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16. Abstract

The overall objective of this study was to evaluate a variety of asphalt additives in the laboratory and in the field to determine their merit in reducing cracking and rutting. In the late 1980s, test pavements were constructed in Texarkana, Sherman, San Benito, and Ft. Worth, Texas. In 1995, cores were obtained from Texarkana and Sherman and evaluated in the laboratory. Asphalt binders, retained during construction and sealed in cans, were tested using the Superpave binder tests and certain chemical tests. Retained binders and aggregates were combined in accordance with the original mixture designs, compacted in the laboratory, and tested using the NCHRP AAMAS test protocols to assess relative resistance to fatigue cracking. The specific objective of the work reported herein was to test the binders and mixtures to determine what properties correlate with the field cracking. A secondary objective was to evaluate the ability of the laboratory tests to identify binders and mixtures susceptible to cracking. Asphalt additives included latex, ethylene vinyl acetate, styrene-butadiene styrene-block (SBS) copolymer, SBS vulcanized with asphalt, manganese organic complex, polyethylene, and carbon black.

Certain polymers when blended with the proper grade of asphalt will significantly reduce fatigue cracking. Field tests indicate that the addition of a polymer will reduce rutting when HMA is overasphalted or will allow the use of higher binder contents. Susceptibility to fatigue cracking was related to low loss tangent values from DSR testing, high levels of oxidation as measured by FT-IR, high amounts of large molecular size (LMS) material from GPC testing, and high asphaltene contents. Binders which failed the Superpave high temperature grading did not produce rut-susceptible mixtures, but did produce crack-resistant mixtures. AAMAS test results showed no correlation with the observed fatigue cracking.

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IMPLEMENTATION RECOMMENDATIONS

Based on the findings of this study, the following implementation concepts are recommended.

1. Usually, only a few asphalt binders and aggregate materials are available for use in asphalt pavements in a given area of the country. In a given area, as long as unmodified asphalts are performing satisfactorily with the available aggregate materials, additives are probably not a necessary expenditure. If a particular pavement performance problem is recurrent, in the area or anticipated for a given project, and asphalt additives can be expected to ameliorate the problem, then additives should be incorporated into the bid item either by generic name or by SHRP specification. (Additives are typically required to meet certain higher PG grades, i.e., when the sum of the absolute value of the 2 numbers in the PG grade equals or exceeds 86, a modifier is usually required.) Most of the polymer additives and latex increase the viscosity of the asphalt at higher performance temperatures (say 50°C and higher) and have very little effect on viscosity at lower performance temperatures (say 5°C and lower.) Proper use of asphalt additives depends on the particular anticipated pavement distress to be addressed.

2. If the anticipated pavement problem is cracking and typical mixtures are rut resistant, then the polymer additive should be incorporated into a softer than usual asphalt. The soft asphalt remains relatively flexible at low temperatures and thus acts to retard thermal and possibly even fatigue cracking while the polymer helps protect the mixture from rutting at high temperatures.

3. If low-temperature cracking has not been a problem and the anticipated pavement problem is rutting, then the polymer additive should be incorporated into the asphalt grade normally used in the particular area. The addition of the polymer should not have negative effects on low-temperature cracking, but will help offset rutting problems. Serious rutting problems cannot be solved by modified asphalts; they must be solved by adjusting aggregate quality or gradation. Larger maximum size aggregate and a gradation affording stone-to-stone contact will reduce rutting.

4. The Superpave asphalt paving mixture design process provides guidelines for selecting the appropriate PG asphalt. The Superpave process allows one to take into consideration climate and traffic conditions. However, other factors should also be considered during mixture design:

- type and condition of substrate (for example) - is substrate a new flexible base or an old cracked pavement;
- placement in pavement structure - the temperature extremes of a base layer will be significantly less than those of a surface layer;
- past performance of asphalt from the given supplier - what are its shortcomings and/or attributes, and to what extent can additives help; and
- number of tanks contractor has available for storage of different binders.

DISCLAIMER

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

There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

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Mr. Paul E. Krugler served as project director on this study. His valuable advice and assistance in establishing and evaluating the test pavements is hereby gratefully acknowledged.

Laboratory testing at TTI was performed by Sidney Greer, Lee Gustavus, Ann Ferry, Charles Hastings, and Jason Chips.

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SUMMARY

The overall objective of this study was to evaluate a variety of asphalt additives in the laboratory and in the field to determine their merit in reducing premature pavement distress such as cracking and rutting. In the late 1980s, test pavements were constructed in Texarkana, Sherman, San Benito, and Ft. Worth, Texas. In 1995, pavement cores were obtained from Texarkana and Sherman and evaluated in the laboratory. Asphalt binders, retained during construction and sealed in cans, were tested using the Superpave binder tests procedures and certain chemical tests. Retained binders and aggregates were combined in accordance with the original mixture designs, compacted in the laboratory, and tested using the NCHRP AAMAS test protocol to assess relative resistance to fatigue cracking. The specific objective of the work reported herein was to test the binders and mixtures to determine what properties correlate with the observed pavement performance (particularly cracking). A secondary objective was to evaluate the ability of certain laboratory tests to identify binders and mixtures susceptible to cracking in a pavement.

Laboratory tests on binders included viscosity and penetration, dynamic shear rheometer (DSR), bending beam rheometer, binder direct tension test, rolling thin film oven test, pressure aging vessel, Brookfield rheometer, capillary tube viscometer, gel permeation chromatography (GPC), and Fourier transform infrared analysis (FT-IR). Laboratory tests on compacted mixtures included resilient modulus, indirect tension, and Australian materials testing apparatus (frequency sweep).

Asphalt additives included latex, ethylene vinyl acetate, styrene-butadiene styrene-block (SBS) copolymer, SBS block copolymer vulcanized with asphalt, a manganese organic complex in an oil base, finely dispersed polyethylene, and carbon black pelletized using 8% oil as a binder.

For the Texarkana project, the ride quality of all the test pavements was good after 9 years in service. However, the EVA pavement was notably rougher than the other pavements. Rut depths were about the same in all the test pavements and, typically, ranged from 3 mm to 6 mm. Slight raveling in a few isolated areas appeared to be associated with minor aggregate segregation that occurred during construction. There was no significant flushing, patching, or alligator cracking in any of the test pavements. There were, however, major differences in cracking between the test pavements. The control sections exhibited moderate cracking. The EVA section exhibited extensive longitudinal, transverse, and random cracking. The fact that the EVA was added to AC-20 instead of AC-10, like all the other additives, likely contributed to its higher rate of cracking. The Latex surface/Latex base section had the least cracking overall with the Styrelf 13 section showing only slightly more longitudinal cracking. The Latex surface/Latex base test section clearly displayed the best looking pavement surface after 9 years in service. There was significant transverse cracking but hardly any longitudinal cracking in the Chemkrete base/Latex surface section. Based on observations before the surface course was placed, the transverse cracks reflected through the Latex-modified surface mixture from the Chemkrete-modified base. A seal coat was applied to these pavements in the summer of 1996 which, of course,

eliminated further observations of surface distresses which could be compared with the original conditions.

For the project near Sherman, the ride quality of all the test pavements was good and mutually equivalent, after 8 years of service. There was no significant rutting, alligator cracking, or patching. Rutting was about the same in all test sections and was in the range of 3 mm to 5 mm. Significant differences in transverse and longitudinal cracking were observed. These cracks were probably reflective cracks from the underlying cracked continuously reinforced concrete pavement (CRCP). EVA exhibited the highest amount of cracking. When both transverse and longitudinal cracking in the additive sections are compared to that for the 76-mm Control section, it appears that polyethylene, SBS, carbon black, latex, and the thicker control overlay are suppressing crack growth. Polyethylene, with no visible cracking, exhibited superior resistance to reflective and/or fatigue cracking. No evidence of pumping at the cracks was observed. Raveling ranged from none to moderate in all the test sections with only minor differences between sections. A seal coat was applied over these pavements in the summer of 1994 which negated further comparative evaluations of any surface distresses.

In Ft. Worth, test pavements were built in the outermost northbound lane of SH-121 to evaluate SBR-modified hot mixed asphalt and engineering fabric to reduce reflective cracking from the underlying CRCP. After 10 years in service, all 4 test pavements were performing about the same. It is noteworthy that they are performing the same, since binder contents of the different sections vary from 7.2% to 8.5% (the mixture contains light weight aggregate). Two test pavements containing 8.5% asphalt *without* latex failed due to rutting and shoving within a few months after construction. Sections with the lower binder contents exhibited slight raveling; whereas, the high asphalt content sections exhibited no raveling. There were no other significant signs of pavement distress. Rut depths were about 3 mm to 6 mm for all 4 test pavements. The fact that unmodified mixtures containing 8.5% binder rutted to failure very soon after construction, and that similar latex-modified mixtures have not rutted in 10 years, indicates that latex significantly reduced the sensitivity of the asphalt mixture to binder content and resisted permanent deformation. These findings indicate that the addition of a polymer will reduce rutting when HMA is overasphalted or will allow the use of higher binder contents.

The San Benito test pavements provided no meaningful results. After only 4 years in service, they were mostly destroyed by subsequent construction of overpasses across the test pavements.

Laboratory experiments using the materials and test procedures described above revealed the following:

- DSR testing indicates that loss tangent values correlate well with observed cracking. High loss tangent at low testing temperatures indicates good resistance to fatigue cracking.

- DSR test results could not be related to reflective cracking which, apparently, is controlled more by differential movements in the underlying pavement than by the properties of the binders in the overlay.

- Binders which failed the Superpave high-temperature grading did not produce rut-susceptible mixtures, but did produce crack-resistant mixtures. Therefore, SHRP binder test results provided evidence that rutting does not relate to binder properties when high quality aggregate is used.

- Asphaltene content measurements (using heptane) indicate that asphalts with asphaltene contents greater than 12.5% are susceptible to fatigue cracking. Asphaltene content could not be related to reflective cracking.

- FT-IR testing indicates the pavements with highly oxidized binders (carbonyl growth greater than 1.400) are susceptible to fatigue cracking. FT-IR test results could not be related to reflective cracking.

- GPC test results indicate that asphalt binders with high amounts of large molecular size (LMS) material (greater than 22%) are susceptible to fatigue cracking. GPC testing could not be related to reflective cracking.

- AAMAS style testing showed no correlation with the observed fatigue or reflective cracking performance.

- Australian Frequency Sweep testing showed no correlation with the observed fatigue or reflective cracking performance.

INTRODUCTION

Relatively expensive asphalt additives are often indiscriminately used in an attempt to produce better, longer lasting asphalt pavements and this may have been accomplished occasionally. However, proper use of asphalt additives should depend on the particular anticipated pavement problem to be addressed. If the anticipated problem is cracking, and history shows conventional mixtures are rut resistant, then the polymer additive should be incorporated into a softer than usual asphalt. The soft asphalt remains relatively flexible at low temperatures and thus acts to retard thermal, and possibly even fatigue cracking, while the polymer helps protect the mixture from rutting at high temperatures. If the anticipated pavement problem is rutting and low-temperature cracking has not been a problem, then the polymer additive should be incorporated into the asphalt grade normally used in the particular area. The addition of the polymer should not have negative effects on low-temperature cracking but will help offset rutting problems. Serious rutting problems, of course, cannot be solved by modified binders; they must be solved by adjusting aggregate quality or gradation.

Guidelines are needed to help pavement designers intelligently specify binders for asphalt pavements. Superpave is most certainly a move in the right direction; however, factors other than temperature and traffic should be considered when selecting the ideal binder. Results of controlled field experiments such as the ones described herein will help provide the knowledge needed to develop meaningful guidelines.

This project began in 1984 as TxDOT HP&R Study 471. Near the end of that study, several additive test pavements were built near Sherman, Texarkana, and San Benito, Texas. Study 187, Task 5 was initiated to monitor performance of the field trials. Latex test pavements built in Ft. Worth in 1985 as part of another study were added to the test pavement monitoring study in 1991. This is the final report of the field evaluations for the four test sites and a report of findings from the detailed laboratory study attempting to relate material properties to the observed cracking performance. The primary goal of the study reported herein was to investigate binder and mixture properties of materials used in construction of additive test pavements near Texarkana and Sherman in an effort to isolate the causes of significant differences in cracking performance. The overall objective of this study was to evaluate selected asphalt additives as economic alternatives to reduce premature asphalt pavement distress such as cracking and rutting.

The test pavements south of Sherman, installed in 1986 in the southbound right lane of U.S. 75, consist of 76-cm overlays over CRCP and contain Novophalt, DuPont EVA, Ultrapave latex, Shell Kraton SBS, Microfil-8 carbon black, 762-mm Control, and 102-cm Control sections. The test pavements north of Texarkana, installed in 1987 on U.S. 59, consist of a 203-cm base and a 50.8-cm surface course, both of which contain the following additives: Goodyear 5812 latex, Exxon Polybilt 102 EVA, Styrelf-13, Chemkrete, and Control sections. Test pavements at Sherman received seal coats during the summer of 1994 which precluded (at least temporarily) further observation of their relative performance. Test pavements near Texarkana received a seal coat in the summer of 1996.

Test pavements installed on U.S. 83 near San Benito in 1986 were destroyed by subsequent construction of overpasses in the same vicinity in 1989. As a result, very little useful information on relative pavement performance was gained from that test.

Test pavements (overlays over CRCP) were built in 1985 in the outermost northbound lane of SH-121 in Ft. Worth which contain various combinations of latex modified asphalt concrete and Petromat fabric at the interface between the old pavement and the new overlay.

Since these pavements were placed, the SHRP and NCHRP AAMAS research studies have been formulated and completed. It appeared prudent to take the opportunity to test the original binders (which were retained from construction) using these procedures to determine what binder properties best correlate with the observed cracking performance. Since the test pavements are showing wide differences in cracking performance, a natural fallout from this study should be the relative ability of the various laboratory tests to detect differences in binders that caused these differences in pavement performance. If the new SHRP and AAMAS tests function as they are designed, it should be possible to determine, for example, why one binder performed well (i.e., did not crack) and why another performed poorly (i.e., exhibited extensive cracking).

The scope of the work reported herein includes coring of the test pavements near Texarkana and Sherman and subsequent testing of the modified and unmodified mixtures. Original asphalts used during construction of the test pavements were retained at TTI. Mixtures (cores) and binders (retained and extracted) were evaluated to determine (to the extent possible) why the pavements exhibited their relative performances, particularly with regard to cracking. In addition, since the only differences in these mixtures (within each location) were the binders, and since the pavements exhibited significant differences in cracking performance, the resulting data should provide an important evaluation of the binder and mixture tests often touted to be related to pavement performance. None of the pavements at Ft. Worth, Sherman, or Texarkana experienced any significant rutting.

Five research reports describing findings from laboratory and field tests have been prepared during this study. They include Research Report 471-1, "Another Look at Chemkrete" (1), Research Report 471-2F, "Asphalt Additives for Increased Pavement Flexibility" (2), Research Report 187-14, "Asphalt Additives in Highway Construction" (3), Research Report 187-18, "Asphalt Additives in Thick Hot Mixed Asphalt Concrete Pavements" (4), and Report 187-22, "Effect of Asphalt Additives on Pavement Performance" (5).

LITERATURE REVIEW

The use of asphalt additives or modifiers as economic alternatives to improve flexibility, and thus resistance to cracking, in asphalt concrete pavements has been evaluated in several studies (1, 2, 3, 4, 5). There are a wide variety of tests that can be performed to predict performance of these pavements; below is a review of some of the current tests.

DYNAMIC SHEAR RHEOMETER ANALYSIS

In previous work (6), correlations were established between dynamic shear rheometer data and the performance of the binders in fatigue experiments. The viscoelastic data showed a good correlation between the loss tangent (ratio of viscous to elastic moduli of an asphalt) and its flexural fatigue life in asphalt concrete (7).

The proper balance of viscoelastic properties in the binder which is necessary for good mixture performance is shown to exist naturally in some asphalts. Data are shown (6) which suggests that the proper balance occurs when an effective elastic network is created by natural molecular associations. The elastic network may also be created by introducing molecular entanglement in asphalt through the use of high molecular weight polymeric additives.

INDIRECT TENSILE TEST AND RESILIENT MODULUS TEST

Fatigue cracking has been related to data obtained from the resilient modulus test and the indirect tensile test through the asphalt aggregate mixture analysis system (AAMAS) (8). If the total resilient modulus and indirect tensile strains at failure for a particular mixture plot above the standard mixture (Federal Highway Administration fatigue curve is recommended), it is assumed that the mixture has better fatigue resistance than the standard mixture (8).

GEL PERMEATION CHROMATOGRAPHY

Gel permeation chromatography (GPC) has been used by numerous researchers to investigate the asphalt fractions contributing to pavement performance (9-16). Goodrich and others (10) have pointed out its limitations and doubted it would ever provide sufficient information to identify good and bad asphalts. Presently, it does have limitations, but still it has shown about as much accuracy as any alternative asphalt test in predicting asphalt mixture cracking (9). GPC gives a rough representation of molecular size distribution but tells nothing directly about molecular structure. The fact that GPC has correlated with asphalt performance at all is probably because of widespread similarity in molecular species in many asphalts. Some asphalts whose performance is better or worse than might be predicted from the GPC have indeed been shown to be structurally anomalous at the molecular level.

Perhaps the most extensive studies of GPC in predicting asphalt pavement performance have been those of Jennings and co-workers (11, 12, 13, 14, 15). Jennings (11) took cores from 39 roads in Montana representing asphalts from all of Montana's four refiners including five penetration grades representing a variety of pavement ages. Jennings found a high degree of correlation between pavement cracking performance and the similarity of the asphalt's GPC to that of the standard, particularly in the large molecular size (LMS) region. He determined the low LMS is desirable to reduce cracking (16).

FOURIER TRANSFORM INFRARED ANALYSIS

Infrared analysis is an extremely valuable tool for investigating the chemical functionality of asphalts. The Fourier transform infrared (FT-IR) method, using attenuated total reflectance, is especially useful for the study of this opaque material. Quick and easy sample preparation, natural state analysis, and clean reproducible spectra are advantages of the FT-IR method over conventional transmission methods (17).

Atmospheric oxidation of certain asphalt molecules with the formation of highly polar and strongly interacting chemical functional groups containing oxygen is the principal cause of age hardening and embrittlement of asphalt used in pavements (18, 19, 20, 21, 22, 23). Thus, the ability to identify and quantify asphalt chemical functionality provides an important tool for assessing the effects of composition on asphalt properties and, thus, the performance of the asphalt in service (24).

SHRP BINDER TESTS

One of the primary products of the Strategic Highway Research Program (SHRP) was a set of performance-related specifications for asphalt cement binders and asphalt concrete mixtures, collectively referred to as Superpave™. The asphalt cement specifications currently in use in North America for asphalt cement are typically based on measurements of viscosity, penetration, ductility, and softening point temperature. These measurements are not sufficient to properly describe the linear viscoelastic and failure properties of asphalt cement that are needed to relate asphalt binder properties to mixture properties and to pavement performance (25).

Dynamic Shear Rheometer

The dynamic shear rheometer is used to measure the rheological properties of the binder in terms of dynamic shear modulus (stiffness), G^*, and the phase angle, δ. In the SHRP binder specification, the parameter $G^* \sin \delta$ relates to fatigue cracking and $G^* / \sin \delta$ relates to permanent deformation (26).

Bending Beam Rheometer

The bending beam rheometer is used to measure the creep stiffness, S, of the asphalt at low temperatures and the slope of the creep stiffness versus loading time curve, "m." In the SHRP binder specification, both of these values relate to low temperature cracking, and "m" is also related to fatigue cracking (26).

Direct Tension Test

The direct tension test is used to measure the low temperature failure properties of the binder. The failure strain at break is used as an indicator of the performance of mixes in cold environments (26).

Artificial Accelerated Aging

The rolling thin film oven test (RTFOT) has been selected as the preferred method to represent binder aging during the construction process, plant aging or short-term aging. Permanent deformation is evaluated in Superpave using RTFOT aged binders. Fatigue and low temperature cracking are evaluated using binders which have been subjected to long-term oxidative aging using the pressure aging vessel (PAV) (26).

SUMMARY OF FIELD TRIALS

References 3 and 4 provide details of the structural and mixture designs, construction, traffic, and environment of the test pavements. Table 1 and the following subsections give a brief summary of these details.

Table 1. Summary of Texarkana and Sherman Test Pavements.

District	Atlanta	Paris
Location	Texarkana U.S. 59	Sherman U.S. 75
Date Installed	1987	1986
No. of Additives	4	5
Layer Thickness, mm	203 mm[1] + 51 mm[2]	76 mm
Type Hot Mix	B[1] + D[2]	C
Type Construction	Reconstruction	Overlay/CRCP
ADT @Construction	11K	23K
Percent Trucks	15%	17%
Expected ESALs in 20 years, millions	5.67	21.4

[1]Base
[2]Surface

TEXARKANA

A 9-km highway construction project MA-F 472 (3) composed of designated test pavements containing asphalt additives was built in the Atlanta District on U.S. 59/71 north of Texarkana in 1987 and 1988. The project is located in Bowie County from 2.9 km north of IH-30 to 1.3 km south of the Red River. This is a fairly flat rural alluvial area in the Red River bottom. The decision to build test pavements on the construction project was made prior to letting out the project for bids. The project consisted of reconstruction of the existing 2-lane pavement and construction of 2 adjacent lanes to provide a 4-lane divided facility. Two test pavements and a control pavement were built in the northbound and the southbound lanes. A map showing the layout of the 6 pavement sections is shown in Figure

A1, Appendix A. The 1.4-km (approximately) test pavements consist of 203 mm of Item 340, Type B (22-mm nominal maximum size) and 51 mm of Item 340, Type D (9.5-mm nominal maximum size) asphalt concrete placed on a 457-mm lime-flyash treated subgrade that had been sealed with an MC-30 prime coat. The Type B mix was placed in 3 lifts. The test pavements were in both lanes. Specific information about these test pavements is furnished in Tables 2 and A1. Details of the materials used, construction process, traffic, and climate are provided in Reference 4 as well as Tables A1-A3, Appendix A.

Table 2. Summary of Materials Used in Texarkana Test Pavements.

Asphalt Paving Mixtures	Base Course	Surface Course
Asphalt Source		
Ultrapave Latex	Fina AC-10	Fina AC-10
Chemkrete-CTI 102	MacMillan AC-20	Fina AC-10+latex*
Polybilt 102 EVA	Lyon AC-20	Lyon AC-20
Styrelf	Exxon	Exxon
Control (no additive)	MacMillan AC-20	MacMillan AC-20
Quantity Additive in Asphalt Cement		
Ultrapave Latex	3.0%	3.0%
Chemkrete	2.0%	3.0% latex*
Polybilt EVA	3.5%	3.5%
Styrelf	3.0%	3.0%
Control	0	0
Aggregate Types	Crushed sandstone + flume sand	

* Chemkrete was replaced with latex in the surface mixture.

Construction (preparation of the subgrade) of the northbound lanes adjacent to the existing highway began in the fall of 1986. The 203-mm asphalt treated base course was placed in the summer of 1987 and the northbound lanes were turned over to traffic. Reconstruction of the existing 2-lane highway (which became the southbound lanes) began in the summer of 1987 and the asphalt treated base course was placed in the fall of 1987. Traffic used these “interim” pavements until the spring of 1988 when the 51-mm surface courses were placed in both the northbound and the southbound lanes.

The following 4 asphalt additives were used in Texarkana:

- Goodyear 5812 - styrene butadiene rubber (SBR) latex, supplied by Fina;
- Exxon Polybilt 102 - ethylene vinyl acetate (EVA), supplied by Exxon;
- Styrelf 13 - neat synthetic SBS block copolymer vulcanized with asphalt; supplied by Elf Asphalt; and
- Chemkrete (CTI-102) - a manganese organic complex in an oil base, supplied by LBD.

The names given to these products are trademarks registered by their suppliers. Both the Type B (base) and Type D (surface) mixes were treated with the same additive in a given test section, with one exception. Chemkrete was removed from the market by LBD shortly after construction of the base course. The surface course placed on the Chemkrete treated base course contained 3% Goodyear latex in the asphalt.

Chemkrete was metered into an in-line mixer and blended with the asphalt at the plant site using a special device furnished by LBD Asphalt Products Company. Polybilt was blended at the plant site in a batch-type operation using a low-shear mixer. Styrelf and Goodyear 5812 were blended with asphalt prior to arrival at the asphalt plant. Mixing and placing of the modified mixtures was generally routine and without any additive related problems. Minor exceptions are listed in the notes pertaining to highway construction using these products (Table A3).

All pavements at Texarkana contained the same aggregates and used basically the same mixture design and construction equipment and procedures. The two control pavements (northbound lanes and southbound lanes) contained MacMillan AC-20. The additive test pavements contained asphalt of various grades from various sources as shown in Table A1. This is not an ideal situation for comparative evaluation of additives, but it was necessary for the contractor to expedite construction of the experimental pavements.

Asphalt paving operations were performed by HMB Construction Company of Texarkana, Texas. The mixtures were prepared using a model 8828 ADM Cedar Rapids plant with a capacity of 400 tons per hour. Placing of the mixes was accomplished using a BSF541 Cedar Rapids paving machine. A 30-ton pneumatic tired compactor was employed as a breakdown roller followed by an 11-ton steel wheel vibratory roller.

SHERMAN

Five asphalt additive test pavements and 2 control test pavements (no additive) were built in 1986 about 5.2-km south of Sherman, Texas on U.S. 75 as part of construction project CSR 47-13-11. The highway is a 4-lane divided controlled access facility. The existing pavement consisted of a transversely cracked and deteriorating 200-mm continuously reinforced portland cement concrete pavement (CRCP). Six of the seven 0.8-km (approximately) test pavements consisted of 76 mm of Item 340, Type C (16-mm nominal maximum size) hot mix asphalt overlays placed in 1 lift as the surface course for rehabilitation of CRCP. One control section was placed as a 100-mm layer in an attempt to determine whether it is more cost effective to place thicker conventional HMA or to use an asphalt additive.

A map showing the layout of the 6 pavement sections is shown in Figure A2, Appendix A. Specific information about these test pavements is furnished in Table A4. Climatic and traffic data are included in Table A5 to indicate the types of environments to which these pavements were exposed.

Five types of additives which appeared likely to improve resistance to rutting and cracking were selected for the study:

- Novophalt (5%)- finely dispersed polyethylene (PE);
- DuPont Elvax 150 (2%) - ethylene vinyl acetate (EVA);
- Ultrapave latex (3%) - styrene butadiene rubber (SBR) latex;
- Kraton D4460X (8.6%*) - styrene-butadiene styrene-block copolymer (SBS); and
- Microfil-8 (12.5%)- carbon black pelletized using 8% oil as a binder.

To the best of the authors' knowledge there is presently only one carbon black product produced specifically for asphalt modification, Microfil-8, supplied by Cabot Corporation. Microfil-8 is a mixture of approximately 92% high-structure HAF grade carbon black plus approximately 8% oil similar to the maltenes portion of asphalts, formed into soft pellets dispersible in asphalt.

Styrene-butadiene latexes are available with a wide variety of monomer proportions, molecular weight ranges, emulsifier types, and other variables. The product selected for use in this field investigation was Ultrapave 70 from Textile Rubber and Chemical Co. It is an anionic emulsion and contains about 70% rubber solids and 30% water.

Thermoplastic block copolymer rubber was supplied from Shell Development Company. Kraton D4460X was composed of 50% SBS + 50% extender oil. Blending difficulties necessitated the inclusion of about 15% Exxon 120/150 pen asphalt in the modified binder. The oil added was a blend of aromatic and naphthenic/paraffinic oils designed to facilitate incorporation of the polymer into asphalt and improve the properties of the final modified product.

Polyethylene was supplied and blended with asphalt by the Novophalt America Corporation. Information on the Novophalt process indicated that normally low density polyethylene (LDPE) is used. Preparation of Novophalt (asphalt and polyethylene) involves a high shear blending process which breaks down the polyethylene into very fine particles, stores the blend in a heat-controlled, agitated tank, and transfers the material directly to the mixing plant. Polyethylene is a linear nonpolar polymer.

Ethylene vinyl acetate (EVA) resin was marketed as Elvax 150 from DuPont Company. EVA has permanent polarity associated with the acetate functionality.

All additives at Sherman were mixed with Total AC-10; whereas, the control mixtures were produced using Total AC-20. All binders contained 0.5% PaveBond LP liquid antistrip additive. Similar aggregates, a blend of crushed limestone and field sand, were used in all test pavements.

Before placing the test pavement layers, the CRCP was treated with a seal coat (under seal) plus a 50-mm lift of Type B HMA level-up course. The test pavements were placed only in the outside (travel) lanes. Details of the materials used, construction process, traffic, and climate are provided in Reference 3 as well as Tables A4 and A5.

FORT WORTH

Test pavements were built in the outermost northbound lane of SH-121 from IH-35 to IH-820 in Fort Worth in June of 1985 to evaluate Dow latex (SBR) modified hot mixed asphalt and engineering fabric to reduce reflective cracking (5). SH-121 is a high traffic volume 6-lane facility. The existing 18 year old CRCP structure contained typical transverse cracks with a spacing from 1.8 m to 3 m. Five 150-m test pavements consisting of 50 mm of asphalt concrete were originally placed as shown in Table 3. A map of the test pavements is shown in Appendix A, Figure A3.

Table 3. Description of Test Pavements Installed at Fort Worth in 1985.

Test Section	Percent Binder	Latex	Fabric
1	8.5	Yes	Yes
2	8.5	Yes	No
3*	8.5	No	No
4*	8.5	No	Yes
5	7.5	Yes	Yes
6*	7.2	No	No

*Test pavements 3 and 4 failed within two weeks after construction due to rutting, shoving, and flushing, and were subsequently replaced. The resulting new 305-m test section was designated section 6.

AC-10 from the Kerr-McGee refinery at Wynnewood, Oklahoma was used in the test sections numbered 1 through 5. AC-20 from the Texaco refinery in Port Arthur, Texas was used in test section number 6. Dow Chemical Company supplied the latex. PaveBond antistrip was used in all the paving mixtures for this project. Synthetic light weight aggregate (Texas Industries), field sand, and limestone screenings were combined to produce the desired aggregate gradation. The test sections are located on either side of the Haltom

Road bridge as shown in Figure A3, Appendix A. Traffic volume, in terms of 2-directional ADT for the 6-lane facility, is about 63,000. The expected total number of 80,000 Newton axle loads during the 20-year period after construction is 40 million. Details of pavement materials, construction processes, traffic, climate, and early performance are given in Reference 5.

SAN BENITO

In August 1986, during construction of Project MA-F-93(40) on US-83/77 in Cameron County, a 4.2-km segment of the project was used to evaluate 4 asphalt additives (3).

- Exxon Polybilt (EVA),
- Ultrapave latex (SBR),
- Kraton D4460X (SBS), and
- Microfil-8 (pelletized carbon black).

The work consisted of new construction. To ensure statistical validity, two 0.4-km test pavements 76 mm thick containing each additive, and similar control sections with no additive were built. In addition, a one 0.4-km control section 102 mm thick was installed. A total of 11 pavement sections were built for the experiment.

Due to subsequent construction of overpasses in the same vicinity in 1990, 7 of the test pavements in the middle of the experimental area were totally or partially destroyed. Furthermore, the northernmost surviving test pavements received many more heavy loads during construction of the overpasses than the southernmost surviving test pavements. For these reasons these test sections were eliminated from further study in 1992. A map showing the layout of the experimental sections and those destroyed is provided in Appendix A, Figure A4.

FINDINGS

OBSERVED PAVEMENT PERFORMANCE

Texarkana Test Pavements

After 9 years in service, the ride quality of all the test pavements in Texarkana was good; however, the EVA pavement was notably rougher than the other pavements. This was a subjective evaluation since no objective measures of ride quality were made. Rut depths were about the same in all the test pavements and, typically, ranged from 3 mm to 6 mm. On the average, the Chemkrete/Latex section exhibited slightly less rutting (2 mm to 3 mm) than all the others. This was probably due to the harder binder in the Chemkrete-modified base layer. Slight raveling in a few isolated areas appeared to be associated with minor aggregate segregation that occurred during construction. There was no significant flushing, patching, or alligator cracking in any of the test pavements. There were, however, major differences in cracking between the test pavements as shown in Figures 1 and 2.

During the early years of the evaluations, cracking was recorded as longitudinal or transverse. After 7 or 8 years, as the cracking progressed, it became increasingly more difficult to distinguish longitudinal and transverse cracking from random cracking. Nevertheless, researchers categorized the random cracks, as well as possible, as either transverse or longitudinal cracks to facilitate plotting for comparative analyses. After 9 years, the control sections exhibited moderate cracking. The Polybilt (EVA) section exhibited extensive longitudinal, transverse, and random cracking. The fact that the Polybilt was added to AC-20 instead of AC-10, like all the other additives, may have contributed to its higher rate of cracking. The Latex/Latex section had the least cracking overall with the Styrelf 13 section showing slightly more longitudinal cracking. The Latex/Latex test section clearly displayed the best looking pavement surface after 9 years in service. Researchers observed only minor, isolated pumping at a few cracks in the northbound Control section, even though rainfall had occurred a few days before the last 2 visual evaluations.

There was significant transverse cracking, but hardly any longitudinal cracking in the Chemkrete/Latex section. Based on observations early in the study, the transverse cracks had reflected through the Latex-modified surface mixture from the Chemkrete-modified base. After one winter of carrying traffic before the surface layer was placed, the 200-mm Chemkrete base layer contained extensive transverse cracks at an average spacing of about 3 m. Since Chemkrete was temporarily pulled from the U.S. market at that time, a surface course containing latex was applied over the Chemkrete. Although no direct comparisons are available, the latex surface course appears to have been reasonably successful in retarding reflection of the transverse cracks.

A seal coat was applied to these pavements in the summer of 1996 which, of course, eliminated further observations of surface distresses which could be compared with the original conditions.

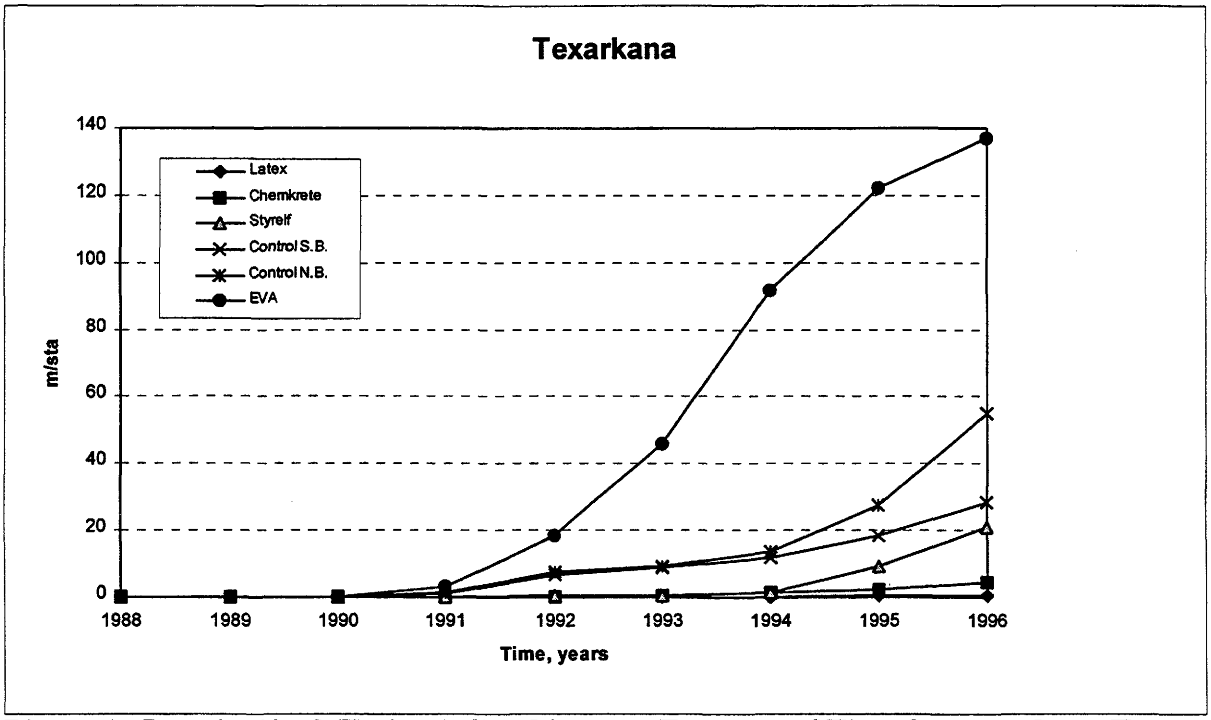

Figure 1. Longitudinal (Fatigue) Cracking as a Function of Time for Texarkana Test Pavements.

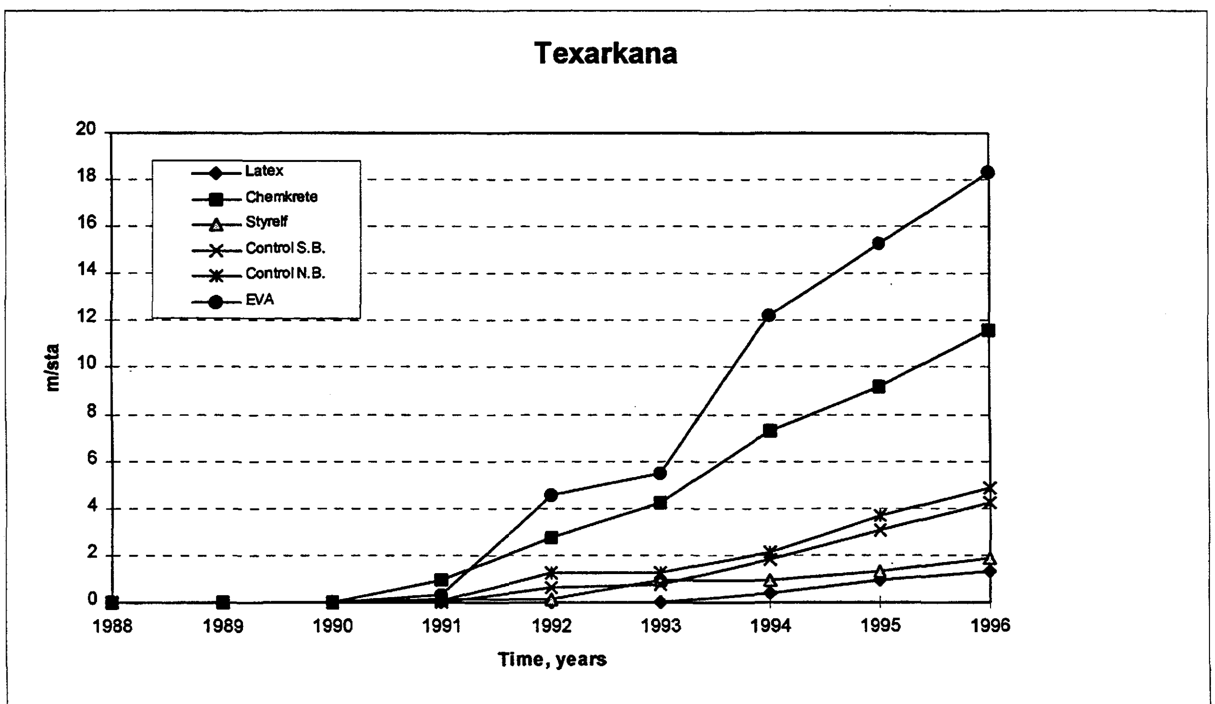

Figure 2. Transverse Cracking as a Function of Time for Texarkana Test Pavements.

Sherman Test Pavements

In 1994, after 8 years of service, the ride quality of all the test pavements near Sherman was good and essentially equivalent. There was no significant rutting, alligator cracking, or patching. Rutting, measured using a 1.2-m straight edge, was about the same in all test sections and was in the range of 3 mm to 5 mm.

Significant differences in transverse and longitudinal cracking were observed. These cracks were probably reflective cracks from the underlying cracked CRCP. The relative severity of these cracks is depicted in Figures 3 and 4. DuPont EVA currently exhibits the highest amount of cracking. When both transverse and longitudinal cracking in the additive sections are compared to that for the 76-mm Control section, it appears that Novophalt, Kraton, Microfil, SBR latex, and the thicker overlay (102-mm Control) are suppressing crack growth. Novophalt, with no cracking, is showing superior resistance to reflective and fatigue cracking. No evidence of pumping at the cracks was observed.

Raveling ranged from none to moderate in all the test sections, with only minor differences between sections. The Novophalt section exhibited the least raveling, and the Carbon Black, Kraton, and the 76-mm Control sections exhibited the most raveling. In 1994, the conventional mixture in the passing lane (with less traffic) adjacent to the test pavements exhibited significantly more raveling than any of the test pavements. Raveling often seemed associated with longitudinal cracks and, further, longitudinal cracks often appeared associated with a "gear-box streak" caused by the paver during construction. Researchers observed an almost continuous narrow band (about 100 mm) of coarse aggregate near the center of the lane containing the test pavements. The longitudinal cracks are almost always near the center of that lane. (This phenomenon is actually not segregation. It is caused by an insufficient supply of HMA at the center of the main screed. During construction, this results in a low area which thus receives comparatively poor compaction [resulting in high voids] and is highly susceptible to raveling. When the fines are eroded away due to raveling, it appears as a coarser textured band which appears to be segregation.)

A seal coat was applied over these pavements in the summer of 1994. This major maintenance negated further comparative evaluations of any surface distresses.

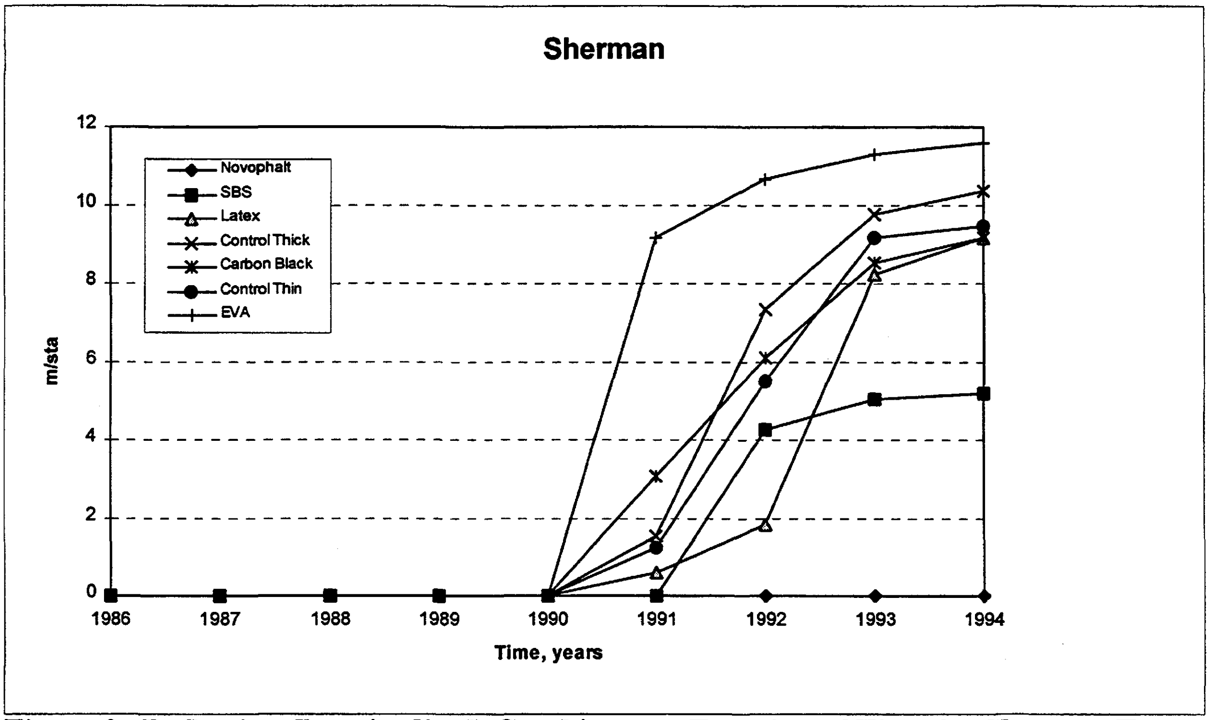

Figure 3. Reflective (Longitudinal) Cracking as a Function of Time for Sherman Test Pavements.

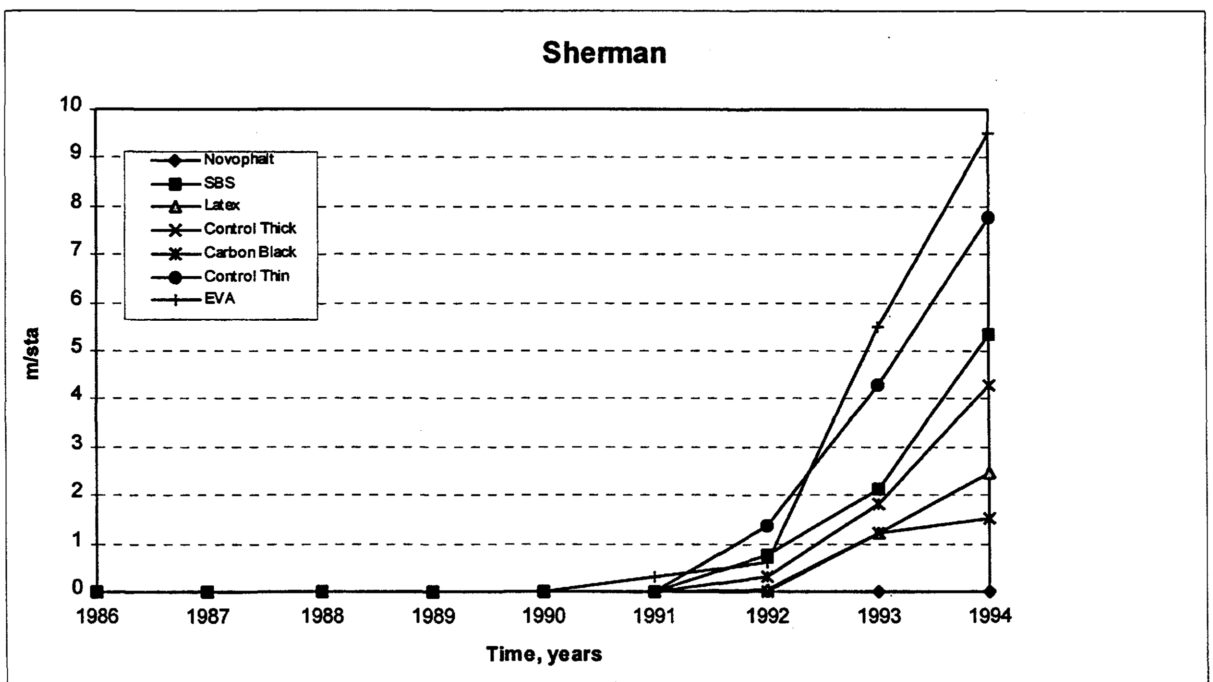

Figure 4. Reflective (Transverse) Cracking as a Function of Time for Sherman Test Pavements.

Ft. Worth Test Pavements

In 1996, visual evaluations of the test pavements in Ft. Worth indicated all four test pavements are performing similarly. This is noteworthy because binder contents of the different sections vary from 7.2% to 8.5% (note: the mixture contains light weight aggregate). The pavements containing 8.5% asphalt *without* latex (test sections 3 and 4) failed due to rutting and shoving and were replaced within a few months after construction. Sections 5 and 6 with the lower binder contents exhibited slight raveling; whereas, the high asphalt content sections exhibited no raveling. There were no other significant signs of pavement distress. Rut depths were about 3 to 6 mm for all four test pavements. There was an almost continuous crack at the construction joint between the test section pavements and the paved shoulder. Those sections containing fabric exhibited slightly less cracking at the joint between the test pavement and the shoulder.

The fact that unmodified mixtures containing 8.5% binder rutted to failure very soon after construction, and that similar latex-modified mixtures have not rutted in 10 years, indicates that latex significantly reduced the sensitivity of the asphalt mixture to binder content. Laboratory work by Little (27) indicates that Novophalt-modified mixtures are less sensitive to binder content than similar mixtures without polyethylene. These findings indicate that the addition of a polymer will reduce dire consequences when HMA is overasphalted or will allow the use of higher binder contents.

San Benito Test Pavements

In the fall of 1989, just before most of the test pavements were destroyed by subsequent construction, all pavements in the experiment were performing identically.

Table 4 shows the relative performance of the surviving test pavements in 1992. There was no rutting, flushing, or alligator cracking, and only slight raveling in the surviving test pavements. The Control sections and the Carbon Black section were demonstrating approximately equivalent performance; whereas, the Kraton section was exhibiting significantly more severe longitudinal cracking. These cracks were up to 50 mm wide and more than 100 mm deep in some places. These deep, wide cracks were apparently a result of poor base or subgrade preparation in this area and had little to do with the presence or type of additive in the 76-mm surface course of asphalt concrete.

Table 4. Performance Data for Surviving Test Pavements Near San Benito - 1992.

Section	Rut Depth mm	Longitudinal Cracking, m	Transverse Cracking, m	Raveling
Control 76-mm	< 3	82	0	Slight
Control 102-mm	< 3	64	0	Slight
Carbon Black	< 3	18	0	Slight
Kraton	< 3	137	3	Slight
Control 76-mm	< 3	0	0	Slight

LABORATORY TESTING

Neat Binder Tests

The primary thrust of this task is a detailed characterization of the neat binders retained at the Texas Transportation Institute (TTI) from the construction projects in Texarkana and Sherman.

Viscosity and Penetration

Capillary viscosity @ 60°C and 135°C and penetration @ 25°C tests were performed on the binders at the beginning of the study in 1987-88. These tests were repeated in this study to determine if any significant changes had occurred in the retained materials and if these changes are related to the modifier. Results from these tests are shown in Tables 5 and 6 as well as Figures 5-10.

Figure 5 shows the change in penetration values for Texarkana test binders. There is relatively little change in the penetration values with the exception of Styrelf which changed from 90 dmm to 63 dmm. Figures 6 and 7 show that the largest changes in viscosity @ 60°C and 135°C for the Texarkana binders occurred in Goodyear 5812 (latex) and Styrelf, both of which increased in viscosity due to age hardening of the binders retained for 9 years. As shown in Figures 5, 6, and 7, Latex and Styrelf consistently exhibit the most hardening. These 2 additives are chemically similar species.

Figure 8 shows the change in penetration values for the Sherman test binders. As expected, the penetration values for the retained binders are generally lower with the exception of Carbon Black which exhibited an increase from 38 dmm to 52 dmm. Accurate measurements of penetration of 2-phase systems such as Carbon Black are difficult because carbon black particles, with a specific gravity of about 2.2, will settle rapidly in static, liquid asphalt. This settlement during sample preparation yields erroneously high penetration values. Differential settlement between specimens may have contributed to this anomalous data. Novophalt, with specific gravity of about 0.9, rises in static, hot asphalt and will also have adverse effects on accurate penetration measurements.

Figure 9 shows a comparatively large change in viscosity @ 60°C in the EVA binder, while Figure 10 depicts a comparatively small change in viscosity @ 135°C in the Control binder.

Table 5. Standard Penetration and Viscosity Values for Texarkana Test Binders.

Test Property	Control MacMillan AC-20	Chemkrete + MacMillan	Lyon AC-20	EVA Polybilt + Lyon	Fina AC-10	Latex Goodyear 5812 + Fina	Styrelf Exxon
	Type of Binder						
Penetration, ASTM D5							
Original Tank Binder:							
@25°C, 100gm, 5s	86	118	64	48	90	83	90
Retained Tank Binder:							
@25°C, 100gm, 5s	83	111	-	47	-	70	63
Viscosity, ASTM D2171							
Original Tank Binder:							
@60°C, Pa-s	221	128	201	257	82.5	177	235
@135°C, Pa-s	0.446	0.469	0.409	0.619	0.257	0.940	0.595
Retained Tank Binder:							
@60°C, Pa-s	221	146	-	277	-	546	476
@135°C, Pa-s	0.437	4.019	-	0.679	-	1.47	0.885

Table 6. Standard Penetration and Viscosity Values for Sherman Test Binders.

Test Property	Type of Binder						
	Control AC-20	AC-10	Carbon Black + AC-10[3]	EVA + AC-10	Novophalt + AC-10	Latex + AC-10	SBS + AC-10[2]
Penetration, ASTM D5							
Original Tank Binder:							
@25°C, 100gm, 5s	92	114	38	127	86	137	115
Retained Tank Binder:							
@25°C, 100gm, 5s	74	102	52	119	78	113	115
Viscosity, ASTM D2171							
Original Tank Binder:							
@60°C, Pa-s	173	100	10000[1]	123	451	101	225
@135°C, Pa-s	0.461	0.339	-[4]	0.449	10.9	0.414	0.513
Retained Tank Binder:							
@60°C, Pa-s	202	109	-[4]	298	459	123	214
@135°C, Pa-s	0.482	0.386	-[4]	0.545	1.31	0.536	0.686

[1]Viscosity measured using Brookfield viscometer.
[2]Also contains some Exxon 120/150 grade asphalt. See text.
[3]Blended in the TTI laboratory using low shear desk top mixer.
[4]Not available due to clogging of capillary tubes by settlement of carbon black.

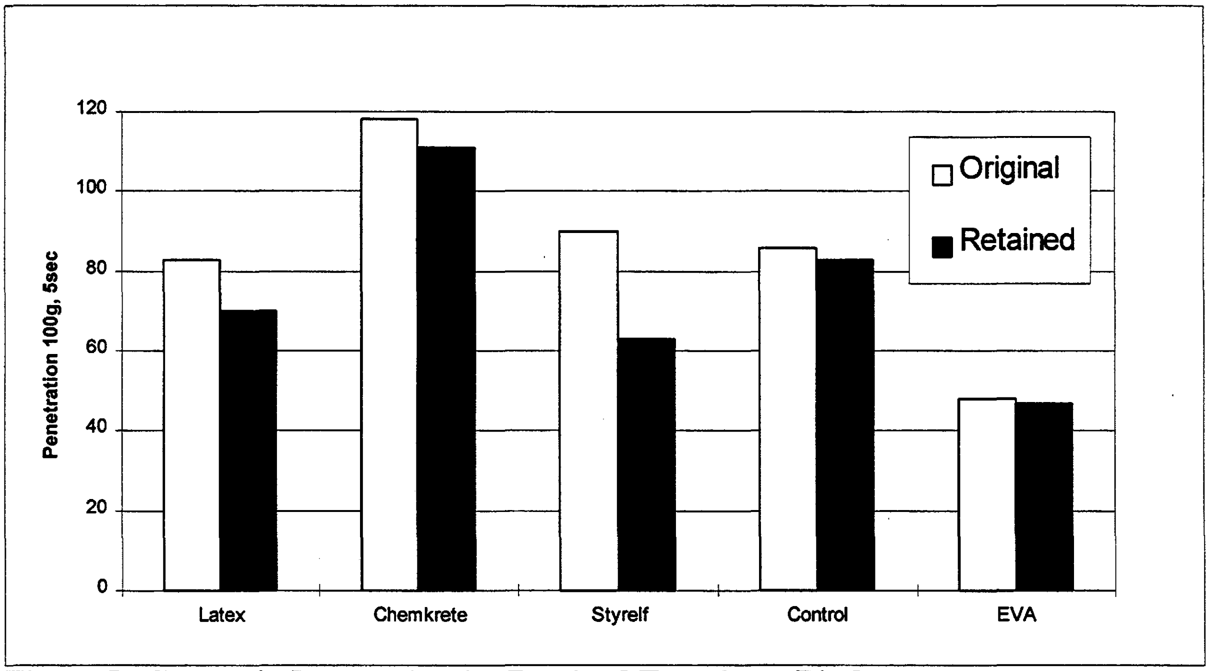

Figure 5. Change in Penetration for Retained Texarkana Binders.

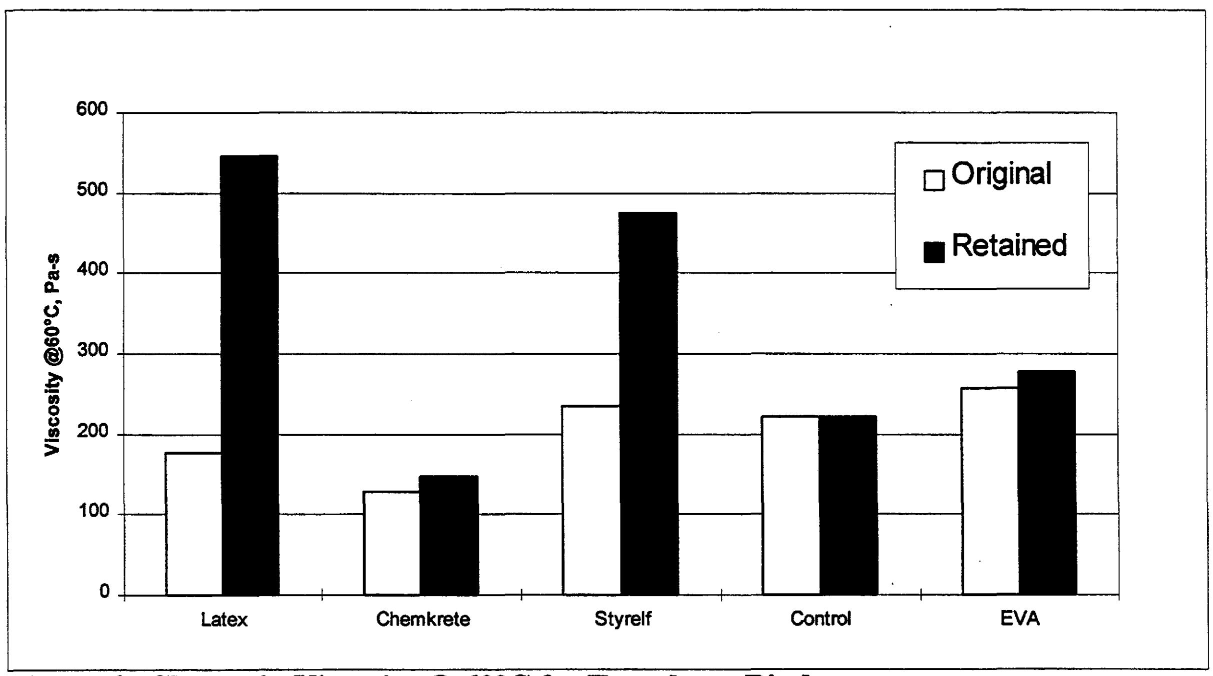

Figure 6. Change in Viscosity @ 60°C for Texarkana Binders.

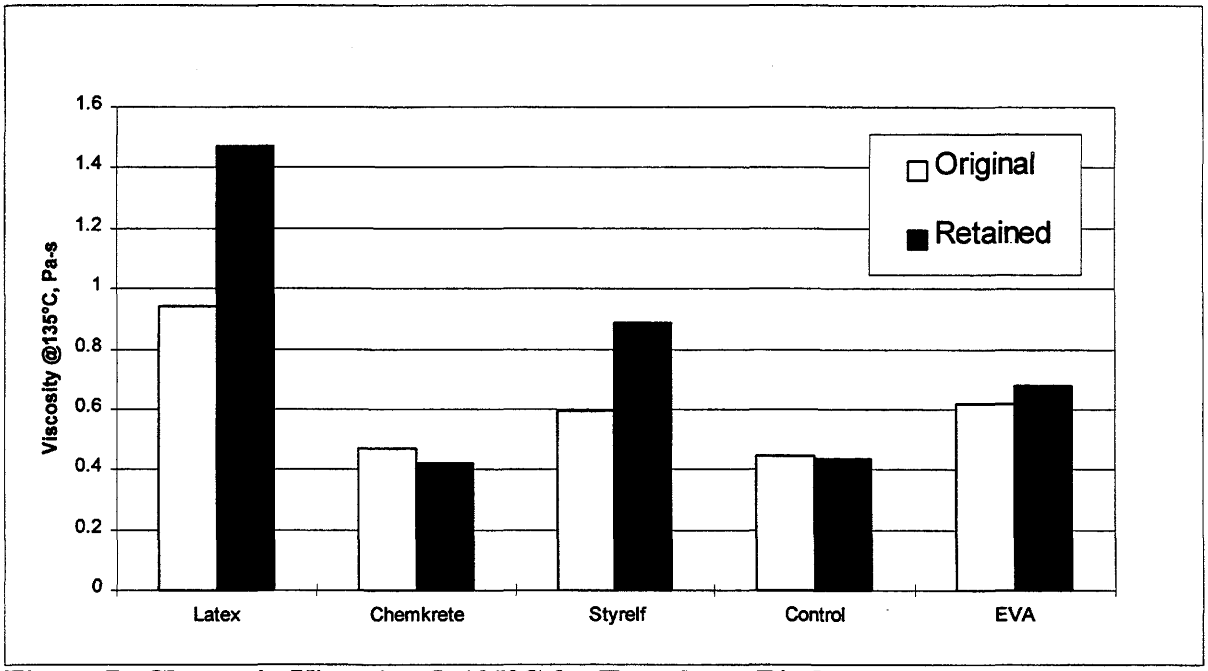

Figure 7. Change in Viscosity @ 135°C for Texarkana Binders.

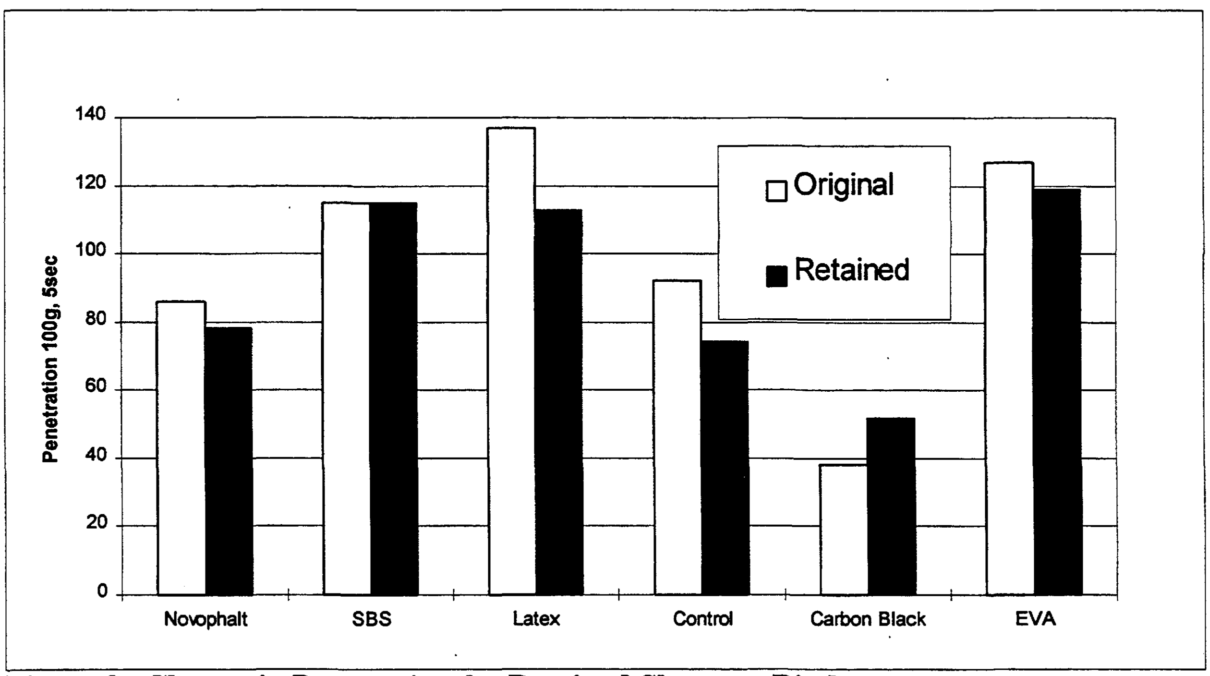

Figure 8. Change in Penetration for Retained Sherman Binders.

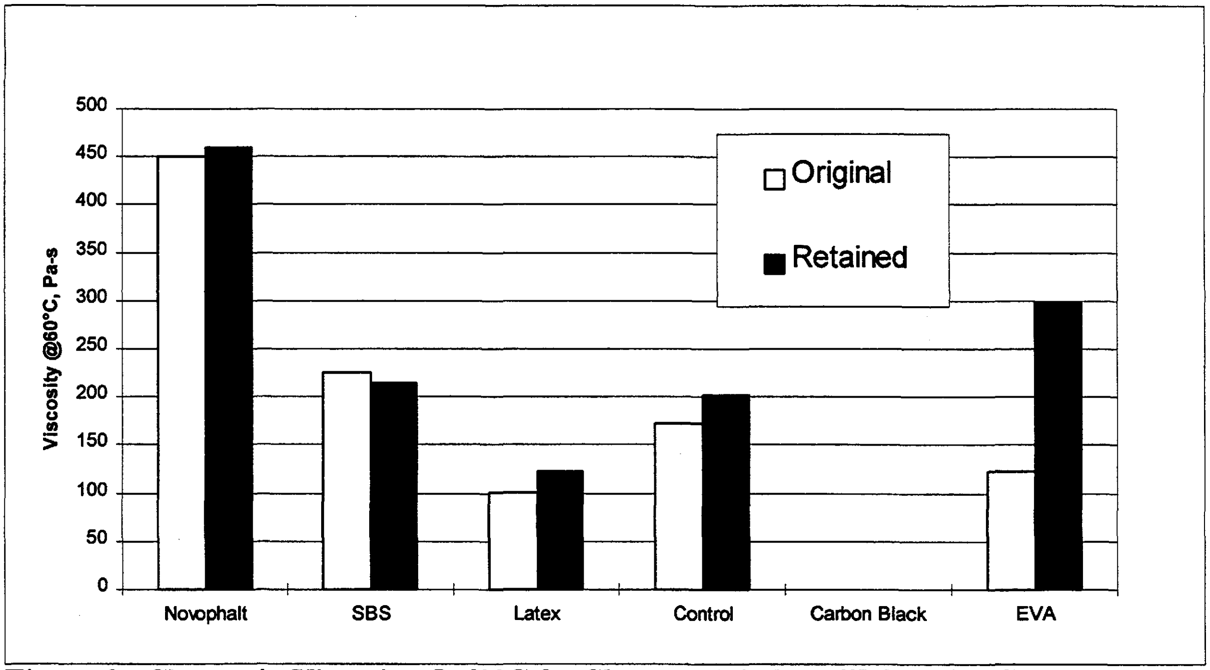

Figure 9. Change in Viscosity @ 60°C for Sherman Binders (Values for Carbon Black could not be obtained due to clogging of capillary tubes).

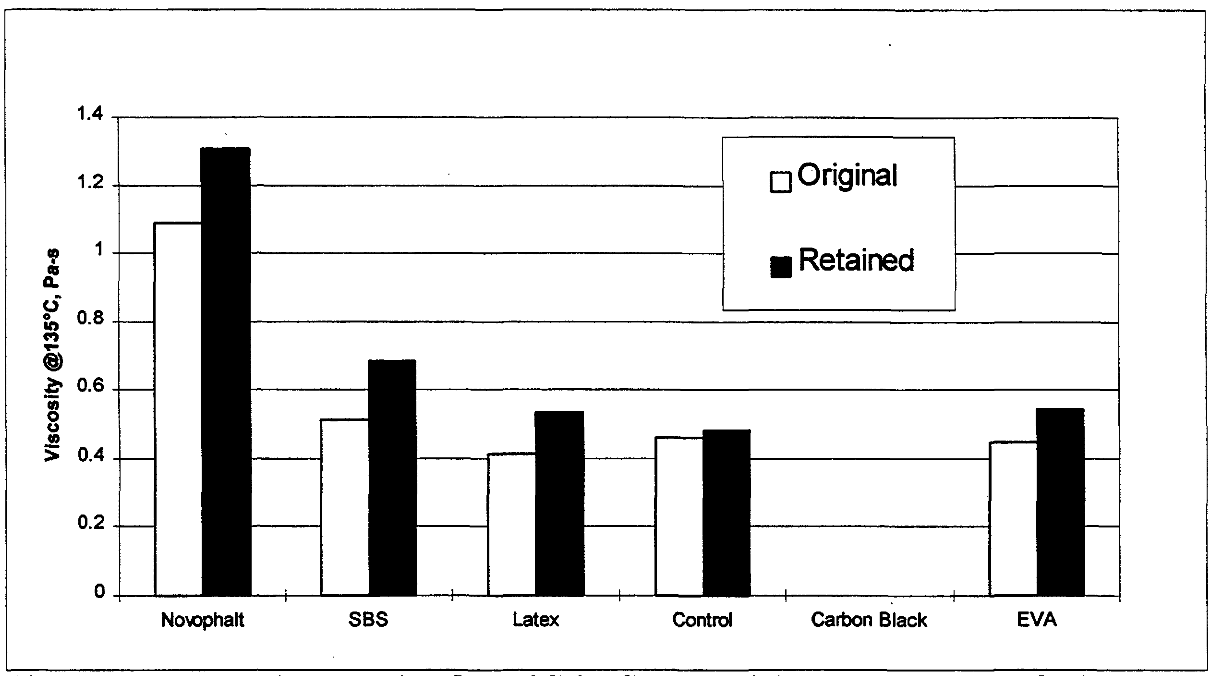

Figure 10. Change in Viscosity @ 135°C for Sherman Binders (Values for Carbon Black could not be obtained due to clogging of capillary tubes).

Dynamic Shear Rheometer (DSR)

A series of DSR tests on pressure aging vessel (PAV) conditioned asphalt binder specimens included those specified by Strategic Highway Research Program (SHRP) found in the next section, but also involved a frequency sweep at selected temperatures to permit calculation of additional rheological parameters. The rheological parameter of importance in this study is the loss tangent (ratio of viscous to elastic moduli of an asphalt). Generally, a higher loss tangent is indicative of better resistance to cracking.

The proper balance of viscoelastic properties in the binder which is necessary for good mixture performance is shown to exist naturally in some asphalts. Goodrich (6) has shown data which suggests that the proper balance occurs when an effective elastic network is created by natural, molecular associations. The elastic network may also be created by introducing molecular entanglement in asphalt through the use of high molecular weight polymeric additives.

Goodrich (6) has also established correlations between DSR data and performance of binders in fatigue experiments. The viscoelastic data showed a good correlation between the loss tangent and its flexural fatigue life in asphalt concrete (7). Similar conclusions have been drawn from DSR tests on the Texarkana binders, but not the Sherman binders.

Table 7 shows the values for the loss tangent of the Texarkana and Sherman binders after TFOT/PAV for temperatures ranging from 10°C to 70°C and a frequency equivalent to 10 radians/sec (1.59 Hz). The 8-mm parallel disk configuration was used for all asphalt binder tests, stresses were kept within the linear viscoelastic region as indicated by stress sweeps, and asphalt specimen thickness between the parallel plates was 2 mm. Some values of the loss tangent at 70°C for the Texarkana binders appear erroneous; a larger diameter disk was probably needed for testing at this higher temperature.

Figure 11 depicts the loss tangent as a function of temperature for the Texarkana TFOT/PAV conditioned binders. Latex performed the best in preventing fatigue cracking, while Chemkrete (in the base) performed the worst, showing excessive thermal cracking after only 1 winter in service. Recall Latex was substituted for Chemkrete in the surface layer because Chemkrete was temporarily pulled from the U.S. market, and it exhibited excessive transverse cracking in the base layer. The Chemkrete/Latex combination ranked second in overall cracking performance. The overall ranking for cracking performance of the test pavements from best to worst is: Latex, Chemkrete/Latex, Styrelf, Control, and EVA. Figure 11 ranks the binders in essentially the same order when viewed from top to bottom.

Figure 12 shows an excellent correlation ($r^2 = 0.92$) between loss tangent at 10 radians/sec and 10°C of the TFOT/PAV conditioned binders and longitudinal (fatigue) cracking in the test pavements. This figure shows that Latex and Styrelf improved cracking resistance over the Control binder. Recall that EVA was blended with AC-20, while all other additives at Texarkana were blended with AC-10. This was surely a factor in the lower loss tangent measured, and the prodigious cracking exhibited by the EVA.

Table 7. Loss Tangent Values @ 10 rad/sec for Retained Binders from Texarkana and Sherman After TFOT/PAV.

Texarkana

Temperature, °C	Latex	Chemkrete	Styrelf	Control	EVA
10	0.85	0.61	0.77	0.74	0.61
20	1.33	0.81	1.26	0.90	0.84
30	2.24	0.97	1.87	1.20	1.13
40	3.51	1.24	2.62	1.83	1.50
50	5.22	1.63	3.08	2.83	2.08
60	31.11	2.25	8.77	15.14	5.64
70	3.63	3.90	21.32	15.04	1.06

Sherman

Temperature, °C	Novophalt	SBS	Latex	Control	Carbon Black	EVA
10	0.56	0.80	0.73	0.64	0.80	0.89
20	0.85	1.07	1.16	0.88	0.89	1.04
30	1.09	1.44	1.49	1.15	1.07	1.27
40	1.40	1.78	1.83	1.54	1.42	1.63
50	1.89	2.10	2.28	2.11	2.10	2.17
60	2.86	2.64	3.44	3.18	3.50	3.23
70	8.54	6.12	79.89	46.73	7.03	16.33

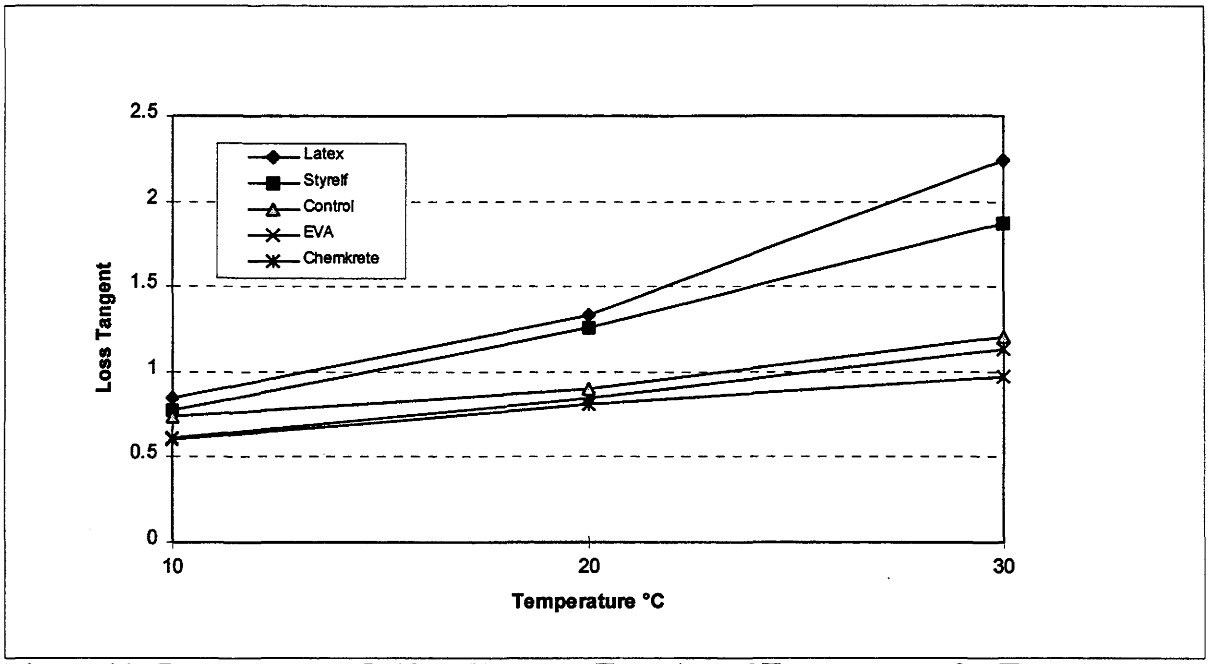

Figure 11. Loss Tangent @ 10 rad/sec as a Function of Temperature for Texarkana TFOT/PAV Conditioned Binders.

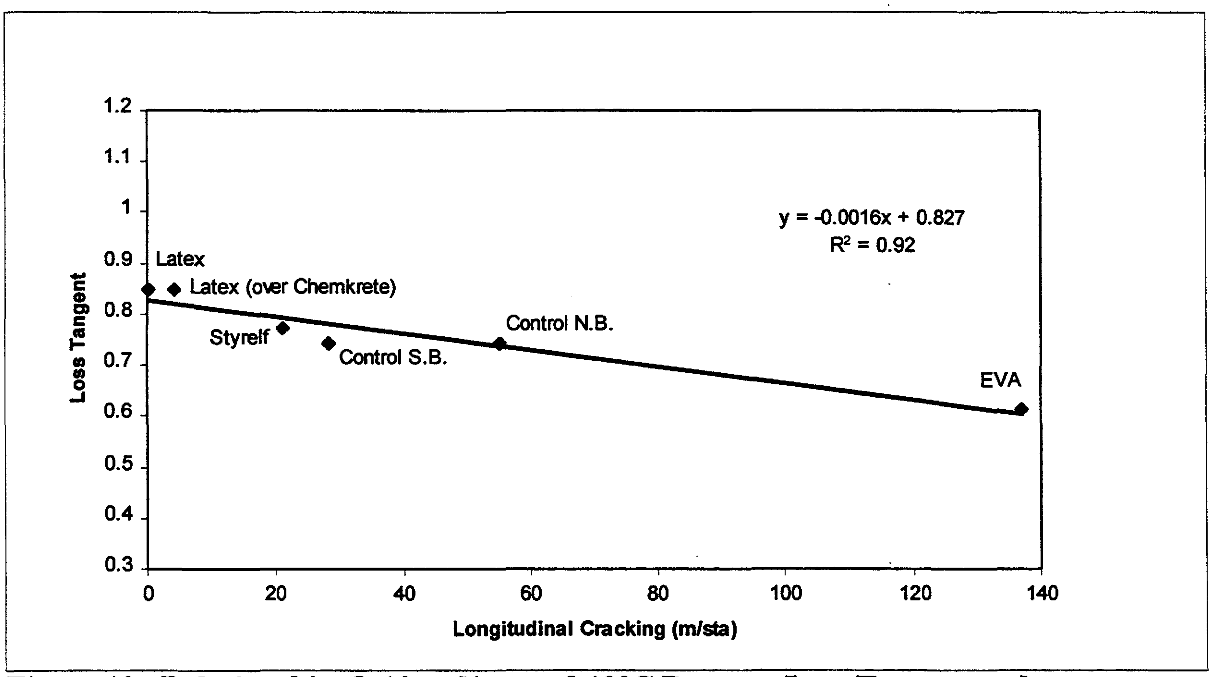

Figure 12. Relationship @ 10 rad/sec and 10°C Between Loss Tangent and Longitudinal (Fatigue) Cracking for Texarkana TFOT/PAV Conditioned Binders.

Figure 13 illustrates the loss tangent as a function of temperature for the TFOT/PAV conditioned binders from Sherman. The overall ranking performance of the test binders from best to worst is: Novophalt, SBS, Latex, Control, Carbon Black and EVA. Notice the irregularity of the loss tangent with respect to cracking performance of the test binders.

Figure 14 illustrates a relationship which is reverse to the relationship established for the Texarkana binders in Figure 12 and is illogical. A poor correlation ($r^2 = 0.38$) exists for the relationship between loss tangent and the reflection cracking observed for Sherman. It should be pointed out that the longitudinal cracking in Texarkana is fatigue cracking; whereas, the longitudinal cracking in Sherman is reflective cracking from the underlying CRCP which was severely cracked. This makes the cracking in the Sherman pavements more difficult to relate to binder properties. Reflection cracking is controlled more by differential movements in the underlying pavement than by the properties of the binders in the overlay.

SHRP Superpave Binder Test Results

In order to develop a performance-based asphalt binder specification, linear viscoelastic properties as influenced by time of loading and temperature must be characterized. These are the fundamental material properties that relate to performance. The binder specification is based primarily on asphalt binder stiffness of both unaged and aged material measured at a specific combination of load duration and temperature. Selected temperatures are related to the environment in which the asphalt binder must serve. Hence, asphalt binder grades are specified for design pavement temperature (28).

Environmental conditions are specified by Superpave as the highest 7-day average of daily maximum pavement temperature and the lowest pavement temperature in a year. According to TxDOT's Materials and Test Division, the high and low temperature ranges for both Texarkana and Sherman, using a 98% confidence level, are 64°C and -22°C, respectively, which correlates with a Superpave performance grade of 64-22.

Table 8 shows low temperature performance grades of -22 for all Texarkana binders except the Control which has a -28. The low temperature performance grade for Styrelf is not available due to a shortage of material. The only binder that does not meet the Superpave requirement is Latex with a 58-22 performance grade. Latex is "too soft" at the high temperature end which probably contributed to its superior performance regarding cracking. This shows that high quality aggregate can resist rutting even when a binder too soft to meet Superpave specifications is used. No correlation between cracking in the test pavements and SHRP performance grades is apparent.

Table 9 shows low temperature performance grades of -22, -28, and -34 for the Sherman binders with no apparent correlation between measured cracking and SHRP performance grades. Latex, EVA, and the Control binders did not meet the Superpave high temperature requirements.

None of the SHRP parameters exhibit any consistent correlations with the cracking observed in the test pavements.

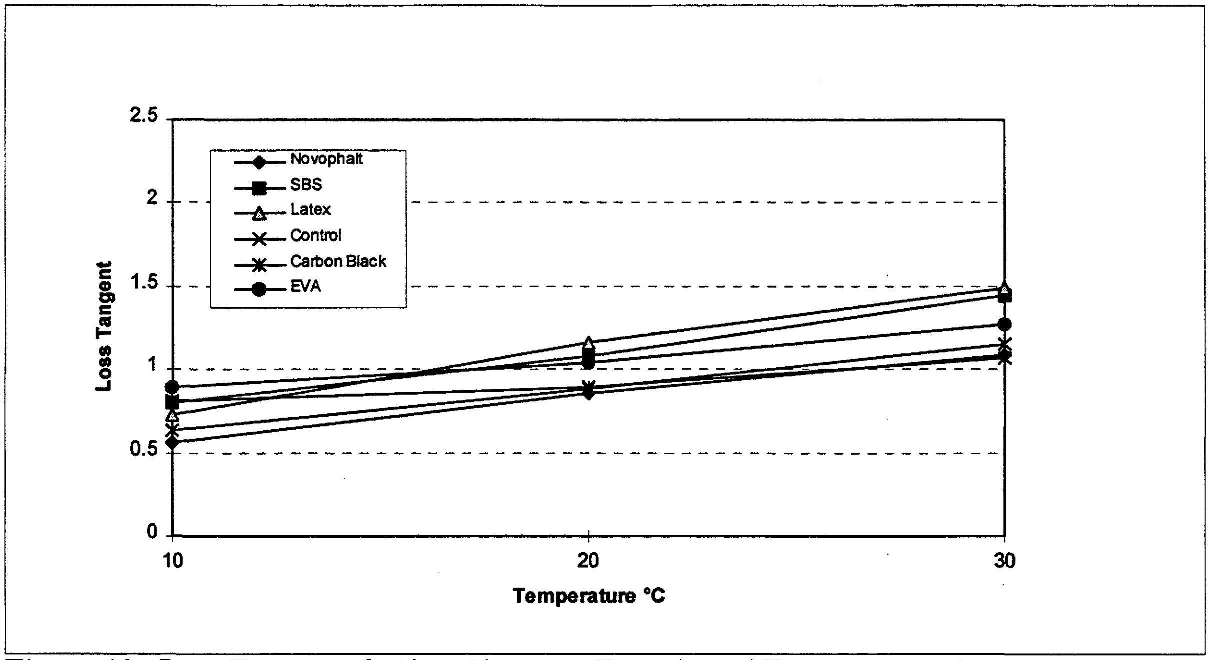

Figure 13. Loss Tangent @ 10 rad/sec as a Function of Temperature for Sherman TFOT/PAV Conditioned Binders.

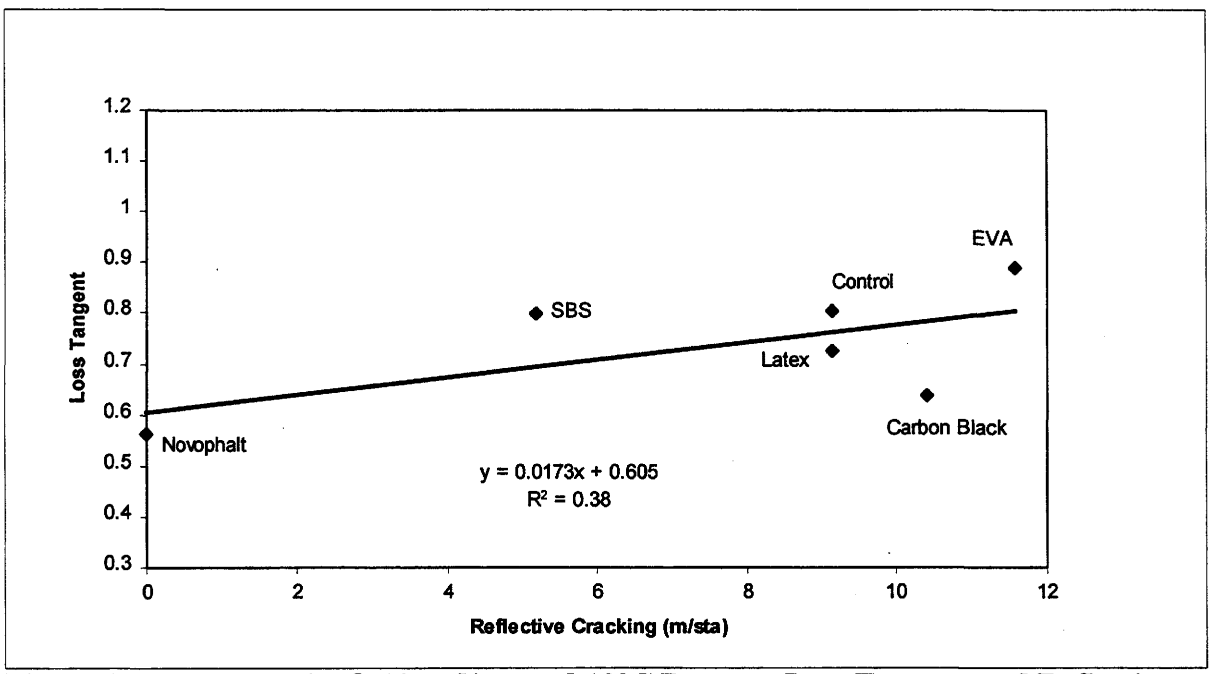

Figure 14. Relationship @ 10 rad/sec and 10°C Between Loss Tangent and Reflective Cracking for Sherman TFOT/PAV Conditioned Binders.

Table 8. Summary of SHRP Binder Test Results for Texarkana Test Binders.

Texarkana Test Binder	Latex	Chemkrete	Styrelf	Control	EVA
Performance Grade (PG)	**PG 58-22**	**PG 64-22**	**PG 64-**	**PG 64-28**	**PG 64-22**
	Original Binder				
Viscosity, ASTM D 4402, Brookfield					
Max. 3 Pa-s @135°C					
Dynamic Shear, AASHTO TP5					
G* / sin(delta), Min. 1.00 kPa	1.25 kPa [OK]	1.33 kPa [OK]	1.29 kPa [OK]	1.62 kPa [OK]	1.13 kPa [OK]
Test Temp @ 10 rad/s, °C	64°	64°C	70°C	64°C	70°C
	Rolling Thin Film Oven Residue (AASHTO T 240)				
Mass Loss, Max 1.00%					
Dynamic Shear, AASHTO TP5					
G* / sin(delta), Min. 2.20 kPa	3.66 kPa [OK]	3.47 kPA [OK]	3.58 kPa [OK]	2.53 kPa [OK]	3.48 kPa [OK]
Test Temp @ 10 rad/s, °C	58°C	64°C	64°C	64°C	64°C
	Pressure Aging Vessel Residue (AASHTO PP1)				
Dynamic Shear, AASHTO TP5					
G* sin(delta), Max. 5000 kPa					
Test Temp @ 10 rad/s, °C					
Creep Stiffness, AASHTO TP1					
S, Max, 300 MPa	229 Mpa [OK]	81 MPa [OK]	not available	154 MPa [OK]	201 MPa [OK]
m-value, Min. 0.300	0.330 slope [OK]	0.320 slope [OK]		0.340 slope [OK]	0.320 slope [OK]
Test Temp @ 60s, °C	-12°C	-12°C		-18°C	-12°C

Table 9. Summary of SHRP Binder Test Results for Sherman Test Binders.

Sherman Test Binder	Novophalt	SBS	Latex	Control	Carbon Black	EVA
Performance Grade (PG)	**PG 70-22**	**PG 64-34**	**PG 58-34**	**PG 64-28**	**PG 58-28**	**PG 58-28**
	Original Binder					
Viscosity, ASTM D 4402, Brookfield						
Max. 3 Pa-s @135°C	1.34 Pa-s [OK]	0.70 Pa-s [OK]	0.51 Pa-s [OK]	0.53 Pa-s [OK]	0.41 Pa-s [OK]	0.57 Pa-s [OK]
Dynamic Shear, AASHTO TP5						
G* / sin(delta), Min. 1.00 kPa	1.32 kPa [OK]	1.27 kPa [OK]	1.44 kPa [OK]	1.35 kPa [OK]	1.61 kPa [OK]	1.86 kPa [OK]
Test Temp @ 10 rad/s, °C	70°C	64°C	58°C	64°C	58°C	58°C
	Rolling Thin Film Oven Residue (AASHTO T 240)					
Mass Loss, Max 1.00%	0.01% [OK]	0.01% [OK]	0.05% [OK]	0.01% [OK]	0.02% [OK]	0.03% [OK]
Dynamic Shear, AASHTO TP5						
G* / sin(delta), Min. 2.20 kPa	3.00 kPa [OK]	2.26 kPA [OK]	2.74 kPa [OK]	2.74 kPa [OK]	3.38 kPa [OK]	3.66 kPa [OK]
Test Temp @ 10 rad/s, °C	70°C	64°C	58°C	64°C	58°C	58°C
	Pressure Aging Vessel Residue (AASHTO PP1)					
Dynamic Shear, AASHTO TP5						
G* sin(delta), Max. 5000 kPa	1197 kPa [OK]	1642 kPa [OK]	1657 kPa [OK]	2331 kPa [OK]	2200 kPa [OK]	1852 kPa [OK]
Test Temp @ 10 rad/s, °C	28°C	19°C	16°C	22°C	19°C	19°C
Creep Stiffness, AASHTO TP1						
S, Max, 300 MPa	84 MPa [OK]	235 MPa [OK]	213 MPa [OK]	177 MPa [OK]	143 MPa [OK]	441 MPa [Fail]
m-value, Min. 0.300	0.317 slope [OK]	0.329 slope [OK]	0.327 slope [OK]	0.307 slope [OK]	0.330 slope [OK]	0.642 slope [OK]
Test Temp @ 60s, °C	-12°C	-24°C	-24°C	-18°C	-18°C	-18°C

Extracted Binders Tests

Penetration

Tables 10 and 11 show the differences in penetration values between tank binder, retained binder, and extracted binder for Texarkana and Sherman, respectively. Tank binder penetrations were measured in 1987 on the original materials. Retained binders had been sealed in metal cans for about 9 years. The extracted binders came from pavement cores drilled in 1995. Considerable age-hardening occurred in these binders as shown in Figures 15 and 16.

Figure 15 facilitates comparisons of penetration values for the Texarkana test binders. Perhaps the most notable change in penetration values is that between the original and extracted Chemkrete which changed from 118 dmm to 8 dmm. Note that the Chemkrete surface layer was replaced with Latex because of excessive cracking in the base layer due to the extreme hardness. Polybilt EVA, which was mixed with AC-20 instead of AC-10 like all the other additives, exhibits the lowest extracted and retained penetrations.

Figure 16 shows penetration values for the Sherman test binders. The extracted penetration values are much lower than the original values except for the Carbon Black binder which indicates softening occurred during storage. Softening is highly unlikely. Further, the original penetration values for Carbon Black appear questionable. Since carbon black settles rapidly when asphalt is hot and static, penetration is dependent on how long and vigorously the binder is stirred during cooling after the penetration specimen has been poured. Consequently, penetration of Carbon Black-modified asphalt is subject to considerable error. In addition, Microfil-8 contains 8% oil that significantly lowers binder consistency at temperatures below about 50°C.

Penetration values at any stage (original, retained, or extracted) do not correlate well with the observed cracking performance.

Chemical Testing

Gel permeation chromatography (GPC) was used to measure molecular size distribution. Fourier transform infrared (FT-IR) analysis was used to measure chemical functionality. Asphaltene content (using heptane) was measured to give an estimate of the very large molecular size hydrocarbons in each binder.

Table 12 shows the asphaltene content for each tank binder. Figure 17 shows the relationship between asphaltene content and fatigue cracking for Texarkana test binders. For the Texarkana binders, lower asphaltene content corresponds to higher fatigue resistance. A reasonable correlation ($r^2 = 0.44$) exists for the relationship between asphaltene content and fatigue cracking for Texarkana binders (Figure 17). Figure 18 shows that there is no correlation between asphaltene content and the observed reflection cracking in Sherman ($r^2 = 0.01$). Recall that 4 different asphalt sources were used in Texarkana, and only one asphalt source was used in Sherman. Similar asphalts exhibit similar asphaltene contents.

Table 10. Comparison of Standard Penetration Values for Texarkana Binders.

Test Property	Type of Binder						
	MacMillan AC-20	Chemkrete + MacMillan	Lyon AC-20	Polybilt + Lyon	Fina AC-10	Goodyear 5812 + Fina	Styrelf Exxon
Penetration, ASTM D5							
Original Tank Binder: @25°C, 100gm, 5s	86	118	64	48	90	83	90
Retained Tank Binder: @25°C, 100gm, 5s	83	111	-	47	-	70	63
Extracted Binder (Base): @25°C, 100gm, 5s	20	8	-	9	-	38	49
Extracted Binder (Surface): @25°C, 100gm, 5s	22	23[1]	-	9	-	25	11

[1]Latex substituted for Chemkrete surface.

Table 11. Comparison of Standard Penetration Values for Sherman Binders.

Test Property	Type of Binder						
	AC-20	AC-10	Carbon Blk. + AC-10[2]	EVA + AC-10	Novophalt + AC-10	Latex + AC-10	SBS[1] + AC-10
Penetration, ASTM D5							
Original Tank Binder: @25°C, 100gm, 5s	92	114	38	127	86	137	115
Retained Tank Binder: @25°C, 100gm, 5s	74	102	52	119	78	113	115
Extracted Binder: @25°C, 100gm, 5s	25	-	53	32	47	50	54

[1]Also contains some Exxon 120/150 grade asphalt. See text.
[2]Blended in the TTI laboratory using low shear desk top mixer.

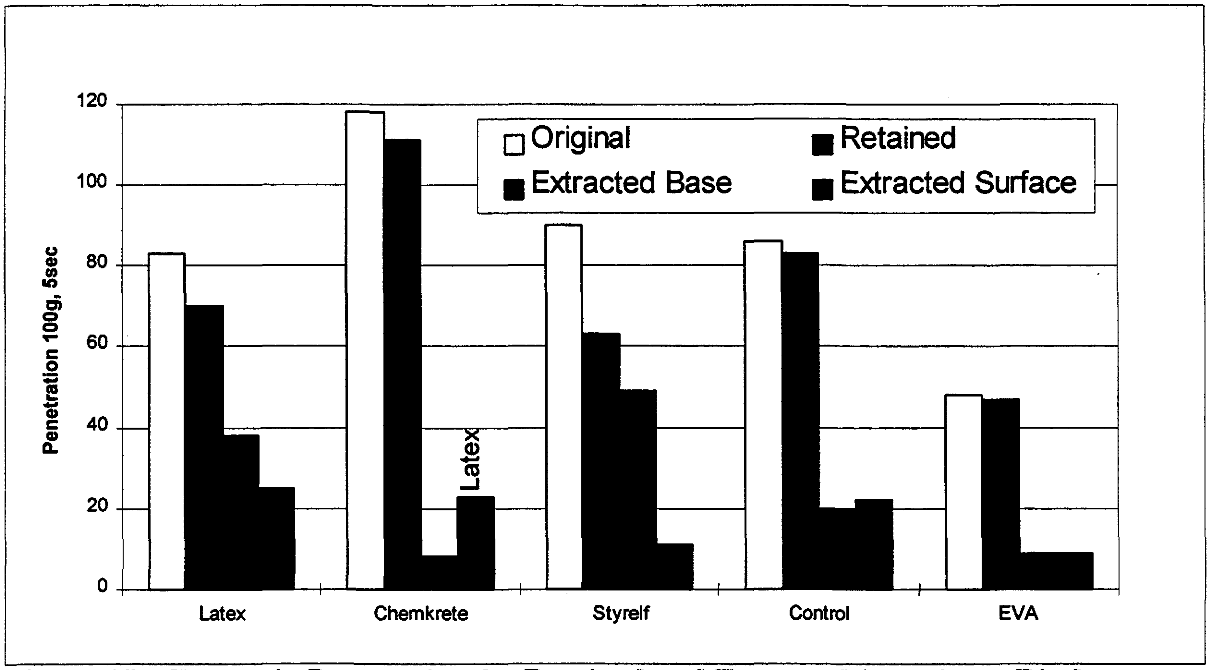

Figure 15. Change in Penetration for Retained and Extracted Texarkana Binders.

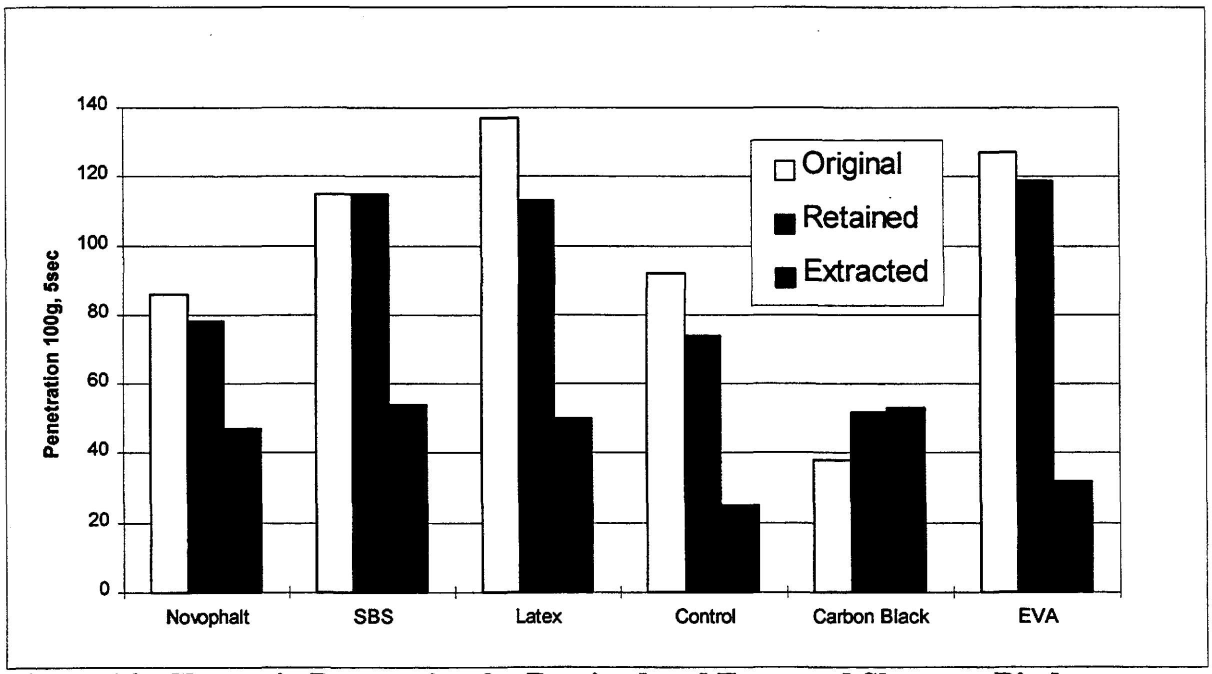

Figure 16. Change in Penetration for Retained and Extracted Sherman Binders.

Table 12. Asphaltene Content for Texarkana and Sherman Tank Binders.

TEXARKANA

Base Sample	Latex	Chemkrete	Styrelf	Control S.B.	Control N.B.	EVA
Asphaltene Content (%)	9.4	14	15.9	14.5	14.5	16.1
Surface Sample						
Asphaltene Content (%)	9.4	9.4[1]	15.9	14.5	14.5	16.1

SHERMAN

Sample ID	Novophalt	SBS	Latex	Control	Carbon Black	EVA
Asphaltene Content (%)	15.6	14.4	14.4	11.3	24.9	12.2

[1]Latex over Chemkrete.

This is the reason for the lack of correlation in the Sherman asphalts (Figure 18). Asphaltenes precipitated from binders containing polymers were hard and not easily crumbled to fine powder, thus indicating that some polymer was likely precipitated along with the asphaltenes. Carbon black, being insoluble in heptane, appeared as asphaltenes. Therefore, the accuracy of the measured asphaltene contents of the modified binders is questionable.

Gel permeation chromatography has been used by numerous researchers to investigate the asphalt fractions contributing to pavement performance. GPC gives a rough representation of molecular size distribution but tells nothing directly about molecular structure. Jennings (16) found a high degree of correlation between pavement performance and molecular size distribution. He determined a low large molecular size (LMS) fraction is desirable to reduce cracking. GPC chromotagraphs illustrating molecular size distributions for each binder are presented in Appendix B, Figures B1-B16.

Table 13 shows the resulting molecular size fraction from the GPC tests. Asphalts were compared by mathematical determination of the areas under the curves. The area was divided into thirds: large molecular size (LMS), medium molecular size (MMS), and small

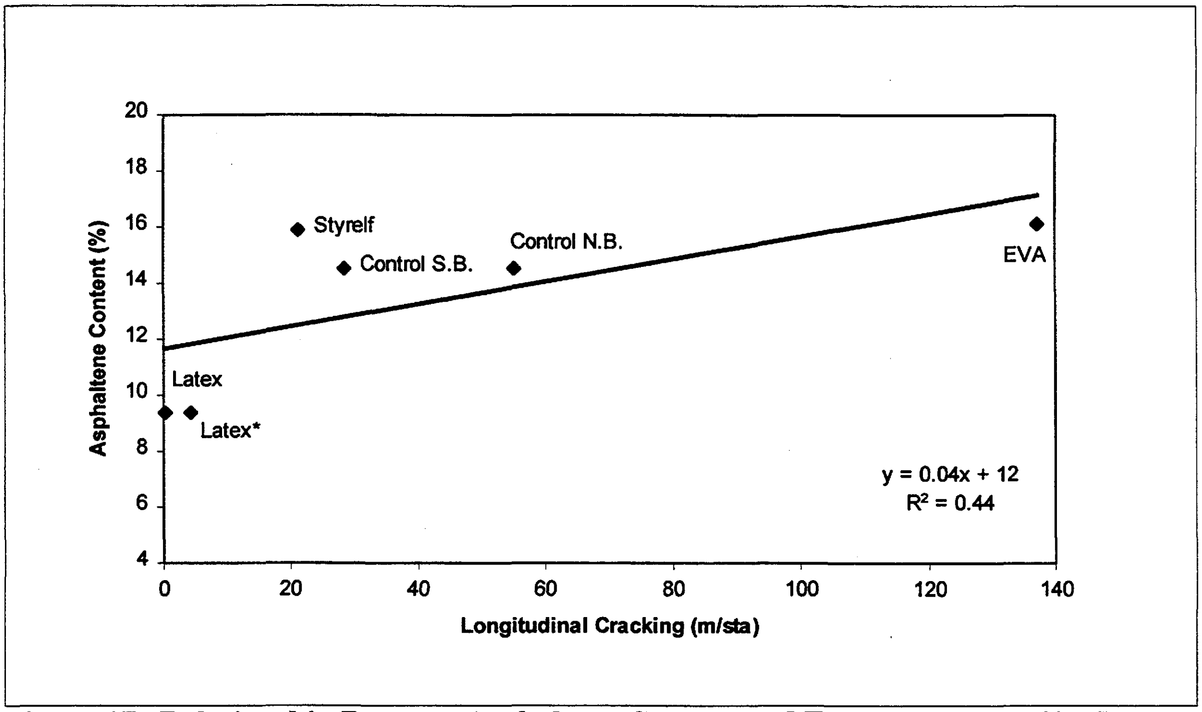

Figure 17. Relationship Between Asphaltene Content and Fatigue (Longitudinal) Cracking for Texarkana Surface Binders (*Latex over Chemkrete).

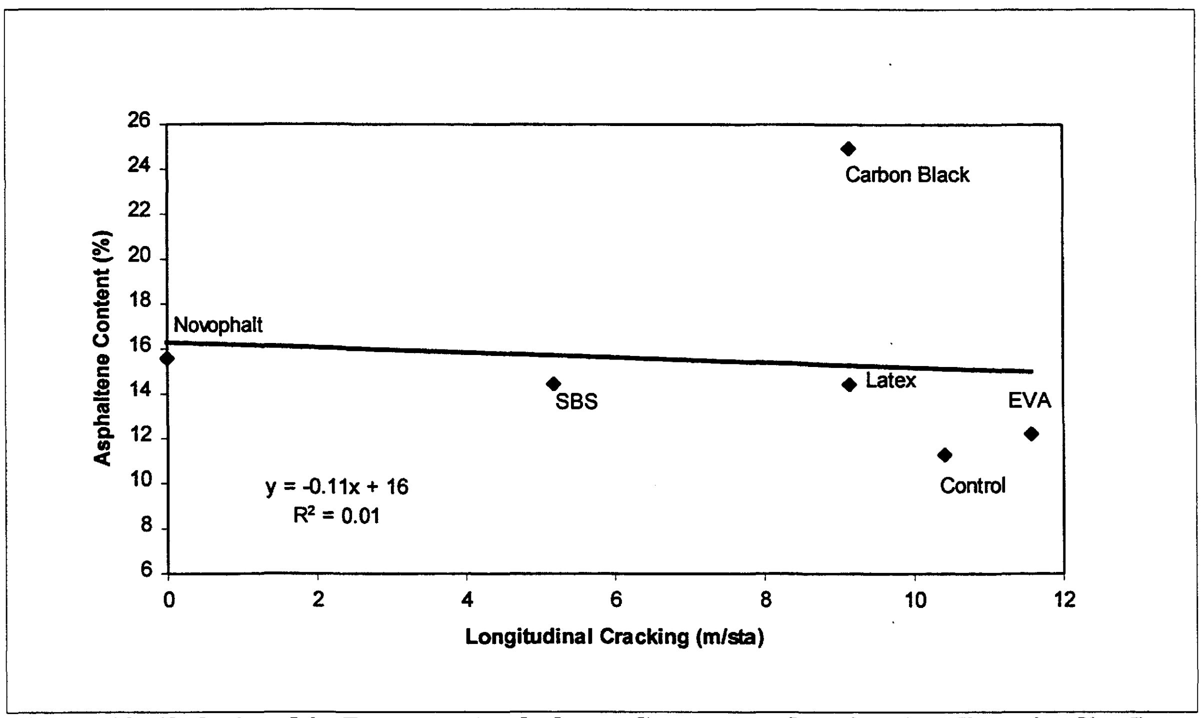

Figure 18. Relationship Between Asphaltene Content and Reflective (Longitudinal) Cracking for Sherman Binders.

Table 13. Molecular Size Fraction for Texarkana and Sherman Tank and Extracted Binders.

TEXARKANA

Tank Binder	Latex	Chemkrete	Styrelf	Control S.B.	Control N.B.	EVA
LMS %	12	21	14	20	20	18
MMS %	74	68	70	69	69	67
SMS %	14	11	16	11	11	15
Extracted Base Binder						
LMS %	16	35	14	21	28	26
MMS %	72	60	73	65	63	63
SMS %	12	5	13	14	9	11
Extracted Surface Binder						
LMS %	22	21[1]	21	25	25	27
MMS %	68	72[1]	64	64	60	63
SMS %	10	7[1]	15	11	15	10

SHERMAN

Tank Binder	Novophalt	SBS	Latex	Control	Carbon Black	EVA
LMS %	18	19	19	20	20	19
MMS %	70	65	69	69	69	67
SMS %	12	16	12	11	11	14
Extracted Binder						
LMS %	26	26	25	27	27	27
MMS %	65	60	63	63	62	63
SMS %	9	14	12	10	11	10

[1]Latex over Chemkrete.

molecular size (SMS). Asphalts were evaluated by comparing the percentage of LMS materials in each. As noted by Jennings (16), low LMS such as Texarkana's latex and Sherman's Novophalt are desirable to reduce cracking (Figures 19 and 20). Figure 19 shows the relationship between LMS fraction of the binder and observed fatigue cracking in the Texarkana test pavements. A fairly strong correlation was observed ($r^2 = 0.72$) between the LMS fraction and fatigue cracking for the Texarkana extracted surface test (Figure 19). A very weak correlation exists ($r^2 = 0.15$) between the LMS fraction and observed reflection cracking for the Sherman extracted test binders (Figure 20). Again, the asphalts used in Sherman were obtained from the same source; whereas, 4 different asphalt sources were used at Texarkana. GPC results are influenced more by asphalt source than the presence or type of additive.

The principal cause of age hardening and embrittlement of asphalt used in pavements is the atmospheric oxidation of certain asphalt molecules with the formation of highly polar and strongly interacting chemical functional groups containing oxygen (18). Thus, the ability to identify and quantify asphalt chemical functionality provides an important tool for assessing the effects of composition on asphalt properties and, thus, the performance of the asphalt in service (24). FT-IR analyses performed on the binders quantified carbonyl content of both tank and extracted binders. As expected, the carbonyl contents of the extracted (oxidized) binders were much higher than those of the tank binders. Asphalts were compared by calculating the difference in carbonyl area of the extracted and tank binders. This difference in carbonyl area is commonly known as carbonyl growth.

Table 14 summarizes carbonyl growth values from FT-IR tests performed on each binder. Growth in carbonyl areas are presented in Appendix B, Figures B17-B32. Figure 21 illustrates the relationship between the carbonyl growth for the Texarkana surface mixtures and fatigue cracking. A correlation exists ($r^2 = 0.60$) between the carbonyl growth and fatigue cracking for the Texarkana surface extracted binders (Figure 21). Figure 22 illustrates the relationship between the Sherman carbonyl growth and reflective cracking. No correlation exists ($r^2 = 0.01$) between carbonyl growth and reflective cracking for the Sherman extracted binders (Figure 22). Asphalts from the same source should exhibit little difference in carbonyl growth with time.

Tests on Laboratory Compacted Mixtures

Materials used in the AAMAS tests were field mix types that were retained during construction in 20 liter containers and later reheated and compacted in the laboratory. All Australian frequency sweep tests used aggregate and binders retained during construction that were mixed and compacted in the laboratory.

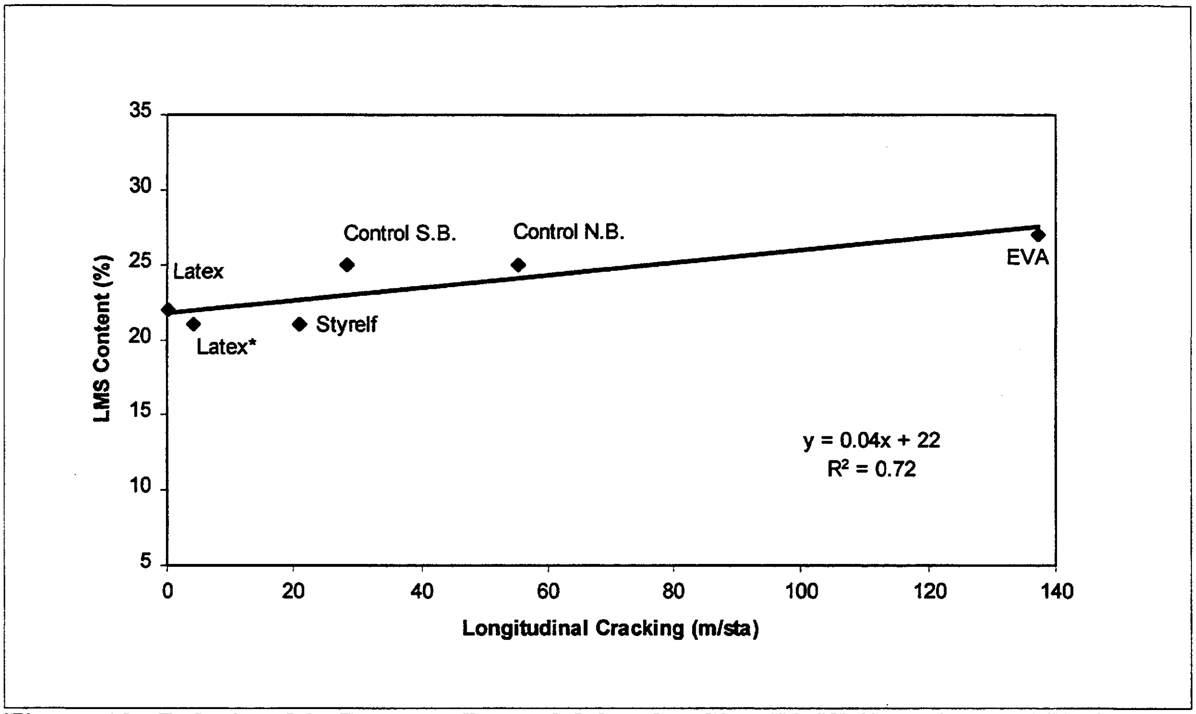

Figure 19. Relationship Between Large Molecular Size (LMS) Fraction and Fatigue (Longitudinal) Cracking for Texarkana Extracted Surface Binders (*Latex over Chemkrete).

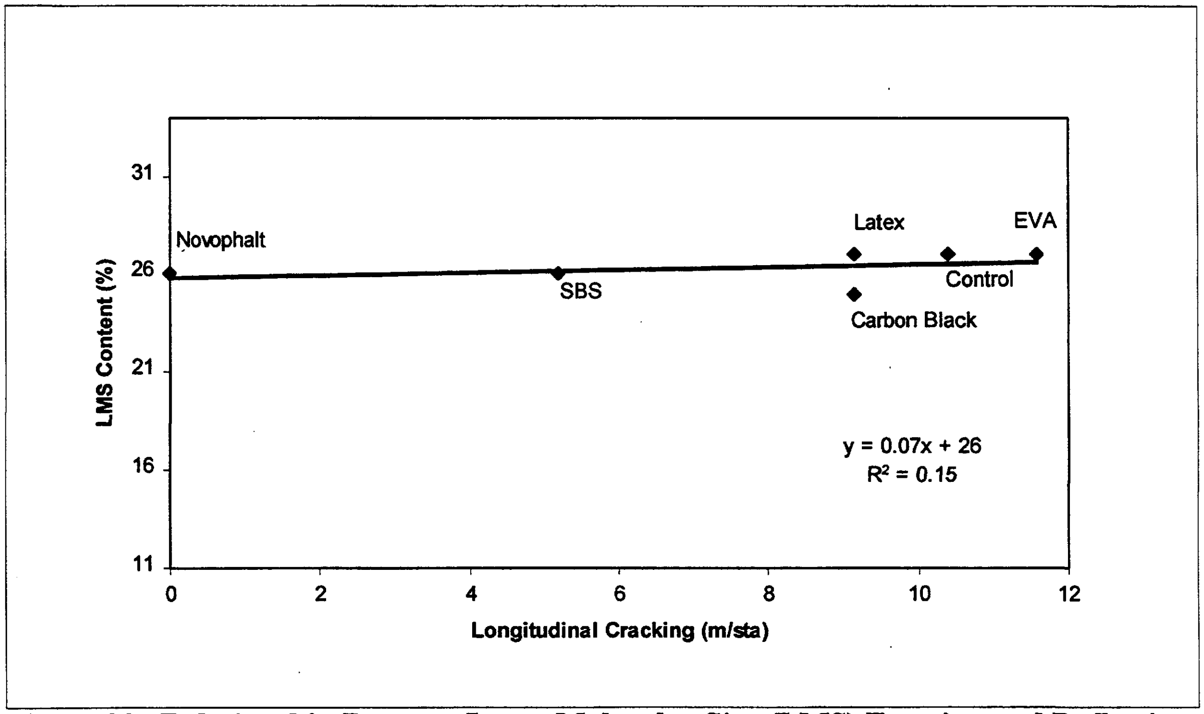

Figure 20. Relationship Between Large Molecular Size (LMS) Fraction and Reflective (Longitudinal) Cracking for Sherman Extracted Binders.

Table 14. Carbonyl Growth to Determine Relative State of Oxidation for Texarkana and Sherman Test Binders.

TEXARKANA

Tank Binder	Latex	Chemkrete	Styrelf	Control S.B.	Control N.B.	EVA
Carbonyl Area	0.446	0.563	0.435	0.468	0.468	0.625
Extracted Base Binder						
Carbonyl Area	0.990	2.587	1.052	1.785	2.003	2.312
Carbonyl Growth	0.544	2.024	0.617	1.317	1.535	1.687
Extracted Surface Binder						
Carbonyl Area	1.768	1.717[1]	1.964	1.838	1.988	2.215
Carbonyl Growth	1.322	1.271[1]	1.529	1.370	1.520	1.590

SHERMAN

Tank Binder	Novophalt	SBS	Latex	Control	Carbon Black	EVA
Carbonyl Area	0.503	0.516	0.615	0.518	0.773	0.725
Extracted Binder						
Carbonyl Area	1.193	1.508	1.398	1.286	1.565	1.521
Carbonyl Growth	0.690	0.992	0.783	0.768	0.792	0.796

[1]Latex over Chemkrete.

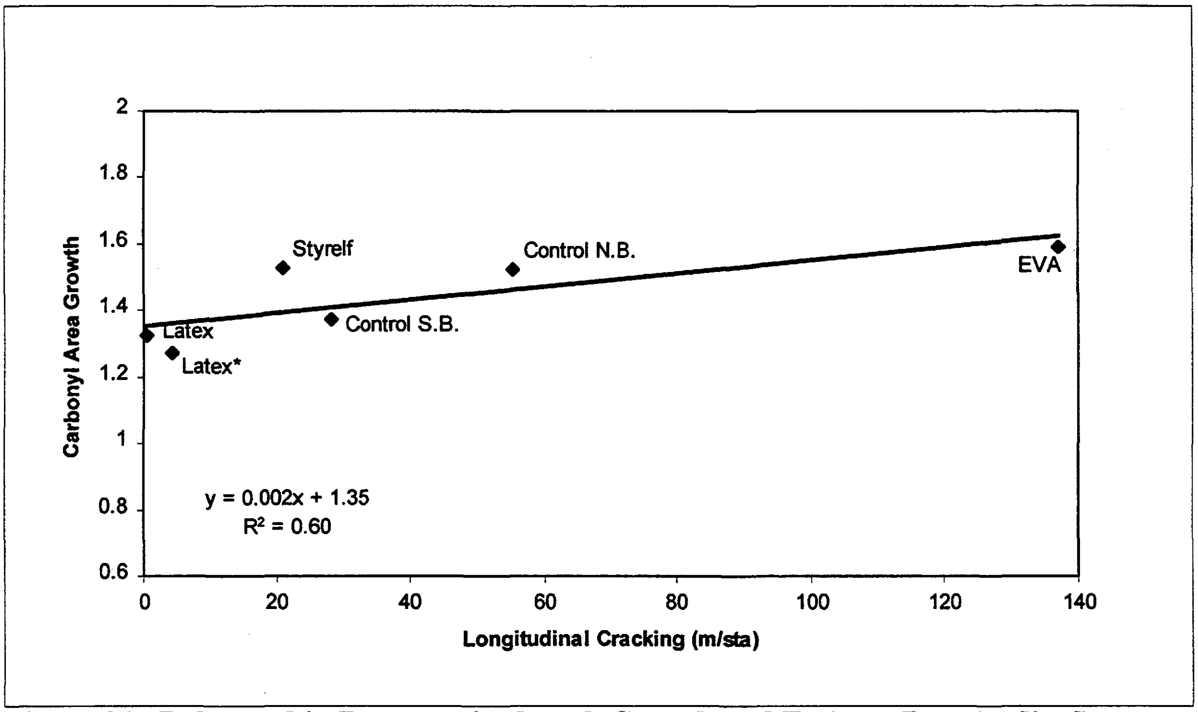

Figure 21. Relationship Between Carbonyl Growth and Fatigue (Longitudinal) Cracking for Texarkana Surface Binders (*Latex over Chemkrete).

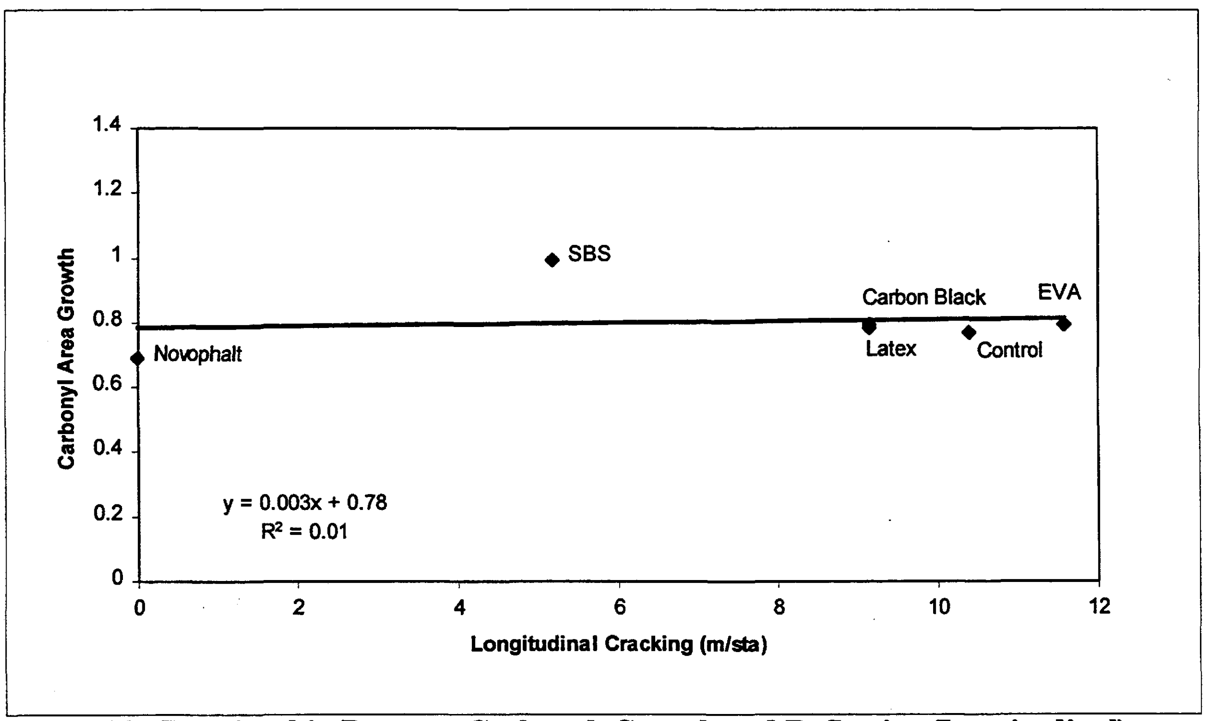

Figure 22. Relationship Between Carbonyl Growth and Reflective (Longitudinal) Cracking for Sherman Binders.

AAMAS Test to Predict Fatigue Cracking

Indirect tension (IDT) tests at 25°C and 51-mm/min and resilient modulus tests at 0°C, 25°C, and 40°C were conducted using a servo-hydraulic closed loop MTS testing machine. Following the procedures set forth by NCHRP's asphalt aggregate mixture analysis system (AAMAS) (8) the IDT data were used along with total resilient modulus (Appendix C, Tables C1 and C2) data to predict the relative resistance of the test pavements to fatigue cracking. Both indirect tension and resilient modulus tests produce biaxial stress fields and are, therefore, sensitive to binder viscosity and content, filler/asphalt ratio, and air void content, some of which varied considerably in the specimens tested.

Figures 23 and 24 show comparisons of the binders to the standard AASHTO Road Test mixture (Federal Highway Administration fatigue curve (29)). If a point plots above the FHWA curve, it is assumed that the mixture has better fatigue resistance than the standard mixture. Based on the plots of these points, none of the test binders for Texarkana or Sherman are predicted to resist fatigue cracking as well as the standard mixture. In fact, based on the scatter of the data, there seems to be no correlation of this test with the observed fatigue performance.

Figure 23 indicates that Styrelf and Latex resists fatigue cracking better than EVA, Chemkrete, and Control pavement sections for the base. Although this result is not entirely accurate (i.e., does not correlate precisely with field observations), it seems useful in identifying potential problem binders (Figure 23). The values used for Figures 23 and 24 were from tests reported earlier (3, 4). These values had to be used due to a shortage of pavement aggregates and binders. As a result, the data is only for the base layer, which explains why the Chemkrete layer is predicted to crack more than the EVA layer (Figure 23). Figure 24 indicates that SBS, Carbon Black, Control, and Latex perform better than EVA and Novophalt pavement sections. In reality, the Novophalt pavement section exhibited the least cracking. The randomness of these results (Figure 24) occurred because the cracking in Sherman was reflection cracking from the underlying portland cement concrete pavement rather than fatigue cracking.

Australian Frequency Sweep Tests

Laboratory compacted specimens 76 mm in diameter and 152 mm in length were subjected to a sinusoidal loading pattern and compliance was measured using strategically located linear variable displacement transducers (LVDTs). Compliance was related to fatigue cracking through a series of calculations and pavement life (number of load cycles to failure) was predicted (Table 15).

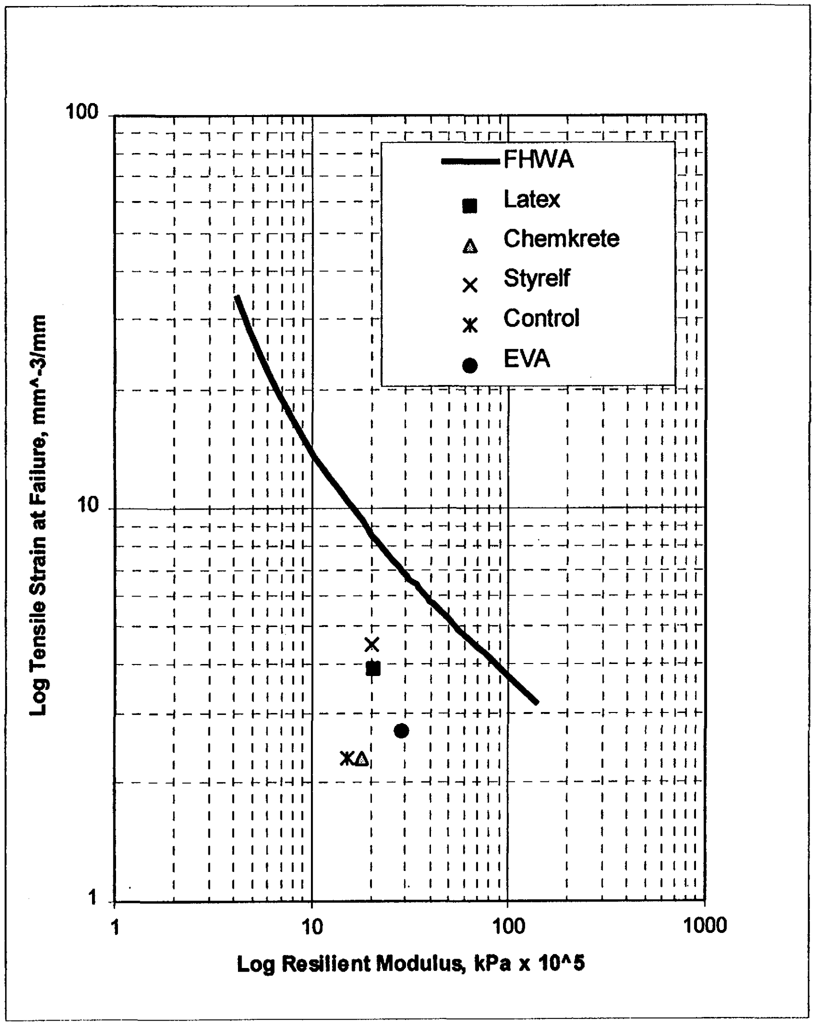

Figure 23. Relationship Between Indirect Tensile Strains and Resilient Modulus for Laboratory Compacted Texarkana Base Specimens Using the Minimum Failure Strains for the FHWA Fatigue Relationship (8).

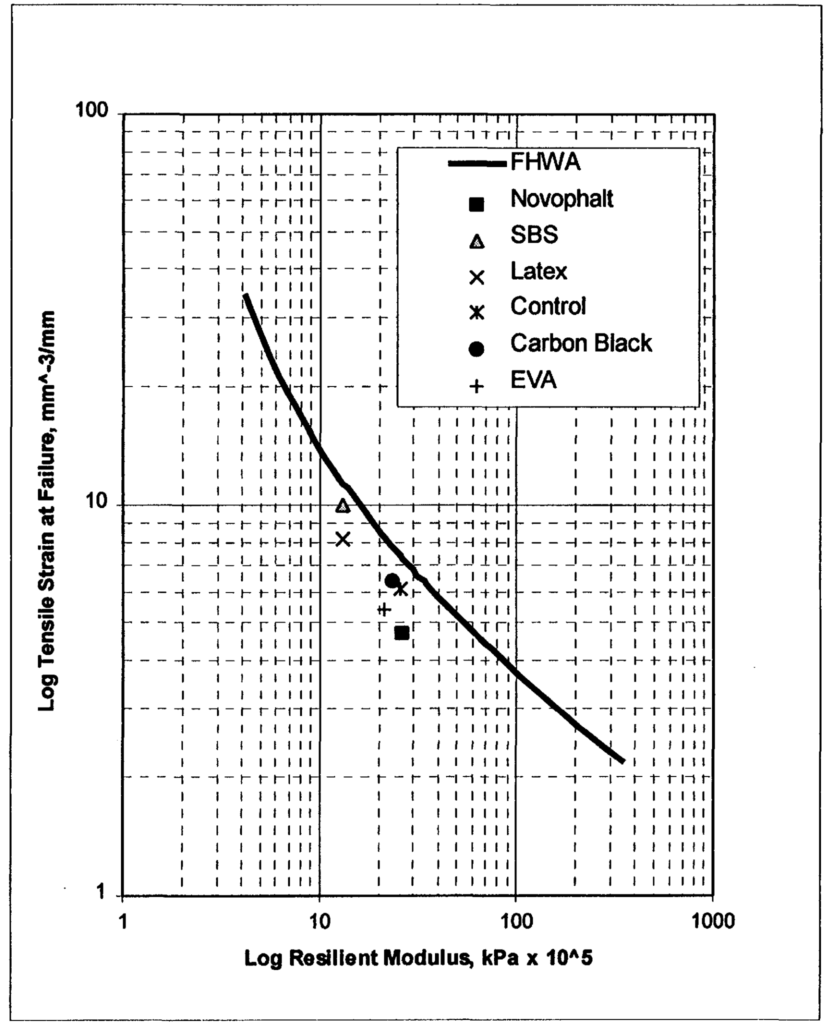

Figure 24. Relationship Between Indirect Tensile Strains and Resilient Modulus for Laboratory Compacted Sherman Specimens Using the Minimum Failure Strains Required for the FHWA Fatigue Relationship (8).

Table 15. Australian Frequency Sweep Pavement Life Predictions Using Laboratory Compacted Specimens.

Texarkana	Number of loads to failure, Nf x 10^4
Latex	5.93
Chemkrete	1.63
Styrelf	3.50
Control	21.2
EVA	4.38
Sherman	
Novophalt	25.2
SBS	308.
SBR	6.90
Carbon Black	867.
Control	1046.
EVA	148.

Tests on Pavement Cores

AAMAS Test to Predict Fatigue Cracking

Researchers obtained pavement cores 100 mm in diameter and conducted indirect tension (IDT) tests at 25°C and 51-mm/min and resilient modulus tests at 0°C, 25°C, and 40°C using a servo-hydraulic closed loop MTS testing machine. Following the procedures set forth by NCHRP's asphalt aggregate mixture analysis system (AAMAS) (8), the IDT data were used with total resilient modulus (Appendix C, Tables C3-C5) data to predict the relative resistance of the test pavements to fatigue cracking.

Figures 25-27 show comparisons of the test binders to the standard AASHTO Road Test mixture (Federal Highway Administration fatigue curve). There are no consistent correlations of these test results with the observed fatigue cracking performance.

Figure 25 indicates that Latex, Styrelf, and EVA resist fatigue cracking better than the standard mixture; whereas, Control S.B., Control N.B., and Chemkrete are less resistant to cracking than the standard mixture. Perhaps the higher air void content in the EVA test pavement contributed to the large difference between the predicted performance and the actual performance. Table C3 shows air void contents for Texarkana base pavement cores. Figure 26 indicates that Latex (placed over Chemkrete) and Control S.B. are more fatigue resistant than the standard mixture; whereas, Control N.B., Latex, Styrelf, and EVA pavement sections (for the surface) are less fatigue resistant. In reality, the Latex pavement section performed the best followed by Latex (over Chemkrete), Styrelf, Control S.B., Control N.B., and EVA test pavements. Figure 27 indicates that EVA and Novophalt are more fatigue resistant than the standard mixture followed by Control, Carbon Black, Latex, and SBS pavement sections. Again, the cracking in Sherman was reflection cracking and not pure fatigue cracking.

Australian Frequency Sweep Tests

Field cores 76 mm in diameter and 152 mm in length were subjected to a sinusoidal loading pattern and compliance was measured with strategically located LVDTs. Compliance was related to fatigue cracking through a series of calculations and pavement life was predicted (Table 16).

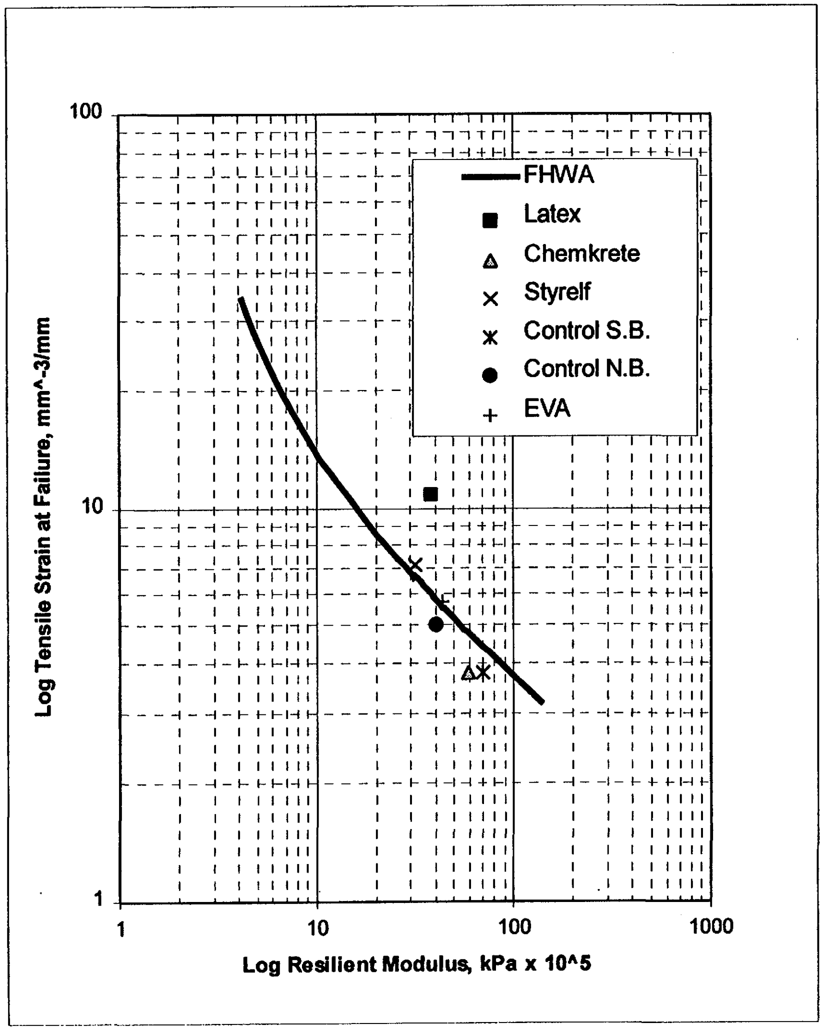

Figure 25. **Relationship Between Indirect Tensile Strains and Resilient Modulus for Texarkana <u>Base</u> Cores Using the Minimum Failure Strains Required for the FHWA Fatigue Relationship (<u>8</u>).**

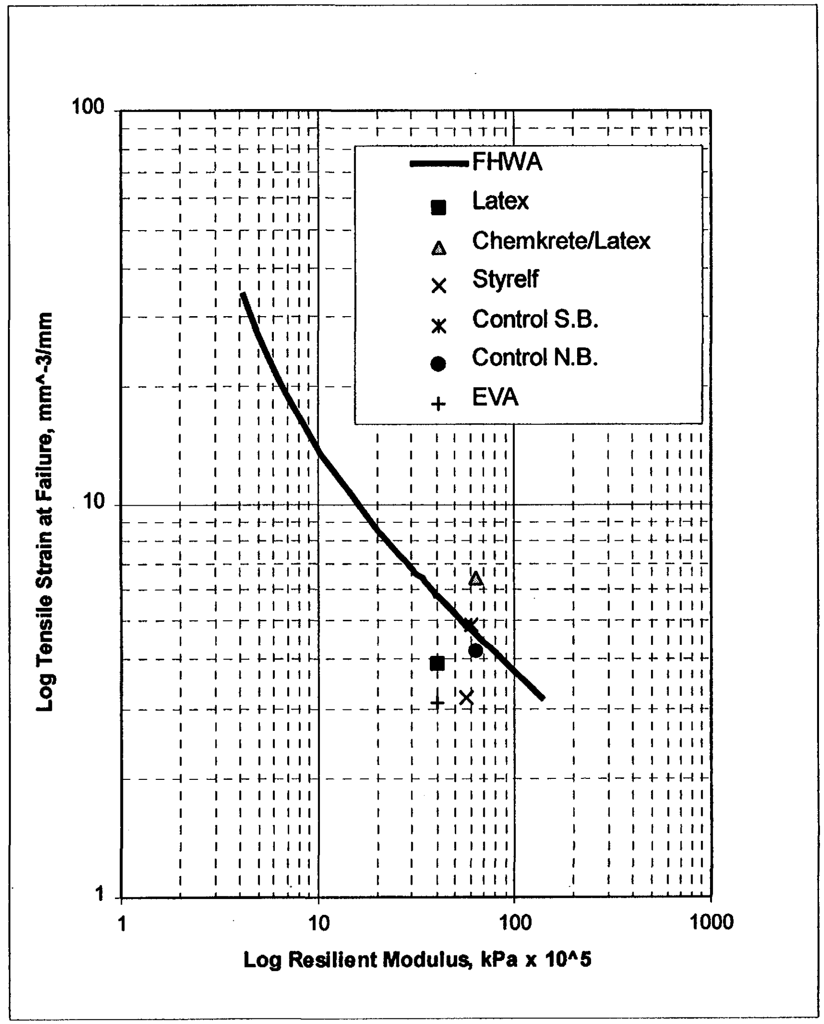

Figure 26. **Relationship Between Indirect Tensile Strains and Resilient Modulus for Texarkana <u>Surface</u> Cores Using the Minimum Failure Strains Required for the FHWA Fatigue Relationship (<u>8</u>).**

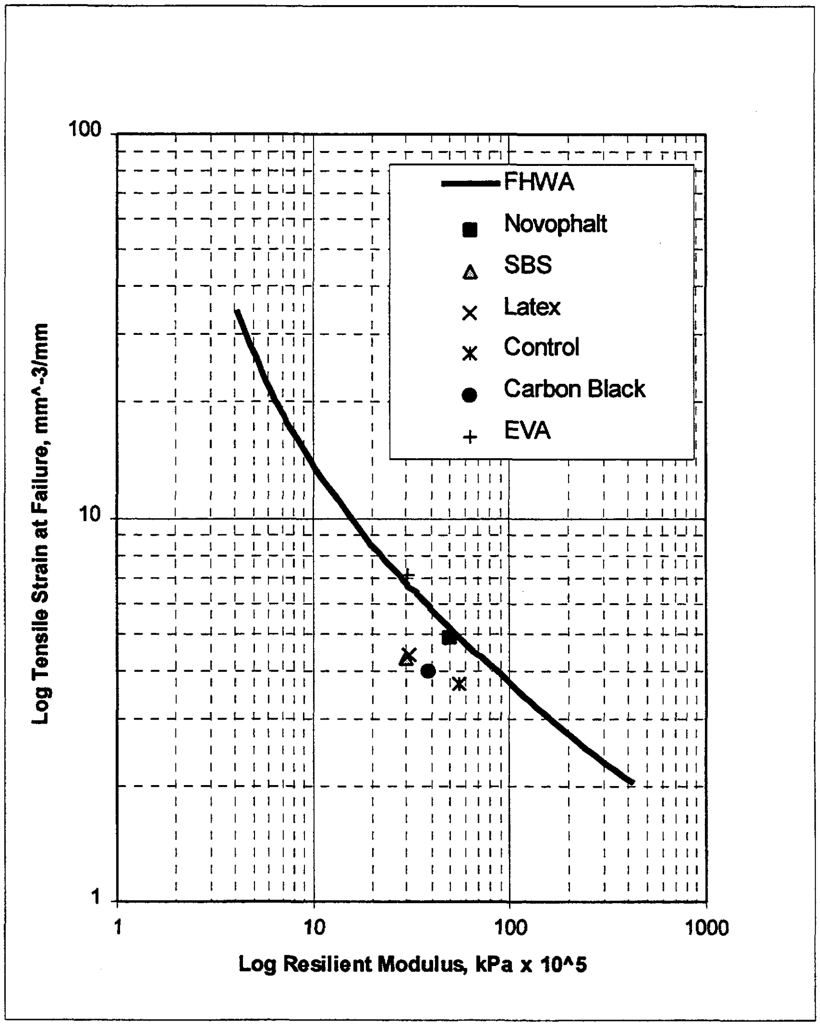

Figure 27. Relationship Between Indirect Tensile Strains and Resilient Modulus for Sherman Cores Using the Minimum Failure Strains Required for the FHWA Fatigue Relationship (8).

Table 16. Australian Frequency Sweep Pavement Life Predictions Using Field Cores.

Texarkana	Number of loads to failure, Nf x 10^6
Latex	05.43
Chemkrete	3.96
Styrelf	1.21
Control S.B.	0.60
Control N.B.	4.30
EVA	40,000,000
Sherman	
Novophalt	182.
SBS	3.40
SBR	7.75
Carbon Black	1.79
Control	8.80
EVA	84.1

APPLICATION OF ADDITIVES IN HIGHWAY PRACTICE

GENERAL

Asphalt pavements comprise more than 95% of the U.S. highway system. This is big business. The number of asphalt modifiers on the market, and the use of modified asphalts has increased remarkably during the last 15 years. Every few years, a new slate of additives and admixtures emerge. Many of these old and new products offer improved performance when properly introduced in properly designed paving mixtures. However, improper selection, testing, analysis, and design could easily reverse the potential benefits. Without question, the industry's asphalt binder and paving mixture test methods, standards, and specifications, as well as mixture design procedures and QC/QA provisions, must be capable of handling modified asphalts.

The concept of asphalt additives is logical and findings from laboratory testing of many different types of additives look positive. Even though field evaluations are incomplete, engineers responsible for pavement quality are willing to gamble because the odds appear to be in their favor. The asphalt modifier industry and associated technology is advancing at a comparatively rapid rate. By the time results from the field are available for the additives being currently marketed, it is reasonable to assume that a whole new generation of asphalt modifiers will be on the market. It is postulated that, in the future, the use of asphalt modifiers is likely to increase. Therefore, the industry must have performance-related laboratory test protocols by which modified asphalt binders and mixtures can be reliably evaluated so that cost effectiveness of alternative pavement construction and rehabilitation techniques can be compared.

Asphalt modifiers are sometimes applied to "make a better pavement." Indiscriminate use of additives may well be self defeating. The optimum type and dosage of modifier as well as the optimum grade of base asphalt *should* depend on the anticipated problem in a particular situation (30, 2, 31). If the anticipated pavement problem is cracking, and the mixtures normally used are rut resistant, then the polymer additive should be incorporated into a softer than usual asphalt. The soft asphalt remains relatively flexible at low temperatures and thus acts to retard thermal and fatigue cracking while the polymer helps protect the mixture from rutting at high temperatures. However, if low-temperature cracking has not been a problem, and the anticipated pavement problem is rutting, then the polymer additive should be incorporated into the asphalt grade normally used in the particular area. The addition of the polymer should not have negative effects on low-temperature cracking, but will help offset rutting problems.

Guidelines or specifications are needed that will enable the pavement designer to select the particular modifier (or a narrowed list of candidates) and the grade of asphalt that have the highest probability of ameliorating his particular problem without causing another problem.

The justification for using an additive falls into 1 or both of 2 categories: (1) solves or alleviates a pavement problem which is likely to occur in the area in which normal paving mixtures are used, or (2) produces an economic, environmental, energy, construction, or performance benefit. In either case, the improvement must ultimately be cost effective. The question is, "Will the additive solve or reduce my anticipated pavement problem?" To reliably answer this question, one must understand the problem, its cause, the alternative treatments that might be available (including additives), and he must be able to effectively match treatments with problems (4, 5).

Modifiers are used to improve on shortcomings of asphalt cement as a pavement binder. Binder shortcomings may lead to fatigue cracking, low-temperature cracking, rutting, water susceptibility, age hardening, and/or flushing. These modified materials are employed in an attempt to cost effectively improve performance of asphalt paving mixtures. With the advent of performance-based specifications from the Strategic Highway Research Program (SHRP), it appears that modified binders will have a place in the asphalt paving industry for the foreseeable future. For some highways in certain areas, unmodified asphalts will not meet the SHRP requirements. In fact, to gain an acceptable level of reliability for a mixture design developed for a high-volume highway in a severe climate, modified asphalt binders are imperative to meet the SHRP specifications (32). When the sum of the absolute value of the 2 numbers comprising the Superpave performance grade equals 86 or more, an additive is normally required to meet the specifications.

SUPERPAVE PRACTICE FOR MODIFIER EVALUATION

The Superpave practice for modifier evaluation provides the means to do the following:

- pinpoint the need for modifier use during the mixture design process;
- estimate the performance capacity of modified asphalt binders and paving mixes under specific climatic and traffic conditions;
- perform simple cost comparisons of modified versus unmodified asphalt binders and paving mixes over extended periods of service; and
- suggest an appropriate modifier for a given situation.

SHRP report A-410 (32) states that a distinct practice for modifier evaluation is unnecessary in the Superpave system since its specifications, test methods, and mixture design system can be applied to any asphalt material, without regard for details of modification. Decisions on the use of a modifier can be made by weighing performance estimates against projected costs.

The Superpave system for modifier evaluation provides a complete procedure for the measurement of the performance characteristics of modified asphalt binders and paving mixtures for hot mix asphalt pavements. It uses performance-based specifications and laboratory binder and mixture tests and the performance models incorporated in the Superpave software. Materials can be evaluated as a single component and/or multiple phase system in the final paving mixture produced for roadway placement. The same performance criteria are used for both unmodified and modified materials.

The practice provides guidance or specific test methods for special properties or characteristics of modifiers or modified asphalt binders such as purity, toxicity, storage, stability, and compatibility. The practice can be used as an integral part of the overall Superpave mixture design system, or alone, to compare the effectiveness of different modifiers, to evaluate novel materials, or to relate the expected benefits of a modified system to its incremental costs.

The practical use of the current Superpave binder specifications in pinpointing the need for modification of binders is seen in the fact that, within the high-temperature range, only a very small percentage of the 42 unmodified asphalt binders evaluated in the SHRP program are adequate for use in areas of the United States where PG-70 grades are needed (Figure 28). Within the low temperature range, the available unmodified asphalt binders are satisfactory for preventing the development of low-temperature cracking only in those areas of the United States and Canada where winter pavement temperatures do not fall below -28°C (i.e., PG-28 grade) (Figure 29). If the Superpave software recommends the use of a performance grade binder outside of these ranges, an appropriate modified asphalt binder must immediately be considered by the material supplier or HMA contractor.

CURRENT NATIONAL STUDY OF ASPHALT ADDITIVES

Pavement engineers need materials specifications, acceptance criteria, and mixture design protocols to include laboratory and field test procedures for quality control and quality assurance that are practical and yet relate to actual performance of the pavement constructed in the field. SHRP has produced the most comprehensive and practical tools to meet these needs that have ever been assembled.

However, SHRP research established test methods and specification limits primarily for neat asphalt cement binders as well as test protocols and performance prediction models for mixtures made using neat asphalt cements. These specification limits, prediction models, and test methods, including volumetric criteria, were established from limited correlations of field performance with laboratory measured properties of neat binders and mixtures made using the neat binders. There is concern that the SHRP Superpave binder and mixture tests may not be suitable for use with certain modified asphalt binder systems now on the market.

As a result, Superpave test protocols may need to be altered to allow characterization or to better characterize the modified asphalt binders and to permit accurate measurement of properties of hot mix asphalt containing modified asphalt binders, so that performance of mixtures can be optimized, and predicted with a reasonable degree of accuracy. Similarly, the specification limits and models developed in the SHRP research may need revision as data on field performance of modified asphalt binders and mixtures containing modified asphalt binders become available. Ongoing NCHRP Project 9-10, "Superpave Protocols for Modified Asphalt Binders," is addressing this problem.

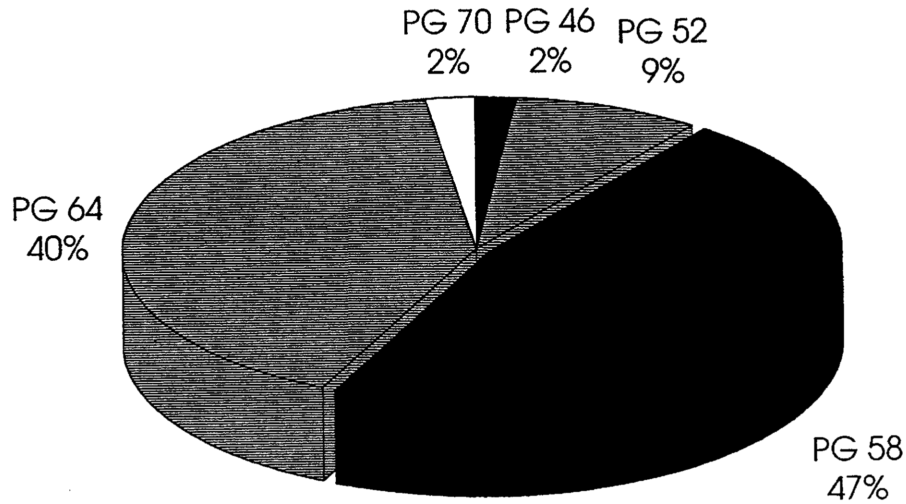

Figure 28. Distribution of Unmodified Asphalt Binders in the SHRP Materials Reference Library Within the High Temperature Performance Grades (AASHTO MP1) (After Reference 32).

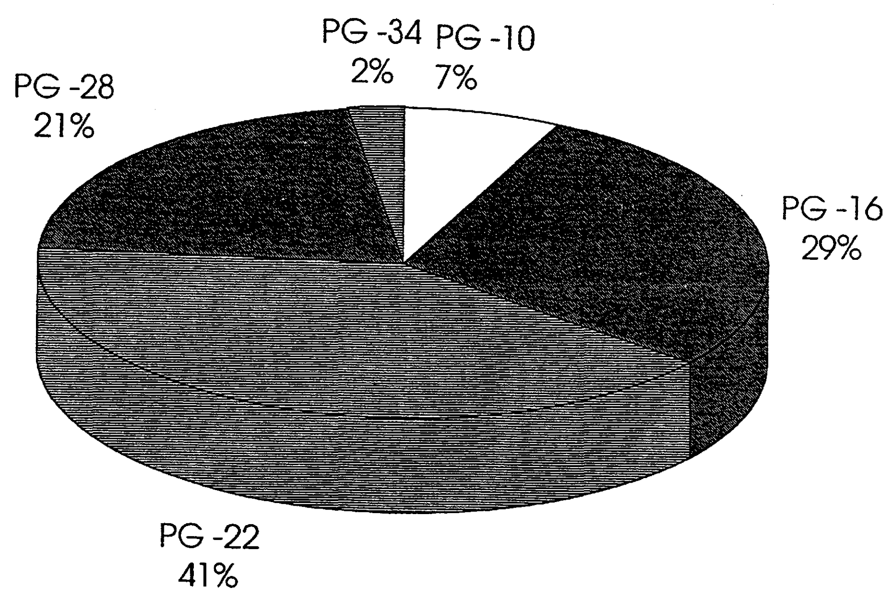

Figure 29. Distribution of Unmodified Asphalt Binders in the SHRP Materials Reference Library Within the Low Temperature Performance Grades (AASHTO MP1) (After Reference 32).

ASPHALT ADDITIVES AVAILABLE

A variety of materials and methods are used to modify asphalt cements. A list of 12 categories of modifiers and examples under each category are shown in Table 17. Some modifiers may actually involve aggregate treatments such as hydrated lime, phenolic resin, even certain polymers (33).

FACTORS RELATED TO ADDITIVE PERFORMANCE

Compatibility

Polymers vary widely in properties and in their effects on asphalt properties. They generally function in 1 of 2 ways in asphalt - as a separate, dispersed phase or as a part of the system when chemically reacted with the asphalt. Dispersed systems are the most common. Typically, these polymers absorb oil from the asphalt and swell to create a polymer network in the asphalt, but they remain a separate phase. Separation is usually a concern with these systems, and there may be compatibility problems between an asphalt and a particular polymer. Polymers that are reacted with the asphalt are less common. However, when the polymer and the asphalt are joined in a chemical reaction, separation is not a problem.

The compatibility of asphalt/polymer systems may be defined in several ways. It may be a thermodynamic definition which requires that a particular morphology be achieved, or it may be an optimization of physical properties based on a desired specification or property trend (31). In practice, it most often refers to storage stability. In all cases, it is directly related to the rheological properties of the binder (34).

Rheology

The additives listed in Table 17 can be divided roughly into 2 categories: those that change the viscosity and those that change the viscoelastic properties such that viscosity measurements alone are insufficient for characterizing the binder. In the former case, there would seem to be little need to change Superpave specifications. If an additive causes any property to change so that the binder fails specifications, the additive would be rejected. This conclusion could be overruled, however, if there were some verifiable improvement in mixture properties. For polymeric additives, especially elastomers, viscosity alone has less meaning, and the low temperature properties such as bending beam will likely be particularly significant.

Table 17. Bitumen Additives Currently Being Used or Tested in Pavements.

1. **Polymers**
   - a. Styrene Butadiene Rubber (SBR) (Latex)
   - b. Block Copolymers
     - i. Triblock Styrene-Butadiene-Styrene (SBS)
     - ii. Radial Block SBS
     - iii. Vulcanized (SBR)
     - iv. Styrene-Isoprene-Styrene (SIS)
     - v. Styrene-Ethylene-Butylene-Styrene (SEBS)
     - vi. Styrene-Ethylene-Propylene-Styrene (SEPS)
   - c. Polyethylene
   - d. Ethylene Vinyl Acetate (EVA)
   - e. Polypropylene
   - f. Crumb Tire Rubber (not included in Project NCHRP 9-10)
   - g. Polychloroprene latex
   - h. Polychloroprene solids
   - i. Natural Polyisoprene
   - j. Synthetic Polyisoprene
   - k. Ethylene Propylene-Diene-Monomer (EPDM)
   - l. Polyisobutylene
   - m. Ethylene/n-butyl acrylate/glycidyl methacrylate terpolymer

2. **Extenders**
   - a. Sulfur
   - b. Fillers

3. **Mineral Fillers**
   - a. Carbon Black
   - b. Hydrated Lime
   - c. Flyash
   - d. Amorphous Silica
   - e. Baghouse Fines

4. **Natural Asphalts**
   - a. Trinidad
   - b. Gilsonite

5. **Chemically modified asphalts**
   - a. Nitration
   - b. Halogenation
   - c. Certain polymers

6. **Antistripping Agents**
   - a. Amidoamines
   - b. Imidazolines
   - c. Polyamines
   - d. Hydrated Lime
   - e. Organo-metallics

7. **Antioxidants**
   - a. Pb/Zn Diethyldithio Carbamates
   - b. Viscosity Modifiers
   - c. Carbon Black
   - d. Hydrated Lime
   - e. Phenols

8. **Hydrocarbons**
   - a. Tall Oil
   - b. Aromatics
   - c. Naphthenics
   - d. Paraffinics/Wax
   - e. Petroleum/Plastic Resins
   - f. Asphaltenes

9. **Gelling Agents**

10. **Viscosity Modifiers**

11. **Catalysts** (e.g., Chemkrete)

12. **Fibers**

The Superpave binder testing protocol calls for a combination of conditioning and property measurement steps (Figure 30). First, the binder is tested in an unaged condition to assess the likelihood that it will produce a tender mix. This is assessed using a dynamic shear rheometer (DRS) $G^*/\sin\delta$ measurement. Second, the binder is subjected to a short-term aging test (TFOT) to simulate the binder aging that would occur in the hot-mix plant and then tested for stiffness to assess the likelihood that permanent deformation would occur in use shortly after construction (again, DSR, $G^*/\sin\delta$). Third, the binder is subjected to a high-temperature, high-pressure aging procedure designed to simulate the aging that would occur over extended periods of time in pavement use. Finally, it is a) tested for binder fatigue at moderate temperatures as an indication of a tendency to fail due to fatigue cracking (DSR, $G^* \sin\delta$) and b) tested at low temperature to obtain a stiffness (S) and the slope of creep stiffness curve (m) to determine the likelihood that the material will fail due to low-temperature cracking and thermal fatigue (bending beam rheometer [BBR], S, and m). Furthermore, the direct tension failure test at low temperature on this long-term aged material may be performed to provide additional information about the likelihood of low-temperature cracking failure.

The methodology of the binder tests is certainly very reasonable, as the rheological tests themselves are performance based. The tenderness and permanent deformation tests which are based on the $G^*/\sin\delta$ parameter have been related to laboratory rutting tests conducted on compacted mixtures. These results have been obtained for both original asphalts and modified asphalt binders. The results are not definitive, however, and need further validation. Nevertheless, it is reasonable that a higher G^* (a higher stiffness) at the maximum pavement temperature and a greater elasticity (smaller $\sin\delta$) will each contribute to a reduced tendency for the binder to deform under load and, therefore, should signify a binder that is less susceptible to rutting. For a viscous material (newly-placed conventional asphalts at maximum pavement temperatures), δ is close to 90° and, so, $\sin\delta$ is close to 1. $G^*/\sin\delta$ then is approximately equal to G^*, which, at those temperatures, is directly proportional to the viscosity of the material. As an elastic component is added to the binder, through modification for example, δ decreases from 90° and $\sin\delta$, therefore, decreases to less than 1 resulting in an increase in $G^*/\sin\delta$. Hence, the material with an equivalent viscosity but a greater elastic component will tend to rut less.

$G^*\sin\delta$ is used to provide a measure of dissipative energy to failure as an indication of a binder's susceptibility to fatigue cracking. This property is measured at moderate temperatures. The specification requiring a maximum value of 5,000 kPa at the desired intermediate performance grade temperature assumes that all binders that reach this level of aging in the PAV will not perform satisfactorily on the road.

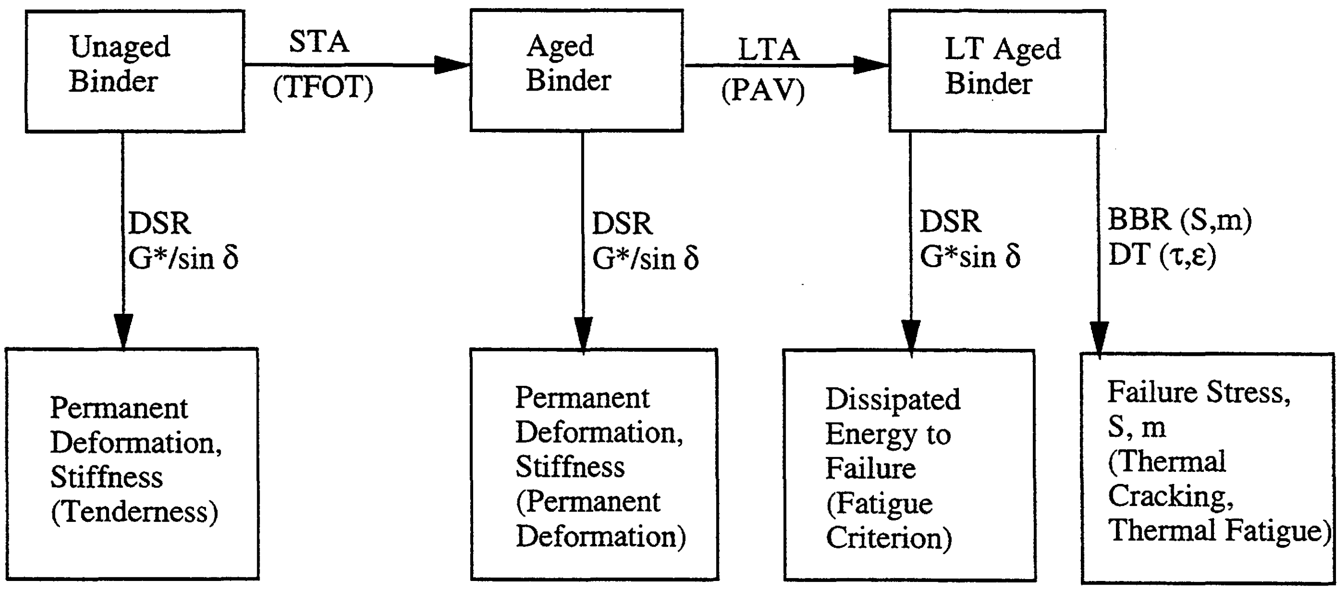

Figure 30. Superpave Binder Test Protocol.

The bending beam stiffness criterion for low-temperature cracking is based upon the notion that all asphalt binders behave essentially the same with respect to failure and stress, i.e., they have the same failure stress at a given temperature. If this is true, then a higher value of stiffness at low temperatures (where the material behaves more like an elastic solid but with creep) results in a lower value of strain at failure (Figure 31). Thus, for an asphalt to accommodate a strain of 1% without failure, the stiffness must be less than the 300 MPa according to the Superpave specification. The direct tension test, when used, is a direct measure of this failure stress-strain behavior.

It should be noted that these specifications also automatically take into account the temperature susceptibility of a binder by setting criteria simultaneously at high, intermediate, and low temperatures (Figure 32). Excessive temperature susceptibility results in missing the specification at one end.

While the Superpave binder specifications are reasonable in principle, they are based upon a set of assumptions which are not necessarily true for modified binders, and must be verified for these systems.

The first assumption is alluded to in the above discussion, i.e., that all binders should have the same failure criteria at a given temperature, even though they may be fundamentally different materials. Is there a fundamental reason that all modified binders should possess the same failure limits on permanent deformation, fatigue cracking, and failure stress as conventional materials? The authors suspect that this assumption will be valid for some of the modified binders, but will be drastically in error for others. For example, the presence of fine, discrete particles (polymer or other material) of the appropriate size may well serve as crack arrestors, thereby inhibiting the propagation of cracks within the binder (35, 36) (e.g., Novophalt in Figures 3 and 4). This would result in a higher failure stress and consequently in a higher acceptable stiffness value at low temperature than for the conventional binders. As a second example, modified binders may be expected to behave differently from conventional asphalts with respect to compatibility of the binder components. While this is of some concern in neat asphalts, it will be of even more concern for modified binders. Undoubtedly, phase transitions and structuring would impose unique challenges to Superpave binder specifications that need to be investigated.

Aging

The second fundamental assumption of the Superpave protocol, if accepted as is for modified binders, is that the aging steps, both short-term and long-term, are valid for the modified binders. They probably are not. For example, it is quite problematical that the short-term aging step, the RTFOT (AASHTO T240) or TFOT (T179), will accurately represent hot mix aging for at least some of the modified materials. TFOT and RTFOT simulate hot mix plant aging fairly well for neat asphalts, but, due to different chemical kinetics, asphalt-additive interactions could be strongly enhanced by the much longer time of the oven tests relative to that of a hot mix plant. In the case of a dispersed material as the modifier, the much longer oven aging times give much greater times for diffusion effects to occur within the particles and, thus, for the asphalt components to interact with the

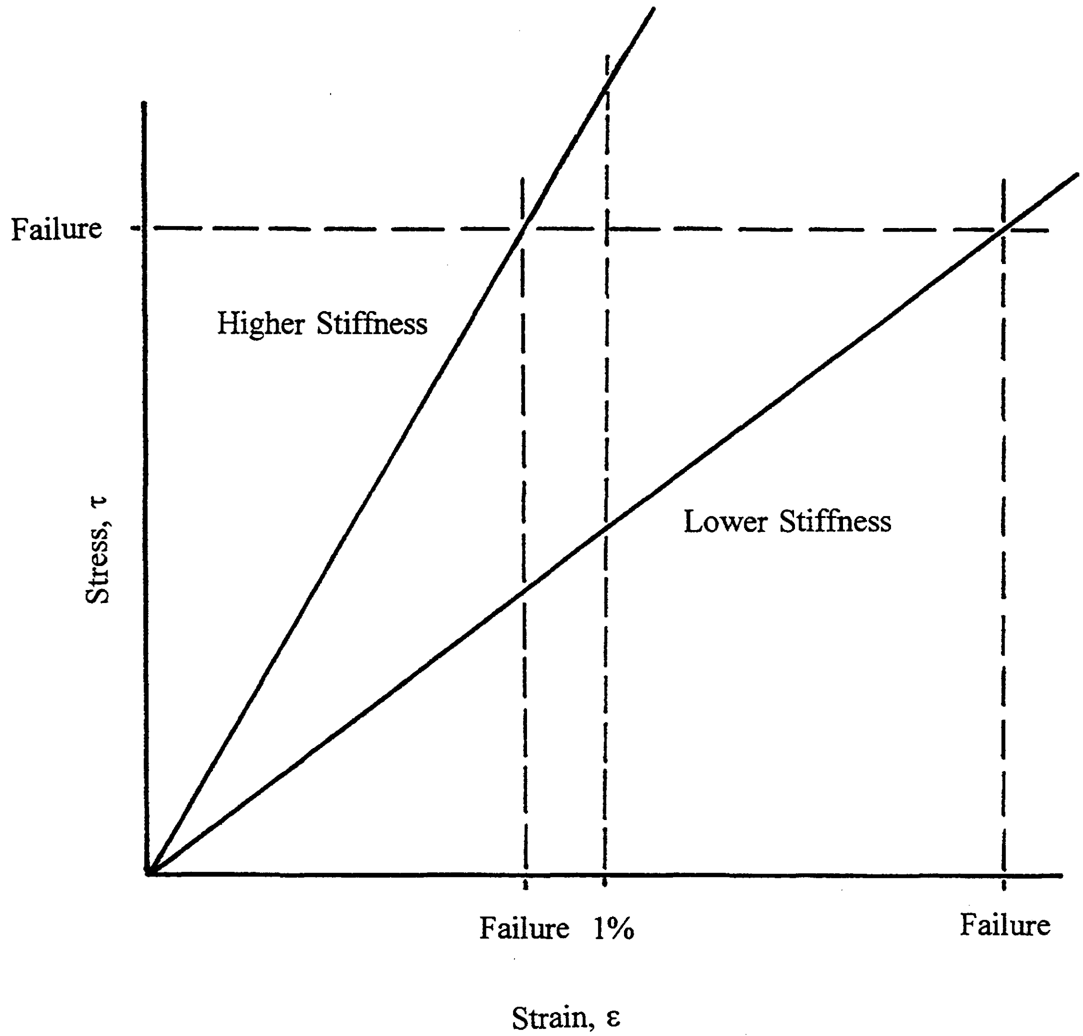

Figure 31. Stress, Strain, Stiffness, and Failure Relationships for Bending Beam Test.

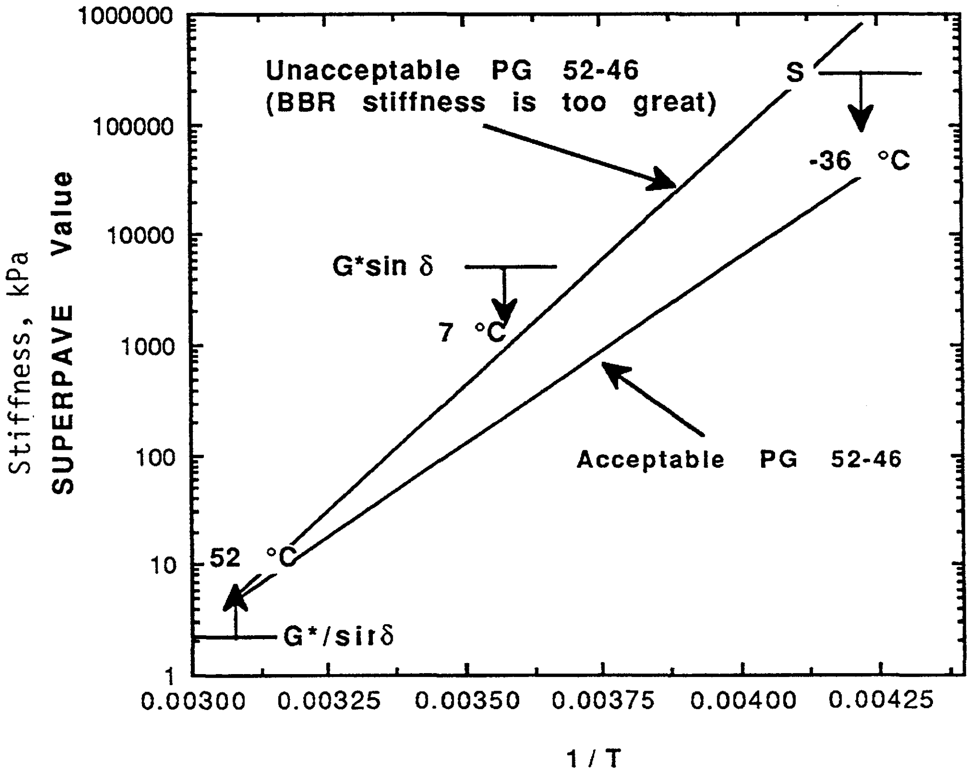

Figure 32. Illustration of Temperature Susceptibility Limitation Built into Superpave Specification.

modifier components. It is anticipated that, in some cases, the modified material after the RTFOT would not be at all like the binder leaving the hot mix plant.

With respect to long-term aging, the ability of the Superpave binder specifications to predict performance rests directly on the ability of the PAV aging procedure to provide a binder with the same properties as the pavement aged materials and at a predictable time in the future. When the modifying components are essentially inert to oxidation, as they will be for some of the cases in Table 17, the aging results for the unmodified binder should be about the same as those for the modified binder. However, some modifiers will be affected by oxidation and at different rates from the asphalt components. In these cases, the relative rate of oxidation of additive and asphalt could be very different at PAV conditions than at road conditions.

Crack Pinning

The real difference that modified asphalt binders will make in the performance of pavements is in their increased resistance to fatigue, thermal, and reflection cracking and their reduction of rutting. There are 3 principal methods by which cracking can be arrested:

1. The modifier is combined or dissolved in the binder and changes the binder properties to be more compliant;
2. The modifier increases the bond between the asphalt binder and the aggregate surfaces, and thus reduces adhesive fracture; and
3. The modifier provides small particulate or fibrous material well dispersed in the asphalt binder and thus provides "crack stoppers" or "crack pinning" to suppress the growth of micro cracks (37).

Testing that has been conducted at TTI (Texas A&M) on the effects of micro cracking and healing has demonstrated that the mean size and the size distribution of micro cracks can be determined as well as the micro crack fracture properties. The analysis methods that have been developed are complex, taking into account the fact that the binder retains a memory of previous loading cycles. Analysis methods have been successfully applied to tensile trapezoidal and haversine loading functions. All of this means that it is now possible to determine, by testing companion binders or mixes, the effect of the type and quantity of modifiers on the fracture properties of the binders and mixes, and on the sizes and growth of micro cracks. This permits a direct comparison of the effects of the modifiers.

Testing with the haversine loading pattern, as applied by the Australian Materials Testing Apparatus (MATTA), is capable of measuring tensile axial stress and strain and radial strain very precisely. During repeated cyclic loading, micro cracks will form and grow and this growth can be monitored by recording the dissipated energy with each load cycle. The method of analyzing these data that has been developed by TTI can then compute the crack growth rate and fracture properties of the binder or mixture. This device is being evaluated in NCHRP Project 9-7 for QA/QC testing.

Rutting results largely from shear strain. The shear properties of a mixture in a pavement are altered and the strength and stiffness are reduced by micro damage in the binder and in the mix. By reducing the growth rate of micro cracking, a modified binder can actually reduce rutting to some limited degree. The shear stiffness and slope of the log stiffness versus log frequency curve, which can be measured in the MATTA, will give a reliable indication of how these material properties are altered by different types and quantities of modifiers.

Crack Retardation

The cracking of an asphalt layer always occurs in 2 stages which have been called crack initiation followed by crack propagation. It was found in the SHRP A-005 project that the "crack initiation" phase is when micro cracks form, grow, and coalesce. The second phase starts when the cracks have grown large enough to propagate. Cracking can be retarded by delaying the crack growth in either phase, but the most efficient phase is during "crack initiation" when micro cracks are forming and healing.

Both processes are important. Some modifiers may retard the micro crack growth by "crack stopping" or "crack pinning." These modifiers provide small particulate or fibrous inclusions in the binder. Other modifiers will have a greater effect upon the healing properties of the binder or the mix. These modifiers raise the compliance of the binder or increase its bond to aggregate surfaces, or both. The healing process is enhanced when soft asphalts are mixed with appropriate modifiers. The soft asphalt promotes healing while the modifier protects from rutting at high temperatures.

CONCLUSIONS AND RECOMMENDATIONS

Samples of pavement mixtures from laboratory and field evaluation were obtained from Texarkana and Sherman. Several test procedures were used to evaluate the binder characteristics of the mixtures: (1) DSR, (2) SHRP Binder Tests, (3) Asphaltene content (using heptane), FT-IR and GPC tests, (4) AAMAS style testing, and (5) Australian Frequency Sweep testing.

CONCLUSIONS

The following conclusions were reached as a result of this study:

- DSR testing indicates that loss tangent values correlate well with observed cracking. High loss tangent at low testing temperatures indicates good resistance to fatigue cracking.

- DSR test results could not be related to reflective cracking which, apparently, is controlled more by differential movements in the underlying pavement than by the properties of the binders in the overlay.

- Binders which failed the Superpave high temperature grading did not produce rut-susceptible mixtures, but did produce crack-resistant mixtures. Therefore, SHRP binder test results provided evidence that rutting does not relate to binder properties when high quality aggregate is used.

- Asphaltene content measurements (using heptane) indicate that asphalts with asphaltene contents greater than 12.5% are susceptible to fatigue cracking. Asphaltene content could not be related to reflective cracking.

- FT-IR testing indicates the pavements with highly oxidized binders (carbonyl growth greater than 1.40) are susceptible to fatigue cracking. FT-IR test results could not be related to reflective cracking.

- GPC test results indicate that asphalt binders with high amounts of large molecular size (LMS) material (greater than 22%) are susceptible to fatigue cracking. GPC testing could not be related to reflective cracking.

- AAMAS style testing showed no correlation with the observed fatigue or reflective cracking performance.

- Australian Frequency Sweep testing showed no correlation with the observed fatigue or reflective cracking performance.

RECOMMENDATIONS

The following are recommended:

- Asphalt modifiers are sometimes incorporated to "make better pavement." Indiscriminate use of additives may well be self defeating. The optimum type and dosage of modifier as well as the optimum grade of base asphalt *should* depend on the anticipated problem in a particular situation. If the anticipated pavement problem is cracking, and the mixtures normally used are rut resistant, then the polymer additive should be incorporated into a softer than usual asphalt. The soft asphalt remains relatively flexible at low temperatures and thus acts to retard thermal and fatigue cracking while the polymer helps protect the mixture from rutting at high temperatures. However, if low-temperature cracking has not been a problem, and the anticipated pavement problem is rutting, then the polymer additive should be incorporated into the asphalt grade normally used in the particular area. The addition of the polymer should not have negative effects on low-temperature cracking, but will help offset rutting problems.

- The Superpave asphalt paving mixture design process provides guidelines for selecting the appropriate PG asphalt. The Superpave process allows the designer to take into consideration climate and traffic conditions. However, other factors that should be considered are:

  a. type and condition of substrate - for example, is substrate a new flexible base or an old cracked pavement;
  b. placement in pavement structure - the temperature extremes of a base layer will be significantly less than those of a surface layer;
  c. past performance of asphalt from the given supplier - what are its shortcomings and/or attributes, and to what extent can an additive(s) help; and
  d. number of tanks contractor has available for storage of different binders.

- Use the findings of this research along with related work by other agencies to upgrade the Superpave specification at the national level.

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36. Rodriguez, M.G., Morrison, G.R., VanLoon, J.R., and Hesp, S.A.M., "Crack Pinning in Particulate-Filled Asphalt Binders and Asphalt Concrete Mixes," Queen's University, Kingston, Ontario, paper submitted to AAPT for publication, 1995.

37. Rodriguez, M.G., Morrison, G.R., VanLoon, J.R., and Hesp, S.A.M., "Low Temperature Failure in Particulate-Filled Asphalt Binders and Asphalt Concrete Mixes," Journal of the Association of Asphalt Paving Technologists, Vol. 65, 1996.

APPENDIX A

ADDITIONAL CONSTRUCTION INFORMATION

Table A1. Summary of Field Project in the Atlanta District North of Texarkana.

General Information

Highway Designation	US 59/71
County	Bowie
Control Section No.	0217-01-018
Construction Project No.	MA-F 472(3)
No. Lanes in each Direction	2
Dates of Construction	
Base (Ultrapave & Chemkrete)	July, 1987
Base (Styrelf & Polybilt)	October, 1987
Surface (all)	May, 1988
Type of Construction	New Construction (Northbound) Reconstruction (Southbound)
Pavement Structure	51 mm ACP Type D (9.5 mm max)
Layer 1 (top)	203 mm ACP Type B (22 mm max)
Layer 2	457 mm lime-flyash treated
Layer 3	subgrade

Asphalt Paving Mixtures

	Base Course	Surface Course
Asphalt Source		
Ultrapave Latex	Fina AC-10	Fina AC-10
Chemkrete-CTI 102	MacMillan AC-20	Fina AC-10 + latex
Polybilt 102 EVA	Lyon AC-20	Lyon AC-20
Styrelf	Exxon	Exxon
Control (no additive)	MacMillan AC-20	MacMillan AC-20
Quantity Additive in Asphalt Cement		
Ultrapave Latex	3.0%	3.0%
Chemkrete	2.0%	3.0% + latex*
Polybilt EVA	3.5%	3.5%
Stryelf	3.0%	3.0%
Control	0	0

* Chemkrete was replaced with latex in the surface mix.

Table A2. Traffic and Environmental Data for Test Site in the Atlanta District - Texarkana.

<u>Traffic Data</u>	
ADT (1985 & 2005)	8,800/13,000
Trucks in ADT, percent	15.2
ATHWLD	12,900
Tandem Ales in ATHWLD, percent	60
Equivalent 18-kip axle loads expected 1985 to 2005	5,670,000
Speed limit, kph	89
<u>Weather Data</u>	
Climate	
Temperature	
Mean Max, °C	24
Mean Min, °C	12
No. Days/yr 32°C & above	64
No. Days/yr 0°C & below	44
Sharp drops	Yes
Frost Penetration, mm	51
Freeze index	0
Precipitation	
Mean annual precipitation, cm	115
Mean annual ice/snow, cm	10

Table A3. Construction Notes from the Atlanta District - Texarkana Test Pavements.

Type of Additive	Method of Incorporating Asphalt Additive	Remarks
None (Control Mix)	Not applicable	Plant temperature was about 149°C. ALL mixes experienced some minor segregation.
Chemkrete	Blended on site using in-line mixer supplied by LBD Asphalt Products Co.	Plant temperature was about 149°C.
Goodyear 5812	Blended at refinery in Port Arthur, TX and shipped to construction site.	Mix sticky and difficult to place at 177°C, lowered to 154°C and eliminated problems. Mix stuck to pneumatic roller at tempertures above 71°C.
Styrelf 13	Blended and reacted at plant in Baytown, TX and shipped to construction site.	Mix stuck to pneumatic roller tires if rolled too hot. Plant temperature was about 154°C.
Polybilt 102 EVA	Blended on site for 30 minutes in a low shear batch-type mixer at 163°C by Cox Paving Co.	Not much different from mixing and compacting control mix. Plant temperature was about 163°C.

Table A4. Summary of Texas Field Projects in the Paris District - Sherman.

Item	South of Sherman
General Information	
Highway Designation	U.S. 75
District Number	1
County and Number	Grayson (92)
Control-Section Number	0047-13
No. of Lanes each Direction	2
Existing Pavement	
Layer 1 (Top)	203 mm CRCP
Layer 2	152 mm Flex Base/ Lime
Layer 3	152 mm Subgrade/Lime
Construction Project No.	CSR 47-13-11
Date of Construction	Oct-86
Type of Construction	HMAC* Overlays
Construction Sequence	Sealcoat+50 mm Type B + Test Pavement
Description of Test Pavements	
Mix type	Type C
Asphalt Source	Total Asphalt Co.
Asphalt Type & Grade w/Additives	AC-10
Asphalt Type & Grade Control	AC-20
Aggregate Type	Crushed Limestone and Field Sand
Antistrip Additives	1/2% Pave Bond LP
Test Pavement Thickness	76 mm
Control Pavement Thickness	76 mm and 102 mm
Asphalt Additives Tested	
Carbon Black	Microfil-8
Ethylene Vinyl Acetate	Elvax 150
Polyethylene	Novophalt
SBR Latex	Ultrapave
SBS Block Copolymer	Kraton D

Table A5. Traffic and Environmental Data for Sherman Test Sites.

<u>Traffic Data</u>	
ADT (1985 & 2005)	17,700/28,800
Trucks in ADT, %	17.1
ATHWLD	13,100
Tandem Axles in ATHWLD, %	80
Equivalent 18-kip axle loads expected 1985 to 2005	21,377,000
<u>Weather Data</u>	
Climate	Humid, subtropical with hot summers
Temperature	
Mean and Record Max, °C	36/43
Mean and Record Min, °C	0/-19
No. Days/yr 32°C & above	94
No. Days/yr 0°C & below	55
Sharp drops	Yes
Frost Penetration, cm	2.5
Freeze Index	0
Precipitation	
Mean annual precip, cm	102
Mean annual ice/snow, cm	13

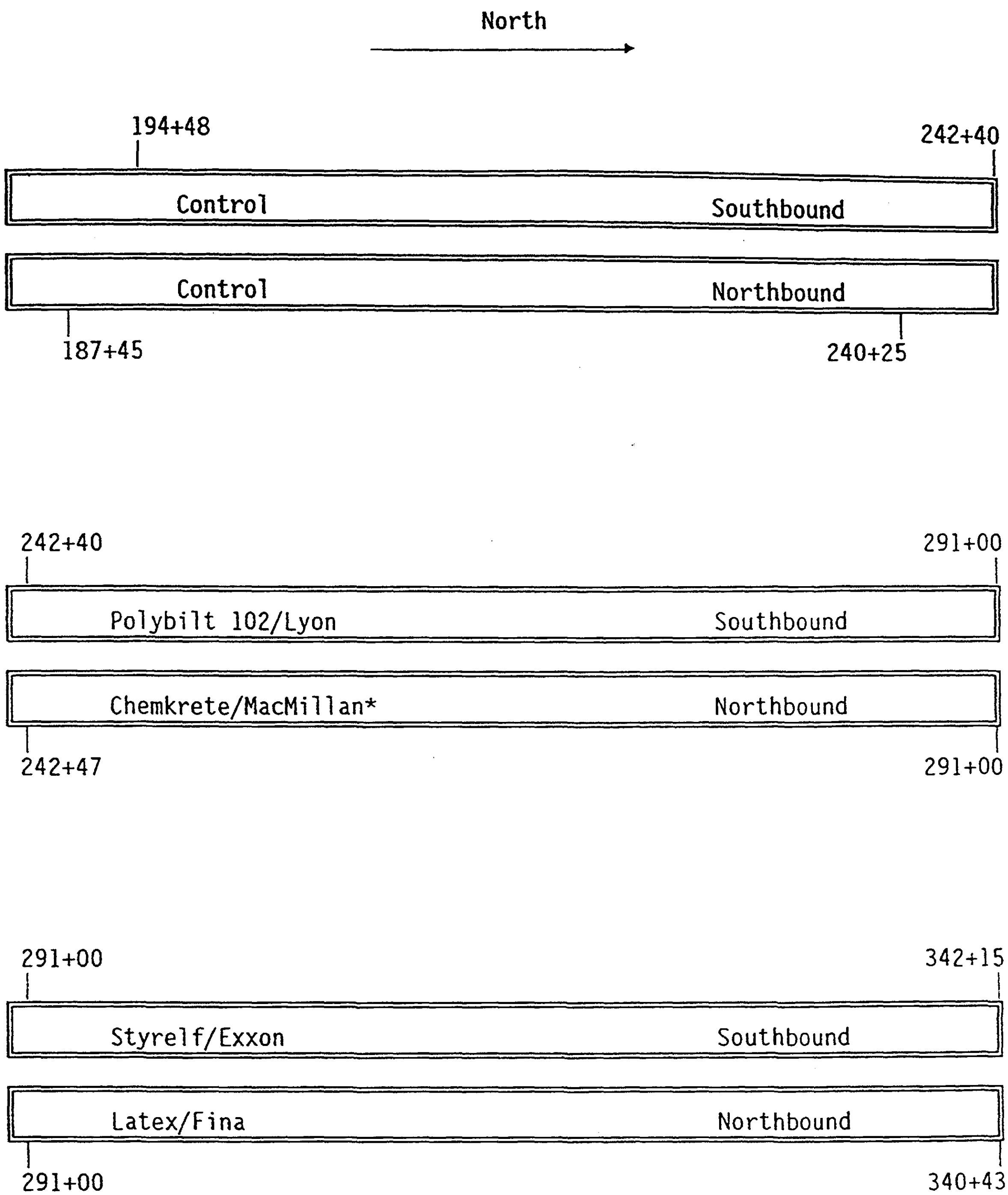

Figure A1. Schematic Showing Texarkana Test Pavement Locations.

*Chemkrete/McMillan was replaced with Latex/Fina in the surface mixture.

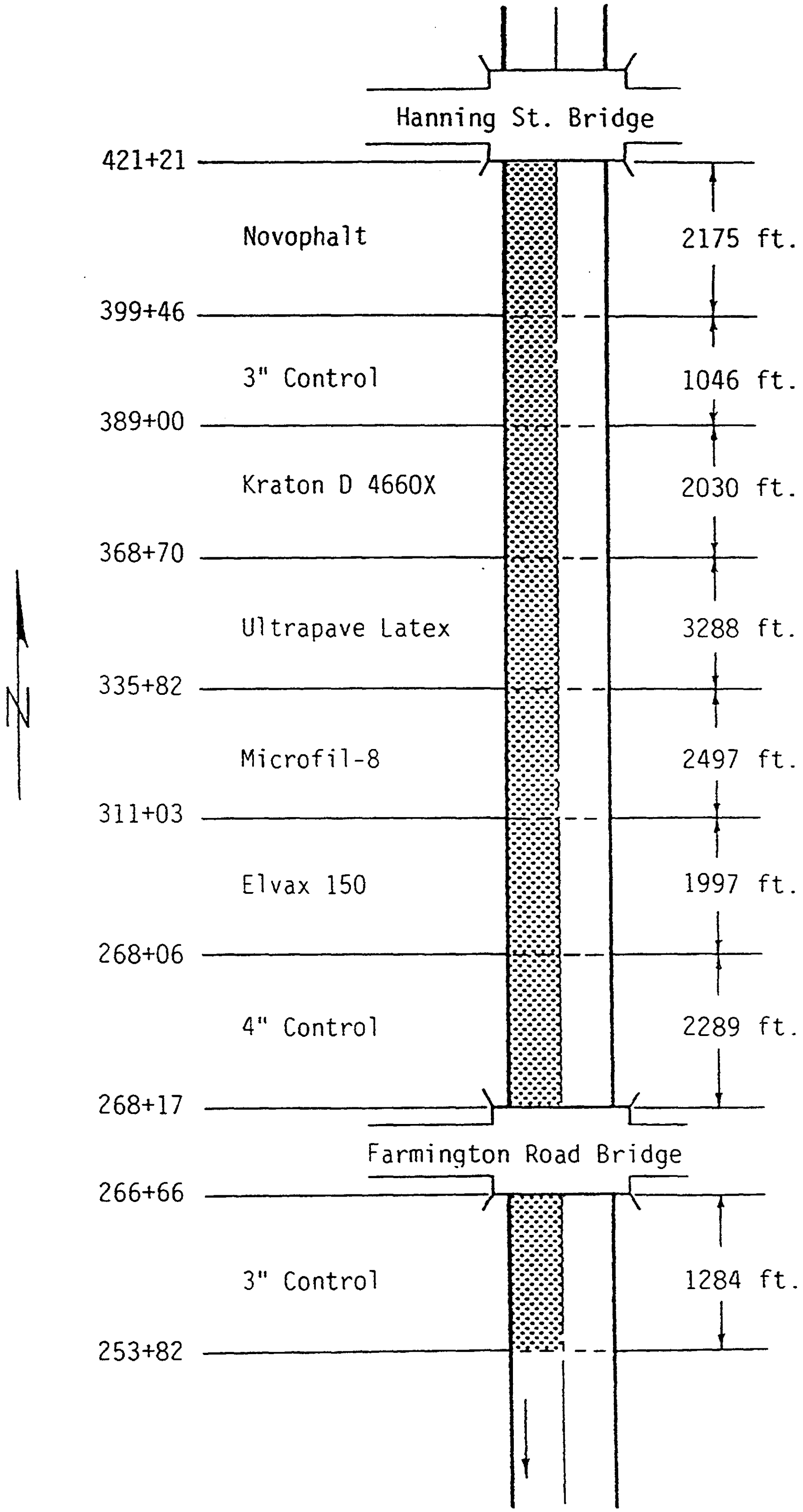

Figure A2. Schematic Showing Sherman Test Pavement Locations.

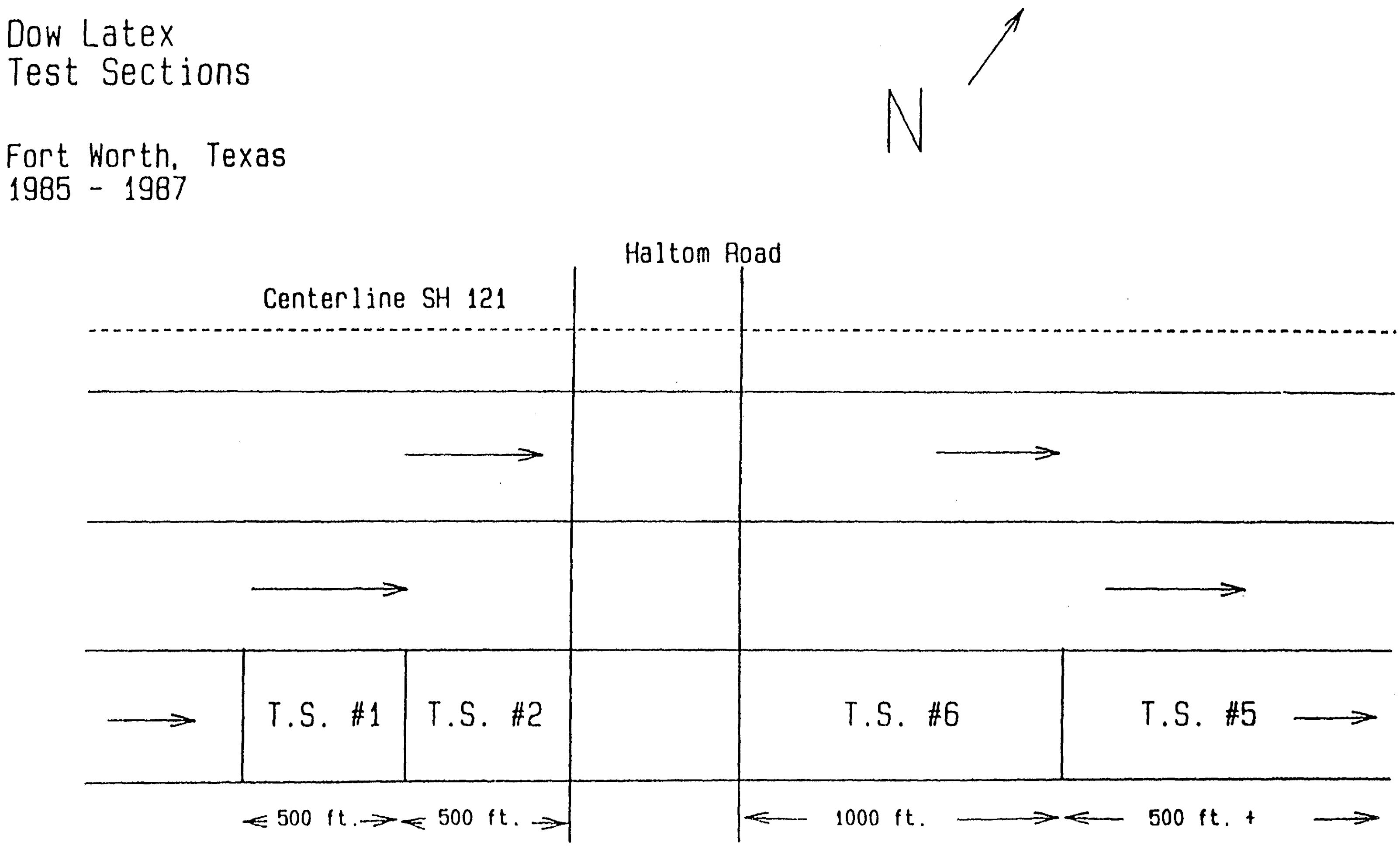

Figure A3. Schematic Showing Ft. Worth Test Pavement Locations.

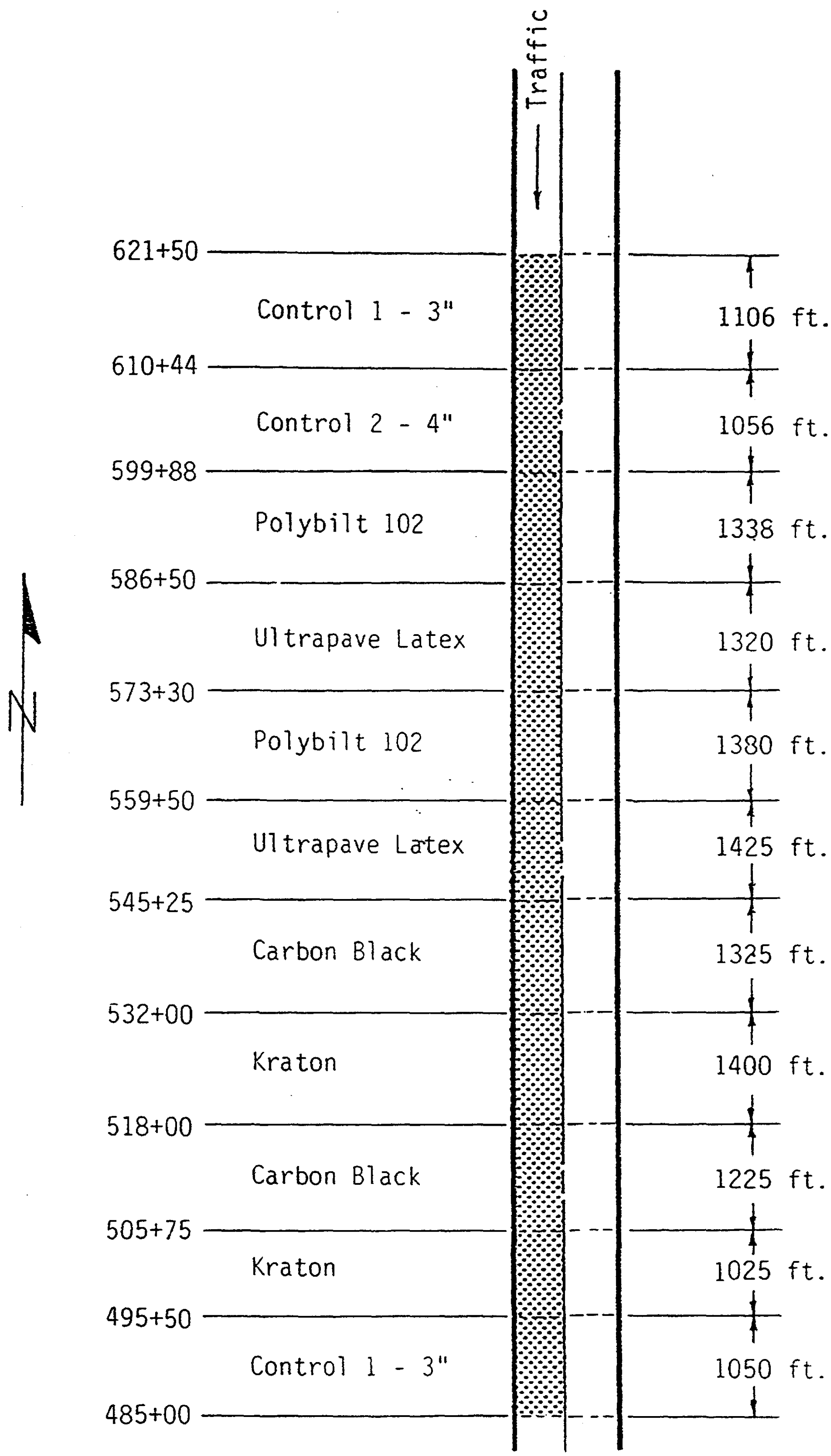

Figure A4. Schematic Showing San Benito Test Pavement Locations.

APPENDIX B

ADDITIONAL CHEMICAL TESTING INFORMATION

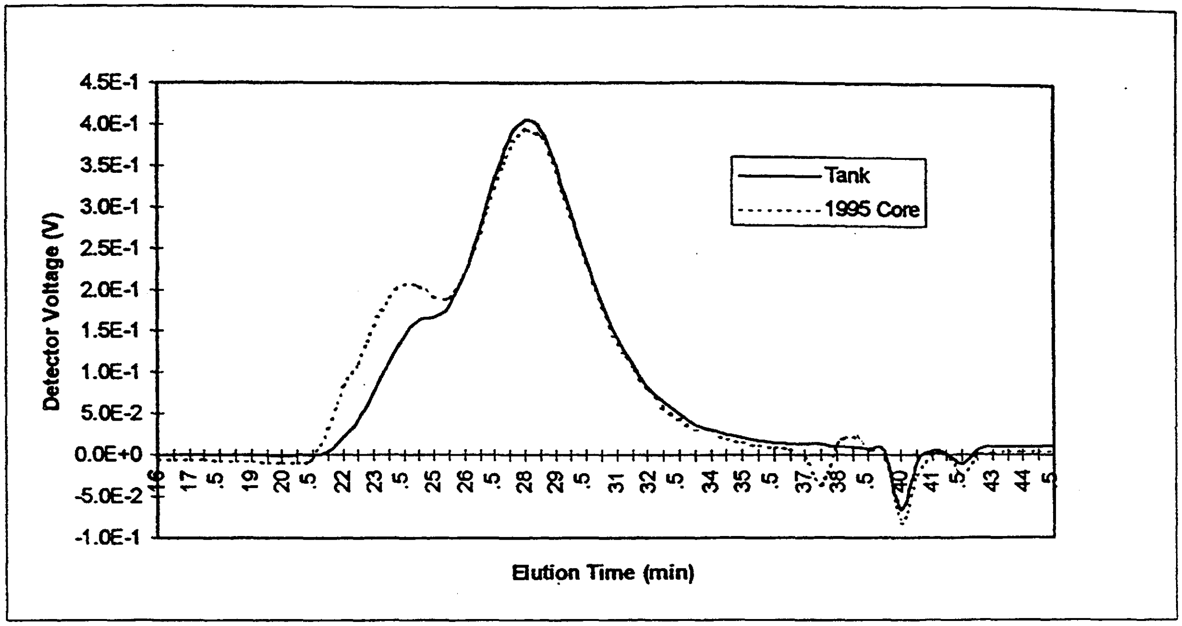

Figure B1. Sherman GPC Results for Novophalt Illustrating Increase in Molecular Size.

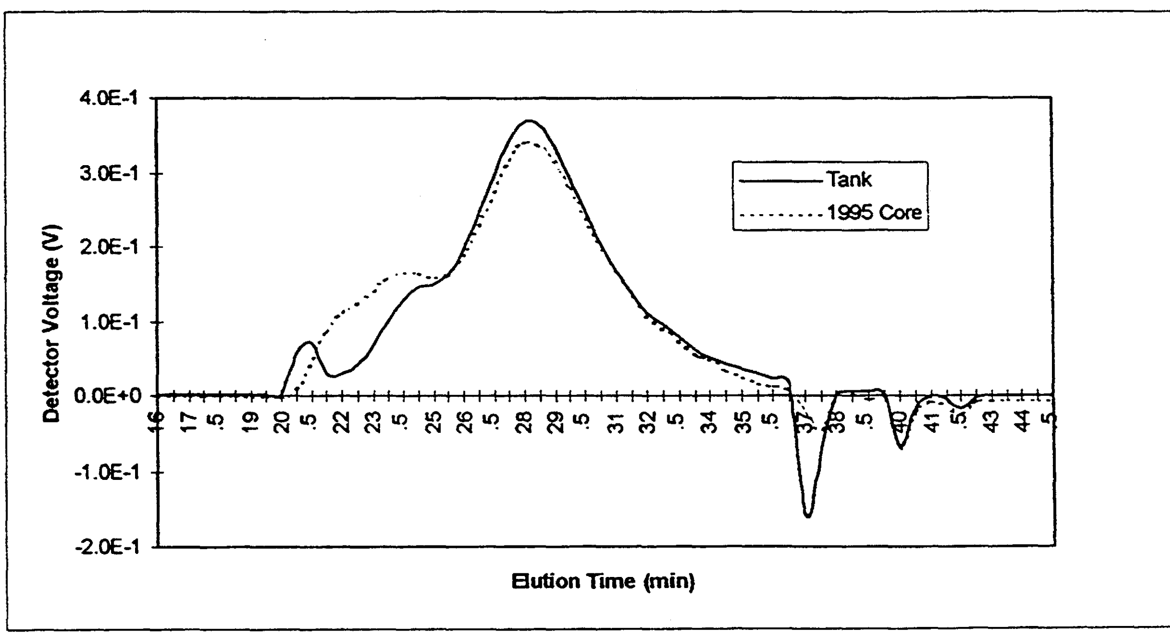

Figure B2. Sherman GPC Results for SBS Illustrating Increase in Molecular Size.

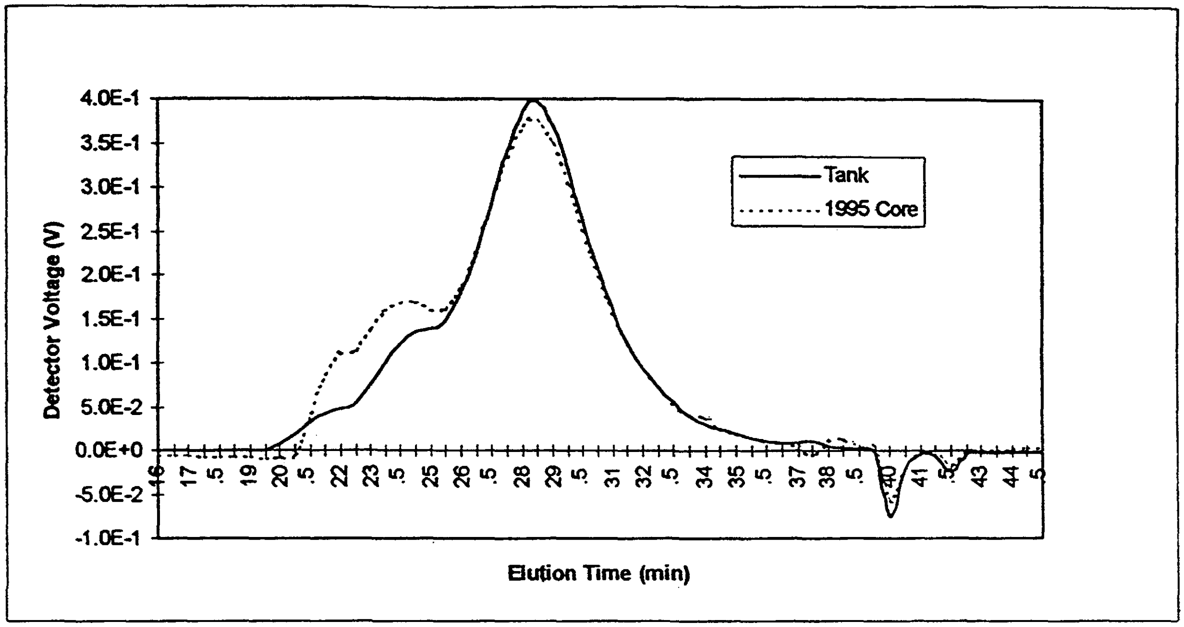

Figure B3. Sherman GPC Results for Latex Illustrating Increase in Molecular Size.

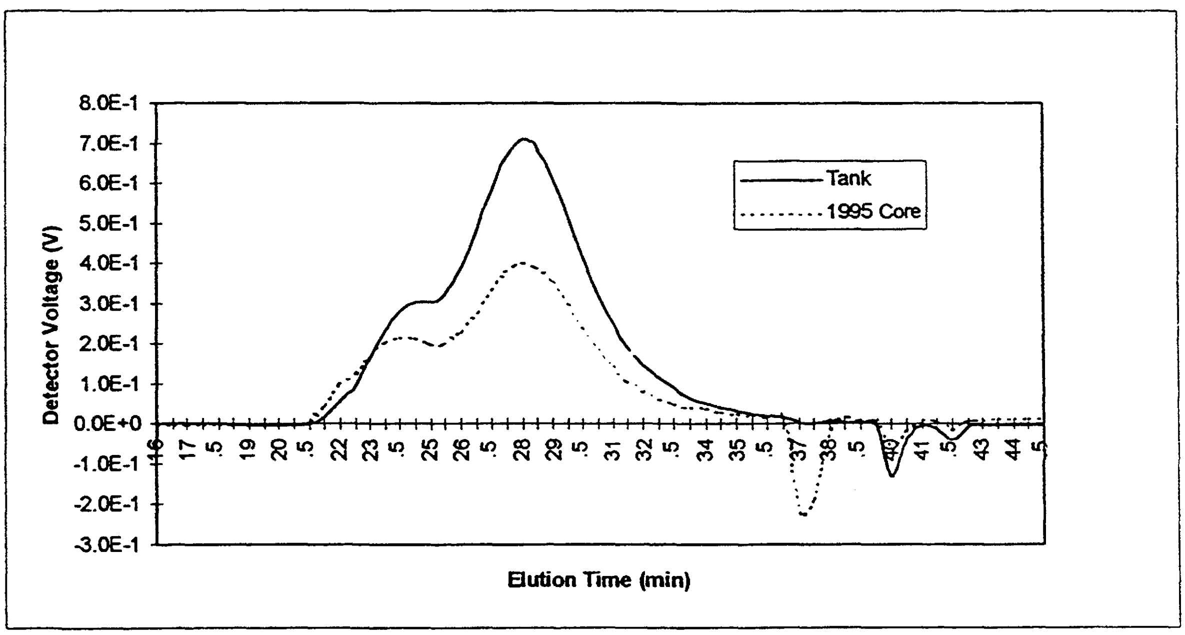

Figure B4. Sherman GPC Results for Carbon Black Illustrating Increase in Molecular Size.

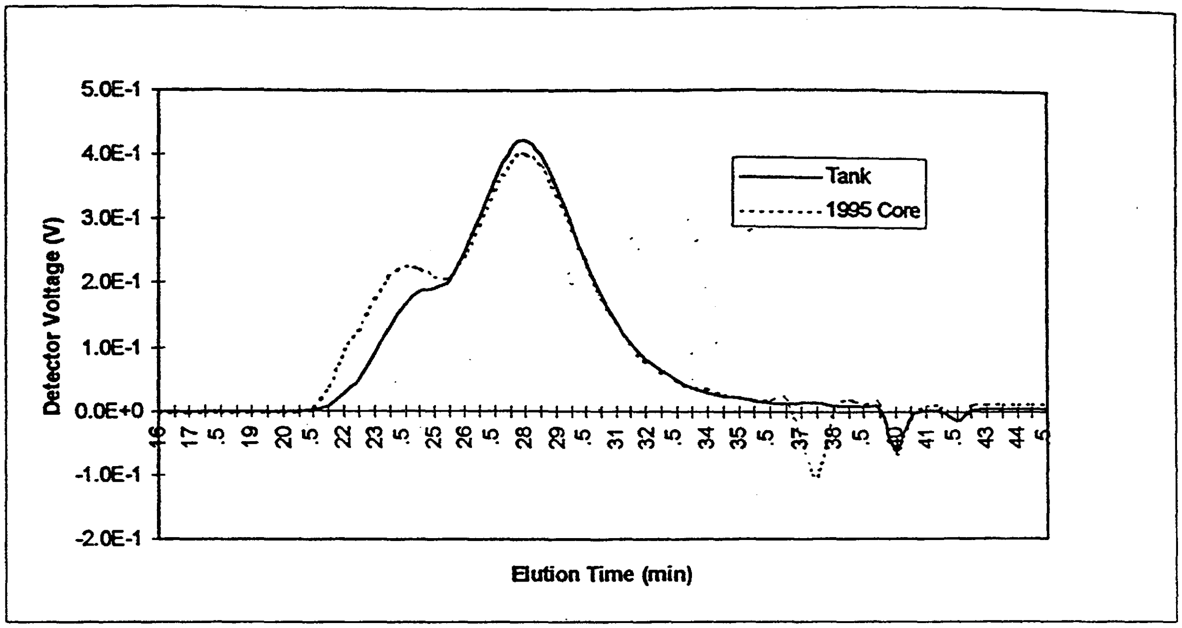

Figure B5. Sherman GPC Results for Control Illustrating Increase in Molecular Size.

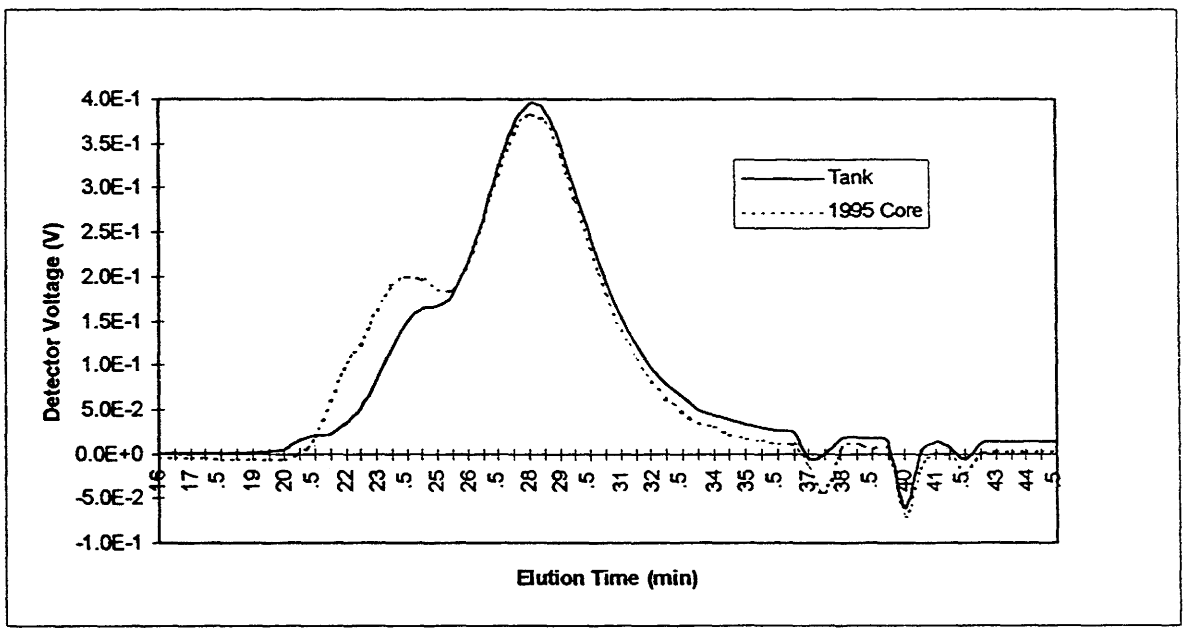

Figure B6. Sherman GPC Results for EVA Illustrating Increase in Molecular Size.

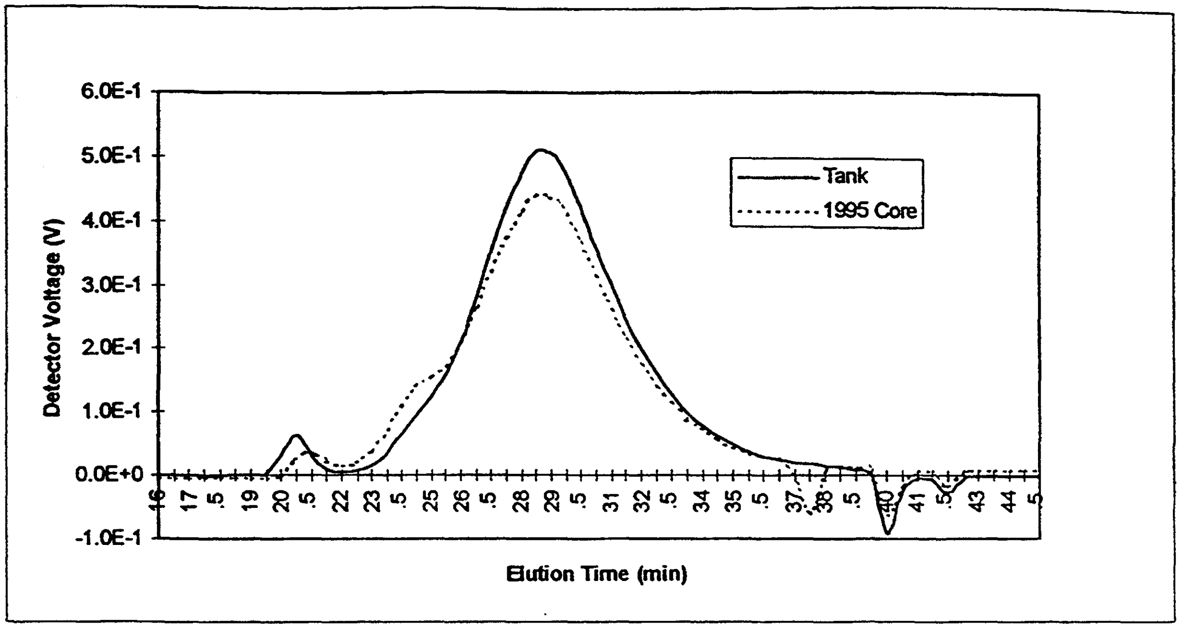

Figure B7. Texarkana GPC Results for Latex Base Illustrating Increase in Molecular Size.

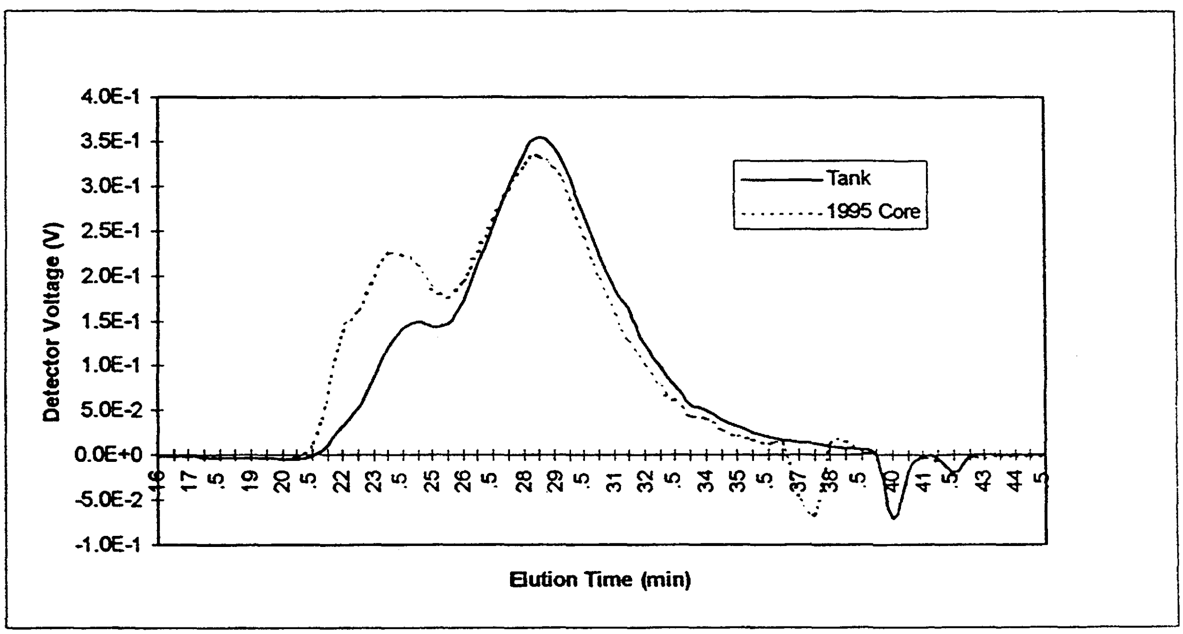

Figure B8. Texarkana GPC Results for Chemkrete Base Illustrating Increase in Molecular Size.

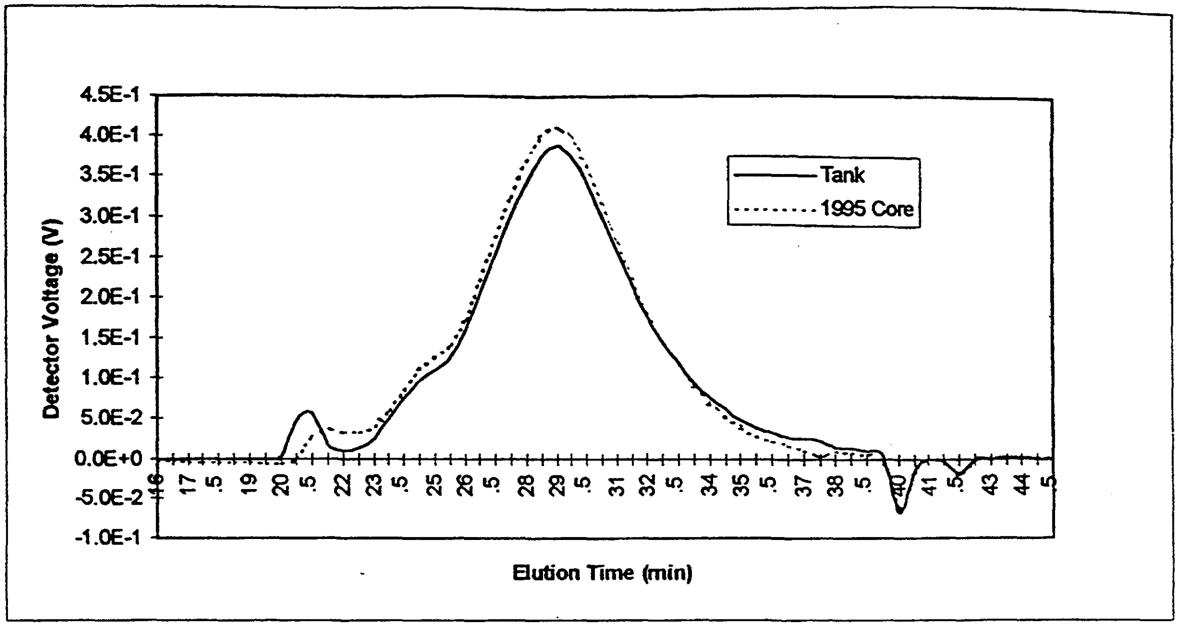

Figure B9. Texarkana GPC Results for Styrelf Base Illustrating Increase in Molecular Size.

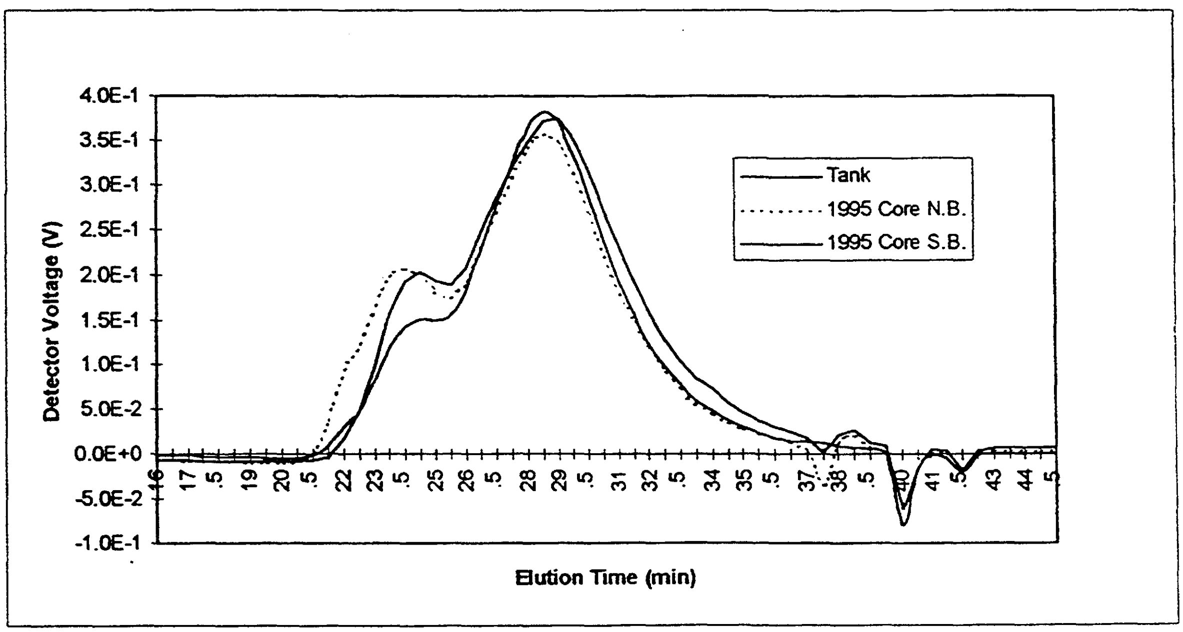

Figure B10. Texarkana GPC Results for Control Base Illustrating Increase in Molecular Size.

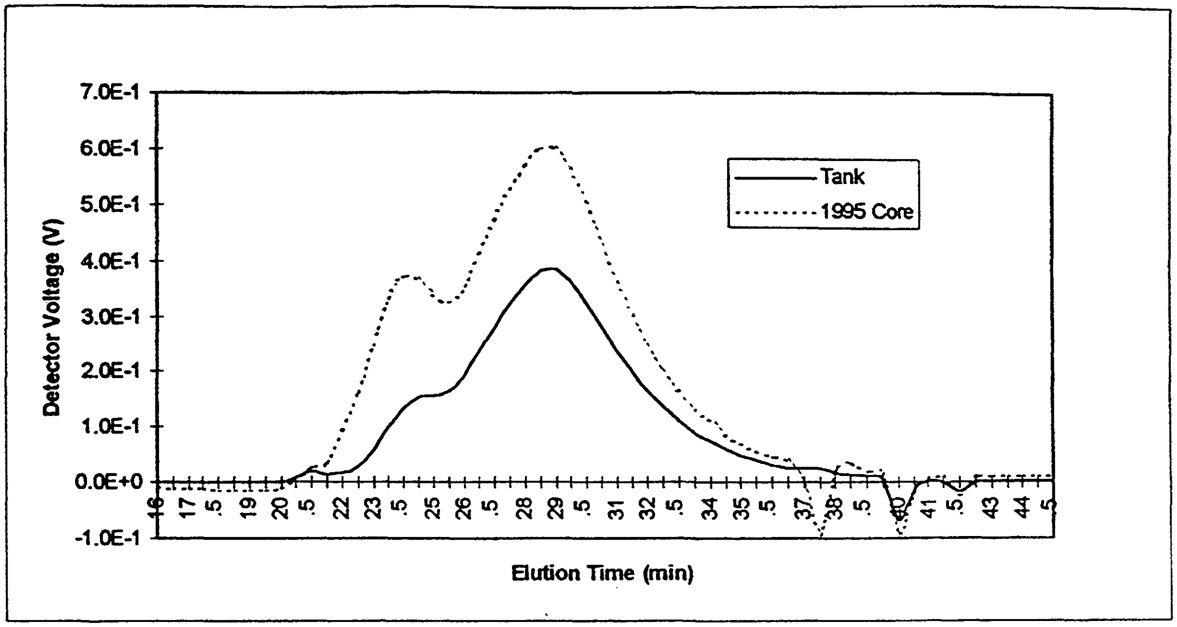

Figure B11. Texarkana GPC Results for EVA Base Illustrating Increase in Molecular Size.

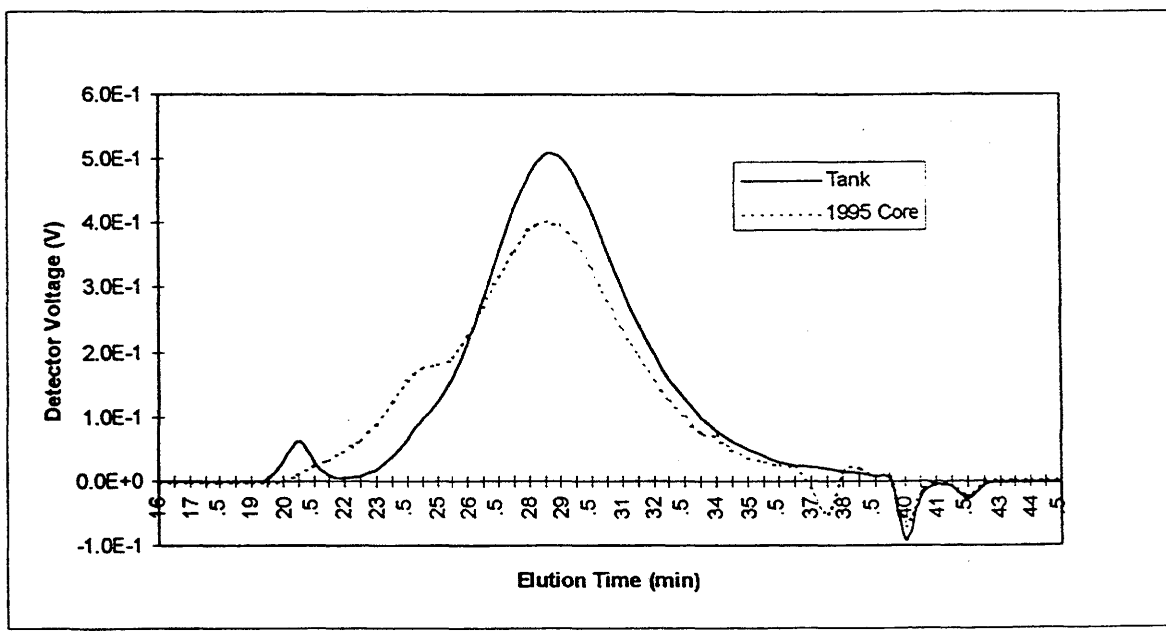

Figure B12. Texarkana GPC Results for Latex Surface Illustrating Increase in Molecular Size.

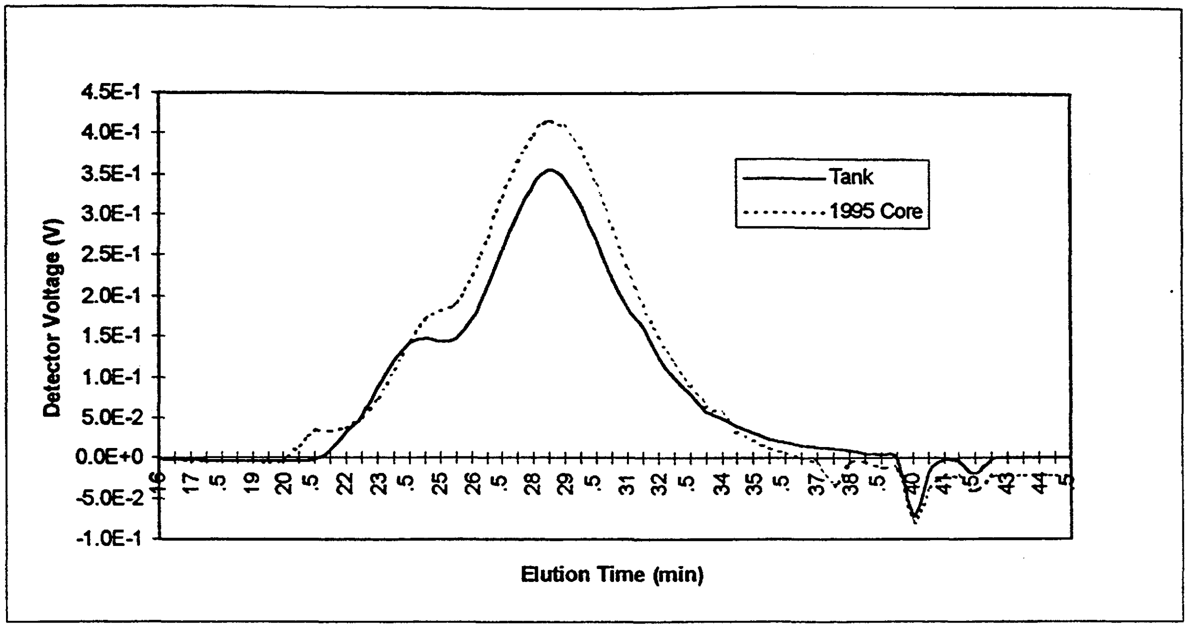

Figure B13. Texarkana GPC Results for Latex* Surface Illustrating Increase in Molecular Size (*Latex over Chemkrete).

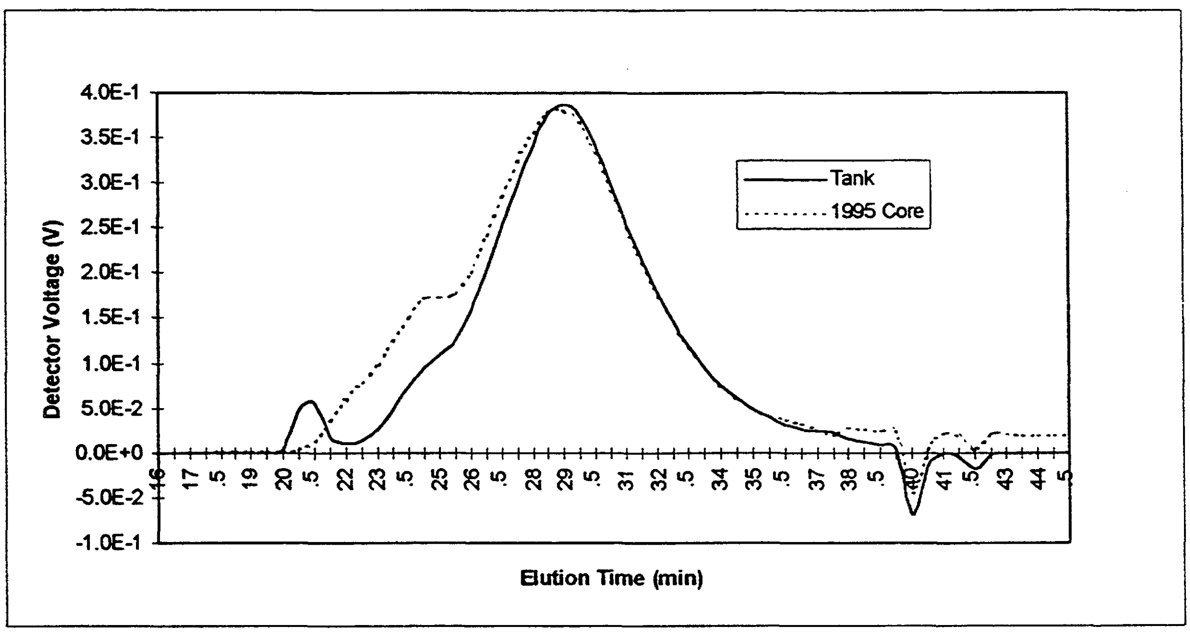

Figure B13. Texarkana GPC Results for Latex* Surface Illustrating Increase in Molecular Size (*Latex over Chemkrete).

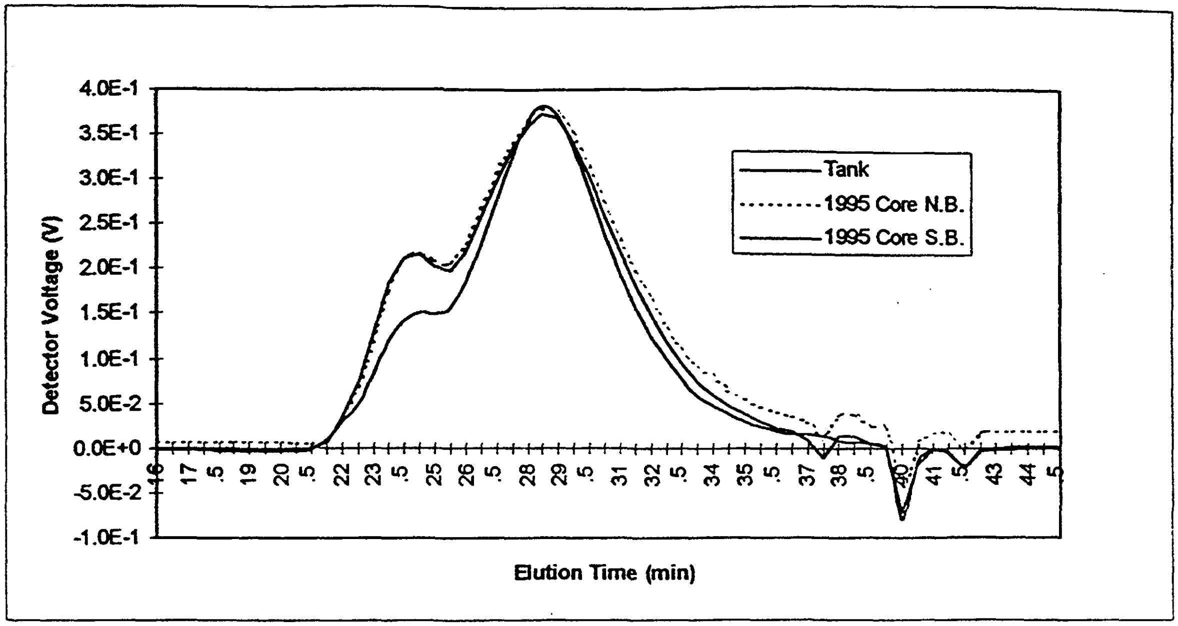

Figure B15. Texarkana GPC Results for Control Surface Illustrating Increase in Molecular Size.

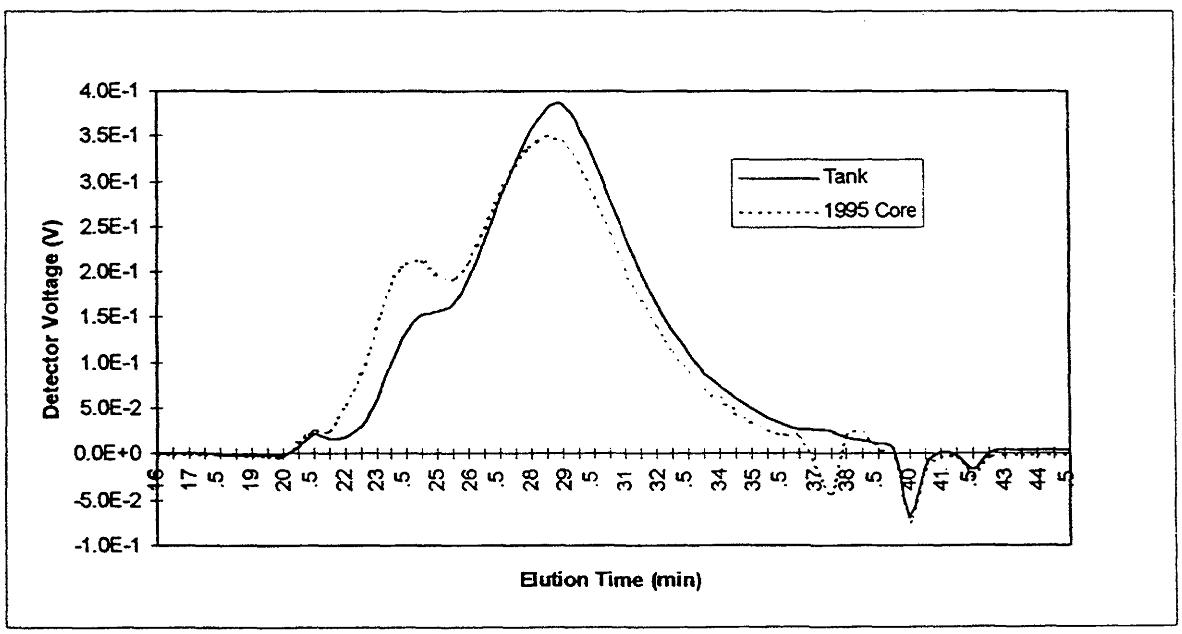

Figure B16. Texarkana GPC Results for EVA Surface Illustrating Increase in Molecular Size.

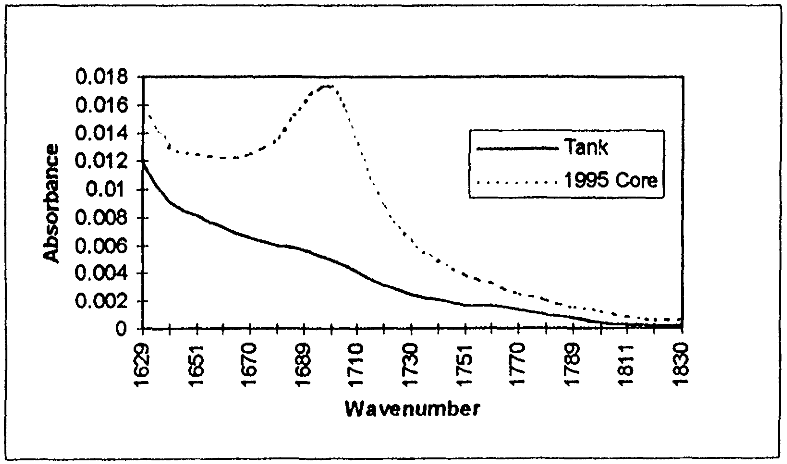

Figure B17. Sherman FTIR Spectra for Novophalt Illustrating Oxidative Aging.

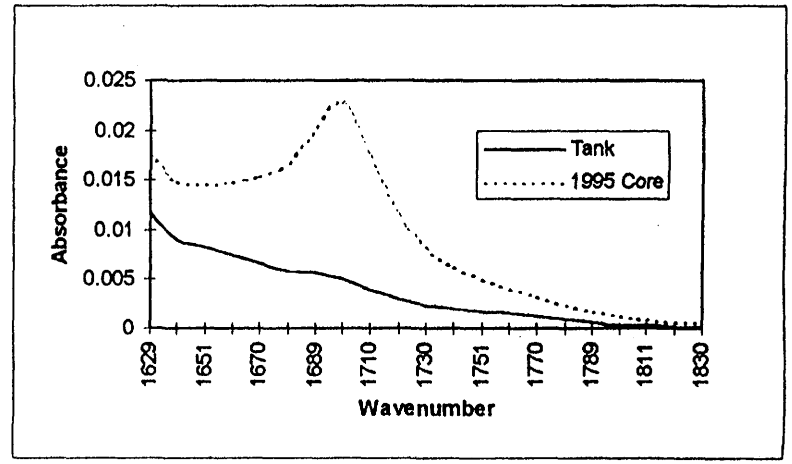

Figure B18. Sherman FTIR Spectra for SBS Illustrating Oxidative Aging.

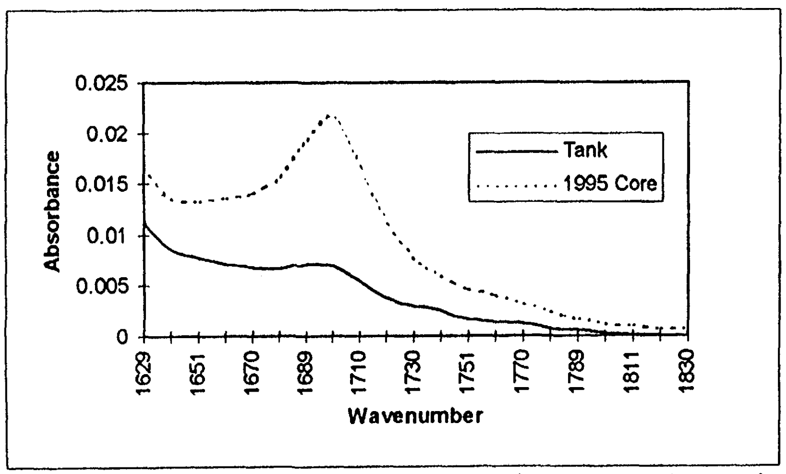

Figure B19. Sherman FTIR Spectra for Latex Illustrating Oxidative Aging.

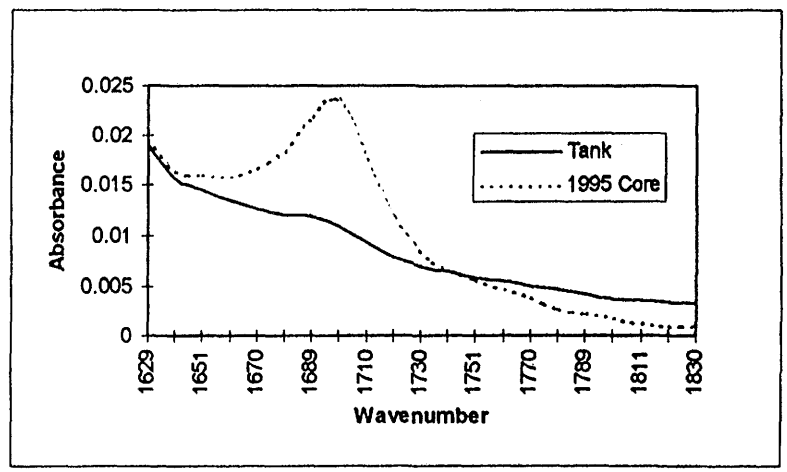

Figure B20. Sherman FTIR Spectra for Carbon Black Illustrating Oxidative Aging.

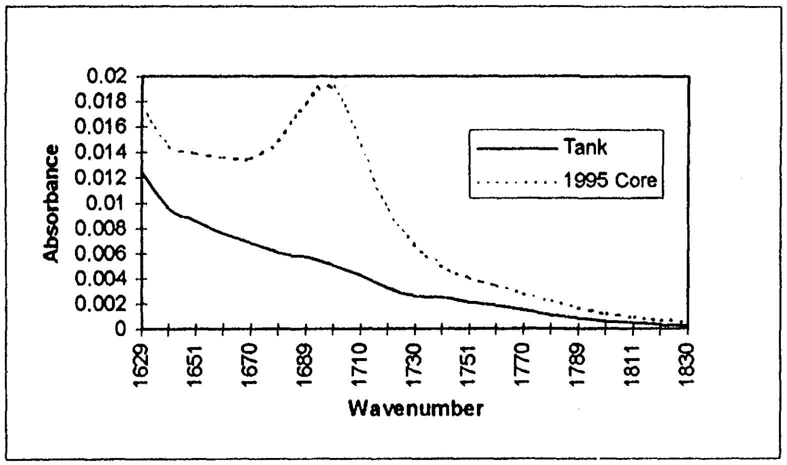

Figure B21. Sherman FTIR Spectra for Control Illustrating Oxidative Aging.

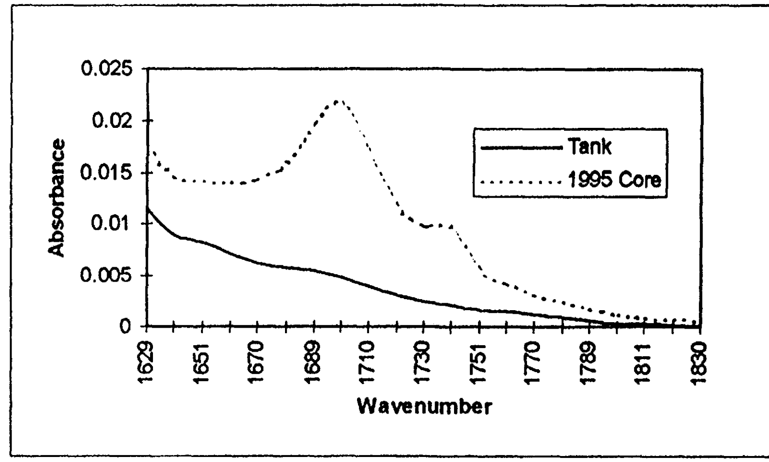

Figure B22. Sherman FTIR Spectra for EVA Illustrating Oxidative Aging.

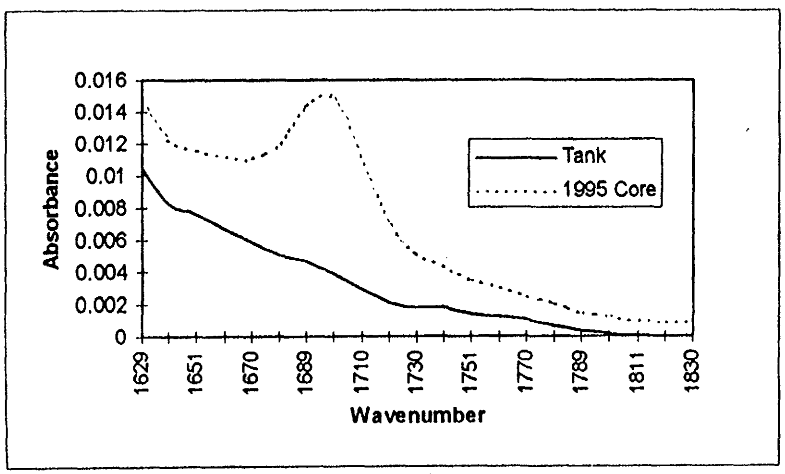

Figure B23. Texarkana FTIR Spectra for Latex Base Illustrating Oxidative Aging.

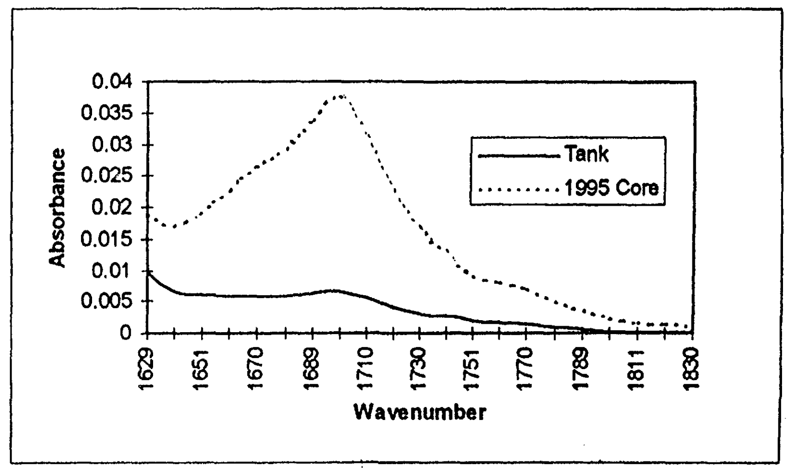

Figure B24. Texarkana FTIR Spectra for Chemkrete Base Illustrating Oxidative Aging.

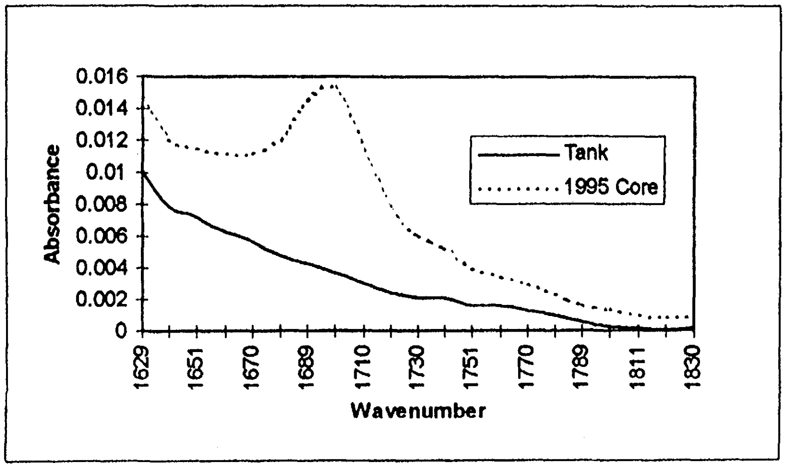

Figure B25. Texarkana FTIR Spectra for Styrelf Base Illustrating Oxidative Aging.

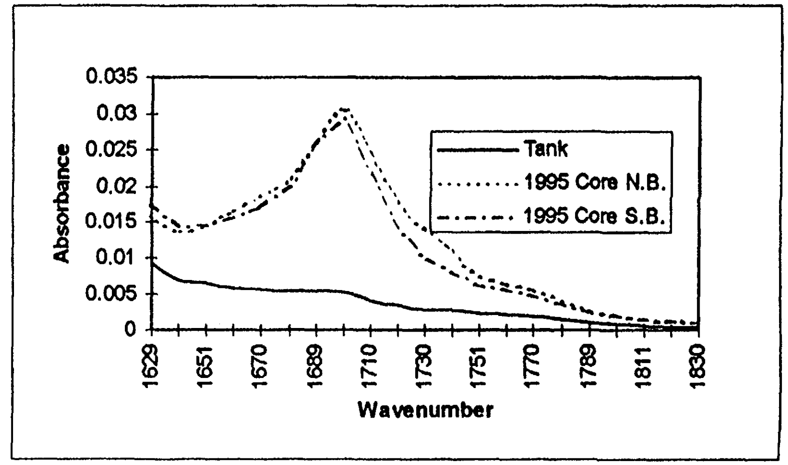

Figure B26. Texarkana FTIR Spectra for Control Base Illustrating Oxidative Aging.

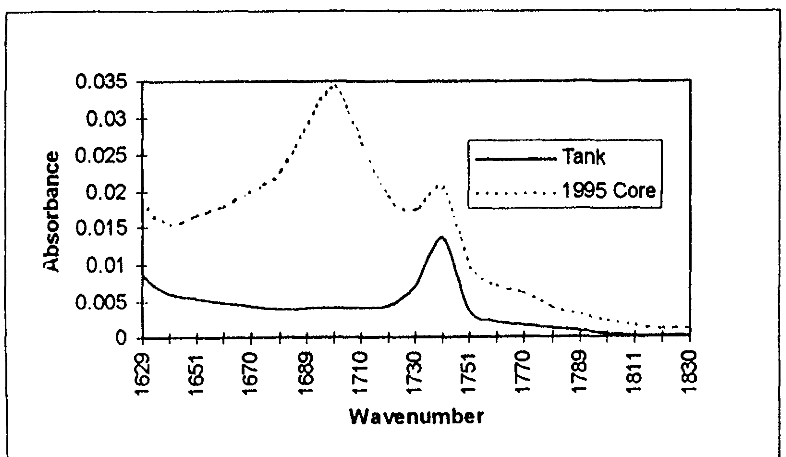

Figure B27. Texarkana FTIR Spectra for EVA Base Illustrating Oxidative Aging.

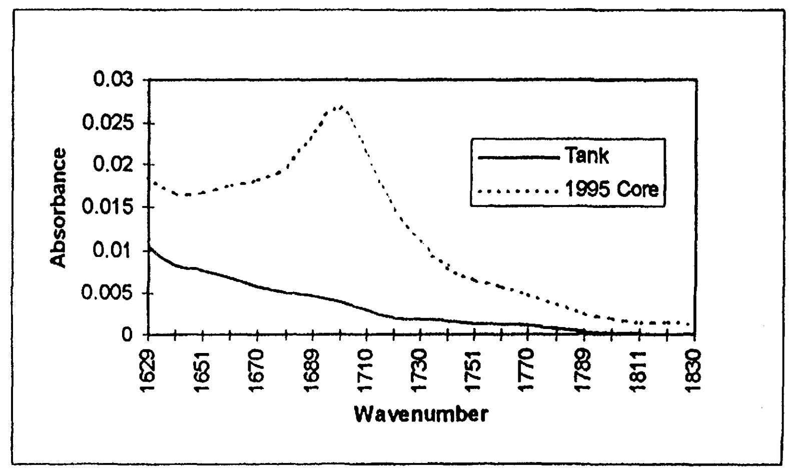

Figure B28. Texarkana FTIR Spectra for Latex Surface Illustrating Oxidative Aging.

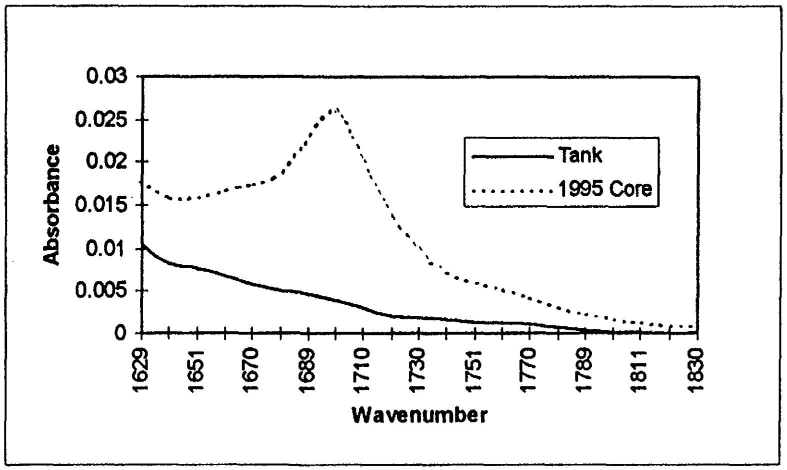

Figure B29. Texarkana FTIR Spectra for Latex* Surface Illustrating Oxidative Aging (*Latex over Chemkrete).

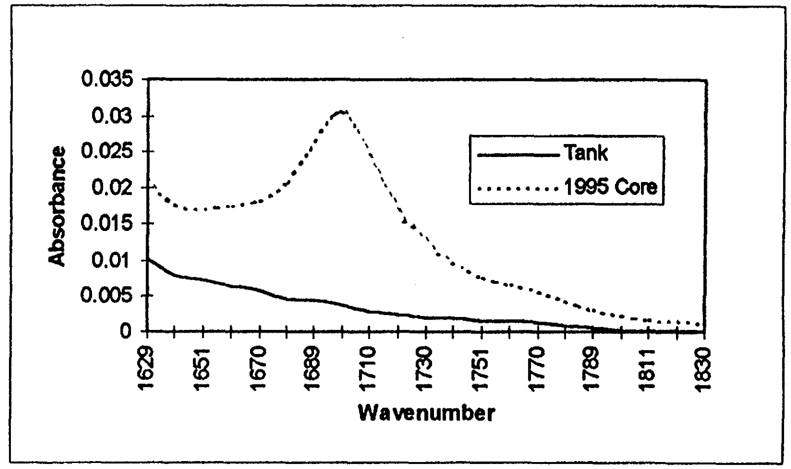

Figure B30. Texarkana FTIR Spectra for Styrelf Surface Illustrating Oxidative Aging.

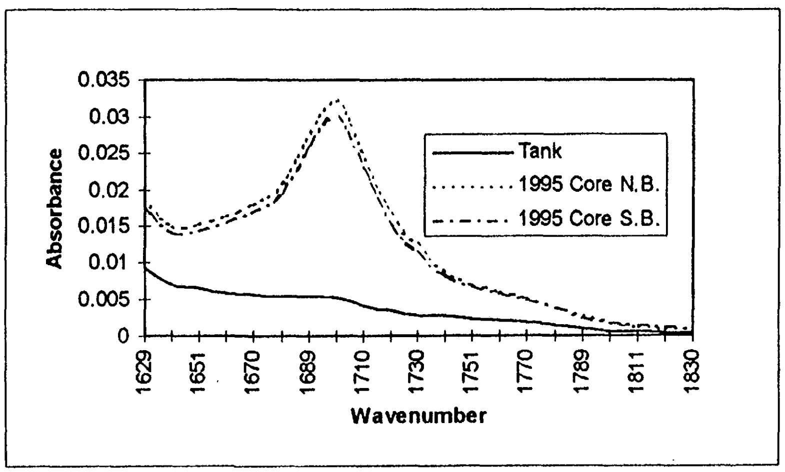

Figure B31. Texarkana FTIR Spectra for Control Surface Illustrating Oxidative Aging.

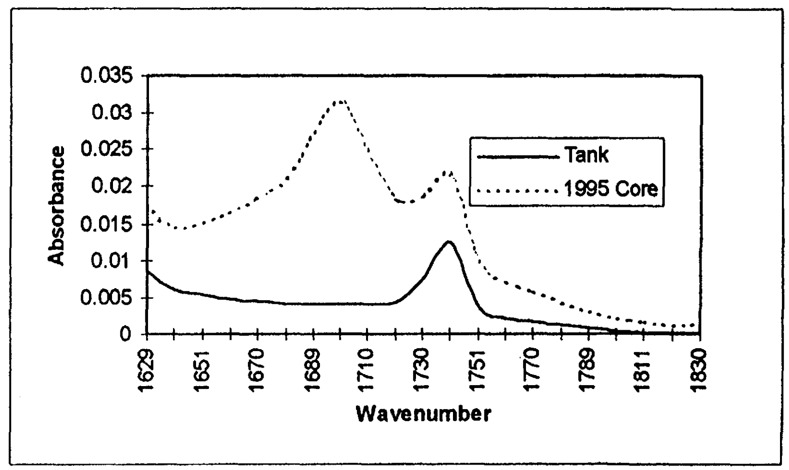

Figure B32. Texarkana FTIR Spectra for EVA Surface Illustrating Oxidative Aging.

APPENDIX C

AIR VOIDS, RESILIENT MODULUS AND TENSILE PROPERTIES OF TEST MIXTURES FOR TEXARKANA AND SHERMAN

Table C1. Properties of Field Mixed-Laboratory Compacted Texarkana Base Course Specimens.

Type Mixture	Air Void Content, percent	Resilient Modulus, kPa			Tensile Properties[1]	
		0°C	25°C	40°C	Tensile Strength, kPa	Tensile Strain, mm/mm
Goodyear/ Fina	4.3	1760	300	38	1379	.0039
Chemkrete/ MacMillan	4.5	1310	260	52	1310	.0023
Styrelf/ Exxon	3.9	1760	290	48	1586	.0045
Control/ NBL	4.4	1200	220	42	966	.0023
Polybilt/ Lyon	4.7	1580	420	56	1517	.0027

[1]Indirect tension tests were performed at 25°C and 50.8 mm/min.

Table C2. Properties of Field Mixed-Laboratory Compacted Sherman Specimens.

Type Mixture	Air Void Content, percent	Resilient Modulus, kPa			Tensile Properties[1]	
		0°C	25°C	40°C	Tensile Strength, kPa	Tensile Strain, mm/mm
Novophalt	3.8	1300	380	110	966	.0047
SBS	2.3	1200	190	59	759	.0100
SBR	3.2	1200	190	58	676	.0082
Control	3.6	1400	370	100	1034	.0061
Carbon Black	5.1	1300	340	89	966	.0064
EVA	2.3	1500	310	87	828	.0054

[1]Indirect tension tests were performed at 25°C and 50.8 mm/min.

Table C3. Properties of Field Cores from Texarkana Base Course.

					Tensile Properties[1]	
	Air Void Content,	Resilient Modulus, kPa			Tensile Strength,	Tensile Strain,
Type Mixture	percent	0°C	25°C	40°C	kPa	mm/mm
Goodyear/	4.9	1821	556	39	1041	0.0096
Fina	4.9	2295	561	42	1048	0.0135
	5.9	1734	523	43	1145	0.0092
Mean	5.2	1950	547	42	1078	0.0108
Chemkrete/	4.7	1511	930	181	524	0.0044
MacMillan	4.9	1469	786	188	524	0.0034
	5.6	993	847	195	490	0.0037
Mean	5.1	1324	854	188	513	0.0038
Styrelf/	4.9	1742	418	28		-
Exxon	3.6	1863	362	30	800	0.0075
	4.0	1798	596	45	869	0.0068
Mean	4.2	1801	459	35	834	0.0071
Control/	5.6	2094	1071	116	800	0.0049
SBL	4.9	1880	977	96	1317	0.0034
	5.4	1881	937	98	1366	0.0033
Mean	5.3	1952	995	104	1161	0.0038
Control/	8.0	1032	373	61	324	0.0011
NBL	8.6	1157	366	56	448	0.0085
	4.8	2537	997	195	476	0.0054
Mean	7.1	1575	579	104	416	0.0050
Polybilt/	10.5	1223	575	88	669	0.0057
Lyon	9.7	1501	671	87	497	0.0015
	10.9	1392	642	98	517	0.0098
Mean	10.4	1372	629	91	561	0.0057

[1]Indirect tension tests were performed at 25°C and 50.8 mm/min.

Table C4. Properties of Field Cores from Texarkana Surface Course.

Type Mixture	Air Void Content, percent	Resilient Modulus, kPa 0°C	Resilient Modulus, kPa 25°C	Resilient Modulus, kPa 40°C	Tensile Properties[1] Tensile Strength, kPa	Tensile Properties[1] Tensile Strain, mm/mm
Goodyear/	7.8	1429	594	99	1379	0.0050
Fina	7.5	1431	546	97	1448	0.0029
	9.6	1364	635	92	1303	0.0038
Mean	8.3	1408	592	96	1377	0.0039
Chemkrete/	6.4	2134	960	110	2000	0.0068
MacMillan	6.5	1742	851	100	1800	0.0065
	6.7	1712	943	77	1876	0.0059
Mean	6.5	1863	918	96	1892	0.0064
Styrelf/	8.4	1498	808	96	1600	0.0028
Exxon	7.7	1467	793	97	1028	0.0028
	7.9	1720	845	102	1607	0.0039
Mean	8.0	1562	815	98	1411	0.0032
Control/	6.5	1926	884	99	1738	0.0051
SBL	6.1	1827	873	75	1738	0.0050
	5.8	1932	826	74	2021	0.0046
Mean	6.1	1895	861	83	1832	0.0049
Control/	6.5	2133	960	110	1972	0.0040
NBL	6.4	1742	851	100	1828	0.0044
	7.2	1711	942	77	1897	0.0041
Mean	6.7	1862	918	96	1899	0.0042
Polybilt/	7.5	1469	589	162	1807	0.0022
Lyon	7.5	1452	545	156	1352	0.0021
	7.1	1381	641	155	1324	0.0049
Mean	7.4	1434	592	158	1494	0.0031

[1]Indirect tension tests were performed at 25°C and 50.8 mm/min.

Table C5. Properties of Field Cores from Sherman.

Type Mixture	Air Void Content, percent	Resilient Modulus, kPa 0°C	25°C	40°C	Tensile Properties[1] Tensile Strength, kPa	Tensile Strain, mm/mm
Novophalt	4.8	1373	724	203	1503	0.0055
	3.4	1576	817	160	1483	0.0045
	3.5	1498	934	160	1690	0.0048
Mean	3.9	1482	825	174	1559	0.0049
SBS	9.7	877	372	86	1021	0.0049
	7.4	1044	481	155	1090	0.0037
	8.7	1098	443	133	986	0.0041
Mean	8.6	1006	432	125	1032	0.0042
SBR	6.2	940	337	95	979	0.0042
	5.1	1203	526	149	710	0.0048
	6.5	1175	496	132	1034	0.0041
Mean	5.9	1106	453	125	908	0.0044
Carbon Black	6.6	1355	574	135	1021	0.0034
	6.3	1171	591	131	1283	0.0039
	3.9	1174	510	132	1310	0.0048
Mean	5.6	1233	558	132	1200	0.004
Control	5.5	1340	805	205	-	-
	4.5	1687	862	191	1779	0.0037
	5.1	1271	726	212	1697	0.0037
Mean	5.0	1433	798	203	1738	0.0037
EVA	7.8	989	370	94	848	0.0067
	7.9	1176	475	116	655	0.0097
	6.6	988	478	110	1152	0.0051
Mean	7.4	1051	441	107	885	0.0071

[1]Indirect tension tests were performed at 25°C and 50.8 mm/min.

APPENDIX D

ADDITIONAL AUSTRALIAN FREQUENCY SWEEP DATA FOR TEXARKANA AND SHERMAN

Table D1. Australian Frequency Sweep Data for Laboratory Compacted Texarkana Specimens.

	Latex		Chemkrete	Styrelf		Control		EVA	
Sigma x	35.44	45.89	92.26	49.48	43.73	39.45	34.59	60.48	41.40
Sigma y	29.74	41.68	90.75	44.32	37.69	33.15	27.91	56.02	35.90
Sigma z	3.71	3.36	1.74	3.44	3.56	4.02	4.26	3.17	3.75
Mean principal stress, psi	22.96	30.31	61.58	32.41	28.33	25.54	22.25	39.89	27.02
Octahedral shear stress, psi	13.81	19.13	42.32	20.60	17.68	15.43	13.01	26.03	16.61
Asphalt concrete modulus, psi	535115	524280	511518	560690	592151	493499	455130	503595	471928
Asphalt content by weight percent	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3
Air voids content, percent	4.8	4.3	2.1	5.2	4.6	2.8	1.4	4.3	3.8
Poisson's ratio @ 12°C, 10Hz	0.119	0.130	0.135	0.110	0.326	0.386	0.128	0.118	0.193
Average daily traffic (truck/day)	1650	1650	1650	1650	1650	1650	1650	1650	1650
Height of A/C surface (in)	2	2	2	2	2	2	2	2	2
Height of Base (in)	8	8	8	8	8	8	8	8	8
D1 (shear)	5.93E-06	1.91E-05	7.33E-06	7.03E-06	8.66E-07	3.28E-06	3.18E-06	5.21E-06	7.68E-07
m (shear)	0.135	0.037	0.161	0.132	0.253	0.255	0.031	0.203	0.219
Tensile strength at 25°C (psi)	200	200	190	230	230	140	140	220	220
D* @12°C, 10Hz	0.271	0.277	0.283	0.259	0.245	0.294	0.319	0.288	0.307
Ni (lab)	20593	11225	4377	7003	11795	31449	82292	5047	18436
Ni (field)	76781	41850	16320	26112	43978	117256	306820	18819	68739
Np (field)	9.569E-06	5.26E-06	1.03E-05	9.47E-06	1.1E-05	1.17E-05	4.2E-06	1.08E-05	1.06E-05
Nf (field)	76781	41850	16320	26112	43978	117256	306820	18819	68739
Nf Mean (number loads to failure)	59315		16320	35045		212038		43779	

Table D2. Australian Frequency Sweep Data for Laboratory Compacted Sherman Specimens.

	Novophalt		SBS		SBR		C.B.		Control		EVA	
Sigma x	1.70	92.04	1.90	18.64	3.41	34.13	1.93	1.12	1.06	1.64	17.30	3.38
Sigma y	8.18	79.01	8.87	21.17	11.46	7.15	9.90	7.23	6.21	9.28	22.06	13.61
Sigma z	25.43	7.24	26.27	49.04	29.98	43.72	27.26	23.94	22.91	26.39	49.21	32.07
Mean principal stress, psi	11.77	59.43	12.35	29.62	14.95	28.33	13.03	10.76	10.06	12.44	29.52	16.35
Octahedral shear stress, psi	10.02	37.29	10.25	13.77	11.12	15.48	10.57	9.65	9.33	10.35	14.06	11.87
Asphalt concrete modulus, psi	285461	318262	295310	313168	241755	260665	366078	362672	314247	385044	342749	391564
Asphalt content by weight percent	5.3	5.3	5.3	5.3	5.3	5.3	5.3	5.3	5.3	5.3	5.3	5.3
Air voids content, percent	6.4	4.7	4.7	4.0	5.8	5.7	4.7	4.0	5.5	3.8	3.4	2.0
Poisson's ratio @ 12°C, 10Hz	0.229	0.791	0.251	0.672	0.355	0.723	0.262	0.16	0.136	0.233	0.665	0.371
Average daily traffic (truck/day)	3950	3950	3950	3950	3950	3950	3950	3950	3950	3950	3950	3950
Height of A/C surface (in)	3	3	3	3	3	3	3	3	3	3	3	3
Height of Base (in)	8	8	8	8	8	8	8	8	8	8	8	8
D1 (shear)	5.42E-06	5.48E-06	5.56E-06	7.58E-06	6.49E-06	9.13E-06	6.49E-07	4.42E-06	5.09E-07	1.77E-05	5.95E-06	3.03E-06
m (shear)	0.274	0.221	0.200	0.199	0.350	0.183	0.287	0.284	0.288	0.140	0.272	0.362
Tensile strength at 25°C (psi)	226	226	150	150	132	132	174	174	252	252	128	128
D* @12°C, 10Hz	0.508	0.456	0.491	0.463	0.600	0.556	0.396	0.400	0.461	0.377	0.423	0.370
Ni (lab)	2182286	323	2648864	22320	587098	10754	1787370	5731651	6304104	2768654	34354	1249803
Ni (field)	5034836	746	6111296	51495	1354516	24810	4123708	13223709	14544438	6387668	79259	2883469
Np (field)	9.23E-06	8.78E-06	8.76E-06	8.82E-06	1.01E-05	8.71E-06	9.14E-06	9.43E-06	8.90E-06	7.86E-06	9.59E-06	1.01E-05
Nf (field)	5034836	746	6111296	51495	1354516	24810	4123708	13223709	14544438	6387668	79259	2883469
Nf Mean (number loads to failure)	2517791		3081396		689663		8673709		10466053		1481364	

Table D3. Australian Frequency Sweep Data for Texarkana Field Cores.

	Latex			Chemkrete			Styrelf			Control S.B.		
Sigma x	29.24	28.34	63.30	18.92	23.74	25.49	29.51	31.72	24.00	30.51	39.45	38.15
Sigma y	22.87	21.33	60.61	13.00	16.88	19.75	23.98	26.01	17.72	22.71	33.66	31.37
Sigma z	4.04	3.93	2.70	6.09	5.43	5.71	4.84	4.60	5.00	4.71	4.75	4.43
Mean principal stress, psi	18.72	17.87	42.20	12.67	15.35	16.98	19.44	20.78	15.57	19.31	25.95	24.65
Octahedral shear stress, psi	10.70	10.26	27.96	5.24	7.55	8.31	10.57	11.68	7.91	10.80	15.18	14.56
Asphalt concrete modulus, psi	469377	506515	509093	208864	287630	250255	319566	358804	307093	413023	357663	440944
Asphalt content by weight percent	5.3	5.3	5.3	5.3	5.3	5.3	5.3	5.3	5.3	5.3	5.3	5.3
Air voids content, percent	6.4	4.7	4.7	4.0	5.8	5.7	4.7	4.0	5.5	3.8	3.8	3.8
Poisson's ratio @ 12°C, 10Hz	0.217	0.130	0.788	0.140	0.172	0.351	0.394	0.401	0.197	0.221	0.535	0.419
Average daily traffic (truck/day)	1650	1650	1650	1650	1650	1650	1650	1650	1650	1650	1650	1650
Height of A/C surface (in)	3	3	3	3	3	3	3	3	3	3	3	3
Height of Base (in)	8	8	8	8	8	8	8	8	8	8	8	8
D1 (shear)	9.15E-06	4.16E-06	3.27E-05	1.78E-05	1.90E-05	2.59E-05	2.32E-06	4.79E-06	3.54E-06	5.68E-06	1.41E-05	2.02E-06
m (shear)	0.197	0.040	0.076	0.187	0.022	0.186	0.343	0.204	0.230	0.101	0.199	0.314
Tensile strength at 25°C (psi)	156	156	156	74	74	74	121	121	121	168	168	168
D* @12°C, 10Hz	0.309	0.286	0.285	0.694	0.504	0.579	0.454	0.404	0.472	0.351	0.405	0.329
Ni (lab)	144721	282311	10030	2359620	547954	274816	211403	199746	565786	315276	65233	101930
Ni (field)	539583	1052579	37395	8797693	2043011	1024634	788203	744739	2109497	1175486	243217	380040
Np (field)	8.809E-06	4.079E-06	6.467E-06	9.243E-06	2.925E-06	9.324E-06	9.946E-06	8.907-06	9.142E-06	6.837E-06	8.878E-06	9.527E-06
Nf (field)	539583	1052579	37395	8797693	2043011	1024634	788203	744739	2109497	1175486	243217	380040
Nf Mean (number loads to failure)	543186			3955113			1214146			599581		

Table D3. (Continued)

	Control N.B.	EVA				
Sigma x	20.80	16.10	28.63	17.91	569.30	26.60
Sigma y	14.88	12.15	24.11	13.31	451.90	19.99
Sigma z	5.99	7.48	6.58	5.64	3.63	4.24
Mean principal stress, psi	13.89	11.91	19.77	12.29	341.61	16.94
Octahedral shear stress, psi	6.09	3.52	9.51	5.06	243.75	9.38
Asphalt concrete modulus, psi	222659	128001	173232	191121	389471	394118
Asphalt content by weight percent	5.3	5.3	5.3	5.3	5.3	5.3
Air voids content, percent	3.8	3.8	3.8	3.8	3.8	3.8
Poisson's ratio @ 12°C, 10Hz	0.208	0.386	0.611	0.231	0.967	0.192
Average daily traffic (truck/day)	1650	1650	1650	1650	1650	1650
Height of A/C surface (in)	3	3	3	3	3	3
Height of Base (in)	8	8	8	8	8	8
D1 (shear)	7.29E-06	7.65E-06	1.38E-05	8.92E-06	2.72E-06	4.86E-06
m (shear)	0.315	0.356	0.208	0.242	0.316	0.213
Tensile strength at 25°C (psi)	60	60	60	81	81	81
D* @12°C, 10Hz	0.651	1.133	0.837	0.759	0.372	0.368
Ni (lab)	1509890	1854246	94586	2756499	3.27E+13	601421
Ni (field)	5629528	6913439	352658	10277432	1.22E+14	2242359
Np (field)	1.047E-05	1.073E-05	9.600E-06	9.685E-06	1.008E-05	9.242E-06
Nf (field)	5629528	6913439	352658	10277432	1.22E+14	2242359
Nf Mean (number loads to failure)	4298542			4.06E+13		

Table D4. Australian Frequency Sweep Data for Sherman Field Cores.

	Novophalt			SBS			SBR		
Sigma x	61.79	1.19	1.00	1.68	3.93	7.16	4.58	1.07	23.60
Sigma y	402.00	8.78	7.59	8.33	13.22	17.16	14.25	6.13	18.33
Sigma z	402.00	25.41	24.00	25.60	32.07	38.12	33.54	22.85	49.12
Mean principal stress, psi	288.60	11.79	10.86	11.87	16.41	20.81	17.46	10.02	30.35
Octahedral shear stress, psi	160.38	10.11	9.67	10.08	11.71	12.90	12.04	9.31	13.45
Asphalt concrete modulus, psi	519546	483378	463007	300263	293540	246519	287368	306017	340408
Asphalt content by weight percent	5.3	5.3	5.3	5.3	5.3	5.3	5.3	5.3	5.3
Air voids content, percent	6.4	4.7	4.7	4.0	5.8	5.7	4.7	4.0	5.5
Poisson's ratio @ 12°C, 10Hz	0.872	0.179	0.139	0.229	0.389	0.505	0.419	0.136	0.699
Average daily traffic (truck/day)	3950	3950	3950	3950	3950	3950	3950	3950	3950
Height of A/C surface (in)	3	3	3	3	3	3	3	3	3
Height of Base (in)	8	8	8	8	8	8	8	8	8
D1 (shear)	1.62E-06	1.51E-05	2.91E-08	5.53E-06	3.23E-06	1.74E-06	4.31E-06	1.29E-05	8.73E-06
m (shear)	0.214	0.016	0.427	0.227	0.289	0.703	0.247	0.197	0.179
Tensile strength at 25°C (psi)	226	226	226	150	150	150	132	132	132
D* @12°C, 10Hz	0.279	0.300	0.313	0.483	0.494	0.588	0.505	0.474	0.426
Ni (lab)	231950834	1859466	2981023	3969475	381493	74422	367254	9694942	13606
Ni (field)	535142601	4290046	6877631	9158127	880157	171701	847307	22367571	31391
Np (field)	8.479E-06	2.217E-06	9.397E-06	9.072E-06	9.508E-06	1.087E-05	9.294E-06	8.974E-06	8.653E-06
Nf (field)	535142601	4290045	6877631	9158126	880157	171700	847306	22367570	31390
Nf Mean (number loads to failure)	182103426			3403328			7748755		

Table D4. (Continued)

	Carbon Black			Control			EVA	
Sigma x	5.65	1.26	5.18	2.51	17.17	1.89	0.92	0.97
Sigma y	19.44	9.54	18.26	10.93	292.30	8.71	3.28	5.66
Sigma z	39.59	26.22	38.05	28.74	329.10	26.11	19.93	22.29
Mean principal stress, psi	21.56	12.34	20.50	14.06	212.86	12.24	8.04	9.64
Octahedral shear stress, psi	13.94	10.38	13.51	10.93	139.18	10.20	8.46	9.15
Asphalt concrete modulus, psi	522881	528522	490627	335206	862018	284643	268111	310872
Asphalt content by weight percent	5.3	5.3	5.3	5.3	5.3	5.3	5.3	5.3
Air voids content, percent	3.8	3.4	2.0	2.0	2.0	2.0	2.0	2.0
Poisson's ratio @ 12°C, 10Hz	0.488	0.194	0.467	0.307	0.905	0.249	0.999	0.112
Average daily traffic (truck/day)	3950	3950	3950	3950	3950	3950	3950	3950
Height of A/C surface (in)	3	3	3	3	3	3	3	3
Height of Base (in)	8	8	8	8	8	8	8	8
D1 (shear)	1.54E-06	7.15E-06	3.93E-06	6.41E-06	2.68E-07	4.09E-06	1.24E-05	5.29E-06
m (shear)	0.295	0.349	0.209	0.222	0.999	0.212	0.194	0.234
Tensile strength at 25°C (psi)	174	174	174	252	252	252	128	128
D* @12°C, 10Hz	0.277	0.274	0.296	0.433	0.168	0.509	0.541	0.466
Ni (lab)	170898	1760257	401587	2796187	2732304	5915460	53456239	19425522
Ni (field)	394285	4061156	926516	6451190	6303802	13647783	123330925	44817362
Np (field)	9.337E-06	9.899E-06	8.724E-06	8.758E-06	1.068E-05	8.573E-06	8.950E-06	9.223E-06
Nf (field)	394285	4061155	926515	6451189	6303802	13647782	123330925	44817361
Nf Mean (number loads to failure)	1793985			8800925			84074143	