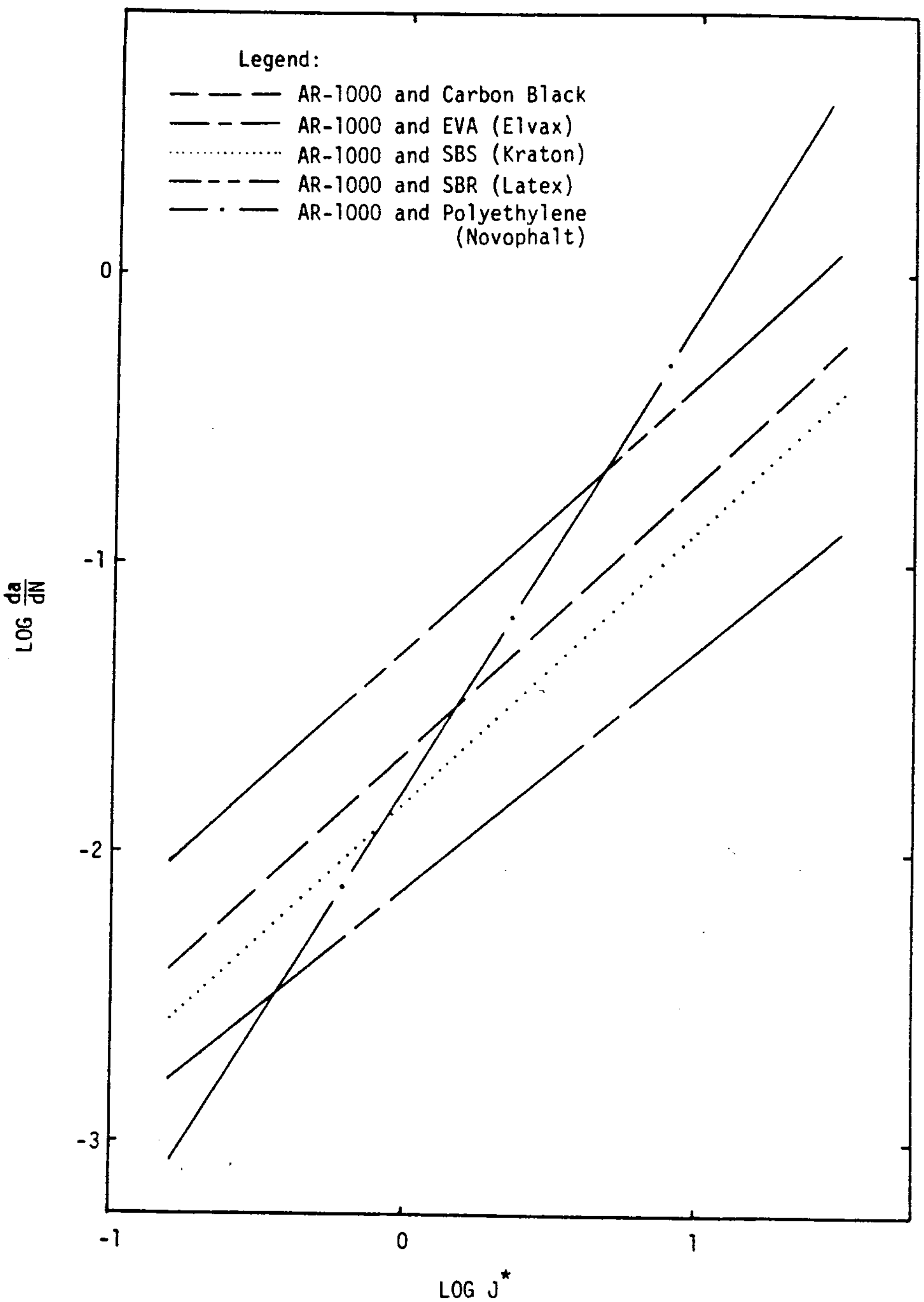

Figure 35. Log-log plot of crack speed versus J-integral at 33°F (1°C) for California Valley asphalts. (After Reference 1)

2. At 33°F, the EVA (Elvax)-AR-1000 blend gave the best results among the blends of additives and San Joaquin Valley asphalts, while the latex (Ultrapave 70)-AC-5 blend gave the best results among blends of additives and Texaco asphalt.

3. Considering the performance of additives from both asphalt sources at 33°F, the SBS (Kraton)-asphalt blends produced the most consistently superior results.

4. At 33°F, the additives blended with Texaco AC-5 demonstrated superior performance when compared to San Joaquin Valley AR-1000 blends. This can be partially explained by the higher penetrations of the AC-5-additive blend at 39.2°F as compared to the AR-1000 blends at 39.2°F. Note also that the Texaco AC-20 asphalt performed slightly better than did the San Joaquin Valley AR-4000 at 33°F (see Tables 15 and 16).

5. At 77°F, the additive blends with the San Joaquin Valley AR-1000 asphalt generally outperformed the blends of additives and the Texaco AC-5 asphalt. Perhaps this is due to the better compatibility between the additives and the San Joaquin Valley asphalt than between the additive and the Texaco asphalt. Furthermore, the base asphalt penetrations are very similar at 77°F so that compatibility may well be the predominant effect. On the otherhand, at 33°F the significant difference in penetration seems to predominate over relative compatibility.

6. The effect of additive-asphalt compatibility at 77°F is most dramatically illustrated by the mixtures composed of blends of EVA (Elvax) and AC-5 and EVA (Elvax) and AR-1000. The controlled displacement samples fabricated with the EVA (Elvax)-AC-5 blend failed in 4, 7 and 10 cycles (Table 15); whereas, samples fabricated with EVA (Elvax)-AR-1000 blends failed in excess of 2000 cycles. Apparently, EVA (Elvax) blended with AC-5 cannot withstand the 0.045-inch crack opening displacement, but the same EVA (Elvax) AC-5 blend performs quite well in controlled-stress fatigue testing at 68°F.

7. At $77^{o}F$, samples fabricated with EVA (Elvax), SBS (Kraton) and latex (Ultrapave 70) blends with AR-1000 demonstrated multiple cracking or "crack branching". This branching of hairline cracks distributes the tensile stresses from the original crack tip and slows the progression of cracks through the sample. As a result, cycles to failure for these samples were often greater than 2000.

8. Mixtures fabricated with carbon black-asphalt blends generally demonstrated the poorest controlled displacement fatigue performance at $77^{o}F$.

CREEP/PERMANENT DEFORMATION TESTING

General

Asphalt concrete mixtures are commonly characterized as viscoelastic materials. The assumption that asphalt concrete behaves in a viscoelastic manner is subject to considerable dispute. However, substantial research in this area has provided credibility to the approach and has developed guidelines to be carefully considered by those evaluating deformation characteristics of asphalt concrete mixtures. This subsection is a summary of test results presented in reference 1.

Experimental Design

Figure 36 represents the experimental design for all direct compression testing on the additive-modified mixtures. Direct compression testing performed in this study was combined with that in the Federal Highway Administration study (1) and reported to both agencies. In doing this, more testing and a much more detailed analyses of the data was made possible than originally proposed to either agency. The silicious river gravel aggregate described in Appendix B was selected as the basic aggregate because it has proven to be much more sensitive to binder properties than the crushed stone aggregates. The Texaco AC-5 was selected as the primary base asphalt for all deformation testing, and the San Joaquin Valley AR-1000 was selected as the secondary asphalt.

Asphalt		Texaco						California Valley					
Temperature, °F	Aggregate \ Additive	Control	Carbon Black	SBR	SBS	EVA	PE	Control	Carbon Black	SBR	SBS	EVA	PE
40	RG	●	●	●	●	●	●						
70	RG	○ ● △	○ ● △	○ ● △	○ ● △	○ ● △	○ ● △	●	●	●	●	●	●
100	RG	●	●	●	●	●	●						

● Normally conditioned specimens.

○ Specimens conditioned by Lottman procedure prior to testing.

△ Specimens aged for 14 days at 140°F prior to testing.

Figure 36. Factorial design of deformation experiments (each symbol represents three replicate samples used to determine an average value for each cell. The tests included in the experiments are: creep compliance, accumulated permanent strain as a function of both load duration and cycles of loading and dynamic moduli). °C=(°F-32)/1.8 (After Reference 1)

This was not a factorial experiment. The purpose of the experiment was not to statistically account for a variety of variables affecting compliance and other deformation responses, but instead, to evaluate the rheological response of the asphalt-additive blends over a temperature range normally encountered by asphalt concrete pavements.

Fabrication of Specimens

A total of seventy-two cylinders 8-inches high and 4-inches in diameter were fabricated using the standard California kneading compactor for the direct compression testing program. Two replicate specimens for each of the cells shown in Figure 36 were fabricated at their respective optimum binder contents as determined eariler (Appendix B). During the fabrication of cylinders for creep compliance testing, temperatures were selected from viscosity-temperature relationships such that the viscosities were 170 centistokes and 280 centistokes, respectively, for mixing and compaction of specimens (Appendix B).

Every effort was made to keep the air voids in the cylinders between six and seven percent. Also, care was taken that the air voids should be distributed equally in the cylinders and that a vertical density gradient would not develop. In order to achieve this, trial cylinders were prepared and then cut into three equal portions and the air void content was determined for each. The compactive effort for the three layers was adjusted based on the results of the previous trial cylinder. The tamping foot pressure was kept constant at 250 psi and only the number of blows was adjusted for the compaction of the three layers. Once a compactive effort was determined for each mixture, it was used for the fabrication of the six cylinders for each binder. The ends of the cylinders were capped using a sulphur capping compound to obtain a smooth and level surface.

The mixing and molding methods used to fabricate the cylinders are outlined in ASTM Methods D 1560 and ASTM D 1561, respectively.

Creep Compliance Testing

All creep tests were performed on a Material Test System 810 closed-loop, feedback control hydraulic tester with a controlled-environment chamber. The creep tests were performed in accordance with the Alternate Procedure II described in the Federal Highway Administration VESYS Users Manual (49). Tests on two specimens each at temperatures of 40^{o}F, 70^{o}F and 100^{o}F were performed. Permanent deformation properties were calculated from the incremental static loading and the creep compliance properties from the 1,000 second response curve for each specimen. A repeated haversine loading was also applied to each specimen in accordance with the VESYS Manual and used to calculate the resilient modulus at the 200th cycle.

The problem of permanent deformation of asphalt layers, which may result in rutting and cause potentially dangerous hydroplaning as well as reduce the service life of the pavement through disintegration of the pavement structure, is a major concern on heavily trafficked asphalt roads. The creep test has been developed into a practical method with which the resistance to permanent deformation of different asphalt mixes can be compared and assessed.

In the creep test, a constant force is applied perpendicularly to the parallel end faces of a cylindrical asphalt specimen. The specimen is placed between two load platens, one of which is fixed and the other, to which the load is applied, is movable in the axial direction as shown in Figures 37 and 38. Deformation of the specimen in the axial direction, occurring under the influence of the load, is measured by linear variable differential transformers (LVDT) as a function of loading time. After removal of the load, the specimen recovers to some extent, which is also measured against time, beginning at the point the load is removed. During the test, the temperature is kept constant.

The results obtained for a particular asphalt mix depend on the chosen test parameters, such as temperature, level of stress applied, preloading conditions, the manner in which load is applied, and the shape and dimensions of the test specimen. It is possible to eliminate the influence of the

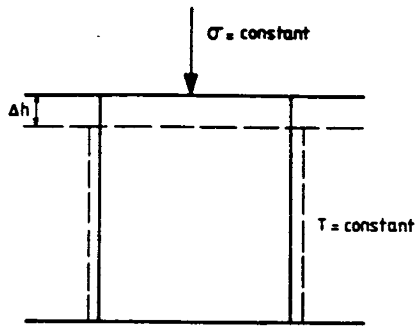

Figure 37. The principle of the creep test procedure.
(After Reference 1)

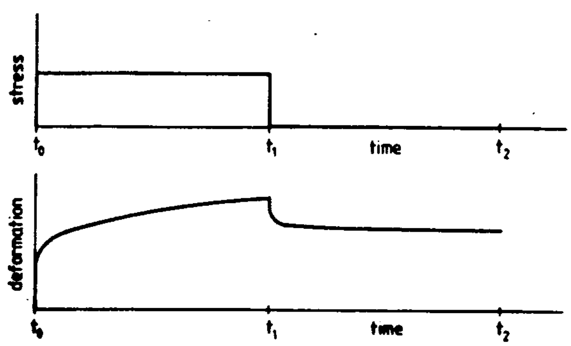

Figure 38. Qualitative diagram of the stress and total deformation during the creep test.
(After Reference 1)

various test parameters by adopting standardized methods. Three standard temperatures of 40°F, 70°F and 100°F were used. A maximum stress level of 20 psi was used, if the deformation under load began to exceed 2500 microunits of strain, the stress level was reduced by 5 psi. If the deformation again began to exceed 2500 micro strain, the stress level was reduced by another 5 psi. For preload conditioning, three ramp loads for ten minutes each were applied, followed by a 10- minute rest period.

For the creep test, five ramp loads were applied for durations of 0.1, 1, 10, 100, and 1000 seconds. Total permanent deformations after two minutes of unload were measured for the 0.1, 1, and 10-second loadings. After the 100-second loading, the specimen was allowed to rest for four minutes then the permanent deformation was measured. The 1000-second load was used to measure the creep compliance as well as the permanent deformation measured after a rest of 8 to 12 minutes.

A repeated haversine loading at the same stress level was then applied for 200 repetitions. Each load application had a load duration of 0.1 second followed by a rest period of 0.9 second. The recoverable strain measured at the 200th cycle was used to calculate the resilient modulus of the specimen. The testing procedure is outlined in detail in the VESYS Manual (49).

VESYS Deformation Parameters

The VESYS structural pavements subsystem uses parameters in the production of permanent deformation. They are called ALPHA and GNU and simply represent mathematical parameters for fitting the relations of permanent strain to cycles of load on a log-log plot.

In developing ALPHA and GNU, researchers (55) decided that it was important to develop a method of representing permanent deformation that was most accurate and sensitive in the region of interest. This region was well past the number of cycles applied during laboratory testing. It was also important to relate the amount of deformation that occurred during a single cycle to the number of previous load cycles so that the permanent strain during any load cycle could be predicted.

The method selected to represent permanent deformation characteristics, ε_a, of material for VESYS IIM involves a linear curve-fit on a log-log plot. This line may be defined by its intercept, I, at one load cycle and its slope, S. Thus,

$$\log \varepsilon_a = \log I + S \log N \qquad \text{Equation 4}$$

or

$$\varepsilon_a = IN^S \qquad \text{Equation 5}$$

The desired permanent strain due to the N^{th} loading is then

$$\varepsilon_p(N) = \varepsilon_a(N) - \varepsilon_a(N-1) \qquad \text{Equation 6}$$

or converting the right side of the equation to a continuous variable

$$\varepsilon_p(N) = ISN^{S-1} \qquad \text{Equation 7}$$

The resilient or elastic strain, ε_r, is essentially a constant after relatively few cycles and is large compared to the permanent strain. Therefore, the fraction of the total strain F(N), that is permanent may be considered to be

$$F(N) = \frac{\varepsilon_p(N)}{\varepsilon_r(N)} = \frac{\varepsilon_p(N)}{\varepsilon_r} = \frac{ISN^{S-1}}{\varepsilon_r} \qquad \text{Equation 8}$$

For convenience, arbitrary definitions were made for mathematical simplification:

$$\mu = IS/\varepsilon_r \text{ (GNU)} \qquad \text{Equation 9}$$

$$\alpha = I - S \text{ (ALPHA)} \qquad \text{Equation 10}$$

As F(N) is the fraction of permanent strain during cycle N, the permanent strain in a compression specimen during cycle N is F(N) multiplied by ε_r. The increment of permanent strain, $\Delta\varepsilon_a$, may also be calculated during the interval of loading, N_1 to N_2, by integrating $\varepsilon_r F(N)$ as follows:

$$\Delta\varepsilon_a = \int_{N_1}^{N_2} \varepsilon_r \; F(N) \; dN = \frac{\varepsilon_r \mu}{1-\alpha} [N_2^{\;1-\alpha} - N_1^{\;1-\alpha}] \qquad \text{Equation 11}$$

The total height (H) reduction of a specimen would be $H \cdot \Delta\varepsilon_a$ during the increments of repetitive load N_1 and N_2.

Both μ and α are considered by VESYS to be constant for a layer of material. In reality, they are quite stress dependent. Thus, μ and α vary with depth in the layer as well as laterally from the center of load.

ALPHA and GNU are difficult parameters to which one may attach physical significance. However, the extensive sensitivity analysis of the VESYS structural subsystem by Rauhut, et al. (55) provided a great step toward understanding the significance of these values. The most important findings in the Rauhut study with respect to this research in terms of ALPHA and GNU are summarized as follows:

1. The ALPHA parameter for asphalt concrete normally occurs within a range of from 0.63 to 0.07.

2. GNU of the surface layer (asphalt concrete) is quite variable and may be as high as 1.5, 2.0 or even higher.

3. ALPHA and GNU are used in VESYS IIM as if they were invariants, but they actually vary with stress, temperature, mix, etc.

4. ALPHA and GNU are very stress-sensitive. Both decrease with increasing deviatoric stress, but at different rates.

5. Temperature should be an important parameter in testing for ALPHA and GNU for the surface layer but it is apparently introduced in VESYS IIM in a different manner. ALPHA and GNU define the fraction of the elastic response to load that will remain when the load is removed. This elastic response is dependent on the stiffness or compliance. For asphalt concrete, a time-temperature shift function revises the master 70°F curve to account for actual temperatures. The assumptions of VESYS IIM are that the effects of

varying layer stiffness with temperature will represent the effects of varying permanent deformation with temperature. It is not known whether or not this is a valid assumption for mixtures containing additives. The proper test temperature for ALPHA and GNU is that used for the master creep-compliance curve (70^{o}F).

6. Both ALPHA and GNU are much more heavily dependent on stress level than upon temperature.

7. A low ALPHA or a high GNU indicate increased rutting and vice versa.

8. Although quite variable, a low ALPHA is usually associated with a low GNU.

9. There is virtually no rutting, slope variance, or deterioration for ALPHA greater than 0.90.

Measuring ALPHA and GNU

ALPHA and GNU are obtained by conducting incremental static-dynamic load tests on 4-inch diameter by 8-inch tall cylindrical specimens. Since these parameters are sensitive to the in situ state of stress and local environments, they should be determined on specimens subjected to realistic in situ stress states and average moisture contents and temperatures expected in the field. The laboratory creep testing specified by the VESYS Manual and used in this study is realistic in that a triaxial stress state is developed during testing. However, various levels of confining pressure are not accounted for nor are variation in moisture conditions except for selected specimens tested following Lottman moisture conditioning.

Straight lines on log-log paper of accumulative strain versus number of load applications were fitted to the data to define the slope, S, and intercept, I. Dynamic, resilient strains were measured at the 200th repetition and were used in the computation of GNU.

Results

Creep Compliance. The 1,000 second response curve was used to calculate the creep compliance, at the loading times of 0.03, 0.1, 0.3, 1.3, 10, 30, 100, 300 and 1,000 seconds. Creep compliance, D(t), is defined as:

$$D(t) = \frac{\text{Total strain observed (function of time)}}{\text{Applied stress}} \qquad \text{Equation 12}$$

Figures 39 and 40 present the results of creep compliance testing for mixtures bound with blends of Texaco AC-5 and additives. These responses are compared to the AC-20 control mixture in each figure. Figure 39 contains data at 40^{o}F and 100^{o}F while Figure 40 contains data at 70^{o}F. Figures 41 and 42 present 70^{o}F compliance data following Lottman conditioning and accelerated aging at 140^{o}F, respectively. The compliance data are tabulated in Appendix E.

From the compliance testing results as depicted in Figures 39 through 42, the following trends were observed:

1. Novophalt exhibited compliance characteristics which were statistically the same as the AC-20 control. In essence, this says that although the resistance of the AC-5 to high temperature deformation is greatly improved by adding Novophalt, the low temperature (40^{o}F) compliance is also reduced giving it essentially the same fracture susceptibility as the AC-20 control.

2. Blends of AC-5 with SBR (Latex), EVA (Elvax), SBS (Kraton) and carbon black all respond with a higher compliance at the low temperature (40^{o}F). The more compliant nature of these blends (compared to the AC-20 control mixture) indicates mixtures which are better suited to relieve stresses induced at lower temperatures and thus better resist low temperature or thermally induced cracking.

3. SBS (Kraton) and carbon black blends with AC-5 respond acceptably at 100^{o}F. Although their compliances at 100^{o}F are significantly higher than those of the control at relatively short load durations (less than 10

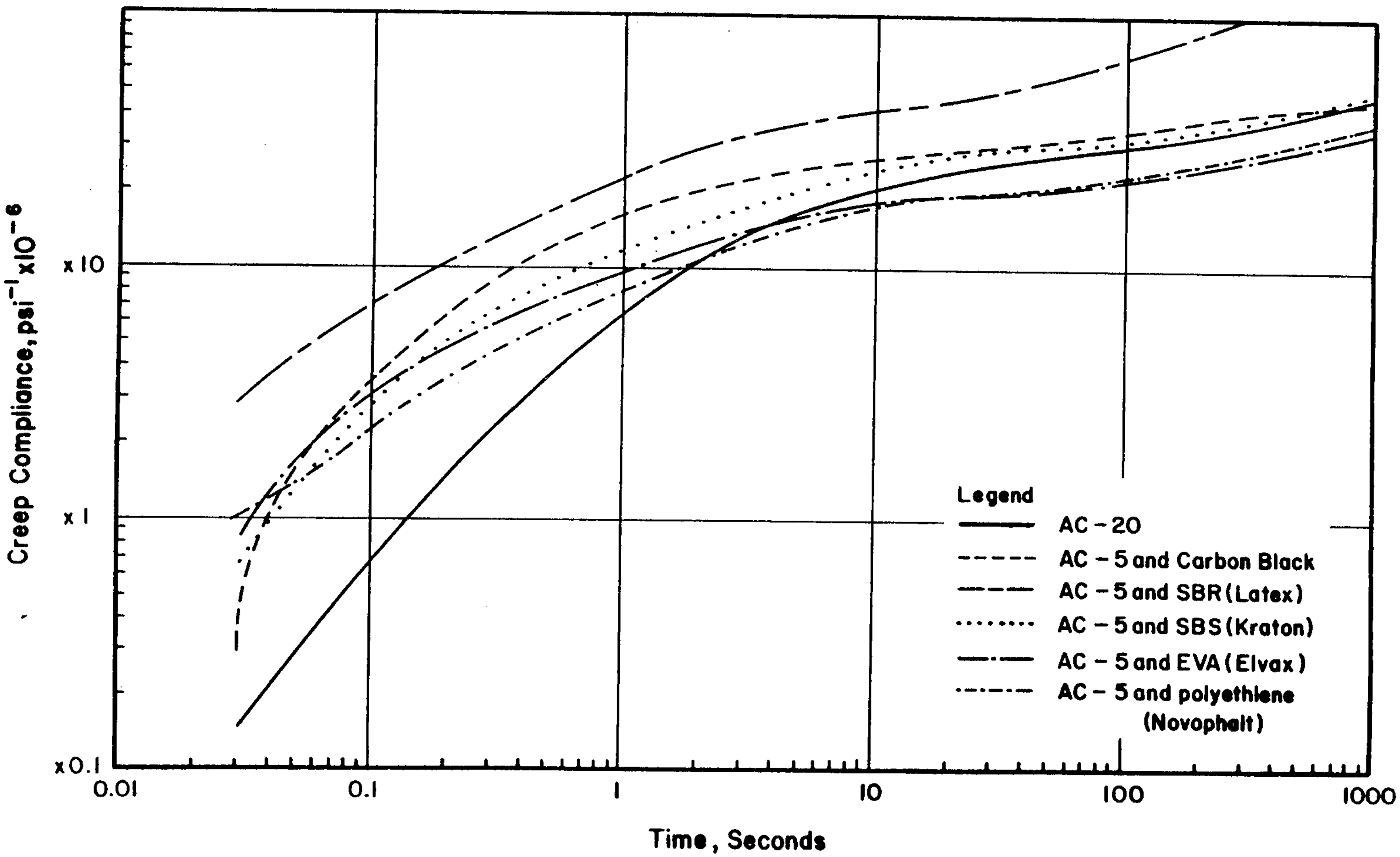

Figure 39. Creep compliance curves at 70°F (21°C) for mixtures containing Texaco asphalts. (After Reference 1)

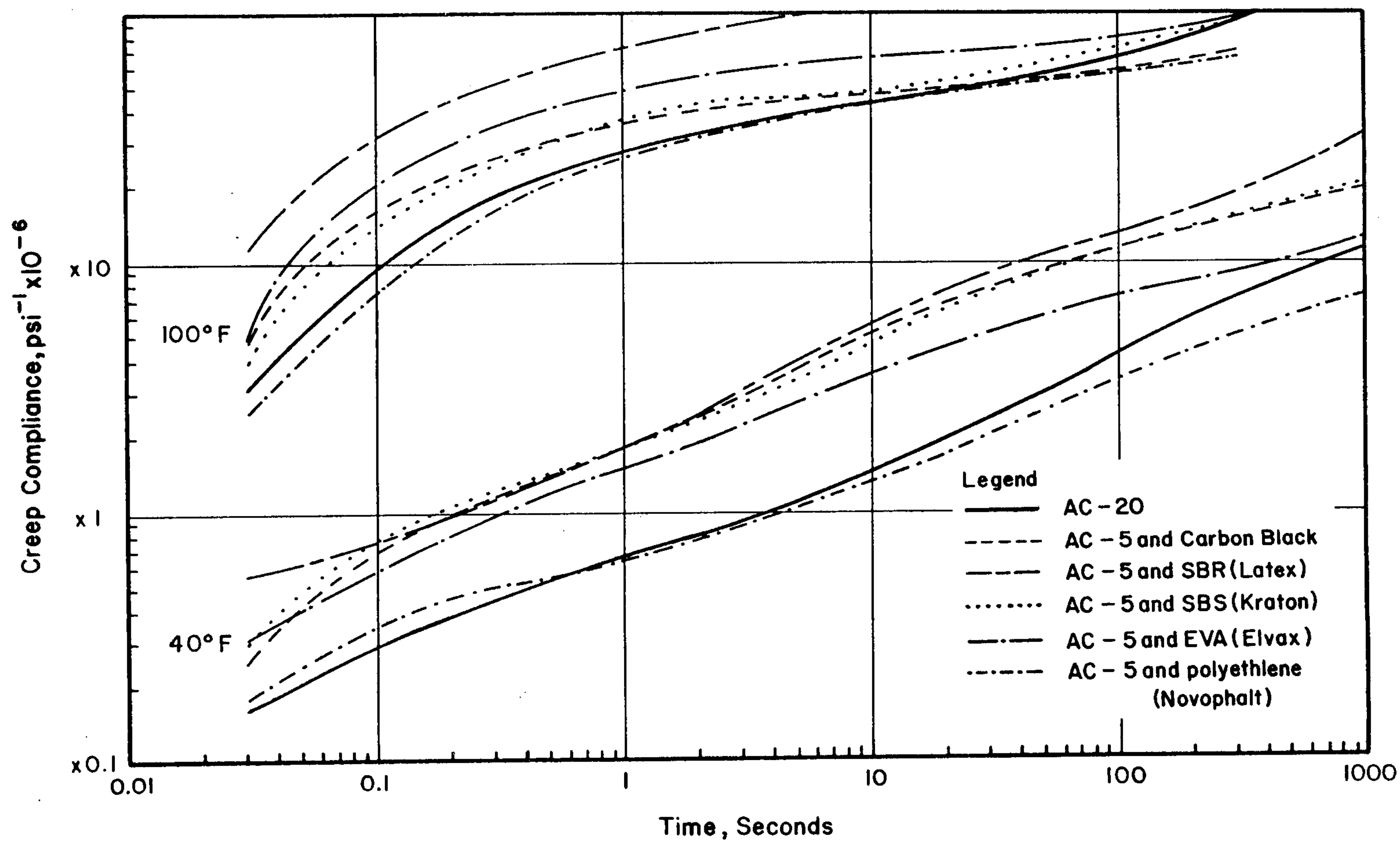

Figure 40. Creep compliance curves at 40°F (4°C) and 100°F (38°C) for mixtures containing Texaco asphalts. (After Reference 1)

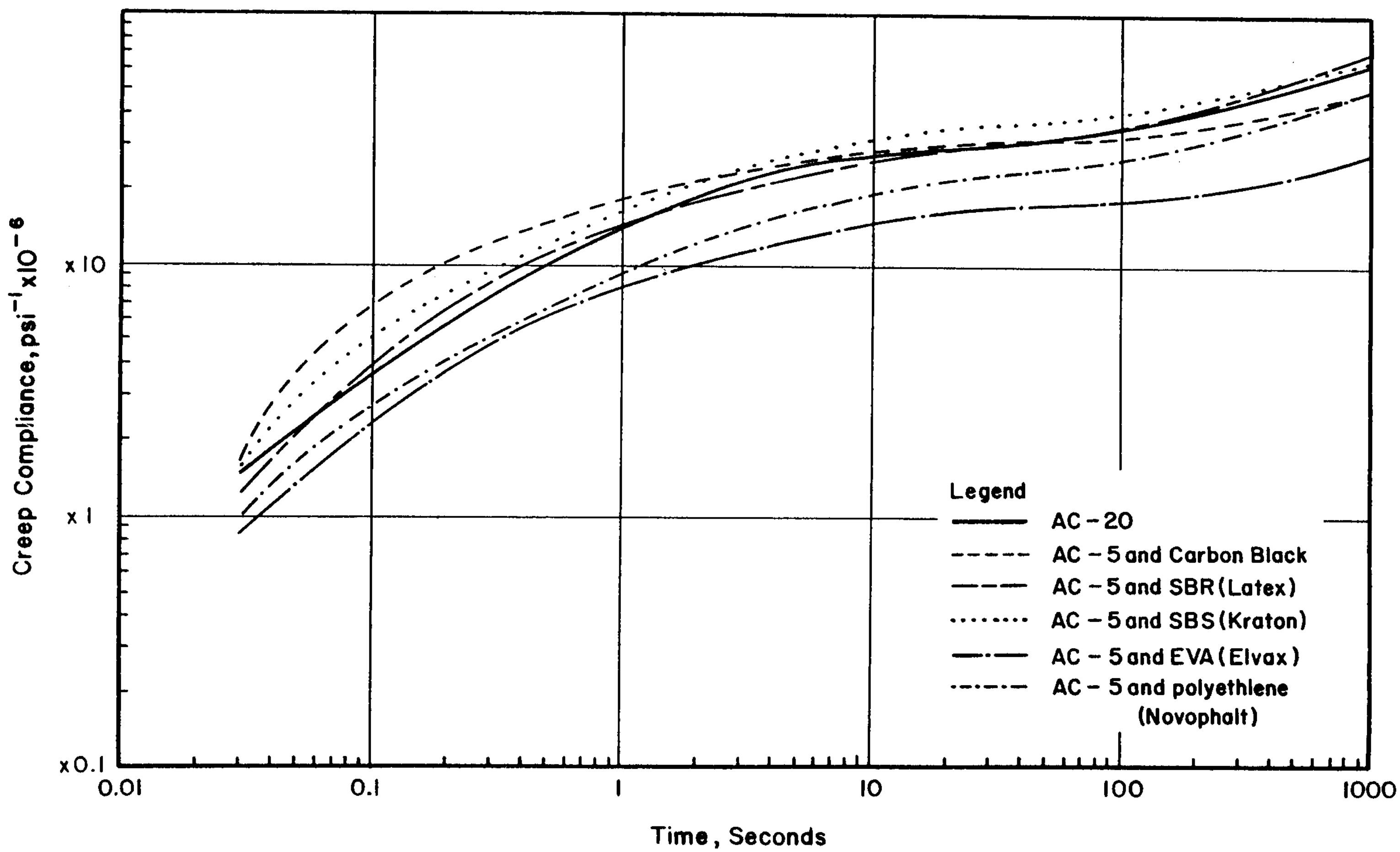

Figure 41. Creep compliance curves at 70°F (21°C) after Lottman moisture conditioning for mixtures containing Texaco asphalt. (After Reference 1)

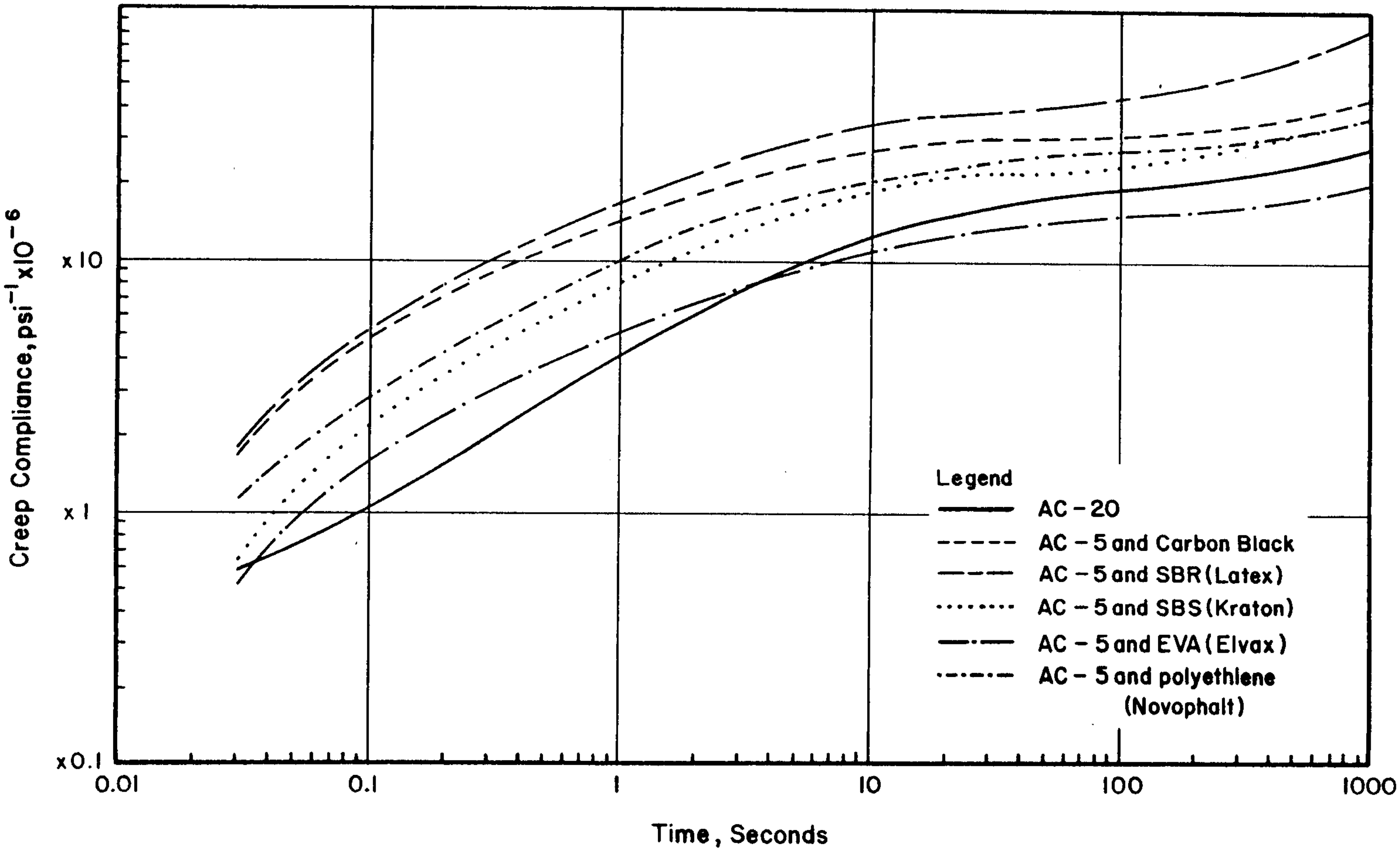

Figure 42. Creep compliance curves at 70°F (21°C) after 7 days at 140°F (60°C) for mixtures containing Texaco asphalt. (After Reference 1)

seconds), the compliances approach those of the control at long load durations, approaching 1000 seconds.

4. The compliances of the AC-5 and SBR (Latex) or EVA (Elvax) at 100^{O}F are significantly higher than those of the control mixture. This is particularly true of the SBR (Latex) blend. From these data, one would expect a reduced potential for load spreading capabilities and excessive permanent deformation at high pavement service temperatures.

5. The effect of Lottman conditioning on the 70^{O}F compliance data was not significant for AC-5 blends with carbon black, polyethylene (Novophalt), SBS (Kraton) and EVA (Elvax). However, the compliances of the AC-20 control mixture were significantly increased (at least at the shorter durations of load) by Lottman conditioning. The compliances of the AC-5 and SBR (Latex) blend showed a significant reduction due to the Lottman conditioning period.

6. A comparison of the 70^{O}F compliance data between normally conditioned specimens and specimens aged for seven days at 140^{O}F reveals no significant difference for any of the mixtures except AC-5 plus EVA (Elvax). The AC-5 and EVA (Elvax) showed a significantly higher compliance before heat aging for load durations of less than 10 seconds. Compliance responses for loading durations of 10 seconds and greater showed no statistical difference.

Based upon Figure 39, it may be stated that, generally, EVA (Elvax), SBS (Kraton), SBR (Latex) and carbon black reduce the temperature susceptibility of AC-5 based on the property of mixture compliance. This occurs because the compliances of all mixtures are significantly higher than for the AC-20 control at 40^{O}F and the compliances of the AC-5 mixtures converge toward those of the AC-20 at the 100^{O}F test temperature. The practical significance of this observation is that such a response is expected of additives which reduce rutting potential at higher temperatures and maintain a compliant (fracture resistant) nature at lower temperatures. The most favorable responses, based on this criterion, occur with the AC-5 blends with carbon black and SBS (Kraton) followed by the EVA (Elvax) blend.

Creep compliance data for mixtures composed of additive blends of the San Joaquin Valley asphalt (AR series) and river gravel at 70^{O}F are shown in

Figure 43. Here the additives were blended with an AR-1000 asphalt and the control mixture was bound with an AR-4000 asphalt. When comparing these compliance data with those of the blends containing Texaco AC-5 asphalt and the AC-20 control, the following observations are made:

1. In general, the compliances are not significantly different between the Texaco and San Joaquin Valley asphalts at loading durations of less then 10 seconds. However, the compliances are significantly higher at load durations above 10 seconds for the San Joaquin Valley Asphalts than for Texaco asphalts.

2. The relative responses of some of the asphalt additive blends are apparently significantly affected by the base asphalts as a result of asphalt-additive compatibility. For example, Novophalt responded essentially the same as the AC-20 control, when the Texaco asphalts were compared. However, the Novophalt AR-1000 blend revealed substantially larger compliances at long loading durations (greater than 10 seconds) than the AR-4000 control mixture. This is a dramatic difference between the two data sets (two asphalt sources). The AR-1000 and SBS (Kraton) blends resulted in the highest compliances. Mixture alterations would probably be necessary in order to bring these compliances into an acceptable range (defined for specific climatic and traffic conditions) in order to reduce the potential for unacceptable deformation.

3. The AR-1000 and carbon black blend produced, as was the case with the AC-5 base asphalt, high compliances at the shorter load durations. However, compliances at the longer load durations were lower than the control. This is the response hoped for when reducing time-temperature susceptibility through the addition of modifiers.

4. The AR-1000 and EVA (Elvax) blend responded very nearly like the AR-4000 control over the loading duration range.

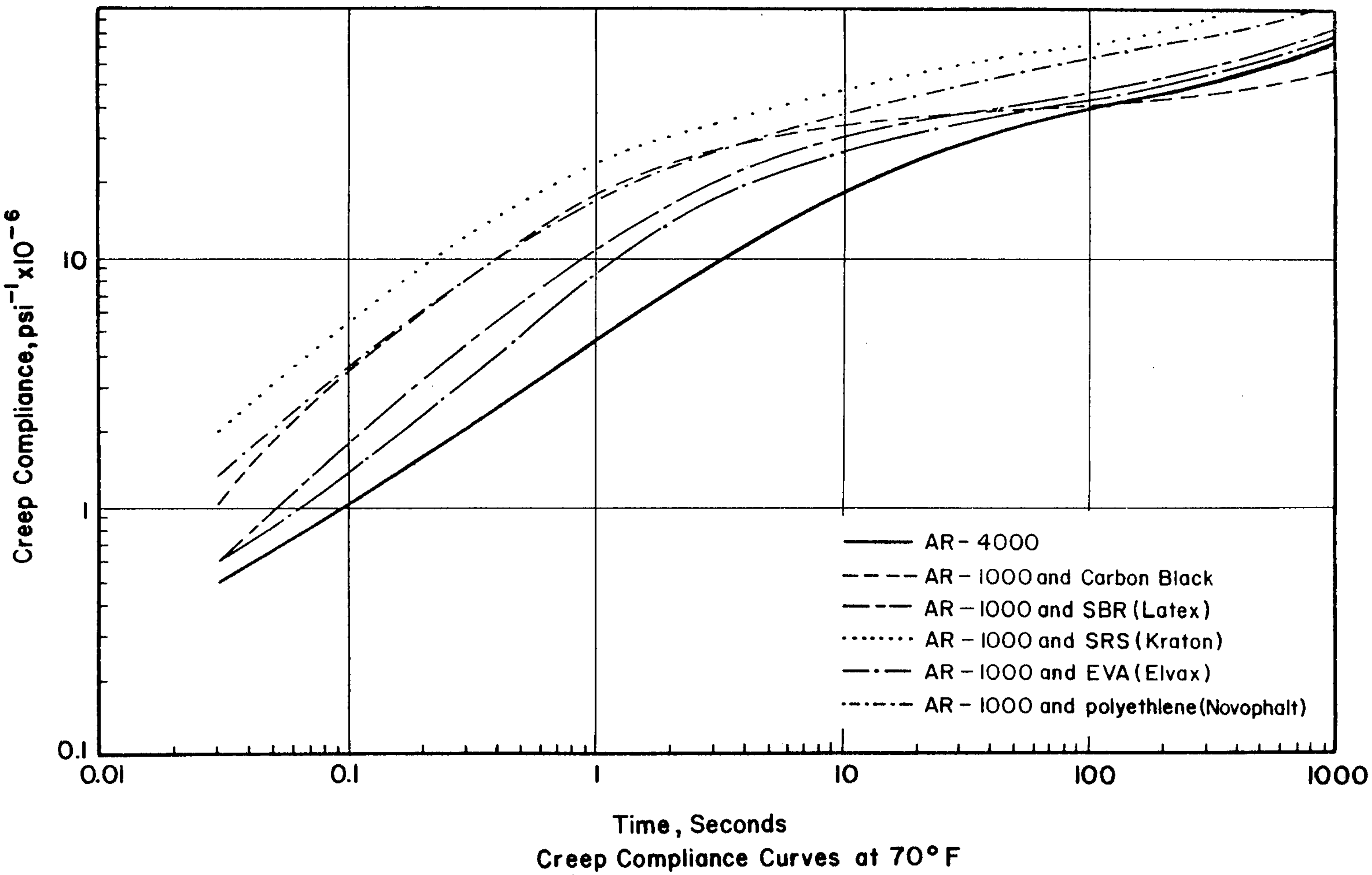

Figure 43. Creep compliance data at 70°F (21°C) for mixtures using SanJoaquin Valley asphalts. (After Reference 1)

In general, with AR-1000 as the base asphalt, the addition of carbon black most effectively produced the favored response of high compliances at short loading times and lower compliances at longer loading times. All other additives were successful to some degree in maintaining the lower compliances at short load durations (attributable to the soft AC-5 asphalt) and producing stiffer, less compliant mixes at the longer load durations. This indicates lower time-of-loading temperature susceptibility.

Permanent Deformation. The total permanent strain at the end of each rest period was plotted on log-log paper as a function of the incremental times of loading: 0.1, 1, 10, 100, and 1000 second. The permanent deformation plots from the incremental static loading tests (performed in accordance with the procedures established in Reference 55) are shown in Figures 44 through 50.

An analysis of the plots of accumulated strain versus incremental loading time from Figures 44 through 50 reveals the following:

1. Mixtures containing AC-5 and SBR (latex) exhibited large deformations during pre-loading (exceeding 2500 micro-strain units) at 70^{o}F and at 100^{o}F, and, in accordance with the VESYS Manual, the level of applied stress was reduced in these cases. Even at the lower level of applied stress, the latex specimens showed the greatest permanent deformation relative to the AC-20 control and the other additives tested.

2. Also at 40^{o}F, the mixture containing the latex blend exhibited significantly higher deformation than the other five mixtures. Perhaps a reduction in binder content (AC-5 and latex blend) within the mixture or an increase in the amount of latex used in the blend (AC-5 plus latex) is warranted to improve the creep and deformation responses.

3. Polyethylene (Novophalt) exhibited a greater resistance to permanent deformation at 40^{o}F and at 70^{o}F than any other mixtures, including the AC-20 control. At 100^{o}F, the carbon black blend yielded the least permanent deformation followed closely by Novophalt. However, the slope of the permanent deformation versus time of loading plot for the Novophalt mixture was statistically smaller than slopes for the other mixtures. This

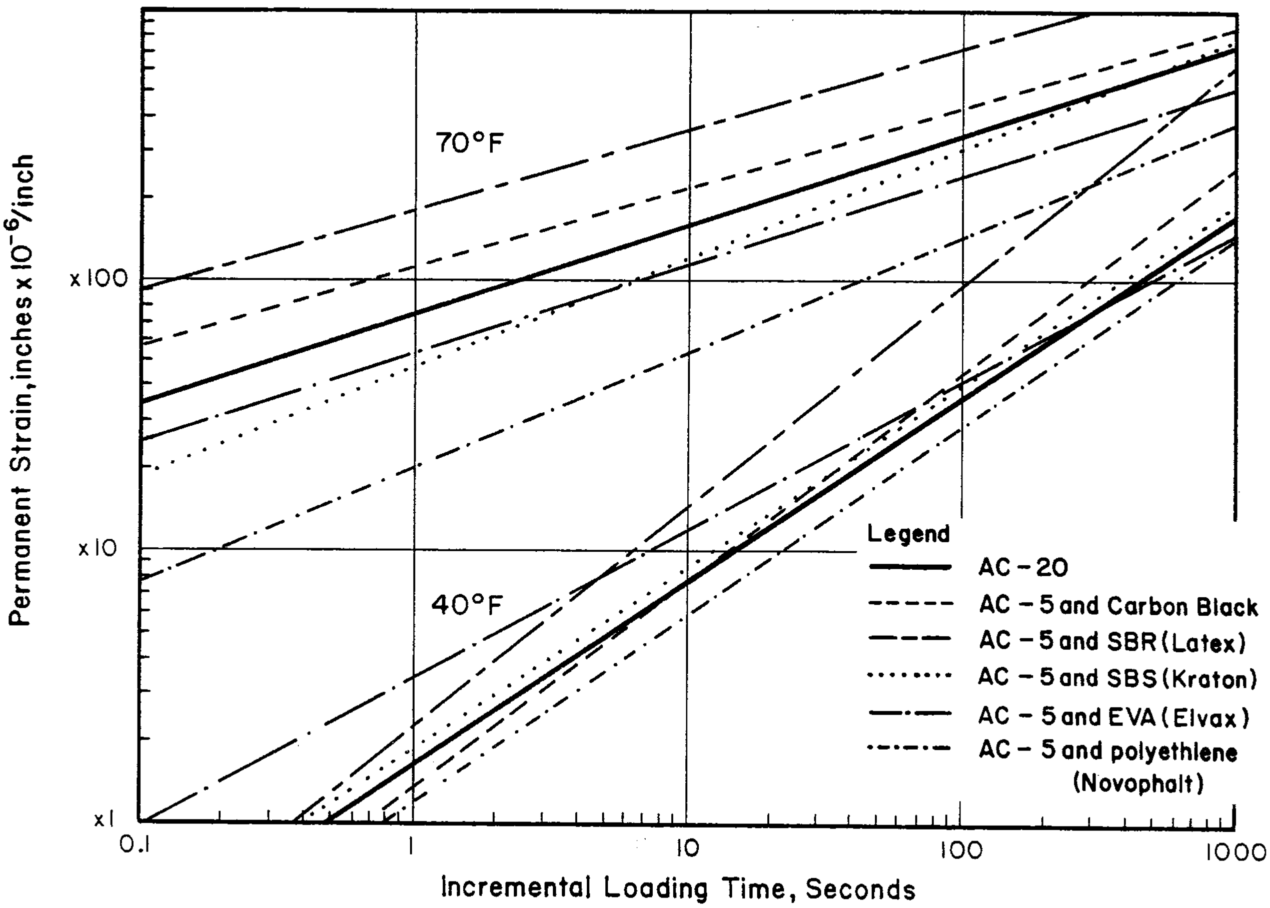

Figure 44. Permanent strain from incremental static loading tests at 40°F (4°C) and 70°F (21°C) for mixtures using Texaco Asphalts. (After Reference 1)

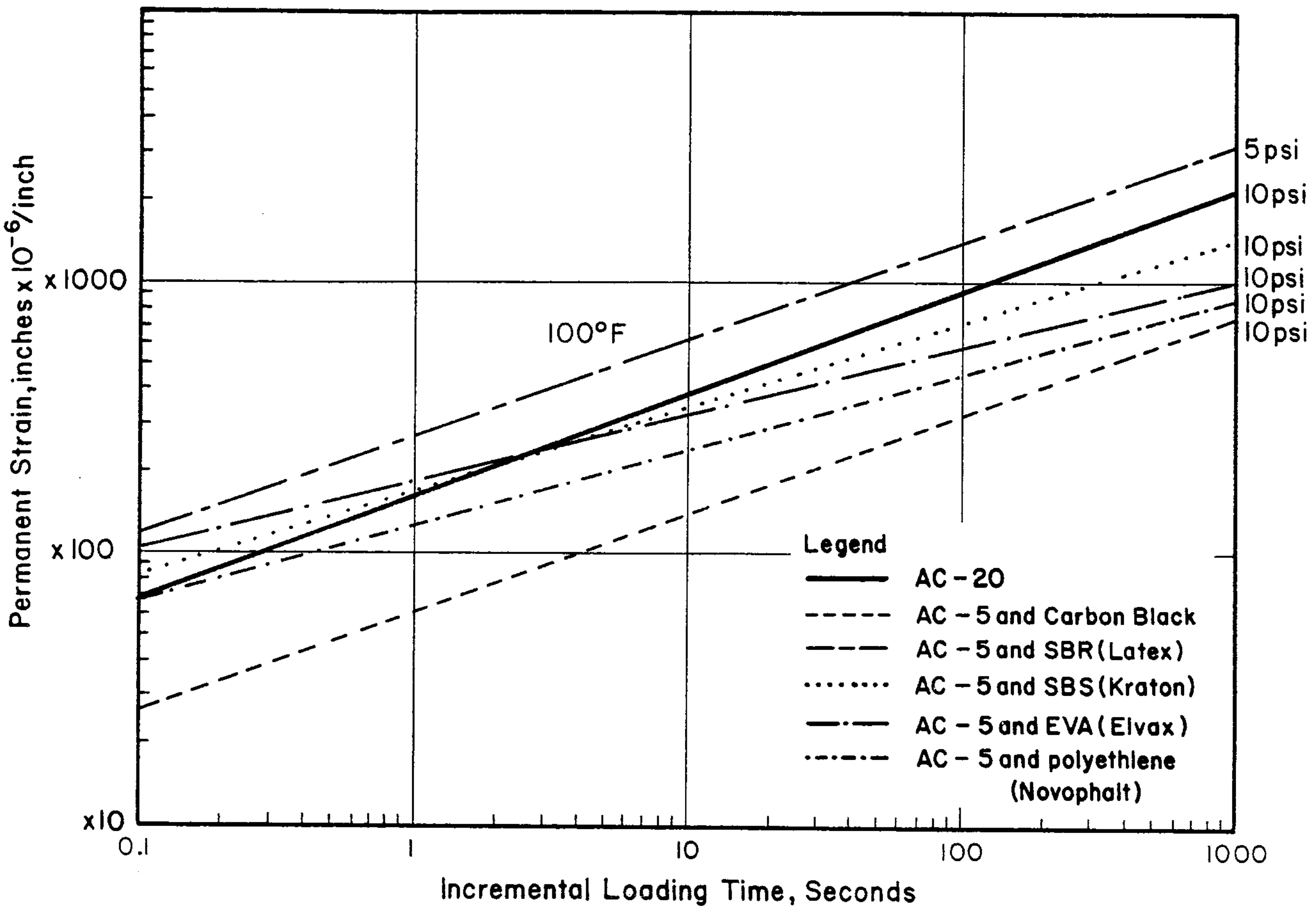

Figure 45. Permanent strain from incremental static loading tests at 100°F (38°C) for mixtures using Texaco asphalts. (After Reference 1)

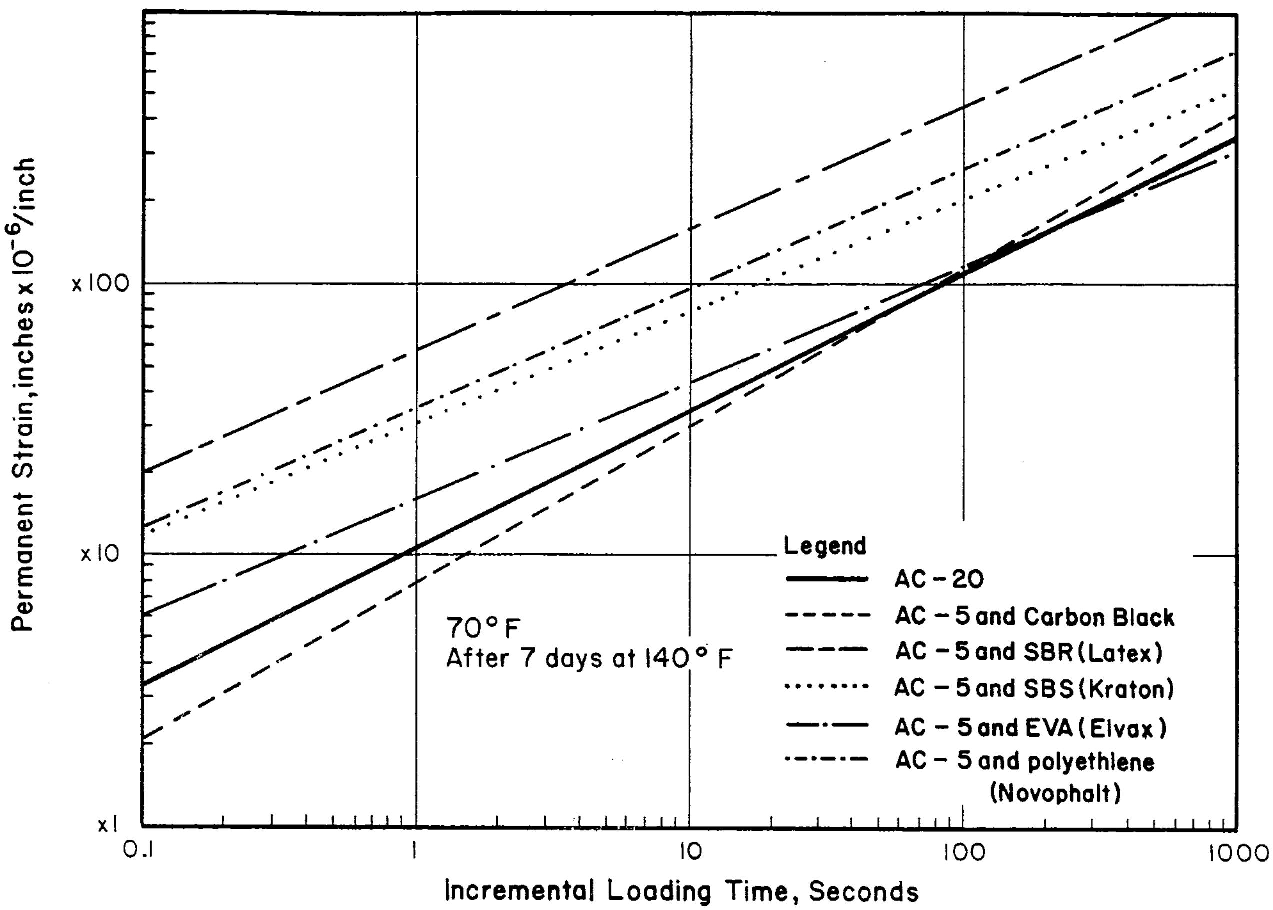

Figure 46. Permanent strain from incremental static loading tests at 70°F (21°C) after 7 days at 140°F (60°C) (all tests at 20 psi) for mixtures using Texaco asphalts. (After Reference 1)

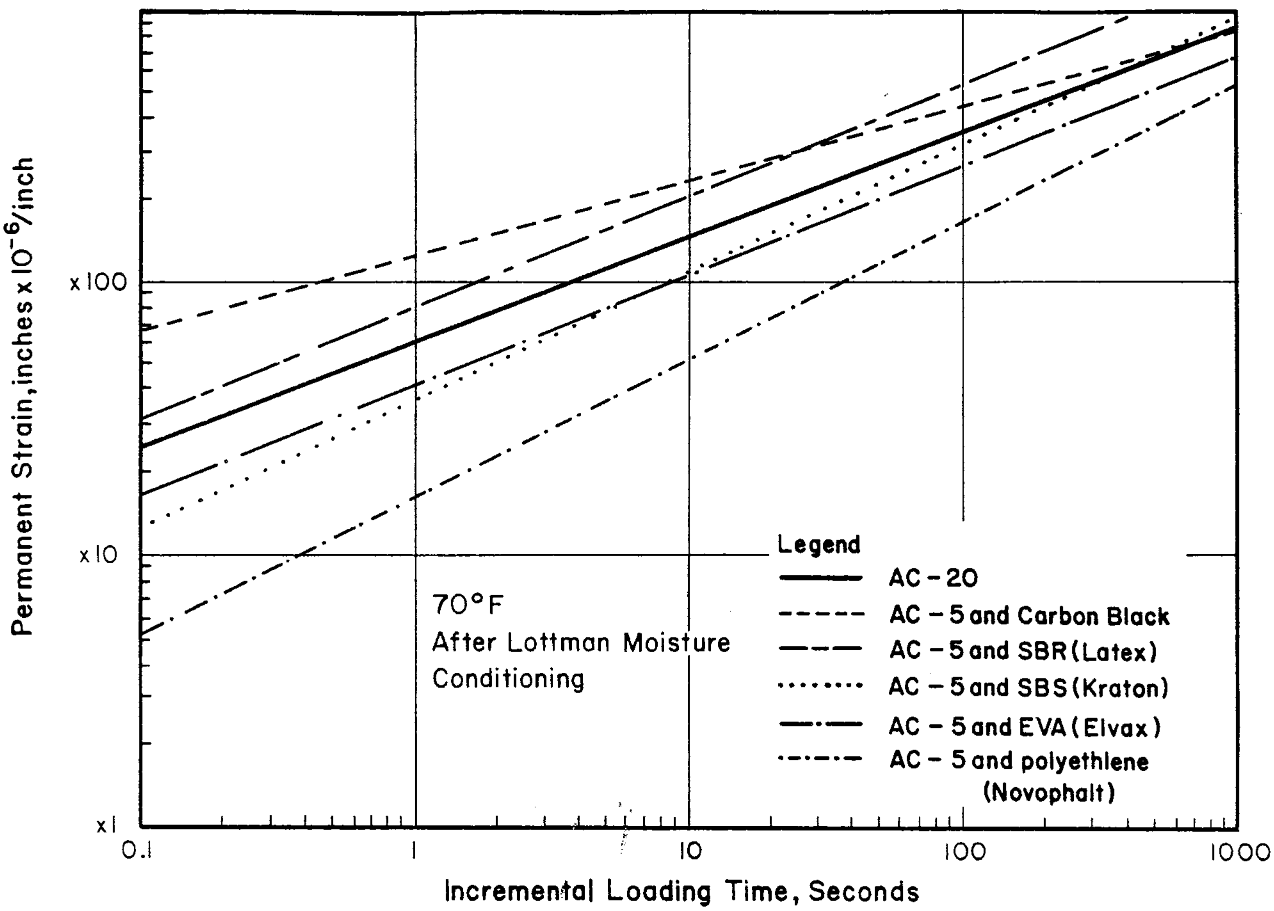

Figure 47. Permanent strain from incremental static loading tests at 70°F (21°C) after one-cycle Lottman for mixtures using Texaco asphalts. (After Reference 1)

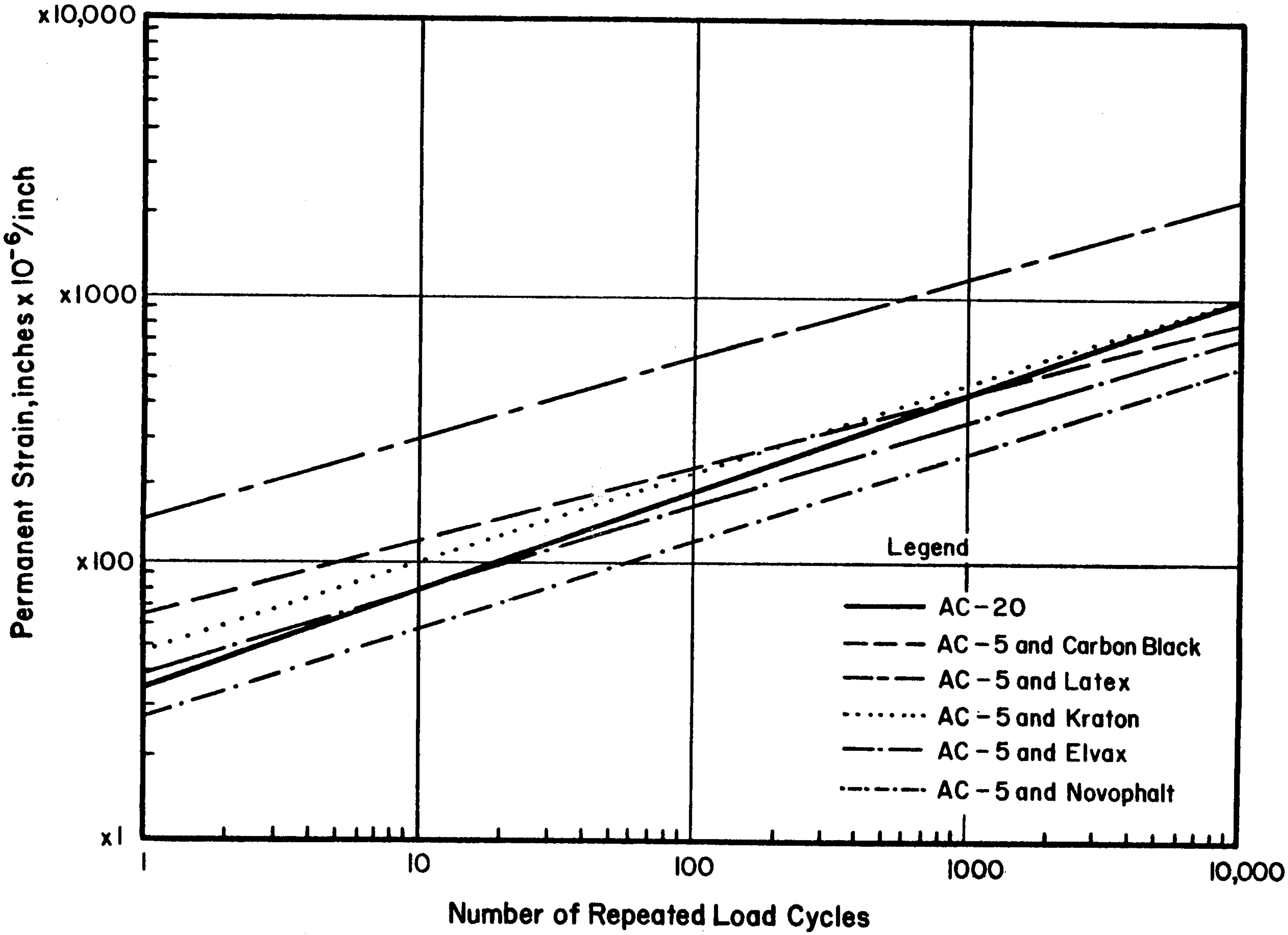

Figure 48. Accumulated strain versus cycles at 100°F (38°C) for additive modified Texaco AC-5. (After Reference 1)

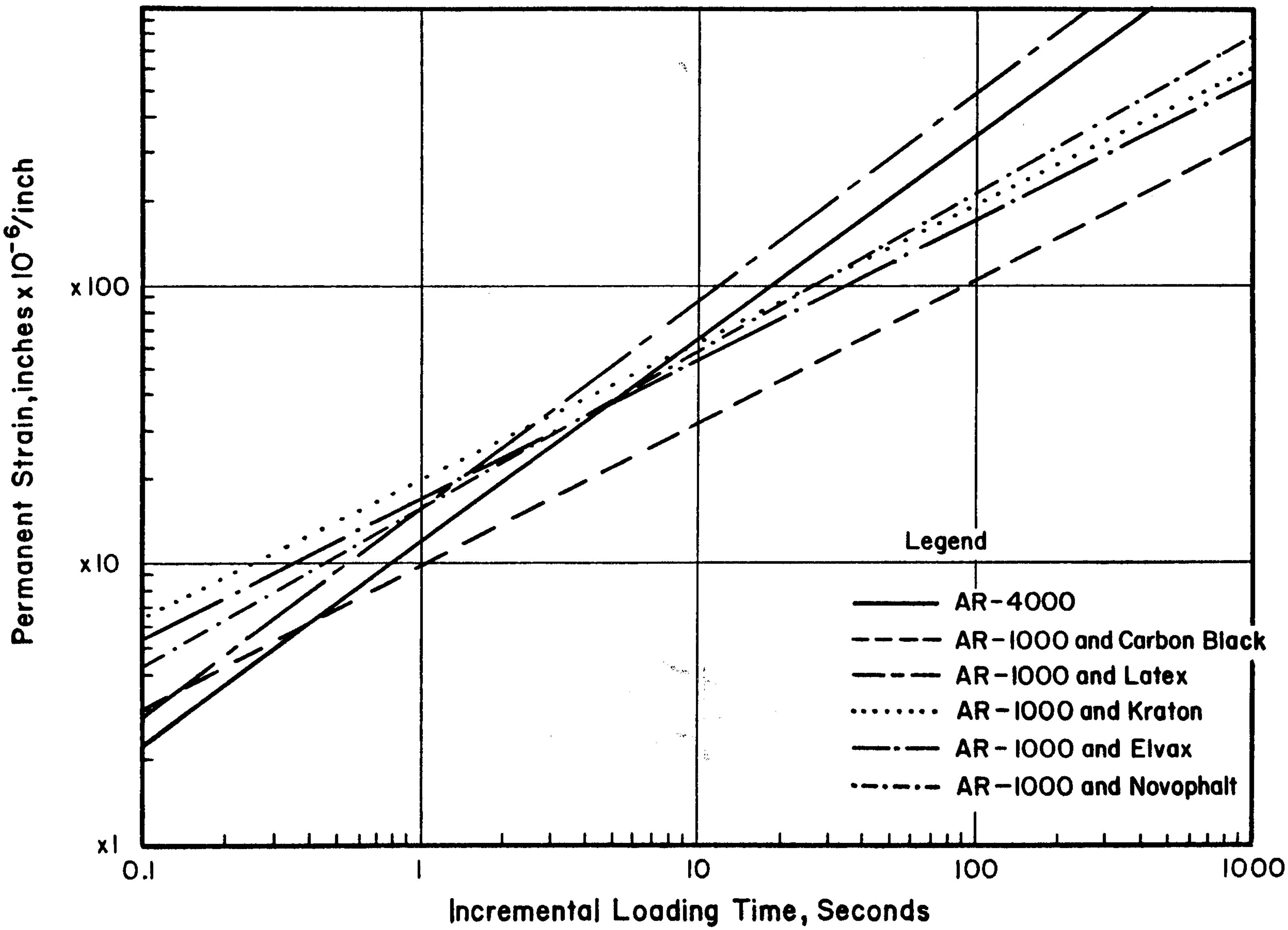

Figure 49. Accumulated strain versus incremental loading time at 70°F (21°C) for additive modified SanJoaquin Valley AR-1000. (After Reference 1)

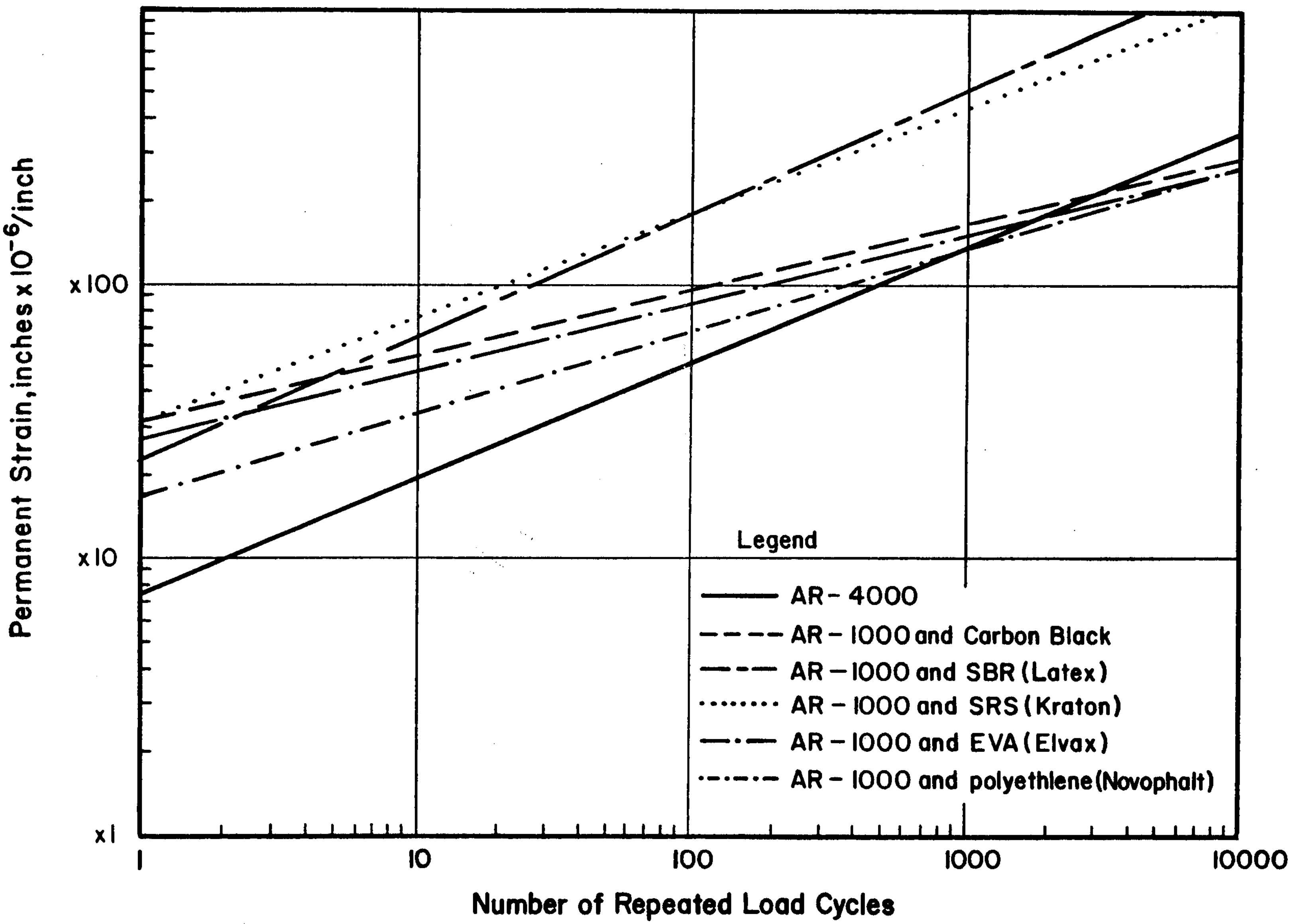

Figure 50. Accumulated strain versus loading cycles at 70°F (21°C) for additive modified San Joaquin Valley AR-1000. (After Reference 1)

fact coupled with the relative position of the plot, with respect to the other mixtures, indicates a greater resistance overall for Novophalt in resisting permanent deformation.

4. It is surmised that the relative positions of the permanent strain versus incremental loading time plots are influenced greatly by the preconditioning procedure. This procedure may not adequately account for material property peculiarities of polymer-modified asphalts. This hypothesis will require further study for evaluation.

5. Mixtures containing EVA (Elvax) and SBS (Kraton) showed permanent deformation responses similar to the AC-20 control.

The effects of heat aging may be evaluated by comparing the results of Figures 44 and 46. An analysis of these results yield the following observations:

1. The susceptibility of mixtures to permanent deformation appeared to be significantly affected based on the different intercept values between Figures 44 and 46. However, the values of permanent strain at incremental, static loading times of 1000 seconds were not significantly different for mixtures containing blends of AC-5 and SBR (latex) or AC-5 and SBS (Kraton). The mixtures containing AC-5 and carbon black, AC-5 and EVA and the control mixture (AC-20) exhibited statistically significant, though not substantial, reductions in accumulated strain at a loading time of 1000 seconds due to heat aging. The mixture containing carbon black and polyethylene actually demonstrated slightly more susceptibility to deformation following accelerated aging.

2. The visual differences between deformation plots before and after aging were a significantly smaller intercept and a significantly steeper slope for the aged samples. The net result was approximately the same accumulated strain at long loading times. Perhaps this was due to an initial set caused by 140^{o}F aging which was overcome during long-term creep.

The effects on one-cycle Lottman conditioning are demonstrated by comparing Figures 44 and 47. Once again, the relative deformation susceptibilites were not markedly altered. In fact, the values of accumulated strain at an incremental static loading time of 1000 seconds were not significantly different between tests for latex, carbon black, the

AC-20 control, EVA or SBS. The AC-5 and polyethylene blend showed larger permanent strains (statistically significant though not practically significant) at the incremental loading time of 1000 seconds.

Figure 48 demonstrates the accumulated strain versus repeated load applications for the six mixtures in which the San Joaquin Valley asphalts were used. These data are recorded at 100^{o}F and can be compared to the data in Figure 45. Although the results are somewhat different from those in Figure 45, the relative behavior of the additives are similar. At 10,000 loading cycles, the order of resistance to permanent deformation is as follows: (1) Polyethylene (Novophalt), (2) EVA (Elvax), (3) Carbon black, (4) SBS (Kraton), (5) AC-20 and (6) SBR (latex). The mixtures were so weak at 100^{o}F that a loading stress of only 5 psi could be used during the test.

Figures 49 and 50 depict the incremental static and repeated load permanent deformation results, respectively, for the AR-1000, San Joaquin Valley asphalt. These tests were performed at 70^{o}F. Based on these results, the following observations are presented:

1. Polyethylene (Novophalt), EVA (Elvax) and carbon black were successful in limiting long-term permanent deformation of the AR-1000 base asphalt to ranges equal to or less than those developed when the AR-4000 binder is used in the mixture.

2. Although the mixture containing SBS (Kraton) responded with high deformation based on the repeated load testing, the responses were similar to other mixtures based on incremental time of loading results. This discrepency may be due in part to the inadequacy of the incremental static load test to account for rebound time.

3. In general, although some anomalies exist, the results of incremental static load induced permanent deformation and repeated load induced permanent deformation are consistent. However, the results should be evaluated by considering not only the relative position of the plot but also the slope. Slopes of the mixtures containing polyethylene, EVA and carbon black are significantly lower than the other mixtures.

4. Where the differences in deformation responses are affected by the asphalt source, the answer may at least partially lie in asphalt-polymer compatibility. However, the SBR and SBS rubber-modified mixtures generally

responded with the least resistance to permanent deformation while EVA demonstrated a substantial resistance.

In conclusion, although a sensitivity of the additives to asphalt source is strongly inferred from the results of permanent deformation testing, the relative effects of the additives in reducing long-term permanent deformation at the higher temperatures are consistent (Table 17). In general, the polyethylene, carbon black and EVA additives were most successful in preventing permanent deformation.

Evaluation of Binder Volume Effects on Compliance and Deformation. Although the mixture design called for a five percent binder content for the latex-AC-5 blends with the river gravel aggregate, the design was re-evaluated for two reasons: first, all other polymer additive mixture designs called for lower binder contents (4.5 to 4.6 percent) and second, the mixtures modified by latex responded as quite susceptible to permanent deformation and exhibited relatively large compliances at high temperature and/or long loading durations. Thus, the properties of mixtures containing latex were re-evaluated using 4.5 percent binder.

Figures 51 and 52 compare the compliance and incremental static load deformation data at 70^{o}F, for the 4.5 and 5.0 percent blends, respectively. From these data one should observe:

1. Reduction of 0.5 percent has a statistically significant effect on compliance as well as accumulated permanent deformation.

2. In terms of deformation susceptibility, the mixture containing 4.5 percent binder improved dramatically but, relative to other mixtures, still ranked most susceptible to deformation.

3. All mixtures containing AR-1000 and latex blends and tested in permanent deformation were prepared at 4.5 percent binder. In these mixtures, the latex modified asphalt also showed the greatest potential to deform at both 70^{o}F and 100^{o}F. However, the response was not significantly different from the mixtures containing Kraton (SBS).

4. The sensitivity of permanent deformation response and creep compliance to mixture properties such as binder content is illustrated by

Table 17. Relative resistance to permanent deformation at long term load durations.

Binder	When additive is mixed with: AC-5 (Texaco)	AR-1000 (San Joaquin Valley)
Control	5 (5)*	5 (4)*
Carbon Black (Microfil-8)	2 (3)	1 (3)
EVA (Elvax)	3 (2)	2 (2)
SBR (Latex)	6 (6)	6 (6)
SBS (Kraton)	4 (4)	3 (5)
Polyethylene (Novophalt)	1 (1)	4 (1)

* Results in parentheses are from repeated load versus accumulated strain testing at 70°F (21°C) for Texaco asphalts and 100°F (38°C) for Valley Asphalt. (After Reference 1)

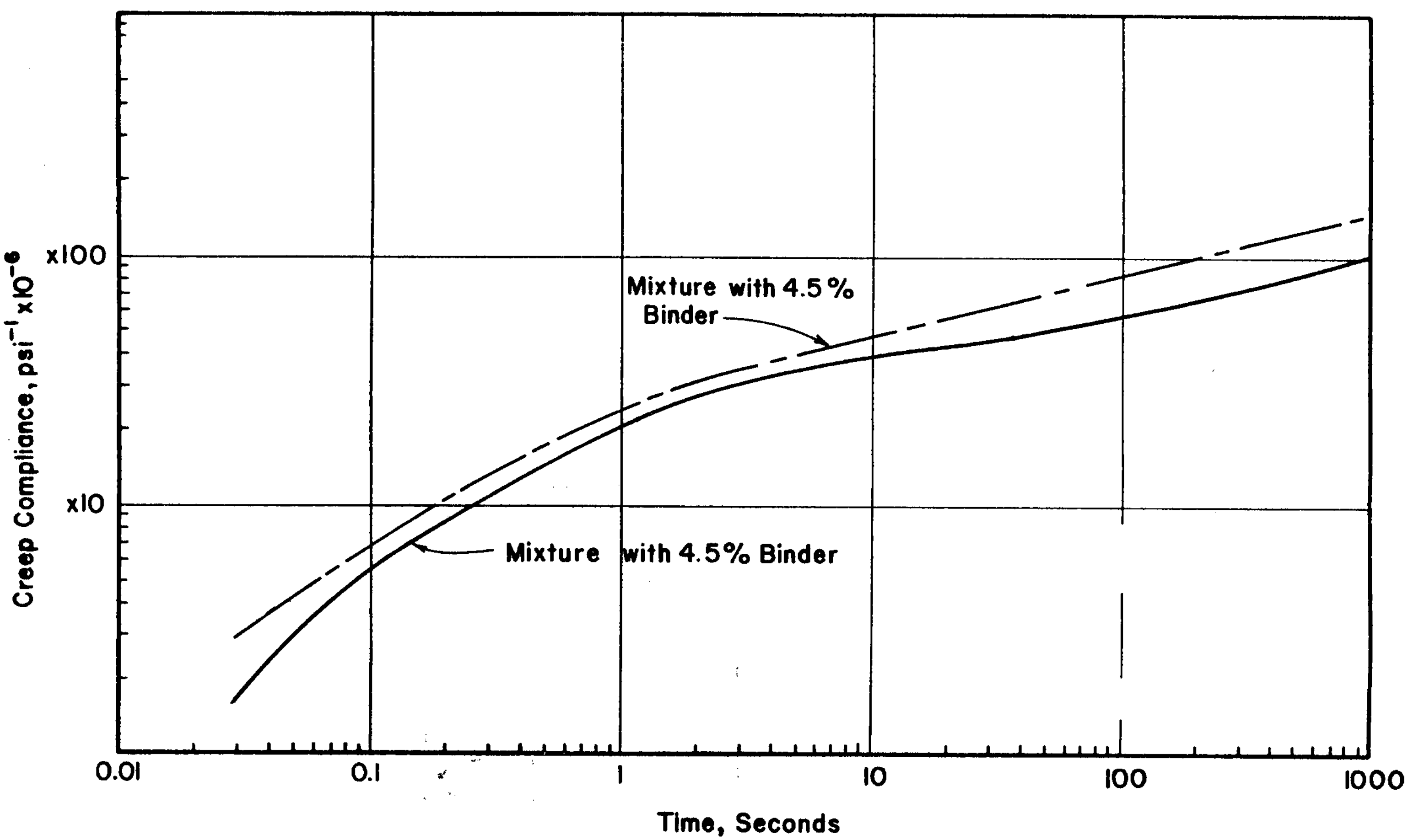

Figure 51. Comparison of creep compliance of mixtures containing 4.5 and 5.0 percent of blends of latex and AC-5. (After Reference 1)

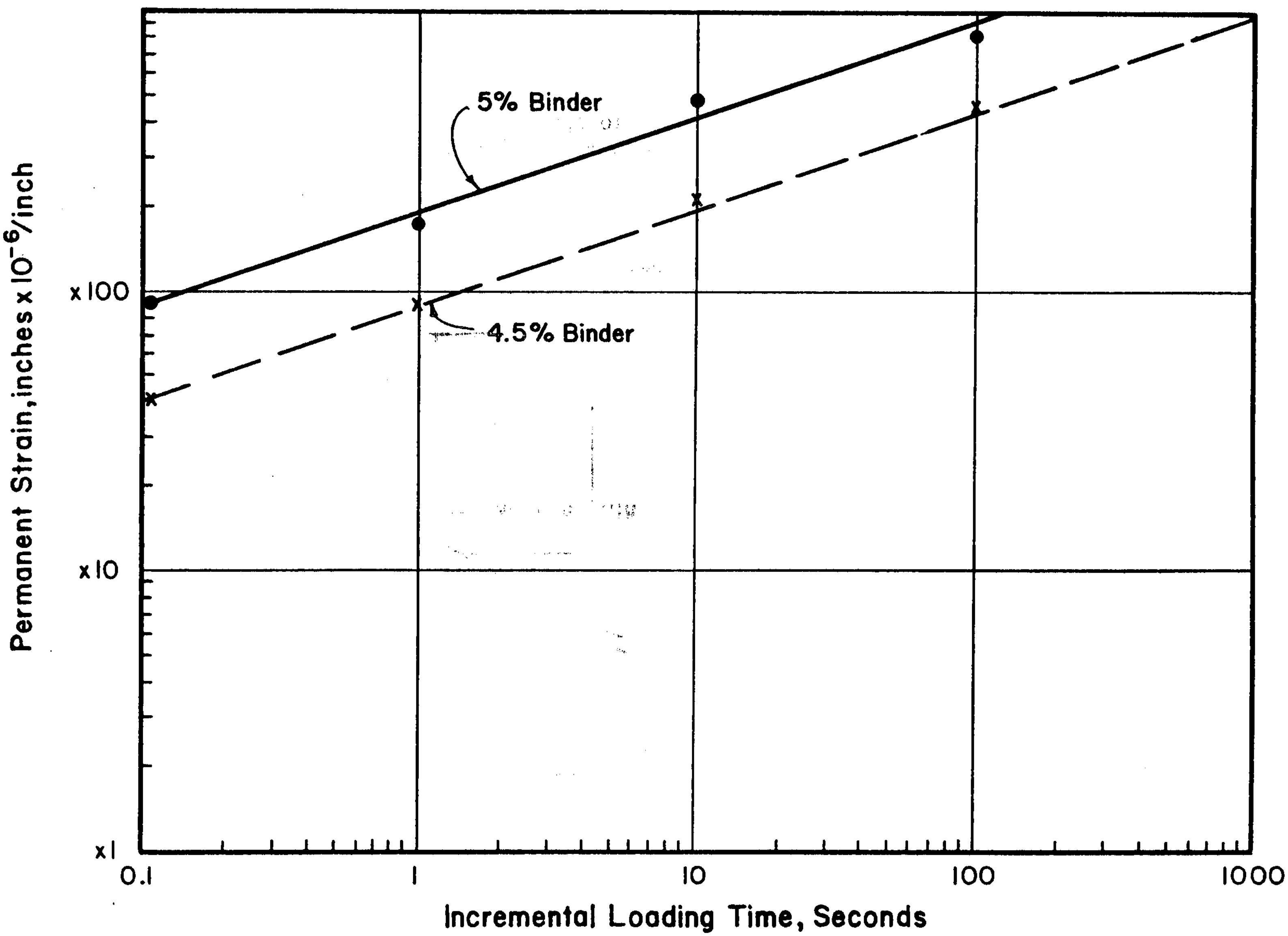

Figure 52. Comparison of incremental loading derived permanent deformation plots for mixtures containing AC-5 and latex (5% binder versus 4.5% binder). (After Reference 1)

these data. Indeed minor changes in the binder content may have as great an effect on permanent deformation performance as the additives themselves.

EVALUATION OF THERMAL CRACKING POTENTIAL

Approach

The thermal cracking potential of the binders and mixtures studied was evaluated using three approaches. The first approach was based on traditionally used concepts of limiting stiffness and critical stress. This approach is based totally on binder properties and is reported in reference 42. The second approach is based on thermal-fatigue cracking and fracture mechanics and the viscous response of the binder in an asphalt concrete matrix. Finally, the indirect tensile strengths over a wide range of temperatures were compared to stresses induced in a pavement. The induced stresses were computed using a viscoelastic slab theory.

Thermal Fatigue Analysis

Models for low temperature cracking have been used with varying degrees of success in the more northerly regions of the United States and Canada. In these areas, the temperature drops low enough that it will reach the "fracture temperatures" of the pavement material. This fracture temperature is defined as the temperature at which the developed tensile thermal stress exceeds the tensile strength of the asphalt concrete mixture. However, in many cases transverse cracking may be common even though the pavement has not been subjected to such temperature extremes.

A mechanism to account for the thermally induced transverse cracking of flexible pavements other than the low temperature cracking model mentioned above is thermal fatigue cracking. It was first described by M. Shahin and B. F. McCullough (56) and is defined to be caused by thermal fatigue distress due to daily temperature cycling, which eventually exceeds the fatigue resistance of the asphalt concrete.

Lytton and Shanmughan (57) have developed a computer model based on fracture mechanisms for predicting transverse cracking due to thermal fatigue cracking in asphalt concrete pavements. Basically, the model uses Shahin's and McCullough's revision to Barber's Equations (58) to compute pavement temperatures based on air temperatures, wind speed and solar radiation; calculates pavement effective moduli and employs the computation of stress intensity factors and fracture mechanics to predict thermal fatigue resistance.

The results of the Lytton-Shanmugham procedure are typically presented as cumulative damage factors. The Lytton-Shanmugham model demonstrated exceptional ability to predict thermal cracking in pavements in Michigan and West Texas. Despite the fundamental superiority of the Lytton-Shanmugham procedure over traditionally used procedures, the Lytton-Shanmugham model was not used in this analysis. It was originally thought that the stiffness versus temperature relationship, as predicted by the Van der Poel equation, could be used in the Lytton-Shanmugham model; however, the modification of asphalt by the addition of polymers substantially altered the viscoelastic behavior of the resultant binder. This alteration may in effect invalidate the use of the Van der Poel model for the prediction of binder stiffness. In addition, the transformation normally used (in the Shell procedure) to predict mixture stiffness from bitumen stiffness has not been substantiated. For these reasons it was decided to evaluate thermal cracking based on mixture stiffnesses actually measured over a range of temperatures. Although the Lytton-Shanmugham model allows direct input of stiffness versus temperature data, not all of these data were available at the time of analysis.

Thermal Cracking Analysis Based on Mixture Properties

The previous analyses were based on mixture stiffnesses predicted from rheological properties of the binder, and aggregate volumetric properties were accounted for strictly in an empirical manner. The final analysis is based on mixture properties.

The available literature suggests that the fracture strength of asphalt concrete is at its highest level at low temperatures and/or rapid loading rates; but fracture occurs at small strains. In fact, Finn (59) defined the limiting strain for asphalt concrete at relatively low temperatures to be approximately 1.0×10^{-3} in/in. This is about one order of magnitude greater than allowable strain levels in fatigue loading.

The critical condition for fracture in asphalt concrete occurs at low temperatures and/or rapid loading rates. It is here that the asphalt behaves in a most brittle manner.

McLeod (60) has used the bitumen stiffness as a fundamental indicator of the asphalt cement characteristics; limits are placed on the bitumen stiffness at a given low temperature to eliminate transverse cracking. McLeod concluded that cracking will not occur if mix stiffness is less than 1 x 10^6 psi at 20,000 seconds loading time for the minimum anticipated temperature. Saal (61) used virtually the same approach as McLeod. Saal calculated that the limiting stiffness for asphalt concrete was approximately 715,000 psi for a loading time of 10^4 seconds and a change in temperature from 32^0F to 14^0F.

Monismith, et. al. (62), using the principles of linear viscoelasticity and creep compliance data, computed by numerical methods the stresses at the surface of an asphalt concrete slab subjected to a range of temperature distributions. They showed that, in the north central U.S. and in Canada, surface stresses in excess of 3,300 psi could be induced. This far exceeds the fracture strength of any asphalt concrete.

The postulated mechanism for cracking is traditionally based on the concept of induced thermal stresses, which exceed the tensile strength.

$$\sigma(\dot{T}) = \alpha_T \sum_{T_0}^{T_f} S_{(\Delta T)} \cdot (\Delta T) \qquad \text{Equation 14}$$

where $\sigma(\dot{T})$ = accumulated thermal stress for a particular cooling rate T,

α = average thermal contraction coefficient over the temperature drop, $T_0 - T_f$,

T_0, T_f = initial and final temperature and

$S_{(\Delta T)}$ = stiffness at the midpoint of discrete temperature intervals T over the range of T_0 and T_f, using a loading time corresponding to the time interval for the T change.

Of course, if the predicted thermal stress as a function of temperature exceeds the tensile strength of the mix at that temperature, fracture occurs.

The above relationship clearly illustrates the fact that very stiff mixtures are susceptible to low temperature fracture even if they possess high fracture strength because of the high stresses induced during cooling.

For a selected cooling rate, $\dot{T}$, temperature drop (cooling rate times period of cooling) and thermal contraction coefficient (approximately equal to 1.5 x 10^{-6} in/in/^{o}F for asphalt concrete between 30^{o}F and -20^{o}F, the only variable affecting $\sigma(\dot{T})$ is $S(\Delta T)$. Stiffness, $S(\Delta T)$, versus temperature was computed for the modified Texaco and San Joaquin Valley asphalt mixtures, respectively. The stiffnesses were derived from the creep compliance tests and the time temperature shift factor, a , discussed earlier in this chapter. The $S(\Delta T)$ values are for loading rates of 7,200 second which is assumed to approximate a 10^{o}F/hr temperature drop and is based on field data. This loading rate is simulated in the laboratory in the indirect tensile mode by a stroke rate of 0.02 to 0.01 in/min.

Once the rate and range of temperature drop are fixed, stiffness is the only variable of consequence. Figures 53 and 54 clearly illustrate the effect of stiffness on producing induced stresses within the AC. The plots in Figures 53 and 54 are for a cooling rate of 10^{o}F/hr and a temperature drop of four hours.

Discussion of Thermal Cracking Analysis

Based on this analysis, the following conclusions are formed:

1. The results of the different methods of analysis are somewhat contradictory. However, in general the softer asphalts (AC-5 and AR-1000), with or without modifiers, performed significantly better than the stiffer, control asphalts.

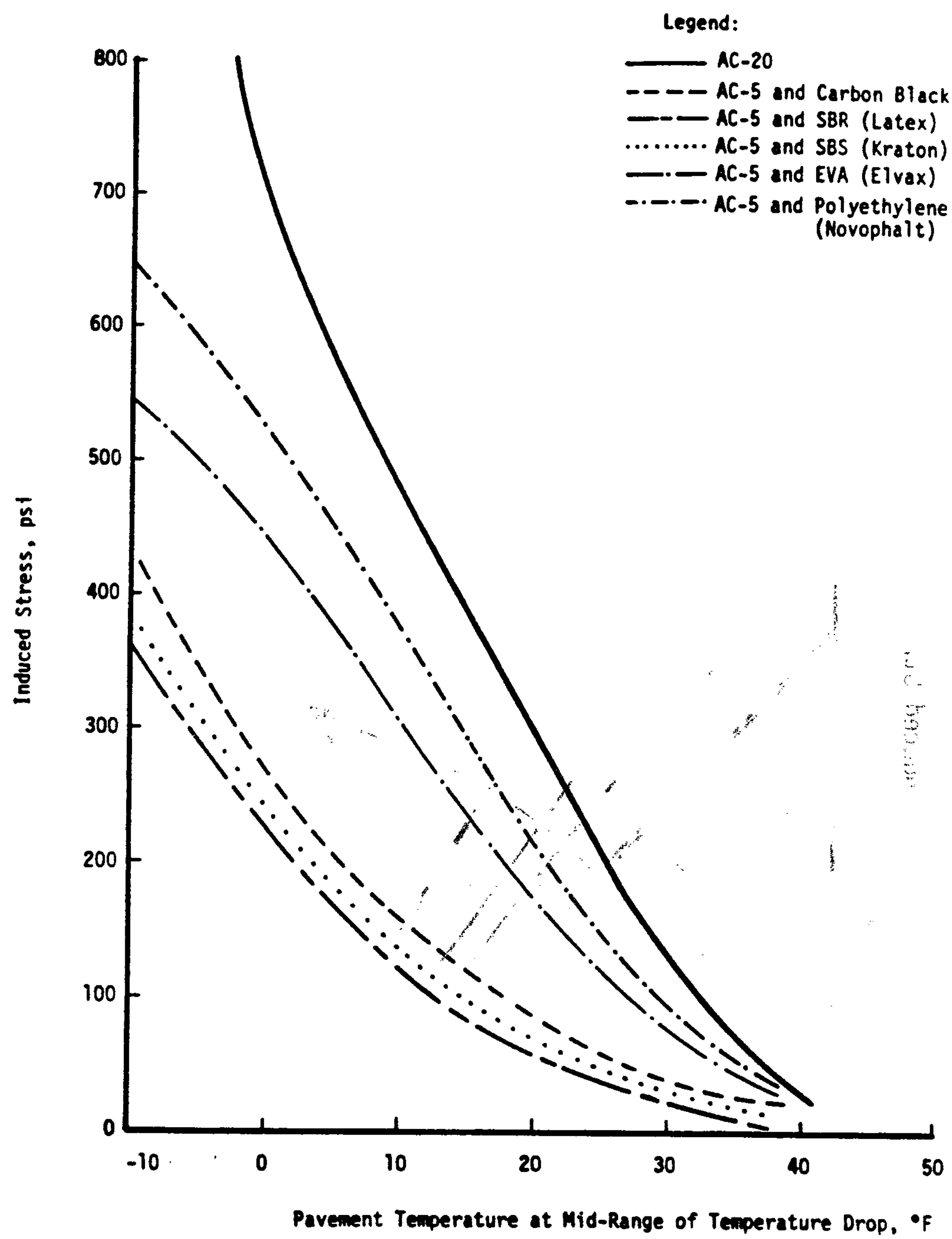

Figure 53. Induced stresses for modified Texaco asphalt mixtures. (After Reference 1)

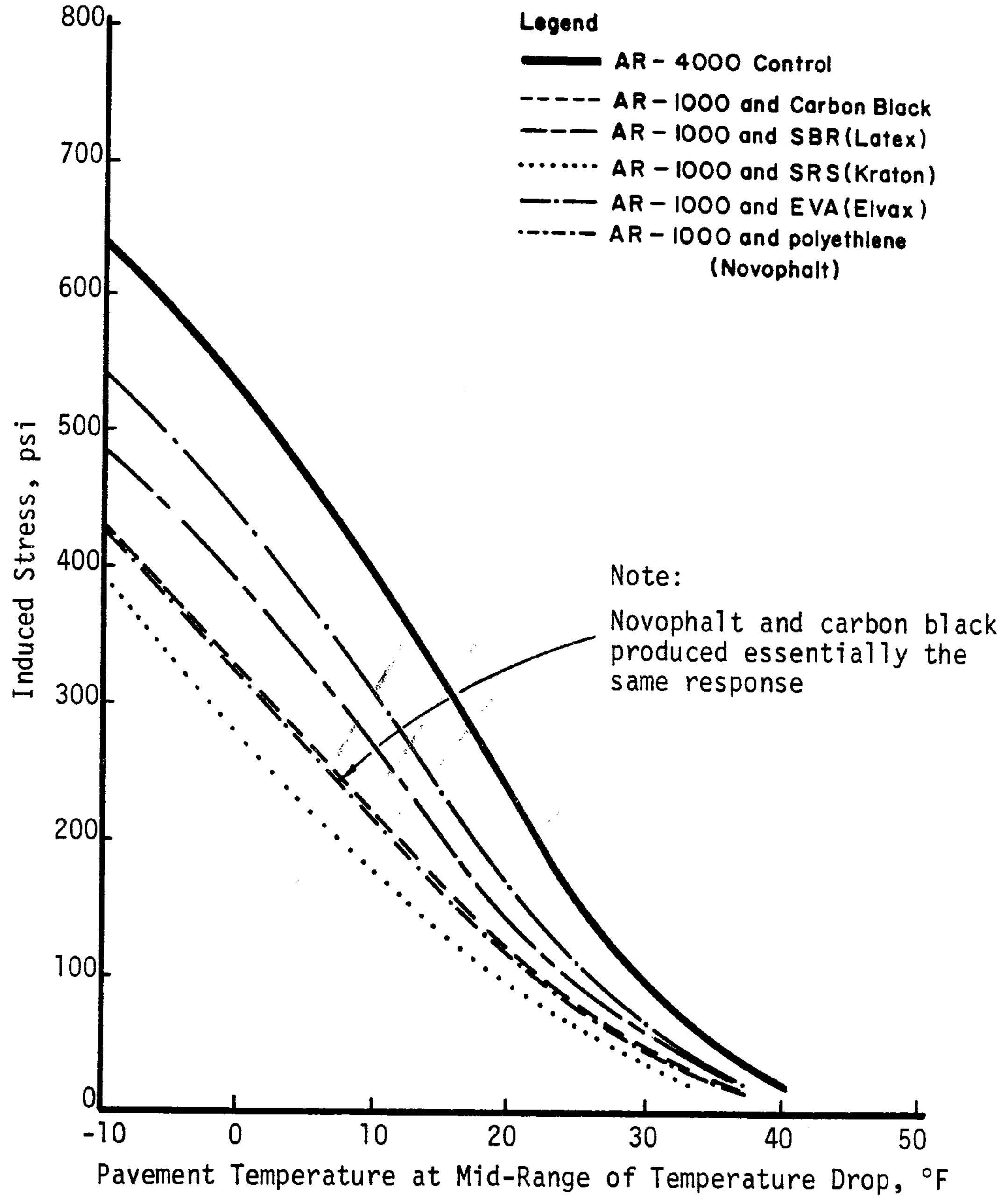

Figure 54. Induced stresses for modified SanJoaquin Valley asphalt mixtures. (After Reference 1)

2. The thermal cracking analysis based on the creep testing of mixtures revealed substantial differences in S_{mix} versus temperature relationships and in the resulting levels of induced stress. Furthermore, indirect tensile data, Figures 17 through 22, indicate similar tensile strength versus temperature results among the mixtures. This underscores the importance of insuring relatively low levels of S_{mix} versus temperature for mixtures subjected to rapid thermal temperature drops.

3. The ability of the modifier to produce favorable S_{mix} versus temperature relationships is highly dependent upon asphalt-additive compatibility.

4. Acceptable low temperature performance is a function of the rheological properties of the base asphalt.

The authors believe that the results of the mixture analysis summarized in Figures 53 and 54 deserve the most credibility as these are based on stiffnesses actually measured in creep compliance testing at loading rates which simulate those actually occurring in the field. These tests have the best chance of evaluating the response of the additive-modified asphalt; whereas, nomographic solutions based on physical properties of the bulk binder only may be biased as they do not account for aggregate effects and are based on empirical data for asphalt cement (unmodified). With this in mind, the general trend is that all additive-soft asphalt blends significantly reduce thermally induced stresses in the mixture. Polyethylene (Novophalt) appears to be quite susceptible to the base asphalt properties, showing a much more compliant nature with the San Joaquin Valley asphalt than with the Texaco asphalt.

MODULUS PROPERTIES OF ASPHALT MIXTURES MODIFIED WITH ADDITIVES

General

The modulus properties of the materials which make up flexible pavement layers are an indispensible part of most up-to-date structural pavement design techniques. In fact, the most commonly used failure criteria in flexible pavement design are tensile strain in the stiffest layer and vertical compressive strain in the subgrade layer. These criteria are extremely sensitive to the respective modulus properties of the pavement layers. Thus, the pavement engineer must not only seek an accurate estimate of the modulus but also the proper definition of modulus for the intended purpose.

Of course, viscoelastic materials such as asphalt concrete add another dimension of difficulty to the task of selecting the correct modulus. These materials have modulus properties which are affected by time (duration of loading) and temperature.

Van der Poel (63) has defined the modulus of asphalt cement as stiffness:

$$S(t,T) = \sigma/\varepsilon \qquad \text{Equation 15}$$

where t = time of loading and T = temperature.

Figure 55 is a simplified illustration of the time of loading dependency of idealized asphalt concrete at a selected temperature. It is easy to trace the change in behavior from an elastic response at short loading times, through a delayed elastic behavior zone and finally to a region where the stiffness is totally a function of the viscous properties of the binder. This representation is helpful in analyzing the creep stiffness data presented previously. Due to the time-temperature superposition of asphalt the abscissa in Figure 55 could be changed to temperature if stiffness were measured at a selected duration of loading.

In this study, the modulus properties of additive-modified asphalts were measured in the following forms:

1. Diametral resilient modulus, M_R.
2. Creep Stiffness.
3. Dynamic modulus, (measured on 4 in by 8 in cylinders).
4. Flexural modulus.

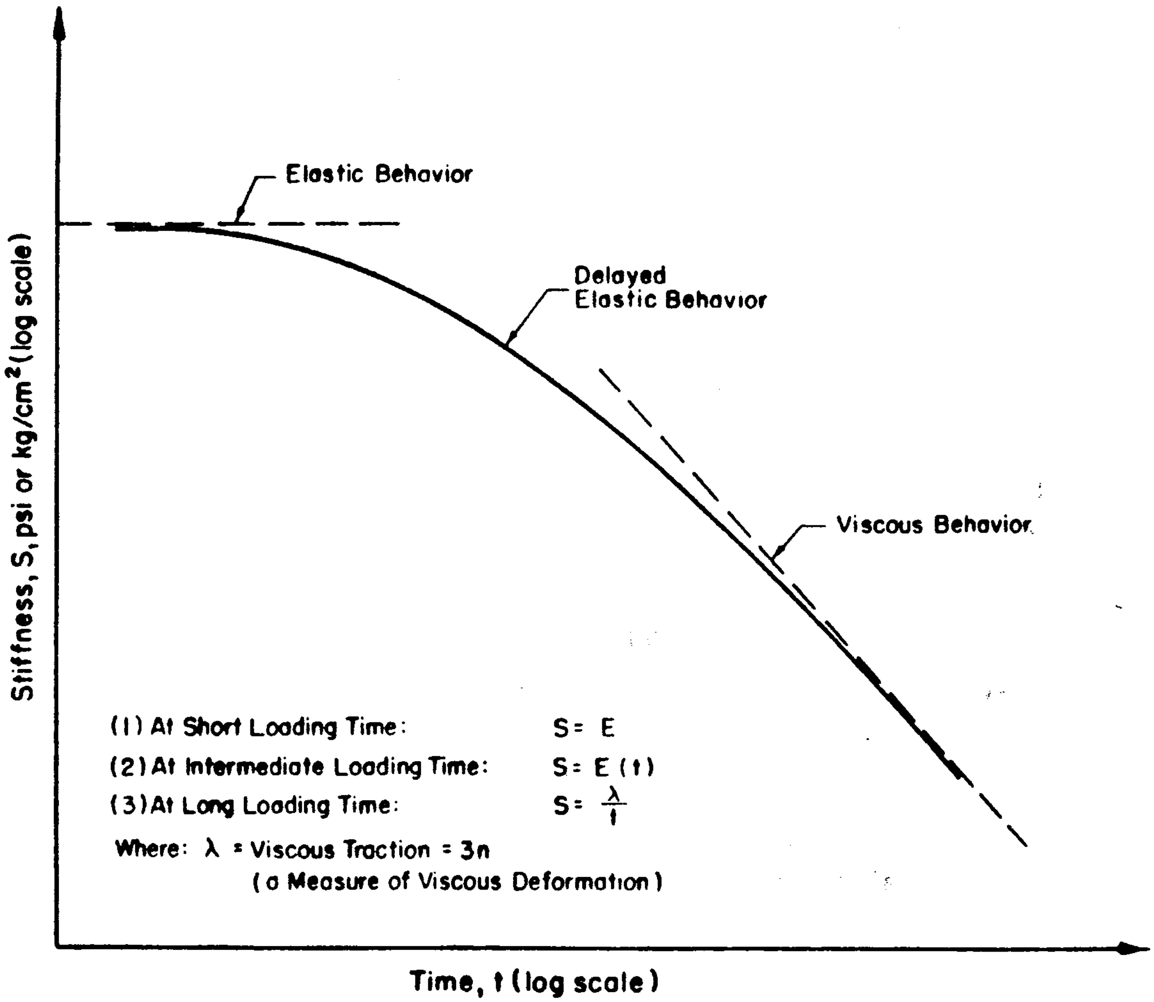

Figure 55. Simplified illustration of components of stiffness: elastic, viscoelastic, and viscous. (After Reference 1)

Resilient Modulus

The resilient moduli, defined as the ratio of induced stress to recoverable strain, were measured by the Mark III device developed by Schmidt (64). The device applies a 0.1-second load pulse once every three seconds across the vertical diameter of a cylindrical specimen (Hveem type specimen) and senses by linear variable transformers the resultant deformation across the horizontal diameter.

The resilient modulus was used throughout this research as a quality assurance measure. Resilient moduli were recorded from aging studies, water susceptibility studies, and mixture design studies. The results of these data are reported earlier in this chapter. All resilient modulus specimens were aged for six days at 50^{o}F prior to testing. The 6-day cure period was selected based on an aging study by Button, et al (51) which revealed that the resilient modulus does not appreciably change in the laboratory following six days of curing at 50^{o}F but does change significantly during the first 6 days.

Creep Stiffness

The diametral resilient modulus is often subjected to criticism because of the light load used, the conditions of biaxial stressing and the rigid assumptions which must be closely adhered to in order for the cylindrical, diametrally loaded specimen to respond elastically. In order to more precisely establish the modular properties of asphalt mixtures under different conditions of loading and different states of stress, other forms of moduli were computed.

Creep stiffness is simply the inverse of the creep compliance. For purposes of comparison creep stiffness was calculated at 0.1 seconds of load duration at 40^{o}F, 70^{o}F and 100^{o}F during the compressive creep test. The resulting values are tabulated in Table 18.

As expected, these moduli do not closely agree with the resilient moduli. However, the same trends are evident as were established with resilient moduli data.

Table 18. Summary of average dynamic moduli, creep stiffness (0.1 sec.) and resilient moduli from all mixtures fabricated with Texaco asphalt.

Binder	Dynamic Modulus $psi \times 10^6$			Creep Stiffness $psi \times 10^6$			Resilient Moduli $psi \times 10^6$			Flexural Modulus $psi \times 10^6$	
	40°F	70°F	100°F	40°F	70°F	100°F	40°F	70°F	100°F	34°F	68°F
AC-20	6.11	0.76	0.27	4.00	0.35	0.10	1.75	0.65	0.12	1.28	0.71
AC-5 and Carbon Black	2.54	0.48	0.19	1.42	0.14	0.075	0.90	0.16	0.040	0.97	0.22
AC-5 and EVA (Elvax)	1.96	0.39	0.06	1.81	0.50	0.050	1.10	0.30	0.055	0.99	0.08
AC-5 and SBS (Kraton)	2.13	0.60	0.13	1.33	0.20	0.080	1.20	0.35	0.060	0.91	0.09
AC-5 and SBR (Latex)	3.43	0.74	0.09	1.33	0.25	0.033	1.00	0.18	0.045	0.81	0.12
AC-5 and Polyethylene (Novophalt)	3.38	0.98	0.23	3.33	0.40	0.14	1.25	0.45	0.080	0.90	0.34

(After Reference 1)

Dynamic Modulus

The dynamic modulus is defined as the ratio of repeated stress applied in an unconfined compressive mode to recoverable elastic strain at the 200th load cycle. This test is performed on 4 in by 8 in cylinders as prescribed in the VESYS Manual (49).

Results of dynamic modulus testing for blends of additives with the Texaco asphalts at three temperatures (40, 70 and 100°F) are presented in Table 18.

Flexural Modulus

Flexural moduli from the Texaco asphalt mixtures are also summarized in Table 18. The flexural modulus is defined as the modulus of the flexural fatigue beams at the 200th load application. The modulus is more clearly defined as

$$E_{flex.} = \frac{Pa(3\ell^2 - 4a^2)}{48I\Delta} \qquad \text{Equation 16}$$

where P = dynamic load applied to deflect the beam (lb),

a = (ℓ/3) (inches),

ℓ = reaction span length (12 in),

I = specimen moment of inertia (in^4) and

Δ = dynamic beam deflection at the center point (in).

Discussion of Results of Modulus Testing

Although the absolute modulus values differ substantially depending on the type of test (Table 18), the general trend is that the AC-20 mixture is stiffest at all temperatures, followed by AC-5 mixtures containing polyethylene (Novophalt). Mixtures containing AC-5 and SBS (Kraton), SBR (Latex) and EVA (Elvax) show somewhat similar trends.

At $100^{o}F$, which represents a nominal high temperature range, the AC-20 control responds with the highest modulus (considering each type of modulus test) followed by the mixture containing AC-5 and polyethylene (Novophalt). Once again, mixtures containing blends of AC-5 and SBS, SBR, carbon black and EVA are not significantly different based on stiffness at $100^{o}F$.

At the nominal low temperature, $40^{o}F$, the results are similar. Once again, the AC-20 control produced the lowest average modulus based on each test type (i.e., diametral, creep and resilient modulus) and the AC-5 polyethylene (Novophalt) mixture ranks as second stiffest. There are no significant differences among the other responses.

Although no clearly defined advantages are established for any additive based on these data, the polyethylene (Novophalt) appears to be most beneficial in reducing temperature susceptibility. This is, of course, the benefical claim of most polymer and/or filler-type additives.

Table 19 summarizes the results of modulus testing for modified San Joaquin Valley asphalts. The results of the diametral resilient modulus testing are graphically presented in Figure 16 of Chapter IV. Based on these results, polyethylene, carbon black and latex all show similar (statistically no difference) effects in improving the temperature susceptibility of the base asphalt (AR-1000). Kraton was slightly (though statistically significant) less effective. At the higher temperatures, Elvax responded statistically the same as the AR-1000, i.e., no stiffening.

Based on creep stiffnesses at a loading time of 0.1 seconds, the AR-4000 control was stiffest by a factor of about 2.5 over the AR-1000 blends with either carbon black, Elvax or Novophalt. The control mixture was 4.5 times stiffer than the Kraton blend and about 60 percent stiffer than the latex blend.

At a loading time of 1000 seconds, the AR-4000 control was essentially the same as AR-1000 blends with either carbon black, Elvax or latex. Blends of AR-1000 and both Kraton and Novophalt were substantially softer. The softness of the AR-1000 and Novophalt blend appears to be a function of asphalt-polymer compatibility.

Table 19. Summary of resilient moduli and creep stiffnesses (0.1 sec - 1000 sec) for all mixtures fabricated with San Joaquin Valley asphalt.

Binder	Resilient Modulus, psi x 10^6			Creep Stiffness, psi x 10^6	
	40°F	70°F	100°F	0.1 sec Loading	1000 sec Loading
AR- 4000	1.75	0.80	0.12	0.80	0.014
AR-1000	0.80	0.20	0.02	—	—
AR-1000 and Carbon Black	1.20	0.38	0.07	0.27	0.018
AR-1000 and EVA (Elvax)	0.80	0.20	0.02	0.30	0.013
AR-1000 and SBS (Kraton)	1.00	0.32	0.04	0.18	0.008
AR-1000 and SBR (Latex)	0.95	0.31	0.05	0.53	0.012
AR-1000 and Polyethylene (Novophalt)	1.20	0.40	0.07	0.27	0.009

(After Reference 1)

CHAPTER 6

UTILITY AND COSTS

METHODS OF ADDITION

Methods of incorporating the additives into asphalt were not specifically addressed in this study. Contacts with highway department and contractor personnel indicate that the preferred method is to combine the asphalt and additive(s) prior to arrival at the mix plant site. This would provide good dispersion of the additive and would not interrupt normal mixing plant operations. It should be pointed out that tank trucks have no positive agitation capability and certain additives, such as polyethylene and carbon black, may separate from the asphalt during hot storage. Therefore, preblending may not always be possible.

Felsinger, Inc., the supplier of Novophalt, has developed a high shear blending apparatus capable of modifying asphalt at the plant site without delaying plant operations. If storage of modified asphalt is necessary, the apparatus is furnished with an integral surge tank which is equipped for remixing as required.

During the course of this work, Monochem Corporation of Atlanta, Texas has developed a proprietary dispersing agent for carbon black which holds it in suspension in hot asphalt for periods of at least 2 weeks. Two field trials have been installed in Texas using preblended carbon black with drum plants in 3-inch hot mix asphalt concrete pavements. Other than increasing mix temperature, no changes in plant operations and paving procedures were required.

Latex (70 percent SBR and 30 percent water) is often added in drum mix plants just downstream from the asphalt inlet or in batch plants shortly after the addition of the asphalt and a brief mixing period. In either case, the relatively small quantity of water is flashed away without consequence (apparently). Additional work is being performed at Texas A&M University to investigate differences in mixing efficiency when the latex is added in the plant or preblended with the asphalt cement.

Prolonged hot storage of modified asphalts can cause degradation of quality other than physical separation of the asphalt and the additive. Laboratory tests and field experience has shown that block copolymers (SBS) and latex (SBR) will "break down" and exhibit a significant decrease in viscosity upon prolonged hot storage or exposure to excessive heat. These "reactions" are without doubt dependent upon the chemical composition of the additive as well as the asphalt. Additional study will be required to define safe limits for storage periods and temperatures. Additive manufacturers should provide this information to their customers.

MIXING AND COMPACTION TEMPERATURES

According to the Asphalt Institute (65), there are certain binder viscosities that should be used for optimum mixing (170 centistokes) and compaction (280 centistokes) of asphalt concrete mixtures. All of the additives addressed in this study produce a significant increase in the 300°F viscosity of the original asphalt. Even when a softer-than-usual grade of asphalt (AC-5) is used with the additive, the high temperature viscosity of the blend may be greater than that of the usual grade asphalt (AC-20). Therefore, in order to assure suitable mixing and adequate compaction time, it may be necessary to increase the plant temperature. Field experience with the additives studied herein has shown that the increase in temperature is necessary to achieve good compaction; however, optimum mixing and compaction temperatures are not simply a function of the viscosity of the binder when asphalt additives are used. These optimum temperatures need to be determined. Viscosity-temperature data for these modified binders can be used as a guide; but, apparently, only field experience can be used to make final decisions.

SPECIAL REQUIREMENTS ASSOCIATED WITH ADDITIVES

The use of specific additives under certain circumstances presents special needs regarding equipment and logistics. Most refineries or asphalt distributors are not presently equipped to properly mix additives into their asphalt products. Therefore, when an additive is specified, special processing is necessary either at the asphalt distribution point or at the mix plant site.

When carbon black is used in conjunction with a batch plant, preweighed polyethylene bags are introduced directly into the pug mill. The polyethylene melts and the carbon black is dispersed into the mix. However, when a drum mix plant is employed, the carbon black must be preblended with the asphalt cement or "blown" into the drum just downstream from the asphalt inlet. Preblended carbon black in hot-stored asphalt will "settle out" (specific gravity of carbon black is 1.7) if not treated with special dispersing agents. Blowing of the carbon black into the drum plant requires special conveying and metering equipment; and there is a high probability of losing much of the carbon black in the stack gases.

Styrene-butadiene rubber (latex) can either be preblended with asphalt or added in the plant. Addition of latex in a plant (drum mix or batch) is usually accomplished after addition of the asphalt. In either plant, special equipment is necessary to measure and transfer the latex. At least one highway district in Texas requires the addition of latex after introduction of the asphalt and initial coating of the aggregate. This is an attempt to eliminate detrimental changes that may occur in latex-modified asphalt during hot storage. When using porous aggregate, some agencies require addition of latex in the drum mix plant to avoid loss in effectiveness of the relatively expensive additive by minimizing the quantity of latex that is absorbed into the aggregate.

Obviously, preblending of additives in asphalt will minimize changes in mix plant operations. However, this blending operation, whether at the refinery or the asphalt distribution point, requires special equipment. If the asphalt producer does not have blending capabilities, special arrangements must be made which could involve shipment of the asphalt to a blending facility before final shipment to the plant site.

APPROXIMATE COSTS FOR ADDITIVES

Costs of the additives examined herein are influenced by the cost of crude oil, as is the cost of asphalt cement. Currently, the price for the polymers ranges from 0.80 to 1.00 dollars per pound and for the carbon black, about 0.50 dollars per pound. This translates into a cost increase of about 4.00 to 9.00 dollars per ton of hot mixed asphalt concrete, depending on the dosage of the additive. Based on an in-place cost of 35.00 dollars per ton of hot mixed asphalt concrete, the additives would increase the paving cost by about 10 to 25 percent. In other words, assuming an average pavement life of 15 years, an additive would need to increase pavement life by 2 to 4 years to be cost effective or decrease maintenance costs accordingly. Based on the laboratory test results reported herein, the polymer and microfiller additives studied can reasonably be expected to provide cost effective pavement performance. Cost effectiveness of the additives discussed in this report is being evaluated in field installations in Districts 1, 19 and 21 under Study 187 (Task 5).

CHAPTER 7

SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

The first phase of this research study included an extensive review of published data on all known asphalt additives and some admixtures. Additives were sought out that showed potential to reduce cracking in asphalt concrete pavements without adversely affecting rutting. It appeared that, for best results, a softer than usual asphalt is used with an additive capable of lowering the temperature susceptibility of the binder. The soft asphalt provides flexibility to reduce cracking at the lower temperatures and the additive increases the viscosity at higher temperatures to reduce the potential for permanent deformation. From both costs and physical properties standpoints, certain types of polymers and carbon black appeared to be promising. Five additives were selected and evaluated in a logical sequence of laboratory tests. The effects of these additives on rheological and physicochemical properties of asphalt cement and on mixture stability, stiffness, tensile properties, and resistance to fatigue and thermal cracking, plastic deformation and moisture damage were assessed. Based on results of laboratory tests and review of the current literature on asphalt additives, the following conclusions are tendered.

1. The Texas SDHPT asphalt mixture design procedure as well as the Marshall and Hveem methods appear acceptable for determining optimum binder contents for additive-modified asphalt mixtures.

2. Although certain binder and mixture properties appeared to be sensitive to compatibility between the asphalt and the additives, overall, the mixture properties demonstrated an ability for each additive to alter mixture temperature susceptibility in a generally favorable manner. The degree of alteration is highly dependent upon the chemical composition and/or physical properties of the asphalt cement. Synergistic effects likely influence the properties of the blend.

3. Hveem stability of mixtures was not significantly altered by the additives. Although Hveem stability is quite sensitive to changes in binder quantity, it is not very sensitive to changes in rheological properties of the binder properties.

4. The additives increased Marshall stability of mixtures when added to AC-5 or AR-1000 but not up to that of mixtures containing AC-20 or AR-4000 with no additive. This should not discourage the use of these additives with asphalts softer than the usual paving grade, particularly where low temperature cracking is a concern.

5. At low temperatures (less than 32^{o}F), the additives had little effect on consistency of the asphalt cements. This was reflected in the diametral resilient moduli (stiffness) of the mixtures. Resilient moduli of AC-5 (or AR-1000) mixtures above 60^{o}F were generally increased by the additives but not up to that of the AC-20 (or AR-4000) mixtures without additives. Although the load spreading ability of asphalt concrete containing a soft asphalt is increased when these additives are employed, the pavement thickness should not be reduced.

6. Indirect tension test results showed that, at the lower temperatures and higher loading rates, the additives increased mixture tensile strength over that of the control mixtures. Strain (deformation) at failure was generally increased by the additives. This is indicative of improved resistance to traffic-induced cracking at low temperatures. At the higher temperatures and lower loading rates, the additives did not appreciably affect the mixture tensile properties as measured by the indirect tension test.

7. The additives had little effect on moisture susceptibility of the mixtures made using the materials included in this study.

8. Flexural fatigue response of mixtures containing AC-5 plus an additive at 68^{o}F and particularly at 32^{o}F was superior to the control mixture which contained AC-20 with no additive. Accelerated aging of test specimens containing additives resulted in a significant decrease in fatigue life; the control specimens, however, exhibited improved fatigue properties after aging.

9. Creep/permanent deformation testing showed that, at high temperatures, all the additives except latex produced equal or better performance than the AC-20 control mixture. (The binder content of the latex mixture was apparently in excess of the true optimum which adversely affected creep/permanent deformation.) At low temperatures, all the additives in AC-5 except polyethylene produced equal or better creep compliance than the AC-20 control mixture.

10. Controlled displacement fatigue testing (overlay tester) at 34^{o}F demonstrated that mixtures containing AC-5 plus an additive gave greater resistance to crack propagation than control mixtures containing AC-20. The "dissolved" additives, EVA, SBR and SBS, showed evidence of improving the distribution of tensile stresses within the mixture. Practically, this could result in retarding crack propagation which should be manifested by resistance to cracking in asphalt concrete overlays.

11. Standard asphalt extraction methods to determine binder content of paving mixtures are unsuitable when polymers or carbon black are used as these materials are insoluble or only partly soluble in standard solvents.

12. Short term aging characteristics of modified binders, as measured by standard tests, did not manifest an appreciable difference from the unmodified asphalts. However, aging of compacted mixture specimens showed significant differences when additives were employed.

13. Force ductility offers promise as a means of estimating compatibility between an additive and asphalt. In addition, the force ductility test may be useful in predicting changes in mixture tensile strength when asphalt additives are employed.

14. Each additive proved to be successful to some degree in improving properties on at least one end of the performance spectrum. However, no additive proved to be a cure-all. There is a need , therefore, to develop an additive selection procedure based on conditions of traffic, pavement structure and climate. To rank the additives according to relative capabilities is a difficult task, as sensitivity to the base asphalt played a significant role. In general, the most effective additives in reducing rutting were EVA, polyethylene and SBS for the Texaco asphalts. For the San Joaquin Valley asphalt, carbon black, polyethylene, and EVA performed most

effectively and without significant difference. In terms of reduction of flexural fatigue cracking, the most successful additives were, in order, EVA, SBS, SBR (latex) and polyethylene which demonstrated essentially equal performance.

RECOMMENDATIONS

1. Future research efforts to evaluate asphalt additives should include a segment to investigate the long-term effects of compatibility with asphalt cement. This study showed no appreciable problem associated with compatibility and short-term mixture properties, however, long-term mixture properties were not evaluated. Since noncompatible products do not form a homogeneous blend, the two phases of the binder may age differently and have deleterious effects upon a pavement structure.

2. Satisfactory methods for extracting modified binders from paving mixtures should be developed. A suitable procedure is necessary for quality assurance regarding binder and additive contents.

3. Investigate force-ductility as a means to evaluate compatibility of additives and binders.

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APPENDIX A

Physical Properties of Binders

Table A1. Blends of carbon black with Texaco asphalts.

	Base Asphalt								
	Texaco AC-5				Texaco AC-10				Texaco AC-20
Asphalt, %	100	100	90	85	100	100	90	85	100
Microfil 8[a], %	...	...	10	15	...	...	10	15	...
Serial No.	7	33	34	35	11	36	37	38	17
Mixing, Blender	No	Yes	Yes	Yes	No	Yes	Yes	Yes	No
Undispersed Carbon Black, % of added Black[b]	...	...	0.34	0.05	...	...	0.06	0.10	...
Viscosity at 140°F[c], P	506	583	871	1850	1080	1210	1950	3540	2040
Viscosity at 275°F[d], cSt	224	...	504	740	332	...	...	...	398
Penetration at 77°F[e], 100 g, 5 s	194	189	179	152	118	112	106	92	75
Penetration at 39.2°F, 100 g, 5 s	20	21	23	21	12	15	16	15	8
Penetration at 39.2°F, 200 g, 60 s	63	65	65	66	41	48	46	48	28
Specific Gravity	1.019	...	...	1.075	...	...	...	...	1.029
Temperature Susceptibility[f], 140 to 275°F	3.42	...	2.97	2.99	3.40	...	...	...	3.52
PVN[g]	-0.3	...	1.0	1.44	-0.3	...	...	...	0.6
P.I.[h] from Penetration at 39.2°F and 77°F	-1.0	-0.8	-0.4	-0.2	-1.1	-0.3	+0.2	+0.4	-0.9
Penetration Ratio[i]	32	34	36	43	35	43	43	52	37

[a] Cabot Corporation, Lot CS-4632.

[b] Retained when solution of asphalt:black blend in VM&P naptha was washed on #325 sieve.

[c] AASHTO T202.

[d] AASHTO T201.

[e] AASHTO T49.

[f] Temperature susceptibility = $(\log\log \eta_2 - \log\log \eta_1)/(\log T_2 - \log T_1)$
where η = viscosity in cP, T = absolute temperature.

[g] Determined from penetration at 77°F and viscosity at 275°F (McLeod, 1976).

[h] P.I. = $(20 - 500\alpha)/(1 + 50\alpha)$:
where $\alpha = [\log(pen_2) - \log(pen_1)]/(T_2 - T_1)$ and T = temperature, °C.

[i] 100 (Pen 39.2°F, 200 g, 60 s)/(Pen 77°F, 100 g, 5 s).

Table A2. Blends of carbon black with San Joaquin Valley asphalts.

	Base asphalt								
	Valley AR-1000				Valley AR-2000				Valley AR-4000
Asphalt, %	100	100	90	85	100	100	90	85	100
Microfil 8[a], %	...	...	10	15	...	...	10	15	...
Serial No.	19	39	40	41	25	42	43	44	31
Mixing, blender	No	Yes	Yes	Yes	No	Yes	Yes	Yes	No
Undispersed carbon black, % of added black[b]	...	...	...	...	...	...	0.27	0.10	...
Viscosity at 140°F[c], P	498	549	942	1640	1100	1160	1940	3110	2170
Viscosity at 275°F[d], cSt	128	...	199	398	185	...	...	...	256
Penetration at 77°F[e], 100 g, 5 s	146	137	123	109	86	75	75	72	57
Penetration at 39.2°F, 100 g, 5 s	10	11	10	10	5	6	6	5	4
Penetration at 39.2°F, 200 g, 60 s	46	49	46	43	25	26	24	24	16
Temperature susceptibility[f] 140 to 275°F	3.94	...	3.80	3.43	3.93	...	...	...	3.92
PVN[g]	-1.6	...	-1.1	-0.1	-1.6	...	...	...	-1.4
P.I.[h] from penetration at 39.2°F and 77°F	-2.0	-1.7	-1.7	-1.4	-2.4	-1.7	-1.7	-2.0	-2.0
Penetration ratio[i]	32	36	37	39	29	35	32	33	28

[a]Cabot Corporation, Lot CS-4632.

[b]Retained when solution of asphalt:black blend in VM&P naphtha was washed on #325 sieve.

[c]AASHTO T202.

[d]AASHTO T201.

[e]AASHTO T49.

[f]Temperature susceptibility = $(\log \log \eta_2 - \log \log \eta_1)/(\log T_2 - \log T_1)$
where η = viscosity in cP, T = absolute temperature.

[g]Determined from penetration at 77°F and viscosity at 275°F (McLeod, 1976).

[h]P.I. = $(20 - 500\alpha)/(1 + 50\alpha)$:

where $\alpha = [\log(\text{pen}_2) - \log(\text{pen}_1)]/(T_2 - T_1)$ and T = temperature, °C.

[i]100(Pen 39.2°F, 200 g, 60 s)/(Pen 77°F, 100 g, 5 s).

Table A3. Blends of SBR (as latex) with asphalt.

Serial No.	11	54	55	56	57	58	25	59	60	61	62
Asphalt	Texaco AC-10						San Joaquin Valley AR-2000				
Latex	None	XUS 40052.00[a] ------->			Ultra Pave 70[b]		None	XUS 40052.00		Ultra Pave 70	
Proportions asphalt: rubber[c]	...	97:3	97:3	95:5	97:3	95:5	...	97:3	95:5	97:3	95:5
Viscosity at 140°F[d], P	1080	3210	2940	7280	3840	7620	1100	2230	4700	2350	5250
Viscosity at 275°F[e], cSt	332	...	1240	3110	...	...	185	966	3830	...	...
Penetration[f] at 77°F, 100 g, 5 s	118	91	96	78	90	78	86	73	75	69	74
Penetration[f] at 39.2°F, 100 g, 5 s	12	10	11	11	20	11	5	4	3	4	3
Penetration[f] at 39.2°F, 200 g, 60 s	41	40	42	45	70	44	25	17	18	17	20
Temperature susceptibility[g]	3.40	...	2.81	2.55	...	...	3.93	2.88	2.26	...	...
PVN[h]	-0.3	...	1.4	2.5			-1.6	0.7	2.8	...	...
Penetration index[i]	-1.1	-0.9	-0.7	-0.1	+1.7	-0.8	-2.4	-2.5	-3.1	-2.4	-3.1
Penetration ratio[j]	35	44	44	58	78	56	29	23	24	25	27

[a]Anionic SBR latex from Dow Chemical USA, 69.7% solids.

[b]Anionic SBR latex from Textile Rubber and Chemical Company, 70.1% solids.

[c]Proportions of asphalt to dry solids from latex; 300 g batches prepared by preheating asphalt in a Waring Blendor jar to 135-155°C (275-310°F), then adding latex slowly while operating the Blendor to flash off the water and disperse the rubber.

[d]AASHTO T202.

[e]AASHTO T201.

[f]AASHTO T49.

[g]Temperature susceptibility = $(\log\log \eta_2 - \log\log \eta_1)/(\log T_2 - \log T_1)$ where η = viscosity in cP, T = absolute temperature.

[h]Determined from penetration at 77°F and viscosity at 275°F (McLeod, 1976).

[i]P.I. = $(20 - 500\alpha)/(1 + 50\alpha)$:

α $[\log(\text{pen}_2) - \log(\text{pen}_1)]/(T_2 - T_1)$, where T = temperature, °C.

[j]100(Pen 39.2°F, 200 g, 60 s)/(Pen 77°F, 100 g, 5 s).

Table A4. Blends of SBR (as latex) with AC-5 and AR-1000 asphalts.

Serial No.	86	131	132	101	133	134
Asphalt	Texaco AC-5			San Joaquin Valley AR-1000		
Latex	None	XUS-40052.00[a]		None	XUS-40052.00	
Proportions asphalt: rubber[b]	....	97:3	95:5	....	97:3	95:5
Viscosity at 140°F[c], P	537	1960	5460	423	4020	10,100
Viscosity at 275°F[d], cSt	217	1020	2780	150	1190	3600
Penetration[e] at 77°F, 100 g, 5 s	186	140	114	164	83	72
Penetration[e] at 39.2°F, 100 g, 5 s	17	15	14	12	6	6
Penetration[e] at 39.2°F, 200 g, 50 s	66	57	54	59	28	29
Softening point[f], °C	41.4	51.6	62.5	41.2	54.2	66.8
Softening point[f], °F	106.5	125	144.5	106	129.5	152
Specific Gravity	1.019	1.014	1.012	1.017	1.014	1.015
Temperature susceptibility[g]	3.42	2.78	2.52	3.71	2.96	2.58
PVN[h]	-0.4	1.8	3.0	-1.2	1.2	2.5
P.I.[i] from penetration at 39.2 and 77°F	-1.4	-0.9	-0.5	-1.9	-1.9	-1.6
P.I. from penetration at 77°F and softening point	0.2	2.4	4.1	-0.4	1.2	3.3
Penetration ratio[j]	35	41	47	36	34	40

[a]Anionic SBR latex from Dow Chemical USA, 69.7% solids.

[b]Proportions of asphalt to dry solids from latex; 300 g batches prepared by preheating asphalt in a Waring Blender jar to 191-199°C (376-390°F), then adding latex slowly while operating the Blender to flash off the water and disperse the rubber.

[c]AASHTO T202.

[d]AASHTO T201.

[e]AASHTO T49.

[f]AASHTO T53.

[g]Temperature susceptibility = $(\log\log\eta_2 - \log\log\eta_1)/(\log T_2 - \log T_1)$ where η = viscosity in cP, T = absolute temperature.

[h]Determined from penetration at 77°F and viscosity at 275°F (McLeod, 1976).

[i]$P.I. = (20 - 500\alpha)/(1 + 50\alpha)$: $\alpha = [\log(pen_2) - \log(pen_1)]/(T_2 - T_1)$, or $[\log 800 - \log(pen_{25°C})]/(T_{SP} - 25)$ where T = temperature, °C.

[j]100 (Pen 39.2°F, 200 g, 60 s)/(Pen 77°F, 100 g, 5 s).

Table A5. Blends of asphalts with thermoplastic block polymers (Kraton).

Serial No.	86	102(1)	128	126	101	130	129	127
Texaco AC-5	100	95	94	88	-	-	-	-
San Joaquin Valley AR-1000	-	-	-	-	100	95	94	88
Kraton TR-60-8774[a]	-	5	-	-	-	5	-	-
Kraton/Dutrex Blend 10 FBP 1000[b]	-	-	6	12	-	-	6	12
Viscosity at 140°F[c], P	537	6720	1160	gel	423	1720	1040	15,500
Viscosity at 275°F[d], cSt	217	873	493	1350	150	431	305	752
Penetration at 77°F[e], 100g, 5s	186	103	145	111	164	134	154	132
Penetration at 39.2°F, 100g, 5s	17	14	17	21	12	11	12	13
Penetration at 39.2°F, 200g,60s	66	49	61	58	59	43	49	56
Softening point[f], °C	41.4	58.6	47.2	79.0	41.2	52.2	49.6	71.0
Softening point, °F	106.5	137.5	117	174	106	126	121	152
Specific Gravity	1.019	1.014	...	...	1.017	1.015	...	...
Temperature susceptibility[g], 140 to 275°F	3.42	2.44	3.11	-	3.71	3.38	3.46	3.78
PVN[h]	-0.4	1.0	0.6	1.8	-1.2	0.3	-0.1	1.2
P.I.[i] from penetration at 39.2°F and 79°F	-1.4	-0.2	0.7	1.0	-1.9	-1.6	-1.8	-1.2
P.I. from penetration at 77°F and softening point	0.2	2.9	1.2	6.7	-0.4	2.4	2.2	6.2
Penetration ratio[j]	35	48	42	52	36	32	32	42

[a] Blend supplied by Shell Development Co. Kraton TR-60-8774 is 50% Kraton D-1101 S-B-S, 50% Kraton DX-1118 S-B.

[b] Blend 10 FBP 1000 is 50% Kraton D-1101 S-B-S, 50% Dutrex 739 Rubber Extender Oil ASTM D2226, type 101.

[c] AASHTO T202.

[d] AASHTO T201.

[e] AASHTO T49.

[f] AASHTO T53.

[g] Temperature susceptibility = $(\log\log \eta_2 - \log\log \eta_1)/(\log T_2 - \log T_1)$, where η = viscosity in cP, T = absolute temperature.

[h] Determined from penetration at 77°F and viscosity at 275°F (McLeod, 1976).

[i] P.I. = $(20 - 500\alpha)/(1 + 50\alpha)$: $\alpha = [\log(pen_2) - \log(pen_1)]/(T_2 - T_1)$ or $[\log 800 - \log(pen_{25°C})]/T_{SP} - 25)$, where T = temperature, °C.

[j] 100(Pen 39.2°F, 200 g, 60 s)/(Pen 77°F, 100 g, 5 s).

Table A6. Dispersions of polyethylenes in Texaco AC-10 asphalt.

Serial No.	11	49	50	51	52	53
Polyethylene[a]	None	Dow 526	Dow 527	Dow 69065P	Dow 2045	Dow 880
Type	...	LDPE	LDPE	HDPE	LLDPE	HMWLDPE
Density (manufacturer's data)	...	0.919	0.921	0.920	0.961	0.932
Melt Index (manufacturer's data)	...	1.0	2.9	1.0	0.60	0.45
No. of passes through mill[b]	None	5	5	6	5	5
Temperature after 1 pass, °C	...	152	158	162	162	166
Temperature after 2 passes, °C	...	162	163	172	170	169
Temperature after 3 passes, °C	...	164	168	178	173	173
Temperature after 4 passes, °C	...	168	172	184	178	179
Temperature after 5 passes, °C	...	170	172	186	181	183
Temperature after 6 passes, °C	...	...	...	186	...	...
Volume of settled layer of swollen polyethylene[c], %	...	20	19	19	29	20
Viscosity at 140°F[d], P	1080	4740	4640	...	...	...
Penetration[e] at 77°F, 100 g, 5 s	118	60	58	36	38	59
Penetration[e] at 39.2°F, 100 g, 5 s	12	7	8	6	5	9
Penetration[e] at 39.2°F, 200 g, 60 s	41	31	33	21	22	27
Penetration index[f]	-1.1	-0.7	-0.2	+0.5	-0.3	+0.2
Penetration ratio[g]	35	52	57	58	58	46

[a]All blends contained 95% asphalt, 5% polyethylene. Additional data for the polyethylenes was presented in Table 1 of Progress Report 3, February 5, 1985.

[b]Probst and Class Vicosator, Model 60.

[c]Specimen in 3-oz tin kept in 150°C oven 3 h, then chilled. Thickness of layers measured under ultraviolet illumination after stripping off the tin.

[d]ASTM D2171, modified Koppers capillary viscometers. Reliable values were not obtained for the blends containing HDPE, LLDPE, and HMWLDPE.

[e]ASTM D5.

[f]P.I. = $(20 - 500\alpha)/(1 + 50\alpha)$:

$\alpha = [\log(pen_2) - \log(pen_1)] / (T_2 - T_1)$, where T = temperature, °C.

[g]100(pen 39.2°F, 200 g, 60 s) / (pen 77°F, 100 g, 5 s).

Table A7. Replicate dispersions of a low-density polyethylene[a] in asphalts.

Asphalt	Texaco AC-5					Texaco AC-10			San Joaquin Valley AR-1000					San Joaquin Valley AR-2000		
Serial No.	7	86	65	84	97	11	49	82	19	101	64	83	113	25	63	81
Polyethylene[a], %	None	None	5	5	5	None	5	5	None	None	5	5	5	None	5	5
No. of batches prepared in mill[b]	...	...	1	1	10	...	1	6	...	...	1	1	10	...	1	6
Volume of settled layer of swollen polyethylene[c], %	...	...	18	33	21	...	20	8	...	...	29	38	38	...	9-19	4
Viscosity at 140°F[d], P	506	537	1990	2410	2200	1080	4740	3410	498	423	1660	1620	1295	1100	2920	2580
Penetration[e] at 77°F, 100 g, 5 s	194	186	95	104	105	118	60	64	146	164	74	104	98	86	44	54
Penetration[e] at 39.2°F, 100 g, 5 s	20	17	11	16	13	12	7	9	10	12	7	9	8	5	3	4
Penetration[e] at 39.2°F, 200 g, 60 s	63	66	42	55	49	41	31	32	46	59	26	38	41	25	15	19
Penetration index[f]	-1.0	-1.4	-0.7	+0.2	-0.5	-1.1	-0.7	-0.1	-0.2	-1.9	-1.3	-1.5	-1.6	-2.4	-2.1	-1.9
Penetration ratio[g]	32	35	44	53	47	35	52	50	32	36	35	37	42	29	34	35

[a] Blends contained 95% asphalt, 5% Dow LDPE 526 (density 0.919, Melt Index 1.0). Blends No. 65 and 84 were made from base asphalt No. 7; blend 97 from base asphalt No. 86; blends 49 and 82 from base asphalt No. 11.

[b] Probst and Class Vicosator, Model 60.

[c] Specimen in 3-oz tin kept in 150°C oven 3 h, then chilled. Thickness of layers measured under ultraviolet illumination after stripping off the tin.

[d] AASHTO T202, modified Koppers capillary viscometers.

[e] AASHTO T49.

[f] P.I. = $(20 - 500\alpha)/(1 + 50\alpha)$: $\alpha = [\log(pen_2) - \log(pen_1)]/(T_2 - T_1)$, where T = temperature, °C.

[g] 100 (Pen 39.2°F, 200 g, 60 s)/(Pen 77°F, 100 g, 5 s).

Table A8. Dispersions of polyethylene[a] in asphalts.

Asphalt	Texaco AC-5		San Joaquin Valley AR-1000	
Serial No.	86	97	101	113
Polyethylene[a], %	None	5	None	5
No. of batches prepared in mill[b]	...	10	...	10
Volume of settled layer of swollen polyethylene[c], %	...	28	...	38
Viscosity at 140°F[d], P	537	2200	423	1295
Viscosity at 275°F,[e], cSt	217	843	150	399
Penetration[f] at 77°F, 100 g, 5 s	186	105	164	98
Penetration[f] at 39.2°F, 100 g, 5 s	17	13	12	8
Penetration[f] at 39.2°F, 200 g, 60 s	66	49	59	41
Softening point[g], °C	41.4	52.2	41.2	47.2
Softening point[g], °F	106.5	126	106	117
Temperature susceptibility[h], 140 to 275°F	-3.42	-2.98	-3.71	-3.33
PVN[i]	-0.4	1.0	-1.2	-0.2
Penetration index[j] from penetration at 39.2°F and 77°F	-1.4	-0.5	-1.9	-1.6
Penetration index from penetration at 77°F and softening point	0.2	1.5	-0.4	-0.2
Penetration ratio[k]	35	47	36	37
After rolling thin film oven test weight change, %	-0.07	-0.06	-1.08	-0.97
viscosity at 140°F, P	1190	6040	893	3840
viscosity at 275°F, cSt	311	1280	180	532
penetration at 77°F, 100g, 5s	112	65	104	59
% of original	60	62	63	60

[a]Blends contained 95% asphalt, 5% Dow LDPE 526 (density 0.919, Melt Index 1.0).

[b]Probst and Class Vicosator, Model 60.

[c]Specimen in 3-oz tin kept in 150°C oven 3 h, then chilled. Thickness of layers measured under ultraviolet illumination after stripping off the tin.

[d]AASHTO T202, modified Koppers capillary viscometers.

[e]AASHTO T201.

[f]AASHTO T49.

[g]AASHTO T53.

[h]Temperature susceptibility = $(\log \log \eta_2 - \log \log \eta_1)/(\log T_2 - \log T_1)$ where η = viscosity in cP, T = absolute temperature.

[i]Determined from penetration at 77°F and viscosity at 275°F (McLeod, 1976).

[j]P.I. = $(20 - 500\alpha)/(1 + 50\alpha)$:

$$\alpha = [\log (pen_2) - \log (pen_1)]/(T_2 - T_1), \text{ or } [\log 800 - \log (pen_{25°C})]/[T_{SP} - 25] \text{ where } T = \text{temperature, °C.}$$

[k]100(pen 39.2°F, 200 g, 60 s)/(pen 77°F, 100 g, 5 s).

Table A9. Dispersion of ethylene-vinyl acetate copolymers in AC-10 and AR-2000 asphalts.

Serial No.	11	70	72	68	67	25	71	73	69	66
Asphalt	Texaco AC-10					San Joaquin Valley AR-2000				
EVA Resin[a]	None	Elvax 40-W		Elvax 150	Elvax 250	None	Elvax 40-W		Elvax 150	Elvax 250
Proportion asphalt:EVA	...	97:3	95:5	97:3	97:3	...	97:3	95:5	97:3	97:3
Viscosity[b] at 140°F, P	1080	1670	2520	1750	f	1100	1780	2320	1640	1640
Penetration[c] at 77°F, 100 g, 5 s	118	101	92	91	82	86	84	91	89	93
Penetration[c] at 39.2°F, 100 g, 5 s	12	13	12	14	13	5	5	7	5	5
Penetration[c] at 39.2°F, 200 g, 60 s	41	44	43	49	42	25	29	30	30	23
Penetration index[d]	-1.1	-0.4	-0.3	0.0	+0.3	-2.4	-2.3	-1.8	-2.5	-2.5
Penetration ratio[e]	35	44	47	54	51	29	34	33	34	25

[a]EVA pellets added to asphalt preheated to 275°F (135°C) in Waring Blendor. Typical properties of resins from manufacturer (DuPont Company):)

Elvax 40-W, 39-42% vinyl acetate, softening point 220°F, (104°C), specific gravity 0.965.
Elvax 150, 32-34% vinyl acetate, softening point 230°F (110°C), specific gravity 0.957.
Elvax 250, 27-29% vinyl acetate, softening point 260°F (127°C), specific gravity 0.951.

[b]AASHTO T202.

[c]AASHTO T49.

[d]P.I. = $(20 - 500\alpha)/(1 + 50\alpha)$:

$\alpha = [\log(pen_2) - \log(pen_1)]/(T_2 - T_1)$, where T = temperature, °C.

[e]100(Pen 39.2°F, 200 g, 60 s)/(Pen 77°F, 100 g, 5 s).

[f]Serial No. 67 blend contained undispersed resin and could not be tested in the capillary viscometer.

Table A10. Blends of ethylene-vinyl acetate copolymers in AC-5 and AR-1000 asphalts.

Serial No.	86	120	121	124	125	101	116	117	118	119
Asphalt	Texaco AC-5					San Joaquin Valley AR-1000				
EVA Resin[a]	None	EX 042		Elvax 150		None	EX 042		Elvax 150	
Proportion asphalt:EVA	...	97:3	95:5	97:3	95:5	...	97:3	95:5	97:3	95:5
Viscosity[b] at 140°F, P	537	634	742	785	1160	423	433	419	852	1180
Viscosity[c] at 275°F, cSt	217	278	368	380	618	150	170	264	281	434
Penetration[d] at 77°F, 100 g, 5 s	186	133	113	202	176	164	176	132	155	161
Penetration[d] at 39.2°F, 100 g, 5 s	17	16	15	20	17	12	10	10	11	12
Penetration[d] at 39.2°F, 200 g, 60 s	66	49	47	63	54	59	40	38	45	50
Softening point[e], °C	41.4	44.8	53.4	42.0	49.0	41.2	42.2	48.0	44.0	44.8
Softening point[e], °F	106.5	112.5	128	107.5	120	106	108	118.5	111	112.5
Temperature suscepti-bility[f],	3.42	3.33	3.16	3.16	2.94	3.71	3.60	3.19	3.45	3.22
PVN[g]	-0.4	-0.5	-0.2	-0.6	1.3	-1.2	-0.9	-0.6	-0.2	0.5
Penetration index[h] from penetration at 39.2°F and 77°F	-1.4	-0.6	-0.3	-1.2	1.2	-1.9	-2.4	-1.8	-2.0	-1.9
Penetration index[h] from penetration at 77°F and softening point	0.2	0.1	2.0	0.9	2.6	-0.4	0.3	1.1	0.4	0.8
Penetration ratio[i]	35	37	42	31	31	36	23	29	29	31

[a]EVA pellets added to asphalt while stirring with Jiffy mixer at 200 rpm. EX 042 dissolved at 325°F(163°C). Elvax 150 dissolved at 347-356°F(175-180°C). Typical properties of resins from manufacturers:
Exxon EX 042 softening point 230°F, specific gravity 0.92.
DuPont Elvax 150, 32-34% vinyl acetate, softening point 230°F (110°C), specific gravity 0.957.

[b]AASHTO T202.

[c]AASHTO T201.

[d]AASHTO T49.

[e]AASHTO T53.

[f]Temperature susceptibility = $(\log \log \eta_2 - \log \log \eta_1)/(\log T_2 - \log T_1)$ where η = viscosity in cP, T = absolute temperature.

[g]Determined from penetration at 77°F and viscosity at 275°F (McLeod, 1976).

[h]$P.I. = (20 - 500\alpha)/(1 + 50\alpha)$: $\alpha = [\log (pen_2)]/(T_2 - T_1)$, or $[\log 800 - \log(pen\ 25°C)]/T_{SP} - 25)$, where T = temperature, °C.

[i]100(Pen 39.2°F, 200 g, 60 s)/(Pen 77°F, 100 g, 5 s).

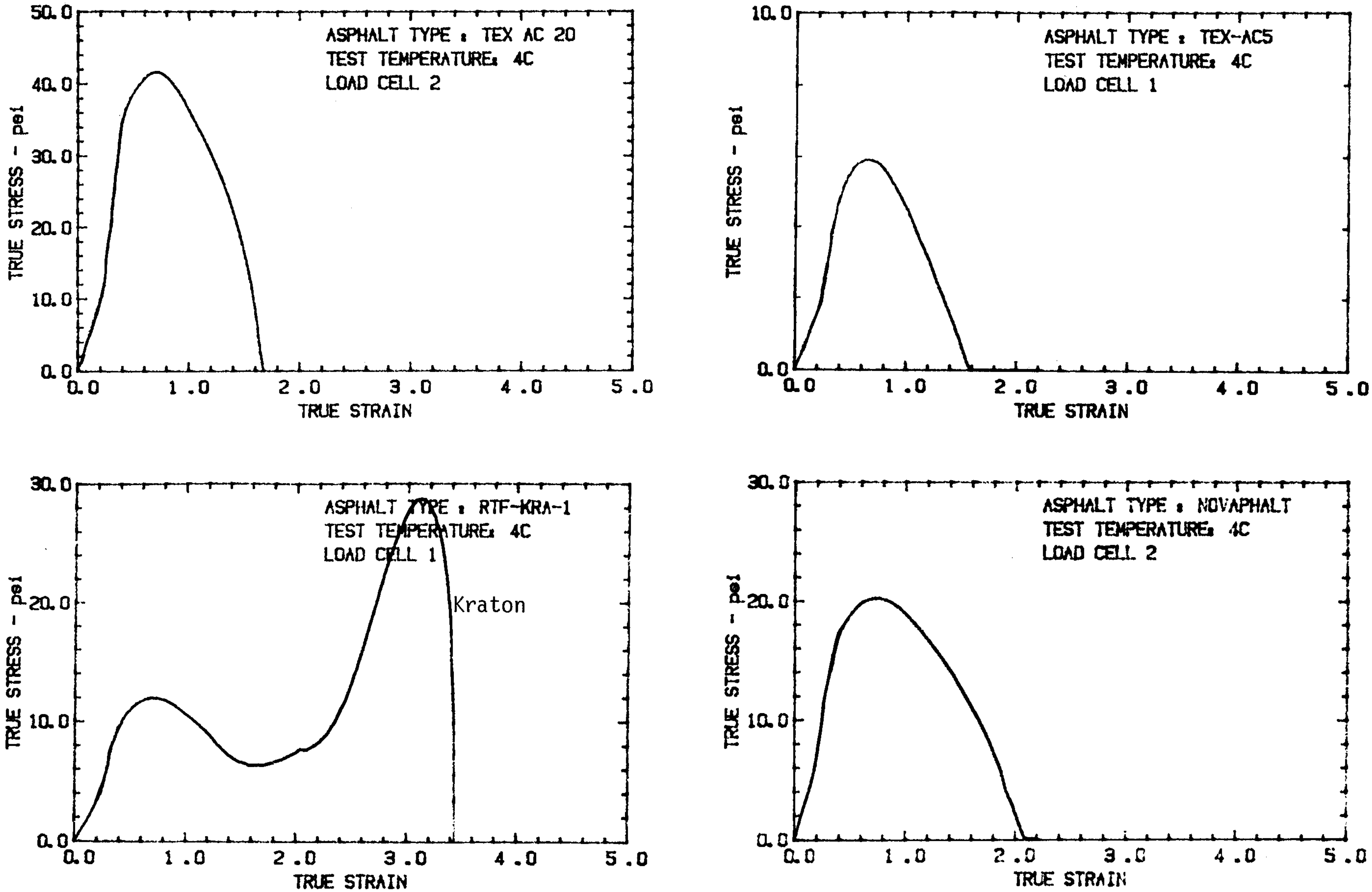
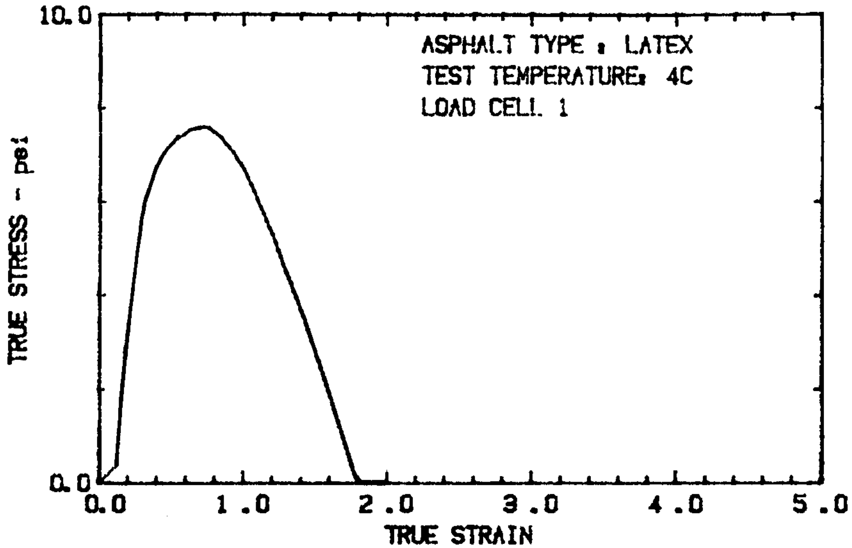
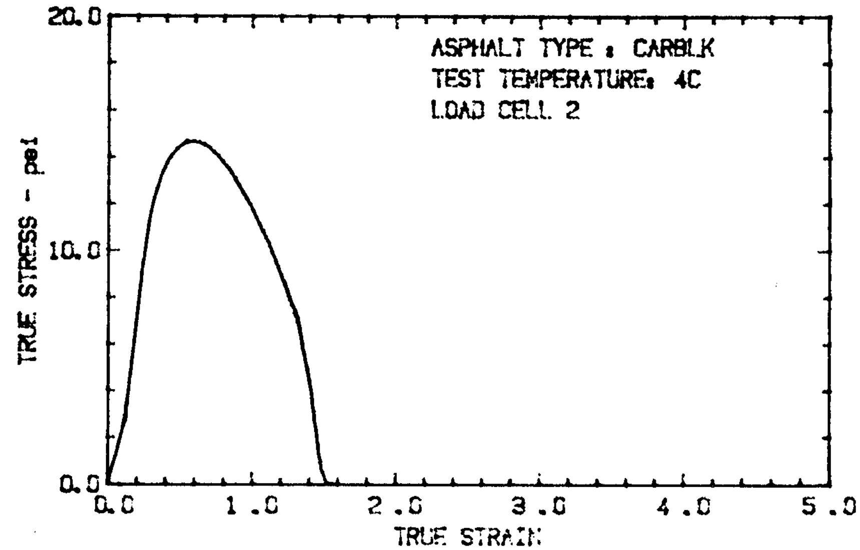


Figure A1. Typical stress-strain curves from force ductility tests at 39.2°F and $5^{cm}/min$ for unmodified and modified Texaco asphalts.





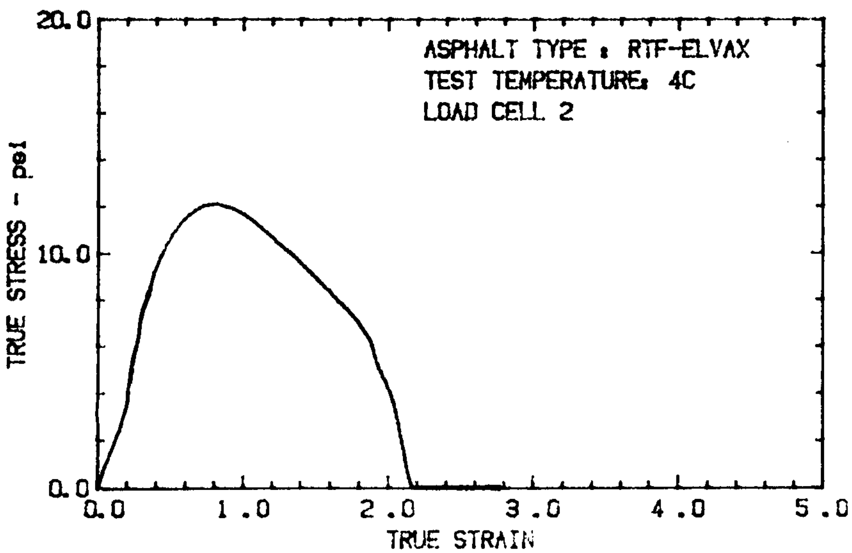


Figure A1. Continued

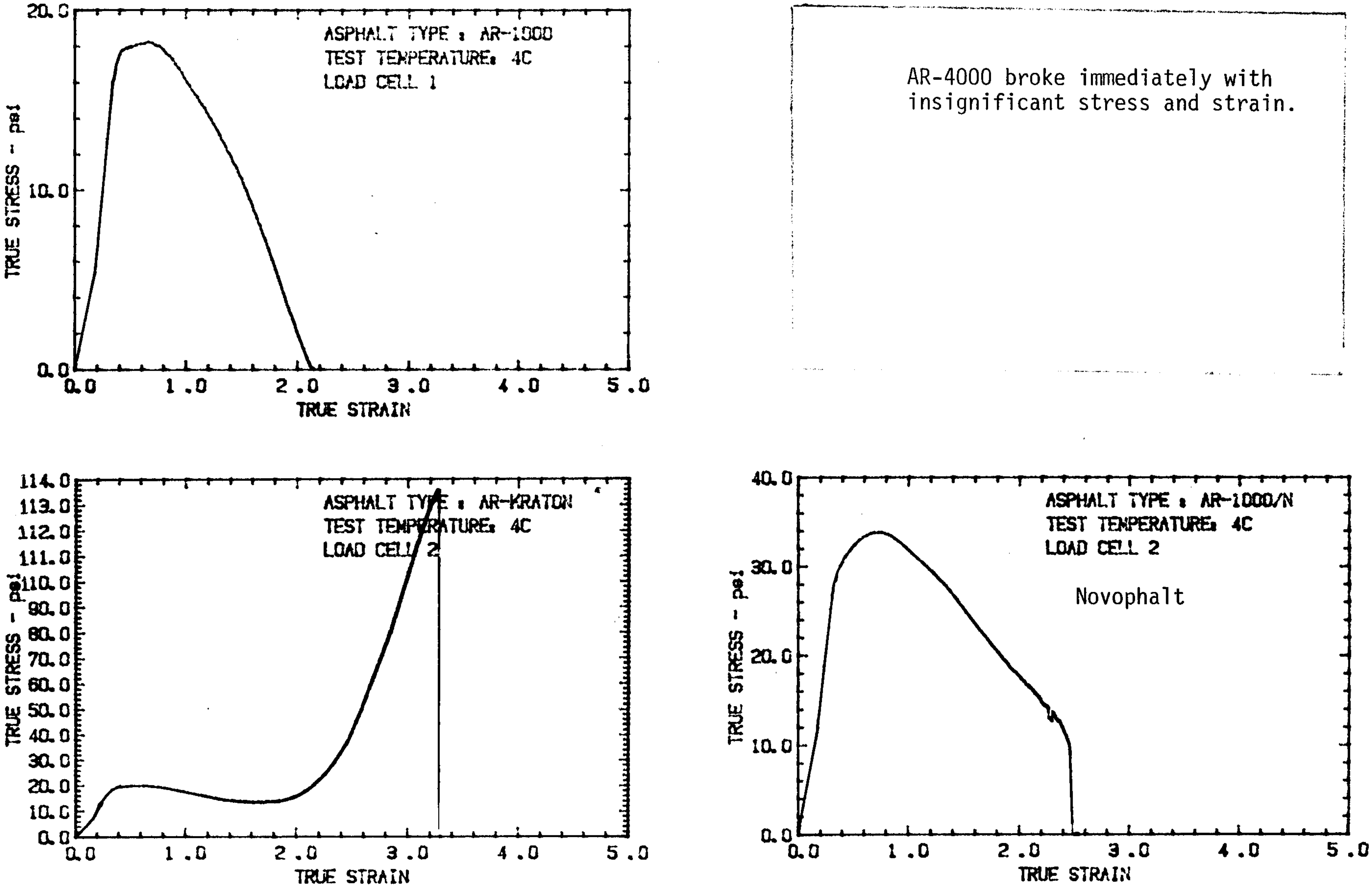


Figure A2. Typical stress-strain curves from force ductility tests at 39.2°F and 5 $^{cm}/min$ for unmodified and modified San Joaquin Valley asphalts.

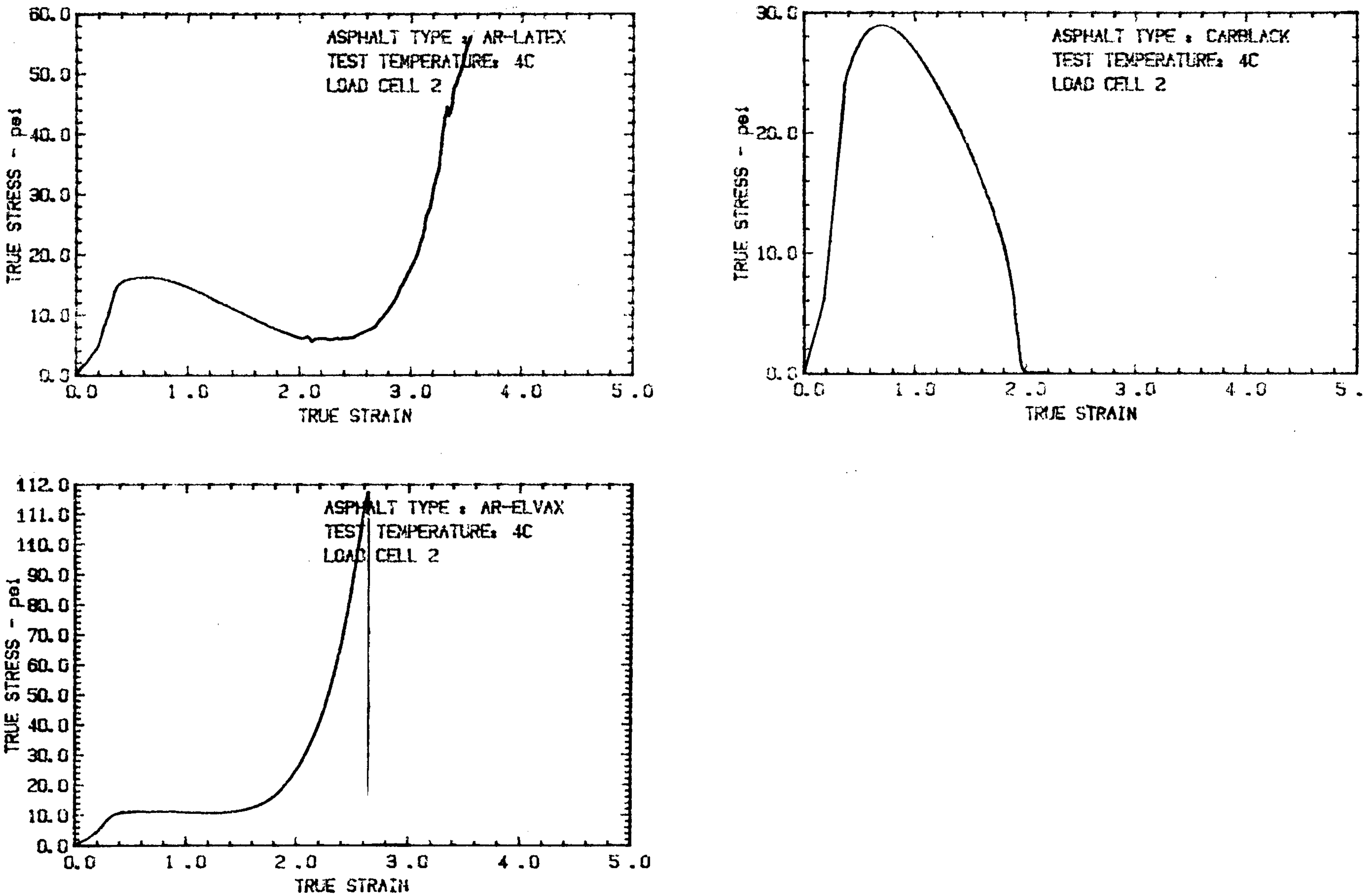


Figure A2. Continued

Table A11. Change in Penetration of Binders After Exposure to 325°F for 24 hours in a Covered Penetration Tin.

Binder	Penetration before Exposure	Penetration after Exposure	Change in Penetration, Percent	Change in Appearance
AC-5	186	159	-15	None
AC-10	118	100	-15	None
AC-20	75	65	-13	None
AC-5 + Carbon Black	152	99	-35	None[a]
AC-5 + EVA	176	138	-21	None
AC-5 + Poyethylene	105	111	+ 6	Crazed crust[b]
AC-5 + SBR	121	145	+20	Lumpy
AC-5 + SBS	103	110	+ 7	None

[a]Carbon black separated and settled to bottom of tin.

[b]Poyethylene separated and rose to top surface of asphalt.

APPENDIX B

Aggregate Properties and Mixture Design Details

Table B1. Individual aggregate gradations for washed pea gravel, washed sand, field sand, and limestone crusher fines.

	Washed Pea Gravel	Washed Sand	Field Sand	Limestone Crusher Fines
Sieve Size	Percent retained	Percent retained	Percent retained	Percent retained
#4	65.6	0.3	1.4	0.1
#8	31.6	13.1	1.1	6.2
#16	1.6	17.7	1.0	18.4
#30	0.4	18.4	0.4	16.1
#50	0	35.4	1.1	11.7
#100	0	11.9	44.8	10.4
#200	0	0.7	28.5	7.1
-#200	0.8	2.5	21.7	30.0
Percentage of each aggregate used in blend	50%	30%	10%	10%

Table B2. Bulk specific gravity, apparent specific gravity, and percent absorption for the pea gravel and combined fines.

	Pea Gravel	Pea Gravel	Combined Fines (washed sand, field sand, limestone fines)
Bulk Specific Gravity	2.575	2.529	2.584
Apparent (maximum) specific gravity	2.658	2.640	2.642
Absorption, percent	1.22	1.68	0.86

Table B3. Mixing and molding temperatures.

	Mixing Temperature (°F)	Molding Temperature (°F)
AC-20	305	275
AC-5	285	266
AC-5 + 15% Carbon Black	341	317
AC-5 + 5% Latex	340	320
AC-5 + 5% Kraton D	340	310
AC-5 + 5% Elvax 150	335	290
AC-5 + 5% Novophalt	345	290

Table B4. Results of Marshall mix designs.

	Optimum Asphalt Content, percent	Air Void Content, percent	Marshall Stability, lbs	Marshall Flow	Voids in Mineral Aggregate, percent
River Gravel and Texaco Asphalt (50 Blow)					
AC-20 (Control)	4.5	3.2	1700	6	12
5% Latex + AC-5	5.0(4.5)*	4.9	1200	6	13
5% Kraton + AC-5	4.5	4.7	1500	6	13
5% Novophalt + AC-5	4.6	5.0	1500	6	13
5% Elvax + AC-5	4.5	4.7	1400	5	13
15% Car. Black + AC-5	4.75	5.0	1400	6	13
AC-10	4.6	4.9	1300	6	13
AC-5	4.6	4.8	900	8	13
River Gravel and San Joaquin Valley Asphalt (50 Blow)					
AR4000 (Control)	4.6	5.0	1200	7	12
5% Latex + AR1000	4.5	5.7	800	7	12
5% Kraton + AR1000	4.5	5.0	900	6	13
5% Novophalt + AR1000	4.5	4.7	1100	6	12
5% Elvax + 5% AR1000	4.5	5.2	700	7	13
15% Car. Black + AR1000	4.7	7.1	1000	6	14
AR2000	4.5	5.4	1000	6	13
AR1000	4.5	4.5	700	6	12
Crushed Limestone and Texaco Asphalt (75 Blow)					
AC-20 (Control)	4.5	4.0	3100	8	-
5% Latex + AC-5	4.5	4.0	2600	8	-
5% Kraton + AC-5	4.5	4.0	2900	10	-
5% Novophalt + AC-5	-	-	-	-	-
5% Elvax + AC-5	4.5	3.0	2400	8	-
15% Car. Black + AC-5	4.7	4.4	3000	9	-

*Later changed to 4.5 percent and used in subsequent tests.
(After Reference 1)

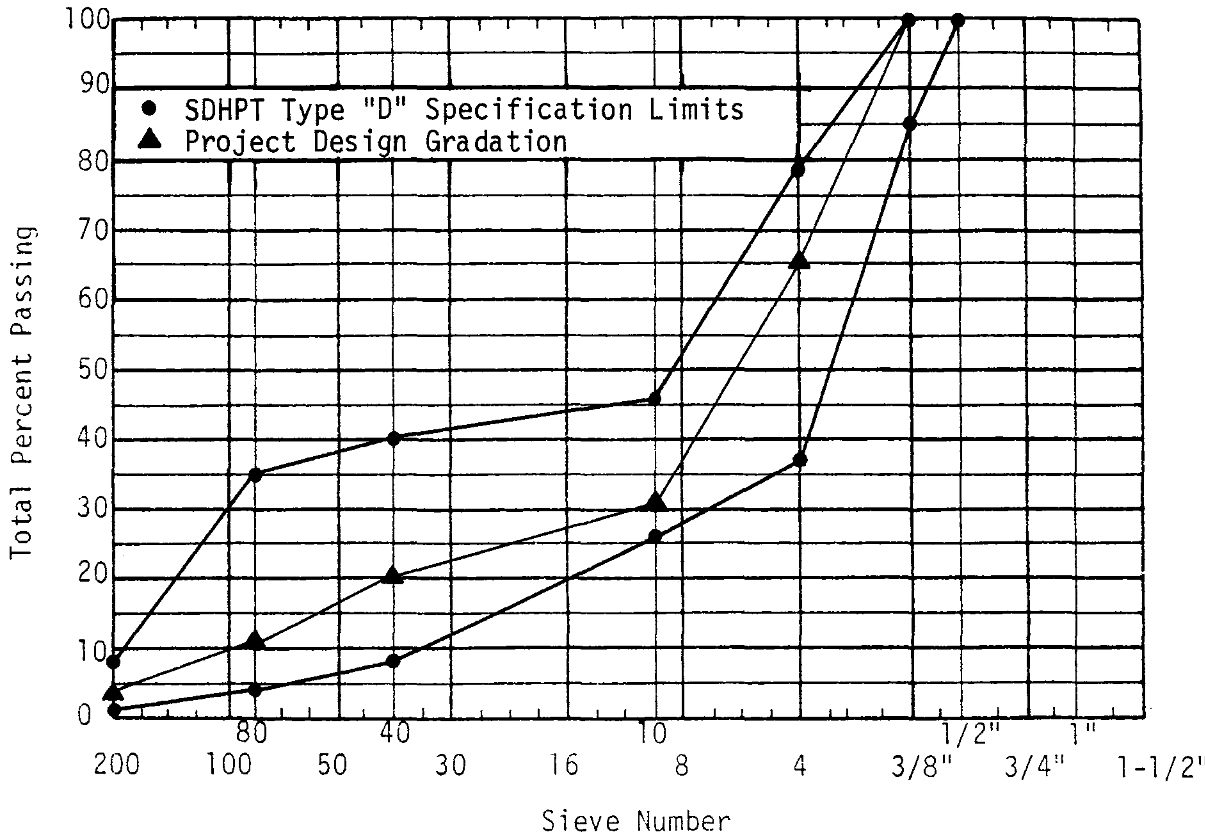


Figure B1. Design gradation specification limits for pea gravel aggregate.

APPENDIX C

Mixture Properties and Binder Extraction Data

Table C1. Tensile properties at 77°F of mixtures made using Texaco asphalt and river gravel.

Type Mixture	Tensile Properties @ 0.02 in/min.			Tensile Properties @ 0.2 in/min.			Tensile Properties @ 2.0 in/min.		
	Tensile Strength, psi	Strain @ Failure, in/in	Secant Modulus, psi	Tensile Strength, psi	Strain @ Failure, in/in	Secant Modulus, psi	Tensile Strength, psi	Strain @ Failure, in/in	Secant Modulus, psi
Control: AC-20	45	0.0028	16,000	83	0.0030	27,700	121	0.0036	34,000
Control: AC-5	16	0.0023	7,100	28	0.0029	9,600	63	0.0031	20,100
AC-5 + 15% Microfil 8	15	0.0030	5,100	33	0.0028	11,900	64	0.0040	17,000
AC-5 + 5% Elvax 150	22	0.0023	9,800	48	0.0023	21,100	87	0.0024	35,300
AC-5 + 5% Kraton D	27	0.0028	9,700	54	0.0025	21,300	112	0.0025	45,700
AC-5 + 5% Latex	15	0.0032	4,900	31	0.0030	10,300	74	0.0031	24,100
AC-5 + 5% Novophalt	28	0.0024	12,100	58	0.0022	26,600	119	0.0025	48,600

Table C2. Tensile properties at 33°F of mixtures made using Texaco asphalt and river gravel.

Type Mixture	Tensile Properties @ 0.02 in/min.			Tensile Properties @ 0.2 in/min.			Tensile Properties @ 2.0 in/min.		
	Tensile Strength, psi	Strain @ Failure, in/in	Secant Modulus, psi	Tensile Strength, psi	Strain @ Failure, in/in	Secant Modulus, psi	Tensile Strength psi	Strain @ Failure, in/in	Secant Modulus, psi
Control: AC-20	211	0.00112	194,000	342	0.00040	866,000	369	0.00039	996,000
Control: AC-5	128	0.00178	72,000	244	0.00156	157,000	376	0.00165	265,000
AC-5 + 15% Microfil 8	132	0.00163	82,000	217	0.00192	114,000	360	0.00139	267,000
AC-5 + 5% Elvax 150	119	0.00138	87,000	241	0.00160	155,000	444	0.00130	349,000
AC-5 + 5% Kraton D	136	0.00118	117,000	300	0.00118	253,000	428	0.00077	569,000
AC-5 + 5% Latex	121	0.00152	80,000	239	0.00182	189,000	399	0.00166	242,000
AC-5 + 5% Novophalt	167	0.00138	121,000	329	0.00118	278,000	436	0.00040	1,133,000

Table C3. Tensile properties at -10 or -18°F of mixtures made using Texaco asphalt and river gravel.

Type Mixture	Tensile Properties @ 0.02			Tensile Properties @ 0.2			Tensile Properties @ 2.0		
	Tensile Strength, psi	Strain @ Failure, in/in	Secant Modulus, psi	Tensile Strength, psi	Strain @ Failure, in/in	Secant Modulus, psi	Tensile Strength, psi	Strain @ Failure, in/in	Secant Modulus, psi
Control: AC-20	(2)413	0.00013	$3.18X10^6$	(1)395	*	*	(2)374	0.00008	$4.68X10^6$
Control: AC-5	(2)327	0.00024	$1.50X10^6$	(1)440	0.00005	$9.17X10^6$	(2)522	0.00012	$4.20X10^6$
AC-5 + 15% Microfil 8	(2)319	0.00053	$0.61X10^6$	(1)424	0.00005	$9.42X10^6$	(2)450	*	*
AC-5 + 5% Elvax 150	(1)381	0.00018	$2.29X10^6$	(1)512	0.00007	$7.52X10^6$	(1)425	0.00006	$9.81X10^6$
AC-5 + 5% Kraton D	(1)404	0.00017	$2.43X10^6$	(1)472	0.00008	$5.90X10^6$	(1)502	0.00011	$4.54X10^6$
AC-5 + 5% Latex	(1)348	0.00025	$1.38X10^6$	(1)352	0.00010	$3.70X10^6$	(2)437	0.00016	$5.60X10^6$
AC-5 + 5% Novophalt	(1)393	0.00010	$4.16X10^6$	(1)444	0.00003	$11.59X10^6$	(1)387	0.00004	$8.98X10^6$

Note: * - Difficult to accurately measure due to very small strain.

(1) - Tensile test performed at -10°F.

(2) - Tensile test performed at -18°F.

Table C4. Properties of mixtures before and after exposure to moisture (Texaco asphalt and river gravel).

Type Mixture	Before Treatment					After Treatment					Resilient Modulus Ratio	Tensile Strength Ratio
	Air Void Content, Percent	Resilient Modulus @ 77°F psiX10^3	Tensile Properties*			Air Void Content, Percent	Resilient Modulus @ 77°F psiX10^3	Tensile Properties*				
			Tensile Strength, psi	Strain @ Failure,	Secant Modulus, psi			Tensile Strength, psi	Strain @ Failure	Secant Modulus, psi		
Control: AC-20	7.4	410	130	0.0032	42,000	7.4	220	110	0.0047	23,000	0.55	0.80
Control: AC-5	5.9	80	50	0.0034	14,000	5.4	100	70	0.0052	13,000	1.30	1.48
AC-5 + 15% Micro-fil 8	7.0	70	60	0.0040	15,000	7.3	60	50	0.0047	11,000	0.88	0.88
AC-5 + 5% Elvax 150	7.6	190	60	0.0024	28,000	7.0	70	70	0.0037	20,000	0.89	1.09
AC-5 + 5% Kraton D	6.4	270	80	0.0024	35,000	6.1	210	80	0.0031	27,000	0.80	1.00
AC-5 + 5% Latex	5.8	140	70	0.0037	19,000	5.8	100	70	0.0050	15,000	0.74	1.01
AC-5 + 5% Novophalt	6.3	320	90	0.0022	42,000	6.0	230	100	0.0031	33,000	0.73	1.07

*Tensile tests at 2 in/min and 77°F.

Table C5. Properties of mixtures after exposure to moisture (San Joaquin Valley asphalt and river gravel).

Type Mixture	After Treatment					Resilient Modulus Ratio	Tensile Strength Ratio
			Tensile Properties*				
	Air Void Content, percent	Resilient Modulus @ 77°F, psiX10^3	Tensile Strength, psi	Strain @ Failure,	Secant Modulus, psi		
Control: AR-4000	3.8	500	170	0.0031	58,000	0.70	0.66
Control: AR-1000	3.4	100	60	0.0044	14,000	0.73	0.78
AR-1000+ 5% Latex	4.1	130	70	0.0047	16,000	0.60	0.76
AR-1000 + 15% Micro-fil 8	4.2	210	90	0.0040	25,000	0.81	0.73
AR-1000+5% Kraton D	4.4	150	90	0.0055	16,000	0.72	0.84
AR-1000 + 5% Novo-phalt	4.6	180	110	0.0039	28,000	0.59	0.82
AR-1000 + 5% Elvax 150	4.6	60	80	0.0072	11,000	0.47	0.77

*Tensile tests performed at 2 in/min and 77°F.

Table C6. Extraction of asphalt concrete specimens containing Texaco asphalts.

Binder	Design Binder Content, %	Extraction Method[a]	Extraction Solvent[b]	Extracted Aggregate, %	Recovered Binder, %	Unaccounted Loss, %	Tests on Recovered Binder: Penetration at 77°F, 100g, 5s	Viscosity at 140°F, P	Viscosity at 275°F, cSt	Comments
Texaco AC-5, no additive	4.6	B	Benzene	95.7	4.3	0	51	930	350	Compare to values of 112 pen., 1190 P at 140°F and 311 cSt at 275°F for AC-5 after Rolling Thin Film Oven Test.
	4.6	B	TCE	95.5	4.5	0	100	1540	336	
Texaco AC-20, no additive	4.5	B	Benzene	95.9	3.8	0.3	32	9300	735	...
	4.5	B	TCE	95.8	4.2	0	28	13,200	841	...
Texaco AC-5 + 5% Microfil 8	4.75	B	Benzene	95.7	4.3	0	92	1710	433	Filter paper stained, but most of carbon black was in the recovered asphalt.
	4.75	B	TCE	95.7	4.3	0	106	1480	364	Filter paper stained, (more than B-7 above) but most of carbon black was in the recovered asphalt.
Texaco AC-5+ 5% Ultrapave Latex	5.0	B	Benzene	96.2	3.8	0	132	1120	439	Extraction very slow due to slow draining through filter. Aggregate obviously still still contained rubber.Recovered asphalt was not tested because extracted was overheated during long extraction time.
	5.0	A	TCE	95.4	4.6	0.0	...	...	...	Aggregate contained trace of rubber, much less than C-10 and C-18. Recovered asphalt was not tested because of long elapsed time between start of extraction and completion of recovery.
Texaco AC-5+ 5% Ultrapave Latex	5.0	A	TCE	95.4	4.6	0	70	6400	960	Aggregate contained trace of rubber.
	5.0	B	TCE	95.6	4.4	0	80	3710	830	Extraction very slow due to slow draining through filter. Aggregate contained rubber and could be lifted from filter as a single, loosely-bound conglomerate.
Texaco AC-5+ 5% Kraton S-B-S	4.5	B	TCE	95.8	4.2	-	61	18,100	1180	Extraction was rapid; aggregate did not appear to contain rubber.
	4.5	B	Benzene	95.9	4.1	0	60	19,200	1180	Extraction was rapid; aggregate did not appear to contain rubber.
Texaco AC-5+ 5% LDPE 526	4.6	B	TCE	95.6	4.4	0	57	8980	1130	Silt separated from extract by centrifuging appeared to contain some polyethylene.
	4.6	N	Benzene	95.8	4.1	0.1	53	6860	761	Silt separated from extract by centrifuging contained some polyethylene.
Texaco AC-5+ 5% Elvax 150	4.5	B	TCE	95.5	4.5	0	71	3090	1100	Extraction very slow due to slow draining through filter.
	4.5	B	Benzene	95.8	4.1	0.1	69	3000	1060	Extraction very slow due to slow draining through filter.

[a] AASHTO T164. Method A used centrifugal "Rotarex" extraction; extraction is carried out at room temperature. In Method B, extraction is carried out at the temperature of boiling solvent. Extracts from both methods were centrifuged to remove silt which passed the primary filters, then distilled to recover the binder by AASHTO T170. Because volume of recovered binder was small, a round-bottom flask was substituted for the flat-bottom flask, to obtain better stripping of solvent by the CO_2 inlet tube.

[b] Reagent-grade benzene or reagent-grade tricholroethylene, as indicated.

(After Reference 1)

Table C7. Extraction of asphalt concrete specimens containing San Joaquin Valley asphalts.

Binder	Design binder content, %	Extraction method[a]	Extraction solvent[b]	Extracted aggregate, %	Recovered binder, %	Unaccounted loss, %	Tests on recovered binder: Penetration at 77°F, 100 g, 5 s	Viscosity at 140°F, P	Viscosity at 275°F, cSt	Comments
AR-4000	4.6	B	Benzene	95.7	4.2	0.1	27	5380	387	...
85% AR-1000 15% Microfil 8	4.7	B	Benzene	96.2	3.8	0	150	644	194	No undispersed carbon black pellets were detected, but some carbon black was retained in the filters and some was removed from the asphalt solution by centrifuging.
95% AR-1000 5% SBR from Dow XUS 40052.00	4.5	B	TCE	96.0	4.0	0	62	16,960	6130	Extraction was very slow. A few pieces of aggregate were bound together by unextracted rubber. Recovered binder was very "rubbery".
		A	TCE	95.7	4.3	0	69	4520	946	No evidence of rubber in the extracted aggregate, but recovered binder did not seem very "rubbery".
95% AR-1000 5% Kraton S-B-S	4.5	B	Benzene	95.8	4.2	0	112	27,640	815	Recovered binder was almost a gel. No evidence of rubber in extracted aggregate.
95% AR-1000 5% LDPE 526	4.5	B	TCE	96.0	4.0	0	52	3880	371	Extraction was very slow. Some of the polyethylene remained in the extracted aggregate.
		A	TCE	95.8	4.1	0.1	47	4050	342	Much of the polyethylene remained in the extracted aggregate.
95% AR-1000 5% Elvax 150	4.5	B	Benzene	95.7	4.3	0	131	1410	458	Recovered binder was not very "rubbery", but viscosity at 275°F was relatively high. No obvious signs of EVA in extracted aggregate.

[a] AASHTO T164. Method A uses centrifugal "Rotarex" extraction; extraction is carried out at room temperature. In Method B, extraction is carried out at the temperature of boiling solvent. Extracts from both methods were centrifuged to remove silt which passed the primary filters, then distilled to recover the binder by AASHTO T170.

[b] Reagent-grade benzene or reagent-grade trichlorethylene, as indicated.

(After Reference 1)

APPENDIX D

Fatigue Testing Data

Table D1. Adjusted mixing and compaction temperatures for different binders.

Type of Binder	From Visc.-Temp. Data		Adjusted	
	Mixing Temp.	Compaction Temp.	Mixing Temp.	Compaction Temp.
AC-20	307	287	307	275
Latex+AC-5	405	374	340	290
Carbon Black + AC-5	339	315	339	315
Kraton+AC-5	339	316	339	290
Novophalt+AC-5	345	322	345	290
Elvax+AC-5	335	311	335	290
AR-4000	290	271	290	271
Latex+AR-1000	414	385	315	290
Carbon Black + AR-1000	310	289	310	289
Kraton+AR-1000	316	291	316	291
Novophalt +AR-1000	313	289	313	289
Elvax+AR-1000	316	292	316	292

°C = (°F - 32)/1.8

(After Reference 1)

Table D2. Summary of K_1 and K_2 values for beam fatigue testing at 34°F and 68°F (normal curing).

Temp.	Binder	K_1	K_2	R^2 of Regression
34°F	AC-20	1.28×10^{-12}	3.77	0.70
	AC-10*	7.26×10^{-11}	3.90	0.80
	AC-5 + Carbon Black	2.56×10^{-17}	5.78	0.94
	AC-5 + EVA	1.92×10^{-10}	3.99	0.74
	AC-5 + SBS	9.76×10^{-13}	4.73	0.91
	AC-5 + Latex	7.84×10^{-11}	4.16	0.96
	AC-5 + Polyethylene (Novophalt)	7.18×10^{-17}	5.91	0.91
68°	AC-20	4.70×10^{-6}	2.63	0.89
	AC-10*	8.00×10^{-9}	3.74	0.72
	AC-5 + Carbon Black	2.63×10^{-6}	2.84	0.88
	AC-5 + EVA	1.28×10^{-7}	3.91	0.94
	AC-5 + SBS	1.64×10^{-5}	3.12	0.68
	AC-5 + Latex	3.63×10^{-5}	3.04	0.78
	AC-5 + Polyethylene (Novophalt)	2.33×10^{-8}	3.38	0.85

*Mixture of AC-10 and crushed limestone.

°C = (°F - 32)/1.8
(After Reference 1)

Table D3. Basic flexural beam fatigue data for Texaco AC-20.

Temperature, ^oF	Sample No.	Stress Level, psi	200th Cycle Bending Strain	Cycles to Failure	
				Predicted	Actual
34	1	440	0.000293	0	1
	2	440	0.000310	0	1
	3	440	0.000275	0	1
	4	200	0.000200	113	200
	5	200	0.000236	60	75
	6	200	0.000133	520	596
	7	150	0.000100	1539	1000
	8	150	0.000126	645	900
	9	150	0.000150	330	460
	10	100	0.000067	7100	9000
	11	100	0.000091	2200	3000
68	1	83	.000097	171662	455331
	2	138	.000081	277438	136201
	3	138	.000205	24000	25375
	4	138	.000205	24000	27275
	5	192	.000347	6012	9660
	6	192	.000222	19471	7945
	7	192	.000243	15379	17985
	8	192	.000208	23078	17286
	9	83	.000189	29802	44137
	10	248	.000292	9516	7151

(After Reference 1)

Table D4. Basic flexural beam fatigue data for Texaco AC-5 and carbon black.

Temperature, °F	Sample No.	Stress Level, psi	200th Cycle Bending Strain	Cycles to Failure	
				Predicted	Actual
34	1	186	.000274	9970	49280
	2	371	.000203	56525	30605
	3	275	.000213	42632	77109
	4	231	.000152	298257	130113
	5	341	.000305	5422	1408
	6	396	.000485	367	434
	7	429	.000460	502	357
	8	231	.000183	103943	256854
	9	220	.000162	205371	180527
68	1	110	.000450	8209	6389
	2	110	.000417	10174	6112
	3	77	.000306	24529	29404
	4	77	.000389	12374	16808
	5	82	.000194	88420	44368
	6	66	.000222	60545	75302
	7	110	.000928	1053	1580
	8	66	.000183	103987	438758
	9	138	.000250	43346	14407

(After Reference 1)

Table D5. Basic flexural beam fatigue data for Texaco AC-5 and SBR (latex).

Temperature, °F	Sample No.	Stress Level, psi	200th Cycle Bending Strain	Cycles to Failure	
				Predicted	Actual
34	1	371	.000284	43982	15461
	2	297	.000389	11944	37181
	3	275	.000315	28800	22105
	4	192	.000132	1070087	729035
	5	231	.000162	451121	931260
	6	418	.000485	4759	8026
	7	462	.000586	2179	1091
	8	440	.000799	600	601
68	1	104	.001226	24988	236846
	2	104	.000716	127809	151147
	3	104	.000690	142727	143999
	4	148	.001332	19401	26507
	5	148	.000818	85218	58215
	6	136	.000818	85218	42122
	7	297	.001279	21960	7868
	8	252	.003944	720	744
	9	260	.002132	4658	2411

(After Reference 1)

Table D6 . Basic flexural beam fatigue data for Texaco AC-5 and Novophalt (polyethylene).

Temperature, °F	Sample No.	Stress Level, psi	200th Cycle Bending Strain	Cycles to Failure Predicted	Actual
34	1	371	.000284	63954	221858
	2	371	.000254	124934	198480
	3	440	.000358	16512	3044
	4	440	.000460	3740	2544
	5	451	.000460	3740	4313
	6	418	.000358	16512	27912
	7	396	.000409	7501	12222
	8	319	.000254	124934	48005
	9	484	.000690	341	404
68	1	138	.000361	9771	8457
	2	138	.000278	23699	10560
	3	138	.000486	3581	5780
	4	77	.000200	72040	123475
	5	77	.000208	62610	112092
	6	165	.000278	23699	19209
	7	192	.000647	1371	1980
	8	192	.000354	10434	4665

(After Reference 1)

Table D7. Basic flexural beam fatigue data for Texaco AC-5 and EVA (Elvax).

Temperature, °F	Sample No.	Stress Level, psi	200th Cycle Bending Strain	Cycles to Failure	
				Predicted	Actual
34	1	371	.000177	183296	6282
	2	371	.000305	21285	93113
	3	371	.000355	11499	30581
	4	445	.000274	32423	22549
	5	312	.000208	97035	303907
	6	342	.000264	37743	279921
	7	440	.000590	1519	375
	8	440	.000647	1056	620
	9	440	.000534	2265	2425
68	1	148	.001599	11313	9316
	2	148	.001812	6930	12682
	3	148	.001972	4977	7664
	4	119	.001023	65021	110964
	5	116	.000869	122847	75680
	6	104	.000818	155754	128497
	7	186	.002132	3668	3804
	8	204	.002452	2122	1794
	9	204	.002665	1531	868

(After Reference 1)

Table D8. Basic flexural beam fatigue data for Texaco AC-5 and SBS (Kraton).

Temperature, ^oF	Sample No.	Stress Level, psi	200th Cycle Bending Strain	Cycles to Failure	
				Predicted	Actual
34	1	371	.000244	116057	83211
	2	371	.000233	141918	16563
	3	371	.000294	47446	22399
	4	371	.000305	40421	56156
	5	400	.000274	66511	10800
	6	297	.000347	21757	368643
	7	297	.000264	79607	465646
	8	342	.000319	32267	373650
	9	440	.000562	2239	2781
	10	440	.000731	648	254
	11	385	.000393	12083	2426
68	1	186	.001865	5443	4894
	2	186	.001439	12237	4074
	3	186	.002398	2484	3440
	4	104	.000767	87199	36163
	5	104	.000946	45308	209287
	6	148	.001492	10923	10249
	7	148	.001151	24591	28043
	8	148	.001279	17675	20796

(After Reference 1)

Table D9. Summary of fatigue parameters K_1 and K_2 computed from unaged and aged (accelerated aging at 140°F) samples.

Specimen History	Binder Identification	K_1	K_2	R^2 of Regression
No Accelerated Aging - Tested at 68°F.	AC-20	4.70×10^{-6}	2.63	0.89
	AC-10*	8.00×10^{-9}	3.74	0.72
	AC-5 + Carbon Black	2.63×10^{-6}	2.84	0.88
	AC-5 + EVA	1.28×10^{-7}	3.91	0.94
	AC-5 + SBS	1.64×10^{-5}	3.12	0.68
	AC-5 + Latex	3.63×10^{-5}	3.04	0.78
	AC-5 + Polyethylene (Novophalt)	2.33×10^{-8}	3.38	0.85
Accelerated Aging at 140°F for 14 Days - Tested at 68°F	AC-20	1.57×10^{-9}	3.94	0.97
	AC-5 + Carbon Black	2.23	1.19	0.75
	AC-5 + EVA	2.68×10^{-4}	2.28	0.97
	AC-5 + SBS	1.05×10^{-2}	1.74	0.92
	AC-5 + Latex	3.58×10^{-5}	2.73	0.96
	AC-5 + Polyethylene (Novophalt)	5.09×10^{-3}	1.88	0.93

*Mixture of AC-10 and crushed limestone.

°C = (°F - 32)/1.8

(After Reference 1)

APPENDIX E

Direct Compression Test Data
(Creep/Permanent Deformation)

Table E1. Average creep compliance from 1000 second creep test.

Test Temperature, °F	Sample ID	Creep Compliance ($psi^{-1} \times 10^{-6}$) at Load Duration Given Below in Seconds									
		0.03	0.1	0.3	1	3	10	30	100	300	1000
40	AC-20	0.16	0.31	0.44	0.64	0.94	1.45	2.55	4.35	7.10	11.90
	Carbon Black	0.25	0.74	1.15	1.85	2.95	4.85	7.40	11.0	15.0	21.0
	Latex	0.547	0.781	1.13	1.96	3.24	5.54	9.79	13.75	20.9	33.9
	Kraton	0.31	0.814	1.285	1.97	3.045	4.845	7.375	11.25	15.90	21.70
	Elvax	0.297	0.648	0.967	1.55	2.24	3.46	4.96	7.08	9.26	12.47
	Novophalt	0.18	0.367	0.50	0.65	0.91	1.35	2.08	3.16	4.90	7.69
70	AC-20	0.156	0.625	2.34	6.17	13.95	21.7	26.4	32.3	38.35	49.9
	Carbon Black	0.312	3.2	8.83	16.5	22.9	28.2	30.85	35.15	39.55	47.3
	Latex	2.92	7.12	13.9	23.6	33.5	43.2	60.15	78.3	100.8	147.5
	Kraton	0.86	2.92	6.74	12.3	18.05	24.45	30.05	34.8	41.5	52.25
	Elvax	0.86	2.78	5.86	10.0	14.15	18.6	21.65	25.75	30.35	36.25
	Novophalt	1.05	2.30	4.57	8.09	12.45	17.6	21.85	26.75	31.75	39.15
100	AC-20	3.12	9.37	18.8	29.6	37.5	45.3	53.1	68.7	95.3	175
	Carbon Black	5.16	13.4	27.5	36.7	42.8	47.8	52.45	60.0	67.8	81.5
	Latex	10.94	31.9	48.45	72.5	92.6	121.5	162	244.5	418	700
	Kraton	3.90	14.05	25.3	37.2	43.25	52.75	60.75	73.70	96.70	148.4
	Elvax	5.0	21.5	35.0	46.5	54.0	64.0	71.0	81.0	97.5	120.0
	Novophalt	2.5	8.15	17.5	27.0	36.0	44.5	51.0	58.5	68.5	93.0

* Additives blended with AC-5
(After Reference 1)

Table E2. Average creep compliance from 1000 second creep test on specimens tested at 70°F.

Sample ID	Treatment	Creep Compliance ($psi^{-1} \times 10^{-6}$) at Load Duration Given Below in Seconds									
		0.03	0.1	0.3	1	3	10	30	100	300	1000
AC-20	Heat Aged for 7 Days at 140°F	0.66	1.02	1.99	4.02	7.30	12.15	16.6	20.65	24.05	28.2
Carbon Black		1.76	4.80	9.06	15.45	21.90	27.70	31.5	34.85	38.5	43.45
Latex		1.88	5.55	10.23	17.90	26.0	34.20	40.05	48.05	58.30	79.45
Kraton		0.90	2.50	4.45	8.40	13.30	18.90	22.95	26.95	30.90	36.35
Elvax		0.55	1.57	2.96	5.08	7.70	10.80	13.20	15.65	17.60	20.10
Novophalt		1.09	3.12	5.58	9.91	14.80	20.15	24.05	27.95	32.30	37.85
AC-20	Over Cycle Lottman Condit-ioning	1.48	3.75	7.42	14.1	21.0	27.95	33.05	39.20	47.70	65.95
Carbon Black		1.64	6.72	11.95	19.80	24.35	28.75	31.60	35.95	40.95	50.75
Latex		1.25	3.98	8.43	14.4	20.85	26.85	31.8	38.20	47.75	68.45
Kraton		1.56	5.0	9.40	16.70	23.90	31.65	37.50	44.50	54.0	67.5
Elvax		0.86	2.34	4.88	8.20	11.65	14.90	17.40	19.50	22.15	28.20
Novophalt		1.09	2.93	5.78	9.94	14.28	19.50	24.20	29.75	36.55	48.70

(After Reference 1)

Table E3. Average creep compliance from 1000 second creep test on specimens at 70°F (San Joaquin Valley asphalt).

Sample ID	Creep Compliance ($psi^{-1} \times 10^{-6}$) at Load Duration Given Below in Seconds*									
	0.03	0.1	0.3	1	3	10	30	100	300	1000
AR-4000	0.78	1.25	2.19	4.69	9.32	19.0	29.4	40.2	52.10	73.2
AR-1000 + 15% Carbon Black	1.02	3.67	8.81	18.75	28.05	35.4	39.85	44.45	49.4	56.3
AR-1000 + 5% Latex	0.62	1.88	4.54	11.1	20.4	30.7	38.0	47.20	59.90	85.5
AR-1000 + 5% Kraton	2.03	5.68	12.1	24.8	37.6	50.8	60.3	73.2	91.3	130.8
AR-1000 + 5% Elvax	0.62	1.35	3.28	9.12	18.6	27.5	34.2	44.8	56.8	78.8
AR-1000 + 5% Novophalt	1.32	3.75	8.04	16.8	28.6	41.6	51.6	63.8	80.2	113.7

* All samples run at 20 psi except Elvax @ 15 psi
(After Reference 1)

Table E4. Average permanent strain from the incremental static compression test at 40°F. All tests at 20 psi applied stress.

Sample ID	Permanent Strain (inch x 10^{-6}/inch) After Load Duration Given Below				
	0.1 sec.	1 sec.	10 sec.	100 sec.	1000 sec.
AC-20	*	*	7.8	39.0	165.0
AC-5 + 15% Carbon Black	*	*	6.88	67.0	225.0
AC-5 + 5% Latex	*	*	13.1	111.5	501.0
AC-5 + 5% Kraton	*	*	9.4	27.4	188.0
AC-5 + 5% Elvax	1.40	2.97	9.7	41.75	127.0
AC-5 + 5% Novophalt	*	1.25	7.50	31.10	104.2

*Deformation too small to measure.
(After Reference 1)

Table E5. Average permanent strain from the incremental static compression test at 70°F. All tests at 20 psi applied stress except for latex. Results of only one test are shown for latex at 10 psi applied stress.

Sample ID	Permanent Strain (inch x 10^{-6}/inch) After Load Duration Given Below				
	0.1 sec.	1 sec.	10 sec.	100 sec.	1000 sec.
AC-20	27.0	98.0	141.1	336	716.6
AC-5 + 15% Carbon Black	19.5	101.6	281.4	519.3	834.4
AC-5 + 5% Latex	93.8	159.0	562.0	656.0	1480.0
AC-5 + 5% Kraton	*	28.9	139.0	295.0	722.5
AC-5 + 5% Elvax	26.6	66.6	95.4	228.0	504.0
AC-5 + 5% Novophalt	*	*	54.7	142.8	375.0

*Deformation too small to measure.
(After Reference 1)

Table E6. Average permanent strain from the incremental static compression test at 100°F. All tests at 10 psi applied stress except for latex, which was tested at 5 psi. Results of only one test each for AC-20 and carbon black are shown.

Sample ID	Permanent Strain (inch x 10^{-6}/inch) After Load Duration Given Below				
	0.1 sec.	1 sec.	10 sec.	100 sec.	1000 sec.
AC-20	65.6	209.0	419.0	809.0	2260.0
AC-5 + 15% Carbon Black	28.1	37.5	156.0	334.0	766.0
AC-5 + 5% Latex	120.0	275.0	628.0	1415.0	2980.0
AC-5 + 5% Kraton	84.4	178.5	334.0	630.0	1570.0
AC-5 + 5% Elvax	110.5	185.0	352.5	515.0	1140.0
AC-5 + 5% Novophalt	69.5	140.0	265.0	425.0	890.0

(After Reference 1)

Table E7. Average permanent strain from the incremental static test at 70°F after specimens were subjected to one cycle Lottman moisture conditioning. All tests at 20 psi applied stress.

Sample ID	Permanent Strain (inch x 10^{-6}/inch) After Load Duration Given Below				
	0.1 sec.	1 sec.	10 sec.	100 sec.	1000 sec.
AC-20	5.47	58.55	154.0	325.0	899.0
AC-5 + 15% Carbon Black	5.00	131.50	274.0	443.5	851.50
AC-5 + 5% Latex	25.55	82.80	229.50	523.50	1300.0
AC-5 + 5% Kraton	*	31.30	127.50	330.0	959.5
AC-5 + 5% Elvax	12.50	51.60	152.50	290.0	516.5
AC-5 + 5% Novophalt	*	10.18	61.75	184.50	533.0

*Deformation to small to measure.
(After Reference 1)

Table E8. Average permanent strain from the incremental static test at 70°F after specimens were heat aged at 140°F for 7 days. All tests at 20 psi.

Sample ID	Permanent Strain (inch x 10^{-6}/inch) After Load Duration Given Below				
	0.1 sec.	1 sec.	10 sec.	100 sec.	1000 sec.
AC-20	*	5.47	49.70	138.0	249.5
AC-5 + 15% Carbon Black	*	4.68	57.80	140.0	326.5
AC-5 + 5% Latex	7.82	57.80	187.50	453.0	1211.5
AC-5 + 5% Kraton	6.25	31.25	108.0	228.0	424.0
AC-5 + 5% Elvax	1.88	17.65	54.4	117.0	211.0
AC-5 + 5% Novophalt	*	34.35	128.8	269.5	501.50

*Deformation too small to measure.
(After Reference 1)

Table E9. Average permanent strain from the incremental static test at 70°F (SanJoaquin Valley asphalt).

Sample ID	Permanent Strain (inch x 10^{-6} inch) After Load Duration Given Below**				
	0.1 sec.	1 sec.	10 sec.	100 sec.	1000 sec.
AR-4000	*	10.9	188	437.5	1064.5
AR-1000 + 15% Carbon Black	8.6	89.1	224	407.5	731.5
AR-1000 + 5% Latex	7.8	104	368	826.5	1810
AR-1000 + 5% Kraton	43.8	189	438	797	1770
	37.5	138	328	788	2190
AR-1000 + 5% Elvax	15.6	129.8	357	740.5	1390
AR-1000 + 5% Novophalt	*	51.6	282.5	720	1955

* Additives blended with AC-5

** All samples run at 20 psi except Elvax @ 15 psi
(After Reference 1)

Table E10. Average accumulated strain for repeated load tests at 70°F (San Joaquin Valley asphalt).

Sample ID	Accumulated Strain After Number of Repetitive Cycles (Average) ($X\ 10^{-6}$) For Number of Cycles Given Below					
	1	10	100	200	1000	10,000
AR-4000	4.69	25.8	108.5	141.9	195.5	282.5
AR-1000 + 15% + Carbon Black	19.55	102.30	188.5	209.5	2545.5	407.0
AR-1000 + 5% + Latex	15.6	93.5	220.5	262.5	399.0	967
AR-1000 + 5% + Kraton	31.2	131.0	253.0	294.0	450.0	912.0
AR-1000 + 5% + Elvax	17.95	70.25	153.35	179.1	--	333.5
AR-1000 + 5% + Novophalt	9.38	56.2	156	180	212	244

*All samples run at 20 psi except Elvax @ 15 psi

(After Reference 1)