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Review of Relationships between Modified Asphalt Properties and Pavement Performance

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Contents

List of Figures

List of Tables

Abstract

The goal of this literature review on asphalt modifiers was to better understand the effects of modifiers on the performance of in-service pavements. The scope of the review was: to collect and summarize information in the technical literature relating modified asphalt properties to field pavement performance; to report the original findings and conclusions of the authors; and to note any trends or consensus. The resources reviewed included: published literature; state highway agency research reports; inquiries to associations, and manufacturers; and a National Technical Information Service on-line computerized database search.

Based on this review, it may be concluded that (a) asphalt modifiers do influence binder and mixture properties and, hence, performance; and (b) the ability to accurately interpret the association between asphalt modifiers and pavement performance has not yet been established through field studies.

Executive Summary

This literature review was initiated to better understand the effects of asphalt modifiers on pavement performance. The objective was to collect and to summarize information in the technical literature as of November 1990 which relates modified asphalt properties (both chemical and physical) to field pavement performance with a goal to identify any consensus in the technical literature regarding relationships between modified asphalt binder properties and pavement performance. Modifiers under consideration include fillers, fibers, extenders, polymers, plastics, anti-strip agents, oxidants and reclaimed rubber. Pavement performance measurements of interest were those investigated by Project A-003A; namely fatigue cracking, rutting, thermal cracking, aging, and water sensitivity.

Although most modified binder research has been relatively recent in the United States, a significant amount of information has been published on the subject. For practical considerations, an exhaustive review of all published literature was not conducted; however, where possible, at least one report was included on each modifier type. Selection was also based on the report's possible relevancy to pavement performance. Several reports were found during the literature search that were not included in this report, since they were not judged to contribute significantly beyond what is included in the reviewed papers. The uncited publications have been included in the bibliography.

This report is not a critical review of the individual publications or research efforts. Instead, the intent is to summarize the findings of the individual researchers and to report any trends or consensus, if any. Only results and conclusions from the original references have been reported. No attempt has been made to pool the data or to perform any additional analysis. The limited field performance data does not lend itself to quantitative analysis; however, a qualitative interpretation of the data has been included.

For this review, a variety of resources were researched as follows:

- Published literature such as AAPT, TRR, ASTM, STP, etc.
- State Highway Agency research reports,
- NTIS computer search, and
- National Asphalt Pavement Association, inquiries to manufacturers, and other sources.

Because the goal of this review is to determine the relationships between modified asphalt binders and field pavement performance, the following set of criteria were used to determine reports suitable for inclusion.

- Reports must include some measurements (field or laboratory) that relate to performance as measured by fatigue cracking, rutting, thermal cracking, aging, and water sensitivity.

- Reports must include at least two of the three followings items: a) Information on properties of the modified binder, b) properties of the mixture prepared with modified binder, or c) field performance measurements for mixes prepared with modified binders.

- Attempts were made to include at least one report on each of the different modifier types available.

The above criteria resulted in a total of approximately 20 reports that are included in this review. Although more reports on the subject were identified, they were not included unless their conclusions were different from those already reported.

Findings

Based on information contained in the reviewed references, it can be concluded that modifiers have an influence on the performance-related properties of asphalt cement and asphalt concrete, as measured in the laboratory. They also have an influence on the performance of asphalt pavements as determined from field test section evaluations.

However, one of the main findings from this review is that there are many problems associated with the non-standard way in which material was tested and characterized and for which relevant data or information was available. These problems make it difficult to establish consensus relationships and trends associated with the use of modified materials.

Field performance of modified binders is generally measured through the use of special test sections placed in a project where the remainder is a control section using unmodified asphalts. Comparisons are then made between the performance of the test section and the control section. Interpretation of results relies on site-specific factors including pavement structure characteristics, traffic and environmental conditions. The strong influence of these site-specific factors makes extrapolation of test results to more generalized conditions very difficult. The interpretation of test results may only be valid for these site specific conditions.

Findings from one investigation do not always agree with those from another investigation. Confounding effects of structure and environment, as discussed previously, are factors that could account for some of the differences. In addition, the base asphalt appears to have a pronounced influence on modifier effectiveness. An important finding from the review is that modifiers may enhance certain properties of a given binder to produce more favorable performance characteristics but not to a level that could be obtained simply by changing asphalt source or grade.

A few European countries have experimented with modified binders including Carbon Black, EVA, SBS, polyethylene, ground rubber, and proprietary materials for the last several years. Recently, a team of asphalt pavement specialists from the United States participated in a two-week study tour of six nations and have provided an overview of conditions in those countries visited. Of the nations visited, France, Italy, Denmark, and the United Kingdom have experience with modifier use. France and Italy, in particular, are reported to use modifiers routinely. Both countries also use proprietary modified mixtures and products. For example, approximately 7% of all asphalt mixes used in France are polymer-modified, compared to 2% to 3% for the United States, but polymer-modified mixes are used only in the surface course. Italy uses polyethylene and rubber; however, the rubber does not come from scrap tires. Although asphalt modification practice is common, contractors and officials visited in the European Asphalt Study Tour suggest that modified binders still represent a small special-purpose niche in the overall pavement market. Subjective impressions of European highways noted by the study team were that the overall condition of European highways is superior to those in the United States, even though vehicle loadings and configurations may be more severe. There are a number of reasons for this situation, which are discussed in the European Asphalt Study Tour 1990, including different practices for pavement structural design, pavement management, mix design, and different contracting procedures and emphasis on research. However, the use of asphalt modifiers may play a significant role in the improved condition.

In contrast, use of modifiers in asphalt pavement construction is a fairly recent practice in the United States and long-term performance data is largely unavailable. In most of the studies, field performance measurements were conducted early in pavement life (1 to 3 years). In many of these, the test and control sections were both in good condition and no discernable difference in performance could be detected between the two. Based on the results of the studies, it appears more long term (greater than 5 years) field observations are needed to help distinguish the effects of the modifiers.

Considerations for Implementation

The cost of asphalt binder modification can be significant. From France's experience, polymer-modified binders cost approximately twice as much as conventional binder, resulting in approximately 20% higher costs for polymer-modified mixtures. Paving contracts in Europe frequently require performance warranties, and contracting approaches favor and encourage contractors to develop innovative mix designs and to utilize modified binders. There is less emphasis on low first cost and a higher emphasis on long-term durability. The cost-effectiveness of modifiers, however, is difficult to establish. Performance data for conditions in the United States is limited and inconclusive. The performance of European roadways would suggest that modifiers are cost effective; however, as previously indicated, more long-term experience and measures of cost-effectiveness are required for U.S. conditions.

Test sections on several of the projects reviewed in this report are scheduled for performance monitoring on an on-going basis. It will be important to continue performance monitoring in future years to better establish benefits and cost-effectiveness of modifiers. The performance period in most studies reviewed is too short to measure the effects with any degree of certainty. Long-term data (5 years or more) from actual field trials will be needed to verify preliminary information.

1
Introduction

1.1 Background

The Strategic Highway Research program (SHRP) is a highly focused, five year $150 million, research program which grew out of recommendations contained in Transportation Research Board Special Report 202, "America's Highways: Accelerating the Search for Innovation". The report documented the serious neglect of highway research in the United States and recommended a concerted research effort to address six high-priority research areas which were later consolidated into four areas: (1) asphalt, (2) concrete and structures, (3) highway operations, and (4) pavement performance.

Part of the asphalt program is designed to investigate how the chemical and physical properties of asphalt binder relate to pavement performance. The results are expected to include improved: (a) characterization of materials, (b) product testing methods, and (c) construction procedures. Other parts of the asphalt program involve the development of accelerated tests for asphalt-aggregate mixtures to better predict the performance of the pavements. Improved understanding of asphalt-binder characteristics and the availability of new asphalt-aggregate tests would also permit the development of more performance-oriented specifications.

In order to accomplish these goals, a series of research contracts have been developed by SHRP. One such contract is SHRP Project A-003A, entitled "Performance-Related Testing and Measuring of Asphalt-Aggregate Interactions and Mixtures," of which this report forms a part.

One of the tasks in project A-003A is SHRP Task 1.4, which has two major objectives: The first objective is to assimilate information in the technical literature relating to chemical and physical properties of asphalts to pavement performance and mixture properties. The second objective is to accumulate test data for incorporation into the national data base.

Another task in this project will be to describe and to standardize test methods for measuring those properties that characterize fatigue cracking, permanent deformation, and low-temperature cracking in asphalt-aggregate mixtures. A wide range of tests will be included in this part of the investigation, including provisions for aging, moisture sensitivity, and temperature conditions representative of a range of field conditions.

Finally, relationships between mixture properties, asphalt chemical and physical properties, and performance for a range of asphalts and aggregates including selected modified asphalts will be established in this project.

1.2 Objectives

The test methods to be developed in SHRP Contract A-003A are designed to measure fundamental properties of asphalt-aggregate mixtures that relate to pavement performance. For the tests to be valid, the tests must be sensitive to the effects of asphalt-binder properties on pavement performance. Modified asphalt binders are included within the scope of binders in the testing phase of project A-003A. In this vein, the test methods must be sensitive to the effects of modified asphalt binders on pavement performance. For example if 1% of Modifier A increases pavement fatigue life in the field by 25%, then the laboratory test should show a similar corresponding increase for a mixture prepared with the modified binder.

To better understand the effects of asphalt modifiers on pavement performance, this literature review was initiated. The objective of this review was to collect and to summarize information in the technical literature as of November 1990 that relates both chemical and physical modified asphalt properties to field pavement performance. This review attempts to determine (a) those properties of modified asphalt cement and mixtures that can significantly influence field pavement performance, and (b) if there is any consensus in the technical literature regarding relationships between modified asphalt binder properties and pavement performance. The emphasis is on properties of modified binders. Modifiers under consideration in this report include fillers, fibers, extenders, polymers, plastics, anti-strip agents, oxidants, and reclaimed rubber. The pavement performance measurements of interest were those investigated by project A-003A; namely fatigue cracking, rutting, thermal cracking, aging, and water sensitivity. A review of unmodified asphalts has previously been completed (Finn et al., 1990); this report is a supplement to the this review.

Both physical and chemical properties of the modified asphalt were considered pertinent to this review. The physical properties typically measured are penetration, viscosity, softening point, temperature susceptibility, binder stiffness, ductility and other rheological characteristics. The chemical properties would include the chemical composition factors and the functionality of asphalt. The specific physical and chemical properties reported are based on information provided in the literature.

1.3 Scope

This report is the result of a review of published literature and of state highway agency, (SHA) research reports on the properties of ***modified*** asphalt binders and their relationship to pavement performance. It is an addendum to an earlier report prepared by Finn et al.

(1990) and is similar in nature, but the focus is on modified asphalt binders and mixes, whereas the earlier report's focus was on unmodified asphalts.

The intent of this review is to report the current technology regarding the field performance of modified asphalts in pavements. Although most modified binder research has been fairly recent, a significant amount of information has been published on the subject. For practical considerations, this review is <u>not</u> intended to be exhaustive; however, where possible, at least one report was included on each modifier type. Several additional reports are available in the literature that were not reviewed; however, they have been included in the bibliography.

Only results and conclusions from the original references have been reported. This report is not a critical review. Instead, the intent is to summarize the findings of the individual researchers and to report any trends or consensus, if any. No attempt has been made to pool the data or to perform any additional analysis as was done in Finn et al. (1990). The limited field performance data does not lend itself to quantitative analysis; however, a qualitative interpretation of the data has been included.

1.4 Sources of Information

Although most of the developments in modified asphalt binder technology have been fairly recent, a significant body of information exists on the subject. Several informational sources were enlisted during the literature search to find reports suitable for review. Information from the published literature provided the largest source, such published literature refers to the body of information that is readily available, such as the Transportation Research Records (TRR), proceedings from the Association of Asphalt Paving Technologists (AAPT), and other generally available sources. They are differentiated from other sources such as internal reports prepared by SHAs or manufacturers, which are typically not published in the usual sources. For this review, a variety of other resources were researched as follows:

1. Published literature such as AAPT, TRR, ASTM STP, etc.
2. State Highway Agency research reports.
3. NTIS computer search.
4. National Asphalt Pavement Association (NAPA), inquiries to manufacturers, and other sources.

Since the goal of this report is to determine the relationships between modified asphalt binders and field pavement performance, the following three criterion were used to determine which reports would be included in this review:

1. Reports must include some measurements (field or laboratory) that relate to performance, as measured by fatigue cracking, rutting, thermal cracking, aging, and water sensitivity;

2. Reports must include at least two of the three followings items (a) Information on properties of the modified binder, (b) properties of the mixture prepared with modified binder, or (c) field performance measurements for mixes prepared with modified binders; and

3. Attempts were made to include at least one report on each of the different modifier types available.

The search of published information consisted of a title search of all reports concerning modified asphalts published in the proceedings of the AAPT, TRR, and ASTM Special Technical Publications. The reports were generally not more than 15 years old. Any report containing reference to modifiers was reviewed more thoroughly to evaluate it in accordance with the criteria above. Reports included in this review are those considered to best fit the criteria.

A few SHA research reports on the performance of pavements prepared with modified asphalts were found and are included in this review. Many of the references were collected and reviewed by the investigators on the project A-004. Investigators in the SHRP project A-004 sent a letter of inquiry to sixty four SHAs (all fifty U.S. states and Canadian provinces) requesting copies of internal or published reports summarizing agency experiences with asphalt modifiers. They received a total of forty-three reports which contained information on the field performance of modifiers. A brief abstract of these reports was prepared by the A-004 investigators. Based on review of these abstracts, a small number of the reports were deemed of interest to the project A-003A. Copies of these reports were subsequently obtained from the project A-004 for a more detailed review.

There are several reasons why only a small number of the forty-three reports are included in this review. In the majority of the projects, the modified asphalt test sections were added to ongoing projects at the request of modifier manufacturers, and typically the project did not include any comprehensive testing and evaluation of the materials. These projects would be considered as uncontrolled test roads, according to the definition used in Finn et al. (1990). Modifiers were generally chosen to study their effects on rutting and on reflective cracking of asphalt overlays. Reflection cracking is not one of the performance measurements considered in the project A-003A. In some of the reports, modifiers were used in asphalt surfaces for bridge decks. This is also outside the scope of the project A-003A. The length of monitoring field performance was generally limited to three to five years after construction, since most SHAs felt that was all that was necessary to assess modifier effectiveness. Performance data was generally limited to visual observations by SHA personnel. Some of the reports included rut depth and crack count measurements.

Laboratory data was also limited and was generally found in the form of mix design and asphalt consistency data. Some reports, however, included detailed laboratory test data. Most of the reports included a control section constructed with the base asphalt for comparison.

A computer search of the National Technical Information Service (NTIS) database was also conducted from 1970 to present to locate references likely to be published by the Federal Highway Administration (FHWA) or SHA research reports funded by the FHWA. The search located approximately five additional references of interest not found in other sources. A user's manual on asphalt modifiers published by the NAPA (Terrel & Epps, 1989) was also reviewed. This reference was an excellent source regarding modifier classification, general properties and performance, and specific references for each modifier class. Finally, inquiries were made to various manufacturers of asphalt modifiers for technical information concerning field performance. Information received generally included reprints of "published" reports found in technical journals such as AAPT and TRR proceedings.

1.5 Organization of the Report

The above criteria resulted in a total of approximately twenty reports to be included in this review. Although more reports on the subject were identified, they were not included unless their conclusions are different from those already reported. This report contains five chapters. This chapter presents the background, objectives, scope, and sources of information for this report. Reports on field studies on pavement performance are included in Chapter 2. Reports with laboratory performance only are included in Chapter 3. A summary of the information, discussion of the results, and conclusions are included in Chapter 4. Finally, references cited in the text and a bibliography are also included.

2
Field Studies

This chapter reviews research reports that contain field studies on the performance of pavements prepared with modified asphalts. The information comes from both SHAs and the published technical literature. For each report reviewed in this chapter, a description of the project is provided, and the results and conclusions of the researchers are then presented.

2.1 Oregon - Various

Description

In 1985, an experimental road section incorporating different asphalt additives was constructed near Bend, Oregon (Hicks et al., 1987). Ten sections were constructed using mix designs furnished by the additive supplier or by the Oregon Department of Transportation. There were two objectives in this study:

1. To evaluate the effectiveness of ten hot, mix overlay test sections, incorporating the use of various additives to extend the life of asphalt concrete pavements; and

2. To determine the cost-effectiveness of each section compared with a conventional asphalt concrete mix.

The ten sections were comprised of:

1. PlusRide® 12 which is a coarse-ground rubber in a mix with modified aggregate gradation and asphalt containing Pave Bond, that is, an anti-stripping agent;

2. Arm-R-Shield, an asphalt concrete containing fine-ground rubber in asphalt in a mix with conventional aggregate gradation;

3. Fiber Pave®, a polypropylene fiber in a mix with asphalt containing Pave Bond and a conventional aggregate gradation;

4. BoniFibers®, a polyester fiber in a mix with asphalt containing Pave Bond and a conventional aggregate gradation;

5. Pave Bond®, an asphalt containing an anti-stripping agent in a mix with a conventional aggregate gradation;

6. Pave Bond® and lime, a lime-treated aggregate and asphalt containing an anti-stripping agent in a mix with a conventional aggregate gradation;

7. Lime, a lime-treated aggregate in a mix with a conventional aggregate gradation;

8. No additive, a conventional asphalt concrete mix;

9. CA(P)-1, a polymer contained in asphalt in a mix with a conventional aggregate gradation; and

10. CA(P)-1 with lime, a polymer contained in asphalt with lime-treated aggregate in a conventional mix.

Hicks et al. (1987) presented mix design, construction process, initial mix properties and 2-year old performance data in their report. A follow-up report with more performance information is expected to be published in late 1990.

The ten sections were part of an asphalt concrete overlay project. Each test section was a minimum of 0.5 miles in length and included a 12-foot wide travel lane. Figure 2-1 illustrates the layout of the test sections. The structural overlay included a leveling course, a 1.5-in. bottom lift, and 1.75 in. for the top lift (which included the additive). An AC-20 asphalt cement from Chevron's Willbridge Refinery in Portland, Oregon, was used in all sections except where the polymer-modified asphalt was used. Prior to the overlay, a pavement condition survey was performed; considerable alligator and thermal cracking as well as patching was present. The temperature at the test road ranges from -10°F in the winter to 100°F in the summer, with daily temperature ranges of 40°F. Snow and ice are present from November through February.

Table 2-1 summarizes the properties of the AC-20 asphalt cement, and Table 2-2, the properties of Chevron's polymer-modified asphalt CA(P)-1. Table 2-3 details the mix design procedures and criteria used for each additive, and Table 2-4 summarizes the resulting asphalt contents used. In general, the quality control tests performed during construction indicated no major problems in the mix with the exception of low densities (high voids).

Results

Shortly after construction in September 1985, 4- and 6-in. cores were taken from each test section. The 4-in. cores were tested for density, voids, modulus and stability, while the 6-in. cores were tested for gradation, asphalt content, and other properties. In addition, box

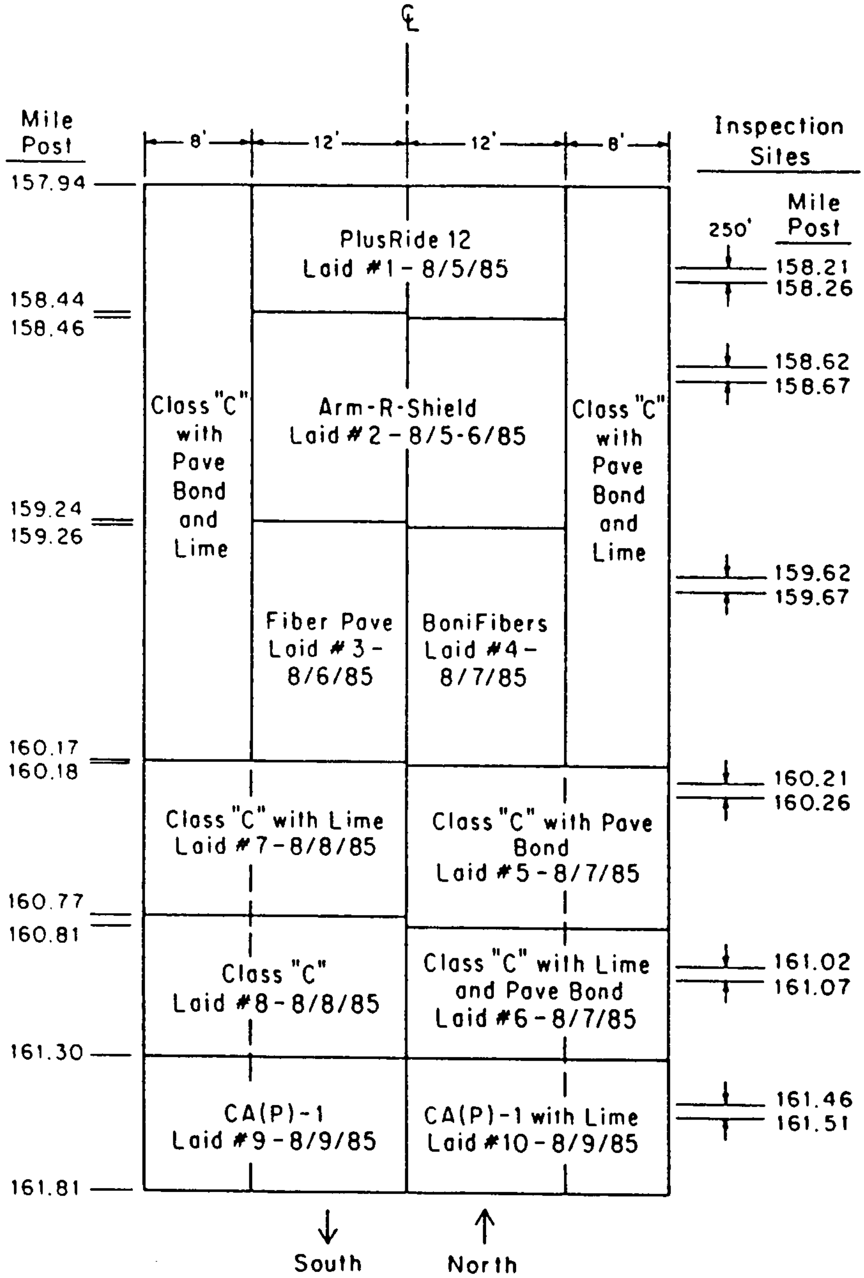


Figure 2-1. Layout of test sections in Oregon (Hicks et al., 1987).

Table 2-1. Properties of AC-20 asphalt cement for Oregon (Hicks et al., 1987).

Property	Actual	Specification
Viscosity at 140°F (poise)	2040	2000 ± 400
Viscosity at 275°F (cSt)	352	230 min
Penetration at 77°F (dmm)	58	50 min
Flash point, COC[a] (°F)		
(AASHTO T-73)	600	450 min
Solubility in trichloroethylene (%)	99.86	99 min
Tests on residue		
Viscosity at 140°F (poise)	6122	8000 max
Ductility at 77°F (cm)	–	75 min

[a]COC = Cleveland open cup.

Table 2-2. **Preliminary product specification, Chevron polymer asphalt CA(P)-1 for Oregon (Hicks et al., 1987).**

Property	ASTM Test Method	CA(P)-1 Specification	CA(P)-1 Properties
Original test Properties			
Penetration at 77°F (dmm)	D 5	85 min	113
Viscosity at 140°F (poise)	D 2171	1600–2400	2092
Viscosity at 275°F (cSt)	D 2170	325 min	676
Flash point, COC (°F)	D 92	450 min	500
Ductility at 77°F (cm)	D 113	100 min	150+
Ductility at 39.2°F (cm) (5 cm/min pull rate)	D 113	25 min	32
Toughness (in.-lb)	–[a]	75 min	124
Tenacity (in.-lb)	–[a]	50 min	101
Properties After Rolling Thin-Film Oven Test			
Viscosity at 140°F (poise)	D 2872	10 000 max	4980
Ductility at 77°F (cm)	D 113	100 min	150+
Ductility at 39.2°F (cm) (5 cm/min pull rate)	D 113	8 min	13
Toughness (in.-lb)	–[a]	100 min	325
Tenacity (in.-lb)	–[a]	75 min	346

[a]Benson method of toughness and tenacity: 20 in./min pull rate, 7/8-in.-diameter tension head.

Table 2-3. Mix design procedures and criteria used, additive suppliers and ODOT for Oregon (Hicks et al., 1987).

Feature	Method	Compactive Effort	Additive	Design Criteria	Comments
PlusRide	Marshall	50 blows/side	3% rubber granules by weight of total mix	3% air voids	Mix is rich in asphalt and filler, has high coarse aggregate content, and is gap graded
Arm-R-Shield	Marshall	75 blows/side	20% rubber by weight of asphalt binder	Stability (lb): 1,500 min Flow: 8–18 in. $\times 10^{-3}$ Voids: 3–5%	Asphalt-rubber is reacted at elevated temperature before use Mix is rich in asphalt
Fiber Pave (polypropylene)	None given	–	0.3% fiber by weight of total mix	Asphalt content increased 0.3% over the standard mix	
BoniFibers (polyester)	None given	–	0.25% fiber by weight of total mix	Asphalt content increased 0.3% over the standard mix	
Chevron CA(P)-1	Hveem	150 blows/500 psi	5.0% of asphalt binder	Stability: 30 min Appearance: shiny	Polymer with and without lime-treated aggregate
All other mixes	Hveem (ODOT)	150 blows/500 psi	0.5% Pave Bond by weight of asphalt binder or 1.0% lime slurry by weight of aggregate, or both	Stability: 30 min Voids: 4–5% IRS: 75% min Modulus ratio: 70% min	For details of mix design procedure see Sullivan et al.(1986)

2-6

Table 2-4. Mix design results (from additive suppliers) for Oregon (Hicks et al., 1987).

Additive	Recommended Asphalt Content (%) With Lime	Recommended Asphalt Content (%) Without Lime
PlusRide	–	8.0
Arm-R-Shield	–	8.0
Fiber Pave	–	6.7
BoniFibers	–	6.7
CA(P)-1	6.5	6.5

NOTE: Percentage by weight of total mix.

samples of the mix were also taken during construction and tested. Tables 2-5 and 2-6 summarize the results of the tests performed on the cores and the box samples. In 1986, cores were again removed and diametral modulus and fatigue tests performed in accordance with ASTM D4123. In March 1986, both modulus and fatigue tests were run at 200 microstrain at 73°F. In June 1986, the tests were conducted at both 100 and 200 microstrain and at 73°F. Table 2-7 summarizes the results of these tests.

Some general observations from the 1985 cores (Tables 2-5 and 2-6) were noted:

1. Modulus values (at 77°F) range from 93,000 psi for Arm-R-Shield to 590,000 psi for the C-mix with Pave Bond and lime-treated aggregate;

2. In-place voids range from 3.7% for PlusRide to 8.1% for BoniFibers;

3. Hveem stability values (in place) are in the normal range except for PlusRide and lime section;

4. The asphalt content and mix gradation were more or less in compliance with the job mix formula;

5. The viscosities of the recovered asphalt from the box samples generally were higher than those measured on the core samples. This is because these loose materials were tested up to one or two months after sampling. The highest viscosity at 140°F was measured on the polymer-modified asphalt in both cases;

6. The viscosities at 140°F of the rubber-modified asphalts were lower than those of the other mixes;

7. The penetration values at 77°F for the rubber and polymer-modified asphalts were higher than for the other materials;

8. The Hveem stability values of laboratory-compacted box samples were all greater than thirty, except for the PlusRide mix;

9. The index of retained strength (IRS) of all mixes was greater than 75% minimum, except for PlusRide; and

10. The modulus ratios after freeze-thaw conditioning were all greater than the 0.70 minimum, except for PlusRide.

Table 2-5. Summary of test results, 4-in. cores (September 1985) for Oregon (Hicks et al., 1987).

Property	PlusRide	Arm-R-Shield	Fiber Pave	BoniFibers	Pave Bond Without Lime	Pave Bond With Lime	Control With Lime	Control Without Lime	CA(P)-1 Without Lime	CA(P)-1 With Lime
Unconditioned modulus at 77°F (1,000 psi)	264	93	111	137	275	590	209	256	352	366
Gravity										
In place (voids)	2.21	2.27	2.28	2.26	2.34	2.31	2.31	2.30	2.36	2.31
	(3.7)	(6.9)	(6.5)	(8.1)	(5.3)	(6.6)	(6.9)	(7.1)	(4.9)	(6.9)
Recompacted (voids)	2.23	2.38	2.42	2.41	2.44	2.45	2.43	2.41	2.46	2.45
	(2.8)	(2.4)	(0.8)	(2.0)	(1.2)	(0.9)	(2.1)	(2.6)	(0.9)	(1.2)
Relative compaction (%)	96.3	93.1	93.5	91.9	94.7	93.4	93.1	92.9	95.1	93.2
Maximum theoretical gravity	2.295	2.438	2.439	2.458	2.470	2.473	2.481	2.475	2.481	2.480
Hveem stability at 140°F										
In place	2	29	12	14	29	16	24	18	25	22
Recompacted	1	18	21	29	19	32	3	34	22	24

Table 2-6. Summary of mix and asphalt property test results, 6-in. cores and box samples (September 1985) for Oregon (Hicks et al., 1987).

	PlusRide		Arm-R-Shield	Fiber Pave	BoniFibers	Pave Bond Without Lime	Pave Bond With Lime	Control With Lime	Control Without Lime	CA(P)-1 Without Lime	CA(P)-1 With Lime
	1	2									
Gradation[a]											
(% passing)											
3/4 in.	100	100	100	100	100	100	100	100	100	100	100
1/2 in.	97	95	98	97	98	97	99	98	98	100	99
3/8 in.	77	72	86	84	85	88	88	83	84	88	85
1/4 in.	47	43	62	64	66	68	67	66	66	66	63
No. 4	42	38	51	54	54	56	56	55	54	54	52
No. 10	35	33	30	32	31	32	32	32	32	32	32
No. 40	18	16	13	14	14	14	14	14	13	14	14
No. 20	7.0	6.2	5.8	6.1	5.8	6.2	6.4	6.7	6.0	5.9	5.5
Asphalt content (%)	8.5	7.8	6.8	7.0	6.9	6.2	6.3	6.3	6.3	7.2	6.4
Asphalt properties[a]											
Viscosity at 140°F (poises)	3319	4479	3302	8064	9130	8120	7637	5650	6560	8013	10 100
Kinematic viscosity at 275°F (cSt)	445	514	849	597	591	624	572	534	568	1040	1137
Penetration (dmm)	40	42	75	27	22	23	22	25	21	40	36
Asphalt properties[b]											
Viscosity at 140°F (poises)	2070	–	2733	5739	7231	7669	2574	6542	8435	9962	12 478
Kinematic viscosity at 275°F (cSt)	374	–	929	539	599	599	635	560	609	1093	1369
Penetration (dmm)	55	–	78	32	29	28	25	26	28	37	34
Mix properties											
Stability	8	5	37	44	39	44	40	39	41	41	37
IRS (%)	64	82	93	94	92	95	100	94	97	84	91
Modulus ratio (freeze-thaw)	0.73	0.63	0.70	0.76	0.84	0.92	0.85	0.90	0.78	0.73	1.00

[a]Box samples.
[b]6-in. cores.

Table 2-7. Summary of modulus and fatigue test data (field cores) for Oregon (Hicks et al., 1987.

Mix Type	Avg Density (pcf)	Avg Modulus[a] (1,000 psi)	Load Applications to Failure
March 1986			
PlusRide	137.6	272	15,942
Arm-R-Shield	141.7	194	4,171
Fiber Pave	144.2	400	6,708
BoniFibers	142.2	387	4,487
Pave Bond without lime	144.0	475	5,347
Pave Bond with lime	145.0	506	6,052
Control	144.9	457	7,094
Control with lime	147.4	511	4,986
CA(P)-1 without lime	148.4	284	21,187
CA(P)-1 with lime	144.4	298	37,375
June 1986			
PlusRide	136.2	341	88,500[b]
Arm-R-Shield	139.4	196	19,876[b]
CA(P)-1 without lime	147.7	304	19,208 122,043[b]
CA(P)-1 with lime	143.8	287	200,598[b] 31,432

[a]Tests run at 73°F and 200 microstrain, except as noted.
[b]Tests run at 73°F and 100 microstrain.

In September and October 1985, the test sections were evaluated for pavement condition, surface deflection, and skid resistance and ride. After one year of service, the performance is good. There is no cracking, rutting or extensive raveling. After the overlay, deflections were reduced in the order of 50% to 70%, and the skid and ride number for all sections were considered good and about the same for all sections.

Conclusions

No definitive conclusions can be drawn with regard to the relationship between performance and the presence of asphalt additives due to the lack of any significant field performance data. However, a follow-up report is expected in late 1990 which will contain performance data for the past five years. Hicks et al. (1987) report that failures in this general area have been experienced within two to four years after construction. There were definite differences, however, in the penetration and in the viscosity for the modified mixes.

2.2 Mt. St. Helens - Rubber

Description

In August 1983, a coarse rubber-modified asphalt pavement overlay was constructed in the Gifford-Pinchot National Forest, in Washington state near Mt. St. Helens as part of a Volcanic Activity Disaster Relief (VADR) project. The rubber material was originally marketed in the early 1970s by two Swedish companies, Skega AB and AB Vaegfoerbaettringar (ABV) under the patented name "Rubit." In the U.S., the trademark "PlusRide" is used. The rubber was obtained from shredded auto and light-truck tires and tire buffings from retreading operations. It was mixed with the aggregate.

The project is 1.11 miles long and has three different asphalt concrete (rubber-modified) thicknesses: 1.75, 2.5 and 3.5 in. The 3.5-in. section was placed in two equal lifts. In addition, a control section was available which was constructed with 3.5 in. of conventional bituminous concrete.

Results

Cores were taken shortly after construction in September 1983 and subsequently in November 1984, July 1985 and June 1986 from both the rubber-modified and the adjacent conventional mix overlays. The tests conducted included:

1. Bulk specific gravity,
2. Diametral resilient modulus,
3. Diametral fatigue,
4. Hveem stabilometer, and
5. Indirect tension.

The diametral resilient modulus test was performed using ASTM D4123 except as noted. Testing was conducted at 22.5 ± 1°C, and a pulse loading with a duration of 0.1 and a frequency of 1 cps was applied. Table 2-8 summarizes the results of these tests for both the rubber-modified and control cores. Figure 2-2 illustrates the effect of time on the modulus for the cores. The results show both materials to be increasing in stiffness with time. The rate of increase is decreasing for both materials with the rubber-modified mix showing a slightly overall increase than the control mixture. Also, the rubber-modified mix is more flexible than the control.

Figures 2-3 and 2-4 illustrate the diametral fatigue life for the rubber modified and the control cores, respectively. The following four observations were recorded:

1. The rubber asphalt mixes were subjected to higher strain levels so that failure would occur in a reasonable length of time.
2. The coefficient of determination (R^2) for the rubber-modified samples (50% to 97%) are lower than for the control mixes (88% to 96%). This could be an indication that diametral testing is not the most appropriate for rubber-modified materials.
3. The slope of the curves for control samples were steeper indicating a greater change in laboratory-fatigue life with a given change in strain level.
4. Both types of samples showed a decrease in expected life with time, reflecting the increase in the modulus in Figure 2-2.

Tables 2-9 and 2-10 summarize the results of the Hveem stabilometer (ASTM D1560) and of indirect tension tests. Hveem stability values increased for the control samples, but remained approximately constant for the rubber-modified mixes. Although these values are not usually acceptable, no rutting was apparent during the field surveys, the last of which was conducted after 3 years of in-service performance. A limited amount of data is available from the indirect tensile tests but would indicate slightly greater tensile strengths for the conventional mixture. Field data collected by the Forest Service and the FHWA in the three years following construction included traffic, deflection roughness, and skid resistance data for 1983-1986 and surface texture (1985). Table 2-11 summarizes the traffic data collected. In addition, visual condition surveys to determine the presence of cracking, raveling or rutting were conducted in November 1983, March 1984, June 1985 and June 1986. From the surveys, both surfaces were in good condition. Deflection results showed very little differences between the rubber-modified and the control sections.

Conclusions

Because of the good performance of the pavements after three years and to the lack of any long-term data, little can be concluded on the effect of the rubber on pavement field performance. However, based on the laboratory tests, Lundy et al. (1987) were able to make the following five conclusions:

1. Moduli of both materials tested increased with time. Rubber-modified mixtures are hardening slightly faster than control mixes;

Table 2-8. Rubber-modified and control modulus values at 23°C for Mt. St. Helens (Lundy et al., 1987).

	Rubber-Modified				Control			
Test Date	Number of Samples	Bulk Specific Gravity	Air Voids (%)	Modulus (ksi)	Number of Samples	Bulk Specific Gravity	Air Voids (%)	Modulus (ksi)
September 1983	17	2.34 (.04)	4.9 (0.8)	225 (47)	9	2.36 (.04)	6.1 (0.6)	346 (62)
November 1984	15	2.37 (.03)	3.6 (0.9)	475 (122)	5	2.38 (.01)	4.0 (0.3)	581 (60)
July 1985	8	2.36 (.03)	2.4 (1.0)	585 (95)	5	2.37 (.04)	6.2 (0.7)	707 (116)
June 1986	18	2.35 (.01)	2.8 (0.5)	443 (51)	12	2.40 (.02)	4.9 (0.7)	610 (48)

NOTES: 1) Numbers in parentheses are standard deviations

2) November 1984 moduli are adjusted to a temperature of 22.5°C

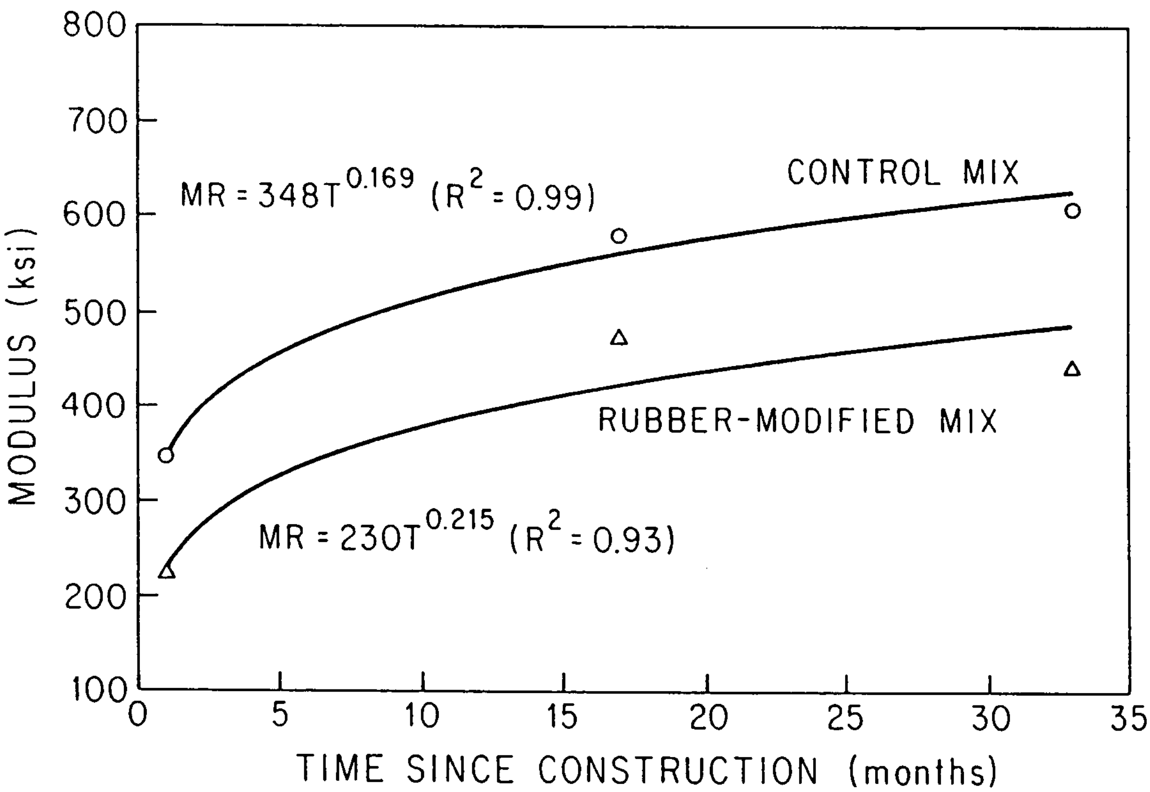


Figure 2-2. Variation of resilient modulus with time for Mt. St. Helens (Lundy et al., 1987).

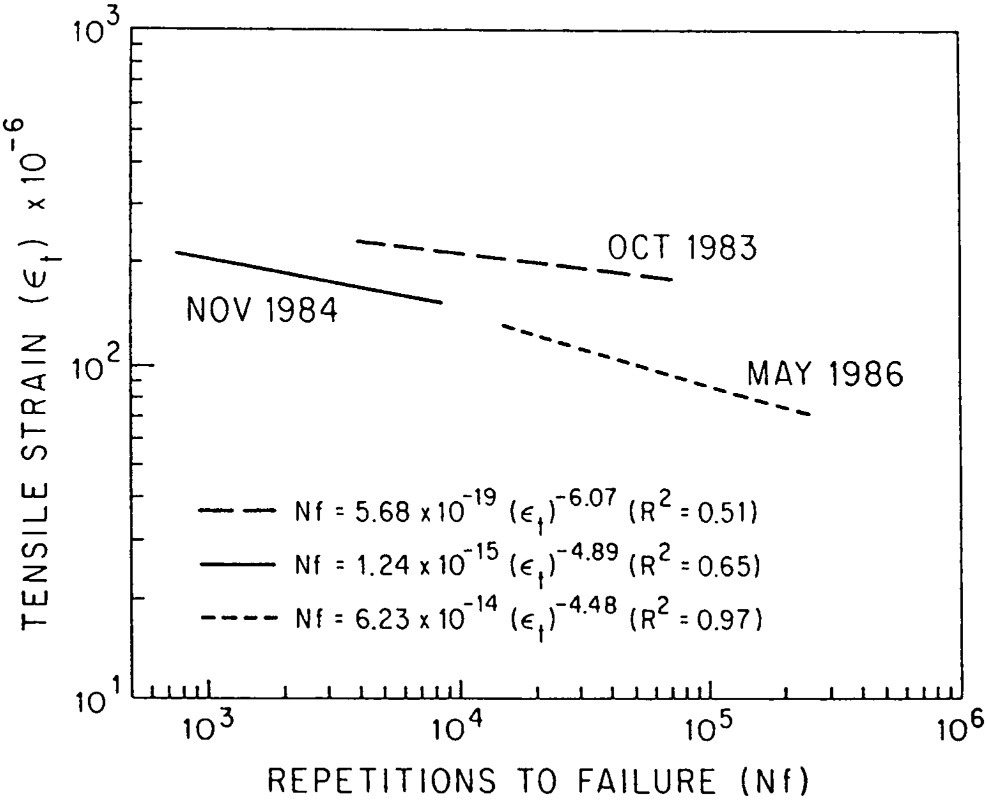


Figure 2-3. Tensile strain vs. fatigue life, rubber samples for Mt. St. Helens (Lundy et al., 1987).

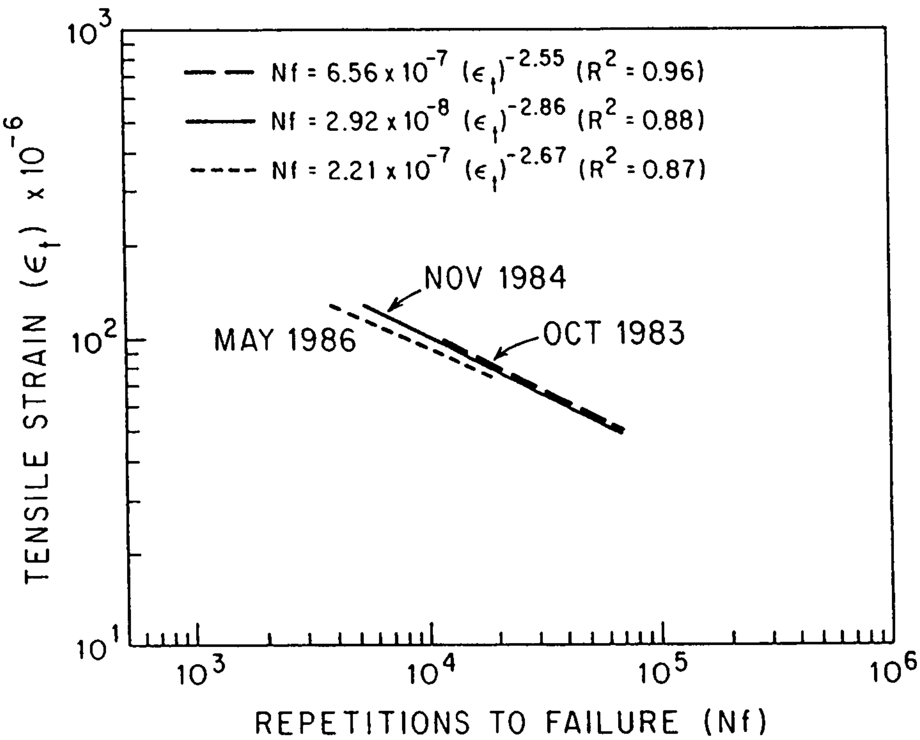


Figure 2-4. Tensile strain vs. fatigue life, control samples for Mt. St. Helens (Lundy et al., 1987).

Table 2-9. Hveem stabilometer test values for Mt. St. Helens (Lundy et al., 1987).

	Rubber	Control
October 1983*	2.8	17.8
November 1984**	2.0	20.5
July 1985**	2.0	21.0
June 1986**	1.0	27.0

*Mean of 6 samples tested.

**Mean of 2 samples tested.

Table 2-10. Indirect tensile test values for Mt. St. Helens (Lundy et al., 1987).

	Rubber	Control
November 1984	117 psi	160 psi
July 1985	103 psi	-
June 1986	101 psi	109 psi

NOTE Tests were not conducted in October 1983 or on control samples July 1985 due to sample shortfalls.

Table 2-11. Traffic summary for Mt. St. Helens (Lundy et al., 1987).

Year	Site 601					Site 351				
	Total Count	Log	Rec	Other	18^{k}* EAL	Total Count	Log	Rec	Other	18^{k}* EAL
1983**	23,464	27%	70%	3%	26,177	44,950	33%	63%	4%	52,875
1984	99,202	35%	58%	7%	116,506	126,410	23%	73%	4%	134,161
1985	93,546	15%	81%	1%	90,675	100,148	25%	69%	6%	106,930
1986	99,691	7%	87%	6%	85,806	101,778	25%	69%	6%	108,670
				Total	319,164				Total	402,636

*18^{k} Equivalent Axle Loads

Log Factor = 1.98

Rec Factor = 0.83

Other Factor = 0.0004

**Since construction (September 1983)

2. Laboratory fatigue lives of both materials are decreasing with time. Laboratory results indicate the expected fatigue life of the rubber-modified mix would exceed that of the control for any given strain level;
3. Control mixtures show a greater increase in stability with time than rubber-modified mixtures. Rubber stability is unacceptably low, but there is no evidence of rutting. This would indicate the stability test is not a valid indicator of field performance for rubber asphalt mixtures of the type investigated;
4. Indirect tensile tests indicate the control has greater strength; and
5. Visual surveys show both materials to be performing well and in good condition.

2.3 FHWA - Sulfur

Description

The term sulfur-extended (SEA), when applied to an asphalt cement, asphalt concrete mix, or pavement, denotes the replacement of a significant portion of the conventionally used asphalt with elemental sulfur. Typically, 20% to 40% of the weight of the asphalt is replaced by sulfur. In 1985, the FHWA organized a task force to conduct a comprehensive SEA field evaluation study (Beatty et al., 1987). The study objectives were (a) to compare the field performance of a representative group of SEA pavements with that of a control group of conventional asphalt concrete pavements (AC) and (b) to determine what differences in performance and in durability existed between the two groups.

Twenty-six SEA projects in eighteen states were selected for evaluation after considering such criteria as geography, climate, sulfur form and content, age, and blending methods. The objective was to obtain a representative set of projects. Table 2-12 summarizes the SEA projects selected. Generally, each project is composed of one or more SEA pavement(s) and of one or more contiguous AC pavements used as controls and built simultaneously with, and to the same specifications as, the SEA pavement(s).

Results

A present condition index (PCI) was determined for each pavement. The PCI is determined through a comprehensive evaluation of visible pavement distress that includes cracking (transverse, longitudinal, and joint reflection) and rutting. A pavement with no visible distresses will have a PCI of 100. The results of the PCI survey are shown in Table 2-13; both the SEA and conventional pavements are, as a group, in satisfactory condition. The predominant distresses were longitudinal, transverse and joint reflection cracking, and some rutting. Alligator cracking was found in the SEA pavements to a greater degree than the control section, but its incidence is small in both types of pavement. No trends were observed in the occurrence of cracking and rutting with variation in the sulfur/asphalt ratio from 20/80 to 40/60. Figures 2-5 and 2-6 suggest that the performance of the SEA pavements was comparable to the AC control group, and that the performance may even be marginally better regardless of the sulfur content of the binder.

Beatty et al. (1987) then used statistical methods to test the significance of observed differences in PCI and to the deduct values between the SEA and the control groups. The analysis concentrated on the effect of sulfur on performance and on durability of the pavements. The students' *t*-test was employed to estimate the significance of the observed differences. The results indicated that no significant differences exist between the SEA and AC control groups; the two types of pavements have performed comparably. Similarly, the performance of the SEA pavement was not significantly influenced by the sulfur content of the paving binder from 20/80 to 40/60. Finally, the researchers attempted to find correlations between the PCI and deduct values with the pavement age and freezing index. No evidence for causal relationships between the age and freezing index and the PCI and deduct values was found.

The results of this study imply that, in most circumstances, the use of sulfur as an extender in asphalt paving mixtures is innocuous and that SEA pavements should perform in a satisfactory manner if they are constructed to proper design with adequate attention to detail.

Conclusions

To summarize, the researchers (Beatty et al., 1987) concluded that:

1. There are no significant differences in performance between SEA and AC pavements. Both types of pavements are performing satisfactorily;

2. The level of sulfur did not have a significant effect on pavement performance or measured levels of distress; and

3. Correlations between SEA and AC pavement and PCI and distress deduct values to pavement age and freezing index was poor.

However, it should be noted that all the pavements were in good condition, which therefore narrowed the range of performance data available for analysis. Secondly, other factors such as the effects of traffic were not included in the analysis.

2.4 California - Styrene-Butadiene-Styrene (SBS)

Description

Two separate test sections were constructed in the California low desert climate to evaluate the age hardening of modified asphalts (Reese, 1989). Tilt-Oven Durability Testing (California Test Method 374) had shown that the modified binders would exhibit less field aging than the control binder. The test sections were constructed on the following two projects:

1. Highway 98 - Imperial County, and
2. Interstate 40 - San Bernardino County.

Table 2-12. Summary of SEA project for FHWA study (Beatty et al., 1987).

State	Review Section Number	Location	Age (Years)[a]	Freezing Index
AZ	850401	Glendale Ave., Phoenix	5.2	0
CA	850601	I-15, West of Baker	3.2	0
CA	860601	Lincoln Ave., Anaheim	4.3	0
CA	860602	Lincoln Ave., Anaheim	4.3	0
DE	851001	US 13 in Greenwood	6.4	0
FL	861201	Southwest 16th Ave. in Gainesville	6.9	0
FL	861202	I-75 North of Gainesville	5.4	0
GA	861301	Bainbridge Bypass (US 27 & US 84)	4.6	0
ID	851601	State Route 14, East of Golden	4.0	500
LA	862201	State Route 22, near Darrow	6.0/7.2[b]	0
ME	852301	I-95, 30 miles so. of Bangor	4.1	1000
ME	852302	I-95, 90 miles no. of Bangor	6.2	2000
MN	862701	Trunk Highway 63, no. of Rochester	7.0	1700
MS	862801	State Route 15, so. of Phila.	4.4	0
NV	853201	US 93-95, no. of Boulder City	8.9	0
NV	853202	US 50 Alternate, near Fernley	5.3	200
NM	853501	US 62/180, near Carlsbad	3.7	0
ND	853801	US 2-82, west of Minot	5.2	2500
PA	854201	Emmaus Ave., near Allentown	4.4	250
TX	854801	I-10, near Fort Stockton	4.2	0
TX	854802	MH153 in College Station	7.4	0
TX	854803	Loop 495, north of Nacogdoches	5.2	0
TX	854804	US 59, near Lufkin	3.2	0
WA	855301	US 2, west of Pullman	6.2	200
WI	865501	State Highway 29, west of Tilleda	3.6	1500
WY	865601	State Route 225, west of Cheyenne	3.7	1250

[a]Age of pavement at time of evaluation.

[b]Ages varied for two design sections.

Table 2-13. Mean PCI summary for FHWA (Beatty et al., 1987).

State Code/ Review Section Number		Mean Present Condition Index for Each Sulfur/Asphalt Ratio Represented					
		0/100	20/80	25/75	30/70	35/65	40/60
AZ	850401	95.0			98.0		
CA	850601	100.0	100.0				100.0
DE	851001	90.0			83.5		
ID	851601	96.6			100.0		
ME	852301	87.0	92.0		84.0		
ME	852302	88.0			82.0		
NV	853201	85.0			88.5		
NV	853202	87.0		89.5			
NM	853501	94.0			100.0		
ND	853801	83.5		80.0	83.0		
PA	854201	95.5			90.0		
TX	854801	100.0			100.0		
TX	854802	57.0			80.0		
TX	854803	85.0				80.0	
TX	854804	82.0			82.0		
WA	855301	90.0			89.0		85.0
CA	860601	97.0			93.0		
CA	860602	100.0			100.0		
FL	861201	96.0			97.0		
FL	861202	90.0			74.0		52.0
GA	861301	86.7			90.4		
LA	862201	90.0			90.0		87.0
MN	862701	65.5					79.0
MS	862801	100.0			100.0		100.0
WI	865501	71.0			85.3		
WY	865601	82.0	84.0				
Mean		88.2	92.0	84.8	90.0	80.0	83.8

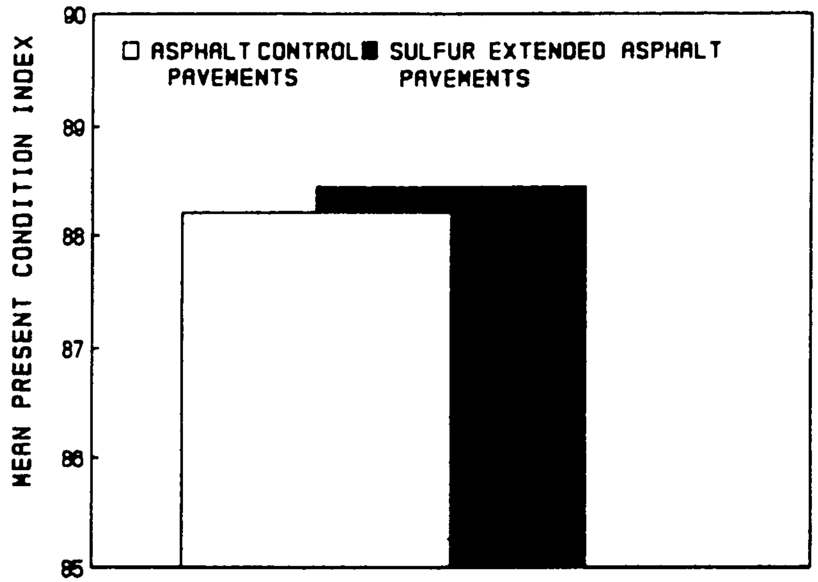


Figure 2-5. Comparison of the *PCI* for SEA and AC pavement sections for FHWA study (Beatty et al., 1987).

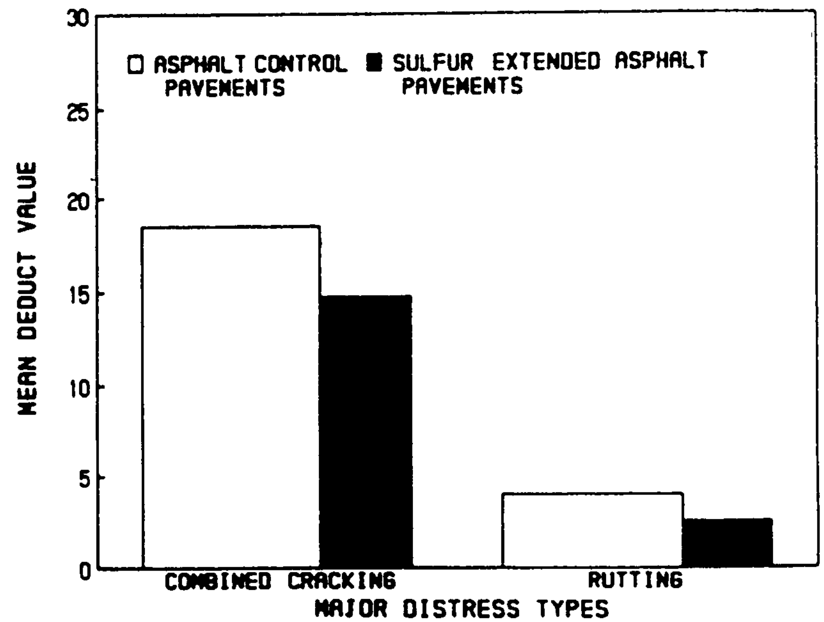


Figure 2-6. Comparison of mean distress deduct values for SEA and AC pavement sections for FHWA study (Beatty et al.,1987).

Highway 98

During December, 1986, a half-mile test section was constructed with modified asphalt on Highway 98 in Imperial County near Ocotillo. A half-mile control section was also placed. The construction was an overlay consisting of a 0.13-foot leveling course with a 0.15-foot surface course. The existing asphalt pavement was block cracked over the entire surface. The control AR-4000 asphalt was supplied by Paramount Petroleum and the modified asphalt was a Witco proprietary blend of paving grade asphalt, extender oil, and styrene-butadiene-styrene block copolymer prepared to meet an AR-4000 specification and to satisfy the additional requirements listed below:

Tests on Residue from Calif. Tilt-Oven Durability Test (Ca. Method 374)	AASHTO Method	Requirements
Absolute Viscosity @ 140°F	T-202	75,000 poise maximum
Penetration @ 77°F	T-49	25 dmm minimum
Ductility @ 77°F	T-51	30 cm minimum

Interstate 40

A test section for three modified binders was placed on Interstate 40 in San Bernardino County near Needles. The test sections were overlays consisting of 0.15 foot surface course. Prior to overlays, an 0.2-foot of the westbound outside lane was milled and replaced with new asphalt concrete. The control AR-4000 was supplied by Newhall Refining Company and the modified asphalts were supplied by Asphalt Supply & Services Company (ASSCO), Edgington, and Witco. The modified asphalts were prepared to meet AR-4000 specifications and the additional requirements listed above for Highway 98.

Results

Highway 98

Asphalt cement properties before and after laboratory and field aging are shown on Table 2-14. The modified binder has a much higher penetration and similar viscosity, therefore, it has a lower temperature susceptibility than the unmodified binder. After the Tilt-Oven Durability Test and one and two years of field aging, the control asphalt had hardened significantly more than the modifier section. After two years, the recovered modified binder had the viscosity of an AR-8000 asphalt. Both control and modified binders after the Tilt-Oven Test had higher viscosities and lower penetrations and ductilities than after two years of field aging. Qualitatively, the differences in magnitude between the control and modified binder properties were similar. Neither of the sections had shown any distress as of February 1989.

Table 2-14. Asphalt cement properties before and after laboratory and field aging on California Highway 98 (after Reese 1989).

Property	Control	Witco
Original Recovered Asphalt Properties		
Pen @ 77° F (dmm)	56	145
Abs. Visc. @ 140° F (p.)	2,800	2,200
Ductility @ 77° F (cm.)	100+	100+
Laboratory Aged Asphalt Properties after Tilt Oven Test		
Pen @ 77° F (dmm)	10	38
Abs. Visc. @ 140° F (p.)	92,000	18,000
Ductility @ 77° F (cm.)	6	55
Field Recovered Asphalt Properties @ 1 Year		
Pen @ 77° F (dmm)	15	76
Abs. Visc. @ 140° F (p.)	20,000	5,700
Ductility @ 77° F (cm.)	13	100+
Field Recovered Asphalt Properties @ 2 Years		
Pen @ 77° F (dmm)	9	63
Abs. Visc. @ 140° F (p.)	53,000	8,000
Ductility @ 77° F (cm.)	8	95
Asphalt Content (%)	-	5.4
Relative Compaction (%)	-	97

Interstate 40

Asphalt cement properties before and after laboratory and field aging are shown on Table 2-15. The binder produced by ASSCO did not meet the ductility requirement of the Tilt-Oven specification although the prequalification sample did. After laboratory Tilt-Oven aging and field aging for one year, the control and ASSCO binders showed similar properties and aged considerably more than the Edgington and Witco binders. As in the Highway 98 project, the Tilt-Oven Test hardened the binders more than one year of field aging, although qualitatively the differences in magnitude between control and modified binders were comparable. As in the Highway 98 project, none of the sections showed any distress as of February 1989.

Conclusions

For both projects, age hardening in the field as measured by penetration, viscosity, and ductility was considerably less in the modified binder sections than in the control sections. The Tilt-Oven Durability Test age hardened the binders more than after one and two years of field aging however, the test was able to distinguish the relative hardening characteristics of the control and modified binders. This is shown by the fact that the hardening after the test and after field aging was very significant for the control binder and relatively low for the modified binder.

2.5 Colorado - Chemkrete

Description

In the fall of 1981, the state of Colorado constructed two 500-foot Chemkrete test sections on an asphalt widening project in northeastern Colorado (Wood and LaForce, 1984). One 500-foot control section was also placed. The sections were located in flat terrain surrounded by irrigated farmland. The Chemkrete sections contained 7 in. of asphalt concrete. The asphalt concrete thickness of the control section was not stated in the report. An AC-10 blended at the rate of 9 parts AC-10 to 1 part Chemkrete was used. The control section was a 50/50 recycled mix in the bottom layers, with 1.5 in. of virgin mix for the surface. The control section contained 1% hydrated lime added to the aggregate, but lime was omitted in the Chemkrete sections because of potential hardening of the asphalt.

Results

No test data on original asphalt or mix properties are reported; however, cores were taken at several time intervals. Viscosities and penetrations of recovered asphalt and resilient modulus values were measured and are shown in Figures 2-7 through 2-9. Crack surveys and rut depth measurements were conducted at the time of coring, and the results are presented in Table 2-16. Recovered penetrations for the top of the cores on the Chemkrete sections are approximately one-half those in the control sections. Recovered viscosities of the top of the cores on the Chemkrete sections are approximately five times higher than the control sections. Recovered penetrations and viscosities from the bottoms of the cores were similar for both Chemkrete and control sections. Resilient moduli values for the top of the cores in the Chemkrete sections are approximately 30% higher than the control sections. Moduli values for the bottoms of the cores were similar between both Chemkrete and control sections, although there is some scatter in the data.

After the first winter, that is, in April 1982, cracking was observed on the Chemkrete section. Wood and LaForce (1984) state that it was the top layer that cracked during the colder winter months with deeper cracks noted at about 50-ft. intervals. The cracking was not distributed evenly throughout the test sections, and there were variations in extent throughout the test section length. Wood and LaForce (1984) did not state the reason, although they did indicate that it could have been a blending problem. The amount of cracking did not increase much from April to September, but increased significantly during the next winter. The control sections still did not crack after the second winter.

Rut depth measurements were taken by a 3-ft gage, and the average rut depths were 0.1 in. for all sections after the first winter. After the second winter, the average rut depth increased to 0.2 in. for the control section and to 0.12 in. for the Chemkrete section.

Conclusions

Test results on cores indicate that the top asphalt concrete layers in the Chemkrete sections have higher resilient modulus, higher recovered viscosities, and lower penetration than the control section. The values for the bottoms of the cores were approximately the same for both Chemkrete and control sections.

Transverse cracking was noted immediately after the first winter in the Chemkrete sections and increased again after the second winter. No transverse cracking was observed in the control section during the study period. Rut depths were slightly higher in the control section, 0.2 in. versus 0.12 in., after the second winter.

Wood and LaForce (1984) indicate that the higher stiffness and viscosity and the lower penetration of the Chemkrete sections were expected, but the transverse cracking was not. They indicated that the Chemkrete supplier felt there was a production error that doubled the chemical concentration on the project.

Table 2-15. Asphalt cement properties before and after laboratory and field aging on Interstate 40 project in California (after Reese 1989).

Property	Control	ASSCO	Edgington	Witco
Original Recovered Asphalt Properties				
Pen @ 77° F (dmm)	36	50	216	116
Abs. Visc. @ 140° F (p.)	3500	9500*	3100	2300
Ductility @ 77° F (cm.)	100+	80	87	100+
Laboratory Aged Asphalt Properties after Tilt Oven Test				
Pen @ 77° F (dmm.)	10	26	38	37
Abs. Visc. @ 140° F (p.)	98,400	74,000	12,100	12,800
Ductility @ 77° F (cm.)	6	16	61	102
Field Recovered Asphalt Properties @ 1 Year				
Pen @ 77° F (dmm)	16	23	119	96
Abs. Visc. @ 140° F (p.)	21,000	28,000	4,900	2,900
Ductility @ 77° F (cm.)	21	27	100+	100+
Asphalt Content (%)	4.8	4.8	4.6	4.6
Air Voids (%)	-	11.0	10	11.0

*This value was confirmed by Reese (1991). The high value is likely the result of "entanglement" of the polymer in the viscosity tubes. Other straight-walled tubes gave similar results.

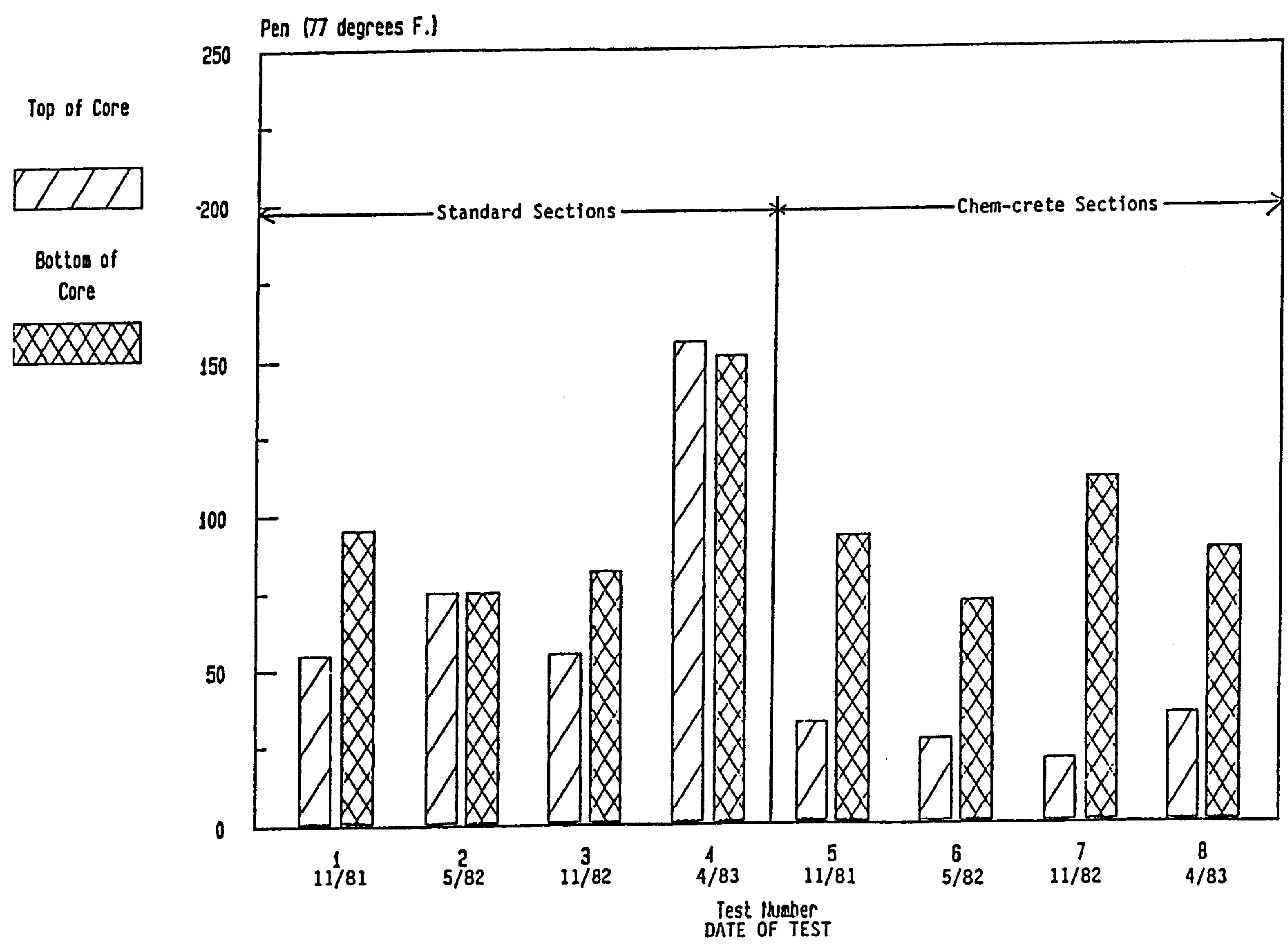


Figure 2-7. Recovered penetrations for Colorado-Chemkrete project (Wood and LaForce, 1984).

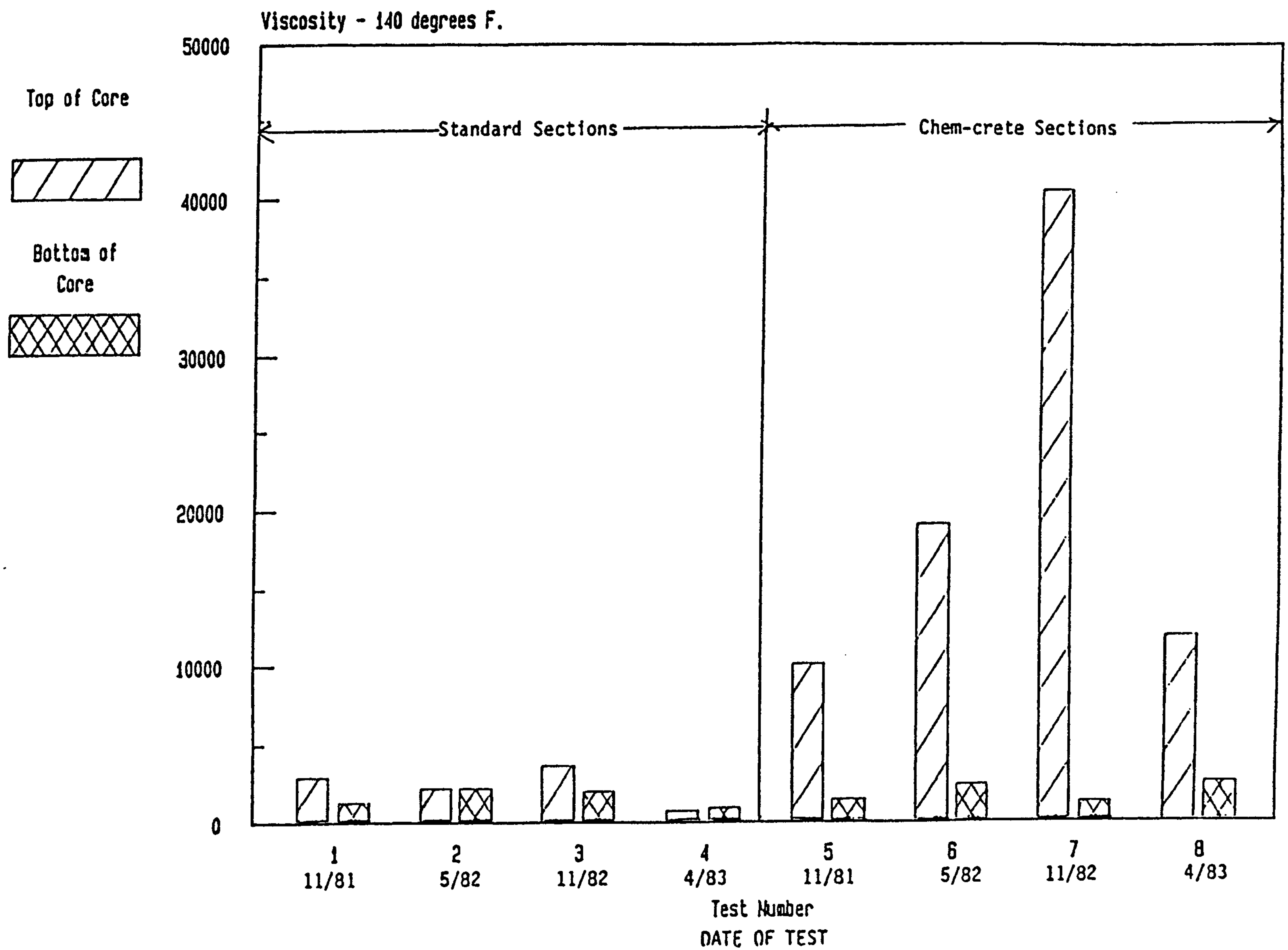


Figure 2-8. Recovered viscosities for Colorado-Chemkrete project (Wood and LaForce, 1984).

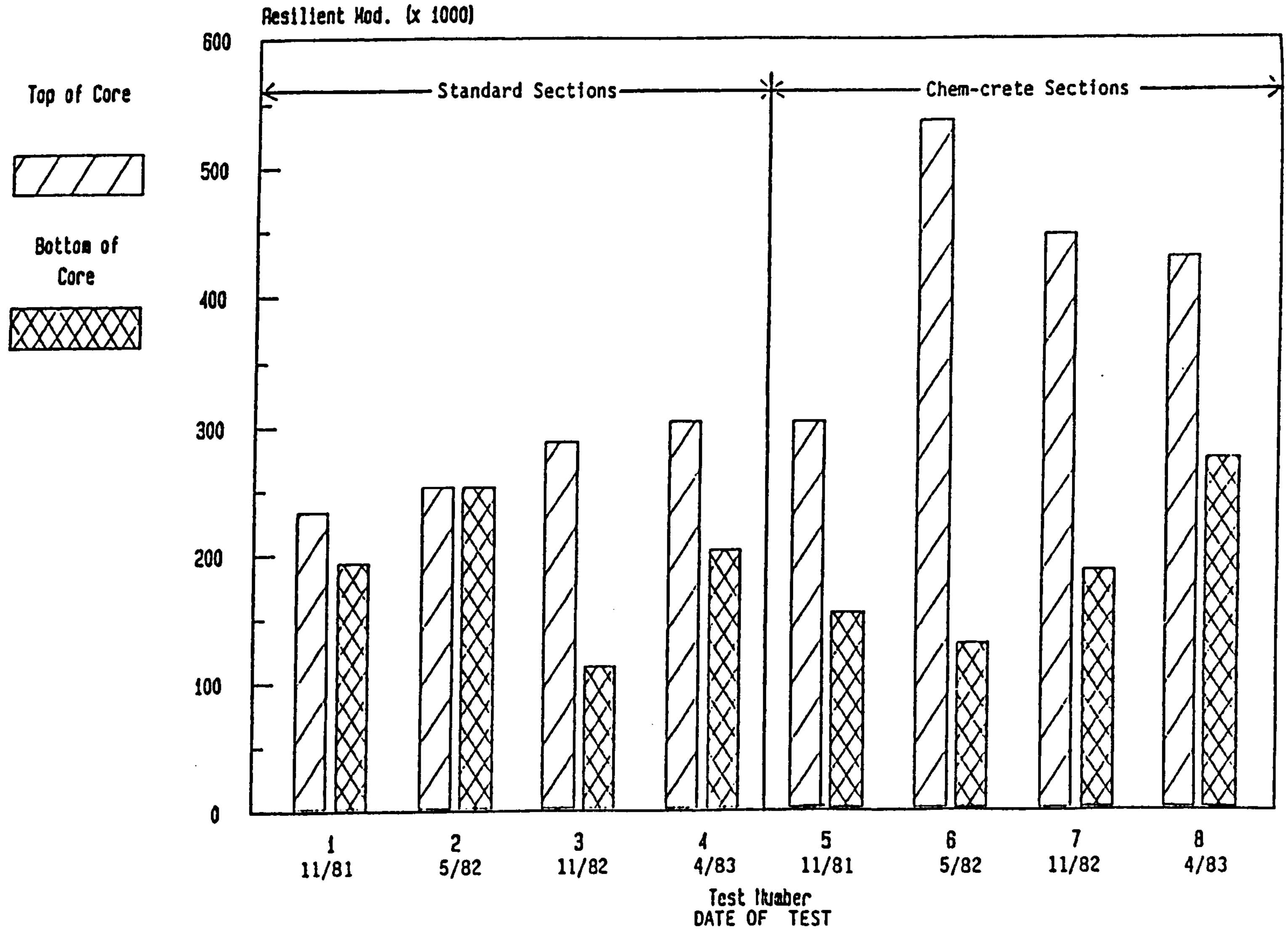


Figure 2-9. Resilient modulus of cores for Colorado-Chemkrete project (Wood and LaForce, 1984).

Table 2-16. **Crack survey and rut depth data for Colorado-Chemkrete project (after Wood and LaForce, 1984).**

	Chemkrete Section 1			Chemkrete Section 2			Control Section		
	Crack Survey Data		Average Rut Depth Data (in.)	Crack Survey Data		Average Rut Depth Data (in.)	Crack Survey Data		Average Rut Depth Data (in.)
Date	Lin. ft.	ft./1000 ft^2		Lin. ft.	ft./1000 ft^2		Lin. ft.	ft./1000 ft^2	
11-24-81	0	0	-	0	0	-	0	0	-
04-27-82	1386	81.4	0.1	146	8.6	0.1	0	0	0.1
09-27-82	-	-	0.1	-	-	0.1	-	-	0.2
09-29-82	1402	82.5	-	148	8.7	-	0	0	-
04-25-83	1712	100.7	0.12	246	14.5	0.12	0	0	0.2

2.6 Illinois - Chemkrete

Description

Three separate projects were built in Illinois (Saner 1987) to evaluate the performance of mixes with Chemkrete modified binder are identified as follows:

1. Illinois 159 Project,
2. US 34 Project, and
3. Illinois 101 Project.

Illinois 159 Project

This project was constructed in 1980 and consisted of widening an existing two lane road into four lanes plus a center turn lane. The widening was 3 in. of asphalt concrete over 8 in. of Portland cement concrete base course. Chemkrete binder was used in both northbound lanes, and conventional binder was used as a control in the southbound lanes. The traffic was uniform in both directions with an ADT of 20,000.

US 34 Project

This project was constructed in 1984 and was a 3-in. overlay of a four lane highway with an ADT of 23,000. Chemkrete was used only in the top 1.5-in. surface course and conventional asphalt was used in the binder course. A small control section was also placed.

Illinois 101 Project

This project consisted of an overlay of rural road in 1985 with an ADT of 650. Three mixes were used: one Class C surface course and two binder courses (Class A and C). Both treated and untreated mixes of each type were placed.

Results

Illinois 159 Project

The results of tests on asphalt binder, mixture, and performance measurements are shown in Table 2-17. The effect of Chemkrete was to reduce original binder viscosity and to increase penetration. In effect, the Chemkrete binder had the viscosity of an AC-10, as opposed to the AC-20 base asphalt. Marshall stability testing was performed on cores. Saner (1987) stated that the manganese content in the Chemkrete binder was 0.25% and that the strength was expected to double between modified and unmodified mixes. Only a one-third strength gain was realized in the surface course and none in the binder. Saner (1987) attributed this gain of 3% to low air voids. Rut depths were essentially the same between Chemkrete and control. The increase in transverse cracks between two and four years performance is reported since a number of transverse cracks were initially present because of the concrete base. Longitudinal cracking, which formed from the surface and propagated down, was also reported. In both cases, the Chemkrete section had more cracking than control.

US 34 Project

Test results for this project are shown in Table 2-18. From the indirect tensile strength data, it can be seen that the temperature susceptibility of the Chemkrete modified mixture was reduced somewhat. No difference in amount, severity, or extent of new cracking was observed between control and Chemkrete sections. No test data were reported.

Illinois 101 Project

Test results and performance data are shown in Table 2-19. The addition of Chemkrete to the AC-5 resulted in a blend with properties of an AC-2.5 asphalt cement. The mix was tender during placement. Indirect tensile strengths at high temperatures for the Chemkrete modified sections were much lower than expected after 15 months. Laboratory aging tests (not reported) had predicted comparable indirect tensile strengths between Chemkrete modified and control mixture. Rut depths are higher in the Chemkrete modified section, and Saner (1987) reported that the differences were a result of lower mix strength at high temperatures. It is probably likely that the lower viscosity of the Chemkrete modified binder contributed to the difference in rut depths. As in the US 34 Project, no difference in amount, extent, or severity of new cracking was observed.

Conclusions

Based on the results of the three evaluations, Saner (1987) concluded that Chemkrete did not improve the performance of Illinois mixes and recommended that no further consideration should be given to the use of Chemkrete.

Table 2-17. Asphalt cement and mixture properties and performance evaluations for Illinois 159 project (from Saner 1987).

Property/Performance	Control	Chemkrete Modified	Difference
Original Asphalt Cement:			
Grade	AC-20	9 parts AC-20, 1 part Chemkrete	
Pen @ 77° F (dmm)	69	103	+34
Abs. Visc. @ 140° F (p.)	1820	995	-825
Kin. Visc. @ 275° F (cs.)	345	344	-1
Manganese Content (%)		0.25	
Marshall Stability (lbs)			
Surface Course			
10 months	1001	1348	+347
26 months	1160	1540	+380
43 months	1022	1421	+399
Binder Course			
10 months	1122	1215	+93
26 months	1360	1310	-50
43 months	1072	1098	+26
Average Rut Depths (in.)			
(outer and inner wheel path)			
2 years	0.14	0.14	0
4 years	0.20	0.14	-0.06
Transverse Cracks			
(increase from 2 to 4 years, %)	6	17	+11
New longitudinal cracks			
(% of section length)	0	54	+54

Table 2-18. **Asphalt cement and mixture properties and performance evaluations for Illinois U.S. 34 project (from Saner 1987).**

Property/Performance	Control	Chemkrete Modified	Difference
Original Asphalt Cement:			
Grade	AC-20	33 parts AC-10 1 part Chemkrete	
Manganese Content (%)		0.06	
Marshall Stability (lbs)			
@ 10 months	1635	2200	+565
Indirect Tensile Strength (psi)			
@ 10 months			
40° F	302.9	288.5	-14.4
125° F	27.1	37.5	+10.4
New Cracking	a	a	a

[a]No difference - actual values and age not reported.

Table 2-19. Asphalt cement and mixture properties and performance evaluations for Illinois 101 project (from Saner 1987).

Property/Performance	Control	Chemkrete Modified	Difference
Original Asphalt Cement:			
Grade	AC-10	AC-5 3% Chemkrete by weight	
Pen @ 77° F (dmm)	-	238	
Abs. Visc. @ 140° F (p.)	-	280	
Kin. Visc. @ 275° F (cs.)	-	152	
Marshal Stability (lbs.)			
@ 15 months			
C - Surface	2177	1468	-709
C - Binder	2230	2185	-45
A - Binder	1812	1334	-478
Indirect Tensile Strength (psi)			
@ 15 months			
@ 40° F:			
C - Surface	338.2	220.5	-117.7
C - Binder	361.6	281.2	-80.4
A - Binder	341.1	189.4	-151.7
@ 125° F:			
C - Surface	31.3	19.8	-11.5
C - Binder	28.6	26.0	-2.6
A - Binder	25.0	23.2	-1.8
Average Rut Depth (in.)			
@ 1 year			
W.B. - Loaded Trucks	0.15	0.24	+0.09
E.B. - Empty Trucks	0.07	0.09	+0.02
New Cracking			
@ 1 year	a	a	a

[a]*No difference - actual values not reported*

2.7 Michigan - Sulfur

Description

In 1977, four one-quarter mile test sections with sulfur-asphalt and two control sections were constructed as part of a resurfacing project on M-18 in Gladwin County (DeFoe, 1983). The sulfur-asphalt test sections had variable sulfur to asphalt ratios and binder contents, as shown in Table 2-20. The purpose of the project was to evaluate sulfur-asphalt rutting and reflection cracking resistance. The resurfacing consisted of a 1.5-in. leveling course and a 1-in. wearing course of an existing asphalt pavement. The existing asphalt pavement was cracked and rutted. An 85/100 pen asphalt was used to make the sulfur-asphalt binder. (No information on traffic is given in the report.)

Results

No original binder or mixture properties are reported. However, core samples were obtained and tested in the laboratory, although the age of the pavement at the time of coring was not reported. Mixture properties reported are presented in Table 2-21. Tensile strength and stiffness modulus were measured by the Indirect Tensile Strength (ITS) test. The loading rate and the temperature were not stated in the report; however, the sulfur-asphalt sections had tensile strength approximately 50% higher as the control sections and stiffness moduli approximately twice as high. Resilient moduli were measured at 0.1 second loading rate at 40° F and 72° F. The sulfur-asphalt mixtures had a higher resilient moduli than control by approximately 30% at 72° F and 50% at 40°F. Contraction coefficients of the sulfur-asphalt mixes were approximately one-half that of control.

Rut depths measured prior to overlay and after one and five years are shown in Figures 2-10 and 2-11. The rut depths in all sections increased slightly over time, and the depths for all sections were approximately equal after one and five years. The rut depths on the original pavement were higher in the sulfur-asphalt test section locations.

Although reflection cracking is not one of the distresses studied in project A-003A, reflection cracking data, which is also reported in DeFoe (1983), indicates that reflective transverse cracking was reduced by 17 % in the sulfur-asphalt sections, as compared to control.

Table 2-20. Composition of sulfur-asphalt mixtures for Michigan sulfur-asphalt test sections (from DeFoe, 1983).

Test Section	Sulfur-to-Asphalt Ratio in Binder, percent by weight	Sulfur-Asphalt Binder in Mixture, percent	
		Leveling Course	Wearing Course
1	30:70	5	5.5
2	30:70	5.5	6
3	50:50	5	5.5
4	50:50	5.5	6

Table 2-21. Properties of core samples from Michigan sulfur-asphalt project (from DeFoe, 1983).

Section	Thickness in.	Unit Weight, P.C.F.	Binder Content, percent		Indirect Tensile Strength, psi			Stiffness Modulus, E, psi			Resilient Modulus, M_r, psi (0.1 sec. loading)		Contraction Coefficient, in./in./°F × 10^5, Composite*
			Wearing	Leveling	Wearing	Leveling	Composite*	Wearing	Leveling	Composite*	72° F	40° F	
Control	2.2	139.8	5.8	---	55	---	---	10,200	---	---	---	---	---
			---	4.5	---	35	---	---	3,800	---	---	---	---
			---	---	---	---	45	---	---	7,600	350,000	1,290,000	1.04
30/70 S-A	2.2	139.6	5.6	---	49	---	---	17,400	---	---	---	---	0.69
			---	4.9	---	40	---	---	8,200	---	---	---	0.34
			---	---	---	---	61	---	---	15,600	410,000	1,880,000	---
50/50 S-A	2.5	140.6	5.8	---	69	---	---	30,900	---	---	---	---	0.48
			---	4.9	---	37	---	---	10,500	---	---	---	0.38
			---	---	---	---	60	---	---	19,200	470,000	1,660,000	---

*Samples consist of wearing and leveling courses together.

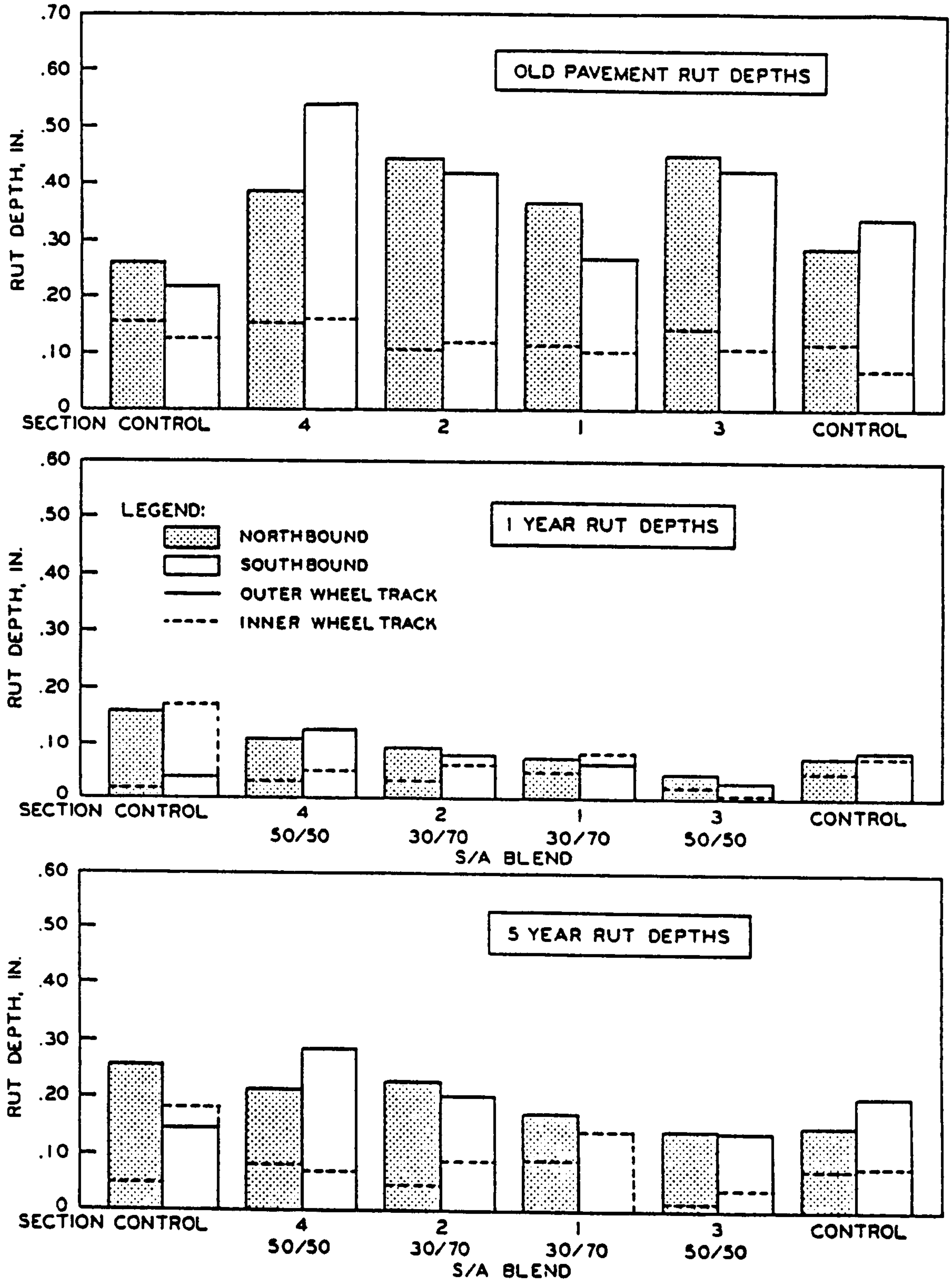


Figure 2-10. Average rut depth on the old pavement and after one and five years for Michigan sulfur-asphalt project (DeFoe, 1983).

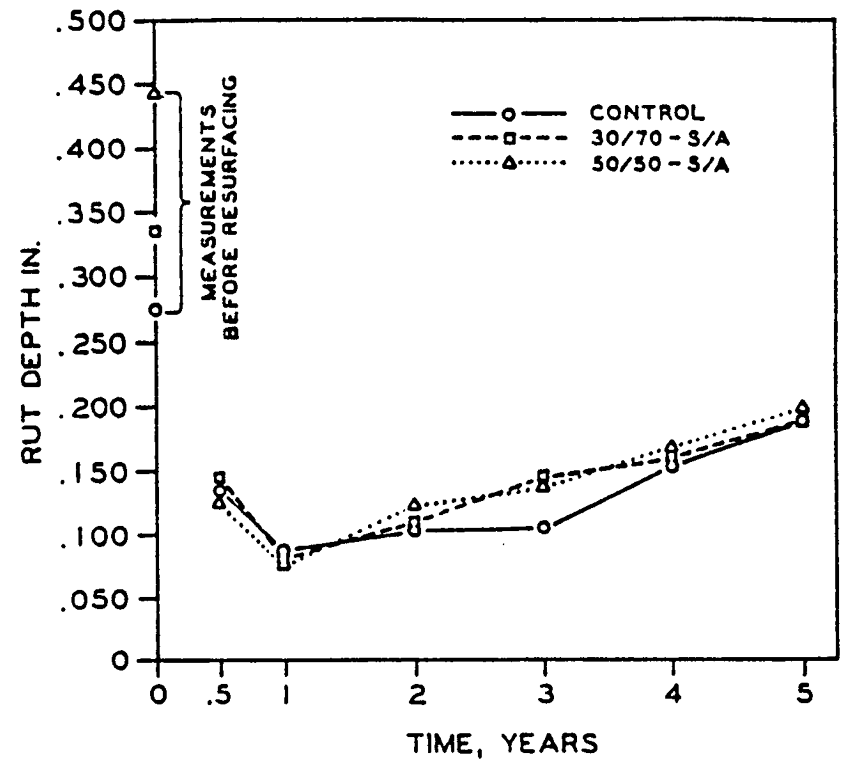


Figure 2-11. History of average rut depths for Michigan sulfur-asphalt project (Defoe, 1983).

Conclusions

DeFoe (1983) concluded that the rutting of the sulfur-asphalt overlay was the same as for the conventional asphalt. He also concluded that the use of sulfur in asphalt mixes resulted in increased tensile strength and stiffness values and in decreased thermal contraction coefficients. From the data, it is apparent that the increased stiffnesses (measured at 40° F and 72° F) did not affect the rutting resistance of the sulfur-asphalt concrete blend.

2.8 Michigan - Rubber

Description

In 1979, eight asphalt rubber test sections and eight paired control sections were placed on M46 in Saginaw County to evaluate rutting and reflection crack resistance (DeFoe, 1985). Two types of tire rubber were used—ground crumb rubber and reclaimed rubber. Ground rubber mixtures were prepared in a batch plant. Fifty-pound bags of ground tire rubber crumb were added to the aggregate in the pug mill immediately prior to asphalt cement addition. The rubber crumb was dry mixed for 10 seconds; the asphalt cement added, and the crumb was mixed an additional 45 seconds. Reclaimed tire rubber was premixed with heated asphalt cement in a heated storage tank then pumped to the batch plant in the normal manner. The test sections were part of an overlay of a jointed concrete pavement with an existing cracked and rutted asphalt concrete overlay. The thickness of the new overlay is not indicated in the report. Two contents of rubber were added: 0.5% and 1.5%. There were two replicates for each section. Aggregate gradations were the same for all control and rubber mixes. Each mix was designed separately using the Marshall procedure. DeFoe (1985) did not indicate the traffic loading on the project.

Results

Indirect tensile strength, tensile strain at failure, and resilient modulus were measured on asphalt concrete sampled during construction. These results are presented in Table 2-22. DeFoe (1985) did not indicate the air void contents of the specimens or field mixtures. In addition, the temperature and loading rate used during the indirect tensile strength test were not reported. Regardless, the control section had the highest tensile strength and resilient modulus and the lowest tensile strain at failure. Of all the rubber mixtures, the 0.5% mixture had the highest tensile strength and modulus and the 1.5% mixture had the lowest tensile strength and modulus (about two-thirds of control values). Both reclaimed rubber mixtures were between these extremes.

Field performance evaluations included rut depth and reflection crack measurements. Rut depth measurements are presented in Figure 2-12. Reflection cracking is outside the scope of project A-003A, as such this information is not considered pertinent to this review. The age of the pavement at the time of measurement is not indicated. The rut depths in the westbound direction were consistently higher than on the eastbound direction. With the exception of sections 7 and 11, less rutting was present on the rubber asphalt sections than on the paired control section. In the westbound lane, sections containing ground rubber and reclaimed rubber reduced rutting by 21% and 8%, respectively, over control. Similar reductions were achieved in the eastbound lane.

Table 2-22. Properties of rubber asphalt mixtures in Michigan (from DeFoe, 1985).

	Mixture				
		Reclaimed		Ground	
Property	Control	1/2%	1-1/2%	1/2%	1-1/2%
Indirect Tensile Strength (psi)	165	130	126	169	115
Tensile Strain at Failure	.0078	.0088	.0105	.0079	.0081
Resilient Modulus (psi) (.1 sec. 72° F)	822,000	625,000	548,000	820,000	442,000

The sections that were 1.5% ground crumb rubber, 2 and 5, disintegrated over the life of the pavement, and frequent patching was required. These mixes had the lowest indirect tensile strengths and resilient moduli of all mixes used on the project, but the values are not abnormally low in comparison with typically reported values.

Conclusions

The addition of rubber decreased the indirect tensile strength and resilient modulus and reduced the tensile strength at failure as compared with the control sections. The asphalt rubber sections also exhibited slightly reduced rutting in comparison to control sections. The sections with the lowest rutting, the 1.5% ground rubber sections, disintegrated over time and required frequent patching.

2.9 University of Nevada - Plastic and Latex

Description

Krater et al. (1988) reported results from pavement test sections containing seven combinations of polyolefin (plastic) and styrene butadiene rubber (SBR) latex placed in five states. The test sections were overlays of existing roadways, all placed in 1986. The states were selected to represent the majority of climatic conditions encountered in the continental United States.

Two different plastics were used. Both plastics contained ethylene and acrylic acid and were in the form of semiclear solid pellets. The latex was incorporated because of possible synergistic effects when combined with the polyolefins. The following seven mixture combinations, listed by amount of additives to total weight of binder system, were used:

1. A control section with no additives,
2. Five percent plastic 1,
3. Five percent plastic 2,
4. Three percent latex,
5. Five percent plastic 1, plus three percent latex,
6. Two percent plastic 1, plus three percent latex, and
7. Two percent plastic 2, plus three percent latex.

The goal of the research was to evaluate:

- Construction feasibility,
- High-temperature performance,
- Low-temperature performance, and
- Moisture susceptibility.

Pavement condition surveys were conducted at each site using the PAVER method. Crack maps were drawn to scale so that cracks occurring in the pavement after construction could be traced to determine amounts of reflective versus thermal cracking. The projects consisted of 1,000-ft sections for each of the seven different modifier combinations listed above. Each 1,000-ft section was divided into ten 100-ft sample units, of which three were selected for sampling of cores as well as the PAVER condition survey.

Texas

This project was constructed on a four-lane portion of US 83 near Mission, Texas, with an ADT of 14,600. Extreme summer temperatures are 90°F to 100°F, winter lows are typically mid-40s and average rainfall is 27 in. The test sections consisted of 2-in. overlays placed over a double asphalt rubber seal coat. Existing distress consisted of low to medium severity transverse, longitudinal, and alligator cracking. The asphalt cement was a Texas Fuel and Oil AC-20 and the aggregate a polished river gravel from the Fordyce pit near Mission, Texas. One percent hydrated lime by dry weight of aggregate was used in the control section to counteract the water susceptibility of the aggregate. Lime was not used in the following sections containing modifiers. This was the only project constructed with lime.

Idaho

This project was a 2-in. overlay constructed on a two-lane road in Ada County, Idaho, with a 1986 ADT of 3,700. Typical summer highs are in the 80°F to 99°F range. Winter temperatures drop into the low teens and below at night. There are nearly 100 air freeze-thaw cycles each year in the area, most of which occur in the spring. Precipitation averages 11.7 in. per year. Existing distress was medium to severe alligator cracking, medium rutting, medium raveling, and low to medium severity longitudinal and transverse cracking. The asphalt used was a Koch AC-10, and the aggregate, a smooth polished river gravel obtained from the Nelson pit in Boise. Because of the mixing problem with the plastic, this

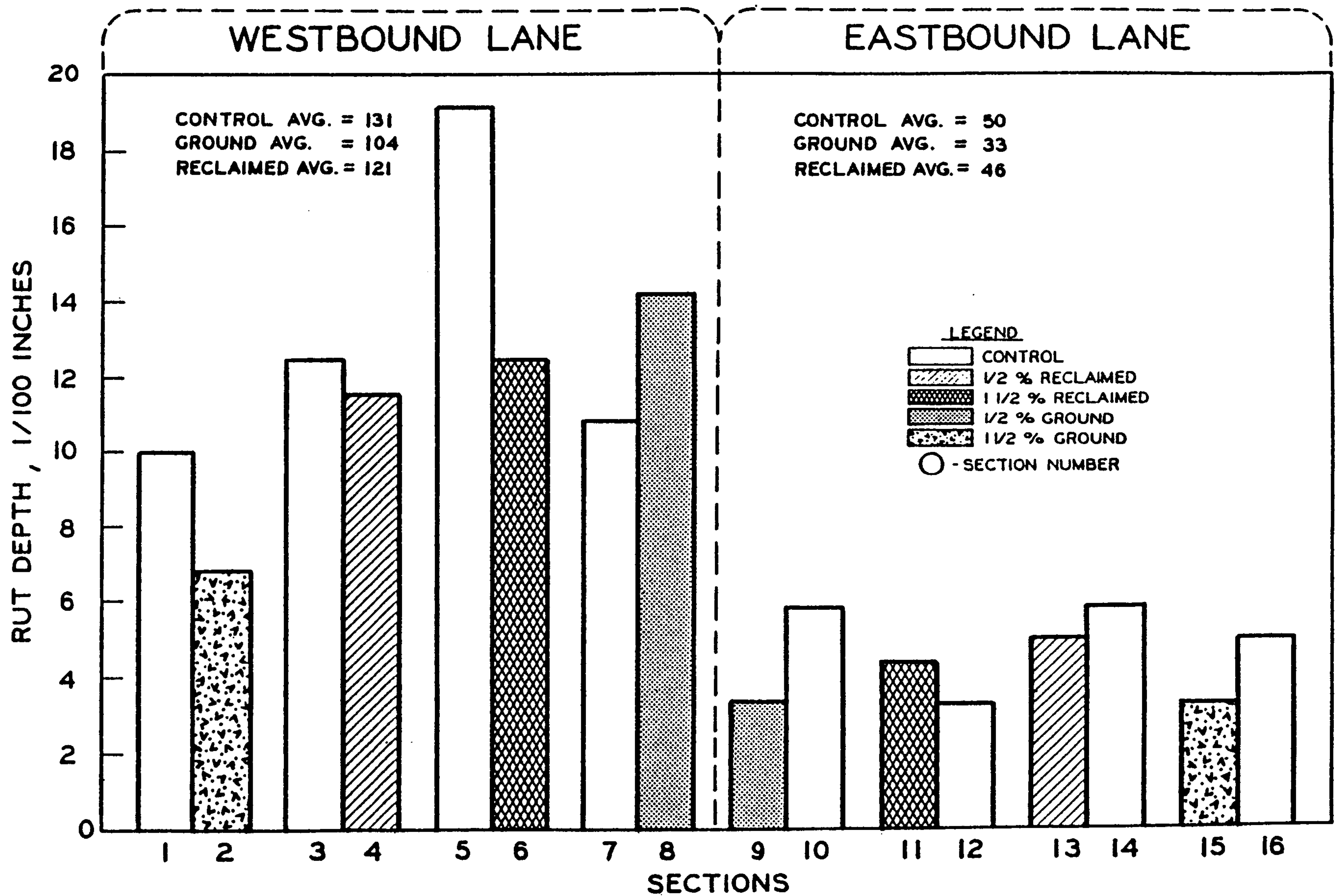


Figure 2-12. Rut-depth measurements for Michigan-Asphalt Rubber test sections and comparable control sections (DeFoe, 1985).

project was scaled back to four sections. (1) the control section, (2) 5% plastic 1, (3) 3% latex, and (4) 3% latex, plus 2% plastic 1.

Maine

The third project was a 2-in. overlay of the southbound travel lane of I-95 in Bangor, Maine. I-95 is a four-lane divided facility with an ADT of 6,695. Summer temperatures range from the mid to the upper 70s; winter temperatures drop into the low teens and below at night. Precipitation averages 41.6 in. per year. The overlay was placed on a milled and leveled surface. Existing distress consisted of medium to high severity transverse cracking even on the milled surface. The asphalt used was an Irving Oil AC-20, and the aggregate a mixture of coarse sand from the Frink pit in Hermon, Maine, and a ledge sand and stone from the Odline Road quarry in Hermon.

Alabama

The fourth project was a 2-in. overlay on a four-lane portion of US-231 in Huntsville, Alabama with a 1986 ADT of 12,000. Summer daytime temperatures range from the low to upper 90s. Winter temperatures drop into the 30s at night. Precipitation averages 54.6 in. per year. Distress present before construction was low to high severity raveling. The asphalt was an AC-30 from the Hunt Refining Company, and the aggregate a Vulcan materials crushed gravel and coarse sand with 10% aggregate lime.

Michigan

This project was a 2-in. overlay with a 1-in. leveling course placed on State Route M-35 near Marquette, Michigan. M-35 is a two lane facility with an ADT of 2,900. Summer daytime temperatures are in the low to upper 70s; winter temperatures drop to near zero at night. Precipitation averages 30.4 in. each year. The existing surface was milled prior to overlay, and existing distress included extensive low to medium severity longitudinal and transverse cracking with some alligator cracking. The asphalt was a 120/150 pen asphalt imported from Spain, and the aggregate a coarse gravel.

Nine cores were extracted from each test section and tested in accordance with the outline on Figure 2-13. The first group of three cores were tested for resilient modulus at temperatures of 10°F, 34°F, 77°F, and 104°F. Testing was performed according to ASTM D4123, with a load cycle of 0.1-sec applied load and a 3-sec pause between loads. Indirect tensile strength was also determined. No loading rate was specified. Theoretical maximum specific gravities were determined by ASTM D2041 and air voids were calculated. The other six samples were run through one cycle of the Lottman (1982) accelerated condition procedure. Resilient modulus values and indirect tensile strengths were determined for dry and wet conditions at 77°F.

Results

Test results for core samples are presented in Tables 2-23 through 2-27. Results from 104°F resilient modulus testing showed that 5% plastic 2 and 2% plastic 1, plus 3% latex provided the highest overall improvement over control mixtures. However, no one modifier consistently provided the high increases in 104°F resilient modulus for all projects. Krater et al. (1988) cautioned, however, that air void contents were great enough to account for the variances in test data.

Krater et al. (1988) found that a majority of the results from 10°F and 34°F resilient modulus testing were within 10% of each other and within one standard deviation. Indirect tensile strengths were measured at 10°F on Alabama cores and were higher for the modified mixes.

Resilient moduli at 77°F are consistently higher for modified mixes than for controlled mixtures. Indirect tensile strengths are approximately the same for both modified and control mixtures.

Results from water sensitivity testing (Tables 2-28 and 2-29) showed that the modifiers did not improve resilient modulus or indirect tensile strength after conditioning. Because resilient modulus and indirect tensile strengths were higher prior to conditioning, retained strengths for the modified mixtures were lower than for control mixtures. It should be recalled that the control mixture for Texas contained 1% hydrated lime.

No field condition survey results are presented in the report. Krater et al. (1988) indicate that follow-up testing was expected to continue for five years from 1986. Cores were to be taken at 1, 3, and 5 years, and PAVER condition surveys were to be conducted annually.

Conclusions

Krater et al. (1988) drew the following five conclusions based on their research:

1. Resistance to thermal cracking should not decline with addition of a combination of polyolefin and latex;

2. Properties of modified mixtures are asphalt source dependent, and mix designs should be performed each time a new modifier is used to test for compatibility with the asphalt cement;

3. No one best polyolefin or polyolefin-latex combination will improve all properties for any one mix;

4. To gain the maximum benefit from the inclusion of modifiers, good construction practices, including close attention to air void content, are necessary; and

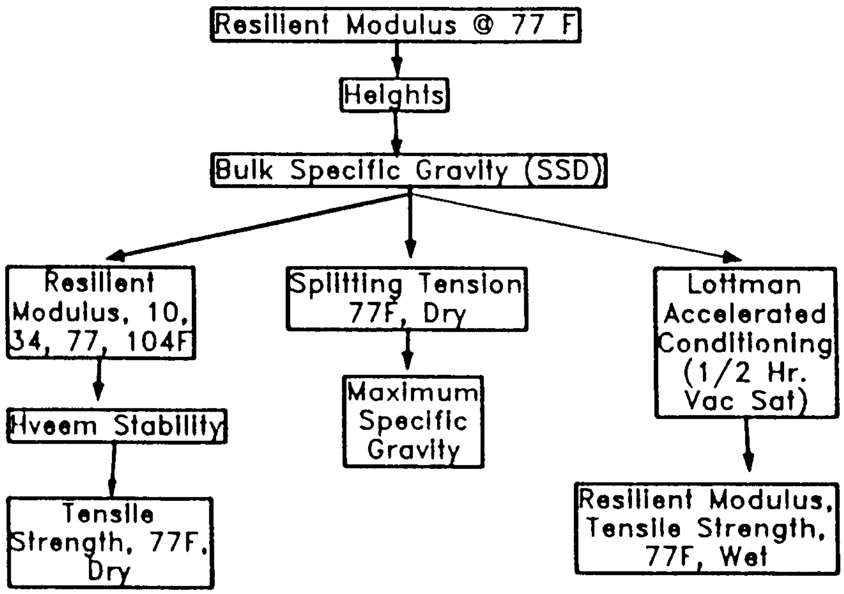


Figure 2-13. Testing outline (Krater et al., 1988).

Table 2-23. Results for Texas cores (Krater et al., 1988).

Test Section Number	Mixture	Resilient Modulus 10°F ksi	34°F ksi	77°F psi	104°F psi	Hveem Stability	Tensile Strength 77°F, psi	Air Voids percent
I	CONTROL	3,331	1,932	271,820	54,480	---	114.1	4.7
V	5%P-1	5,145	1,694	276,040	66,470	---	90.7	4.6
VI	5%P-2	3,123	3,193	334,400	65,890	---	127.2	4.2
II	3%LATEX	3,464	2,043	298,450	58,690	---	110.4	4.9
IV	5%P1+3%L	3,796	1,505	310,360	77,390	---	111.2	4.6
III	2%P1+3%L	3,717	2,013	343,590	66,890	---	106.2	5.7
VII	2%P2+3%L	3,518	2,904	361,980	60,880	---	126.0	3.1

Note: (---) indicates data not available.

Table 2-24. Results for Idaho cores (Krater et al., 1988).

Test Section Number	Mixture	Resilient Modulus 10 F ksi	34°F ksi	77°F psi	104°F psi	Hveem Stability	Tensile Strength 77°F, psi	Air Voids percent
I & IV	CONTROL	1,839	1,586	91,616	13,086	---	63.5	9.7
II	5%P-1	1,617	1,081	125,228	22,627	---	58.0	9.4
	5%P-2	---	---	---	---	---	---	---
III	3%LATEX	1,685	2,058	132,027	24,482	---	66.1	10.7
	5%P1+3%L	---	---	---	---	---	---	---
V	2%P1+3%L	3,454	---	165,334	---	---	---	8.2
	2%P2+3%L	---	---	---	---	---	---	---

Note: (---) indicates data not available.

Table 2-25. Results for Maine cores (Krater et al., 1988).

Test Section Number	Mixture	Resilient Modulus 10°F ksi	34°F ksi	77°F psi	104°F psi	Hveem Stability	Tensile Strength 77°F, psi	Air Voids percent
I	CONTROL	3,302	2,304	102,748	19,187	---	60.8	5.2
II	5%P-1	3,195	2,412	197,378	31,015	---	67.6	5.7
III	5%P-2	2,993	3,104	209,760	41,136	---	67.6	5.7
IV	3%LATEX	3,447	2,600	167,430	29,712	---	67.2	5.0
VII	5%P1+3%L	1,996	1,980	211,008	26,916	---	62.3	6.6
VI	2%P1+3%L	2,869	3,658	210,232	29,841	---	64.4	7.4
V	2%P2+3%L	2,951	3,173	206,651	32,908	---	70.2	5.5

Note: (---) indicates data not available

Table 2-26. Results for Alabama cores (Krater et al., 1988).

Test Section Number	Mixture	Resilient Modulus 10°F ksi	34°F ksi	77°F psi	104°F psi	Hveem Stability	Tensile Strength 10°F, psi	Air Voids percent
I	CONTROL	2,352	2,411	143,071	32,768	11.1	189.6	13.5
II	5%P-1	2,766	1,643	171,360	39,073	20.3	266.7	10.7
III	5%P-2	2,184	1,376	160,766	45,871	---	---	14.2
IV	3%LATEX	2,059	1,303	185,839	44,507	---	---	12.8
VII	5%P1+3%L	2,264	2,284	215,290	37,502	19.0	277.8	11.4
VI	2%P1+3%L	2,362	2,560	230,341	53,015	14.8	274.5	11.1
V	2%P2+3%L	2,797	1,473	200,889	53,715	21.5	288.0	10.6

Note: (---) Indicates data not available.

Table 2-27. Results for Michigan cores (Krater et al., 1988).

Test Section Number	Mixture	Resilient Modulus 77°F psi	10°F ksi	34°F ksi	104°F psi	Hveem Stability	Tensile Strength 77°F, psi	Air Voids percent
I	CONTROL	74,237	2,831	1,186	12,803	3.75" COR	51.5	3.2
V	5%P-1	94,803	2,956	1,244	18,201	N/A	50.1	5.9
VI	5%P-2	93,327	2,629	1,562	13,124	N/A	40.9	6.3
II	3%LATEX	129,125	3,123	1,443	17,635	N/A	53.9	4.4
IV	5%P1+3%L	74,332	2,772	1,206	13,494	N/A	46.4	5.7
III	2%P1+3%L	155,395	3,108	1,345	24,097	N/A	65.0	4.3
VII	2%P2+3%L	148,670	3,466	1,704	22,407	N/A	60.7	4.7

Table 2-28. Water sensitivity test results for Texas cores (Krater et al., 1988).

Test Section Number	Mixture	Resilient Modulus, psi 77°F Before	77°F After	Retained Strength percent	Tensile Strength, psi 77°F Before	77°F After	Retained Strength percent	Air Voids percent
I	CONTROL	271,820	120,990	44.5	114.1	88.8	77.8	4.7
V	5%P-1	276,040	74,960	27.2	90.7	33.5	36.9	4.6
VI	5%P-2	334,400	149,420	44.7	127.2	70.8	55.7	4.2
II	3%LATEX	298,450	59,010	19.8	110.4	33.3	30.2	4.9
IV	5%P1+3%L	310,360	104,500	33.7	111.2	48.7	43.8	4.6
III	2%P1+3%L	343,590	83,340	24.3	106.2	36.2	34.1	5.7
VII	2%P2+3%L	361,980	135,040	37.3	126.0	51.0	40.5	3.1

Table 2-29. Water sensitivity test results for Idaho cores (Krater et al., 1988).

Test Section Number	Mixture	Resilient Modulus, psi 77°F Before Lottman	77°F After Lottman	Retained Strength percent	Tensile Strength, psi 77°F Before Lottman	77°F After Lottman	Retained Strength percent	Air Voids percent
I & IV	CONTROL	91,616	46,733	51.0	63.5	36.2	56.9	9.7
II	5%P-1	125,228	56,620	45.2	58.0	39.7	68.5	9.4
	5%P-2	---	---	---	---	---	---	---
III	3%LATEX	132,027	45,758	34.7	66.1	29.2	44.2	10.7
	5%P1+3%L	---	---	---	---	---	---	---
V	2%P1+3%L	165,334	---	---	---	---	---	8.2
	2%P2+3%L	---	---	---	---	---	---	---

Note: (---) indicates data not available.

5. Further follow-up testing during the next five years of the project will yield much more significant results.

2.10 Montana Big Timber - Chemkrete and Carbon Black

Description

In 1983, Montana constructed 20 test sections on Interstate 90 in the south-central plains of Montana (Jennings et al., 1988). The main variables in the test sections were the asphalt source and the type of additive. This study has previously been reviewed by Finn et al. (1990); however, that review did not emphasize the properties of the modified binders.

Details concerning the site, pavement structure, and traffic are included in Finn et al. (1990) and are not presented here. Asphalt came from four refiners. According to Jennings et al. (1988), the asphalt grade, with only three exceptions, was a 120/150 pen asphalt which met Montana's standard specifications. The exceptions were a 85/100 pen section from Refiner B, a section with 200/300 pen asphalt from Refiner B containing Chemkrete and a 200/300 pen asphalt from Refiner B containing Microfil 8 (carbon black).

Test sections containing 1.5% fly ash, 1.5% hydrated lime, and 0.5% liquid antistrip additive (ACRA) were constructed from each asphalt source in addition to control sections with no additives. The mixture variables for this project are shown in Table 2-30. Dry additives (hydrated lime, fly ash, Microfil 8) were fed directly into the drum dryer, where the first contact was with the asphalt cement. Liquid additives (antistripping agent, Chemkrete) were added to the asphalt cement by means of an in-line feeder located just before the entrance to the mixture.

Results

Cores were obtained immediately after construction, and the associated recovered asphalt properties and mixture properties are shown on Table 2-31. Performance data obtained four years after construction are shown on Tables 2-32 and 2-33. Test section 19 (Chemkrete) does not appear on the tables since this section began to experience rutting in the driving lane soon after it was opened to traffic. The rutting was not uniform and had reportedly reached depths of 0.75-in. at some places when an overlay was applied to the driving lane for reasons of safety. Only two or three short random cracks were found. Jennings et al. (1988) believed these unusual problems with Chemkrete probably stem from several sources: the use of asphalt that was too soft (200/300 pen); over-rolling, which may have sealed the surface to access of oxygen; inadequate curing time; or possibly an improper combination of asphalt type and carrier for Chemkrete.

Tables 2-34 and 2-35 show transverse crack and rutting measurements for control, lime, fly ash, and antistripping additive mixtures. From the transverse cracking data, Jennings et al. (1988) concluded that the effects of additives are dependent on both the type of additive and the refiner, or crude source. The researchers concluded that cracking performance of a given asphalt may be beneficially altered by the proper choice of

Table 2-30. Components of Montana test sections (Jennings et al., 1988).

	Additive			
Refiner	None	Lime	Fly Ash	Antistripping Agent
C[a]	1	2	4	3
B[b]	15	16	14	17
A[c]	6	8	5	7
D[d]	10	9	12	11

NOTE: All are 120-150 grade AC.
[a]Blend.
[b]200-300 with Microfil 8.
[c]200-300 with Chemcrete.
[d]85-100.

Table 2-31. Results from upper lift of Montana cores (Jennings et al., 1988).

Test Section	Components (asphalt/ additive)	AC (%) Design	AC (%) In-Place	Additive (%) Design	Additive (%) Constructed	Marshall Stability, Field	Marshall Flow, Field	Voids In-Place (%)	Penetration, 77°F (recovered)	Viscosity, 275°F (cSt)	Ductility, 40°F (cm)
1	C	6.6	6.2			1,637	8	8.9	60	437	9
2	C/lime	6.2	6.5	1.5	1.5	1,876	9	9.5	47	441	7
3	C/antistrip	6.6	6.9	1.0	1.0	1,952	8	9.3	50	440	7
4	C/fly ash	6.2	6.0	1.5	1.9	1,762	8	8.7	63	410	10
5	A/fly ash	6.3	7.2	1.5	1.8	1,801	8	6.2	77	344	26
6	A	6.8	7.6			1,419	8	6.1	69	368	17
7	A/antistrip	6.8	8.0	1.0	0.9	1,502	9	5.8	75	340	24
8	A/lime	6.6	7.2	1.5	1.6	1,967	9	8.5	54	421	8
9	D/lime	6.3	6.7	1.5	1.5	1,874	8	7.6	64	463	7
10	D	6.6	6.6			1,661	9	9.0	78	378	6
11	D/antistrip	6.5	7.1	1.0	0.9	1,432	9	8.9	54	409	6
12	D/fly ash	6.3	6.9	1.5	1.9	1,727	8	7.8	54	437	6
13	Blend	6.5	6.8			1,638	7	8.5	89	263	39
14	B/fly ash	6.2	7.5	1.5	1.8	1,763	8	7.7	62	318	10
15	B	6.8	6.8			1,814	9	6.4	59	361	7
16	B/lime	6.2	6.5	1.5	1.6	1,602	9	10.8	61	328	8
17	B/antistrip	6.8	7.0	1.0	0.9	1,551	9	6.8	84	249	28
18	85/100	6.5	6.4			1,777	8	8.9	52	396	7
20	200-300 Microfil 8	6.2	6.9	1.0	0.9	1,576	8	9.5	57	317	7

additives. From the rutting data, they stated that the results are not readily related to additive type because of the confounding effects of asphalt content and percent of air voids.

Figures 2-14 and 2-15 present the results of Tables 2-34 and 2-35 in graphical form; however, the measurements from the Microfil 8 (carbon black) section were also included. Comparing transverse crack counts from the carbon black section with the control section (Refiner B), it can be seen that transverse cracking was reduced by about two-thirds. Rut depths for the carbon black section are approximately half that of its companion control section. Although there are possible confounding effects of air voids (9.5% for carbon black, 6.4% for control) and asphalt content (0.7% higher than design for carbon black, at design asphalt content for control), the effect of carbon black on rutting resistance could be significant.

Conclusions

Conclusions by Jennings et al. (1988) relative to the effects of asphalt modifiers on the performance of test sections are summarized below:

- The effect of additives is both asphalt and additive dependent. The underlying chemical cause of this dependency is unknown.

- Within the 120/150 pen asphalt grade, both rutting and cracking differ with asphalt source. Performance is modified by additives.

2.11 Ontario - Sulfur

Description

In 1975, the first of three test roads was constructed by the Ontario Ministry of Transportation and Communications (Fromm and Kennepohl, 1979; Fromm et al., 1981). One of the objectives of this project was to see if the laboratory results could be confirmed in the field. Laboratory results had indicated that sulfur could produce a superior product that would have less tendency to crack at low temperatures.

Test Road 1 had two sulfur percentages that were used in the mix, 40% and 50%. A 150/200 pen grade asphalt cement was used for the test road with a separate portion constructed with 300/400 pen grade asphalt at the end of the test section. The average annual daily traffic (AADT) was 950 with 20% heavy logging trucks. The second test road was constructed in 1977 in the northern part of the province. Again, the road was designed to test a normal grade of asphalt for this area, which is a 150/200 pen, against softer asphalt, 300/400 pen, both containing 40% sulfur by weight of asphalt. Two surface thicknesses were used—1.5 in. and 3 in. The AADT was 1150 with 20% heavy logging trucks. In 1978, the third test road was constructed in the warmer, southern part of the province under heavy traffic. This was a full-depth section where heavy traffic was in operation at all times of the year. The AADT was 11,600 daily. The normal grade of asphalt generally used in this area was 85/100 pen. A softer grade of asphalt would help

Table 2-32. Rutting measurements for Montana (Jennings et al., 1988).

Section No.	Components (asphalt/additive)	Rut Depth (in.)	
10	D	0.16	
12	D/fly ash	0.16	
2	C/lime	0.22	
16	B/lime	0.22	$\lesssim 0.25$
20	200-300/Microfil 8	0.24	
1	C	0.25	
18	85-100	0.25	
9	D/lime	0.27	
4	C/fly ash	0.33	
11	D/antistrip	0.33	
17	B/antistrip	0.34	$\lesssim 0.5$
3	C/antistrip	0.38	
8	A/lime	0.39	
13	Blend	0.43	
5	A/fly ash	0.52	
14	B/fly ash	0.52	
7	A/antistrip	0.56	> 0.5
15	B	0.56	
6	A	0.62	

Table 2-33. Crack counts for Montana (Jennings et al., 1988).

Section No.	Components (asphalt/additive)	Total Cracks
2	C/lime	0
13	Blend	0
1	C	1
3	C/antistrip	1
12	D/fly ash	2
20	200-300/Microfil 8	8
9	D/lime	7
4	C/fly ash	7
5	A/fly ash	12
6	A	18
10	D	19
16	B/lime	19
11	D/antistrip	23
15	B	23
8	A/lime	28
7	A/antistrip	29
17	B/antistrip	33
14	B/fly ash	36
18	85-100	>52

Table 2-34. **Effect of additives on transverse cracking for Montana (Jennings et al., 1988).**

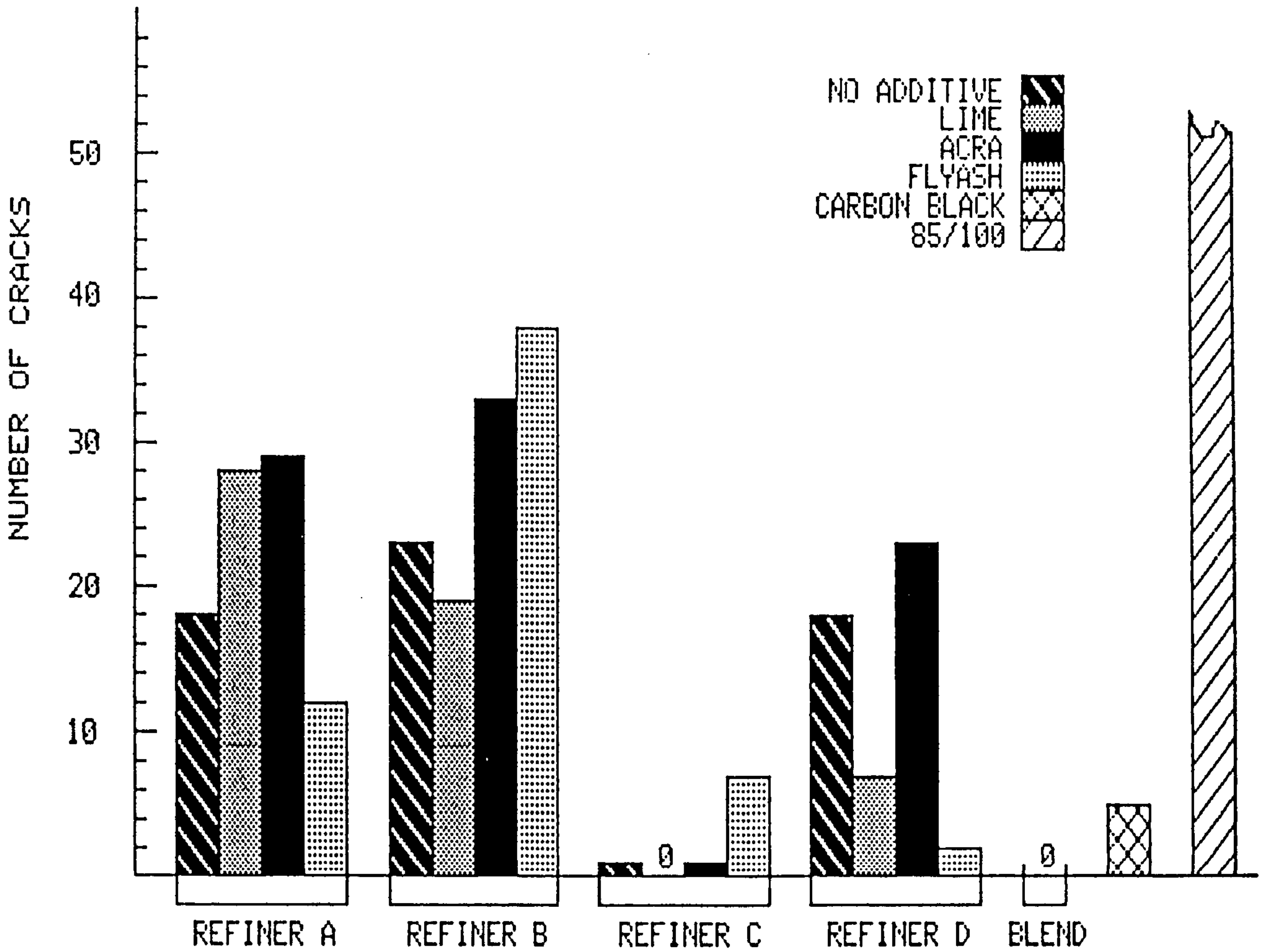
Refiner	Without Additive	With Lime	With Fly Ash	With Antistripping Agent
C	1	0	7	1
D	19	7	2	23
B	23	19	36	33
A	18	28	12	29

NOTE: Total number of transverse cracks per test section (full-width plus one-lane cracks).

Table 2-35. **Effect of additives on rutting for Montana (Jennings et al., 1988).**

Refiner	Without Additive	With Lime	With Fly Ash	With Antistripping Agent
C	0.25[a] (6.2;8.9)[b]	0.22 (6.5;9.5)	0.33 (6.0;8.7)	0.38 (6.9;9.3)
D	0.16 (6.6;9.0)	0.27 (6.7;7.6)	0.16 (6.9;7.8)	0.33 (7.1;8.9)
B	0.56 (6.8;6.4)	0.22 (6.5;10.8)	0.52 (7.5;7.7)	0.34 (7.0;6.8)
A	0.62 (7.6;6.1)	0.39 (7.2;8.5)	0.52 (7.2;6.2)	0.56 (8.0;5.8)

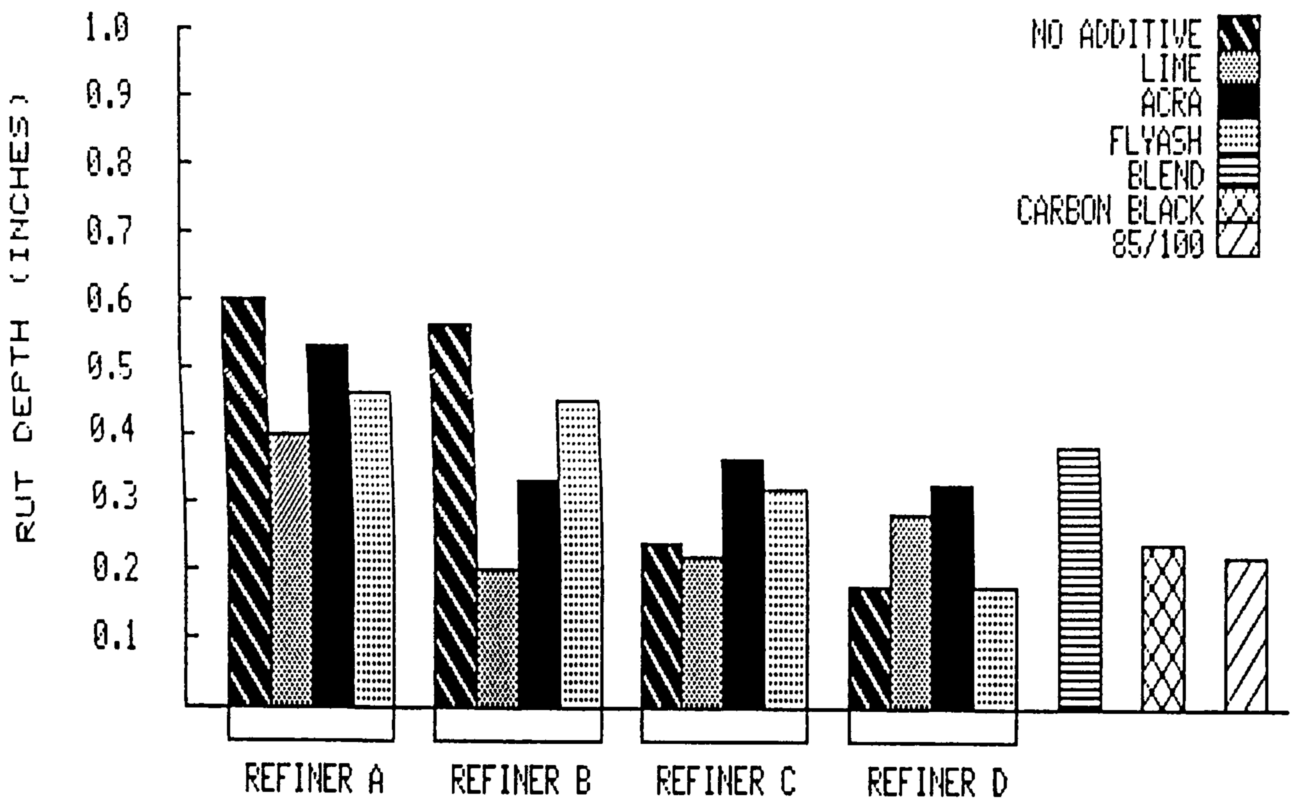
[a] Rut depth in wheelpath, inches.
[b] (% asphalt; % voids).



Notes:

1. ACRA is a liquid anti-stripping additive.
2. Special asphalt blend composed of 75% Refiner B, 20% Refiner D.
3. Carbon black section constructed with 200/300 pen asphalt from Refiner B.
4. 85/100 pen asphalt is from Refiner B.

Figure 2-14. Cracking in Montana pavements (Jennings et al., 1988).



Notes:

1. ACRA is a liquid anti-stripping additive.
2. Special asphalt blend composed of 75% Refiner B, 20% Refiner D.
3. Carbon black section constructed with 200/300 pen asphalt from Refiner B.
4. 85/100 pen asphalt is from Refiner B.

Figure 2-15. Rutting in Montana pavements (Jennings et al., 1988).

Table 2-36. Asphalt cement specifications for Ontario (Fromm & Kennepohl, 1979).

Specification	GRADE					
Designation	85/100		150/200		300/400	
Requirements	Min.	Max.	Min.	Max.	Min.	Max.
Flash Point - C.O.C. °C	450	-	425	-	350	-
Pen. @ 25 °C	85	100	150	200	300	400
Visc. @ 135 °C Kin. Ctsks.	280	-	200	-	120	-
Thin Film Oven Test 50g @ 163 °C, 5 hrs.						
% Loss Wt.	-	0.85	-	1.3	-	1.3
% Ret. Pen. @ 25 °C	47	-	42	-	35	-
Ductility of Residue @ 25 °C, mm	75	-	100	-	100	-
Ductility @ 4 °C 10 mm per min.	6	-	10	-	15	-
Solubility in TCE	99.5	-	99.5	-	99.5	-

Table 2-37. Test Road 1 - Formula and tests, HL-2 sand-asphalt mixes, 150/200 pen asphalt cements (Fromm & Kennepohl, 1979).

Properties	50/50 Hi-Binder	40/60 Hi-Binder	40/60 Lo-Binder	0/100 Control
Stone, % Wt.	-	-	-	-
Sand, % Wt.	60	60	60	60
Scalped Sand, % Wt.	40	40	40	40
Sulfur/Asphalt Ratio, % Wt.	50/50	40/50	40/50	0/100
Binder, % Wt.	9.8	9.0	7.3	7.3
Voids, % Vol.	4.8	4.4	8.5	4.5
VMA, % Vol.	22.2	21.2	21.7	21.1
Marshall Stab. lbs @ 140°F	2588	1511	1304	969
Marshall Flow 0.01 in.	11.5	8.5	8.0	7.0

Table 2-38. Test Road 1 - Formula and tests, HL-4 surface mixes, 150/200 pen asphalt cements (Fromm & Kennepohl, 1979).

Properties	50/50 Hi-Binder	40/60 Hi-Binder	40/60 Lo-Binder	0/100 Control
Stone, % Wt.	45	45	45	45
Sand % Wt.	44	44	44	44
Scalped Sand, % Wt.	11	11	11	11
Sulfur/Asphalt Ratio, % Wt.	50/50	40/50	40/50	0/100
Binder, % Wt.	6.7	6.2	5.0	5.0
Voids, % Vol.	3.7	4.1	6.8	4.6
VMA, % Vol.	16.1	16.0	16.2	16.4
Marshall Stab. lbs @ 140°F	4288	2280	1706	1278
Marshall Flow 0.01 in.	10.5	9.5	8.0	8.5

Table 2-39. Test Road 2 - Formula and tests on HL-4 surface mix (Fromm & Kennepohl, 1979).

Property	Control	Sulfur - Asphalt	Sulfur Asphalt
Stone, % Wt.	45	45	45
Screenings, % Wt.	13	13	13
Sand, % Wt.	42	42	42
Asphalt Grade	150/200	150/200	300/400
Sulfur-Asphalt Ratio	0/100	40/60	40/60
Binder, % Wt.	5.1	5.7	5.7
Voids, % Vol.	5.6	6.0	6.2
VMA, % Vol.	17.2	16.5	16.7
Marshall Stability lbs @ 140°F	1145	1633	1605
Marshall Flow 0.01 in.	8.9	9.5	8.0

Table 2-40. Test Road 3 - Tests on HL-2 and HL-4 mixes (Fromm & Kennepohl, 1979).

Property	HL-2 Control	HL-2 Sulfur-Asphalt	HL-4 Control	HL-4 Sulfur-Asphalt
Asphalt Grade, pen.	85/100	300/400	85/100	300/400
Sulfur-Asphalt Ratio, wt.	0	40/60	0	40/60
Binder, % Wt.	7.4	8.4	5.5	6.3
Voids, % Vol.	5.0	5.0	3.0	3.0
VMA % Vol.	19	19	17.4	17.4
Marshall Stab. lbs @ 140°F	930	930	1500	1500
Marshall Flow 0.01 in.	10.0	10.0	11.0	11.0

reduce thermal cracking. To give the sulfur-asphalt blend a thorough test, 300/400 pen asphalt cement was used blended with 40% by weight of sulfur. The eastbound lanes were paved with sulfur-asphalt using 300/400 pen, and the westbound lanes were paved with 85/100 pen asphalt as a control. The total depth of the asphalt surfacing was 5 in. The specifications for the asphalt cements used are summarized in Table 2-36. Two of the test roads used a sand-asphalt mix as a binder course (HL-2) and all use a general purpose mix (HL-4). Tables 2-37 through 2-40 summarize the properties of the tests of all the mixes used for Test Roads 1, 2, and 3.

Test Road 4 was constructed in July 1979 and reasonably heavy traffic is present; 6,600 AADT with 14% trucks. Four different thicknesses of 150/200 pen in a 40/60 sulfur-asphalt blend were built—3.5 in., 4.5 in., 6 in., and 7.5 in. These thicknesses were contrasted with similar thicknesses of regular 85/100 pen asphalt pavements. The surface course on this course was a HL-1 mix, which is a wear-resistant mix that uses hard traprock as the stone fraction (passing 5/8 in.--retained No. 4). All of the Ontario test roads were constructed using the Gulf Canada, Ltd., process in which liquid sulfur is emulsified in the asphalt before blending with the aggregate in the pug mill.

Results

In 1979 the initial results of the study by Fromm & Kennepohl, (1979) were published. The first three test roads showed that the addition of sulfur to the asphalt resulted in a binder of lower temperature susceptibility. The use of sulfur emulsified in the asphalt binder produces mixes that had greater stiffness at higher temperatures of use. Thus, a softer asphalt than usual can be used. Such a sulfur-asphalt blend had the load-bearing characteristics of a stiffer asphalt at summer road temperatures and the crack resistance (low-temperature flow properties) of a soft asphalt at winter temperatures.

Rutting measurements on the test roads have suggested that the sulfur-asphalt pavements are stronger than normal asphalt pavements. A possible conclusion is that sulfur asphalt pavements could be constructed thinner than regular pavements and thereby reduce costs. This observation suggested that a fourth test road be built to determine layer equivalencies for sulfur-asphalt pavements relative to regular asphalt pavement. There have also been indications that the additional sulfur aided the asphalt in wetting the aggregate surface. The 1981 Fromm et al. report was a follow-up of the continued testing of the first three test roads and also describes construction and testing of the fourth test road.

Test Road 1 had passed through five winters as of 1981. Cracking occurred over the first winter and the cracks increased in number each year. Cracking in the 150/200 pen sulfur-asphalt sections was of the same frequency as over the remainder of the road, which was the control section. The lowest temperature recorded on Test Road 1 was -34°C. There has been very little difference in cracking since 1978.

Test Road 2 had passed through three winters as of 1981. There was very little cracking in evidence after the first winter. After the second winter, several cracks appeared in the north and south control sections, and in the sections where 150/200 pen asphalt was used.

However, Sections C and D (300/400 pen sulfur-asphalt sections) remain crack-free. Only transverse (thermal) cracks were recorded; other cracking such as centerline, random, and edge cracking were not recorded. This result suggests that sulfur does have some effect on the low-temperature flow properties of the asphalt cement. To gain the benefits of the sulfur-asphalt mixes, this would indicate that one should use an asphalt one or two grades softer than is normally used in the area. However, this high frequency of cracking has not occurred on Test Road 1 where similar asphalt grades are under test. Test Road 3 had passed through two winters as of 1981, and Test Road 4, one winter--both roads remain crack free on both the sulfur-asphalt and control asphalt sections.

Rut depths were measured for all four test roads during the late summer of 1980, and these are summarized in Tables 2-41 through 2-44. On Test Road 1 (Table 2-41), the rutting on Section C-F is less than the east and west control sections (Note: Sections A-D and B-E have a higher asphalt content than the control). The greater strength of sulfur-asphalt can be seen when Sections G and H are compared. Section H used 300/400 pen asphalt cement and has rutted deeper than Section G, which used 300/400 pen asphalt in a 50/50 sulfur-asphalt binder.

On Test Road 2 (Table 2-42), the sulfur-asphalt Sections A and B tend to show less rutting than the two control sections; all four sections used 150/200 pen asphalt. The control section had a lower asphalt content as well. The 300/400 pen sulfur-asphalt Sections C and D show slightly less rutting than the control sections and display excellent stability. Sections B and C which are 3 in. thick, show less rutting than the corresponding Sections A or D, which are only 1.5 in. thick. On Test Road 3 (Table 2-43), where 300/400 pen sulfur-asphalt was compared with 85/100 pen regular asphalt, the average rutting was identical for both sections. This indicates that a pavement made with sulfur-asphalt using an asphalt cement two grades softer has about the same strength as the regular asphaltic concrete pavement. On Test Road 4 (Table 2-44), sulfur-asphalt sections show less rutting than the control sections (an asphalt cement one grade softer than the control was used). Table 2-44 also shows that the thicker pavement sections have progressively less rutting than the thinner sections.

On all the test roads, the surface of the sulfur-asphalt seemed tighter than the regular surfaces. This was particularly noticeable after a light shower. The sulfur-asphalt surface drained much quicker, and no trace of rain remained, while the control surfaces still showed traces of wetness. The skid resistance of all four test roads were measured using an ASTM brake force trailer equipped with Type D-501-76 tires and tested according to ASTM E274. The skid numbers of the sulfur-asphalt sections were observed to be at least equivalent to those of the control mixes, and in several instances, slightly better. Skid numbers of Test Roads 1 and 2 for both trial and control mixes are higher than the usual values, because of the angular nature of the aggregates used in both of those localities. Skid numbers were measured in the late summer of 1980.

Laboratory work has shown that some aggregates, which were difficult to coat with asphalt cement and required an antistripping agent, could be coated with sulfur-asphalt blends. Test Road 1 used such an aggregate which required an antistripping agent. Core samples were taken from the road each year to determine if any difference existing in the amount of stripping between the sections using an antistripping agent and those that did not use one. As of 1981, there appeared to be no difference between the two groups. Test Road 4 was also constructed with aggregates—regular sand, blending sand, and stone fractions known to require an antistripping agent. Because of the experience with Test Road 1, no such agent was used on Test Road 4 in the sulfur-asphalt sections. Although Fromm et al. (1981) report that samples taken one year after construction have shown apparent stripping, no manifestation of the distress is apparent at the surface.

Conclusions

The behavior of the four asphalt test roads indicated that sulfur is a practical material to use in combination with asphalt cement, both from a construction and performance point of view. The additional sulfur has produced denser surfaces, which has reduced the penetration of moisture. These surfaces have skid resistance values as good, if not better, than standard-mix surfaces. Sulfur permits the use of softer asphalt cements, which could reduce low-temperature cracking. Pavements made with these softer asphalt cements and sulfur have shown less rutting than pavements made with one penetration grade stiffer asphalt cement. This indicates that sulfur imparts greater strength in the mix at normal pavement temperatures. This latter property may lead to construction of thinner pavements which would still retain the same strength as those presently constructed.

In summary, the conclusions made by the researchers (Fromm et al., 1981) were:

- Pavements made with softer asphalts using sulfur will develop fewer thermal cracks than those paved with a harder grade of asphalt cement without sulfur;
- Mixes made with sulfur asphalt have the stability to resist rutting and deformation better than those mixes made with a stiffer grade of asphalt cement;
- The skid resistance of sulfur asphalt is equal to, or better than, regular asphalt mixes made from the same aggregates;
- The presence of sulfur asphalt may increase the wetting properties of the binder and reduce the need for antistripping agents; and
- To date, there is no indication that the presence of sulfur in the binder would decrease the performance or lasting qualities of the pavement.

Table 2-41. Rut depths, Test Road 1, 1980 for Ontario (Fromm et al., 1981).

Depths in mm

Test Section	Eastbound			Westbound			Grand Average
	IWP	OWP	AV.	IWP	OWP	AV.	
A and D 50/50 Hi-B 150/200	0.2	1.3	0.8	0.0	0.0	0.0	0.4
B and E 40/60 Hi-B 150/200	0.1	1.0	0.6	0.0	0.1	0.0	0.3
C and F 40/60 Lo-B 150/200	0.0	0.2	0.1	0.0	0.1	0.0	0.0
G 50/50 300/400	0.1	0.4	0.3	-	-	-	0.3
H 300/400	0.8	1.6	1.2	-	-	-	1.2
East Control 150/200	0.4	0.3	0.4	0.8	0.5	0.6	0.5
West Control 150/200	0.0	0.6	0.3	0.4	0.0	0.2	0.2

Table 2-42. Rut depths, Test Road 2, 1980 for Ontario (Fromm et al., 1981).

Test Section Thickness, mm	Northbound IWP	Northbound OWP	Northbound AV.	Southbound IWP	Southbound OWP	Southbound AV.	Grand Average
A (S/A 150/200) 38	0.4	1.0	0.7	0.1	2.7	1.4	1.0
B (S/A 150/200) 76	0.2	0.4	0.3	0.4	1.0	0.7	0.5
C (S/A 300/400) 76	0.0	1.0	0.5	1.0	0.9	1.0	0.8
D (S/A 300/400) 38	0.0	1.3	0.6	1.0	1.8	1.4	1.0
South Control 150/200 38	0.4	1.6	1.0	1.8	1.2	1.5	1.2
North Control 150/200 38	0.0	1.0	0.5	1.2	1.6	1.4	1.0

Depths in mm

Table 2-43. Rut depths, Test Road 3, 1980 for Ontario (Fromm et al., 1981).

Depths in mm

Test Section Thickness, mm	Driving Lane			Passing Lane			Grand Average
	IWP	OWP	AV.	IWP	OWP	AV.	
S/A, 300/400 127	1.4	2.0	1.7	0.7	1.2	1.0	1.4
Control 85/100 127	1.3	2.0	1.6	1.5	0.8	1.2	1.4

Table 2-44. Pavement ruts, Test Road 4, 1980 for Ontario (Fromm et al., 1981).

Depths in mm

Test Section Thickness, mm	Northbound			Southbound			Grand Average
	IWP	OWP	AV.	IWP	OWP	AV.	
S/A 90	2.2	2.0	2.1	1.4	0.6	1.0	1.6
S/A 115	1.6	1.9	1.8	1.0	0.2	0.6	1.2
S/A 152	1.2	0.9	1.0	0.7	0.1	0.4	0.7
S/A 190	1.0	1.8	1.4	0.4	0.1	0.2	0.8
Control 115	3.0	2.2	2.6	4.0	2.4	3.2	2.9
Control 190	3.0	0.3	1.6	3.2	1.2	2.2	1.9

2.12 California - Sulfur

Description

In 1981, the California Department of Transportation (Caltrans) began a study of sulfur-extended asphalt (SEA) with three objectives (Predoehl, 1989). The first was to determine whether the incorporation of sulfur with a soft grade of asphalt could change the temperature-viscosity relationship of the binder in hot and cold climates. The other objectives were to determine the performance of SEA pavements and to develop a laboratory test procedure to predict SEA binder durability.

Two field test sections (AC overlays) were constructed. A hot climate SEA test section was installed on I-15 near Baker in 1982, and a cold climate section on US 6 at Benton in 1984. Tables 2-45 and 2-46 list the physical, climatic and preexisting structural conditions of each site. Both projects utilized 20% and 40% sulfur by weight of binder, and used an AR-2000 from Newhall Refinery. However, in the hot climate (Baker) test section, the SEA binder was preblended prior to mixing with the aggregate in a drum mixer, while the SEA binders used in the cold climate section (Benton) were blended during the mixing in the pug mill of a batch plant mixer. Tables 2-47 and 2-48 summarize the mix design data and binder contents used on both test sections. Since mix type, binder content, and physical properties of the aggregate on both sections are very similar, the mixes should be very similar.

Field testing of the sections consisted of skid resistance testing and deflection measurement. Cores were also taken for testing, and crack and condition surveys were performed. The binders were recovered from the cores using the Abson recovery procedure and tested for viscosity, penetration, and ductility.

Results

Tables 2-49 and 2-50 summarize the test results from the testing of the Abson recovered residue at different pavement ages. The results in Table 2-49 (Baker) indicate slightly less weathering (softer residue) after 54 months in the No. 2 lane, except for the AR-4000. However, Predoehl (1989) hypothesized that this could be due to the greater compaction effort produced by more truck traffic in this lane.

Higher specific gravities are also present in the No. 2 lane, confirming that air voids are probably lower than in the No. 1 lane. Void content has been shown in previous studies to affect the rate of hardening. Table 2-50 shows that the binders exhibit slightly more hardening in the northbound lanes after 36 months (higher voids than southbound). However, other factors (temperature, voids, aggregate porosity, etc.) also affect hardening rate. Figure 2-16 illustrates the change in absolute viscosity with time for the test sections. The results show that the hardening rates for both sides are very similar, especially for the AR-2000 control binders. The addition of sulfur appears to have reduced the hardening rate.

Crack and condition surveys were performed before construction and periodically afterwards. Figures 2-17, 2-18, and 2-19 summarize the results of these surveys. The evaluations indicate that the Baker test section resisted cracking for a significantly longer period (approximately 3 years) than the Benton section (Figures 2-17 and 2-18). The deflection values are also higher at the Benton site. The results of both alligator cracking and deflection measurements for both test sites are seen in Figure 2-19. It appears that the combination of greater deflections, a thinner overlay in the northbound lanes, and the colder climate at Benton have made a significant contribution toward the higher incidence of cracking in the Benton sections.

The cracking data also reveals that the SEA pavements were more susceptible to reflected transverse (thermal) cracking than the conventional pavements (Figures 2-20 and 2-21). In the 40% SEA portion of the northbound lanes at Benton, significant transverse cracking (42%) were reflected after only 16 months. The 20% SEA section did not approach this level of cracking until more than 28 months after overlaying. However, in the case of alligator cracking, the SEA sections resisted this longer than the control section. It appears that SEA binders may be more thermally sensitive in colder climates, especially for higher sulfur contents. In the warmer Baker section, the SEA and AR-2000 control sections have approximately equal cracking after 66 months.

Finally, California Tilt-Oven Durability (CATOD) tests were also performed on the SEA and control binders for each project after construction. The tests were used in an attempt to estimate SEA binder durability and were based on previous studies that had correlated the test with durability in a hot weather setting using unmodified asphalts. The results indicated that only the AR-4000 binder approaches the CATOD predictions and that the Baker test site has a less severe weathering regime than the CATOD standard, which is the Indio or low desert climate. Table 2-51 summarizes the CATOD results. Comparing the penetration at 77°F of the SEA and the AR-2000 binders, it is clear that the AR-2000 binders should harden at approximately the same rate.

Conclusions

Predoehl (1989) arrived at the following conclusions:

- SEA binders have a slower or equal hardening rate in comparison with the AR-2000 control;

- SEA binders appear to resist alligator cracking better than the AR-2000 control sections; however, the higher sulfur content (40%) are more prone to reflection cracking at an early age;

Table 2-45. California Test Section Physical Conditions (Predoehl, 1989).

Test Section	Terrain	Average Altitude (feet)	Weather Conditions* Temperatures (°F) Summer (Maximum) 5 Yr. Avg.	Winter (Minimum) 5 Yr. Avg.	Yearly Average 5 Yrs	Yearly Average Normal	Rainfall (inches) Yearly Average 5 Yrs	Rainfall (inches) Yearly Average Normal
Baker (Hot)	Level and Straight	1700	112	20	67.6	67.2	5.47	3.81
Benton (Cold)	Level and Straight	5300	105	9	55.5	56.1	8.8	

*Data from U.S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA) Climatological Summaries for 1982 thru 1986. Baker T. S. data from Daggett FAA, (elevation 1922 feet, 25 miles west). Benton T. S. data from Bishop, (elevation 4100 feet, 30 miles south) for temperatures and Benton Inspection Station (elevation 5461 feet, 2 miles east) for rainfall. From conversations with local Benton residents, it is believed that Benton maximum and minimum temperatures should be about 10°F lower than Bishop temperatures.

Table 2-46. Pre-existing conditions - structural, traffic and deflection data in California (Predoehl, 1989).

Location	Preexisting Structural & Traffic Conditions*	Original Construction Data (age)	Section	(80th Percentile) Preexisting Deflection Measurements (inches)**	Deflections After Overlay 80th Percentile (inches)**	Actual DGAC Overlay Thickness (feet)***
Baker						
			AR-4000			
	0.04' OGAC	1964	NB #1	-	0.006	0.21
	0.54' DGAC	(18 years)	NB #2	0.007	0.006	0.21
	0.67' AB					
	0.75' AS		AR-2000			
			NB #1	-	0.010	0.22
	1979 ADT = 15,300 (15% trucks) 1995 Estimated ADT = 26,000 Estimated 10 years T.I. = 11.5		NB #2	-	0.010	0.19
			SEA 20%			
			NB #1	-	0.008	0.21
			NB #2	0.013	0.012	0.23
			SEA 40%			
			NB #1	-	0.007	0.19
			NB #2	-	0.012	0.21
Benton						
			AR-2000			
	0.08 DGAC (1977)	Prior to 1950	NB	0.019	0.024	0.14
	0.08 DGAC (1963)		SB	0.018	0.020	0.26
	0.25 DGAC					
	0.50 Imported Borrow					
	1979 ADT = 1900 (15% trucks) 1995 Estimated ADT = 2500 Estimated 10 Years T.I. = 9.0					
			SEA 20%			
			NB	0.019	0.025	0.18
			SB	0.018	0.022	0.24
			SEA 40%			
			NB	0.019	0.018	0.16
			SB	0.018	0.021	0.26

* Thickness in feet, O.G.A.C. = Open Graded Asphalt Concrete
D.G.A.C. = Dense Graded Asphalt Concrete
A.B. = Aggregate Base
A.S. = Aggregate Subbase
O.G.A.C. was removed prior to construction of SEA T.S. overlays.

** Preexisting deflections - 1980, Postconstruction Deflections - Baker 5 weeks, Benton 12 months, after overlaying

***Recommended overlay thicknesses based on deflections and traffic conditions = Baker T.S. = 0.25 feet, Benton T.S. = 0.15 feet. Actual thicknesses are average of two or more cores taken at 10 and 11 months from the Baker and Benton, T.S.

Table 2-47. Mix design data for California (Predoehl, 1989).

		Calif. Test	Baker T.S.	Benton T.S.
1.	Design Method		Hveem (Calif. Test 367)	Hveem
2.	Mix Type, Size, Grading (Calif. Stand. Spec. Sect 39)		Type A, 1/2" Maximum, Coarse	Type A, 1/2" Maximum, Coarse
3.	Aggregate (source)		Opah Ditch Pit (10 miles west of Baker)	Milner Fan Pit (at PM 7.8 south of highway)
4.	Aggregate Properties			
	Specific Gravity Coarse	206	2.56	2.64
	Fine	208	2.69	2.67
	LART Abrasion Loss after 500 Revolutions	211	26%	19%
	K_c Factor	303	1.2	0.9
	K_f Factor	303	1.1	1.0
	K_m Factor	303	1.1	1.0
	Surface Area ft^2/pound	303	26	26.3
	Sand Equivalent	217	64	79
5.	Blending Asphalt - AR-2000		Newhall	Newhall
6.	Sulfur		Commercial Grade 99.4% Pure	
7.	Binders		AR-4000, AR-2000 SEA 20%, SEA 40%	AR-2000, SEA 20% SEA 40%

Table 2-48. Mix design binder recommendations* in California (Predoehl, 1989).

Test Section	Binder	Percent** Binder	Design Briquette Values: Stability Calif. Test 366	Percent Voids*	Specific Gravity*
BAKER	AR-4000***	5.4	43	5.3	2.30
	AR-2000	5.4	40	6.0	2.25
	SEA 20%	6.0	41	6.0	2.26
	SEA 40%	8.0	38	4.5	2.27
BENTON	AR-2000	5.5	38	4.5	2.31
	SEA 20%	6.0	39	4.5	2.33
	SEA 40%	7.5	46	4.8	2.31

* California Test 367 (Hveem)
** By weight of dry aggregate
***AR-4000 mix design by District 08 Lab in San Bernardino. Job asphalt.

Table 2-49. Summary of test results of recovered binder residues* - Baker test section in California (Predoehl, 1989).

Test on Residue	Test Method No. AASHTO	Overlay Age (months)	AR-2000 #1 Lane	AR-2000 #2 Lane	SEA 20% #1 Lane	SEA 20% #2 Lane	SEA 20% #2** Lane D.B.A.	SEA 40% #1 Lane	SEA 40% #2 Lane	AR-4000 #1 Lane	AR-4000 #2 Lane
Absolute Viscosity at 140°F (poise)	T-202	Orig.	2675		1101		1828	1103		4447	
		10	4402	4400	1810	1909	2171	1797	1849	9314	10904
		28	6653	9285	8241	4027	7035	3390	2699	16415	26360
		54	11495	9728	9371	7631	8837	6544	5424	35256	39100
Kinematic Viscosity at 275°F (cst)	T-201	Orig.	356		155		161	154		445	
		10	396	431	190	199	200	165	167	564	590
		28	499	580	389	242	299	230	203	707	881
		54	595	565	342	362	327	306	334	915	980
Penetration at 77°F (dmm)	T-49	Orig.	40		68		51	70		27	
		10	31	30	53	50	46	51	53	21	19
		28	27	21	24	32	29	33	40	18	13
		54	22	22	22	27	26	27	26	13	13
Softening Point (°F)	T-53	Orig.	127		121		125	121		131	
		10	131	131	123	125	127	123	124	139	139
		28	134	135	136	129	132	129	125	139	145
		54	139	142	140	139	140	138	136	151	154
Ductility at 77°F (cm)	T-51	Orig.	100+		100+		100+	100+		100+	
		10	100+	100+	100+	98	64	88	96	100+	100+
		28	100+	100+	67	81	33	100+	100+	18	13
		54	91	87	37	24	39	72	71	10	10

*Original mix from windrow, remaining samples from 12-inch cores.
Test on binder residue recovered by California Test 380 (Abson recovery).

Table 2-50. Summary of recovered binder* test results - Benton test section in California (Predoehl, 1989).

Test on Residue	Test Method No. AASHTO-	Overlay Age (months)	Test Results of Binders (Average of two or more). AR-2000 NB Lane	AR-2000 SB Lane	SEA 20% NB Lane	SEA 20% SB Lane	SEA 40% NB Lane	SEA 40% SB Lane
Absolute Viscosity at 140°F (poise)	T202	Orig.	2025		1477		1605	
		10	4848	5202	2775	2871	2813	2205
		36	9835	7702	6262	5323	7089	6686
Kinematic Viscosity at 275°F (cst)	T201	Orig.	293		154		150	
		10	443	440	214	230	207	189
		36	560	540	382	453	296	277
Penetration at 77°F (dmm)	T49	Orig.	50		56		57	
		10	30	27	40	41	39	44
		36	23	25	27	30	27	27
Penetration at 39.2°F (dmm)		Orig.	15		16		17	
		36	8	9	13	14	11	12
Softening Point (°F)	T53	Orig.	123		122		121	
		10	131	133	127	128	127	127
		36	136	134	135	132	135	134
Ductility at 77°F (cm)	T51	Orig.	150+		94		84	
		10	100+	100+	92	83	100+	82
		36	100+	100+	53	68	40	51

*Original Mix - from windrow, remaining samples 12 inch cores. Test on binder residues recovered by California Test 380 (Abson recovery).

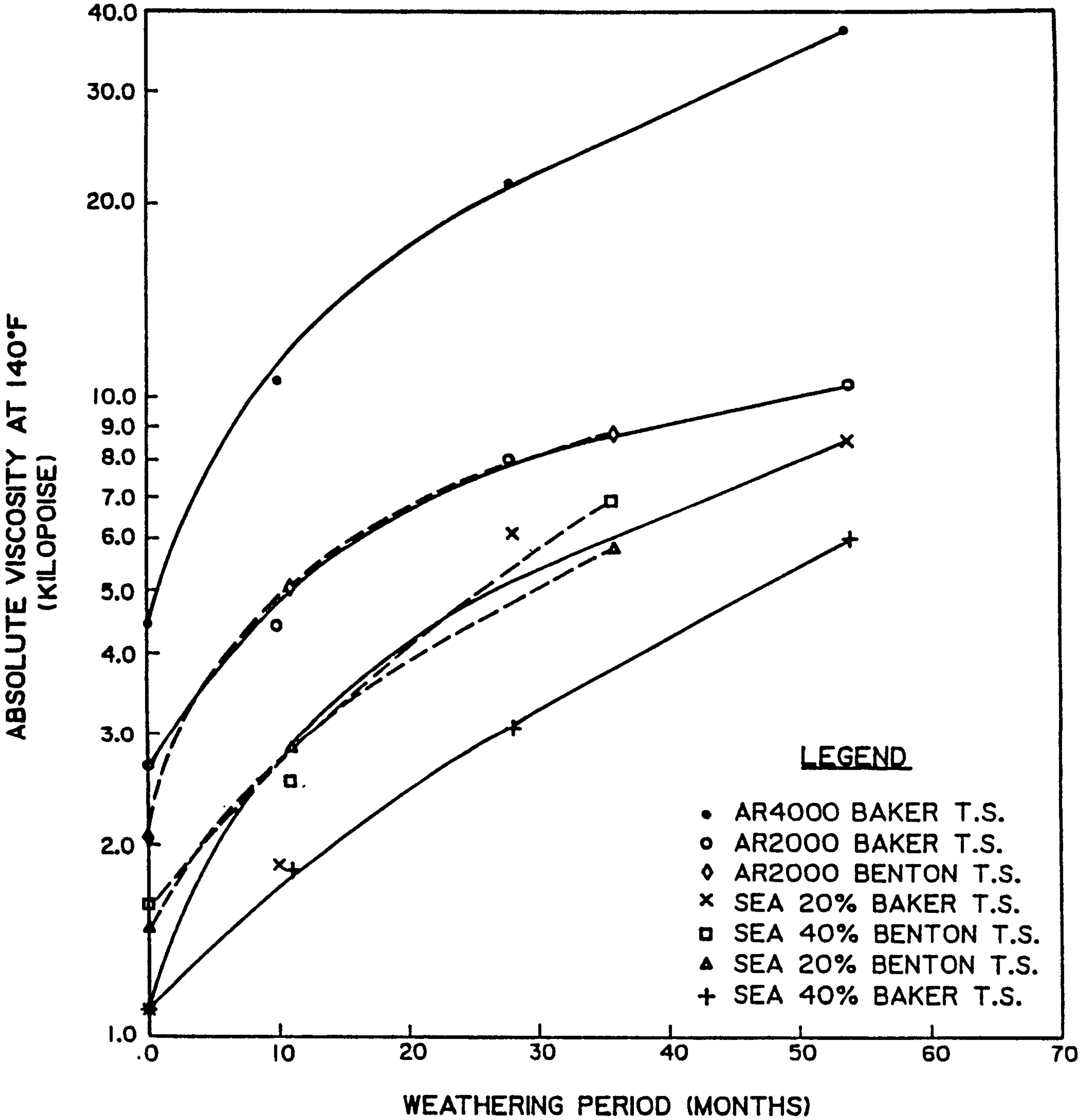


Figure 2-16. Comparison of binder hardening rates of Baker and Benton test sections in California (Abson recovered residues) (Predoehl, 1989).

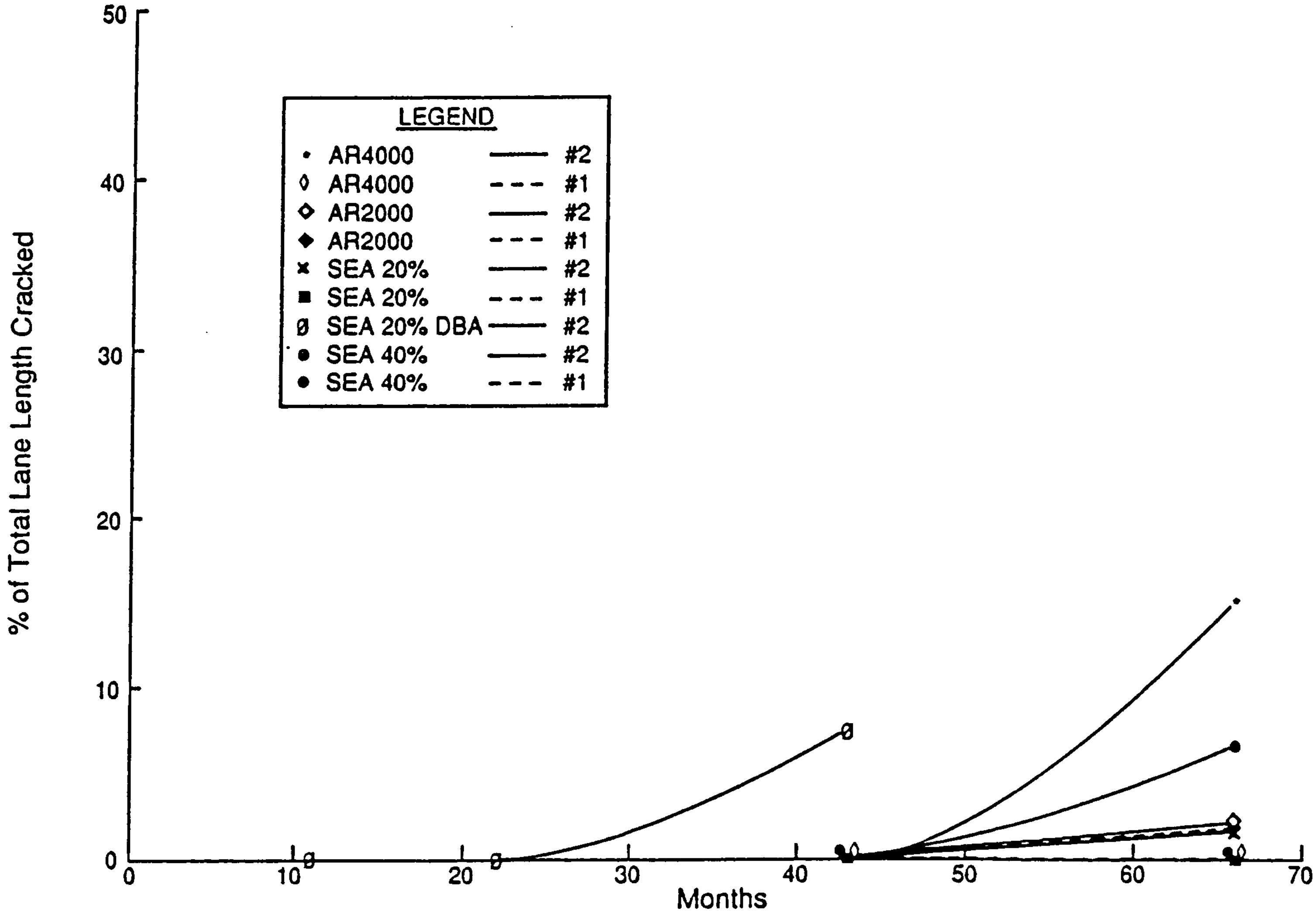


Figure 2-17. Percent of total lane length cracked - Baker test section in California (Predoehl, 1989).

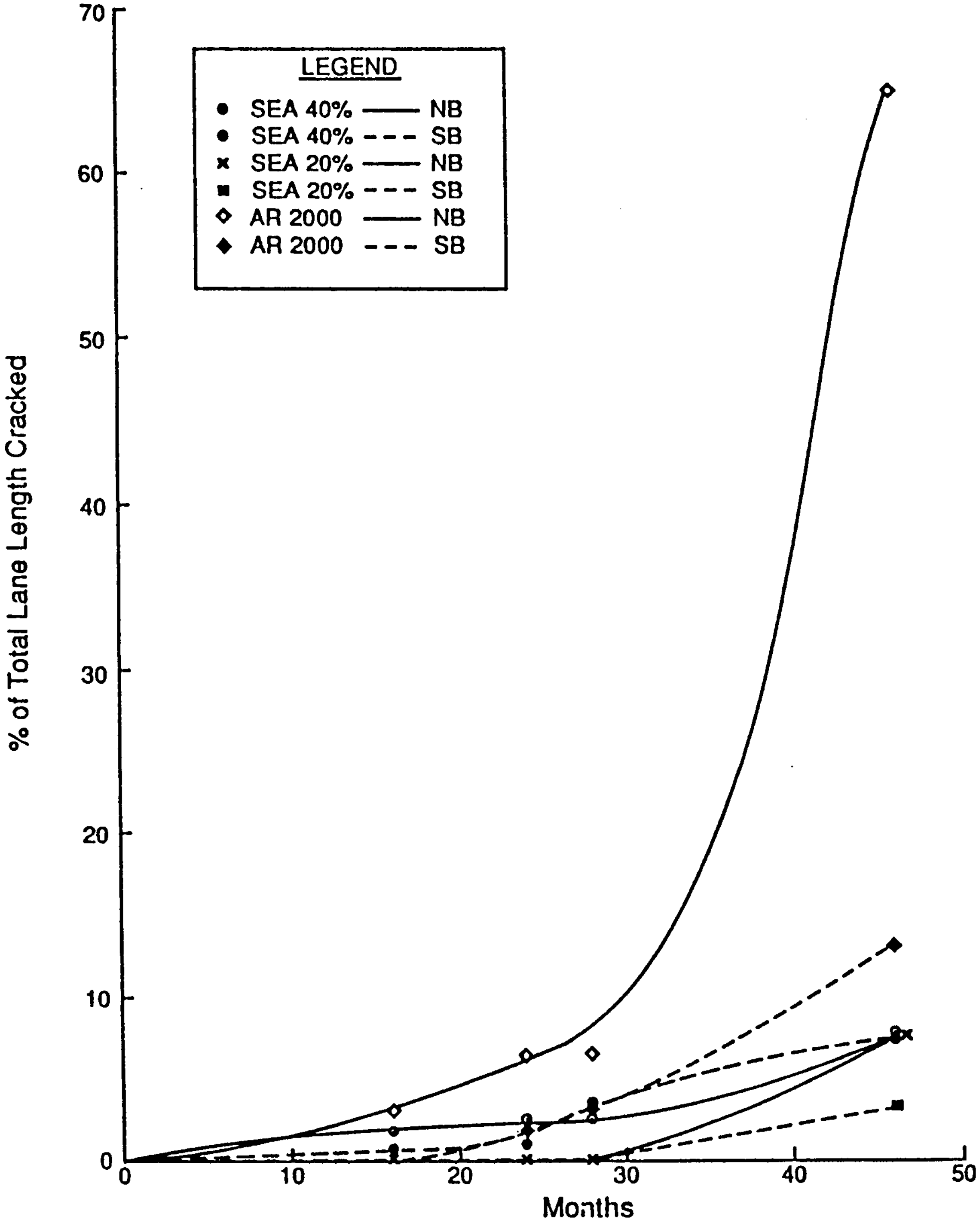


Figure 2-18. Percent of total lane length cracked Benton test section in California (Predoehl, 1989).

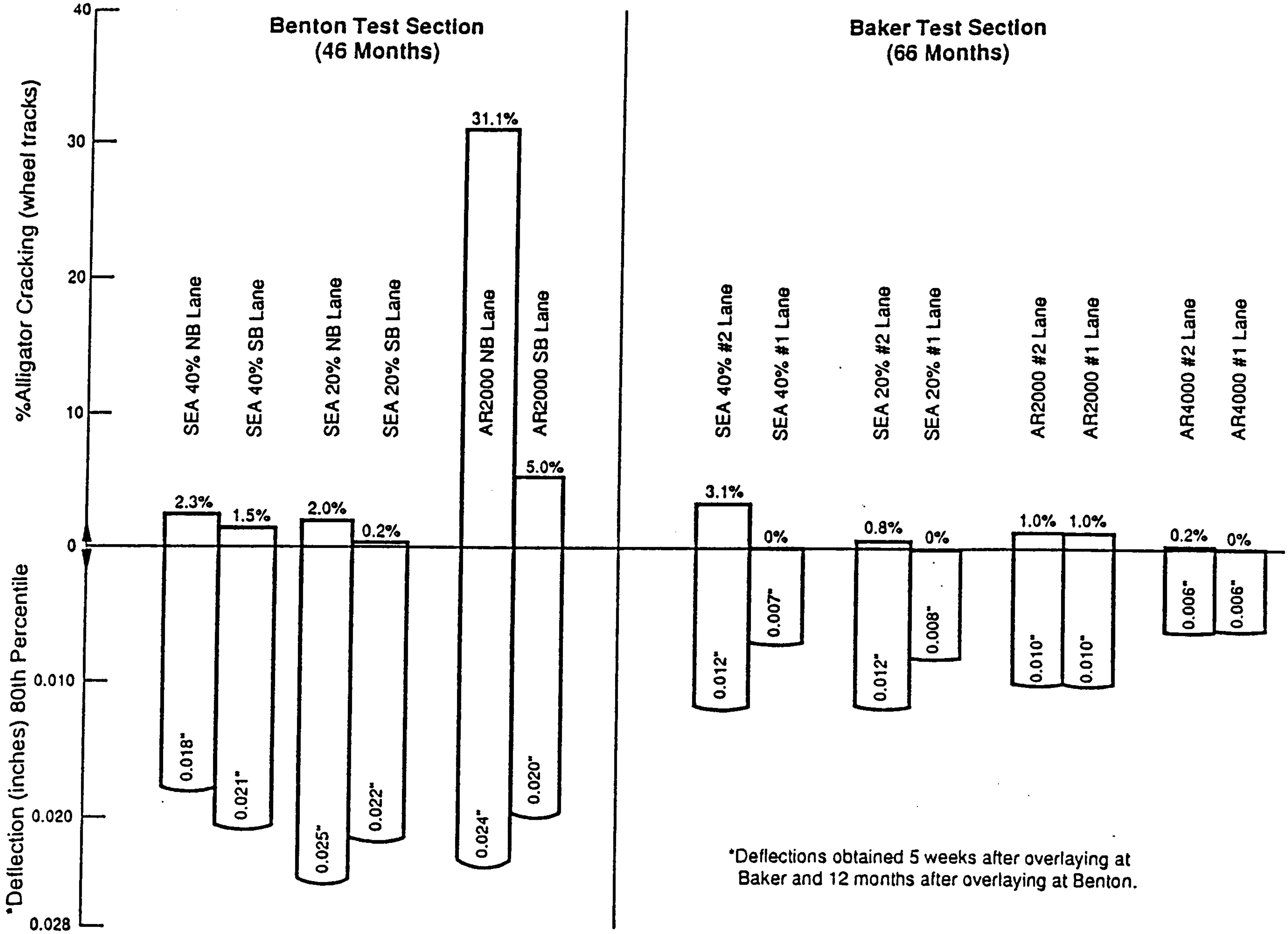


Figure 2-19. Comparison of % alligator cracking and deflection measurements at Baker and Benton test sites in California (Predoehl, 1989).

- The skid resistance characteristics are not affected by the addition of sulfur;

- The hardening rate of the AR-4000 binder was higher than the AR-2000 and SEA binders at Baker;

- The addition of sulfur reduces the hardening rate; however, the SEA binders have poorer ductile properties as they age;

- The CATOD test cannot be used to directly predict the age hardening of binders, since it was developed for a hot, desert test site;

- The pavements in the Baker test section resisted cracking for approximately three years longer than the comparable thickness pavements in the Benton test section. The earlier cracking of the Benton pavements appears to have been the result of a combination of greater deflections, a thinner overlay in one lane, and colder ambient temperatures;

- It appears that SEA blends may be more susceptible to thermal stresses in colder climates as the sulfur content increases. The SEA 40% sections at the Benton site (colder climate) has significant transverse (thermal) cracking 16 months after overlaying, while the adjoining SEA 20% and control sections required more than 27 months to reach nearly similar levels of transverse cracking. Transverse cracking was minimal in all the sections at the warmer Baker test site; however, the SEA sections were the first to exhibit beginning transverse cracking at approximately 66 months;

- The pavements constructed with SEA blends in the colder Benton site appeared to resist alligator cracking better than the pavements constructed with the control AR-2000 asphalt. The AR-2000 pavements had significant alligator cracking after 46 months, while the SEA pavements had minimal alligator cracking. Cracking of all the sections in the warmer Baker test sections was minimal 66 months after overlaying;

- Overall cracking frequency of the pavements at each site appeared to correlate well with binder hardening; and

- The SEA 20% has the best performance in both test sections; however, the control AR-2000 was equal in performance at the warmer Baker site. In warmer areas it appears that the primary factor affecting the cost-effectiveness of SEA binders is whether the cost of sulfur is no more than 50% as much as the asphalt. In colder areas, it appears that SEA mixes with about 20% sulfur by weight may offer greater resistance to alligator cracking than conventional asphalt mixes.

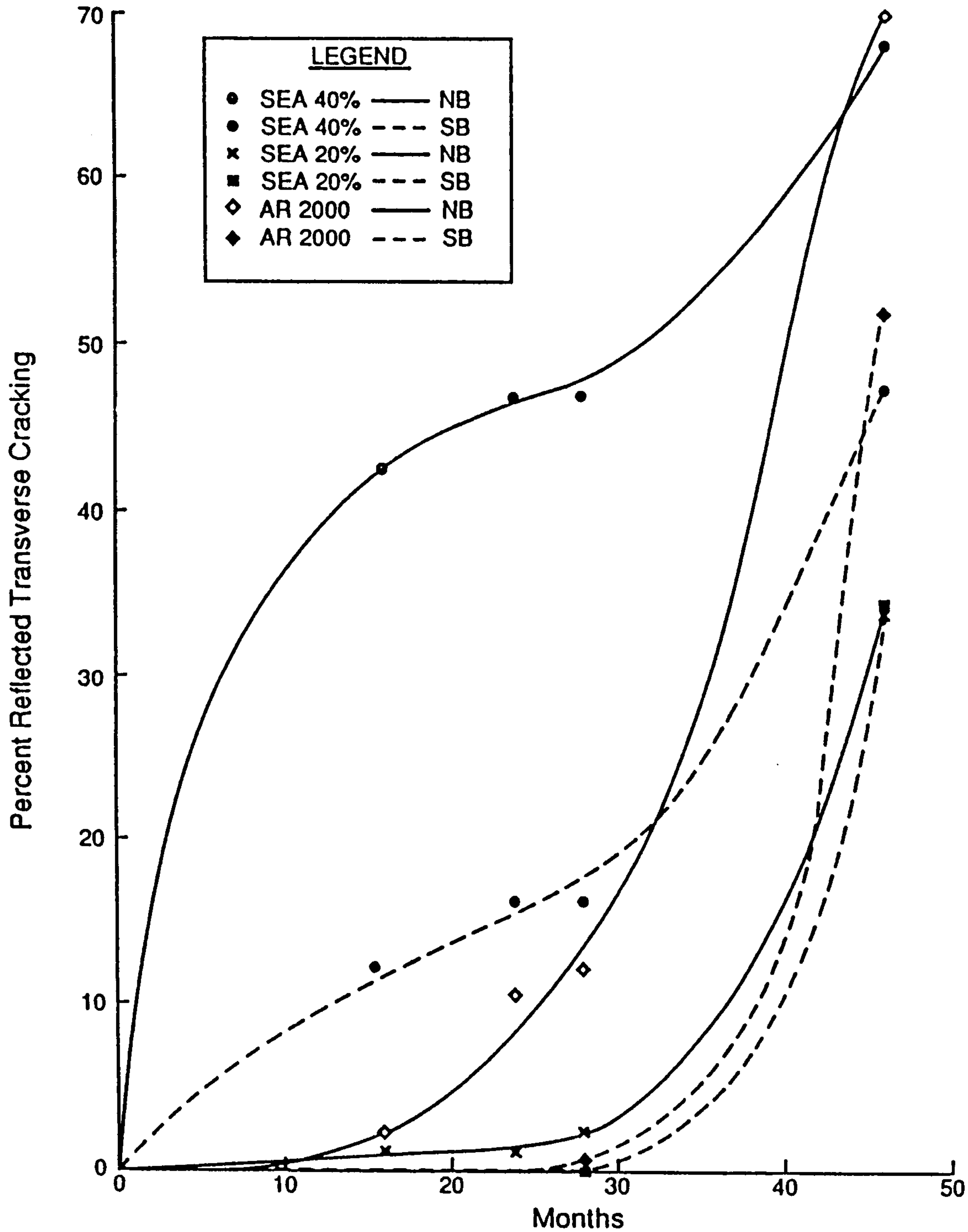


Figure 2-20. Reflected transverse cracks - Benton test section in California (Predoehl, 1989).

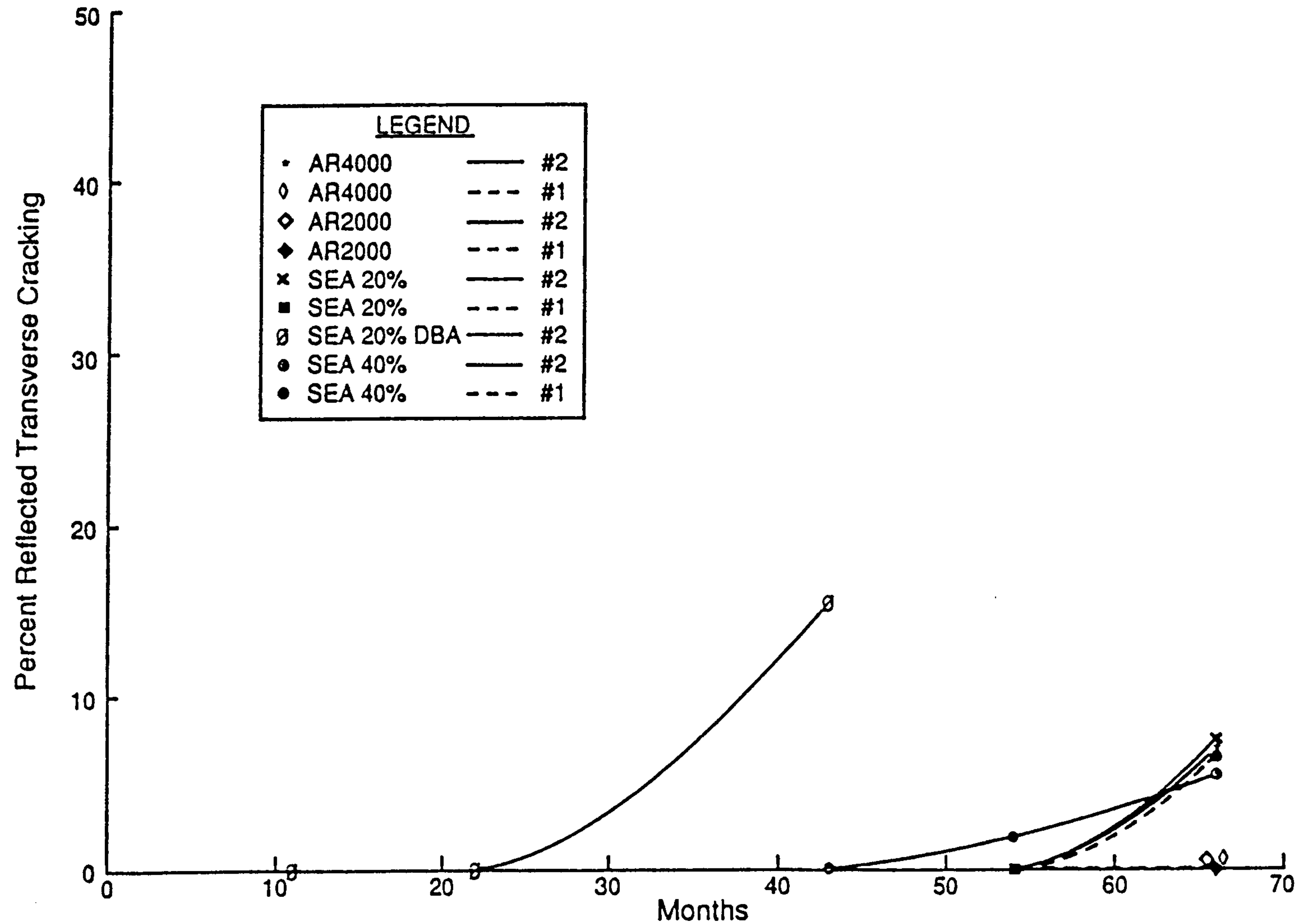


Figure 2-21. Reflected transverse cracks - Baker test section in California (Predoehl, 1989).

Table 2-51. Comparison of field residue test results to CATOD test results in California (Predoehl, 1989).

Test on Residue	Test Method AASHTO	Site	AR-4000 Field**	AR-4000 CATOD	AR-2000 Field	AR-2000 CATOD	SEA 20% Field	SEA 20% CATOD	SEA 40% Field	SEA 40% CATOD
Absolute Viscosity at 140°F (poise)	T202	Baker	37,178	102,601	10,611	83,592	8,613	348,518	5,984	90,326
		Benton			8,768	101,468	5,792	541,225	6,887	122,346
Kinematic Viscosity at 275°F (centistokes)	T201	Baker	947	1,435	580	1,240	344	1,305	320	680
		Benton			550	1,375	417	1,875	281	657
Penetration at 77°F (dmm)	T49	Baker	13	9	22	12	25	11	26	14
		Benton			24	12	29	11	30	14
Softening Point (°F)	T53	Baker	153	158	141	158	140	167	137	160
		Benton			135	152	133	163	135	155
Ductility at 77°F (cm)	T51	Baker	10	6	89	7	33	2	71	2
		Benton			100+	5	60	1	45	1

Test Results on Residue (Averages)

* Field residue test results on Abson recovery residue (California Test 380).
CATOD = California Tilt-Oven Durability (California Test 374) predicts effects of approximately two years in hot desert climate.

**Field results = Baker, 54 months, Benton, 36 months.

3
Laboratory Studies

This chapter reviews research reports on the laboratory studies on the performance of pavements prepared with modified asphalts. Field performance information was not measured in these reports. As in Chapter 2, this information comes from both SHA research and published technical literature. For each report reviewed in this chapter, a project description, results and conclusions of the researchers are presented.

3.1 Goodrich - Polymers

Background

Five asphalt binders (three conventional and two polymer-modified) were studied by Goodrich (1988). The three conventional asphalts represented the extremes of temperature susceptibility (identified as "A", the most temperature susceptible; "B", the least susceptible; and "C", intermediate). The two polymer-modified asphalts (identified as "P1" and "P2") contained 5% polymer, and each was blended into asphalts similar to lower viscosity versions of asphalt "A." The five asphalts chosen were selected to have, as closely as possible, absolute viscosities at 140°F in the AR-4000 and AC-20 range.

Several areas of investigation were studied and are presented in Goodrich (1988):

- Rheology and physical properties of the binders,
- Thermally induced cracking in asphalt concrete,
- Flexural fatigue life of asphalt concrete,
- Permanent deformation of asphalt concrete, and
- Aging: chemical and rheological changes.

The focus of the investigation was to determine which test properties of asphalt and polymer-modified asphalt best related to performance. Dynamic mechanical analysis was used as well as flexural fatigue, creep, rolling thin film oven (RTFO) and long term durability (LTD) tests.

Dynamic Mechanical Analysis

Dynamic mechanical analysis experiments were conducted with two similar rheometers: a rheometrics mechanical spectrometer, and a rheometrics dynamic analyzer. The dynamic analysis allowed finger printing of the asphalts through measurement of the viscous and elastic characteristics of the binders over wide ranges in temperature and in loading time. During testing, sinusoidal oscillatory shear strains were imposed on samples in a parallel disk viscometer configuration (Figure 3-1). The amplitude of stress was determined by measuring the torque transmitted through the sample in response to the imposed strain. Strain amplitude and frequency were input variables, set by the operator. The phase shift angle was measured by accurately determining sine wave forms of the strain and the torque.

When cold and brittle, asphalt may behave as a nearly ideal solid, where the stress will exactly follow the sinusoidal input strain (Goodrich, 1988). At elevated temperatures, most asphalts will approach ideal liquid (Newtonian) behavior (Verga et al., 1975). In this case, the maximum stress will occur when the rate of strain is greatest. This occurs 90° out-of-phase with the peak strain. Therefore, with ideal liquids the peak stress lags 90° behind the peak input strain (Goodrich 1988).

In between the ideal elastic solid behavior and the ideal viscous fluid response are the viscoelastic materials, such as asphalt. Depending on the temperature and strain frequency, the peak stress of viscoelastic materials can lag anywhere from 0° to 90° behind the maximum applied strain (Figure 3-2). The lag, or phase shift angle, is referred to as delta (δ) (Figure 3-3).

Goodrich (1988) defined the parameters that are measured during dynamic mechanical analysis. In dynamic mechanical analysis an asphalt sample will reach a steady state condition after a limited number of cycles (Dickinson & Witt, 1974). The ratio of the peak stress to the peak strain (Figure 3-2) is the absolute value of the modulus, referred to as the *complex shear modulus*, $|G^*|$:

$$|G^*| = \text{peak stress/peak strain}$$

The in-phase component of $|G^*|$ is called the *shear storage modulus*, or G'. The storage modulus equals the stress in phase with the strain divided by the strain, or:

$$G' = |G^*| \cos(\text{delta})$$

Delta is the phase shift angle between the applied maximum strain and the maximum stress (Figure 3-3).

The out-of-phase component of $|G^*|$ is called the *shear loss modulus*, or G''. G'' represents the viscous components of $|G^*|$. The loss modulus equals the stress 90° out-of-phase with the strain divided by the strain, or:

$$G'' = |G^*| \sin(\text{delta})$$

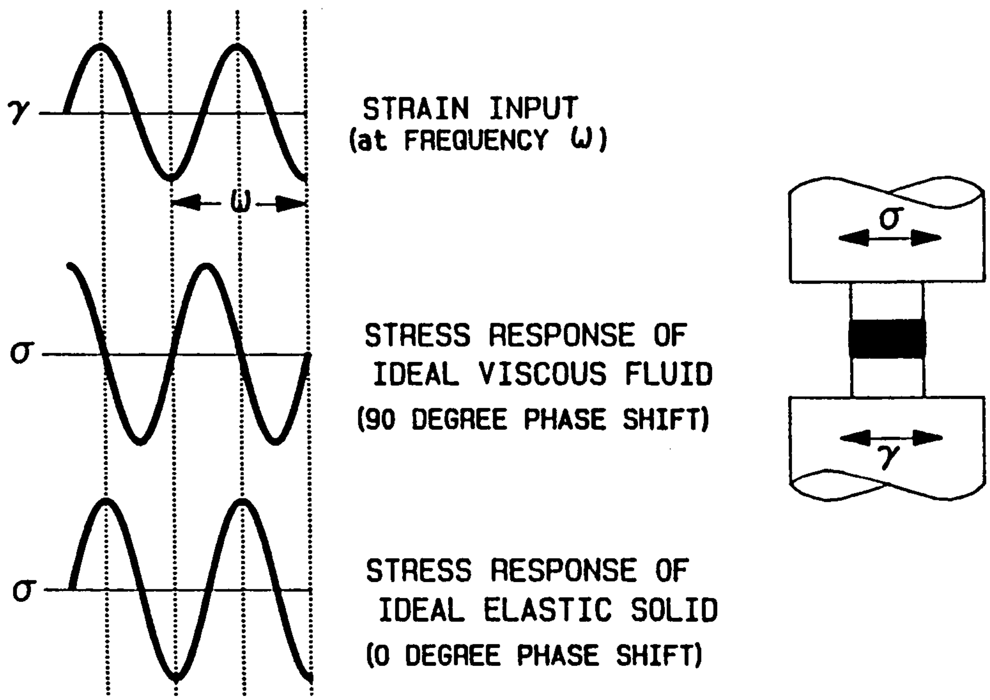


Figure 3-1. Illustration of dynamic mechanical analysis (Goodrich 1988).

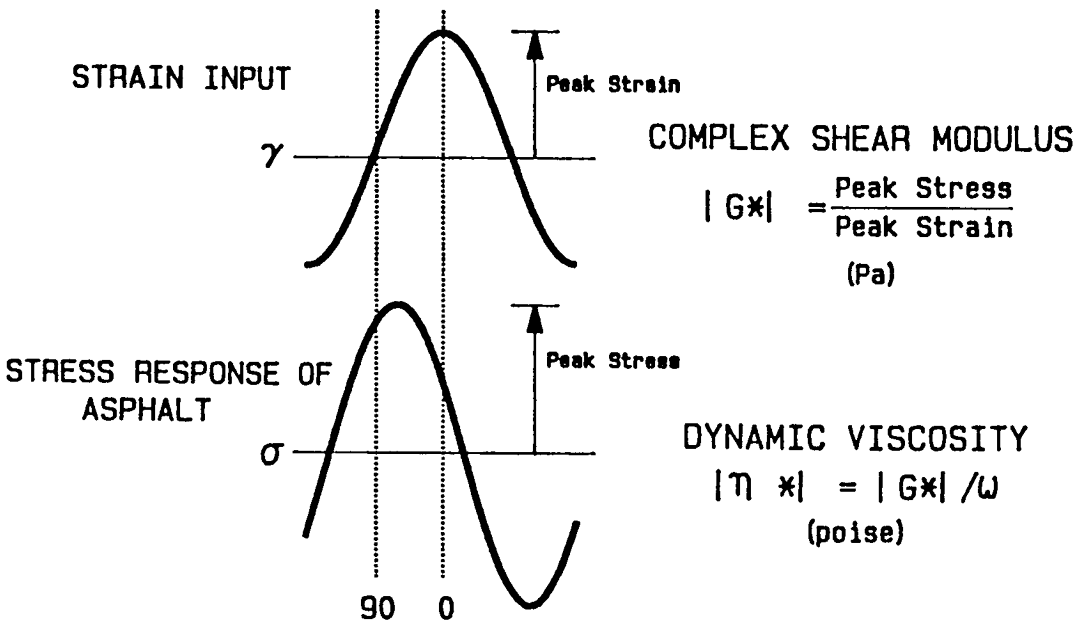


Figure 3-2. Illustration of peak strained stress, dynamic mechanical analysis (Goodrich 1988).

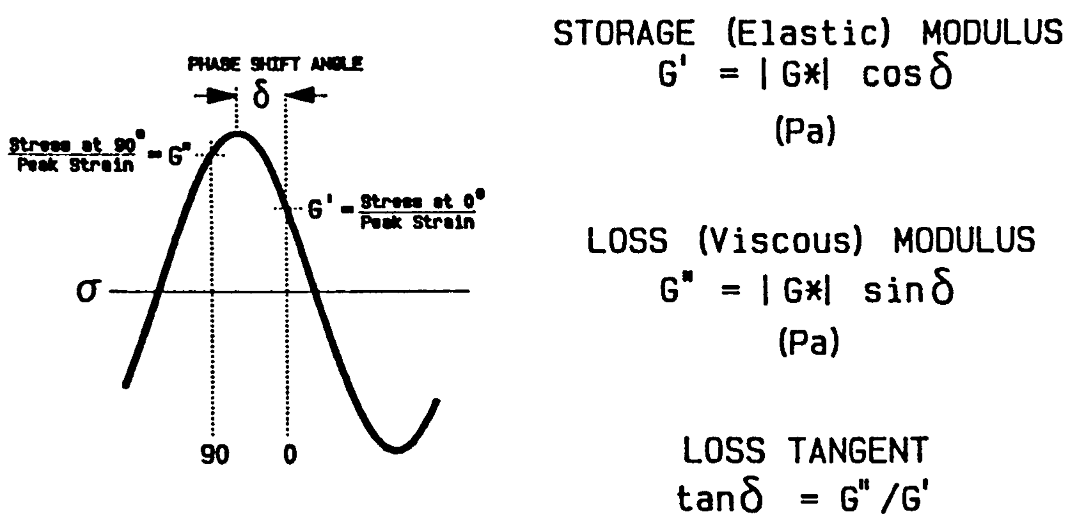


Figure 3-3. Illustration of phase sift-angle, dynamic mechanical analysis (Goodrich 1988).

Typical units for $|G^*|$, G' and G'' are Pascals (SI) or dynes/cm^2 (cgs).

The *complex dynamic shear viscosity*, |eta*|, is computed from the complex shear modulus (in dynes/cm^2) and the strain frequency, omega (radians/sec):

$$|\text{eta}^*| = |G^*| \;/\; \text{omega (poise)}$$

The ratio of the viscous to the elastic moduli is referred to as the *loss tangent*. The loss tangent is the ratio of energy lost to the energy stored in a cyclic deformation:

$$\text{Loss tangent} = \tan\,(\text{delta}) = G''/G'$$

If the dynamic testing is done using small strains within the linear viscoelastic region, then according to the principle of superposition, the data obtained at higher and lower temperatures can be equated simply and graphically with lower and higher frequencies, respectively. Conversely, data obtained at higher and lower frequency can be transposed into lower and higher temperatures, respectively. An example of time (frequency)-temperature superposition is given in Figure 3-4. The figure shows the actual data points obtained over a range of temperatures and a narrow range of frequencies. By applying the superposition principle, each data set obtained at a particular temperature can be shifted along the time axis to form a smooth curve. The degree to which succeeding curves must be shifted to form a smooth curve is referred to as the *shift factor* and is related to the temperature susceptibility of the material.

Results

Binder Properties

Conventional binder properties (penetration, viscosity, etc.), force ductility (Shuler et al., 1985), and toughness-tenacity (Benson, 1955) were conducted on the binders. Details of the force ductility test and the toughness-tenacity test procedures are discussed later in the review of the New Mexico - Styrenic Block Copolymers (Section 3.2). Measured and calculated binder properties of the five binders are shown in Tables 3-1 and 3-2. Recall that the binders were chosen to have absolute viscosities at 140°F in the AC-20/AR-4000 range and that Binders A and B are the most and least temperature susceptible, respectively. Force ductility testing showed that the polymer-modified binders had the highest maximum stress and strain. Toughness-tenacity test results were also highest with the polymers.

Dynamic Mechanical Analysis

These tests were conducted on aged residues from RTFO and LTD tests. The testing of RTFO residues was done to more closely reflect the binder properties in the mixtures subjected to performance related tests. The testing of LTD residues were accomplished to evaluate the effects of aging on binder rheology.

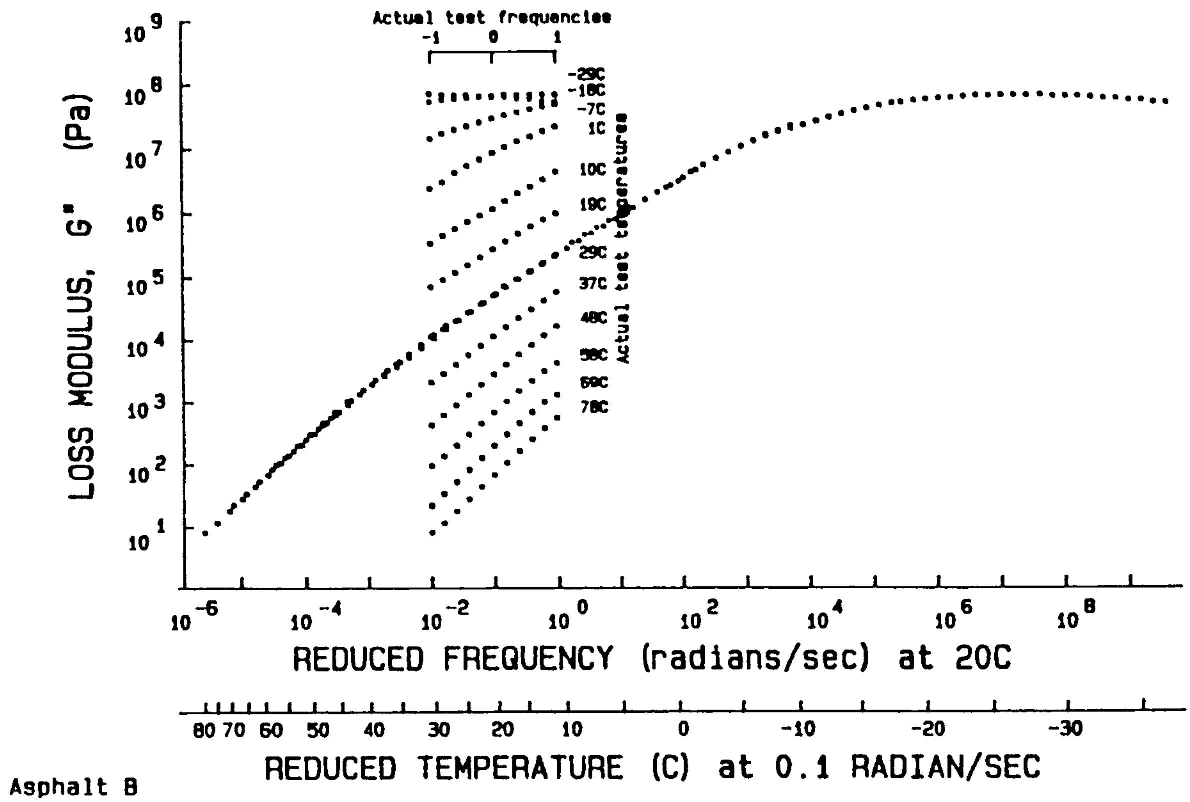


Figure 3-4. Illustration of time-temperature superposition (Goodrich 1988).

Table 3-1. Measured binder properties (Goodrich 1988).

ASPHALT IDENTIFICATION:	Conditions	A	B	C	P1	P2
ORIGINAL ASPHALT PROPERTIES						
Penetration						
200gm, 60 sec	@ 4 C, dmm	16	62	25	40	38
100gm, 5 sec	@ 4 C, dmm	1.5	14.0	2.8	8.2	5.6
100gm, 5 sec	@ 25 C, dmm	52	126	56	114	110
Viscosity, Absolute	@ 60 C, poise	2182	1425	2381	1870	2239
Viscosity, Kinematic	@ 135 C, cSt	317	392	488	541	821
Softening Point (R.&B.)	, C	50.0	44.5	49.0	48.1	66.1
	, F	122	112	120	115	151
Ductility (5 cm/min)	@ 25 C, cm	150+	150+	150+	150+	82
Ductility (5 cm/min)	@ 4 C, cm	0	21	0	32.2	100+
Freess Brittle Point	, C	-19.8	-26.7	-23.3	-23.2	-25.0
	, F	-3.7	-16.0	-10.0	-9.7	-13.0
RTFO OVEN PROPERTIES						
Loss (-gain), Wt.	, %	0.17	0.49	-0.12	0.15	0.48
Penetration						
200gm, 60 sec	@ 4 C, dmm	13	34	21	26	28
100gm, 5 sec	@ 4 C, dmm	1.2	4.6	2.2	3.0	3.5
100gm, 5 sec	@ 25 C, dmm	33	70	48	61	71
Viscosity, Absolute	@ 60 C, poise	4870	4172	4178	4847	2949
Viscosity, Kinematic	@ 135 C, cSt	429	838	824	977	800
Softening Point (R.&B.)	, C	55.0	52.5	54.0	52.8	55.0
	, F	131	126.5	129.2	127	131
Ductility (5 cm/min)	@ 25 C, cm	150+	101.5	150+	94.3	150+
Ductility (5 cm/min)	@ 4 C, cm	0	5.7	0	16.2	59.0
Force Ductility						
"True" Parameters	@ 4 C					
Maximum Stress	, psi	212	155.0	250	1210	1660
Maximum Strain	, in/in	0.15	1.94	0.33	2.27	3.31
Area Under Stress-Strain Curve	, psi	16	240	53	1260	1570
Asphalt Modulus	, psi	3100	250	1900	1170	1420
Asphalt-Polymer Modulus	, psi	0	0	0	1320	1430
Toughness-Tenacity	@ 25 C					
Peak Load	, lb	266	93	150	109	91
Peak Elongation	, in	0.44	0.35	0.42	0.37	0.31
Toughness (area under load-elongation curve)	, in-lb	214	60	117	370	235
Tenacity (area under load-elongation curve after peak elongation)	, in-lb	88	10	83	305	186
Toughness-Tenacity	@ 20 C					
Peak Load	, lb	58	120	222	145	127
Peak Elongation	, in	0.11	0.44	0.48	0.47	0.40
Toughness (area under load-elongation curve)	, in-lb	22	72	125	265	426
Tenacity (area under load-elongation curve after peak elongation)	, in-lb	8	44	6	182	357
Freess Brittle Point	, C	-20.8	-26.5	-24.1	-22.6	-23.7
	, F	-5.5	-15.7	-11.3	-8.7	-10.7
LTD OVEN PROPERTIES						
Penetration						
200gm, 60 sec	@ 4 C, dmm	6	19	11	6	12
100gm, 5 sec	@ 25 C, dmm	11	31	20	10	21
Viscosity, Absolute	@ 60 C, poise	66590	53100	26808	88878	44289
Viscosity, Kinematic	@ 135 C, cSt	1210	1863	1335	1293	1391
Softening Point (R.&B.)	, C	68.9	67.8	63.9	65.0	80.0
	, F	156	154	147	149	176
Ductility (5 cm/min)	@ 25 C, cm	8.7	19.2	78.8	9.5	28

Table 3-2. Calculated binder properties (Goodrich 1988).

ASPHALT IDENTIFICATION:	A	B	C	P1	P2
TEMPERATURE OF EQUIVALENT STIFFNESS (20000 psi, 10000 s)					
Original asphalt, C	-16.8	-38.0	-21.0	-27.0	-25.5
, F	1.8	-36.4	-5.8	-16.6	-13.9
After RTFO aging, C	-14.8	-24.5	-18.0	-20.0	-20.0
, F	5.4	-12.1	-0.4	-4.0	-4.0
EQUAL VISCOSITY (280 cSt) TEMPERATURE [57]					
EVT (original asphalt), C	138	143	148	151	155
, F	280	289	298	304	310
EVT (RTFO residues), C	153	154	153	166	153
, F	308	309	308	330	307
PENETRATION INDEX					
(Pen @ 25 C - R&B SP) original	-1.11	-0.18	-1.18	-0.01	4.85
(Pen @ 25 C - R&B SP) after RTFO	-0.83	0.26	-0.34	-0.04	0.80
(Pen @ 25 C - R&B SP) after LTD	-0.39	1.32	-0.20	-1.12	2.31
(Pens @ 25 C, 4 C) original	-3.57	-0.83	-2.88	-2.52	-2.85
(Pens @ 25 C, 4 C) after RTFO	-3.22	-2.14	-2.84	-2.71	-2.71
(Pens @ 25 C, 4 C) after LTD	-2.07	-2.31	-2.81	-1.86	-2.56
PENETRATION RATIO					
(Pens @ 25 C, 4 C) original	30.8	49.2	44.6	35.1	34.5
(Pens @ 25 C, 4 C) after RTFO	39.4	48.6	43.8	42.6	40.8
(Pens @ 25 C, 4 C) after LTD	54.5	81.3	55.0	60.0	57.1
PENETRATION/VISCOSITY NUMBER, PVN					
(25 C, 60 C) original	-0.85	0.01	-0.76	0.14	0.27
(25 C, 60 C) after RTFO	-0.86	0.15	-0.43	0.08	-0.18
(25 C, 60 C) after LTD	-0.02	1.27	0.02	-0.11	0.52
(25 C, 135 C) original	-1.23	0.03	-0.56	0.41	0.58
(25 C, 135 C) after RTFO	-1.22	0.06	-0.38	0.50	-0.02
(25 C, 135 C) after LTD	-0.88	0.68	-0.26	-0.86	-0.17
VISCOSITY-TEMPERATURE SUSCEPTIBILITY, VTS					
(60 C, 135 C) original	3.76	3.41	3.44	3.26	3.22
(60 C, 135 C) after RTFO	3.80	3.44	3.46	3.18	3.36
(60 C, 135 C) after LTD	3.93	3.56	3.57	3.89	3.70
VISCOSITY RATIO					
RTFO Vis. @ 60C/ original Vis. @ 60 C	1.91	2.93	1.75	2.59	1.32
LTD Vis. @ 60 C/ original Vis. @ 60 C	31	37	11	37	20
% of ORIGINAL PENETRATION					
after RTFO	63.5	55.6	85.7	53.5	64.5
after LTD	21.2	24.6	35.7	8.8	19.1

Goodrich plotted G″, G′ and eta* versus temperature for each of the five asphalts. Results are shown on Figures 3-5 through 3-9. After review of the curves for Asphalts A and B, Goodrich (1988) noted that:

- Both Asphalt A (high temperature susceptibility) and Asphalt B (low temperature susceptibility) have nearly the same ultimate viscosity at low temperatures, but Asphalt A reaches the maximum viscosity at a higher temperature. Asphalt A becomes brittle before Asphalt B.

- G′ of Asphalt A diverges from G″ much faster as the temperature increases than is the case for Asphalt B. This means that Asphalt A flows more like a Newtonian fluid at high temperatures or at low frequencies. Mixes made using Asphalt A should be more susceptible to rutting than mixes made with Asphalt B.

Low Temperature Diametral Creep

These tests were performed on 4 in. by 2.5 in. diameter high mixture specimens using an 80 psi loading applied for 60 minutes with a 30 minute recovery. Tests on the briquettes were conducted at 4°F, -7°F, 17°F and -29°F. Using the time-temperature superposition principle for linear viscoelastic materials, master curves of modulus versus temperature were developed. Limiting stiffness temperatures (LST), the temperature at which the modulus of the mix is a given value after a defined period of creep loading, were determined from the master curves (Figure 3-5). Goodrich (1988) used a limiting stiffness of 1.5×10^6 psi suggested by Schmidt (1975) and found that the LSTs for the five asphalt mixes ranged from -4°C to -22°C (Table 3-3, Figure 3-10). The polymer modified asphalts showed lower LSTs than the unmodified control Asphalt A. However, the polymer-modified binders (P1 and P2) did not have lower LSTs than the least temperature susceptible conventional asphalt (Binder B) included in the test series.

Goodrich states that the reduction of the LST did not result from the addition of the polymer. Instead, it is due to the asphalt selected for modification. Polymers increase the high temperature viscosity of an asphalt. Therefore, the polymers used in the P1 and P2 binders were added to lower than normal viscosity asphalts. Effectively, the result is an AC-20 asphalt with the low temperature properties of an AC-10, for example. Although the polymer used in P2 has a lower glass transition temperature than the polymer in P1, the LST of the P1 mix was lower. Due to slight differences in the base asphalts of the two binders. The asphalt modified in P1 remains softer at lower temperatures than does the asphalt modified in P2.

Goodrich correlated the binder test results to the LST which is assumed to be an indicator of low temperature cracking performance as shown in Figure 3-11. Penetration (4°C, 200 g, 60 sec.) showed the best correlation. Tests with poor correlation to the LST were PVN, RTFO Ductility at 4°C, RTFO Tenacity at 20°C, and RTFO Force Ductility-Asphalt Polymer Modulus at 4°C. Goodrich noted that these are the tests that are often used to show the enhancements of polymer modified asphalts but the tests do not equate with improved low temperature performance as determined by the LST.

Table 3-3. Properties of asphalt concrete mixes (Goodrich 1988).

ASPHALT IDENTIFICATION:	A	B	C	P1	P2
Stability, @ 60 C					
Hveem Stabilometer Value	45.3	40.9	43.7	41.8	40.6
Resilient Modulus					
@ 4 C , psi	3.71E+06	1.41E+06	2.05E+06	2.61E+06	2.51E+06
@ 25 C , psi	6.86E+05	1.61E+05	2.32E+05	2.70E+05	2.06E+05
@ 40 C , psi	1.53E+05	5.40E+04	4.59E+04	1.05E+05	3.45E+04
Limiting Stiffness Temperature					
10.3 GPa (1.5E+06 psi), 10000 sec , C	-4.4	-22.2	-14.4	-15.0	-17.8
, F	24	-8	6	5	0
17.9 GPa (2.6E+06 psi), 1800 sec , C	-5.0	-25.0	-15.0	-16.7	-18.9
, F	23	-13	5	2	-2
Beam Flexural Fatigue Life @ 25 C					
200 uin initial strain , cycles	85000	1300000	170000	1100000	780000
400 uin initial strain , cycles	7200	230000	30000	85000	85000
600 uin initial strain , cycles	1000	61000	11000	14000	18000
Creep (axial) @ 40 C					
172 KPa (25 psi) axial pressure,					
Deformation after 60 min , uin/in	1659	1277	1570	1338	1391
Modulus , psi	15060	19570	15910	18680	17970
Surface Abrasion, California Test No. 360					
Method "B" (steel balls), @ 40 F					
Weight loss , grams	21.3	13.5	14.9	11.4	7.2
Split Tension @ -17.8 C (0 F)					
Peak load , lbs	7300	7050	8200	8700	7450
Tensile stress at break , lb/in	581	561	652	682	583
Tensile strain at break , in/in	0.34	1.02	0.78	0.57	0.47
RECOVERED ASPHALT PROPERTIES					
Penetration					
200gm, 60 sec @ 4 C , dmm	11	29	19	17	19
100gm, 5 sec @ 25 C , dmm	34	57	49	83	55
Viscosity, Absolute @ 60 C , poise	7616	6133	4455	9297	4881
Viscosity, Kinematic @ 135 C , cSt	530	753	637	1417	772
Ductility (5 cm/min) @ 25 C , cm	150+	150+	150+	150+	93
@ 4 C , cm	0	5	0	5	25

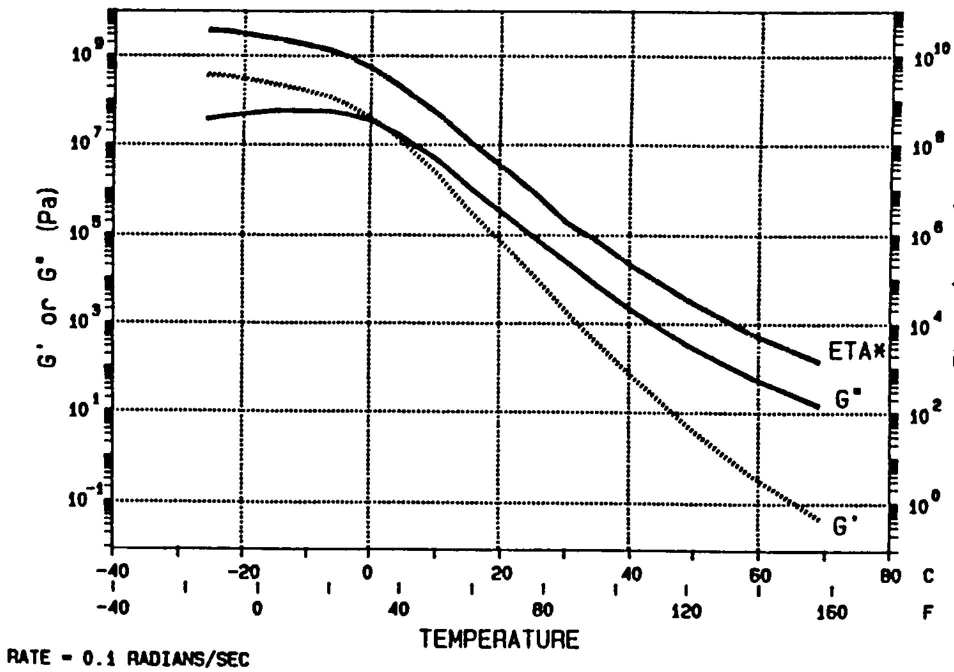


Figure 3-5 Dynamic mechanical analysis of Asphalt A (Goodrich 1988).

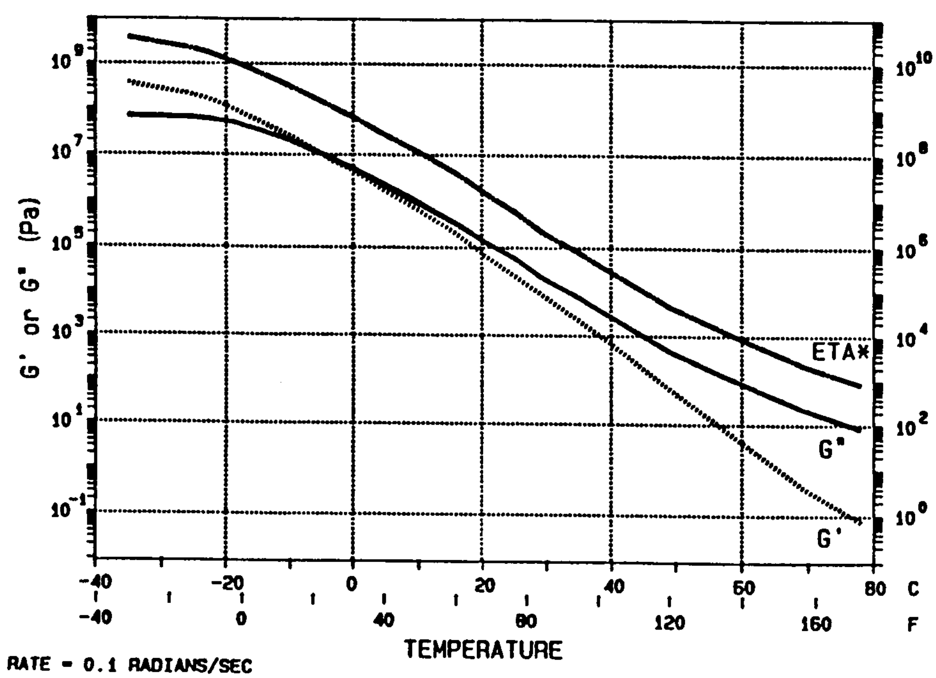


Figure 3-6. Dynamic mechanical analysis of Asphalt B (Goodrich 1988).

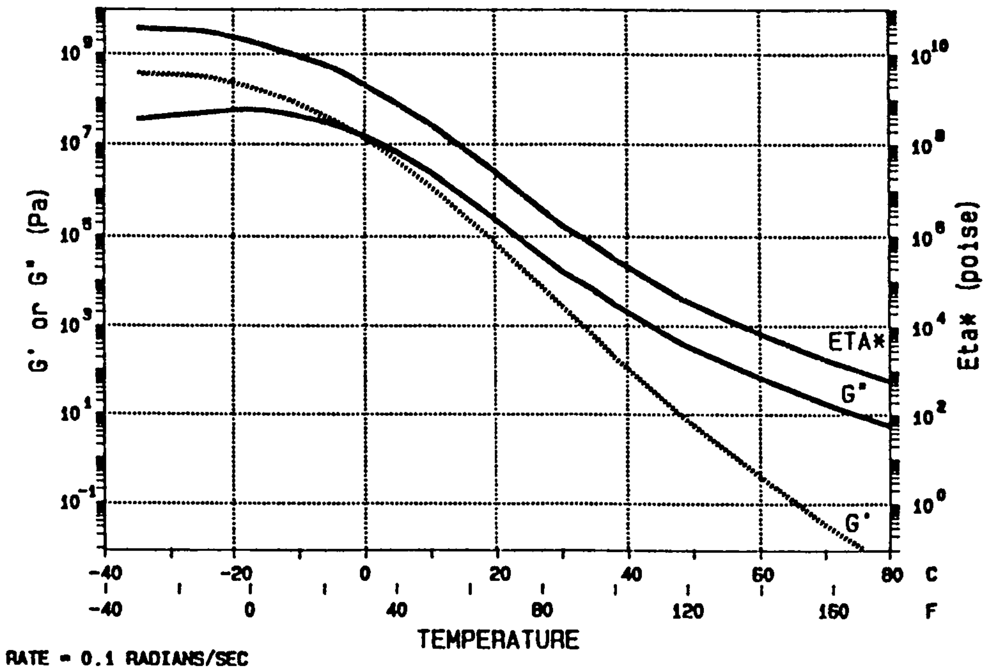


Figure 3-7. Dynamic mechanical analysis of Asphalt C (Goodrich 1988).

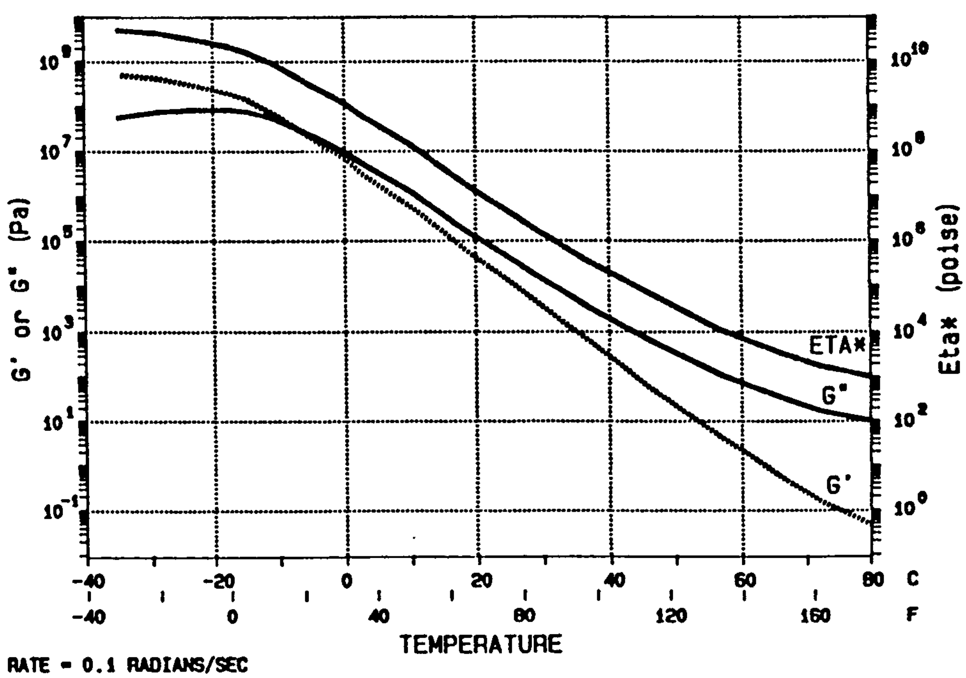


Figure 3-8. Dynamic mechanical analysis of Asphalt P1 (Goodrich 1988).

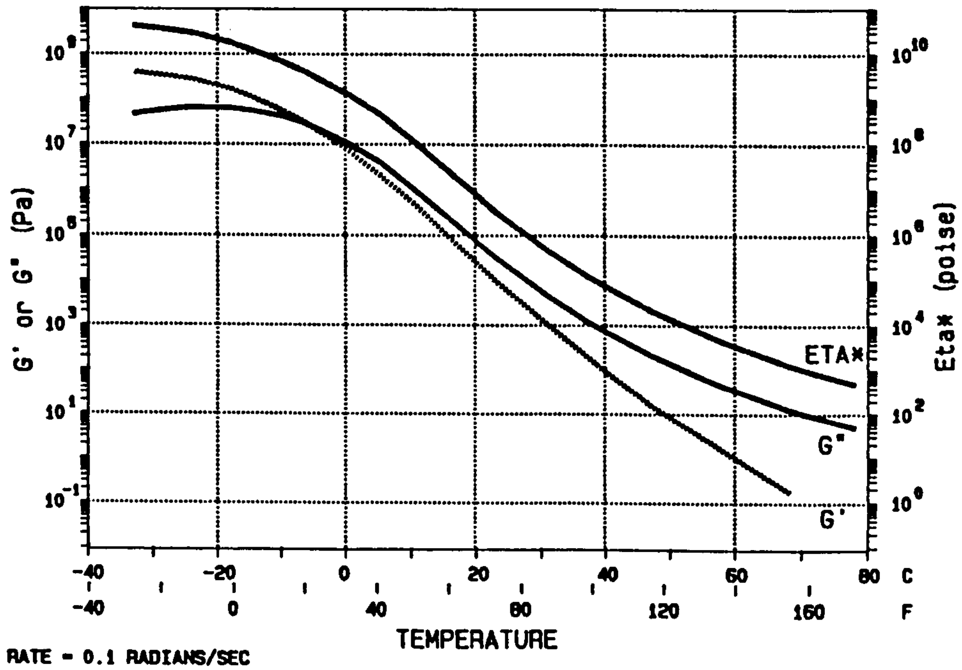


Figure 3-9. Dynamic mechanical analysis of Asphalt P2 (Goodrich 1988).

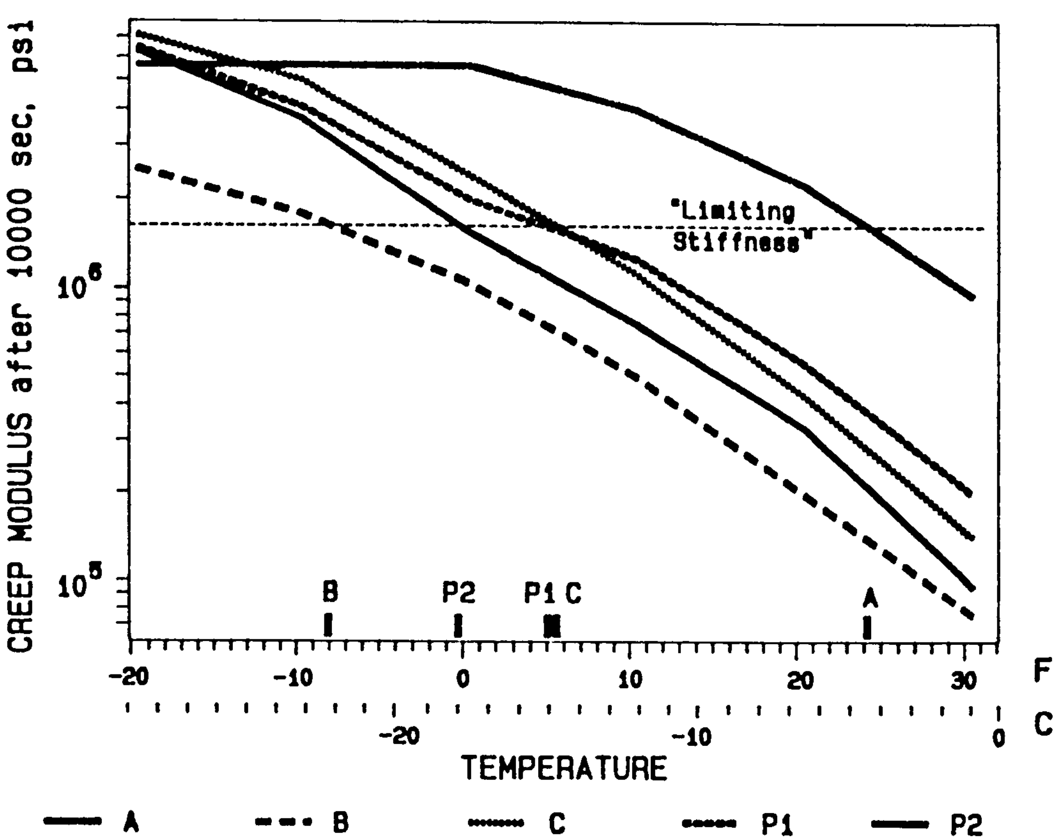


Figure 3-10. Limiting stiffness temperature of asphalt concrete (Goodrich 1988).

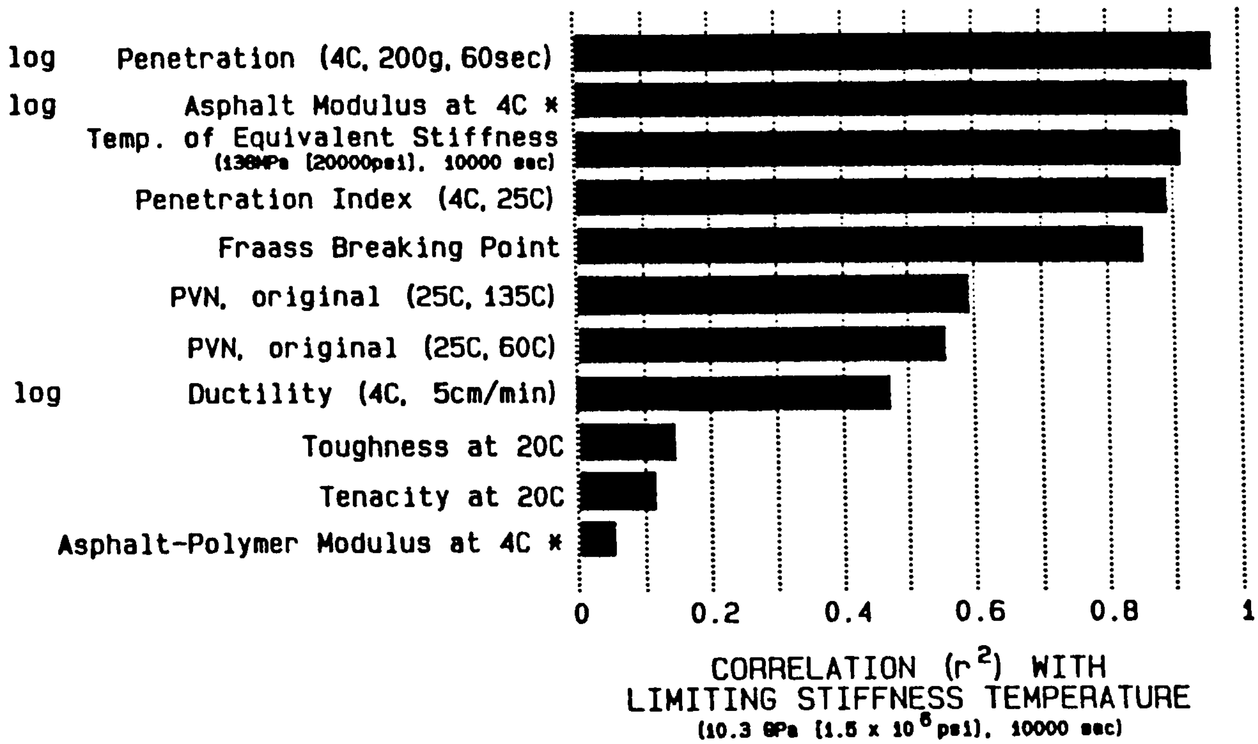


Figure 3-11. Correlation of conventional binder properties and asphalt concrete limiting stiffness temperature (Goodrich 1988).

From dynamic mechanical analysis of the binders over the same temperature range as used to test the asphalt concrete briquettes (-30°C to +4°C), the temperature at which the loss tangent, (tan delta or G″/G′), equaled 0.4 (Figure 3-12) was determined. This is the point where the contribution of the elastic nature, G′, of the binder is 2.5 times the contribution from the viscous nature, G″. The empirical correlation between tan delta = 0.4 and LST is r^2 - 0.92. Therefore, Goodrich concluded that the association between the dynamic mechanical properties of the five tests asphalts and the LST of mixes made from those asphalts is fundamental and the peaking of the G″ (Figures 3-5 to 3-9) at low temperature is a property of an asphalt which needs to be offset to lower temperatures for improved resistance to thermally induced cracking.

Flexural Fatigue

Controlled tests for stress flexural fatigue were performed at 25°C on 1.5 in. by 1.5 in. by 15 in. specimens using a 0.05-sec. load pulse at 100 cycles per minute. Fatigue life curves are shown on Figure 3-13.

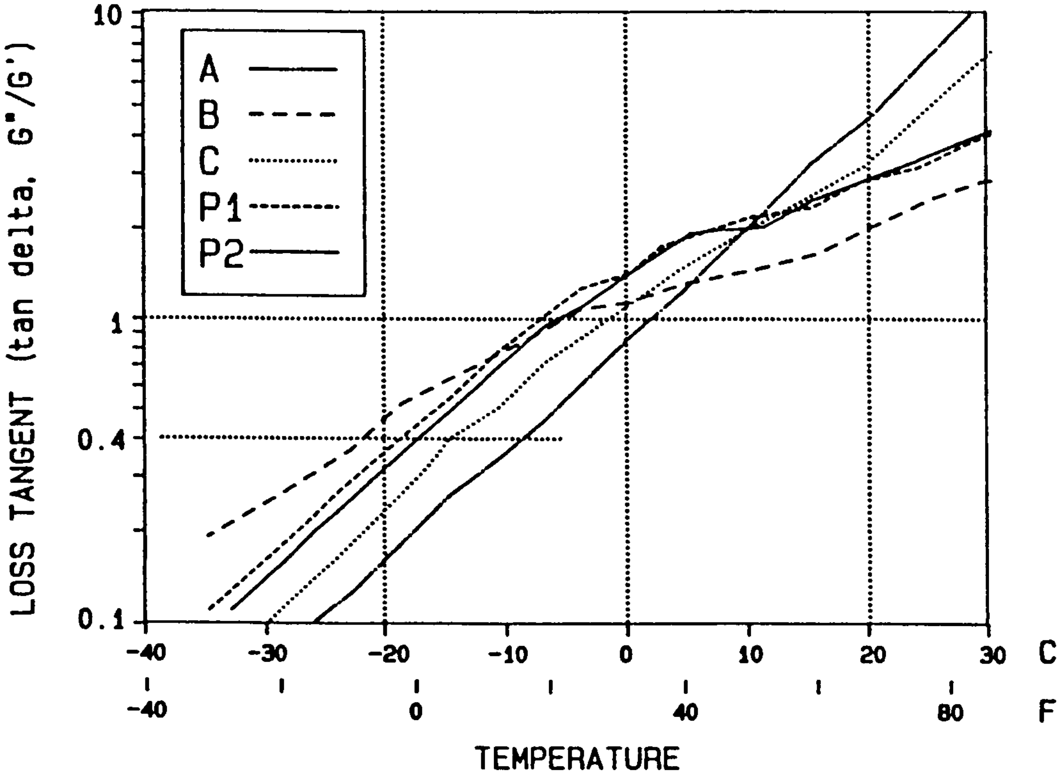
The fatigue lives of the polymer-modified asphalts P1 and P2 at 400 microinch/in. strain were improved significantly over the unmodified Binder A. However, the polymer-modified binder mixes did not have greater fatigue life than the Binder B. Goodrich (1988) stated that this was unexpected. Conventional tests indicated that the stiffer the binder at low temperatures, the poorer the beam flexural fatigue Figure 3-14, Tables 3-1 and 3-2). Penetration (4°C, 200 g, 60 sec.), Force Ductility-Asphalt Modulus at 4°C, and temperature of equivalent binder stiffness (20000 psi, 10000 sec) correlated well ($r^2 > 0.95$) with the flexural fatigue life at 25°C. As with the low temperature creep test, the RTFO Ductility at 4°C, RTFO Force Ductility-Asphalt Polymer Modulus at 4°C and the RTFO Tenacity at 25°C did not correlate well with beam fatigue life.

From dynamic mechanical testing, the loss tangent (tan delta or G″/G′) was found to correlate extremely well ($r^2 = 0.98$) to flexural fatigue life at 25°C, whereas the storage and loss modulus (G′ and G″) separately had a poor correlation (Figure 3-15). Goodrich (1989) noted that Brodynyan (1958) has suggested that asphalts with higher storage modulus (G′) and loss tangent (tan delta) may have improved fatigue resistance.The data obtained in this study support the second part of the suggestion.

Goodrich (1989) noted that this may be a case of cross correlation. Re-examination of Figure 3-12 shows that the asphalts with high loss tangent (relatively high G″) at low temperature appear to be the same binders which show relatively low loss tangents at 25°C.

High Temperature Creep

Creep measurements were conducted at 40°C on 4 in. diameter by 8 in. high cylindrical specimens. After preload conditioning of the specimen (Yao and Monismith, 1986), creep deformation was measured during a 60-minute loading period and a 30-minute recovery period. Creep deformation (Figure 3-16) versus time showed that polymer modification of



RATE = 0.1 RADIANS/SEC

Figure 3-12. Loss tangent vs. temperature (RTFO Residue) (Goodrich 1988).

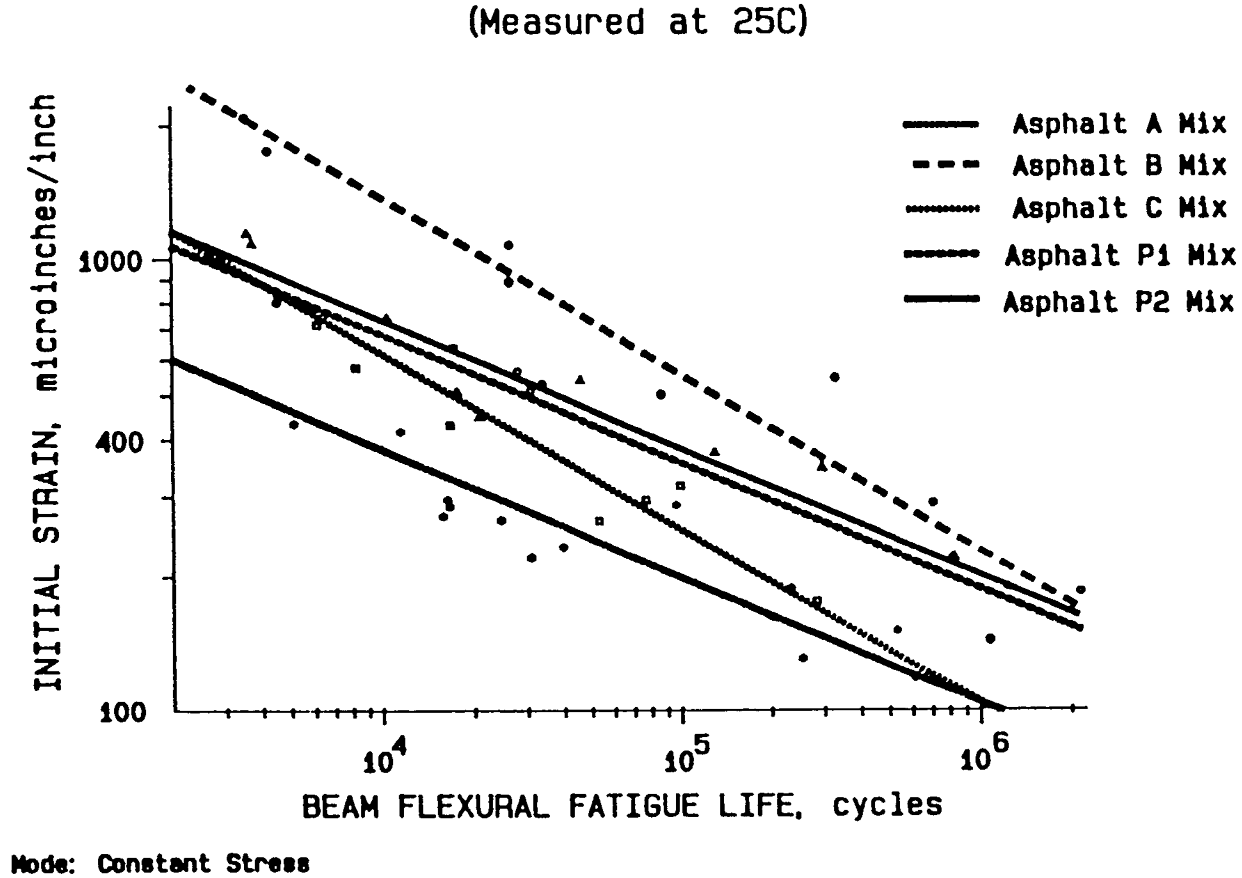


Figure 3-13. Fatigue life vs. initial strain (Goodrich 1988).

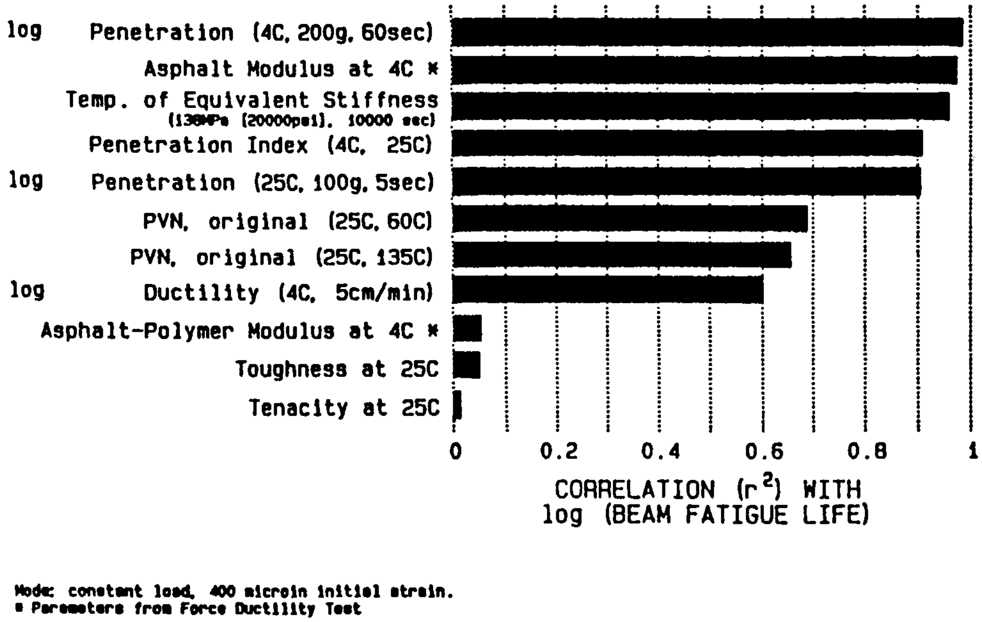


Figure 3-14. Correlation of conventional binder properties and beam flexural fatigue life at 25°C (Goodrich 1988).\

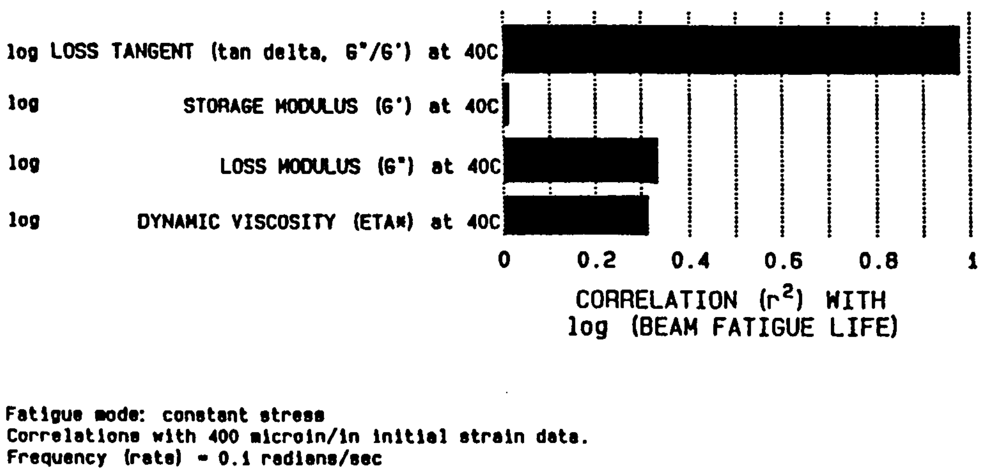


Figure 3-15. Correlation of dynamic mechanical properties with beam flexural fatigue life at 25°C (Goodrich 1988).

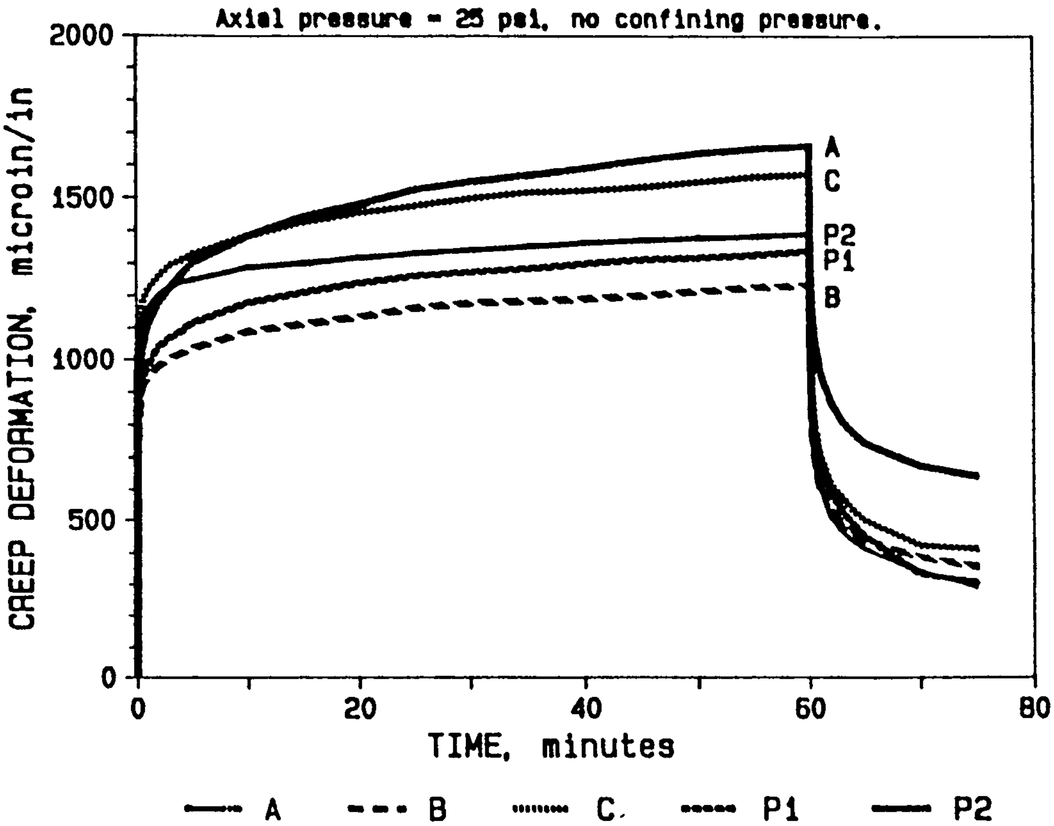


Figure 3-16. Creep deformation of asphalt concrete at 40°C (Goodrich 1988).

Binder A did reduce creep but not to the level of Binder B. As with the fatigue data, Goodrich (1988) stated that this was also unexpected.

Correlations between the creep deformation of the mixes and conventional binder properties are given in Figure 3-17. No conventional test gave an r^2 above 0.9. The best correlations were with tests at low temperature (4°C). The dynamic mechanical data showed that the loss tangent of the binder had an excellent correlation with the creep deformation with an r^2 of 0.98 (Figure 3-18). The storage modulus G′ also had a fairly high correlation with an r^2 of 0.76.

Laboratory Aging

The five binders were subjected to an extended Tilt-Oven Durability test. The test is a 7-day, 111°C version of the RTFO test and was designed to approximate the properties of asphalt recovered from cores aged for two years in the California desert (Kemp and Predoehl, 1981).

Measurement of oxidation by neutron activation and infrared analysis (Figure 3-19) showed that modified asphalts oxidize just as conventional asphalts despite only relatively small viscosity changes after aging. The bar graphs in the upper right corner in each of the five plots indicate the absolute increase in oxygen content was similar for all five asphalts (the unaged oxygen content of asphalt P1 is higher due to the presence of oxygen in its polymer). Goodrich concluded that the polymer modification does not reduce binder oxidation.

Goodrich used size exclusion chromatography (SEC) analysis to study the changes in apparent molecular size distribution. Figure 3-20 shows the large molecular size portion of the chromatogram. Larger size fractions are eluted first and appear on the left side of the plots. The height of the traces is in proportion to the amount of material passing out of the columns at a particular time. The chromatograms indicate a normal shift in the asphalt size distribution as a function of aging, that is, an increase in the large apparent size. The polymer in the unaged binder P2 is revealed as the peak centered at 25 minutes retention time. Note the changes that occur in the P2 polymer as a function of aging. The distribution of the polymer molecular weight becomes broader and shifted to a smaller average size. Goodrich stated that the P2 polymer is not stable and exhibits chain scission as a result of oxidation. The polymer in binder P1 is not apparently affected by aging since the peak (approximately 26.5 minutes retention time) does not shift. The increased height of the peak is accounted for by the changes in the aged asphalt. Goodrich concluded that for binder P2, the lower viscosity ratio (Table 3-2) is due to the degradation of the polymer concurrent with normal oxidation of the base asphalt. The bottom line is a reduced apparent increase in viscosity.

Dynamic mechanical analysis of LTD residues was also conducted. Plots of complex modulus, $|G^*|$, versus temperature are shown in Figure 3-21. In the lower temperature ranges, Binder A has the highest complex modulus, followed in order of decreasing modulus by Binder C, P2, B and P1. According to Sisko & Brunstrum (1969), "Large

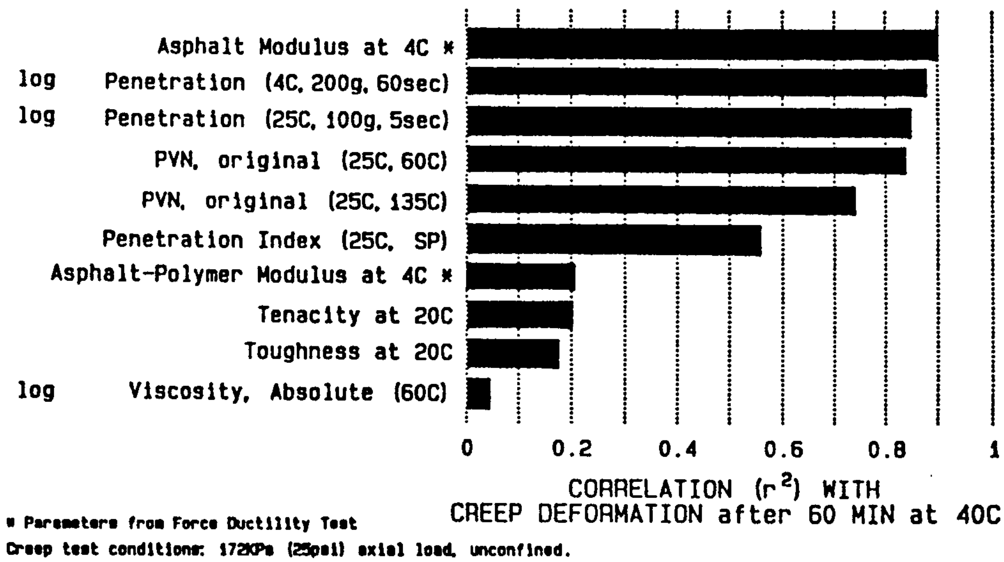


Figure 3-17. Correlation of conventional binder properties with asphalt concrete creep deformation at 40°C (Goodrich 1988).

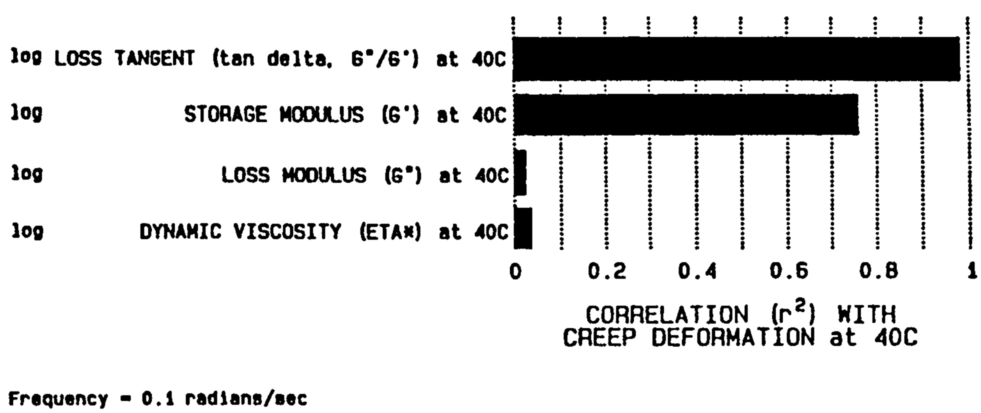


Figure 3-18. Correlation of dynamic mechanical properties and creep deformation of asphalt concrete at 40°C (Goodrich 1988).

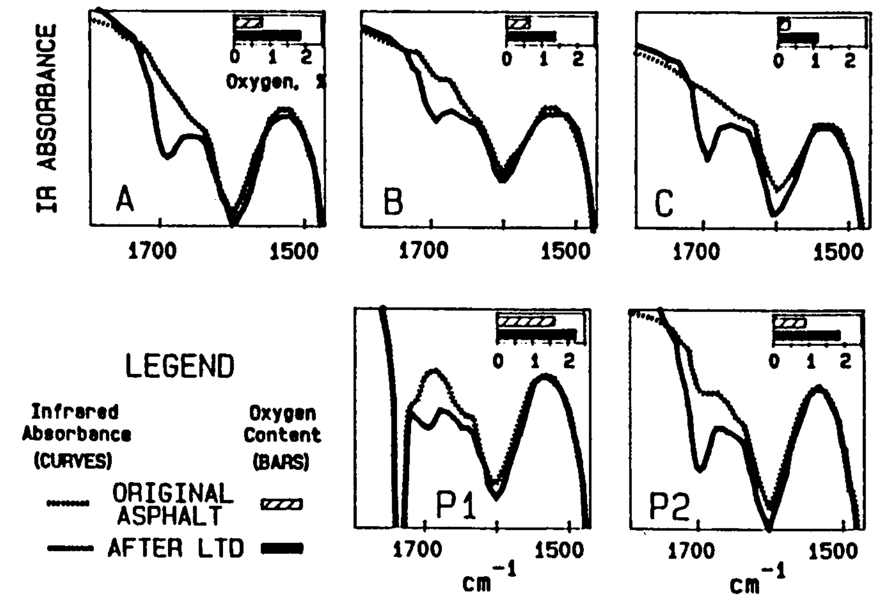


Figure 3-19. Oxidation of asphalts (Goodrich 1988).

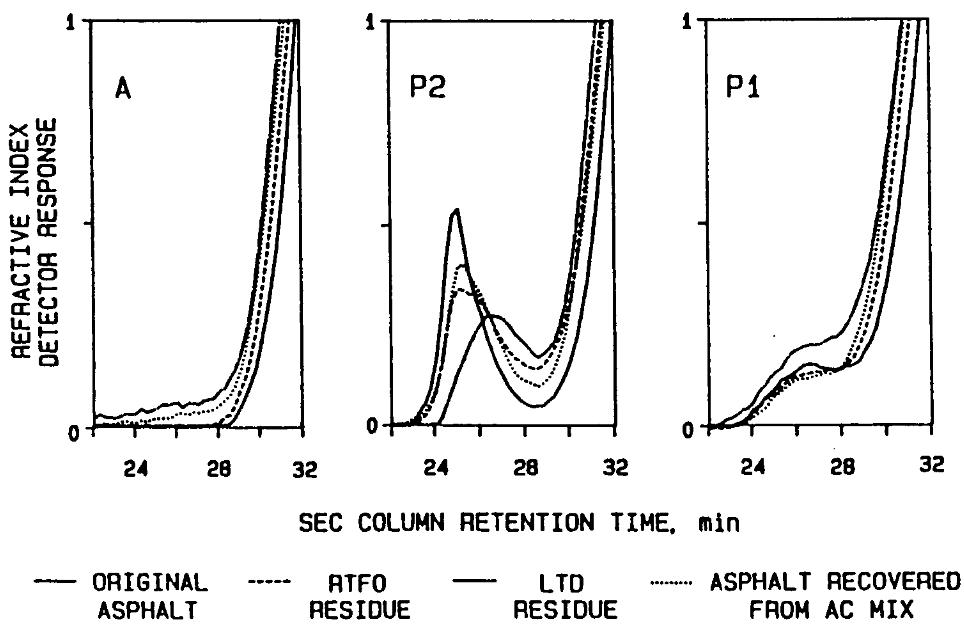


Figure 3-20. Effect of aging on the molecular size distribution of polymers in asphalt (Goodrich 1988).

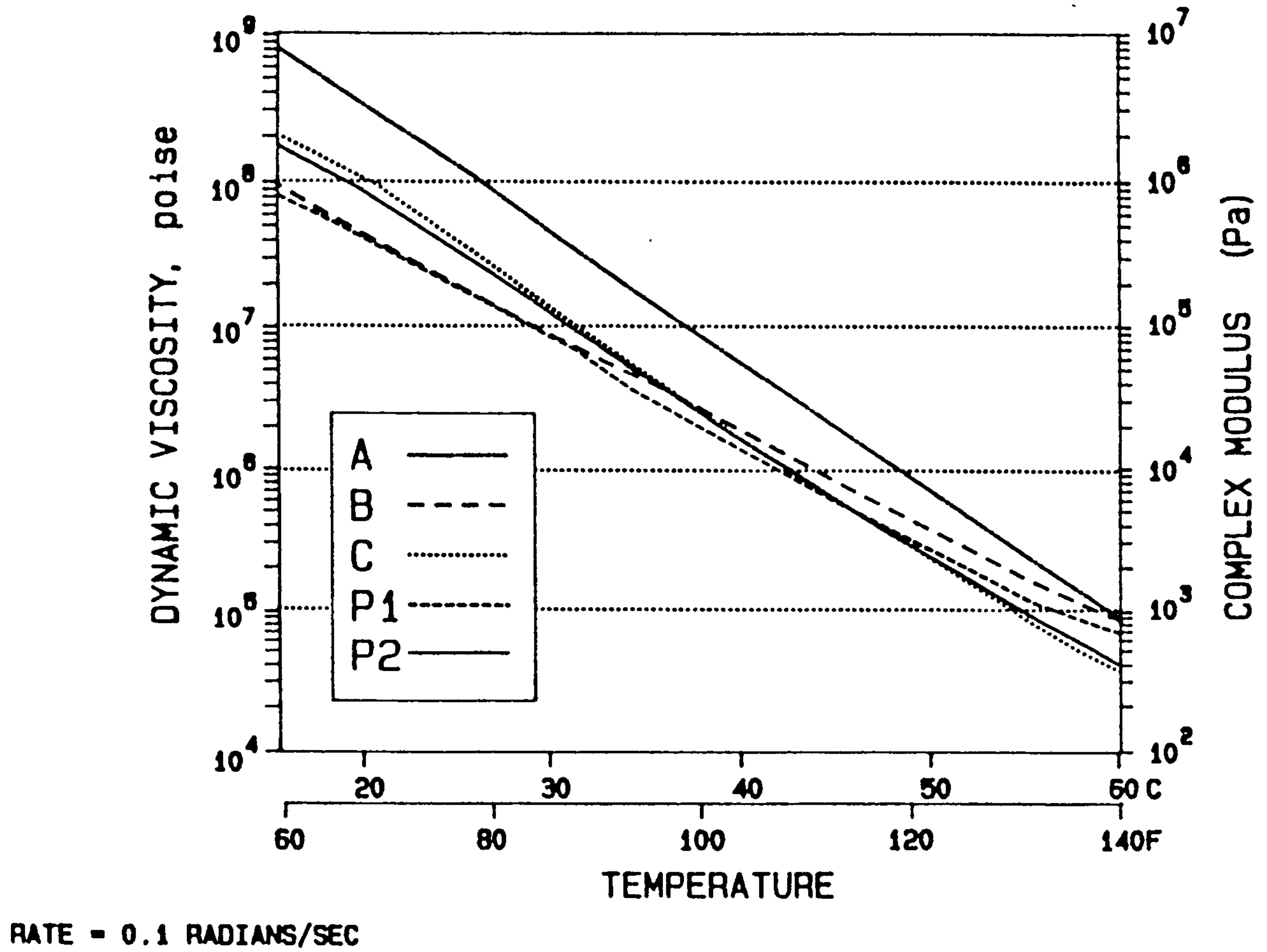


Figure 3-21. Complex modulus vs. temperature for LTD residues (Goodrich 1988).

increases, induced by aging, in the hardness of the asphalt binder (as measured by the complex modulus) are associated with road cracking." Using Sisko's findings, Goodrich stated that extended aged Asphalts P1 and B could be expected to show the least "road cracking" of the five asphalts.

Conclusions

From the results from the low temperature creep, the fatigue, and the permanent deformation studies together, Goodrich (1988) found that those asphalts that performed best in all three experiments have a molecular structure (association or entanglement) which provides elastic stability at high temperatures. He stated that asphalts may be thought of as having an elastic sponge structure that is filled with a viscous fluid. At low temperatures, the viscous flow of the asphalt which permits creep and thus resists thermally induced cracking is principally derived from the viscosity of the modified base asphalt. As the temperature of the binder increases, the viscosity of the base asphalt lowers, this allows the elastic nature of the sponge to become a functional property of the asphalt.

Goodrich also found that a conventional asphalt Binder B, consistently performed better than the polymer modified asphalts. He concluded that the proper balance of the viscous fluid-elastic sponge properties existed naturally in some asphalts, which occurs because of an effective elastic network created by molecular associations, and could also be formed by creating molecular entanglement in an asphalt through the use of high molecular weight polymeric additives. He concluded that some asphalts may not require modification of their natural viscoelastic properties to improve their performance in asphalt concrete.

Thus, Goodrich recommended that those tests that merely characterize the presence of modifiers in asphalt binders be distinguished from those tests that provide data that correlate with the improved performance of asphalt concrete mix. He concluded that some conventional asphalt tests were usefully related to mix performance properties; while other tests, especially those involving very high strains, were not.

3.2 New Mexico - Styrenic Block Copolymers

Description

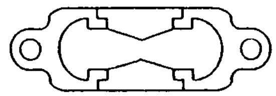
Shuler et al. (1987) reported mechanical properties of asphalts containing styrenic block copolymers and of dense-grade asphalt concrete mixtures made with these binders. The purpose of the study was to characterize the physical and rheological properties of the asphalt binder and the performance of asphalt concrete prepared with these binders so that desirable properties in the binder that lead to performance improvements in the asphalt concrete could be determined.

Exxon Baytown AC-5 and AC-20 asphalts were used as the control binders. Polymer-modified asphalt samples were prepared by blending styrenic block copolymers with the AC-5 asphalt at 3% and 6% by weight of the total blend. The 3% binder contained 1.5% each of styrene-butadiene (Kraton D1118X Rubber) and a styrene-butadiene-styrene block copolymer (Kraton D1101 Rubber). The 6% polymer-modified blend contained the same amount of polymers at levels of 3% each. Following the addition of the polymer to the asphalt, the material was mixed with a high shear mixer for approximately 30 minutes at 180°C (356°F) to obtain a homogeneous mixture.

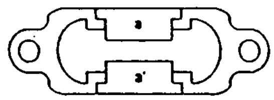
Force Ductility Test Description

The force-ductility test is a modification of the asphalt ductility test (ASTM D 113). The principal alteration of the test consists of adding two force cells in the loading chain. A second major alteration of the standard ASTM procedure involves the test specimen shape. The mold is modified for force ductility testing by fabricating new pieces a and a^1 (Figure 3-22). This mold produces a test specimen with a constant cross-sectional area for a distance of approximately 3 cm. The modified shape of the force ductility specimen allows computation of material stress and strain characteristics. The polymer-modified asphalts evaluated have two characteristic primary and secondary loading regions in stress-strain plots (Figure 3-23). The modulus of elasticity was evaluated by measuring the slope of the loading portion of the stress-strain curve in both regions. The initial slope of the stress-strain curve in the linear region under primary loading is referred to as the "Asphalt Modulus." The second slope observed is secondary loading and is referred to as the "Asphalt-Polymer Modulus."

Unmodified asphalt cement exhibits a stress-strain curve that appears like the left half of the polymer-modified stress-strain curve. However, as unloading occurs after peak engineering stress, both modified and unmodified asphalt cement unload to the point where the polymer-modified asphalt demonstrates secondary loading. At this point, the unmodified asphalt cement continues to unload while the modified asphalt begins to increase in load. Shuler et al. (1987) attribute this secondary increase in load to the presence of the polymer, and state that the polymer, at this point, is beginning to carry the load. Thus, the terms asphalt modulus and asphalt-polymer modulus were chosen.



ASTM D-113 ductility mold.



Force-ductility mold.

Figure 3-22. Standard ductility mold and force-ductility mold (Shuler et al., 1987).

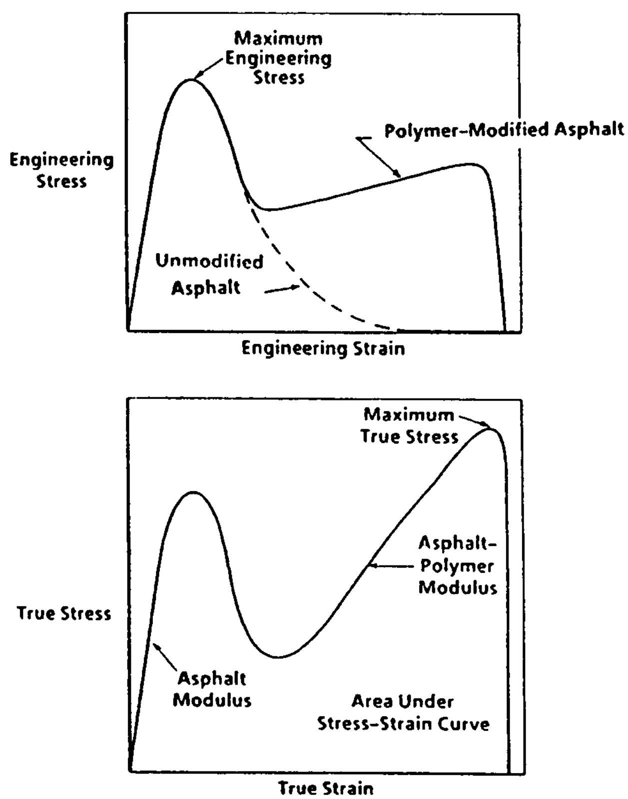


Figure 3-23. Force-ductility characteristics of polymer-modified asphalt (Shuler et al., 1987).

Results

Binder Physical Properties

Binder property data for both polymer-modified and unmodified asphalts are shown in Table 3-4. The polymer-modified asphalts demonstrated increased ring-and-ball softening point, reduced penetration, increased ductility, and increased toughness and tenacity. These effects become more pronounced with increasing polymer content. For example, the AC-5 + 6% polymer has a ring-and-ball softening point well above that of the AC-5 and AC-20 controls (82°C versus 44°C and 52°C, respectively). The 4°C (39°F) ductility of the AC-5 + 6% polymer (63 cm) is also significantly higher than that of the unmodified AC-5 (10 cm) or the AC-20 control (1 cm). Also, the toughness and tenacity of the polymer-modified AC-5 specimens are substantially higher than those of the AC-5 and AC-20 controls.

The absolute and Brookfield viscosities of the polymer-modified AC-5 + 3% specimen are approximately equivalent to those of the unmodified AC-20 specimen. Due to the relatively low shear rates generated in the vacuum and rotational viscometers, the absolute viscosity increased by a factor of five when the polymer content in the AC-5 was increased from 3 to 6%. However, the Brookfield viscosity of the AC-5 + 6% polymer specimen is only a factor of three greater than the AC-5 + 3% polymer specimen at 100°C.

Force-Ductility Testing

Parameters from force-ductility testing at 4°C and 1 cm./min. deformation rate are shown in Table 3-5. The values shown in Table 3-5 are averages obtained from each of three replicate tests conducted for each material. Analysis of variance (ANOVA) techniques were used to determine if significant differences exist between material types for each parameter. The lower-case letters in parentheses in Table 3-5 indicate whether means are significantly different using the Newman-Keuls multiple range test. Letters of the same type for each parameter indicate no significant difference in means at an alpha of 0.05.

According to Shuler et al.(1987), two parameters that appear to measure asphalt properties are maximum engineering stress and asphalt modulus. These two parameters indicate no change in mean value for the AC-5, AC-5 + 3%, or AC-5 + 6%, but indicate a significant difference for the AC-20. The other three parameters, maximum true stress, area under true stress-strain curve, and asphalt polymer modulus appear sensitive to polymer-modified material properties as shown by significant differences in mean values between the AC-5, AC-5 + 3%, and AC-5 + 6% specimens. Shuler et al.(1987) concluded that asphalt-polymer modulus is aptly named due to its ability to distinguish between the quantity of polymer present in the AC-5. Likewise, they concluded that asphalt modulus is a proper label for the primary stress-strain slope since it distinguishes between AC-5 and AC-20, but is not sensitive to the presence of polymer in the AC-5. The parameters maximum true stress and area under the stress-strain curve for the AC-20 are not significantly different than the AC-5 + 3%.

Table 3-4. Physical properties of unmodified and polymer-modified binders (Shuler et al., 1987).

Properties	Binder Types			
	AC-5[a]	AC-5 + 3%[b]	AC-5 + 6%[c]	AC-20[a]
Ring-and-ball softening point, °C	4	51	82	52
Penetration @ 25°C (77°F), dmm	122	100	83	64
Ductility @ 4°C (39°F), cm	10	48	63	1
Toughness, in. lb	49	165	328	167
Tenacity, in. lb	18	126	284	22
Absolute viscosity @ 60°C (140°F), poise	691	1855	9940	1958
Brookfield viscosity, cP				
@ 100°C (212°F)	1700	3250	9500	2750
@ 120°C (248°F)	500	910	2550	750
@ 140°C (284°F)	200	370	850	325
@ 160°C (320°F)	88	160	385	125
@ 180°C (356°F)	50	88	205	75

[a] Asphalt cement binders from Exxon, Baytown, Texas Refinery.
[b] 3% by weight neat Kraton polymer, D1101/D1118X (50:50).
[c] 6% by weight neat Kraton polymer, D1101/D1118X (50:50).
Conversion factors:
1 cm = 0.39 in.
1 in. lb = 0.1129 N · m.

Table 3-5. Force-ductility parameters for unmodified and polymer-modified binders (Shuler et al., 1987).

Parameter	Material			
	AC-5[a]	AC-5 + 3%[b]	AC-5 + 6%[c]	AC-20
Maximum engineering stress, psi[d]	26.2 (a)	28.0 (a)	28.0 (a)	99.5 (b)
Maximum true stress, psi	34.3 (a)	170.0 (b)	592.0 (c)	140.9 (b)
Area under true stress-strain curve, psi	41.5 (a)	162.7 (b)	357.7 (c)	191.0 (b)
True asphalt modulus, psi	155.7 (a)	163.7 (a)	170.8 (a)	454.4 (b)
True asphalt-polymer modulus, psi	n/a	156.4 (a)	786.3 (b)	n/a

NOTE—Letters of the same type in parentheses indicate no significant difference exists between binders for a given parameter at alpha = 0.05.

[a] Asphalt cement binders from Exxon, Baytown, Texas Refinery.
[b] 3% by weight neat Kraton polymer, D1101/D1118X (50:50).
[c] 6% by weight neat Kraton polymer, D1101/D1118X (50:50).

Dynamic Rheological Properties

Plots of complex viscosity, |Eta|*, and loss tangent, tan δ, versus temperature are presented on Figures 3-24 and 3-25. The |Eta|*-temperature curves for the unmodified asphalt specimens are approximately parallel. However, |Eta|* for the AC-20 control specimen is approximately four times that of the AC-5 control, and the |Eta|* values for the polymer modified AC-5 binders are essentially equivalent to that of the unmodified binder up to 25°C. However, between 25° and 75°C, |Eta|* for the polymer-modified binders decreases more slowly with increasing temperature; this effect is more pronounced at the higher polymer content. Tan δ values of the unmodified and polymer-modified AC-5 and AC-20 specimens are essentially equivalent from -10 to +20°C. Over this temperature range, tan δ increases from approximately 0.5 to 2.5. However, above 25°C, tan δ for the unmodified AC-5 increases rapidly and at 50°C (122°F), the unmodified AC-5 and AC-20 asphalts are essentially purely viscous fluids (high tan δ). In contrast, at high polymer levels, tan δ is lower indicating a more elastic material.

Indirect Tensile Test

Indirect tensile peak stress and peak strain were measured on asphalt concrete samples according to ASTM C 496 at -18°C, 25°C and 40°C. Results are presented in Figures 3-26 and 3-27. Although a small difference exists between the AC-5 and AC-5 + 3%, the most pronounced effect on tensile properties occurs for binder modified with 6% polymer. Tensile stress at failure for the AC-5 + 6% mixtures is significantly higher than the control or AC-5 + 3% mixes at all three temperatures tested. The AC-5 + 6% mixtures at -18°C have significantly higher tensile strains at failure than the control or AC-5 + 3% mixes; however, at other temperatures, tensile strain does not appear to be affected significantly by polymer addition.

Conclusions

Shuler et al. (1987) noted that those properties that are measured at relatively low strains at low to moderate temperatures--the asphalt modulus in the force ductility test, and |Eta|*--are roughly equivalent for the polymer-modified and unmodified asphalt binders and the asphalt concrete. However, tests with relatively large deformations of the specimens at low temperatures--toughness and tenacity by the Benson method, the asphalt-polymer modulus phase of the force-ductility test, and the indirect tensile measurements--show improved properties for the polymer-modified materials. This was especially apparent when the binder contained 6% block copolymer. Shuler et al. (1987) stated that (a) modification of the asphalt binder with SBS block copolymers does impart elastomeric characteristics to the binder, and (b) the mechanical properties of the binder and the asphalt concrete with the modified binder are improved under large deformations.

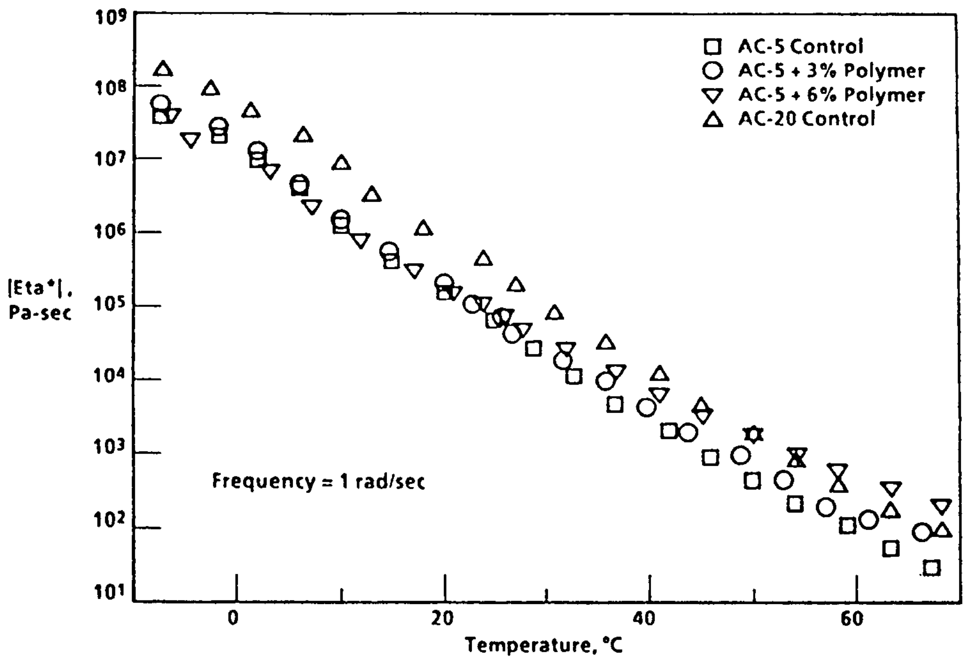


Figure 3-24. |Eta*| versus temperature (Shuler et al., 1987).

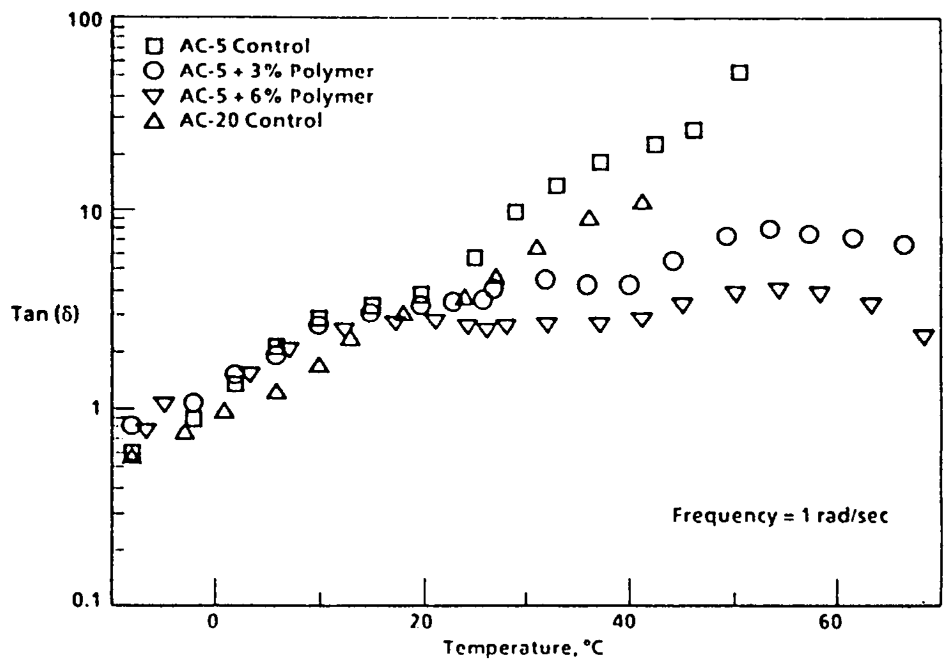


Figure 3-25. Tan(δ) versus temperature (Shuler et al., 1987).

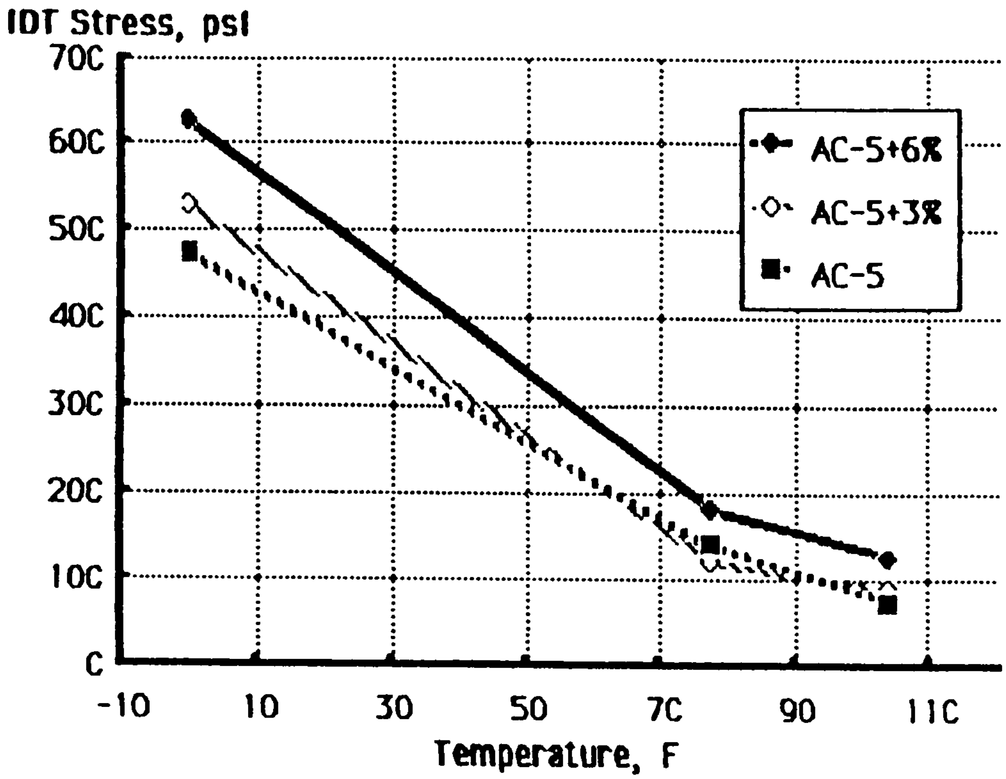


Figure 3-26. Indirect tensile stress versus temperature (Shuler et al., 1987).

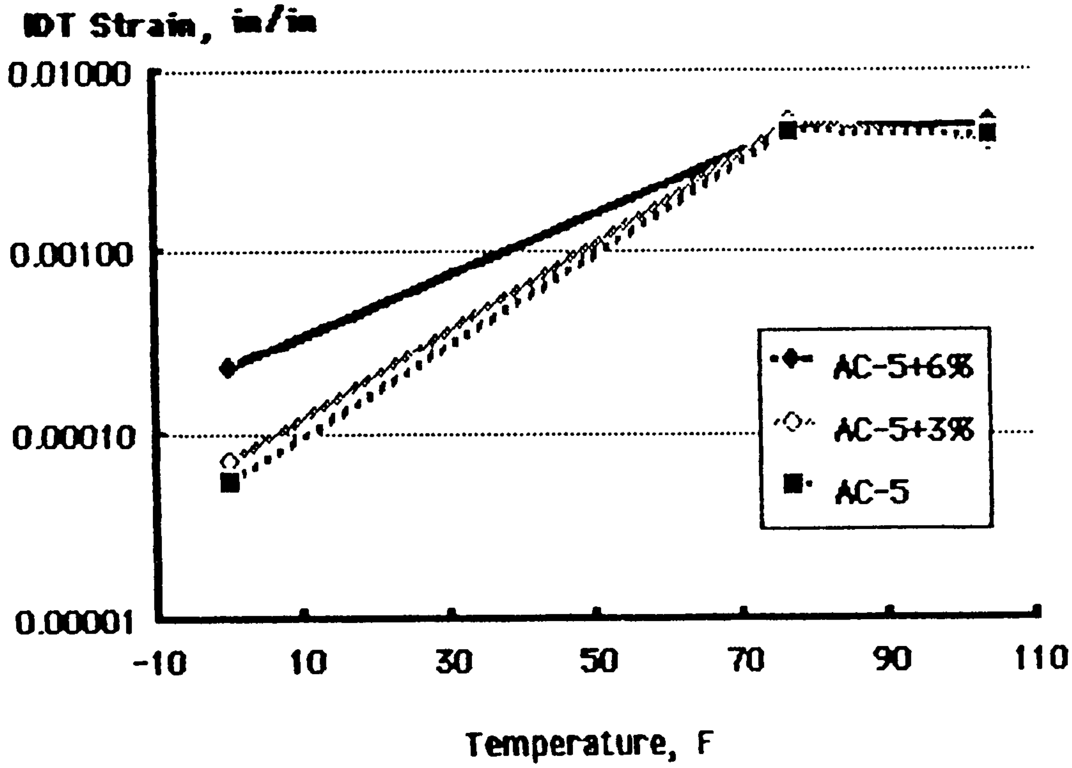


Figure 3-27. Indirect tensile strain versus temperature (Shuler et al., 1987).

Additionally, Shuler et al. (1987) noted the following conclusions:

- The addition of certain styrenic block copolymers to Exxon Baytown AC-5 at 3 and 6% by weight imparts increased viscosity, ductility, softening point, toughness, and tenacity to the original asphalt.

- Rheology of the modified binders indicates that these materials exhibit much greater elasticity at elevated temperatures compared to the control binders.

- Rheology of the modified binders indicates that elastic modulus is reduced at low and increased at high test temperatures compared to the control materials.

- Two moduli can be evaluated from force ductility testing of polymer-modified asphalt. The first appears to be a measure of the unmodified asphalt modulus of elasticity. The second appears to represent a composite modulus, or asphalt-polymer modulus, unique to polymer-modified asphalt.

- Indirect tension results for polymer-modified asphalt concrete indicate the following differences compared with control asphalt concrete
  (*a*) Increased failure tensile stress at -18°C, 25°C, and 40°C, and
  (*b*) Increased failure tensile strain at -18°C, no change at 25°C and 40°C.

3.3 Texas A & M - Various

Description

The researchers at Texas A & M University conducted a study to evaluate the performance of materials added to asphalt concrete mixtures for the purpose of reducing pavement cracking with or without rutting potential (Button et al., 1987). A laboratory test program was designed to examine stiffness, brittleness, and flexibility at low temperatures and at high loading rates, and to evaluate the resistance to fatigue-type tensile loads caused by vehicular loading and thermal variations. Asphalt cements with and without additives were tested for chemical, rheological, elastic, fracture and thermal properties, as well as sensitivity to heat and oxidation and also compatibility between asphalts and additives. Asphalt concrete mixtures were tested for stability, compactability, and water sensitivity, as well as stiffness, tensile, fatigue, and creep/permanent deformation properties as functions of temperature.

The five following additives were selected:

1. Latex (emulsified styrene-butadiene-styrene),
2. Block copolymer rubber (styrene-butadiene-styrene),
3. Ethylene vinyl-acetate,
4. Finely dispersed polyethylene, and
5. Carbon black.

Two styrene-butadiene latexes were selected, Latex XUS 40052.00 (Dow Chemical) and Ultra Pave 70 (Textile Rubber & Chemical Co.). Both are anionic and contain about 70 percent solids. A thermoplastic block copolymer rubber, Kraton TR60-8774, was obtained from Shell Development Company in the form of dry crumbs. Two EVA resins, Elvax 150 (DuPont) and EX042 (Exxon) were selected. Elvax 150 was used in the mixture study. Dow 526 was the polyethylene (Novophalt) selected for most of the study (six dispersions varying in density, molecular weight, and melt index were prepared). Finally, Microfil 8 (Cabot Corp.), formed into soft pellets dispersable in asphalt, was selected for the Carbon Black preparation.

The asphalt cements used in this study were from two sources known to have different compositions and temperature susceptibility. Three grades of paving asphalt were obtained from each source: AC-5, AC-10 and AC-20 grades from a Texas Coastal Refinery and AR-1000, AR-2000 and AR-4000 from a California refinery that processes crude oil originating in the San Joaquin Valley. Table 3-6 summarizes the component composition of these asphalts. San Joaquin Valley asphalts have relatively low asphaltene contents and a high content of nitrogen bases; whereas the Texas Coastal Refinery component is a solvent for asphaltenes and makes asphaltenes compatible with the other maltene fractions.

After the asphalts and the additives have been blended, standard rheological tests were performed on the blends. These results are summarized in Tables 3-7 and 3-8. All five additives demonstrated the ability to decrease the temperature susceptibility of both asphalts. Since the additives are much more effective at increasing high temperature viscosity than in decreasing low temperature penetration, additives were incorporated in the soft asphalts for evaluation in the mixture study. Generally, the additives increase the high temperature viscosity to resist rutting, while not appreciably affecting the cracking resistance of the low viscosity base asphalts at low temperatures.

The forced ductility test, which is a modification of the asphalt ductility test, was performed on all asphalt cements and was used to measure tensile load deformation characteristics of asphalt and asphalt rubber binders. The results of this test suggest two things: 1) Changes in the stress-strain properties imparted by the additives are highly dependent upon the properties of the base asphalt; 2) the stress-strain curves develop a second peak for polymers that are dissolved in the asphalt or develop a continuous network of microscopic strands. However, it was noted in the discussion that carbon black and polyethylene threads break when pulled, so ductility may not be an appropriate evaluation for such discrete additives. Figure 3-28 shows that a relationship exists between the maximum stress of the binders and tensile strength of corresponding paving mixtures. The forced ductility test may be useful in predicting changes in mixture tensile strength when asphalt additives are employed.

Marshall and Hveem stability, resilient modulus, and direct tension tests were performed on unmodified and modified mixtures composed of river gravel with two asphalts. No single additive demonstrated the ability to produce mixtures with consistently higher values of stability, stiffness or strength. Figure 3-29 summarizes the resilient modulus results as a function of temperature for these mixtures, and Table 3-9 summarizes the indirect tension test results for the modified and unmodified mixtures.

Table 3-6. Component composition of asphalts from Texas A & M (Button et al., 1987).

Property	Texas Coastal Asphalts AC-5	AC-10	AC-20	San Joaquin Valley Asphalts AR-1000	AR-2000	AR-4000
Corbett Analysis[a]						
Asphaltenes, %	14.6	-	14.8	5.0	-	6.0
Saturates, %	13.4	-	10.1	13.7	-	10.0
Napthene 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[c] $(N+A_1)/(P+A_2)$	1.13	0.73	-	1.50	1.32	-
Sulphur, %	-	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 (3)

Table 3-7. Summary of binder data for unmodified and modified Texas Coastal asphalts from Texas A & M study (Button et al., 1987).

Test Value	AC-20	AC-5	AC-5 + 15% Microfil-8	AC-5 + 5% Dow Latex	AC-5 + 3% Dow Latex	AC-5 + 5% Kraton	AC-5 + 5% Elvax 150	AC-5 + 5% Polyethylene
Penetration@25°C(77°F) 100g, 5 sec[1]	75	194	152	114	140	103	176	105
Penetration@4°C(39°F) 100g, 5 sec	8	20	21	14	15	14	17	13
Penetration@4°C(39°F) 200g, 60 sec	28	63	66	54	57	49	54	49
Viscosity@60°C(140°F)[2]	2040	506	1850	5480	1960	6720	1160	2200
Viscosity@135°C(275°F)[3]	398	224	740	2780	1020	870	618	840
R&B Soft Point°C(°F)[4]	-	41(107)	-	63(145)	52(125)	59(138)	49(120)	52(126)
Ductility 4°C/25°C[5]	-	150+/150+	-/11	150+/150+	150+/150+	69/98	24/45	-/35
Viscosity Temp. Suscep.[6] (60°C-135°C)	3.52	3.42	2.99	2.52	2.78	2.44	2.94	2.98
Pen-Vis Number (PVN)[7]	0.6	-0.3	1.4	3.0	1.8	1.0	1.3	1.0
Pen Index[8]	-0.9	-1.0	-0.2	-0.5	-0.9	-0.2	1.2	-0.5
Penetration Ratio[9]	37	32	43	47	41	48	31	47

[1]AASHTO T53.

[2]AASHTO T202.

[3]AASHTO T201.

[4]AASHTO T49.

[5]AASHTO T51, 5^{cm}/min.

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

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

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

[9]100(Pen 39.2°F, 200 g, 60 s)/(Pen 77°F, 100 g, 5 s).

Table 3-8. **Summary of binder data for unmodified and modified San Joaquin asphalts for Texas A & M study (Button et al., 1987).**

Test Value	AR-4000	AR-1000	AR-1000+15% Microfil-8	AR-1000+5% Dow Latex	AR-1000+3% Dow Latex	AR-1000+5% Kraton	AR-1000+5% Elvax 150	AR-1000+5% Polyethylene
Penetration@25°C(77°F) 100g, 5 sec[1]	57	146	109	72	83	134	161	98
Penetration@4°C(39°F) 100g, 5 sec	4	10	10	6	6	11	12	8
Penetration@4°C(39°F) 200g, 60 sec	16	46	43	29	28	43	50	41
Viscosity@60°C(140°F)[2]	2170	498	1640	10,100	4020	1720	1180	1295
Viscosity@135°C(275°F)[3]	256	128	398	3600	1190	431	434	399
R&B Soft Point°C(°F)[4]	-	41(106)	-	67(152)	54(130)	52/(126)	45(113)	47(117)
Ductility 4°C/25°C[5]	-	150+/130+	-/7	150+/144	36/131	141/83	-	-/26
Viscosity Temp. Suscep.(60-135°C)[6]	3.92	3.94	3.43	2.58	2.96	3.38	3.22	3.33
Pen-Vis Number[7]	-1.4	-1.6	-0.1	2.5	1.2	0.3	0.5	-0.2
Pen Index[8]	-2.0	-2.0	-1.4	-1.6	-1.9	-1.6	-1.9	-1.6
Penetration Ratio[9]	28	32	39	40	34	32	31	37

[1] AASHTO T53.

[2] AASHTO T202.

[3] AASHTO T201.

[4] AASHTO T49.

[5] AASHTO T51, 5 cm/min.

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

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

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

[9] 100(Pen 39.2°F, 200 g, 60 s)/(Pen 77°F, 100 g, 5 s).

Table 3-9. Tensile strength of mixtures made using Texas asphalt and river gravel in Texas A & M study (Button et al., 1987).

Type Mixture	Tensile Strength, psi								
	0.02 in/min			0.2 in/min			2.0 in/min		
	25°C (77°F)	1°C (33°F)	-15°C (-26°F)	25°C (77°F)	1°C (33°F)	-15°C (-26°F)	25°C (77°F)	1°C (33°F)	-15°C (-26°F)
Control: AC-20	45	211	413	83	342	395	121	369	374
Control: AC-5	16	128	327	28	244	440	63	376	522
AC-5 + 15% Microfil-8	15	132	319	33	217	424	64	360	450
AC-5 + 5% Elvax 150	22	119	381	48	241	512	87	444	425
AC-5 + 5% Kraton D	27	136	404	54	300	472	112	428	502
AC-5 + 5% Latex	15	121	348	31	239	352	74	399	437
AC-5 + 5% Novophalt	28	167	393	58	329	444	119	436	387

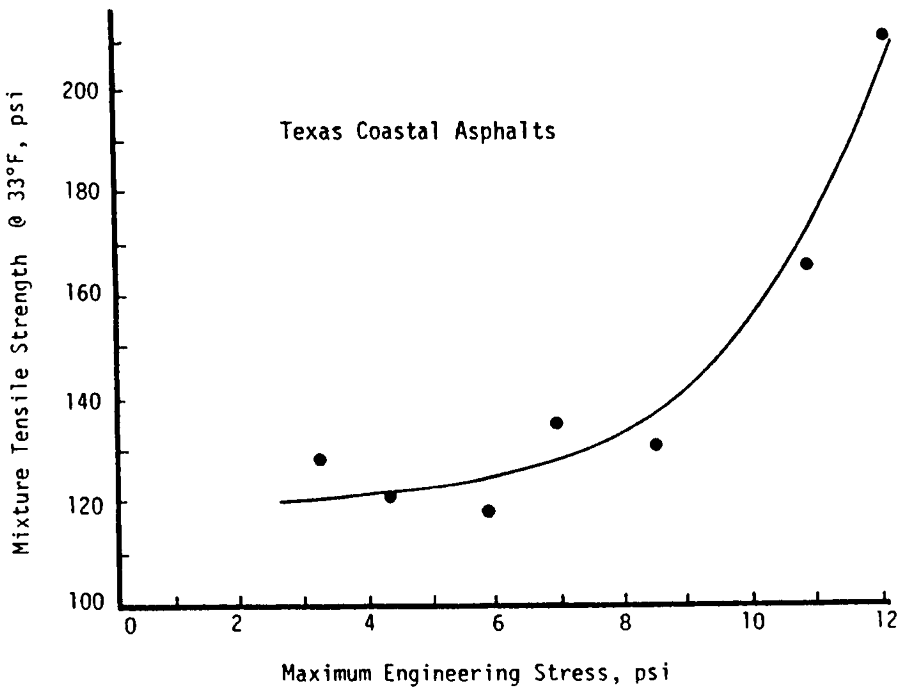


Figure 3-28. Mixture tensile strength as a function of maximum engineering stress. (Mixture tensile strength was measured at 33°F and 2 in./min. using indirect tension test. Force ductility data at 4°C after RTFOT were used.) (Button et al., 1987).

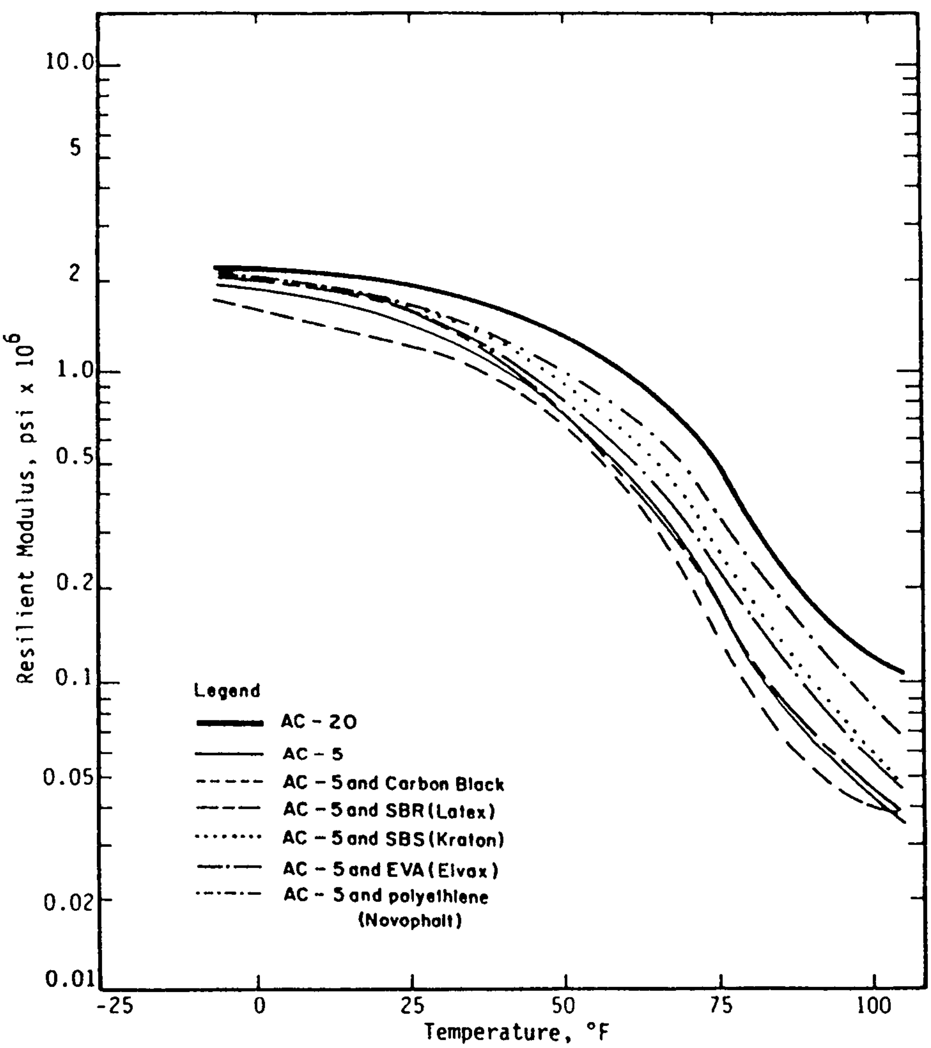


Figure 3-29. Resilient modulus as a function of temperature for river gravel mixtures containing Texas asphalts with and without additives in Texas A & M study (Button et al., 1987).

The modified accelerated Lottman moisture treatment procedure was used on mixtures containing both asphalts. It appeared that, generally, the additives had little effect on the moisture susceptibility of the mixtures used in this study.

Two approaches were used to evaluate the fatigue cracking potential of the mixes. The first approach is a controlled-stress flexural fatigue test, and the second approach is based on fracture mechanics. Fracture mechanics techniques are used to evaluate the energy required to propagate the crack through the material. The following results were noted:

1. At 20°C, each additive blend with AC-5 produced a mixture with statistically superior fatigue properties compared with the control mixture using AC-20 asphalt as the binder. This is illustrated in Figure 3-30. The mixtures containing EVA (Elvax), SBR (Latex), and SBS (Kraton) performed the same for all practical purposes and performed better than the mixtures containing either carbon black or polyethylene (Novophalt).

2. At 0°C, the modified AC-5 asphalt blend again performed better than the control. However, fatigue results on the mixtures containing polyethylene, SBS, SBR and EVA were not significantly different. Figure 3-31 illustrates these results.

From the fracture mechanics tests, the following results were noted. Table 3-10 summarizes the results of the controlled displacement tests.

1. At 1°C, all additive-soft asphalt blends demonstrated significantly superior resistance to crack propagation compared to the control mixtures, which were bound with a harder asphalt without an additive. The EVA AR-1000 blend gave the best results among the additives and San Joaquin Valley asphalts, while the latex AC-5 blend gave the best results among the blends of additives from Texas Coastal Asphalt. Apparently, synergistic interactions affect performance. Considering the performance of additives from both asphalt sources at 1°C, the SBS asphalt blend produced the most consistently superior results.

2. At 1°C, the additives blended with Texas Coastal AC-5 demonstrated superior performance in comparison to San Joaquin Valley AR-1000 blends. This can be partially explained by the higher penetrations of the AC-5 additive blend at 4°C. Note also that the Texas Coastal asphalt performed slightly better than did the San Joaquin Valley AR-4000 at 1°C (See Table 3-10).

3. At 25°C, the additives blends of the San Joaquin Valley AR-1000 asphalt generally outperformed the blends of additives in the Texas Coastal AC-5 asphalt. The samples fabricated with EVA, SBS, and latex blends with AR-1000 demonstrated multiple cracking or crack branching.

4. Mixtures fabricated with Carbon black asphalt blends generally demonstrated the poorest controlled displacement fatigue performance at 25°C.

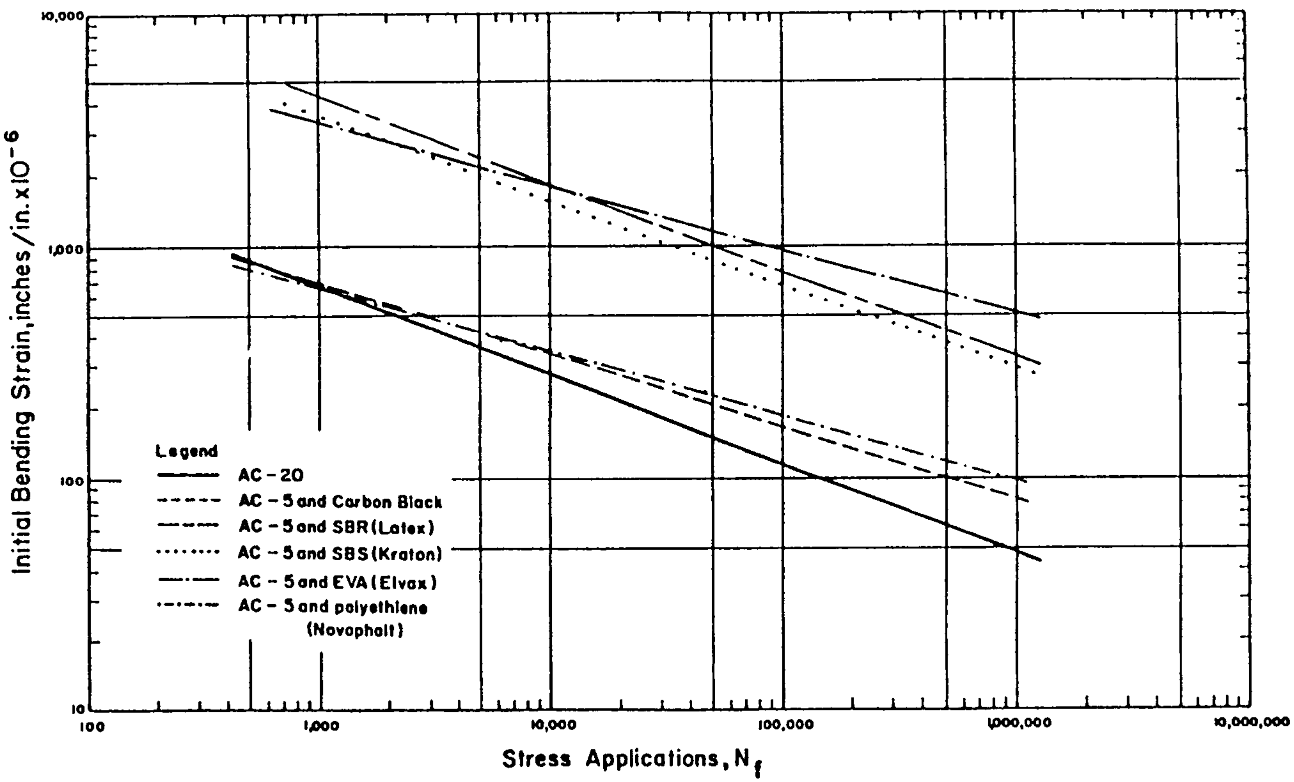


Figure 3-30. Controlled stress flexural beam fatigue results at 20°C in Texas A & M study (Button et al., 1987).

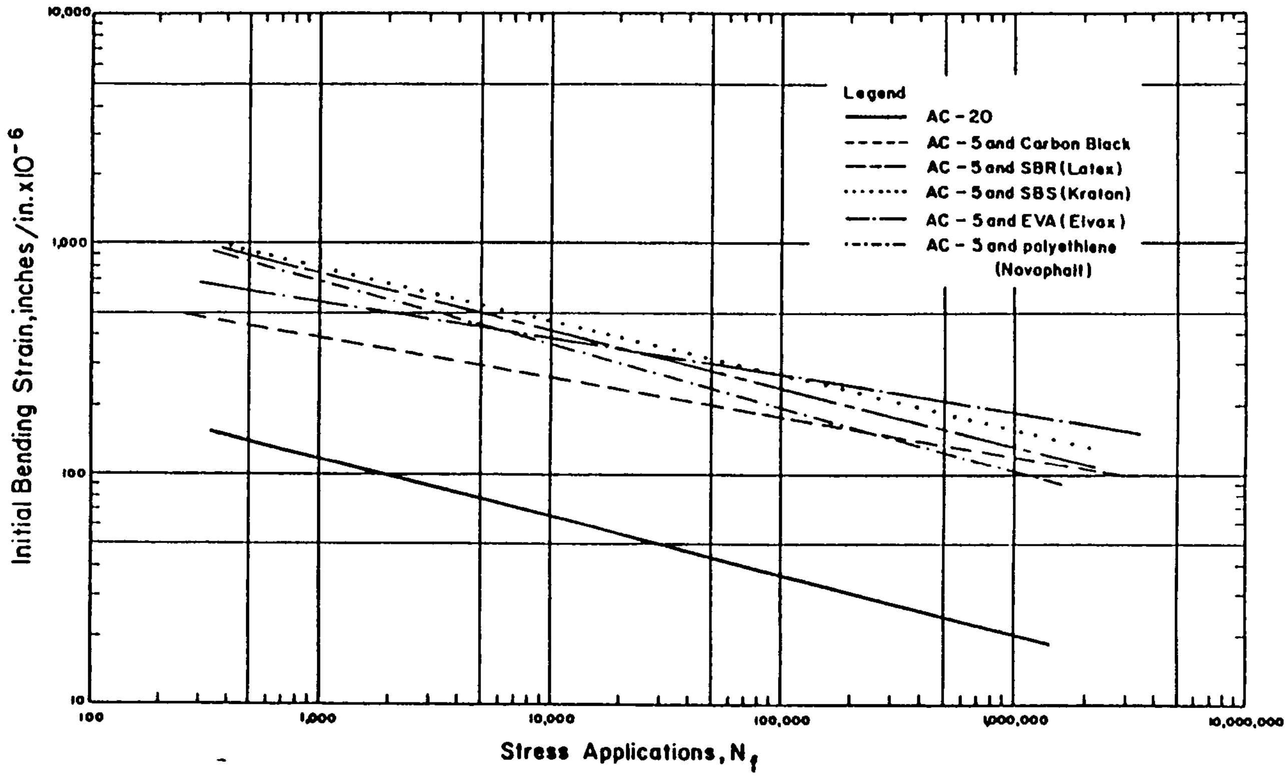


Figure 3-31. Controlled stress flexural beam fatigue results at 0°C in Texas A & M study (Button et al., 1987).

Table 3-10. Summary of controlled displacement fatigue results[a] in Texas A & M study (Button et al., 1987).

Base Asphalt	Mixture Type	1°C (33°F) Air Voids, Percent	1°C (33°F) No. Cycles to Failure	25°C (77°F) Air Voids, Percent	25°C (77°F) No. Cycles to Failure
Texas Coastal	AC-20	5.9	6	5.9	250
	AC-5 + Car. Blk.	7.0	590	6.7	530
	AC-5 + Elvax	5.9	390	6.0	7[b]
	AC-5 + Kraton	5.6	860	5.8	350
	AC-5 + Latex	6.0	1190	6.2	740
	AC-5 + Novophalt	6.0	1230[b]	6.5	190
San Joaquin Valley	AR-4000	6.3	1	6.0	110
	AR-1000 + Car. Blk.	7.1	250[b]	6.5	490
	AR-1000 + Elvax	6.9	740	6.5	>2000
	AR-1000 + Kraton	6.3	370	6.8	>2000
	AR-1000 + Latex	6.5	90	6.6	>2000
	AR-1000 + Novophalt	6.7	180	6.4	782

[a]Each value represents an average of at least two values.

[b]These values represent an average of three values.

For the creep permanent deformation testing, asphalt concrete cylinders 8 in. high by 4 in. in diameter were fabricated using the standard California kneading compactor for the direct compression testing program. Air voids in the cylinders were between 6 and 7 percent. Tests on two specimens each at temperatures of 40°, 70° and 100°F were performed. Permanent deformation properties were calculated from the incremental static loading and the creep compliance (defined as total strain at different times divided by applied stress) properties from the 1000-second response curve for each specimen. From the compliance test results (Figure 3-32), the following trends were observed:

1. Polyethylene in AC-5 exhibited compliance characteristics which were statistically the same as the AC-20 control. Although the resistance of the AC-5 to high temperature deformation is greatly improved by adding polyethylene, the low temperature compliance is also reduced, giving it essentially the same fracture susceptibility as the AC-20 control.

2. Blends of AC-5 with SBR, EVA, SBS and carbon black all responded with a higher compliance than the AC-20 control at the low temperature. The more compliant nature of these blends indicates mixtures which better resist low temperature cracking.

3. The compliances of the AC-5 with SBR or EVA at 100°F are significantly higher than those of the control mixture. From these data, one would expect excessive permanent deformation at high pavement service temperatures.

4. Generally, it may be stated that EVA, SBS, SBR and carbon black provide reduced temperature susceptibility. Such a response is expected of additives that reduce rutting potential at higher temperatures and maintain a compliant, fracture resistant nature at lower temperatures, and

5. Test results on modified San Joaquin Valley asphalt mixtures were substantially different from the Texas Coastal asphalts, indicating synergistic effects probably related to asphalt-additive compatibility.

The results of the permanent deformation tests are shown in Figure 3-33 and may be summarized:

1. Mixtures containing SBR showed the greatest permanent deformation relative to the AC-20 control and the other additives tested,

2. Polyethylene exhibited a greater resistance to permanent deformation at 40° and 70°F than any other mixtures including the AC-20 control. At 100°F, the carbon black blend yielded the least permanent deformation, followed closely by the polyethylene, and

3. Mixtures containing EVA and SBS showed permanent deformation responses similar to the AC-20 control. At 10,000 loading cycles, the order of resistance to permanent deformation is polyethylene, EVA, carbon black, SBS, AC-20 and SBR.

Conclusions

Button et al. (1987) derived the following conclusions from this study:

- For best results, a softer than usual asphalt should be used with an additive capable of lowering the temperature susceptibility of the binder. The soft asphalt provides flexibility and reduced cracking at the lower temperatures and the additive increases the viscosity at higher temperatures to reduce the potential for permanent deformation,

- Hveem stability of the mixtures was not significantly altered by the additives indicating an insensitivity to changes in rheological properties of the binder,

- The additives increased the Marshall stability of mixtures when added to AC-5 or AR-1000, but not up to that of mixtures containing the AC-20 or AR-4000 with no additives,

- At low temperatures, less than 0°C, the additives had little effect on the diametral resilient modulus of the mixtures. Resilient moduli of AC-5 or AR-1000 mixtures above 60°F were generally increased by the additives but not up to that of the AC-20 or AR-4000 mixtures without additives,

- Indirect tension test results show that, at the lower temperatures and higher loading rates, the additives increase mixture tensile strength over the other control mixtures,

- Strain or deformation at failure was generally increased by the additives. At higher temperatures and lower loading rates, additives did not appreciably affect the mixture tensile properties,

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

- Flexural fatigue responses of mixtures containing AC-5 plus additive at 20° and particularly at 0°C were superior to the control mixture (AC-20 with no additive),

- 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. At low temperatures all the additives in AC-5 except polyethylene produced equal or better creep compliance than the AC-20 control mixture, and

- Controlled displacement fatigue testing at 1°C demonstrated mixtures containing AC-5 plus an additive gave greater resistance to crack propagation than control mixes containing AC-20. The "dissolved" additives, EVA, SBR and SBS showed evidence of retarding crack propagation.

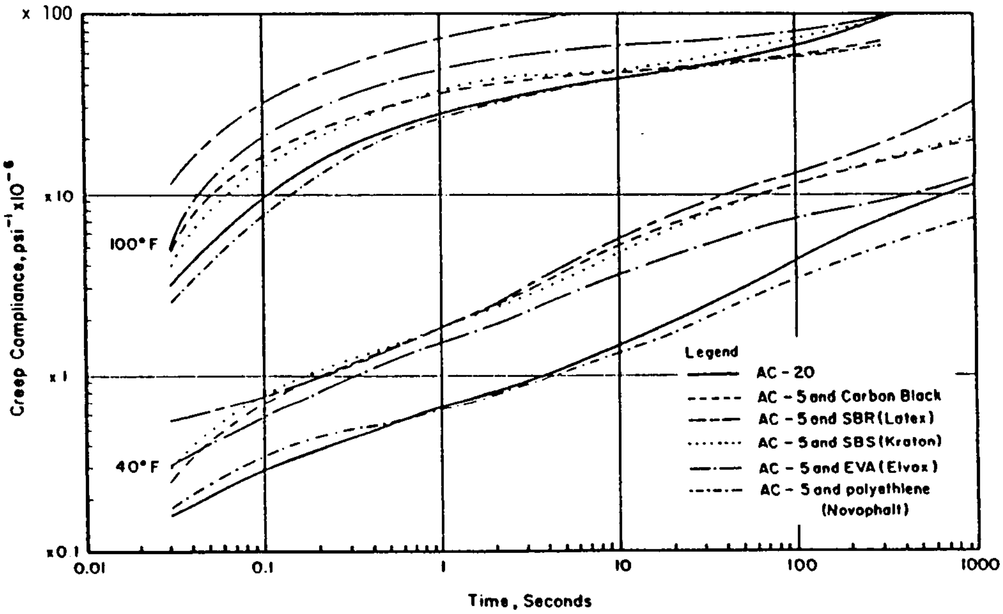


Figure 3-32. Creep compliance curves at 40°F and 100°F for mixtures containing Texas Coastal Asphalts in Texas A & M study (Button et al., 1987).

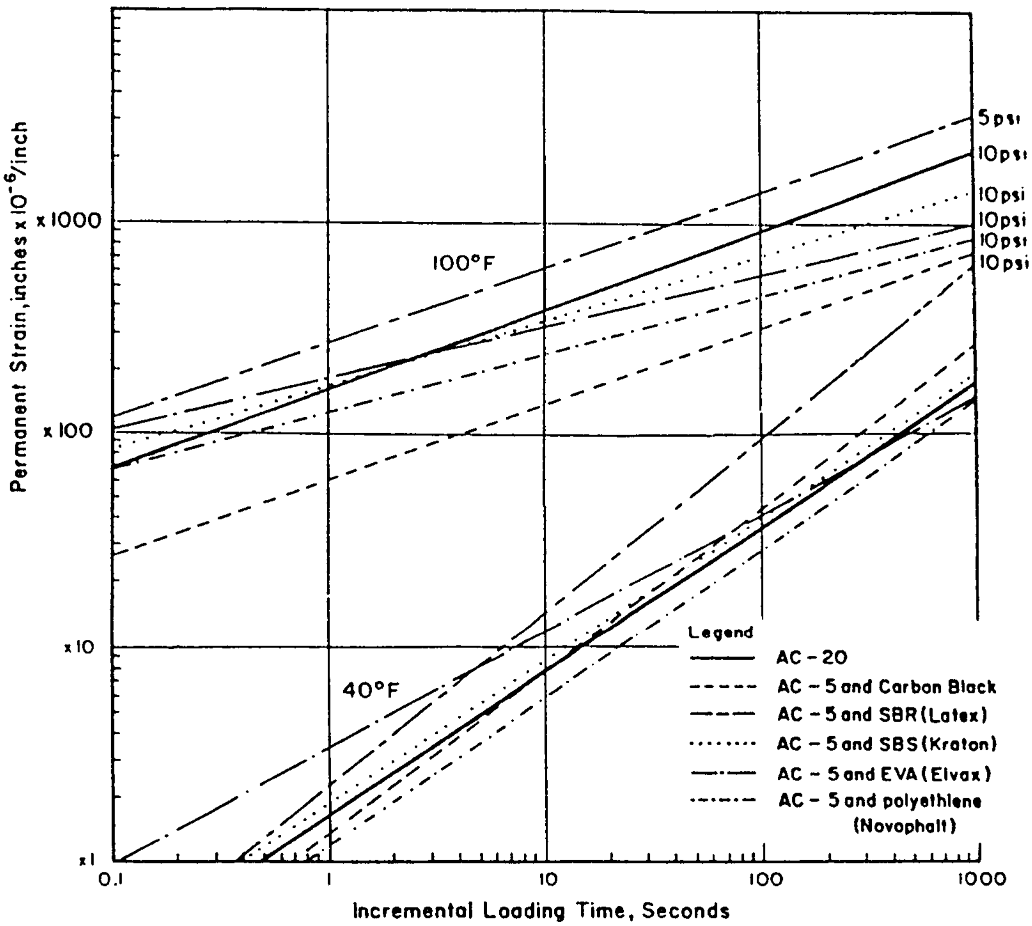


Figure 3-33. Permanent strain from incremental loading tests at 40° and 100°F for mixtures containing Texas asphalts in Texas A & M study (Button et al., 1987).

Finally, 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 panacea. To rank the additives according to relative capabilities is a difficult task, as sensitivity to the base asphalt plays a significant role.

3.4 Western Research Institute - Lime

The effects of lime on asphalt age hardening were evaluated Petersen et al., (1987). The study considers the effects of both lime type, concentration and asphalt composition. In addition to age hardening, the effects of lime on asphalt stiffness at higher temperatures and on asphalt low-temperature flow properties were also evaluated. High temperature stiffness and low temperature flow properties are important in controlling permanent pavement deformation and thermally induced cracking.

Four AC-10 asphalts were used in the study; two of the asphalts, California Coastal and Boscan (Venezuela), were produced from a single crude oil, while the other two asphalts were produced from North Slope-Maya and West Texas-Maya blends. High-calcium hydrated lime and dolomitic hydrated lime were supplied by the National Lime Association and used in the preparation of lime-treated asphalts. The lime was finer than the No. 200 mesh and was used without further treatment. In addition, a high-calcium limestone, pulverized to match the physical fineness of the hydrated lime was used in some comparative experiments. Mixtures containing 10%, 20% and 30% weight high-calcium lime and 20% weight dolomitic lime were prepared for tests. Two of the asphalts (Boscan and West Texas-Maya) were treated with pulverized limestone in the same manner as with hydrated lime.

The following five tests were performed on these mixtures:

1. Thin-film accelerated aging test (TFAAT),
2. Removal of hydrated lime from lime treated asphalts,
3. Infrared analyses,
4. Rheological measurements, and
5. Tensile elongation test.

The TFAAT test is a laboratory aging test that simulates the level of oxidative aging and volatile loss that typically occurs in about ten years of pavement service. It was developed by Western Research Institute and is a modification of the rolling microfilm test developed by Chevron Research. Chemical functionalities naturally present or formed during oxidative aging were quantitatively determined using a differential infrared spectrometric technique. Rheological data were obtained using a mechanical spectrometer. The tensile elongation test was used to determine the low temperature flow properties of untreated and lime-treated asphalts before and after aging. The stiffness modulus was calculated from the data obtained at 1% elongation. Maximum tensile stress during elongation or at break was also determined.

Results

The aging index is the viscosity of the asphalt after the TFAAT divided by the viscosity before aging. This index was used to quantify the effects of asphalt age hardening. The data in Table 3-11 shows that significant reductions occurred in the aging index of lime-treated asphalts. For example, high-calcium lime at 10% concentration reduced the aging index of the West Texas-Maya blend (most sensitive to age hardening) from 338 to 66 and for the North Slope-Maya blend (least sensitive asphalt) from 90 to 30. Figure 3-34 illustrates the same results graphically. Again, the two asphalts most sensitive to age hardening (the Boscan and West Texas-Maya blend) showed the greatest reduction in age hardening after lime treatment. Also significant is that most of the reduction to the aging index was realized between 0% and 10% lime concentration. In most cases, 20% lime seemed about optimum. Also, Table 3-12 shows incremental increases in the log viscosity of the Boscan asphalt from the lime filler effect, together with the incremental decreases in the log viscosity related to asphalt compositional changes. These incremental changes were obtained by comparing the viscosities of the asphalts before and after lime removal. Data for the unaged asphalt show incremental increases in viscosity with increasing lime concentration from only the lime filler effect. Data for the TFAAT aged asphalt show both incremental increases in viscosity from the lime filler effect and incremental decreases in the viscosity from compositional changes. The greatest incremental decrease from compositional change is between 0% and 10% lime, which is consistent with the data in Figure 3-34, with the maximum decrease observed at 20% lime.

Changes in the rheological properties at higher temperatures (60°C) of aged, lime-treated asphalts, which may be of potential importance to pavement performance is illustrated in Figure 3-35. All lime-treated, aged asphalts (except California Coastal) asphalt show lower complex dynamic shear moduli, G*, than their untreated aged counterparts. The complex modulus is a measure of asphalt stiffness and is determined under dynamic conditions, which may qualitatively relate to flexing under traffic loading--a cause of fatigue cracking in aged pavement.

Increased initial stiffness in newly constructed pavements from the use of lime-treated asphalt may be highly desirable in reducing early rutting, shoving, and other forms of permanent pavement deformation. The rheological data shows that lime treatment significantly increases stiffness of unaged asphalt (60°C) in the temperature range of permanent deformation a change that can be troublesome. In all cases, the complex dynamic shear modulus, G*, was increased by the addition of lime to asphalt. Corresponding increases in the complex, dynamic shear viscosity are also apparent from the data on unaged asphalts shown in Table 3-11. These results suggest that lime treatment should increase rutting resistance in all cases.

The log of the complex dynamic shear modulus of 60°C shown in Figure 3-36 for all unaged asphalts and for both lime types and all loading levels. Unlike the response to age hardening, which shows the maximum effect between 0% and 10% concentrations, the dynamic stiffness modulus of the asphalts generally show a regular increase with an increasing lime content. This result is expected because the major contributor to the increased stiffness is the lime filler effect. Figure 3-36 also shows that the Boscan asphalt

Table 3-11. Effectiveness of lime in reducing oxidative age hardening of asphalts in WRI study (Petersen et al., 1987).

Asphalt	Lime treatment Type	%	Dynamic viscosity (η^*), Pa·sec Unaged[1]	Aged[2]	Aging index[3]
Boscan	None	0	8.38×10^1	1.79×10^4	214
	High calcium	10	1.51×10^2	6.38×10^3	42
	High calcium	20	2.02×10^2	5.40×10^3	27
	High calcium	30	2.47×10^2	9.62×10^3	39
	Dolomitic	20	2.01×10^2	6.23×10^3	31
California Coastal	None	0	1.28×10^2	1.72×10^4	134
	High calcium	10	1.72×10^2	1.28×10^4	74
	High calcium	20	2.71×10^2	1.41×10^4	52
	High calcium	30	2.75×10^2	2.65×10^4	96
	Dolomitic	20	2.54×10^2	2.67×10^4	105
W. Texas-Maya blend	None	0	1.17×10^2	3.96×10^4	338
	High calcium	10	1.31×10^2	8.60×10^3	66
	High calcium	20	1.64×10^2	8.49×10^3	52
	High calcium	30	1.81×10^2	9.79×10^3	54
	Dolomitic	20	1.53×10^2	8.68×10^3	57
N. Slope-Maya blend	None	0	8.76×10^1	7.91×10^3	90
	High calcium	10	1.22×10^2	3.64×10^3	30
	High calcium	20	1.52×10^2	4.95×10^3	33
	High calcium	30	1.91×10^2	4.55×10^3	24
	Dolomitic	20	1.41×10^2	3.70×10^3	26

[1] Rheological data at 60 C and 15.85 rad/sec
[2] Rheological data at 60 C and 0.126 rad/sec
[3] Ratio of viscosity aged/viscosity unaged

Table 3-12. **Relative effects on viscosity changes of Boscan asphalt resulting from lime filler effect and lime-altered changes in asphalt composition for WRI study (Petersen et al., 1987).**

Lime Treatment		Dynamic viscosity[1] (η^*), Pa·sec		Incremental changes in log η^*		
Type	%	Lime in	Lime removed	From lime filler effect[2]	From asphalt compositional changes[3]	Sum[4]
			Unaged			
None	0	$8.38x10^1$	$8.38x10^1$	0.00	0.00	0.00
High calcium	10	$1.51x10^2$	$7.76x10^1$	0.29	-0.03	0.26
High calcium	20	$2.02x10^2$	$8.32x10^1$	0.39	0.00	0.39
High calcium	30	$2.47x10^2$	$8.32x10^1$	0.47	0.00	0.47
Dolomitic	20	$2.01x10^2$	$8.71x10^1$	0.36	0.02	0.38
			TFAAT-aged			
None	0	$1.79x10^4$	$1.79x10^4$	0.00	0.00	0.00
High calcium	10	$6.38x10^3$	$3.16x10^3$	0.30	-0.75	-0.45
High calcium	20	$5.40x10^3$	$2.45x10^3$	0.34	-0.86	-0.52
High calcium	30	$9.62x10^3$	$2.57x10^3$	0.57	-0.84	-0.27
Dolomitic	20	$6.23x10^3$	$2.75x10^3$	0.35	-0.81	-0.46

[1] Dynamic viscosity at 60 C on unaged asphalts reported at 15.85 rad/sec, on TFAAT-aged asphalts at 0.126 rad/sec
[2] Difference between values for lime in and lime removed
[3] Difference between values for untreated and lime removed
[4] Difference between values for untreated and lime in

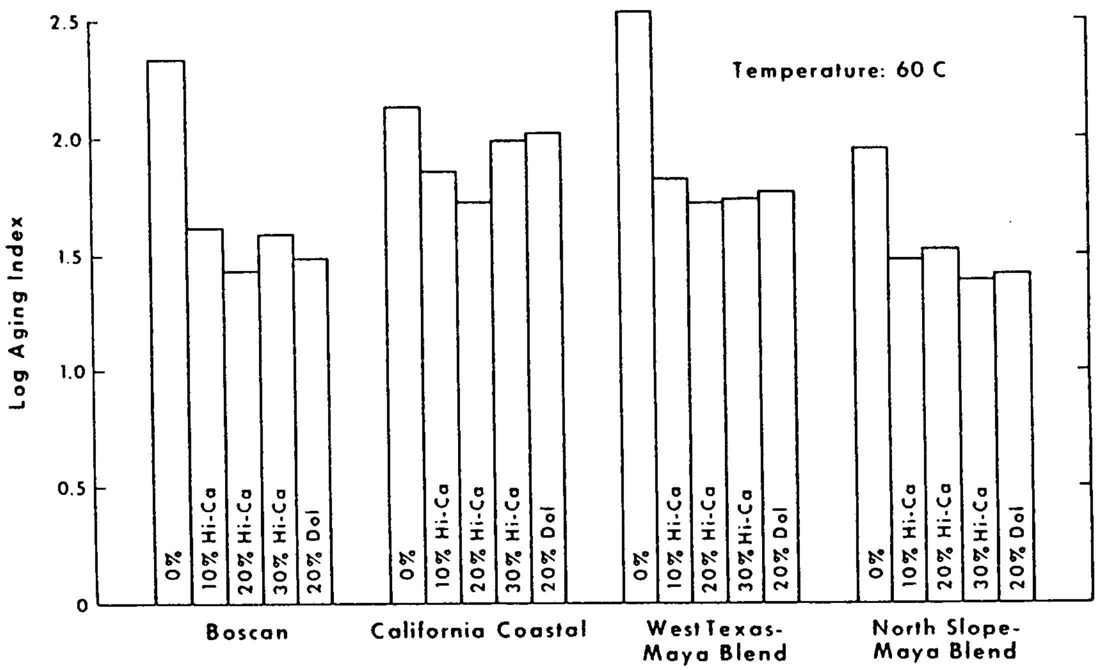


Figure 3-34. Effect of lime on asphalt aging index for WRI study (Petersen et al., 1987).

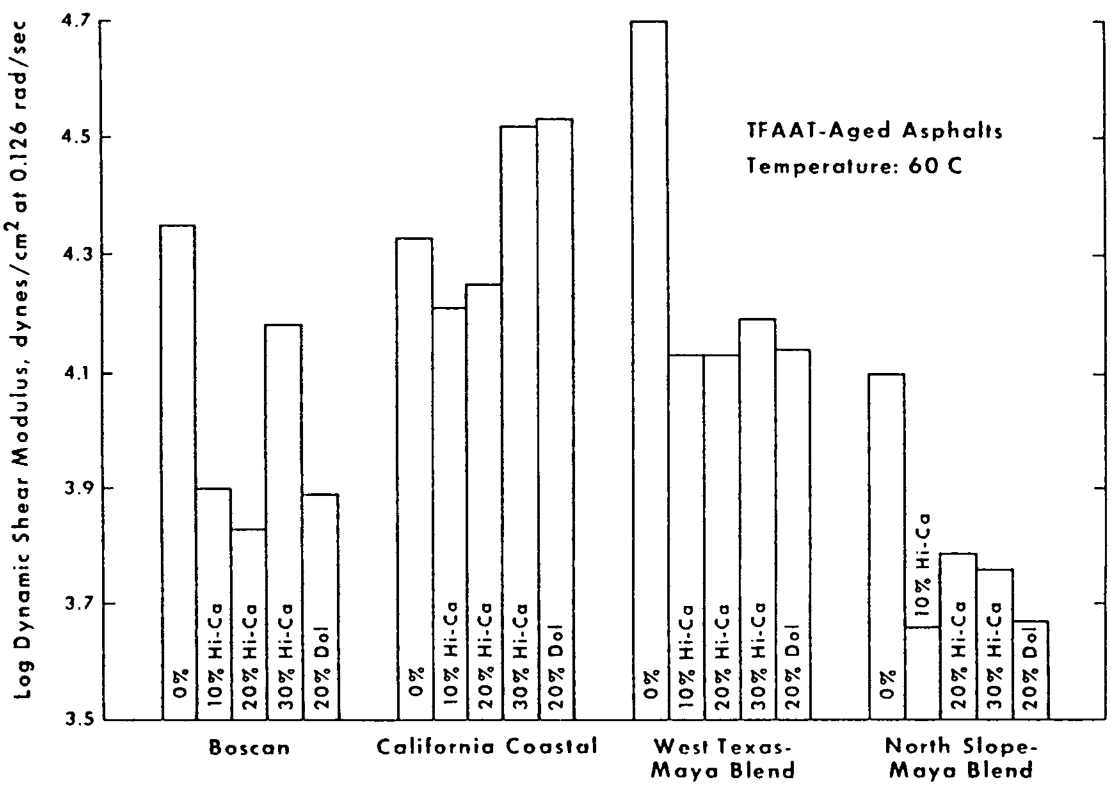


Figure 3-35. Effect of lime on complex dynamic shear modulus of aged asphalts in WRI study (Petersen et al., 1987).

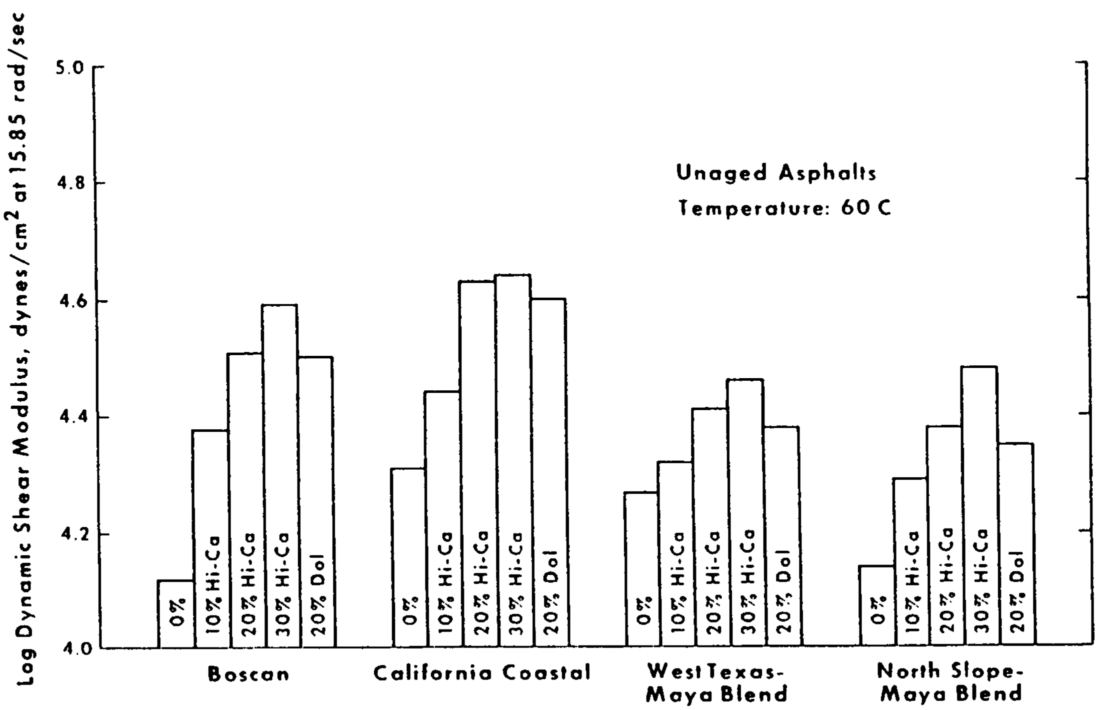


Figure 3-36. Effect of lime on complex dynamic shear modulus of unaged asphalts in WRI study (Petersen et al., 1987).

was most responsive to modulus increase on lime treatment and that the West Texas-Maya blend was least responsive. Thus, as with the effects on the aging characteristics, the magnitude of the stiffening effect of lime on asphalts was highly source or composition dependent. Finally, comparison of the data in Figure 3-36 for both 20% high calcium and 20% dolomitic lime showed that the effects of the two lime types on asphalt stiffness was similar.

The low temperature ductile flow properties of the untreated and lime-treated asphalts are shown in Figure 3-37. The data were obtained in a tensile-elongation test, in which the asphalt was elongated intentionally at a very slow rate, near and at temperatures near its brittle fracture region. It is proposed that under these conditions, the data can be related to the tensile stresses responsible for the low-temperature, thermally induced transverse cracking in pavements. The higher tensile stress at break of the lime-treated asphalts suggests stronger pavements, but the increased elongation at break is more significant with regard to low temperature pavement cracking. Elongation-to-break is more important because as pavements shrink from cooling, the asphalt in the pavement is put in tension. If the asphalt is unable to elongate through ductile flow and the tensile strength of the asphalt is exceeded, it breaks in brittle fracture. On the other hand, if the tensile stress can be released by ductile flow, no cracking will result. The West Texas-Maya blend, although improved significantly by lime addition, was less improved as measured by increased low temperature ductile flow than the other three asphalts. This asphalt also had the highest aging index in the untreated state (Table 3-11).

Finally, the effects of a high-calcium limestone on selected asphalt properties were compared with the corresponding effects of high-calcium hydrated lime. The aging index results in Figure 3-38 show that although high calcium lime significantly reduced the aging index, the pulverized limestone caused an increase in the aging index. Thus, the beneficial effects of lime in reducing age hardening are not seen for the pulverized limestone. The low temperature flow property data in Figure 3-38 show that although the pulverized limestone increases the tensile strength at break over the untreated asphalts, the elongation to break was greatly reduced by the limestone. This is in contrast to effects of hydrated lime on these properties of aged asphalts. Hydrated lime produced both an increase in tensile strength and an increase in elongation to break. The low temperature stiffness moduli for the aged, limestone-treated asphalts were considerably higher than the corresponding moduli for either the untreated or lime-treated, aged asphalts. These low temperature data suggest that resistance to transverse cracking aged pavements would not be improved by the pulverized limestone, as it most likely would be by hydrated lime, but might actually be adversely affected.

Conclusions

This study showed that (Peterson et al., 1987) lime treatment of asphalts reduced asphalt age hardening, increased the high-temperature stiffness of unaged asphalts, reduced the stiffness of aged asphalts at higher temperatures, and increased the asphalt tensile-elongation at low temperatures. These effects should benefit asphalt pavements by increasing asphalt durability, by reducing rutting, shoving and other forms of permanent

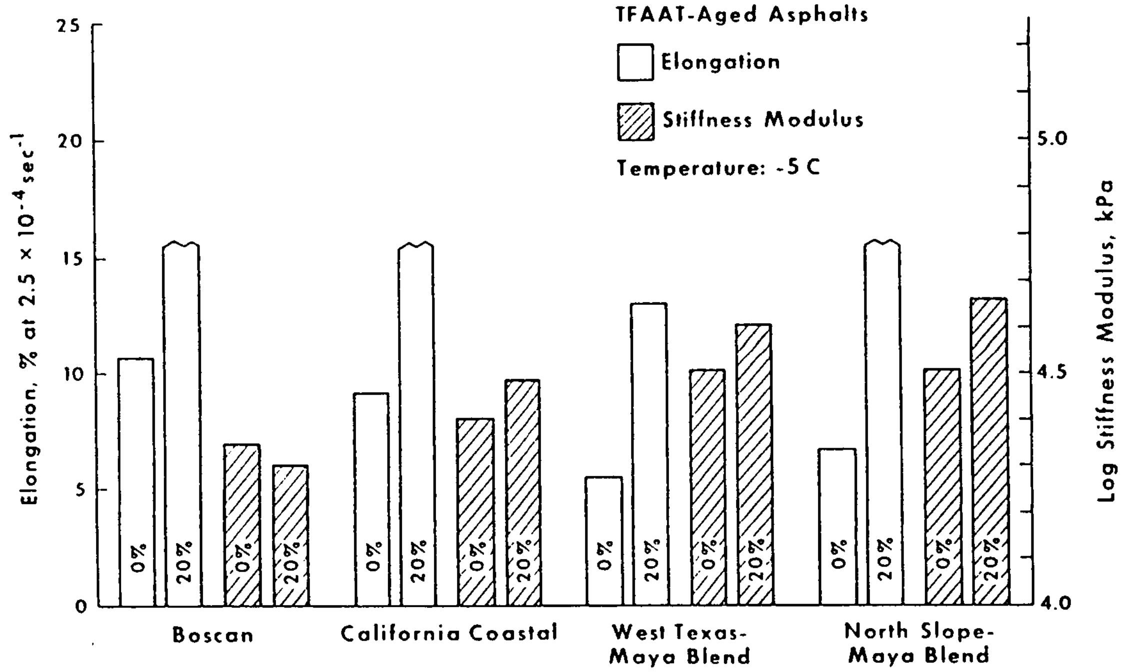


Figure 3-37. Effect of high calcium lime on low-temperature tensile-elongation and stiffness modulus of aged asphalts in WRI study (Petersen et al., 1987).

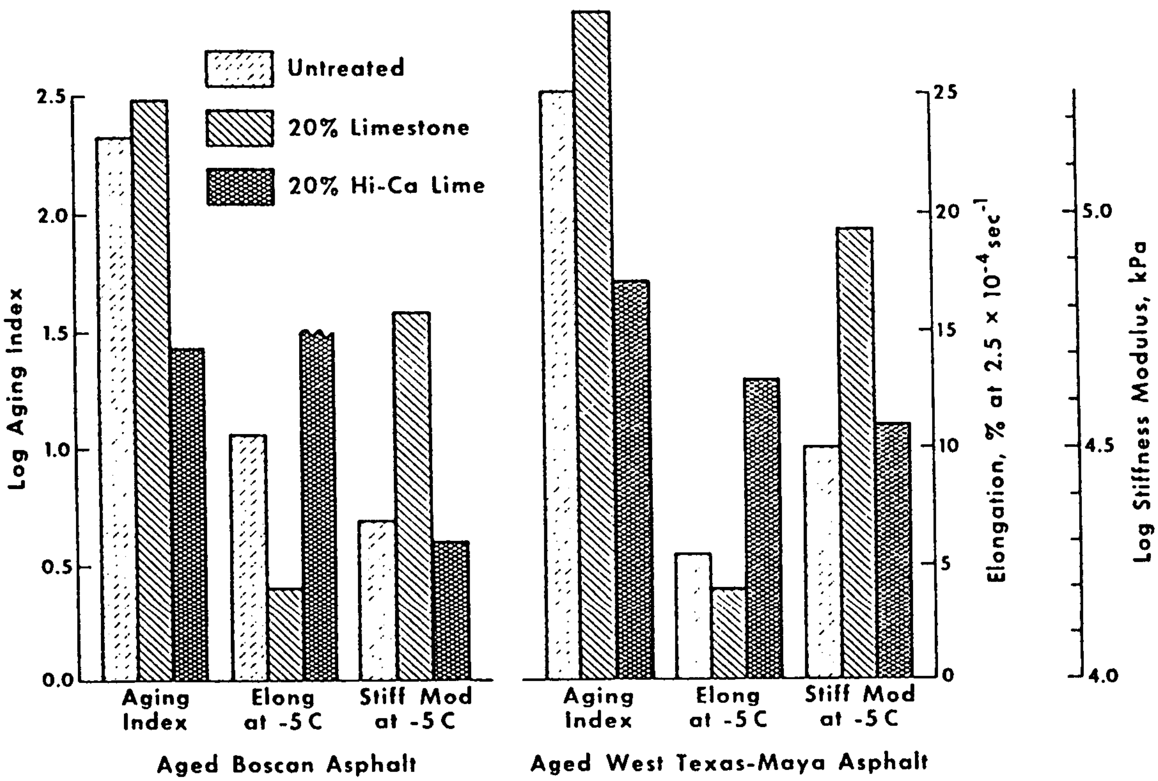


Figure 3-38. Comparison of the effects of pulverized limestone and high-calcium lime on selected properties of aged asphalts in WRI study (Petersen et al., 1987).

pavement deformation, by improving fatigue resistance in aged pavements, and by improving pavement resistance to low temperature transverse cracking. Although the relative response to lime treatment varies as a function of asphalt source, all sources studied benefited significantly by lime treatment. The net result of the combined effects of lime treatment should result in longer-lasting pavements, with improved performance during the life of the pavement. The beneficial effects of hydrated lime on the aging characteristics and on the low temperature flow properties were not found for pulverized limestone.

3.5 Bradford, England - Various

Description

In this report, Salter and Rafati-Afshar (1987) used the indirect tensile test to investigate the effects of ethylene vinyl acetate, polypropylene fiber, rubber, and sulfur as additives on the fatigue and strength characteristics of bituminous mixtures. The objectives of the research program were 1) to use the indirect tensile test to obtain relationships between a number of load repetitions to failure and the resilient characteristics of bituminous mixtures and 2) to investigate the effects of incorporating additives in bituminous mixtures.

The mixes in the experimental program used hard limestone aggregate with continuous grading, and the binder was a 50 pen bitumen supplied by Croda Hydrocarbons, Ltd. The ethylene vinyl acetate used was Evatane 18-150 (Imperial Chemical Industries) supplied in the form of pellets approximately 2 to 3 mm in diameter. Polypropylene fiber (Don Fibres Ltd.) were supplied in the form of 500 denier fiber staples at a length of 50 mm and a tensile strength of 4.5-5.5 g per denier. The melting point of the fiber was given as 167°C with some softening to be expected at 150°C; 10 g of the fiber were incorporated into each of the fiber-modified specimens. Rubber additives (Rubber Latex, Ltd.) were supplied in the form of Pulvatex rubber powder, an unvulcanized rubber powder manufactured from concentrated natural rubber latex with 60% natural rubber and 40 percent by mass of a separator to keep the rubber particles from agglomerating. The rubber/bitumen ratio was 5/95 by mass. In the sulfur-modified specimens, the heated sulfur powder is mixed with bitumen before addition to the aggregate. The compaction of specimens was carried out using the standard Marshall molds with 50 blows of the standard Marshall compaction hammer. Samples were cured for three weeks before testing. Tables 3-13 and 3-14 summarize the binder contents used in the test specimens and the Marshall test data, respectively.

The indirect tensile testing of the specimens was carried out by applying a repeated compressive load that acted parallel to and along the vertical diametral plane. The load was transmitted to the specimen through a 20 mm wide curved loading strip of the same curvature as the specimen being tested. Testing was carried out in a controlled-temperature laboratory, with temperatures in the 22°-24°C range for all tests. Loading frequency was at a rate of 1 cps with a load duration of 0.4 sec. and a rest period of 0.6 sec.

Table 3-13. Test specimens used in experimental program at Bradford, England (Salter & Rafati-Afshar, 1987).

Bituminous Specimens			Bituminous Specimens Containing Fiber			Bituminous Specimens Containing Evatane (copolymer)			Bitumen Specimens Containing Sulfur (20 percent of bitumen weight)			Bituminous Specimens Containing Sulfur (50 percent of bitumen weight)			Rubber-Modified Bituminous Specimens (5 percent of bitumen weight)		
Bitumen Content (%)	No. of Speci-mens	Stress Level (kPa)	Bitumen Content (%)	No. of Speci-mens	Stress Level (kPa)	Binder Content (%)	No. of Speci-mens	Stress Level (kPa)	Binder Content (%)	No. of Speci-mens	Stress Level (kPa)	Binder Content (%)	No. of Speci-mens	Stress Level (kPa)	Binder Content (%)	No. of Speci-mens	Stress Level (kPa)
5	3	283	5	3	283	5.0	4	283	5.5	4	283	5.0	4	283	4.8	4	283
	3	496		3	496		4	496		4	496		4	496		4	496
	3	662		3	662		4	662		4	662		4	662		4	662
6	3	283	6	3	283	6.0	4	283	6.0	4	283	6.0	4	283	5.2	4	283
	3	496		3	496		4	496		4	496		4	496		4	496
	3	662		3	662		4	662		4	662		4	662		4	662
7	3	283	7	3	283	6.5	4	283	6.5	4	283	6.5	4	283	5.6	4	283
	3	496		3	496		4	496		4	496		4	496		4	496
	3	662		3	662		4	662		4	662		4	662		4	662
8	3	283				7.0	4	283	7.0	4	283	7.0	4	283	6.0	4	283
	3	496					4	496		4	496		4	496		4	496
	3	662					4	662		4	662		4	662		4	662
						8.0	4	283									
							4	496									
							4	662									

Table 3-14. Marshall test data: Density and Marshall quotient at optimum binder content for bituminous and modified bituminous mixes at Bradford, England (Salter & Rafati-Afshar, 1987).

Type of Specimen	Optimum Binder Content (%)	Stability (kN)	Flow (mm)	Density (g/mL)	Percentage of Voids	QM[a] (kN/mm)
Bituminous	6	10.2	5.0	2.40	3.8	2.04
Bituminous containing fiber	6	9.7	5.6	2.26	9.1	1.72
Evatane-modified bituminous (5:95 EVA:bitumen ratio)	6.5	13.6	4.3	2.42	2.4	3.21
Sulfur S20–modified bituminous (20:80 sulfur:bitumen ratio)	5.5	10.0	3.1	2.42	1.3	3.22
Sulfur S50–modified bituminous (50:50 sulfur:bitumen ratio)	6.5	18.4	3.0	2.42	3.7	6.13
Rubber-modified bituminous (5:95 rubber:bitumen ratio)	5.2	9.5	3.0	2.40	3.6	3.22

[a]QM = Marshall quotient.

Results

Table 3-15 summarizes the fatigue test results for all the specimen types obtained by indirect tensile testing. It can be seen that the polymer modified binders offered improved fatigue life compared with mixes that contain the other evaluated additives or unmodified mixes. A linear relationship was found to exist between the logarithm of applied stress and the logarithm of fatigue life expressed as:

$$N_f = K_2 \left(\frac{1}{\sigma}\right)^{n_2}$$

where: N_f = fatigue life,
σ = applied stress, and
K_2,n_2 = constants that depend on mixture properties and temperatures.

The experimental values for the constants K_2 and n_2 for all types of specimens are given in Table 3-16. The K_2-values range between 4.5×10^{10} and 4.03×10^{13}. The n_2-values for all fatigue test results vary between 2.62 and 3.75. Comparison with fatigue results obtained by different test methods and from different researchers indicate that the differences are mainly in the K_2-values. It is thought to be due to differences in loading and in environmental testing conditions, as well as the composition of the specimens.

Fatigue life relationships were also expressed in terms of initial strain for controlled stress tests and repeated strain for controlled strain tests. The relationships between fatigue life and initial strain for bituminous specimens in additive-modified bituminous specimens are as shown in the general equation below:

$$N_f = K_1 \left(\frac{1}{\varepsilon_{mix}}\right)^{n_1}$$

where: N_f = fatigue life,
ε_{mix} = initial strain in the mixture, and
K_1,n_1 = constants.A summary of the values of K_1 and n_1 is given in Table 3-17.

The values of K_1 range from 7.93×10^{-7} to 3.10×10^{-4} for bituminous and modified bituminous mixtures. Values of n_1 ranged from 2.61 to 3.74. Again, it was difficult to compare the values of K_1 and n_1 with those obtained from previous flexural axial load and dynamic indirect tensile tests because the mixtures, bitumen contents, testing temperatures and testing procedures are different for each type of test.

Conclusions

The researchers (Salter and Rafati-Afshar, 1987) had the following conclusions to make:

- An analysis of the indirect tensile test fatigue results indicated low coefficients of variation that exhibited a general decrease with an increase in binder content. Tests carried out on different binder contents showed that several mixes had an optimum

Table 3-15. Mean, standard deviation, and coefficient of variation of fatigue life for bituminous specimens at Bradford, England (Salter & Rafati-Afshar, 1987).

Type of Specimen	Bitumen Content (%)	Stress Level (kPa)	No. of Specimens	Mean Cycles to Failure	Standard Deviation	CV (%)
Bituminous	5	283	3	18,193	12237	67.0
	5	496	3	4,740	779	16.4
	5	662	3	2,050	292	14.2
	6	283	3	22,322	10113	45.3
	6	496	3	2,863	742	25.9
	6	662	3	1,247	54	4.3
	7	283	3	27,956	9503	34.0
	7	496	3	4,790	714	14.9
	7	662	3	1,380	212	15.4
	8	283	3	20,519	3229	16.0
	8	496	3	2,930	453	15.5
	8	662	3	1,130	61	5.4
Bituminous containing fiber	5	283	3	26,826	1861	6.9
	5	496	3	3,500	320	9.1
	5	662	3	880	162	18.4
	6	283	3	17,200	2040	11.9
	6	496	3	4,200	1407	33.5
	6	662	3	1,536	418	27.3
	7	283	3	21,941	1846	8.4
	7	496	3	3,105	740	23.8
	7	662	3	1,288	423	32.8
Bituminous containing 5% EVA	5	283	4	39,100	3158	8.0
	5	496	4	5,195	1388	26.7
	5	662	4	2,560	723	28.2
	6	283	4	46,030	3210	7.0
	6	496	4	5,790	1151	20.0
	6	662	4	2,750	633	23.0
	6.5	283	4	41,300	2786	6.7
	6.5	496	4	6,100	729	12.0
	6.5	662	4	2,020	396	19.5
	7	283	4	35,731	2439	6.8
	7	496	4	5,160	386	7.5
	7	662	4	1,855	253	13.6
	8	283	4	27,700	3558	12.8
	8	496	4	3,570	620	17.4
	8	662	4	1,497	105	7.0
Bituminous containing 20% sulfur/80% bitumen	5.5	283	4	18,457	1770	9.6
	5.5	496	4	3,354	453	13.5
	5.5	662	4	1,101	195	17.7
	6	283	4	20,596	2938	14.3
	6	496	4	3,707	463	12.5
	6	662	4	1,215	232	19.1
	6.5	283	4	17,819	1935	10.8
	6.5	496	4	3,492	416	11.9
	6.5	662	4	1,441	188	13.0
	7	283	4	15,976	3535	22.1
	7	496	4	3,315	662	20.0
	7	662	4	1,297	451	34.7
Bituminous containing 50% sulfur/50% bitumen	5.5	283	4	24,025	2731	11.4
	5.5	496	4	4,393	1041	23.7
	5.5	662	4	1,375	294	21.4
	6.0	283	4	26,277	2895	11.0
	6.0	496	4	4,900	1631	33.3
	6.0	662	4	1,645	420	25.5
	6.5	283	4	36,956	2575	7.0
	6.5	496	4	5,290	1347	25.5
	6.5	662	4	2,390	960	40.0
	7.0	283	4	31,500	3286	10.4
	7.0	496	4	4,012	603	15.0
	7.0	662	4	1,850	263	14.0
Bituminous containing rubber	4.8	283	4	20,533	2856	14.0
	4.8	496	4	3,578	577	16.0
	4.8	662	4	1,350	209	15.5
	5.2	283	4	23,474	3437	14.6
	5.2	496	4	3,232	477	14.7
	5.2	662	4	1,422	168	11.8
	5.6	283	4	26,540	2777	10.5
	5.6	496	4	4,202	737	17.5
	5.6	662	4	1,600	366	22.6
	6.0	283	4	22,181	3613	16.3
	6.0	496	4	3,820	1285	33.6
	6.0	662	4	1,410	320	22.7

Table 3-16. Experimental values of K_2, K_2', and n_2 at Bradford, England (Salter & Rafati-Afshar, 1987).

Type of Specimen	Bitumen Content (%)	No. of Specimens	K_2	K_2'	n_2	Correlation Coefficient
Bituminous	5.0	9	4.50×10^{10}	1.69×10^{12}	2.62	0.98
	6.0	9	3.54×10^{12}	3.73×10^{14}	3.34	0.99
	7.0	9	9.97×10^{12}	1.24×10^{15}	3.48	0.99
	8.0	9	3.65×10^{12}	3.74×10^{14}	3.36	0.99
Fiber-modified bituminous	5.0	9	4.03×10^{13}	7.00×10^{15}	3.74	0.99
	6.0	9	1.22×10^{12}	9.73×10^{13}	3.15	0.98
	7.0	9	4.54×10^{12}	4.69×10^{14}	3.39	0.98
Polymer-modified bituminous	5.0	12	4.11×10^{12}	3.90×10^{14}	3.28	0.99
	6.0	12	8.05×10^{12}	8.56×10^{14}	3.37	0.99
	6.5	12	2.00×10^{13}	2.66×10^{15}	3.54	0.99
	7.0	12	1.18×10^{13}	1.45×10^{15}	3.47	0.99
	8.0	12	7.38×10^{12}	8.53×10^{14}	3.44	0.99
Sulfur-modified bituminous	5.5	12	2.11×10^{12}	1.97×10^{14}	3.28	0.99
(20/80 sulfur/bitumen ratio)	6.0	12	2.47×10^{12}	2.42×10^{14}	3.29	0.99
	6.5	12	3.05×10^{11}	1.79×10^{13}	2.95	0.99
	7.0	12	2.93×10^{11}	1.82×10^{13}	2.96	0.98
Sulfur-modified bituminous	5.5	12	3.63×10^{12}	3.61×10^{14}	3.33	0.98
(50/50 sulfur/bitumen ratio)	6.0	12	1.81×10^{13}	2.40×10^{14}	3.60	0.98
	6.5	12	3.75×10^{12}	3.60×10^{14}	3.27	0.99
	7.0	12	6.78×10^{12}	7.65×10^{14}	3.40	0.99
Rubber-modified bituminous	4.8	12	1.43×10^{12}	1.10×10^{14}	3.20	0.99
(5/95 rubber/bitumen ratio)	5.2	12	2.82×10^{12}	3.14×10^{14}	3.30	0.99
	5.6	12	3.46×10^{12}	3.52×10^{14}	3.31	0.99
	6.0	12	1.93×10^{12}	1.69×10^{14}	3.24	0.98

Table 3-17. Experimental values of K_1 and n_1 at Bradford, England (Salter & Rafati-Afshar, 1987).

Type of Specimen	Bitumen Content (%)	No. of Specimens	K_1	n_1	Correlation Coefficient
Bituminous	5.0	9	6.35×10^{-4}	2.61	0.98
	6.0	9	2.76×10^{-6}	3.34	0.99
	7.0	9	7.93×10^{-7}	3.48	0.99
	8.0	9	1.23×10^{-5}	3.36	0.99
Fiber-modified bituminous	5.0	9	1.59×10^{-6}	3.74	0.99
	6.0	9	6.18×10^{-5}	3.14	0.98
	7.0	9	4.41×10^{-5}	3.38	0.98
	7.0	9	4.41×10^{-5}	3.38	0.98
Polymer-modified bituminous	5.0	12	6.32×10^{-6}	3.26	0.99
	6.0	12	2.67×10^{-6}	3.37	0.99
	6.5	12	1.14×10^{-6}	3.55	0.99
	7.0	12	5.33×10^{-6}	3.48	0.99
	8.0	12	2.44×10^{-6}	3.44	0.99
Sulfur-modified bituminous	5.5	12	3.13×10^{-5}	3.28	0.99
(20/80 sulfur/bitumen ratio)	6.0	12	2.29×10^{-5}	3.30	0.99
	6.5	12	3.10×10^{-4}	2.94	0.99
	7.0	12	4.71×10^{-4}	2.95	0.98
Sulfur-modified bituminous	5.5	12	2.28×10^{-6}	3.32	0.98
(50/50 sulfur/bitumen ratio)	6.0	12	2.87×10^{-6}	3.26	0.98
	6.5	12	3.46×10^{-6}	3.27	0.99
	7.0	12	3.62×10^{-6}	3.41	0.99
Rubber-modified bituminous	4.8	12	5.51×10^{-6}	3.20	0.99
(5/95 rubber/bitumen ratio)	5.2	12	2.33×10^{-6}	3.32	0.99
	5.6	12	3.40×10^{-6}	3.32	0.99
	6.0	12	9.70×10^{-6}	3.22	0.98

binder content for maximum fatigue life. These binder contents were within ±1% of the optimum derived from the Marshall test,

- The addition of 20% sulfur to the binder did not increase Marshall stability compared with that of a normal bituminous specimen, but did decrease the flow which resulted in an increased Marshall quotient,

- Fiber specimens produced disappointing test values because of low densities caused by mixing and compacting difficulties,

- Mixtures with rubber modified binders also had low stability values but decreased flow values. The two most successful mixes were those with ethylene vinyl acetate binders and those with binder modified by 50% sulfur, and

- The use of ethylene vinyl acetate as a binder additive (at optimum binder content) produced the highest fatigue life improvement followed by the use of a 50% sulfur additive. Rubber-bitumen mixes and 20% sulfur also gave an improvement in fatigue properties in these tests at some stress levels. Difficulty was experienced in the preparation of fiber reinforced specimens. This resulted in low densities, high void contents, and shorter fatigue lives.

3.6 Illinois - Polymers

Description

This paper presents the results of laboratory testing to characterize performance differences among five different polymer blends and three unmodified asphalts (Carpenter & VanDam, 1987). The base asphalt is an AC-5 that was used in all polymer blends, remaining unmodified asphalts are an AC-10 and an AC-20. The polymers used were various amounts and types of Kraton®, a proprietary polymer (Shell Development Co.).

The asphalts prepared for use in this study were made from an AC-5 modified with the following polymers: 3% & 6% of Kraton D-1101, 3% Kraton G-1650, an experimental polymer, and a mix with 2.85% D-1116 and 1.14% D-1107. Table 3-18 summarizes the properties of the asphalt cements forming these polymer combinations. Table 3-19 summarizes the optimum asphalt content values as well as voids used in this study. The aggregate was a crushed limestone blended to the dense gradation required by the Illinois Department of Transportation for new interstate overlay mixes. The California kneading compactor was used to compact all cylinders for testing.

Four types of tests were used to evaluate the specimens: (1) stiffness, (2) tensile strength, (3) permanent deformation, and (4) thermal coefficient of expansion. Stiffness determinations were performed using the diametral resilient modulus device, with a 0.1 pulse load applied along the vertical diameter of the sample. Testing was performed at 40°F, 72°F, and 100°F. The tensile strength of the compacted asphalt concrete specimens was typically determined by the indirect tensile test or the Brazilian split test. The load is applied at a constant rate and increased until failure occurs. Indirect tensile strength, stiffness, tensile strain, compressive strain, Poisson's ratio, and vertical deformation at failure can be calculated. Testing was performed at 72°F at a rate of deformation of 2 in./min. Subsequent testing was conducted at 40°, 20°, 0°, and -20°F at a deformation rate of 0.05 in./min. to provide an indication of the low temperature performance of the mixes.

The permanent deformation or rutting resistance of the mixture was evaluated using cylindrical samples. Loads were applied to the vertical axis of the cylinder and total deformation of the load is recorded by an non-contacting sensor. The test equipment can perform a standard creep compliance test and a continual repeated load test. The procedure used in this study was the FHWA incremental static procedure that defines the ALPHA and GNU parameters for VESYS. The determination of the coefficient of expansion or contraction is an integral part of the analysis of the thermal behavior of mixes. Asphalt concrete bars were placed in an environmental chamber for twenty four hours at each temperature, and the length of the bar was recorded after the 24-hour temperature cycle. Temperature levels investigated included 72°F, 40°F, 20°F, 0°F, and -20°F. Readings were taken during both the cooling and the heating cycles for comparison.

Results

Figure 3-39 summarizes the results for the diametral resilient modulus test, where the plots of stiffness vs. temperature are shown. The influence of the polymers on the stiffness of the mixes is readily apparent, and the following comparisons between them can be made: The polymer modified mixes are stiffer than the base AC-5 until the temperature drops below approximately 20°F. Below this temperature, the polymer mixes are softer than the AC-5. The modified mixes show stiffnesses intermediate between those of AC-20 and the AC-5 at elevated temperatures. Treatment 2 produced stiffness values similar to those of the AC-20 at 72°F. Extrapolating the data below 40°F indicates that the blends will remain more flexible than the AC-10 and AC-20, and below 20°F, polymer blends are more flexible than the base asphalt.

At 72°F, the polymer modifiers increased the tensile strength of the mixes above the levels provided by the base AC-5 asphalts cement but not above that of an AC-10 asphalt cement. The tensile strain at failure for the polymer treatments was generally greater than for the AC-10 and the AC-20 mixtures. Increased strain at failure indicates that more strain energy is required to fail the specimen of that mixture, which is sometimes interpreted as a "tougher" mixture. The improvement at 72°F, however, has been more in terms of increased performance. Table 3-20 summarizes stiffness and tensile strength data for the Marshall compaction samples.

The results of the tensile strength and strains from the low temperature tests were slightly different. Typically, the softer grade of asphalt cement provided greater resistance to low temperature cracking by lowering the temperature at which failure occurs as well as increasing the tensile strength at failure. The polymer modified mixes demonstrate distinct differences from the untreated asphalt mixes. At approximately 10°F, all mixes possess the strength of the AC-20 mixtures. Below 10°F, even at very low temperatures, all polymer

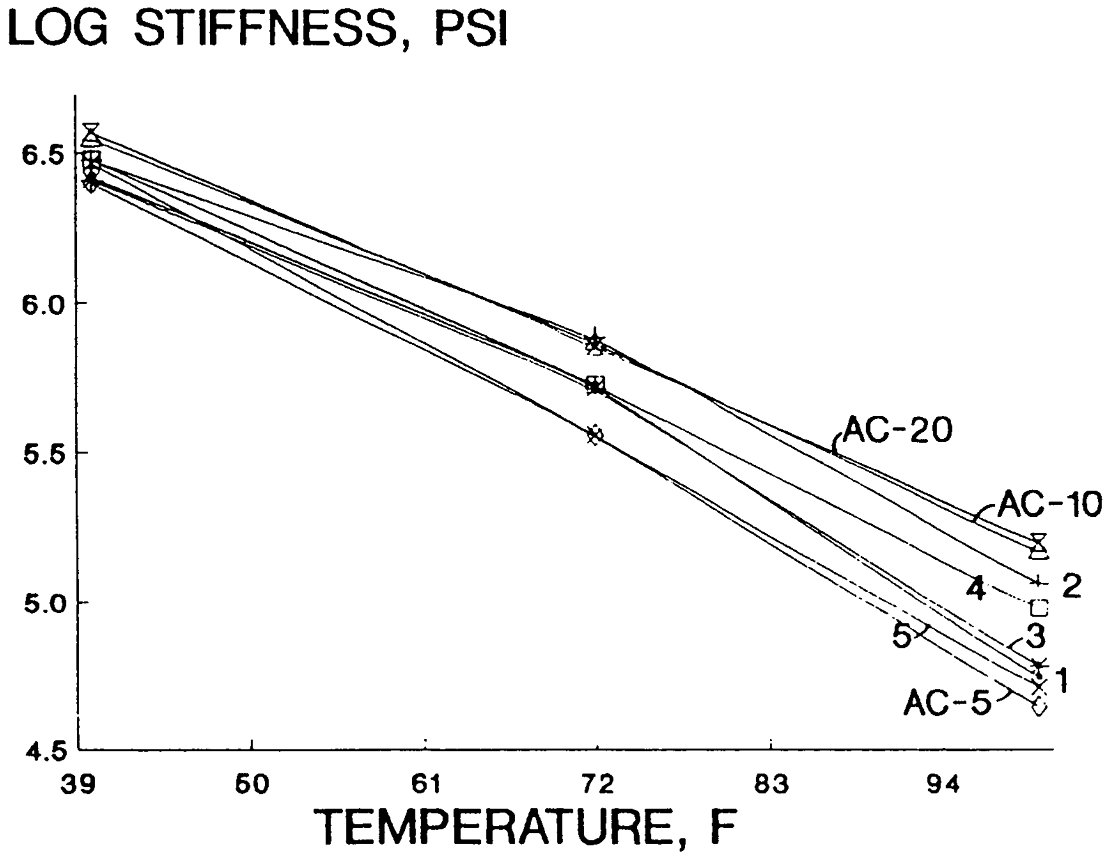


Figure 3-39. Stiffness vs. temperature for treated and untreated samples in Illinois study (Carpenter & VanDam, 1987).

Table 3-18. Properties of asphalt cements evaluated in Illinois study (Carpenter & VanDam, 1987).

Property	Modified Exxon AC-5 Base Asphalt with Treatment					Unmodified		
	1[a]	2[b]	3[c]	4[d]	5[e]	Exxon AC-5 with Treatment 6	Shell AC-10 with Treatment 7	Shell AC-20 with Treatment 8
Penetration at 25°C (dmm)	100	78	104	103	97	128	84	59
Viscosity at 80°C (poises)	560	–	200	282	628	78	131	207
Viscosity at 135°C (cPs)	570	1675	525	660	905	250	365	480
Ductility, 4°C, 5 cm/min (cm)	98	91	23	34	104	31	3	0
Softening point, ring and ball (°F)	121	193	118	120	130	112	118	123
Penetration index	+0.5	+6.8	+0.2	+0.5	+1.0	−0.9	−0.6	−0.7
Pen-vis no.	+0.2	+1.0	+0.2	+0.5	+1.0	−0.9	−0.6	−0.5
Toughness (in./lb)	85	171	85	177	144	17	42	73
Tenacity (in./lb)	67	141	20	147	126	10	11	12

[a]3 percent Kraton® D-1101.
[b]6 percent Kraton® D-1101.
[c]3 percent Kraton® G-1650.
[d]Experimental polymer.
[e]2.86 percent Kraton® D-116 with 1.14 percent Kraton® D-1107, 4 percent polymer content.

Table 3-19. Mix design properties of samples tested in Illinois study (Carpenter & VanDam, 1987).

Treatment	Asphalt Content (%)	G_{mm}[a]	G_{mb}[b]	Air Voids (%)
1	5.75	2.457	2.384	2.97
2	5.75	2.457	2.388	2.77
3	6.5	2.431	2.407	1.02
4	6.2	2.442	2.399	2.09
5	6.25	2.440	2.399	1.68
6	6.0	2.448	2.379	2.80
7	6.25	2.450	2.392	2.37
8	6.5	2.432	2.400	1.32

[a]G_{mm} = maximum theoretical density or specific gravity of an asphalt mixture.
[b]G_{mb} = density or bulk specific gravity of an asphalt mixture.

mixes maintain a higher strength than does the AC-20 mixture. It is thought that the peak strengths of several of the polymer blends may not have been reached because of temperature equipment control limitations that precluded going below -20°F. Figures 3-40 and 3-41 summarize the effects of the different treatments on indirect tensile strength and strain as a function of temperature for the untreated and polymer modified blends. The strain and failure in normal asphalt mixes rapidly decrease to a minimum value, even at moderate temperature level, and remain at this level to extremely low temperatures. The polymer modified blends, however, show a significantly different relationship.

The tensile strain at failure for these remain much higher than those for normal asphalt cements. Most significantly, at -20°F, the failure strain in the polymer mixes is two or three times greater than those of normal asphalt cements. This corresponds to the lower resilient modulus stiffness values indicated for these mixes at low temperatures. From this data, it would appear that Mixture 2 provides the best strain at failure and that Mixtures 1 and 3 performed quite well. It is evident that the polymer significantly modified the low temperature performance of the mixes. This modification is not possible with normal asphalts, even if they are of very different grades.

Figures 3-42 and 3-43 show the accumulation of rutting (@ 72°F & 100°F, respectively) for the untreated and polymer-modified asphalts sampled. Rutting is given as a relative unit for comparison only. At 100°F, the untreated asphalt samples show a dramatic increase in the potential for rutting as compared to the samples @ 72°F, as expected. The polymer-treated asphalts also show an increase in rutting potential, but the increase is not nearly as dramatic as it is for the untreated samples.

Asphalt 4, in particular, did not show any changes in its potential for rutting when compared with the 72°F curves, which demonstrates a very stable temperature influence. Treatments 1 and 3 showed the largest increase in rutting potential, but they still performed better than the AC-10 and nearly as well as the AC-20 at similar asphalt contents.

The coefficients of thermal contraction are given in Table 3-21 for each of the temperature ranges examined. These coefficients are typical for any dense graded mixture and did not appear to be affected by the asphalt grade used or the type of polymer treatment. The polymer-treated mixtures did show a difference from the untreated asphalt cements in that the coefficients did not show the same linear relationship with temperature. The polymer-treated mixes exhibited a non-linear relationship in the 40°F temperature range. Although this does not cause any significant difference in performance, it may be indicative of the polymer's influence.

In addition, samples were run through the Lottman vacuum saturation freeze-thaw procedure to induce stripping. However, the combinations of virgin asphalt and limestone aggregate used in this study are apparently not moisture sensitive. Therefore, any potential of the modified blends for improving resistance to stripping cannot be investigated with these mixtures. Nonetheless, it is just as important to note that the modified asphalt cements did not increase the potential for moisture sensitivity in these mixes.

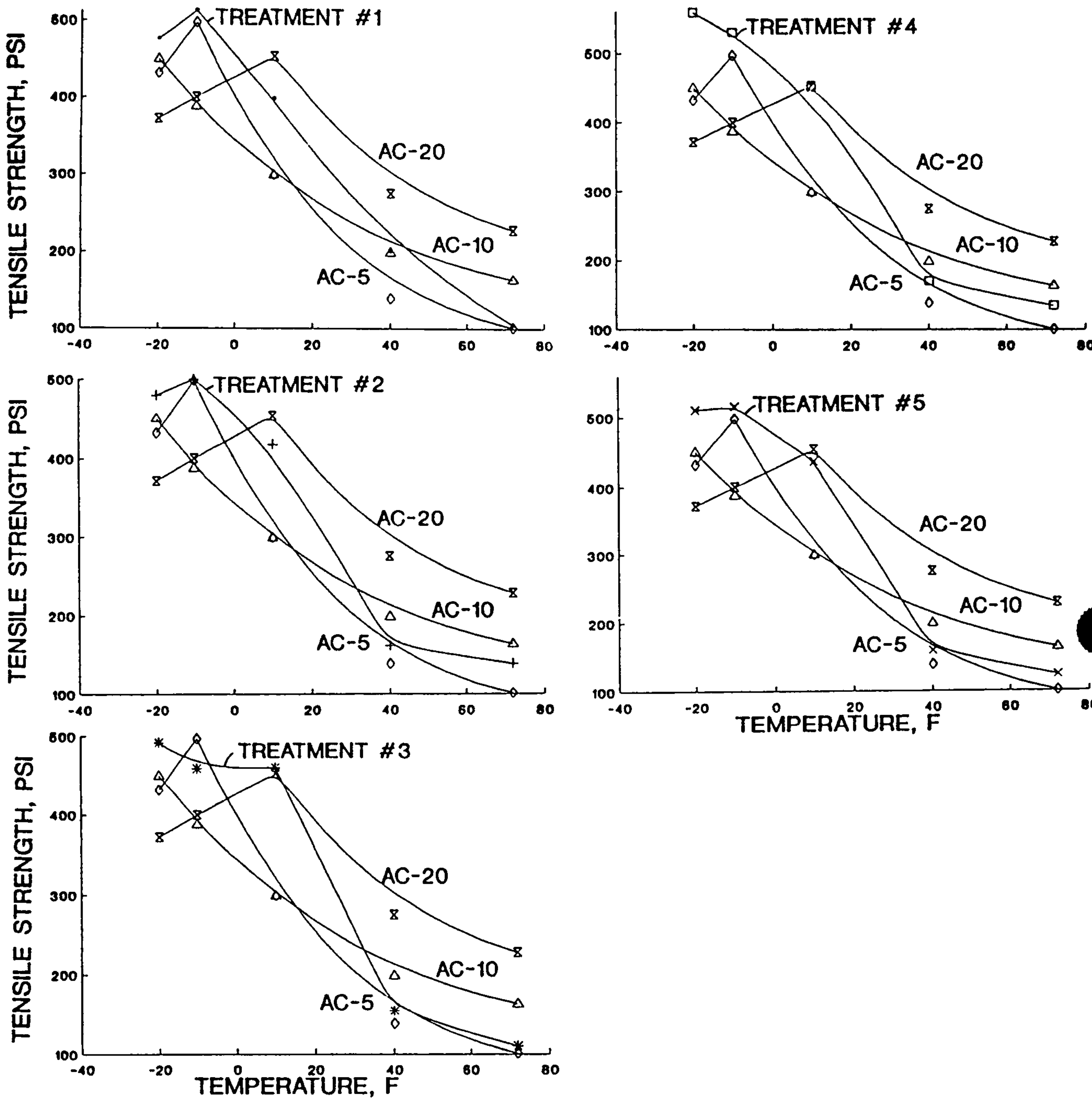


Figure 3-40. Indirect tensile strength as a function of temperature in Illinois study (Carpenter & VanDam, 1987).

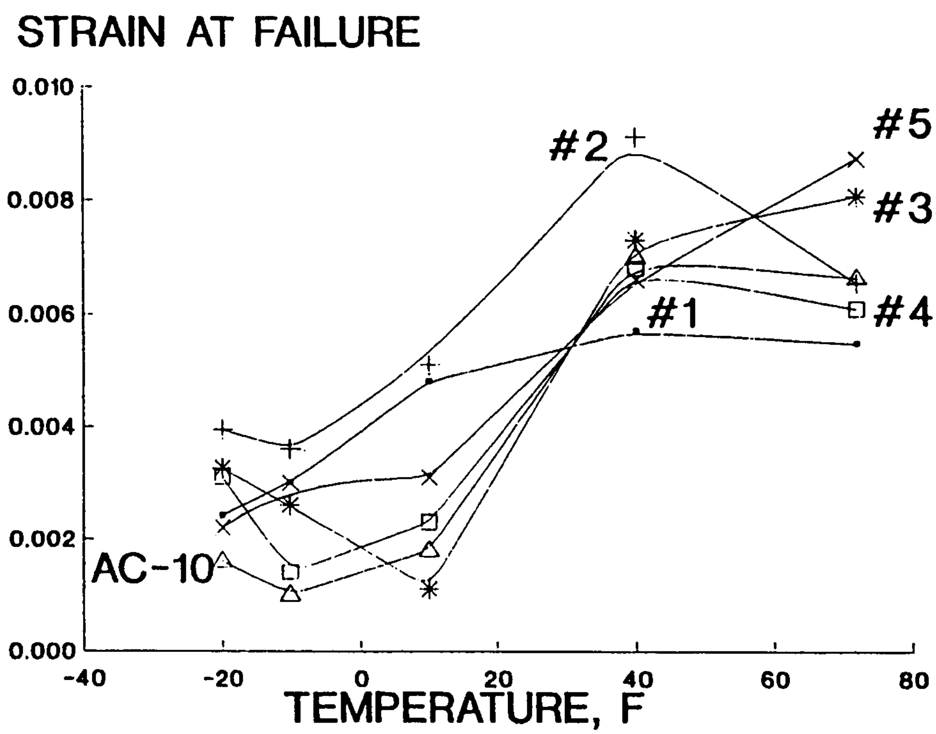


Figure 3-41. Indirect tensile strain as a function of temperature in Illinois study (Carpenter & VanDam, 1987).

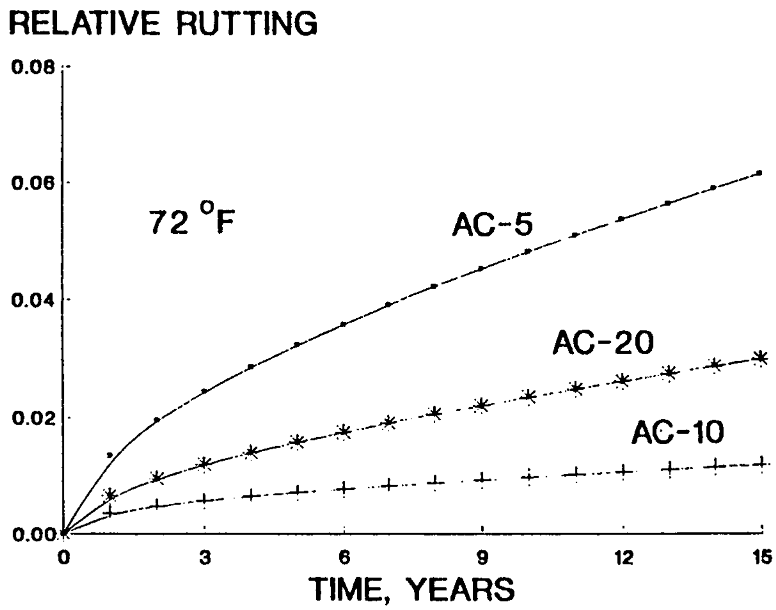


Figure 3-42a. Development of rutting in untreated samples at 72°F (Carpenter & VanDam, 1987).

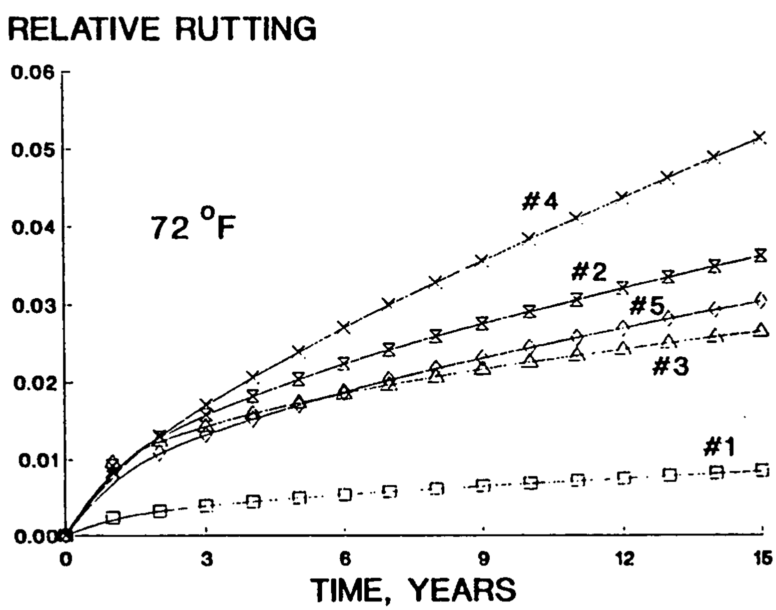


Figure 3-42b. Development of rutting in treated samples at 72°F (Carpenter & VanDam, 1987).

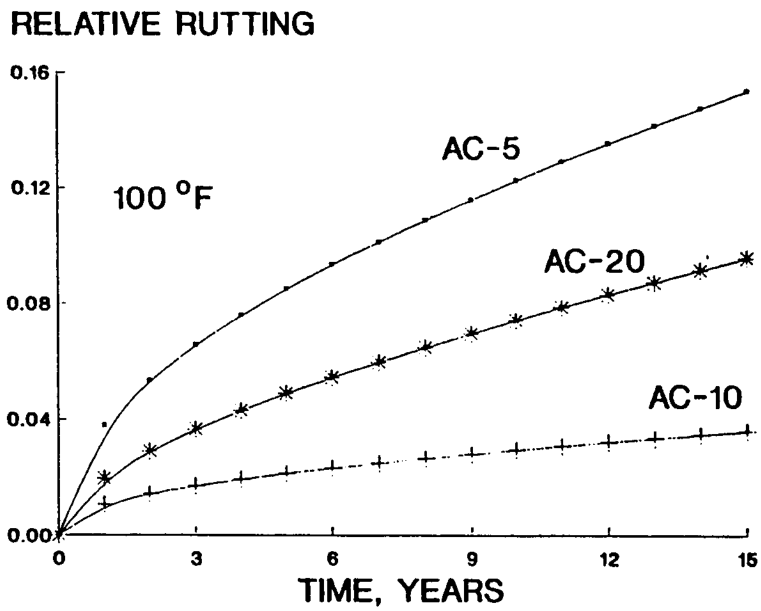


Figure 3-43a. Development of rutting in untreated samples at 100°F (Carpenter & VanDam, 1987).

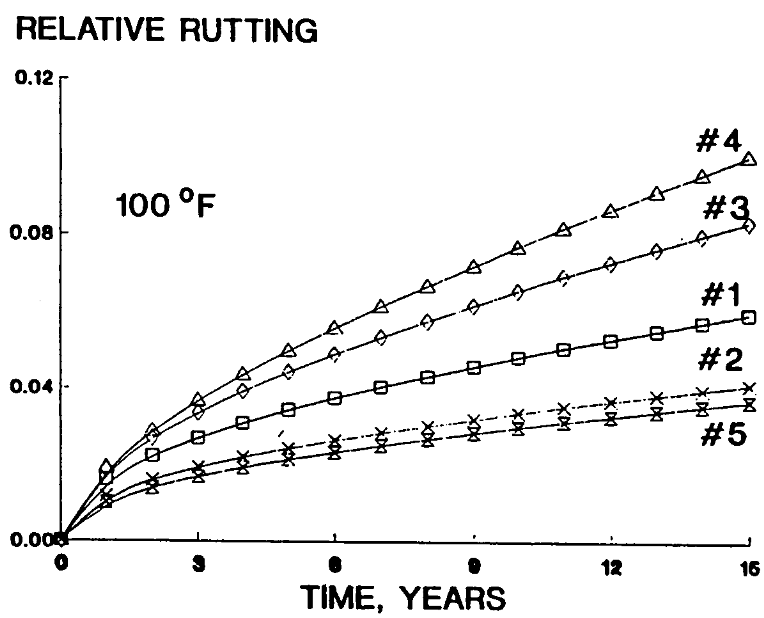


Figure 3-43b. Development of rutting in treated samples at 100°F (Carpenter & VanDam, 1987).

Table 3-20. Stiffness and tensile strength data for Marshall compacted samples in Illinois study (Carpenter & VanDam, 1987).

Treatment	Compaction Level					
	50			75		
	AC %	S_{it}	Stiffness	AC %	Sit	Stiffness
1	5.75	110.6	554600	5.75	130	470200
	5.75	105.2	517800	5.75	132.9	647000
2	5.25	128.5	821250	5.25	148.8	700850
	5.75	134.8	785950	5.75	121	546950
	5.75	143.2	759950	5.75	124.7	662150
	6.25	136.1	693400	6.25	138	651550
3	6.00	124	560000	5.25	130.3	752100
	6.00	98.1	484400	5.75	137.9	754050
	6.50	124	511350	5.75	133.3	801500
	6.50	119.6	592700	6.50	124	757750
4	5.75	147.6	702300	5.25	163.6	870100
	5.75	140.5	539800	5.25	146.7	875150
	6.25	129.6	527300			
5	5.75	120.2	484400	5.00	145	754350
	6.25	132.3	535200	5.50	121.3	500900
	6.25	115.5	458200	5.50	134.4	607000
	6.75	118.9	510850	6.00	133.6	555350
6	5.5	92.2	267150	5.50	88.4	467600
	6.00	93.2	338700	6.00	101.8	369000
	6.00	96.9	390150	6.00	88.9	359350
	6.50	84.6	270100	6.50	94.8	281550
7	5.75	162.2	729850	5.00	155.8	773100
	6.25	170.8	743100	5.50	151.9	732100
	6.25	156.4	738900	5.50	151.9	603300
	6.75	143.1	723300	6.00	166.6	817550
8	6.50	202.8	743000	4.75	221.8	1083350
	6.50	226.2	692050	5.25	230.4	873700
	7.00	185	699150	5.25	226.7	1288500
				5.75	225.9	977050

Conclusions

The study presented herein (Carpenter & VanDam, 1987) was designed to illustrate differences in performance of several polymer blends. The laboratory test data developed clearly illustrates that polymer modification of an asphalt cement produces a mixture that is quite different from the normal asphalt cement. Beginning with a base AC-5 asphalt, performance can be enhanced to a level expected of an AC-10, and in some instances to a level of that found in an AC-20. At temperatures around 72°F, the properties of the polymer-modified asphalts is better than the untreated asphalt cements of similar consistency (i.e., penetration at 77°F). Performance of these asphalts is most similar to that of an AC-10. At low temperatures, the performance of the modified asphalts approaches and surpasses that of an AC-5, and at higher temperatures, the performance of the modified asphalt can meet or exceed that of the AC-20 in the permanent deformation test. The following conclusions were drawn by the researchers:

- The dynamic resilient modulus-temperature relationship of the base asphalt can be improved to provide a stiffer asphalt at elevated temperatures and yet maintain a stiffness below that of the base asphalt at low temperatures.

- The fatigue resistance of a mixture is improved slightly with the polymer modified asphalt cement, particularly at high temperatures.

- The potential for low temperature thermal cracking is significantly reduced by use of a polymer-modified asphalt cement. Significantly higher tensile strengths are provided while a lower stiffness, is maintained which provides increased resistance to thermal cracking at temperatures in the range of -20°F to 0°F. The tensile strengths at extremely low temperatures are above those of an untreated asphalt concrete sample,

- An improvement in low temperature performance properties is seen in the tensile strain at failure at low temperatures for the modified binder. This is the most important factor in polymer modification for low temperature performance. With increased strains at failure, the polymer mixes are not as brittle as unmodified mixes and provide improved resistance to thermal cracking, and

- Polymer modification provides significant improvement in rutting resistance in comparison with the mixtures prepared with the base asphalt. The performance of the modified blends is similar to but not significantly better than that of an AC-10 or an AC-20 at 72°F. The improvement provided by the polymer modification is seen when rutting at 100°F is compared. At these elevated temperatures, the improvement provided by the polymer treatment over the untreated AC-5 is quite dramatic. Several of the polymer blends actually show no increase in rutting at the elevated temperature. At the elevated temperature, the performance of the polymer-modified blends was substantially equal to that of the untreated AC-10 and AC-20 mixtures.

Table 3-21. Thermal coefficients of contraction (× $10^{-5}/°F$) in temperature ranges in Illinois study (Carpenter & VanDam, 1987).

	Temperature Range (°F)			
Asphalt	72 to 40	40 to 20	20 to 0	0 to −20
1	1.06	1.10	1.22	1.10
2	0.99	1.24	1.22	1.14
3	1.04	1.24	1.40	1.16
4	0.92	1.08	1.33	1.25
5	0.96	1.29	1.49	1.23
6	–	1.51	1.33	1.10
7	1.37	1.32	1.41	1.21
8	1.42	1.66	1.33	1.22

3.7 Texas A & M -Lime

Description

In this study (Button, 1984), laboratory and field tests were conducted to evaluate the use of hydrated lime as an antistrip additive in hot mix asphalt concrete. The field study evaluated techniques for adding dry lime and slurry lime in batch and drum mix plants. The four primary objectives of the research program were (1) to determine the effectiveness of lime as an antistrip additive when added either dry or in slurry, (2) to investigate the effect of time delay after lime treatment of aggregate, (3) to evaluate the point of entry of lime in the production system, and (4) to assess the differences in mixtures produced in the batch and the drum mix plants.

An AC-20 asphalt cement from the Exxon refinery in Baytown, Texas, was used throughout this study. The cement was produced by the propane deasphalting process. Properties of the original asphalt cement are given in Table 3-22. The aggregates used were siliceous and were combined from pea gravel, washed sand and field sand. Dry hydrated lime was supplied in bags. When used as the slurry, it was mixed in a slurry mixer at a 70:30 weight ratio of water to lime. Lime (dry or slurry) was added at a rate of 1.5% of dry lime by weight of aggregate treated. The hot mix asphalt concrete mixture used in the study met the Texas State Department of Highways and Public Transportation specifications for fine graded surface course. The asphalt content was 5% and the air void content was 4.5% from the mix design. Hveem stability value was 41 and average density was 95.5% of the theoretical maximum.

Laboratory mix and compacted specimens were fabricated by applying the lime by seven different methods. One and one-half percent lime by weight of aggregate was added dry or in slurry to fine and individual coarse aggregates and to the total aggregate. Selected lime-treated aggregates were allowed to cure for 2 or 30 days before mixing with an asphalt. One set of specimens was made by adding 1.5% silica flour to the total aggregate in an attempt to determine the effects of merely adding an "inert" filler instead of lime.

Samples of the field mixtures were also obtained from the asphalt mixing plant. They were transported to the laboratory and compacted using a gyratory molding press to fabricate 4-in. diameter briquettes. The methods of adding hydrated lime to the field mixtures and identification of the codes used in subsequent figures are given in Table 3-23.

Results

Resilient moduli at 25°C were determined before and after vacuum saturation and soaking in water for seven days at 25°C. The resilient modulus ratio was computed by dividing resilient modulus after moisture treatment by its corresponding original value before moisture treatment. Most of the resilient modulus ratios are greater than 1 (Table 3-24), which indicates higher values of resilient modulus after moisture treatment than before. This unlikely phenomenon may be due to evaporative cooling of the saturated specimens during testing.

Table 3-22. Properties of original asphalt cement at Texas A & M (Button et al., 1984).

Characteristic Measured	Measurement
Viscosity	
77°F (25°C), poise	2.75×10^6
140°F (60°C), poise	1983
275°F (135°C), poise	3.78
Penetration	
77°F (100 gm, 5 s)	60
39.2°F (4°C) (100 g, 5 s)	0
39.2°F (4°C) (200 g, 60 s)	12
Softening point, °C	50 (122°F)
Flash point, °C	315+ (600+°F)
Specific gravity	1.03
After thin film oven test	
viscosity at 60°C (140°F)	5316
penetration at 25°C (77°F)	31
weight loss, %	0[a]
ductility, cm	150+
viscosity ratio	2.68
retained penetration, %	52

[a]Actually a slight gain in weight (0.07%) was indicated by repeated tests.

Figure 3-44 reveals that those laboratory mixtures treated with lime slurry consistently yielded the highest resilient modulus ratios. Those mixtures treated with slurry for two days or more before mixing and compacting yielded higher resilient modulus ratios than those treated only five minutes before mixing and compacting.

The indirect tension test was conducted at a temperature of 25°C and a deformation rate of 2 in./min. Cylindrical specimens of 4 in. diameter were tested before and after moisture treatment using the accelerated Lottman freeze/thaw procedure. Tensile strength ratios are plotted in Figure 3-45. Notably higher tensile strength ratios are exhibited by those mixtures treated with slurry. This indicates that, of those methods tested, slurry is the most beneficial form of lime application.

The most frequently reported theory regarding the ability of lime to decrease moisture induced damage in concrete involves the direct contact of wetted lime on the aggregate surface. Lime is purported to alter the surface chemistry of the aggregate, thus producing a more tenacious bond between the asphalt and the aggregate. However, in this experiment, mixtures DL and LA were made without any water and both of them exhibited improved resistance to moisture (Figure 3-45), therefore, other mechanisms appear to be involved. From the standpoint of tensile strength retention, no significant advantage to aging the lime-treated aggregate appears before mixing with asphalt.

Similar tests were performed on the field mix/laboratory compacted mixtures. Figure 3-46 indicates that tensile strength was considerably higher for those mixtures containing lime. Furthermore, those mixtures containing lime slurry generally yielded higher retained strength than those containing dry lime. There were no detectable differences in tensile strength ratio when the aggregate was treated with lime slurry immediately before mixing or was stockpiled for two days. Tensile strength ratios of resulting mixtures did, however, appear to decrease when slurry-treated aggregate was stockpiled for thirty days. There are no measurable differences in tensile properties or in water susceptibility in mixtures made in the batch plant or in those made in the drum mix plant.

Conclusions

Button (1984) made the following conclusions:

- Hydrated lime is effective in reducing moisture-induced damage of the paving mixture considered in this study,
- The most effective method for applying lime is in the presence of moisture. The moisture may be on the surface of the aggregate or the lime may be introduced as a slurry,
- Lime is an effective antistrip additive in both batch and drum mix plants. There were no significant differences in properties of the mixture prepared in the batch plant and those prepared in the drum plant,

Table 3-23. Explanation of codes used on figures for field mixtures at Texas A & M (Button et al., 1984).

Method of Adding Lime	Code Used on Figures
BATCH PLANT	
Control (no lime)	C_b
Dry lime in pugmill[a]	dry
Slurry on total aggregate + 2*da* in stockpile	*S*
DRUM MIX PLANT	
Control (no lime)	C_d
Dry lime on total aggregate at cold feed belt	CFB } dry
Dry lime at center of drum thru fines feeder	CD } dry
Slurry on field sand at cold feed belt	FS } slurry
Slurry on washed sand at cold feed belt	WS } slurry
Slurry on pea gravel at cold feed belt	PG } slurry
Slurry on total aggregate at cold feed belt	TA } slurry
Slurry on field sand + 2*da* in stockpile	FS } 2-day slurry
Slurry on washed sand + 2*da* in stockpile	WS } 2-day slurry
Slurry on pea gravel + 2*da* in stockpile	PG } 2-day slurry
Slurry on total aggregate + 2*da* in stockpile	TA } 2-day slurry
Slurry on total aggregate + 30*da* in stockpile	TA + 30 day

[a]Dry lime was added and mixed for 20 s before addition of asphalt cement.

Table 3-24. Mean values of data from laboratory mixed and compacted specimens[a] before and after freeze-thaw at Texas A & M (Button et al., 1984).

	Mixture Type								
	Control	Silica Flour	Dry Lime	Lime in Asphalt Cement	Slurry on Total Aggregate 5 min	Slurry on Fine Aggregate 2-days	Slurry on Coarse Aggregate 2-days	Slurry on Total Aggregate 2-days	Slurry on Total Aggregate 30-days
Test Procedure	C	SF	DL	LA	ST5	SF2	SC2	ST2	ST30
Overall average Air-void content, %[b]	7.3	7.2	7.5	9.1	5.6	7.4	7.4	6.0	7.9
Resilient modulus, psi × 10^3									
−13°F	1980 (7.3)	1860	1 990 (6.8)	1790 (8.9)	1 930 (4.8)	2 430 (7.3)	2070 (7.4)	2 080 (5.7)	1930
33°F	1650	1780	1 860	1480	1 590	1 670	1490	1 370	1480
68°F	493	467	550	363	505	370	381	511	343
104°F	44	36	50	29	37	25	27	40	32
77°F[b]	271 (7.3)	234	344 (7.2)	230 (9.1)	283 (5.6)	217 (7.4)	201 (7.4)	261 (6.0)	182
77°F (after freeze-thaw)	245 (7.5)	208	315 (8.4)	235 (9.7)	329 (7.2)	315 (7.1)	286 (7.3)	304 (6.5)	283
77°F (after soak)	281 (7.4)	280	391 (7.4)	255 (8.8)	362 (5.2)	291 (7.7)	277 (7.7)	371 (5.7)	298
Marshall test									
stability, lb	280 (7.1)	260	290 (7.1)	140 (8.8)	310 (5.1)	180 (7.2)	200 (7.1)	330 (5.7)	230
flow, 0.01 in.	12	14	19	15	17	17	18	16	15
stab (after soak)	...	450	430 (7.4)	240 (8.8)	490 (5.2)	320 (7.7)	360 (7.7)	610 (5.7)	320
flow (after soak)	...	15	19	18	18	19	18	18	15
Hveem stability									
stability	19 (7.4)	19	16 (7.4)	15 (8.8)	14 (5.2)	14 (7.7)	14 (7.7)	16 (5.7)	15
stab (after soak)	25	23	18	20	17	19	19	20	19
Splitting tensile test[c]									
stress, psi	110	110	110 (6.8)	80 (8.9)	110 (4.8)	90 (7.3)	90 (7.4)	120 (5.7)	100
strain, in./in.	0.012 (7.3)	0.012	0.010	0.011	0.010	0.011	0.012	0.010	0.014
secant Mod, psi	9400	8900	10 600	6800	11 500	7 600	7500	12 200	6600
stress (after freeze-thaw)	90	90	90 (8.4)	70 (9.7)	120 (7.2)	110 (7.1)	110 (7.3)	120 (6.5)	100
strain (after freeze-thaw)	0.013 (7.5)	0.013	0.010	0.009	0.010	0.010	0.012	0.009	0.013
secant Mod (after freeze-thaw)	7400	7100	9 700	8000	11 500	10 800	9300	14 200	8200

[a]Each value represents an average for three tests. Numbers within the table in parentheses indicate average air-void content of the three specimens tested to produce the data.

[b]Each value represents an average for twelve tests.

[c]Values were obtained at point of specimen failure.

NOTE: 1 psi = 6894 Pa, 1 lb = 0.4536 kg, and 1 in. = 0.0254 m.

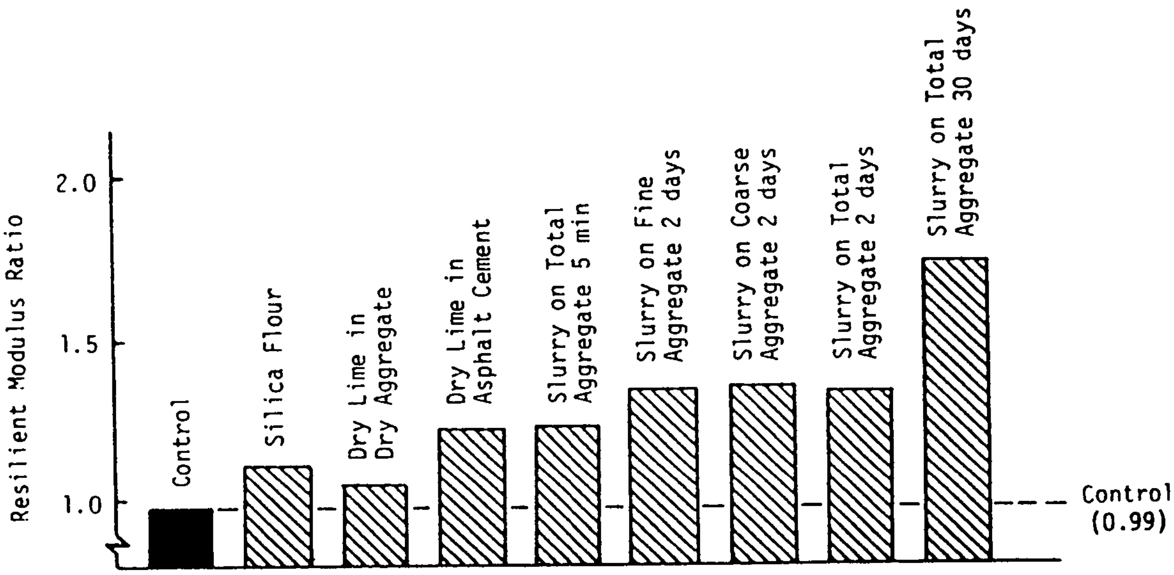


Figure 3-44. Resilient modulus ratios for seven-day soak moisture treatment on laboratory mixed/laboratory compacted specimens (measured at 25°C (77°) at Texas A & M) (Button et al., 1984).

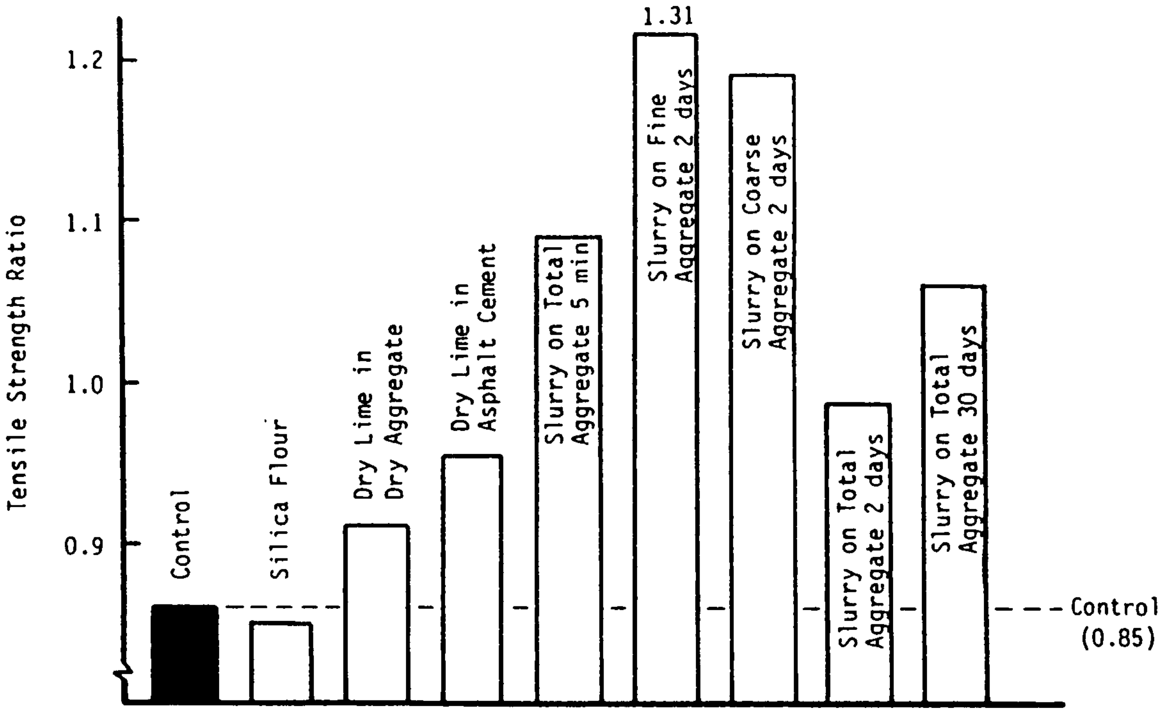


Figure 3-45. Tensile strength ratio for laboratory mixed and compacted specimens after freeze-thaw treatment at Texas A & M (Button et al., 1984).

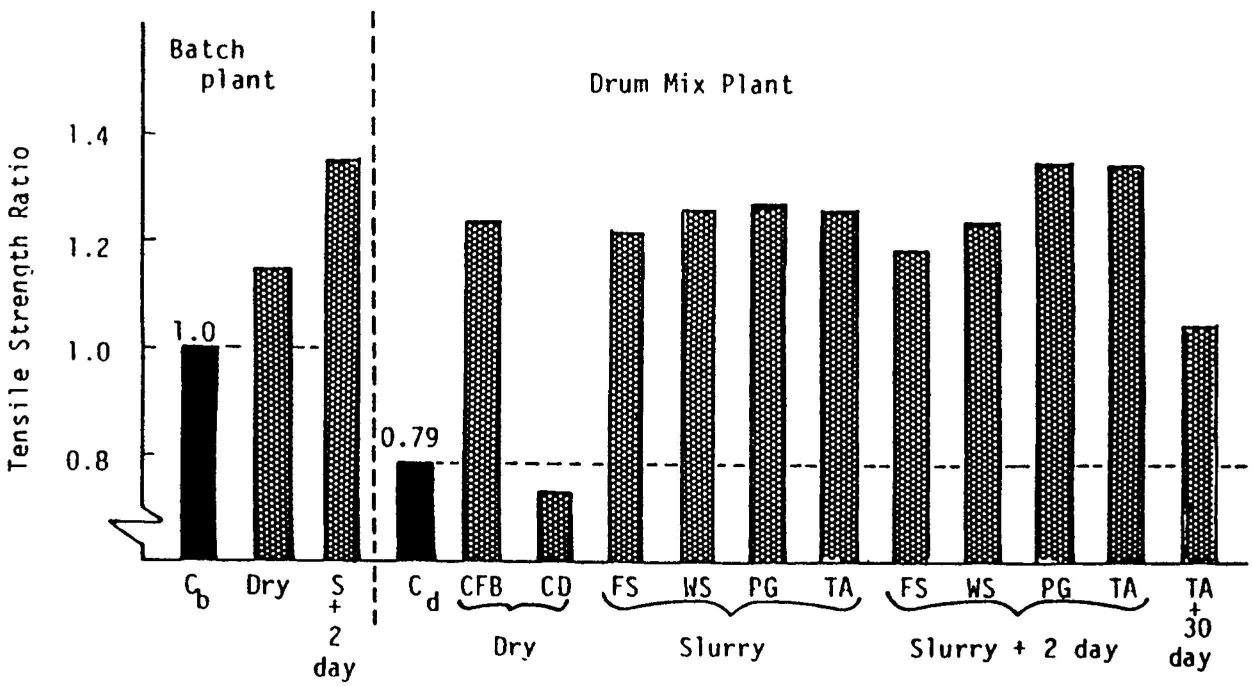


Figure 3-46. Tensile strength ratios before and after freeze-thaw moisture treatment for field mixed/laboratory compacted mixtures at Texas A & M (Button et al., 1984).

- Ratios of tensile strength and resilient modulus before and after treatment with moisture appear to be satisfactory laboratory procedures for estimating resistance to moisture damage of asphalt concrete mixtures, and

- The mixture containing 1.5% silica flour generally exhibited properties similar to the control mixture.

3.8 Washington - Wood Lignin

Description

The overall objective of this research (Terrel & Rimsritong, 1979) was to develop methods to use lignin as an extender for asphalt for the purpose of reducing the dependence on this material in highway paving mixtures. Specific objectives were (1) to develop and to verify a systematic procedure for evaluating and demonstrating the influence of lignin as an extender for asphalt in paving mixtures and the resultant effect on pavement performance, and (2) using the procedures developed above, to determine mix design alternatives that will result in a saving of asphalt material. As part of the overall program, the research reported herein is limited to the investigation of binders prepared by blending lignin and asphalt only in an open vessel. While other approaches are also being evaluated, the results of those studies were not included in this report.

Two paving grade asphalts were provided by Chevron Asphalt Company. The control asphalt binder, AR 4000 was selected, since it is commonly used by the Washington State Department of Transportation. A softer grade of asphalt cement, AC-5, was selected for mixing with various lignin materials. The properties of both the AR-4000 and AC-5 are shown in Tables 3-25 and 3-26. Dry powdered kraft lignin was used for this phase of the study. The material was a kraft pine Indulin AT produced by Westvaco Company of Charleston, South Carolina. Basic properties of the lignin and the particle size distribution are shown in Figure 3-47. The mineral aggregate used in all mixtures was a crushed glacial gravel with a predominance of granitic rock. The gradation met specifications for Washington Class "B" aggregates.

The powdered lignin is first oven-dried to remove the small amount of water that is retained when stored. Both the asphalt cement and the lignin are then preheated to 135°C and then the materials are hand-mixed with a spatula for approximately five minutes. Mixing is continued until a smooth mixture is obtained. After considerable testing, researchers concluded that the binders prepared in this manner were essentially physical mixtures of the two materials, and that neither was soluble in the other. The finely powdered kraft lignin tends to act as a reinforcement of the asphalt as noted by increasing viscosity and other properties. Tests included specific gravity, penetration, and ring and ball softening point. These results are summarized in Table 3-27. The mixture study reported was limited to binders with 30% kraft lignin.

Both resilient modulus and indirect tensile strengths were measured using diametral loading on the asphalt concrete briquettes. Three separate mixture designs were compared. The

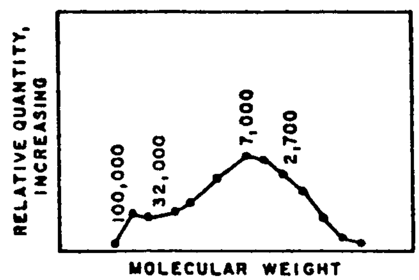
Typical Properties of Kraft Lignin "INDULIN" from Westvaco Company.

Physical Form ----- Free Flowing Brown Powder
Lignin content, %(on dry basis) ---------- 99
pH of 2% aqueous slurry at 25°C ----------- 6
Insoluble in warm 5% NaOH solution ----- 0.05
Surface tension of 1% solution of sodium salt
of INDULIN at -------------------------- 43
Flash point, °F ------------------------- 349
Fire point, °F -------------------------- 389
Sintering temperature, °F --------------- 370
Weight loss on heating in air, %
300°F ---------------------------------- 1
400°F ---------------------------------- 40
600°F ---------------------------------- 75
Bulk density, lb/ft^3, loose -------------- 26
packed ------------- 32
Ash, % (on dry basis) -------------------- 1
Moisture, % ----------------------------- 3
Specific gravity ------------------------ 1.3

Solubility in organic solvents

Benzene - nil
Hexane - nil
Methyl Ethyl Ketone - Partial
Methyl alcohol - Partial
Ethylene glycol - complete
Dioxane - complete
Monoethanolamine - complete
Dimethyl formanide - complete
Cellosolve - complete
50:50 Ketone Alcohol - complete

Molecular Weight-Distribution-
Gel Filtration, 0.1% NaOH
Solution



Particle Size Distribution of Powderlike
Kraft Lignin

Sieve No.	Size of Opening	% Passing*
100	150 μm	96.5
200	75 μm	60.1
270	53 μm	42.8
325	45 μm	29.8
400	37 μm	26.2

* Determined by Matrecon Inc.

Figure 3-47. Properties of Kraft Lignin in Washington study (Terrel & Rimsritong, 1979).

control mix was made with AR-4000 asphalt cement. The other mixtures contained AC-5 asphalt cement only, and lignin blended with AC-5 with L/A ratio of 30/70% by weight.

To evaluate lignin-asphalt mixtures as compared with conventional asphalt concrete, a series of diametral tension tests were conducted. In this study, all specimens were 4 in. diameter by 2.5 in. high. The resilient modulus tests for the three mixtures indicated that in each case, the AR-4000 mixture was typically highest and AC-5 mixtures were lowest, with the L/A in between. The variables studied included temperature, load duration and load frequency. Because of the general sensitivity to water for most asphalt mixtures, as well as the concern for L/A mixtures, specimens were also subjected to vacuum saturation and freeze/thaw tests. Essentially the results showed that the L/A mixtures behave much like asphalt concrete when subjected to these conditions. The results of the indirect tensile strength test were similar to those of the resilient modulus; again, the L/A mixtures were consistent with results for pure asphalt mixes.

Fatigue tests were conducted using the diametral tension approach. The results indicated that L/A mixtures should perform approximately the same as the AR-4000 mixtures over a reasonable range of repeated loads. This is illustrated in Figure 3-48. Again using repeated loading, the susceptibility of the several mixtures to permanent deformation is shown in Figure 3-49. As with previous results, it appears that the L/A mixtures fall somewhere between those with asphalt binders. An attempt was also made to predict the performance of these mixtures using the VESYS computer program. The creep compliance curves are shown in Figure 3-50; again, the L/A falls between the AR-4000 and the AC-5.

Conclusions

The results from this laboratory study show that the L/A binders produced have positive attributes. During the laboratory experiments, the L/A binders exhibited good qualities with respect to coating, workability, compaction, low temperature properties, and fatigue resistance. The principal findings may be summarized as follows:

- Hardness, softening point, and viscosity properties of lignin-asphalt binders increased directly with percent lignin,

- The resilient modulus of lignin extended asphalt mixtures appear to be dependent on the same factors as for other bituminous materials. The resilient moduli of the L/A mixtures at various temperatures were much higher than the AC-5 mix and were about the same as the AR-4000 mixes,

- The tensile strength of the L/A mixture was between that of the AC-5 and AR-4000 mixes,

- The L/A mixtures had a resistance to moisture damage and freeze/thaw damage at least as good as for conventional asphalt concretes,

- Creep characteristics of the lignin-asphalt mix was similar to normal asphalt concrete,

- Lignin-asphalt mixtures had a much higher resistance to fatigue failure than did AC-5 mixtures and was nearly the same as for the AR-4000 mixtures,

- Under repeated loading, the ability of the L/A mixture to resist permanent deformation was much better than for AC-5 mixtures and similar to AR-4000 mixtures, and

- Material layer equivalencies of the L/A mixture and conventional asphalt concrete are as follows:

  1 in. of L/A mix = 1.3 in. of AC-5 mix
  1 in. of L/A mix = 0.9 in. of AR-4000 mix.

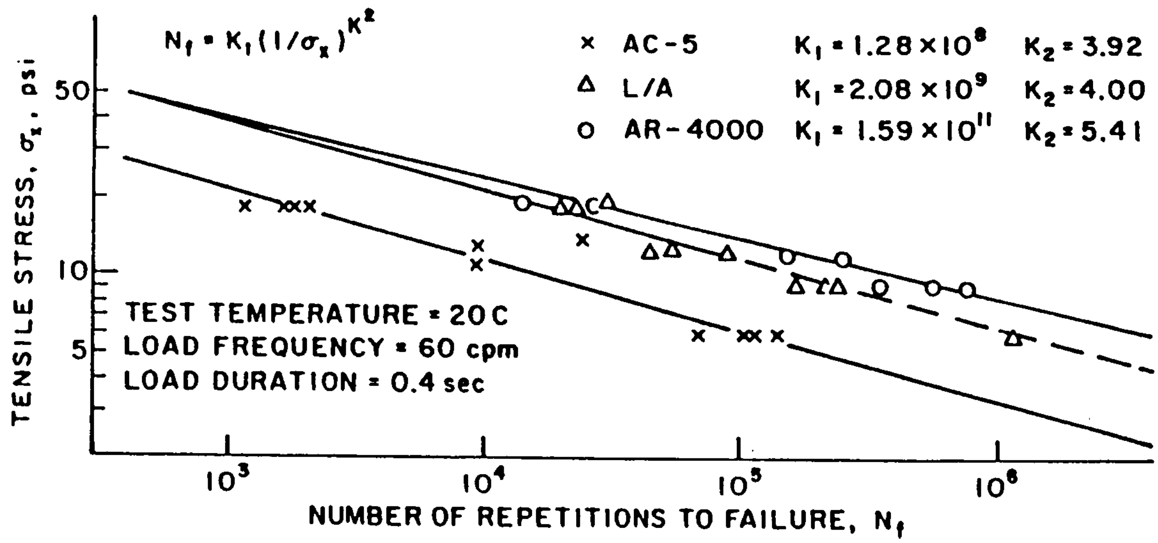


Figure 3-48. Fatigue response for various mixtures in Washington study (Terrel & Rimsritong, 1979).

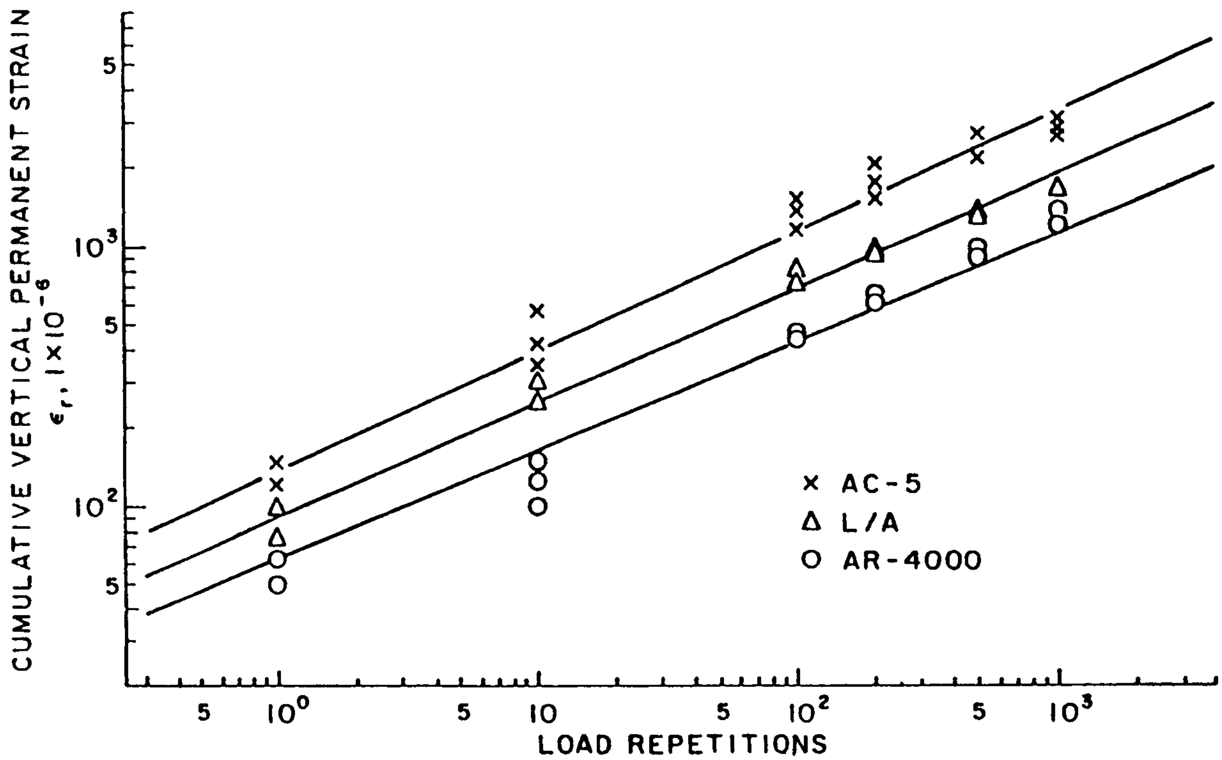


Figure 3-49. Effect of repeated loading on permanent strain in Washington study (Terrel & Rimsritong, 1979).

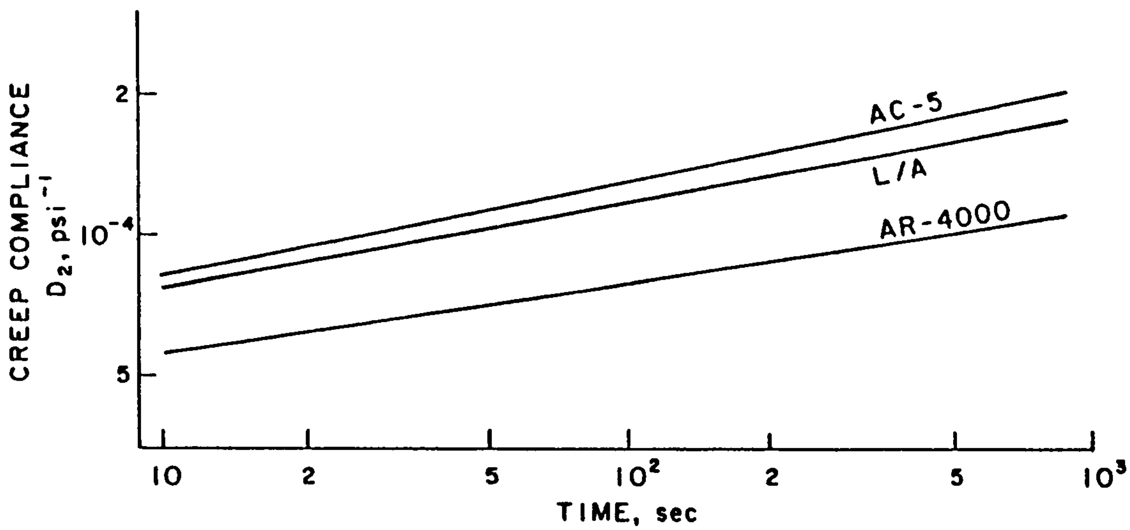


Figure 3-50. Creep compliance for various mixtures in Washington study (Terrel & Rimsritong, 1979).

Table 3-25. Physical properties of asphalt cements in Washington study (Terrel & Rimsritong, 1979).

PHYSICAL PROPERTIES	AR-4000*	AC-5**
Flash Point by COC, F	440	550
Penetration @ 77F, .1 mm	25	183
Viscosity @ 275F, CST	275	205
Viscosity @ 140F, POISE	3000-5000	553
Ductility @ 77F, CM	75	125+
Penetration Retained After RTFO @ 77F, %	45	47.5
Solubility in Trichlorethylene, %	99+	99+

* Specification from AASHTO M226

** Results of laboratory tests by Chevron Asphalt Co.

Table 3-26. Composition of AC-5 paving asphalts in Washington study (Terrel & Rimsritong, 1979).

COMPOSITIONS	AC-5*
Asphaltenes, %	19.9
Nitrogen Bases, %	24.0
First Acidaffins, %	18.1
Second Acidaffins, %	26.9
Paraffins, %	11.1(waxy)

* Determined by Matrecon Inc.

Table 3-27. Physical properties of lignin extended asphalt binders in Washington study (Terrel & Rimsritong, 1979).

BINDER DESCRIPTION	PENETRATION @ 4C, .1 mm	SOFTENING POINT, C	VISCOSITY @ 25C, POISE 10^5	SPECIFIC GRAVITY
AC-5	81	35	2.5	1.011
AC-5 with 10% Lignin	73	37	4.5	1.001
AC-5 with 20% Lignin	65	39	9.1	0.999
AC-5 with 30% Lignin	50	44	39	0.997
AC-5 with 40% Lignin	47	58	52	0.992
AC-5 with 50% Lignin	32	100+	*	0.989

* Could not make a film of binder for testing.

4
Summary and Conclusions

Although this literature review on asphalt modifiers was not exhaustive, it was conducted to better understand the effects of modifiers on the performance of in-service pavements. The goal was to collect and summarize information in the technical literature that relates modified asphalt properties to pavement performance and to report the findings and conclusions of the original authors. From this information, an attempt was made to determine whether modifiers significantly effect the properties of asphalt and also whether they significantly improve pavement performance.

Several modifier categories were considered in this report including fillers, fibers, extenders, polymers, plastics, antistrip agents, oxidants, and reclaimed rubber. An attempt was made to include at least one report for each modifier type available. Many additional reports exist that contain information that may link modified binders to performance (mostly laboratory performance). The objective was to obtain a representative sample of the available information by including at least one report for each modifier category.

Considerations for Data Interpretation

One of the main findings is that there are many problems associated with the nonstandard way in which material was tested and characterized and for which relevant data or information was available, which make it difficult to establish consensus relationships and trends associated with the use of modified materials. These same problems have previously been discovered with the performance of unmodified asphalts and are also reported in Finn et al. (1990).

Field performance of modified binders is generally measured through the use of special test sections placed in a project where the remainder is a control section using unmodified asphalts. Comparisons are then made between the performance of the test section and the control section. Interpretation of the test results relies on site-specific factors, including pavement structure characteristics and traffic and environmental conditions. The strong, confounding effects of these site-specific factors makes extrapolation of test results to more generalized conditions very difficult. The interpretation of test results may only be valid for these site-specific conditions.

In most of the studies, field performance measurements were conducted early in pavement life, that is, at 1 to 3 years. In many of these, the test and control sections were both in good condition and no discernible difference in performance could be detected between the two. The use of modifiers in asphalt pavement construction is a fairly recent practice in the United States compared with the Europeans, for example, and long-term performance data is largely unavailable. Based on the results of the studies, it appears that more long-term (greater than 5 years) field observations are needed to help distinguish the effects of the modifiers.

For most of the studies reporting field performance, asphalt properties and/or asphalt mixture properties were evaluated using traditional tests such as penetration, viscosity and ductility for asphalt and Marshall or Hveem stability for mixtures. As stated in Finn et al. (1990), these properties have proven useful in the past, and may be used in the future; however, the emphasis of SHRP is on new and innovative testing designed to be more strongly related to pavement performance.

Several of the laboratory studies report test results likely to be used in the future to measure physical properties of the asphalt binder such as dynamic viscoelastic properties. Such tests provide information such as complex shear modulus, dynamic viscosity, storage (elastic) modulus, viscous modulus, and loss tangent (ratio of viscous modulus to elastic modulus). It is expected that these properties will provide improved correlations with the mechanical properties of the asphalt-aggregate system. Similarly, some of the laboratory studies report test results to measure mix properties that are likely to be used in the future. Measurements of elastic, viscoelastic, shear and fatigue properties have been accomplished in some of the laboratory studies. Unfortunately, these measurements were rarely included in the field studies to establish the necessary links between laboratory and field performance.

Several nontraditional test methods (force-ductility, toughness-tenacity) have been introduced to measure the influence of modifiers on binder properties (force-ductility, toughness-tenacity). Generally, the test methods impose load or deformation characteristics (high strains), which may not represent actual conditions in the field. There is some controversy regarding whether binder property improvements, as measured by these particular nontraditional test methods, are reflected in actual field performance.

The intent of this report is not to make comparisons in methods; instead, the purpose is to identify possible associations between modified binder properties to performance. Thus, each method is evaluated on its own merits and comparisons have not been attempted.

Finally, findings from one investigation do not always concur with those from another investigation. Confounding effects of structure and environment, as discussed previously, are factors that could account for some of the differences. In addition, the base asphalt used has a pronounced influence on modifier effectiveness. A modifier may enhance certain properties of a given binder to produce more favorable performance characteristics but may not raise those characteristics to the level that may be obtained simply by changing asphalt source or grade.

Recognizing that problems do exist in attempting to interpret information from these studies, a cautious effort has been made to extract both qualitative and quantitative information from such data and field performance.

Findings tend to be inconclusive based on information available in the United States. However, general impressions from the European experience suggests improved performance, but higher costs (European Asphalt Study Tour, 1990).

Interpretation of Modifier Effects

From the preceding reviews of field and laboratory studies on modified asphalt binders and mixes, some general trends were noted as discussed later in this chapter. Due to minimal field data and inconsistencies in performance observations between projects, only qualitative interpretations can be established. Tables 4-1 and 4-2 summarize the conclusions that are reported by the researchers for field and laboratory studies, respectively.

Table 4-1. **Summary of Results for Field Studies.**

Modifier	Performance Parameters	Section	Conclusions
Arm-R-Shield	No long-term performance data (pavement condition is satisfactory after 1+ years)	2.1 Oregon (Hicks et al., 1987)	▪ Modulus values (@ 73°F) was 194 ksi vs. 457 ksi for control. ▪ In-place voids (after construction) was 6.9% vs. 7.1% for control. ▪ Absolute viscosity (@ 140°F) was ⅓ that of control, but kinematic viscosity was similar @ 275°F. ▪ Penetration (@ 77°F) was twice as high as control. ▪ Index of retained strength (IRS) was 93% compared to 97% for control. ▪ Modulus ratio (after freeze-thaw) was 0.70 compared with 0.78 for control. ▪ No. of repetitions to failure was approximately half that of control.
FiberPave	No long-term performance data (pavement condition is satisfactory after 1+ years)	2.1 Oregon (Hicks et al., 1987)	▪ Modulus values (@ 73°F) was 400 ksi vs. 457 ksi for control. ▪ In-place voids (after construction) was 6.5% vs. 7.1% for control. ▪ Both absolute viscosity and kinematic viscosity were lower than control by 30% and 12%, respectively. ▪ Penetration (@ 77°F) was similar to control. ▪ Index of retained strength (IRS) was 94% compared to 97% for control. ▪ Modulus ratio (after freeze-thaw) was similar to control. ▪ Number of repetitions to failure was 95% that of control.

Table 4-1. Summary of Results for Field Studies.

Modifier	Performance Parameters	Section	Conclusions
Bonifibers	No long-term performance data (pavement condition is satisfactory after 1+ years)	2.1 Oregon (Hicks et al., 1987)	▪ Modulus values (@ 73°F) was 387 ksi vs. 457 ksi for control. ▪ In-place voids (after construction) was 8.1% vs. 7.1% for control. ▪ Absolute viscosity (@ 140°F) was 85% of control; kinematic viscosity was similar. ▪ Penetration (@ 77°F) was similar to control. ▪ Index of retained strength (IRS) was 92% compared to 97% for control. ▪ Modulus ratio (after freeze-thaw) was 0.84 vs. 0.78 for control. ▪ Number of repetitions to failure was 63% that of control.
Pave Bond	No long-term performance data (pavement condition is satisfactory after 1+ years)	2.1 Oregon (Hicks et al., 1987)	▪ Modulus values (@ 73°F) was 475 ksi vs. 457 ksi for control. ▪ In-place voids (after construction) was 5.3% vs. 7.1% for control. ▪ Absolute viscosity (@ 140°F) was 90% that of control; kinematic viscosity was similar. ▪ Penetration (@ 77°F) was the same as control. ▪ Index of retained strength (IRS) was 95% compared to 97% for control. ▪ Modulus ratio (after freeze-thaw) was 0.92 vs. 0.78 for control. ▪ Number of repetitions to failure was 75% that of control.

Table 4-1. Summary of Results for Field Studies.

Modifier	Performance Parameters	Section	Conclusions
CA(P)-1	No long-term performance data (pavement condition is satisfactory after 1+ years)	2.1 Oregon (Hicks et al., 1987)	■ Modulus values (@ 73°F) was 284 ksi vs. 457 ksi for control. ■ In-place voids (after construction) was 4.9% vs. 7.1% for control. ■ Absolute viscosity (@ 140°F) was 20% higher than control; kinematic viscosity was almost 180% higher. ■ Penetration (@ 77°F) was 32% higher than control. ■ Index of retained strength (IRS) was 84% compared to 97% for control. ■ Modulus ratio (after freeze-thaw) was 0.73 vs. 0.78 for control. ■ Number of repetitions to failure was three times that of control.
Sulfur	Pavement Condition Index (Longitudinal/transverse/ reflection cracking, rutting, alligator cracking)	2.3 FHWA (Beatty et al., 1987)	■ No significant difference in performance between sulfur-extended asphalt pavements and control sections. ■ No relationship between PCI and pavement age and freezing index. ■ All pavements surveyed were in good condition.

Table 4-1. Summary of Results for Field Studies.

Modifier	Performance Parameters	Section	Conclusions
Sulfur	Reflection transverse cracking, Rutting	2.7 Michigan (DeFoe, 1983)	■ Sulfur asphalt mixes had higher resilient modulus than control by approximately 30% @ 72°F and 50% @ 40°F. ■ Rut depths similar for both sections after five years. ■ Thermal contraction coefficients of sulfur-asphalt mixes were ½ that of control mix. ■ Sulfur-asphalt had 50% higher indirect tensile strengths.
Sulfur	Rutting, thermal cracking, skid resistance	2.11 Ontario (Fromm et al., 1979 1981)	■ Marshall stability and flow increased with an increase in sulfur content. ■ Rut depths decreased with addition of sulfur. ■ Skid resistance equal to or better than control.
Sulfur	Aging, alligator cracking, skid resistance, thermal cracking	2.12 California (Predoehl, 1989)	■ Sulfur reduces hardening rate as measured by absolute viscosity @ 140°F compared to control. ■ SEA pavements resist alligator cracking better than control pavements. ■ Skid resistance not affected by addition of sulfur. ■ SEA pavements may be more susceptible to thermal cracking in colder climates as sulfur content increases (40% vs. 20% sulfur). Results are mixed. ■ 20% sulfur pavement had best field performance.

Table 4-1. **Summary of Results for Field Studies.**

Modifier	Performance Parameters	Section	Conclusions
PlusRide	No long-term performance data (condition is satisfactory after 1+ years)	2.1 Oregon (Hicks et al., 1987)	▪ Modulus values (@ 73°F) was 341 ksi vs. 457 ksi for control after one year of service. ▪ In-place voids (after construction) was 3.7% vs. 7.1% for control. ▪ Absolute viscosity (@ 140°F) and kinematic viscosity (@ 275°F) were lower than control. ▪ Penetration (@ 77°F) was almost twice as high as control. ▪ Index of retained strength (IRS) was 64%. ▪ Modulus ratio (after freeze-thaw) was 0.69 (average) vs. 0.78 for control. ▪ No. of load applications to failure was more than 10 times higher than control.
Rubber (PlusRide)	No long-term performance data available (pavement is in satisfactory condition after 1+ years).	2.2 Mt. St. Helens (Lundy et al., 1987)	▪ Modulus (@ 22.5°C) increased with time from 225 ksi at construction to 443 ksi at 33 months. Control mixes had higher moduli, from 346 ksi to 610 ksi. ▪ No difference in deflection results. ▪ Rubber-modified mixes hardening faster than control mixes.

Table 4-1. **Summary of Results for Field Studies.**

Modifier	Performance Parameters	Section	Conclusions
Rubber (styrene-butadiene-styrene block copolymer)	Aging	2.4 California (Reese, 1989)	▪ Modified binder has higher penetration @ 77°F after field aging (63-119 dmm vs. 9-16 for control). ▪ Modified binder has lower absolute viscosity @ 140°F after field aging (2,900-8,000 P vs. 21,000-53,000 P for control). ▪ Modified binder has higher ductility at 77°F after field aging (from 8 cm for control to 95 cm, from 21 cm for control to 100+ cm). ▪ Age hardening as measured above was lower for modified mixes than for controls. ▪ Tilt-Oven test was able to qualitatively distinguish relative hardening characteristics.
Rubber (Ground crumb and reclaimed)	Rutting, reflection cracking	2.8 Michigan (DeFoe, 1985)	▪ The control section had a higher indirect tensile strength (165 psi vs. 115-169 psi). ▪ The control sections had the highest resilient modulus @ 72°F (822 ksi vs. 422-625 ksi). ▪ Control section had lowest tensile strain at failure (0.0078 vs. 0.0079-0.015). ▪ Reclaimed rubber sections showed less extreme effects than ground crumb rubber. ▪ Rubber sections had lower rutting than controls. However, sections with 1.5% ground rubber (lowest rutting) disintegrated over time.

Table 4-1. Summary of Results for Field Studies.

Modifier	Performance Parameters	Section	Conclusions
Chemkrete	Rutting, Transverse cracking	2.5 Colorado (Wood & LaForce, 1984)	■ Recovered penetration for Chemkrete section are half of control. ■ Recovered viscosities (@ 140°F) at top of cores for Chemkrete section are 5 times higher than control. ■ Resilient modulus @ top of cores for Chemkrete section are approx. 30% higher than control. ■ Recovered viscosities (@ 140°F) and resilient modulus at bottom of cores are similar for both sections. ■ Rut depths were similar for both sections (0.12" for Chemkrete, 0.20" for control). ■ Transverse cracking observed on Chemkrete sections (after first and second winters) but none on controls.
Chemkrete	Rutting, Transverse cracking	2.6 Illinois (Saner, 1987)	■ Chemkrete reduced original binder viscosity @ 140°F from 1820 P to 995 P. ■ Chemkrete increased original binder penetration @ 77°F from 69 dmm to 103 dmm. ■ Kinematic viscosity (@ 275°F) was the same. ■ Rut depths were inconsistent between projects. ■ One chemkrete section had 17% transverse cracking vs. 6% for control section.
Chemkrete	Rutting	2.10 Montana Big Timber (Jennings et al., 1988)	■ Chemkrete section failed soon after construction. Rut depths reached 0.75 in.

Table 4-1. Summary of Results for Field Studies.

Modifier	Performance Parameters	Section	Conclusions
Fly Ash	Rutting, transverse cracking	2.10 Montana Big Timber (Jennings et al., 1988)	■ Rutting data confounded by air voids and asphalt content. ■ Transverse cracking performance confounded by asphalt source. ■ Recovered penetration @ 77°F generally increased by varying amounts depending on refinery. ■ Recovered viscosity @ 275°F and ductility @ 40°F results confounded by refinery source.
ACRA (anti-stripping agent)	Rutting, transverse cracking	2.10 Montana Big Timber (Jennings et al., 1988)	■ Transverse cracking increased in all anti-strip sections compared to control. ■ Rutting data confounded by air voids and asphalt content. ■ Recovered penetration @ 77°F, viscosity @ 275°F, and ductility @ 40°F results confounded by refinery source.
Carbon Black (Microfil 8)	Rutting, transverse cracking	2.10 Montana Big Timber (Jennings et al., 1988)	■ Transverse cracking was reduced by ⅔ compared to control. ■ Rut depths were ½ that of control. ■ Possible confounding effects due to air voids.

Table 4-1. Summary of Results for Field Studies.

Modifier	Performance Parameters	Section	Conclusions
Lime	Rutting, transverse cracking	2.10 Montana Big Timber (Jennings et al., 1988)	▪ Transverse cracking generally decreased depending on refinery. ▪ Rutting data confounded by air voids and asphalt content. ▪ Recovered penetration @ 77°F generally decreased with the addition of lime depending on refinery. ▪ Recovered viscosity @ 275°F generally increased with addition of lime depending on refinery. ▪ Recovered ductility @ 40°F generally decreased with addition of lime depending on refinery.
Lime	No long-term performance data (pavement condition is satisfactory after 1+ years)	2.1 Oregon (Hicks et al., 1987)	▪ Modulus values (@ 73°F) was 511 ksi vs. 457 ksi for control. ▪ In-place voids (after construction) was similar to controls. ▪ Absolute viscosity (@ 140°F) was 78% that of control; kinematic viscosity was 92% that of control. ▪ Penetration (@ 77°F) was similar. ▪ Index of retained strength (IRS) was 94% compared to 97% for control. ▪ Modulus ratio (after freeze-thaw) was 0.90 compared to 0.78 for control. ▪ Number of repetitions to failure was 70% that of control.

Table 4-1. Summary of Results for Field Studies.

Modifier	Performance Parameters	Section	Conclusions
Plastic (polyolefin)	Thermal cracking, moisture susceptibility (No field performance reported)	2.9 Univ. of Nevada (Krater et al., 1988)	▪ No one modifier consistently provided highest increase in resilient modulus @ 104°F. However, 5% Plastic 2 and 2% Plastic 1 with 3% latex provided highest improvement overall.
Latex (Styrene-butadiene-rubber)			▪ Majority of results from 10°F and 34°F resilient modulus testing were within 10% of each other and within one standard deviation. ▪ Resilient modulus @ 77°F were consistently higher for modified mixes. ▪ Indirect tensile strength @ 10°F were higher for modified mixes. At 77°F, they were the same. ▪ Modifiers did not improve resilient modulus or indirect tensile strength after moisture conditioning.

Table 4-2. Summary of Results for Laboratory Studies

Modifier	Performance Parameters	Section	Conclusions
Polymers (no other identification)	Laboratory fatigue, creep, aging	3.1 Goodrich (Goodrich, 1988)	■ Force ductility tests showed polymer-modified binders had the highest maximum stress and strain. ■ Toughness-tenacity test results were highest with polymers. ■ Polymer modified asphalts had lower limiting stiffness temperatures than control, but this is due to the asphalt binder selected for modification rather than the polymer. ■ Flexural fatigue lives increased significantly over control. However, improvement was not to the level of the least temperature susceptible asphalt without polymer. ■ Creep @ 40°C was reduced with addition of polymer compared with control. However, creep was not reduced to the level of the least temperature susceptible asphalt without polymer. ■ Viscosity of polymer-modified binders only slightly reduced after aging. However, oxidation as measured by infrared analysis and neutron activation was the same as control. ■ Polymer modified binders had lower complex modulus than the control binder.

Table 4-2. Summary of Results for Laboratory Studies

Modifier	Performance Parameters	Section	Conclusions
Rubber (styrene-butadiene-styrene and styrene-butadiene block copolymers)	Dynamic modulus, resilient modulus, indirect tensile strength	3.2 New Mexico (Schuler et al., 1987)	▪ Polymer modified asphalts had lower penetration @ 77°F (83-100 vs. 122 for control), however, not to the level of AC-20 (64). ▪ Polymer modified asphalts had higher ductility @ 39°F (48-63 vs. 10 cm for control). ▪ Polymer modified asphalts had higher toughness (165-328 in.-lb. vs. 49) and tenacity (126-284 to 18 in.-lb.) versus control. ▪ Polymer modified asphalts had higher absolute viscosity @ 140°F (1855-9940 P vs. 691 P for control). The viscosity of AC-20 was 1958 P. ▪ Above effects were more pronounced with greater polymer contents. ▪ Force ductility tests showed the same "asphalt modulus" and higher "asphalt polymer modulus" than control. ▪ |Eta|* from dynamic modulus tests decreases more slowly with increasing temperature for polymer modified binders within the temperature range of 25°C to 75°C. ▪ Tan δ above 25°C increases much more rapidly for unmodified binders as compared with modified binders. ▪ Small differences in indirect tensile strengths and strains exist between polymer modified and unmodified binders with the exception of the significantly higher strength for the 6 percent polymer modified at -18°C.

Table 4-2. Summary of Results for Laboratory Studies

Modifier	Performance Parameters	Section	Conclusions
Latex (emulsified SBS) Rubber (dry crumbs) Ethylene vinyl-acetate Polyethylene (Novophalt) Carbon Black (Microfil 8)	Creep, moisture susceptibility, fatigue, thermal cracking	3.3 Texas A & M (Button et al., 1987)	▪ Penetration (@ 77°F) all decreased with the addition of modifiers; Viscosity (@ 140°F & 275°F) increased significantly. ▪ Resilient modulus (@ different temperatures ranging from 0 to 100°F) of modified mixes generally increased; there was no change at temperatures less than 0°C. ▪ Indirect tensile strength increases at lower temperatures (-15°) and higher loading rates (2 in./min). ▪ Deformation at failure was generally increased by additives. ▪ Little effect on moisture susceptibility. ▪ Flexural fatigue response superior @ 0°C and 20°C. ▪ At 100°F, all additives except latex produced better creep performance than control. At 40°F, all except polyethylene produced equal or better creep compliance. ▪ Greater resistance to crack propagation @ 1°C.

Table 4-2. Summary of Results for Laboratory Studies

Modifier	Performance Parameters	Section	Conclusions
Lime	Thermal cracking, permanent deformation (Laboratory)	3.4 WRI (Petersen et al., 1987)	■ Lime treatment reduced age hardening as measured by the dynamic viscosity. ■ Lime treatment increased stiffness of unaged asphalts @ 60°C and reduced stiffness of aged asphalts. Suggests increased rutting and fatigue cracking resistance. ■ Lime treatment increased asphalt tensile-elongation @ -5°C. Should improve resistance to thermal cracking.
Lime	Moisture susceptibility (Laboratory)	3.7 Texas A & M	■ Higher values of resilient modulus after vacuum saturation and soaking in water for 7 days @ 25°C were found for lime mixes. ■ Lime slurry had highest resilient moduli ratios and higher tensile strength ratios.
Ethylene vinyl acetate (EVA)	Laboratory fatigue	3.5 Bradford, England (Salter et al., 1987)	■ Polymer modified binders have improved fatigue lives compared with control and other additives. EVA had best fatigue life improvement.
Polypropylene fiber			■ Regression equations were developed relating fatigue life to stress and strain.
Rubber (powder)			■ Addition of 20% sulfur decreased flow but did not change Marshall stability.
Sulfur			■ Fiber specimens had mixing/compacting problems.

Table 4-2. Summary of Results for Laboratory Studies

Modifier	Performance Parameters	Section	Conclusions
Kraton (polymer)	Permanent deformation, fatigue, thermal cracking (Laboratory)	3.6 Illinois (Carpenter et al., 1987)	▪ Polymer modified mixes are stiffer (using resilient modulus) until temperature < 20°F, when they become softer than control. ▪ Tensile strength increased @ 72°F. ▪ Tensile strain at failure increased @ 72°F and at low temperatures. ▪ Fatigue resistance is improved particularly at high temperatures. ▪ Higher tensile strengths (increased resistance to thermal cracking) are provided while a lower stiffness @ -20°F to 0°F is maintained. ▪ Rutting resistance is improved with modified mixes @ 100°F; no difference @ 72°F.
Wood Lignin	Moisture susceptibility, creep, fatigue, permanent deformation	3.8 Washington (Terrel et al., 1979)	▪ Hardness and viscosity properties of lignin-asphalt mixes increased compared to control. ▪ Resilient modulus and tensile strength increased compared with control's. ▪ Creep characteristics were similar to control. ▪ Greater resistance to moisture damage and freeze/thaw damage. ▪ High resistance to fatigue. ▪ Resisted permanent deformation better than control.

Field Performance

Good field performance for both control and modifier test sections was reported in the Oregon and Mount St. Helens studies, but no field data was reported in the University of Nevada study. For these studies, indications are that modifiers have not significantly affected short term performance.

Rutting: Rutting measurements were reported in the following studies: FHWA-Sulfur, Colorado-Chemkrete, Illinois-Chemkrete, Michigan-Sulfur, Michigan-Rubber, Montana-Chemkrete and Carbon Black, and the Ontario-Sulfur. Sulfur slightly decreased average rut depths in the FHWA and Ontario studies and had no significant effect in the Michigan study. Chemkrete had an inconsistent effect on rutting, slightly lower in Colorado, mixed results in Illinois, higher in Michigan, and a failure in Montana. Rut depths decreased when ground and reclaimed rubber was used in Michigan. No other studies of asphalt rubber reported rut depths for comparison. Rut depths obtained from the Montana study showed Carbon Black sections had only one half the rut depth of the control sections. The authors cautioned, however, that air voids may have confounded the results.

Cracking: Cracking measurements are included in the following studies: FHWA-Sulfur, Colorado-Chemkrete, Illinois-Chemkrete, Montana-Chemkrete and Carbon Black, and the California-Sulfur. Sulfur sections in the FHWA study had slightly less cracking of all types than control sections, but in the California study, more reflected thermal and less reflected alligator cracking was exhibited in the sulfur sections. Predoehl (1989) found that the sulfur-extended binders may be more susceptible to thermal stresses in cold climates as sulfur content increases. Chemkrete was associated with increased transverse cracking in the Colorado and Illinois studies. In the Montana study, the Chemkrete section failed prematurely; therefore, no cracking data was available. Crack data from the Montana study also revealed reduced transverse cracking with Carbon Black and lime and increased transverse cracking in the ACRA (anti-strip) sections.

Field Aging: Field aging was evaluated in the California-SBS and the California-Sulfur studies by measuring recovered binder properties at various pavement ages. In both studies, binders were aged in the laboratory using the Tilt-Oven Durability Test (California Test Method 374). Aged penetration, viscosity, and ductility were then compared to recovered properties after field aging. Predoehl (1989) indicates that the Tilt-Oven Test is intended to simulate a low desert climate. In the SBS study, the Tilt-Oven Test was shown to be severe enough to qualitatively distinguish the relative hardening characteristics of the binders; i.e., the test predicted significantly more hardening for unmodified binders. However, quantitatively, the test was more severe than after two years of aging in the low desert field conditions on the study. In the sulfur study, the Tilt-Oven Test overpredicted field aging for all binders regardless of climate (cold or hot). Predoehl (1989) concluded the test could not be used to directly predict age hardening of the binders for field conditions at the sulfur test sites. From field aging results, Predoehl (1989) found that sulfur extended binders had slower or equal hardening rates in comparison with the control base asphalt. He also found that overall cracking frequency appeared to relate well with binder hardening.

Laboratory Performance

A wide variety of laboratory tests were used to characterize binders and mixtures made with and without modifiers. For binders, viscosity, penetration, and ductility, force ductility, and toughness-tenacity test results are reported. Marshall stability, resilient and complex modulus, fatigue, creep and water sensitivity tests were used to characterize mixes.

Conventional binder properties such as viscosity, penetration, and ductility were used to measure temperature susceptibility and hardening characteristics. Viscosity was generally "controlled" by selecting an appropriate base asphalt grade for modification to produce modified binders with viscosities comparable to typical paving grade asphalts (e.g. AC-20, AR-4000). In general, most of the modifiers increased the absolute viscosity (140°F) of the binders and decreased the penetration at 77°F, although Arm-R-Shield, CA(P)-1 and PlusRide did not follow this trend. The Goodrich-Polymer and New Mexico-Styrenic Block Copolymer studies both evaluated force-ductility and toughness-tenacity. Both studies showed that modified binders were capable of increased stress and strain in the force-ductility test and gave higher toughness-tenacity results than control binders.

Mixture resilient modulus results varied by modifier: Arm-R-Shield, sulfur, plastic, SBS latex, and Kraton gave higher resilient modulus than the controls at room temperatures. At lower temperatures, Kraton had a lower modulus, but the plastics generally had higher moduli than the controls. PlusRide had consistent modulus values of approximately 2/3 of control, and both the ground and reclaimed rubber mixes also had lower moduli. The CA(P)-1 product was also approximately 2/3 that of the control, and fibers provided resilient moduli comparable to the control. The dynamic modulus (Eta*) for polymers reported by Goodrich were generally lower than control binders throughout the temperature range of -20°C to 60°C. Dynamic modulus (Eta*) for the styrenic block copolymer modified mixes reported in the New Mexico study were approximately equivalent to the control below 20°C, but were consistently higher than control with increasing temperatures above 20°C. Loss tangent (tan δ) values in both studies were lower for polymer modified binders than for the controls at higher temperatures. In the Goodrich study, the loss tangent of the polymer modified binders did not reach the level of Binder B, the least temperature susceptible binder. Higher tan δ values indicate more Newtonian behavior, and Goodrich (1988) states these binders may be more susceptible to rutting.

Fatigue properties: Fatigue properties were generally improved by the presence of modifiers. The CA(P)-1 polymer PlusRide, EVA, rubber powder, polypropylene fiber, Kraton, wood lignin, and the polymers studied by Goodrich all improved fatigue life. The FiberPave and Bonifiber modifiers gave similar or slightly reduced the fatigue lives when compared to the control specimens. Arm-R-Shield and the PaveBond modifiers produced fatigue lives lower than the controls.

Low-temperature diametral creep: Low-temperature diametral creep was measured by Goodrich to determine the temperatures at which the mixture stiffnesses reached a limiting value of 1.5×10^6 psi. The polymer modified binders had lower limiting stiffness temperatures than control binders but improvement was not to the level of the least temperature susceptible asphalt (Binder B). High-temperature creep deformation was lower for the polymer modified binders but not to the level of Binder B. In the Texas A & M study of various polymers, the low-temperature creep compliance properties of AC-5 binder modified with SBR, EVA, SBS, and Carbon Black were better than the AC-20 control, and the polyethylene modified AC-5 exhibited similar compliance to the AC-20 control. Button et al. (1987) indicate that compliant mixtures may better resist low-temperature cracking. At high temperature, the AC-5 with polyethylene provided resistance to creep deformation equivalent to the AC-20 control. The AC-5 with EVA or SBR allowed significantly higher deformation than the controls. Button et al. (1987) indicated that excessive permanent deformation at high service temperatures could be expected for these mixtures.

Water sensitivity: Water sensitivity was measured for several mixtures through the use of retained resilient modulus or indirect tensile strength after freeze-thaw conditioning. The resilient modulus ratio (modulus after freeze-thaw conditioning to original) was measured in the Oregon-Various modifiers study. Results ranged from 0.63 for PlusRide to 1.0 for CA(P)-1 with lime. In general, the granular rubber modifiers (PlusRide and Arm-R-Shield) had lower or equal ratios as the control binder (0.78). Lime consistently improved the modulus ratio when used alone (0.90), with Pave Bond (0.85), and with CA(P)-1. The best improvement came from the combination of lime and CA(P)-1 with a ratio of 1.0. PaveBond also improved the modulus ratio with (0.85) and without (0.92) lime. It is interesting to note that the combination of lime and PaveBond did not perform as well as the two modifiers used alone. Lime also reduced water sensitivity in the Texas A & M-Lime study. Dry lime gave higher retained indirect tensile strength ratios than control mixtures. The best results were obtained when lime was added to moist aggregate or in slurry form. Resilient modulus and indirect tensile strength testing after Lottman freeze-thaw conditioning in the University of Nevada-Plastic and Latex study showed lower retained ratios for modified mixtures than control mixes. Results varied with modifier type and content, and no general trends could be determined.

Laboratory aging: Laboratory aging was measured by Petersen et al. (1987) for binders with and without lime. Thin-film accelerated aging tests developed by the authors were used to harden the binders, and aging indices (aged viscosity divided by original viscosity) were significantly lower for lime modified binders. In addition, complex dynamic shear moduli (G^*) for aged lime modified binders were lower than for control binders for three of four asphalt sources. Finally the loss tangent ($\tan \delta$) values were higher for aged lime modified binders relative to control binders. Petersen et al. (1987) states that higher $\tan \delta$ values suggest increased ability for lime modified binders to undergo stress-releasing plastic deformation, as opposed to micro-crack formation and tensile fracture during flexing.

Conclusions

1. Modifiers have an influence on the performance-related properties of asphalt cement and asphalt concrete as measured in the laboratory,

2. Modifiers have an influence on the performance of asphalt pavements as determined from field test section evaluations; however, the magnitude of this influence is difficult to quantify due to the confounding effects of environment, traffic, construction, and other factors,

3. The association of modifier effects and performance has been more qualitative than quantitative. Conventional binder and mixture properties, which are typically used to characterize modified binders and mixtures, do not lend themselves easily to analytical models for use in predicting performance,

4. Unconventional test methods involving high strains have identified potential benefits of modified binders. However, controversy exists regarding whether these tests are an accurate measurement of field performance,

5. There is a need to use standard or baseline test procedures to measure modified binder and mixture properties as part of field investigations to evaluate their effects on pavement performance,

6. There is also a need to use a standard or baseline procedure for measuring and recording the condition of in-service pavements,

7. Relatively little long-term field performance data exists for pavements constructed with modified binders. Most modifiers have only recently been incorporated into asphalt pavements, and consequently most studies have only short-term field performance data. Many times, pavement deterioration is minimal and modifier effects are inconclusive. To complicate matters, formulations may change as the technology improves, potentially making prior information obsolete,

8. Although modifiers may improve the performance-related properties for a given asphalt binder, the performance may not be raised to the level that may be obtained simply by changing asphalt source or grade, and

9. The cost-effectiveness of modifiers should be weighed when determining their suitability for use.

An overall summary of information would suggest that (1) asphalt modifiers do influence binder and mixture properties and, hence, could or should influence pavement performance, and (2) the ability to accurately interpret the association between asphalt modifiers and pavement performance has not yet been established through field studies in the United States.

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