NCHRP

REPORT 459

NATIONAL
COOPERATIVE
HIGHWAY
RESEARCH
PROGRAM

Characterization of Modified Asphalt Binders in Superpave Mix Design

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state and local governmental agencies, universities, and industry; its relationship to the National Research Council is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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The members of the technical committee selected to monitor this project and to review this report were chosen for recognized scholarly competence and with due consideration for the balance of disciplines appropriate to the project. The opinions and conclusions expressed or implied are those of the research agency that performed the research, and, while they have been accepted as appropriate by the technical committee, they are not necessarily those of the Transportation Research Board, the National Research Council, the American Association of State Highway and Transportation Officials, or the Federal Highway Administration, U.S. Department of Transportation.

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FOREWORD

By Staff
Transportation Research Board

This report presents the findings of a research project to evaluate the applicability of AASHTO MP1, “Standard Specification for Performance Graded Asphalt Binder,” to modified asphalt binders. Its main finding is that the current AASHTO MP1 specification typically underestimates the potential performance of modified asphalt binders; changes to the specification and its supporting test methods are recommended to remedy this situation. The report will be of particular interest to materials engineers in state highway agencies as well as to materials suppliers and paving contractor personnel responsible for the production and use of modified asphalt binders for hot-mix asphalt pavement construction.

Modification of asphalt binders can serve several purposes. It can enhance the overall performance of a binder by widening the range between the binder’s high- and low-temperature grades, or it can target a specific improvement in a binder’s performance in response to a particular severe-service condition, such as a pavement carrying a very high traffic volume or a high percentage of slow-moving, heavy vehicles.

Many diverse materials are added to neat asphalt cement as modifiers. The elastomers styrene-butadiene rubber and styrene-butadiene-styrene block copolymer are widely used. Plastomers such as polyethylene and ethylene vinyl acetate are also marketed as modifiers, as are hydrated lime, elemental sulfur, gilsonite, and crumb rubber prepared from scrap tires. However, modification may also entail processing neat asphalt cement to enhance its performance; airblowing (i.e., oxidation) and steam distillation are good examples of such processes.

Estimates of the extent of use of modified asphalt binders in hot-mix asphalt (HMA) pavement construction vary, but it is likely that modified binders represent as much as 15 percent of the total annual tonnage of asphalt binder used in the United States; this percentage is expected to increase in the coming decade. These binders represent a significant cost component of HMA construction. Depending on the type of modification, the cost per ton of a modified binder may be 50 to 100 percent greater than that of neat asphalt cement, translating to an increase of 10 to 20 percent in the cost of in-place HMA.

These increased costs are reasonable if modified asphalt binders truly improve pavement performance to the degree expected. From 1987 through 1993, the Strategic Highway Research Program (SHRP) carried out a major research program to develop the Superpave® performance-based specifications and test methods for asphalt binders and similar tests and a mix design practice for HMA mixes. One goal of this program was to make the specifications and tests “transparent” to the use of modified binders—that is, to ensure that the specifications would accurately measure the enhanced performance characteristics of the modified binders. However, the SHRP asphalt research was carried out almost exclusively with unmodified asphalt cements, so the applicability of the Superpave specifications and test methods to modified binders was not validated.

In practice, modified asphalt binders graded according to the requirements of AASHTO MP1 do show marked improvements in selected performance characteristics compared with neat asphalt cements. However, users and producers of modified asphalt binders remain concerned that the current specification and test methods do not fully measure the performance enhancement contributed by the modification.

Under NCHRP Project 9-10, “Superpave Protocols for Modified Asphalt Binders,” the Asphalt Institute of Lexington, Kentucky, supported by the University of Wisconsin Asphalt Group of Madison, Wisconsin, and the National Center for Asphalt Technology of Auburn, Alabama, was assigned the tasks of recommending changes to AASHTO MP1 and its supporting test methods to fully characterize modified asphalt binders and validating those recommendations through laboratory performance testing of modified HMA.

The research team (1) surveyed highway agencies and materials suppliers on the use and performance of modified asphalt binders; (2) planned and carried out a comprehensive laboratory testing program to fully characterize a large, representative set of modified binders; (3) explored possible changes to the current AASHTO MP1 and its supporting test methods to better characterize the performance of modified binders; and (4) carried out a program of laboratory performance testing of modified HMA mixes to validate the effectiveness of the potential changes to the binder specification and test methods.

The research team found that the current AASHTO MP1 specification does not adequately characterize the performance of modified asphalt binders; typically, the binders’ potential performance is underestimated. The concepts of viscous flow and energy dissipation were explored in an effort to derive binder parameters that more effectively relate binder to mixture performance. Suggested specification parameters and test protocols were developed for three concepts: (1) permanent deformation, the viscous component of the binder creep stiffness, G_V, measured by a repeated creep test in the dynamic shear rheometer (DSR); (2) fatigue cracking, the number of cycles to crack propagation, N_p, measured by a repeated cyclic loading test in the DSR; and (3) low-temperature cracking, a direct measurement of the binder’s glass-transition temperature combined with failure stress and strain for a region-specific design cooling rate measured with the bending beam rheometer and the direct tension device. In addition, the research developed a practice in the AASHTO format for characterization of modified asphalt binders and new test methods for storage stability, particulate additive content, and laboratory mixing and compaction temperatures. Finally, the research found that the current Superpave mixture tests (AASHTO TP7, “Standard Test Method for Determining the Permanent Deformation and Fatigue Cracking Characteristics of Hot Mix Asphalt [HMA] Using the Simple Shear Test [SST] device,” and TP9, “Standard Test Method for Determining Creep Compliance and Strength of Hot Mix Asphalt [HMA] Using the Indirect Tensile Test Device”) satisfactorily characterize the performance of modified HMA mixes.

This final report includes a detailed description of the experimental program, a discussion of the research results, and five supporting appendixes:

- Appendix A, Summary of Mix Designs;
- Appendix B, Internal Advisory Group;
- Appendix C, Practice for Laboratory Evaluation of Modified Asphalt Binders;
- Appendix D, Results of Survey of Modified Asphalts; and
- Appendix E, Recommendations for Changes to the Present AASHTO Standards.

The entire final report, including all appendixes and several previously unpublished interim and topical reports, along with selected interim reports and the final reports for NCHRP Projects 9-14 and 9-19, will be distributed as a CD-ROM (*CRP-CD-9*). The research results have been referred to the TRB Binder Expert Task Group for its review and possible recommendation to the AASHTO Highway Subcommittee on Materials for adoption as new or revised specifications, test methods, and recommended practices.

CONTENTS

AUTHOR ACKNOWLEDGMENTS

The research report herein was performed under NCHRP Project 9-10, “Superpave Protocols for Modified Asphalt Binders.” The guidance of the Internal Advisory Group that was formed during the project from experts involved in the TRB Binder Expert Task Group is greatly appreciated. Many of the important achievements in this project would not have been possible without their critical review and recommendations.

This report is a collective effort of many individuals. The principal investigator was Dr. Hussain U. Bahia at the University of Wisconsin-Madison. The co-principal investigator was Mr. Douglas I. Hanson at the National Center for Asphalt Technology. The other authors are Dr. Menglan Zeng, Dr. Huachun Zhai and Mr. Arif M. Khatri at the University of Wisconsin-Madison, and Mr. Michael R. Anderson at the Asphalt Institute. The authors wish to acknowledge the contributions of:

- Ms. Pamela Turner and Mr. Michael Anderson, the Asphalt Institute, for supervising the collection of the mixture FSCH, RSCH and Beam Fatigue testing data, and preparing the mixture designs with the limestone aggregates.
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- Mr. Barry Paye, University of Wisconsin-Madison, for collecting materials for the validation tasks under Task 10B.

CHARACTERIZATION OF MODIFIED ASPHALT BINDERS IN SUPERPAVE MIX DESIGN

SUMMARY

This report documents and presents the results of a study of the applicability of the Superpave® specification and protocols developed for asphalt cements to modified asphalt binders. A survey of users and suppliers of modified binders indicated the existence of a wide variety of modifiers that are used either routinely or on an experimental basis to produce PG grades. The survey also indicated that a majority of state agencies intend to increase the future use of modified binders. However, serious concerns remain about the lack of knowledge about how modified binders affect performance, as well as with issues related to storage stability, suitability of the Superpave aging procedures, and the lack of protocols to determine mixing and compaction temperatures.

The focus of NCHRP Project 9-10, "Superpave Protocols for Modified Asphalt Binders," was on solving the most cited concerns in the survey. To evaluate the applicability of Superpave rheological protocols to modified binders, advanced rheological characterization of a selected set of 50 binders was conducted to define the range in properties of commonly used modified binders. The binders included performance grades varying between PG 82-22 and PG 58-40 that were produced with two base asphalts and 17 types of generic modifiers. The results indicated that the simplifying assumptions in the current Superpave specification (AASHTO MP1, "Standard Specifications for Performance Graded Asphalt Binder") limit its applicability to the majority of the modified binders tested. Moreover, the results indicated that many modified binders are highly nonlinear and sensitive to mechanical working. Finally, it was found that the modified binders vary significantly in their sensitivity to traffic speed, to traffic volume, and to stress or strain level that varies according to pavement structure.

Based on this fuller understanding of modified binder behavior, specialized rheological and failure characterization of a selected set of nine binders and 36 mixtures was conducted using revised protocols based on the binder testing equipment developed under the Strategic Highway Research Program. The selected set included binders modified with elastomers and plastomers or modified by oxidation. The mixtures were produced with limestone and gravel aggregates, each combined in a fine and a coarse gradation that meet the Superpave mixture requirements. The results indicated that the present AASHTO MP1 specification parameters did not currently rank the modified binders relative to their contribution to mixture damage. The concepts of nonlinear viscoelasticity

and energy dissipation were explored to derive better binder parameters that can more effectively relate binder to mixture behavior. It was found that characterization of damage behavior of binders is necessary and can be achieved using the test equipment currently supporting AASHTO MP1 with modification of protocols. Test protocols and specification parameters were developed as means to integrate the new concepts in a future specification.

To establish a better rating of the role of binders in mixture rutting, the parameter G_v, defined as the viscous component of the creep stiffness, is recommended to replace the current binder parameter $G^*/\sin\delta$. This parameter is measured with a newly developed repeated creep test conducted with the dynamic shear rheometer (DSR).

To establish a better rating of the role of binders in mixture fatigue, the parameter N_p, defined as the number of cycles to crack propagation, is recommended to replace the current binder parameter $G^*\sin\delta$. The new parameter is measured with a newly developed repeated cyclic loading conducted with the DSR.

As for thermal cracking, the study focused on measuring the effect of modifiers on the binder's thermo-volumetric and failure properties. Binders of equal PG grade were found to vary significantly in their glass transition behavior, in their low temperature ductility (i.e., strain at failure), and in their strength (i.e., stress at failure). Cooling rate plays a role in changing failure properties and thermal stress development. The effect of cooling rate is highly modifier specific. It is, therefore, recommended that that binder specification include a direct measure of the glass transition temperature and a consideration of a region-specific design-cooling rate.

Revisions to the binder grading system are also suggested to include a three-level grading scheme depending on which factors are considered in the binder selection. For Level I grading, only climate is considered; for Level II, traffic conditions are added; and for Level III, climate, traffic, and pavement structure are all considered.

A storage stability test and a particulate additive test were also developed and proposed as standard tests to evaluate modified binders. The concept of low shear viscosity was introduced to evaluate the effect of modified binders on laboratory mixing and compaction, and a viscosity level was selected to avoid excessive heating and to consider the shear-rate dependency of modified binders.

Superpave mixture testing protocols were also evaluated for their effectiveness in characterizing the performance of mixtures produced with modified asphalts. It is found that the frequency sweep constant height (AASHTO TP7) procedure could be made more useful by considering the effect of pavement structure through using multiple strain levels during testing. The repeated shear constant height (AASHTO TP7) was modified to include higher stress levels and higher temperatures. Although the beam fatigue (AASHTO TP8) test was found to be useful, its protocol could be more effective if the test is extended to a greater number of cycles and if the results are analyzed in light of the concept of dissipated energy ratio by defining the number of cycles at which crack propagation starts. Low-temperature testing of mixtures was found to be more effective when the optional 1,000-s testing time was used.

Although the suggested test protocols and specification revisions are based on known scientific concepts and were shown to improve the correlation between binder and mixture behavior, a field validation plan is recommended to rigorously test the validity of the concepts and derive specification criteria.

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

This section of the report presents the statement of the problem, objectives, and descriptions of the phases and tasks of the research project.

1.1 STATEMENT OF PROBLEM

NCHRP Project 9-10, "Superpave Protocols for Modified Asphalt Binders," was initiated to confirm whether the binder and mixture test methods of Superpave®, an asphalt–aggregate mixture design and analysis system developed under the Strategic Highway Research Program (SHRP), are generally suitable for use with modified asphalt binders. If they are not, the Superpave methods may need to be altered to better characterize modified binders and hot-mix asphalt (HMA) containing modified binders. By the same token, the specification limits, criteria, and models developed through the SHRP asphalt research program may require revision.

1.2 OBJECTIVES

The objectives of this research project were to

1. Recommend modifications to Superpave asphalt binder tests for modified asphalt binders, and
2. Identify problems with the Superpave mixture performance tests in relation to mixtures made using modified binders.

The project did not include a significant field validation of the results. Additional research work, using field performance data from other projects, is needed to evaluate and refine the findings of this project. This additional research, if conducted, should be focused on establishing appropriate binder specification limits for modified asphalt binders, as well as criteria and models for mixtures containing modified asphalt binders.

1.3 DESCRIPTIONS OF PROJECT TASKS

The following 11 tasks were undertaken to accomplish the project objectives:

Task 1. Define, identify, and categorize asphalt modifiers; evaluate available information and research (completed and ongoing) on modified asphalt binders in the Superpave system; survey organizations using Superpave binder and mixture procedures; and determine issues associated with the use of modified asphalt binders and any other information related to the objectives of this project.

Task 2. Prepare an experimental plan and develop a protocol to fully characterize a representative set of modified asphalt binders (i.e., base asphalt, modifiers, and levels of modification).

Task 3. Not later than 6 months from project initiation, submit an interim report documenting the results of the work accomplished in Tasks 1 and 2, and provide a detailed plan for Tasks 4 and 5.

Task 4. Conduct the modified asphalt binder characterization program as approved in Task 3.

Task 5. Based on the results of the Task 4 characterization and any appropriate additional information, evaluate and recommend modification (if necessary) to the Superpave asphalt binder tests to allow their use with modified asphalt binders.

Task 6. Prepare an experimental plan to evaluate the effect of modified asphalt binders on the Superpave mixture performance tests; include a consideration of materials and data from the FHWA Long-Term Pavement Performance (LTPP) Specific Pavement Studies (SPS-9), "Field Validation of the SHRP Asphalt Specifications and Mix Design," and NCHRP Project 9-7, "Field Procedures and Equipment to Implement SHRP Asphalt Specifications."

Task 7. Not later than 12 months from the project initiation, submit an interim report documenting the results of the work accomplished in Tasks 1 through 6 and include a list of AASHTO Superpave binder tests, standards, and specifications proposed for modification; also list any new protocols that need to be developed; and present the experimental plan developed in Task 6.

Task 8. Conduct the mixture performance test program as approved in Task 6 on a limited number of materials.

Task 9. Evaluate and revise the Superpave mixture design mixing and compaction temperature requirements for modified asphalt binders; evaluate the effect of modified asphalt binders on the quality control/quality assurance (QC/QA) practice developed in NCHRP Project 9-7.

Task 10. Recommend detailed revisions to the AASHTO Superpave asphalt binder tests, standards, and specifications; use AASHTO MP2 ("Standard Specification for Superpave

Volumetric [Level I] Design”), AASHTO PP28 (“Standard Practice for Superpave Volumetric [Level I] Design for Hot-Mix Asphalt [HMA]”), and AASHTO PP5 (“Standard Practice for Laboratory Evaluation of Modified Asphalt Binders”) as reference materials; recommend revisions (if necessary) to the NCHRP Project 9-7 QC/QA practice as it relates to modified asphalt binders; also recommend protocols for other Superpave asphalt binder tests (as necessary) in AASHTO format.

Task 11. Prepare a final report that documents the entire research effort and includes the Task 9 products as a stand-alone appendix.

1.4 OBJECTIVES AND EXPERIMENTAL PLANS OF PROJECT PHASES

The project was divided into three phases. For each phase, a set of objectives was defined and a separate experimental plan was developed.

1.4.1 Phase I

Two main objectives were defined for the first phase of the project:

1. To identify the most commonly used modified asphalts and concerns regarding the use of such asphalts, and
2. To develop an experimental plan for an advanced characterization of approximately 50 modified binders of various grades.

To best accomplish these objectives the research team followed the methodology described in Figure 1.1.

The research team conducted a literature search focused on modified asphalt testing and on asphalt modifiers. In addition, a detailed survey questionnaire was developed and sent to state agencies, industry organizations, and academic institutions involved in using, producing, or studying modified binders.

The results of the survey and the literature search were presented in the first interim report and summarized in a stand-alone report that was delivered in the second quarter of 1997. The summary of the survey is discussed in Chapter 2.

1.4.2 Phase II

The results of Phase I of the project were used to revise the working plan originally proposed for Phase II. A new classification system of modified binders was suggested that separated modified binders into simple binders and complex binders. A total of 17 modifiers and two base asphalts with different chemical compositions were selected for the study. The main revisions were focused on Task 4. The revised experimental program for Task 4 included three main work elements:

- Task 4A: Characterization of Selected Simple Modified Binders;
- Task 4B: Characterization of Selected Complex Modified Binders; and
- Task 4SS: Special Studies to Evaluate Properties of Modified Binders:
  - Subtask 4SS1: Effect of Size and Concentration of Particulate Additives,
  - Subtask 4SS2: Protocol for Measuring Potential of Thermal Degradation of Additives,
  - Subtask 4SS3: Protocol for Separation of Effects of Oxidative Aging During the Pressure-Aging Vessel (PAV) Procedure, and
  - Subtask 4SS4: Protocol for Measuring Potential for Phase Separation of Additives.

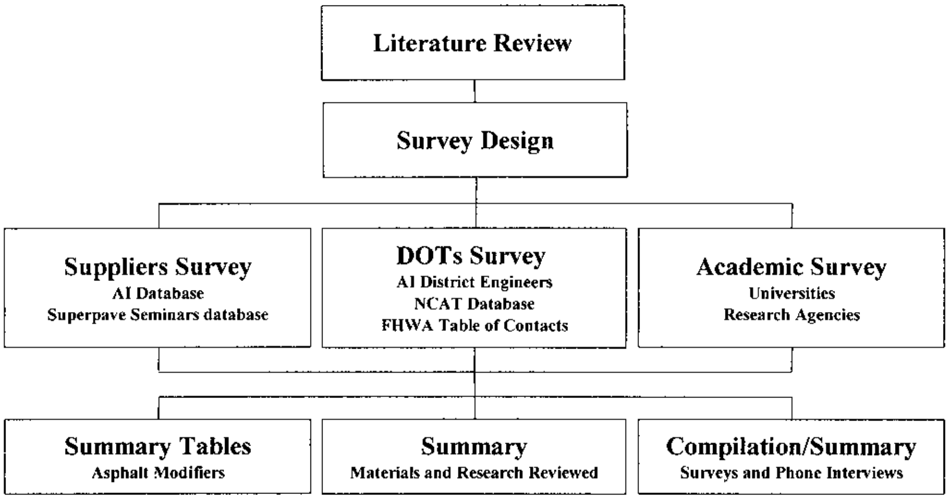


Figure 1.1 Information-gathering plan conducted in Phase I.

Task 4A, which was the largest task, focused on testing simple modified binders. Task 4B tested binders modified with selected additives known to result in complex binder systems. Task 4SS focused on measuring special characteristics of modified binders and on developing specific testing protocols that are not part of the current Superpave testing protocols.

The hypothesis in Phase II was that the role of simple binders in mixture and pavement performance could be estimated using the existing (or revised) Superpave binder protocols, regardless of their constituents or the method of production. On the other hand, the role of complex binders in mixture and pavement performance cannot be estimated using binder testing; mixture testing is necessary. It was hypothesized that an asphalt binder can be classified as a complex binder because of the physical characteristics of the modifier or the nature of the effect of the modifier. Binders modified with particulate matter can be complex because of their dependency on sample geometry. Other binders can be complex because they are thixotropic or strain dependent.

The experimental testing plan for Phase II included 17 generic modifiers selected after a review of the existing literature about modifiers used in practice. The supplier of each modifier was asked to modify two base asphalts to a grade chosen as the basis of its marketing strategy. Table 1.1 shows the possible combinations that were targeted.

The testing plan included an extended set of protocols to characterize the rheological, failure, stability, and aging properties of the selected binders. Numerical modeling of the collected data was used to estimate some of the important rheological and failure characteristics. Table 1.2 lists the testing protocols that were used in order to fully characterize the selected modified binders.

The results of Phase II and the analysis are presented in the second interim report of the project, which was completed in the first quarter of 1998. A summary of these results is discussed in Chapter 2.

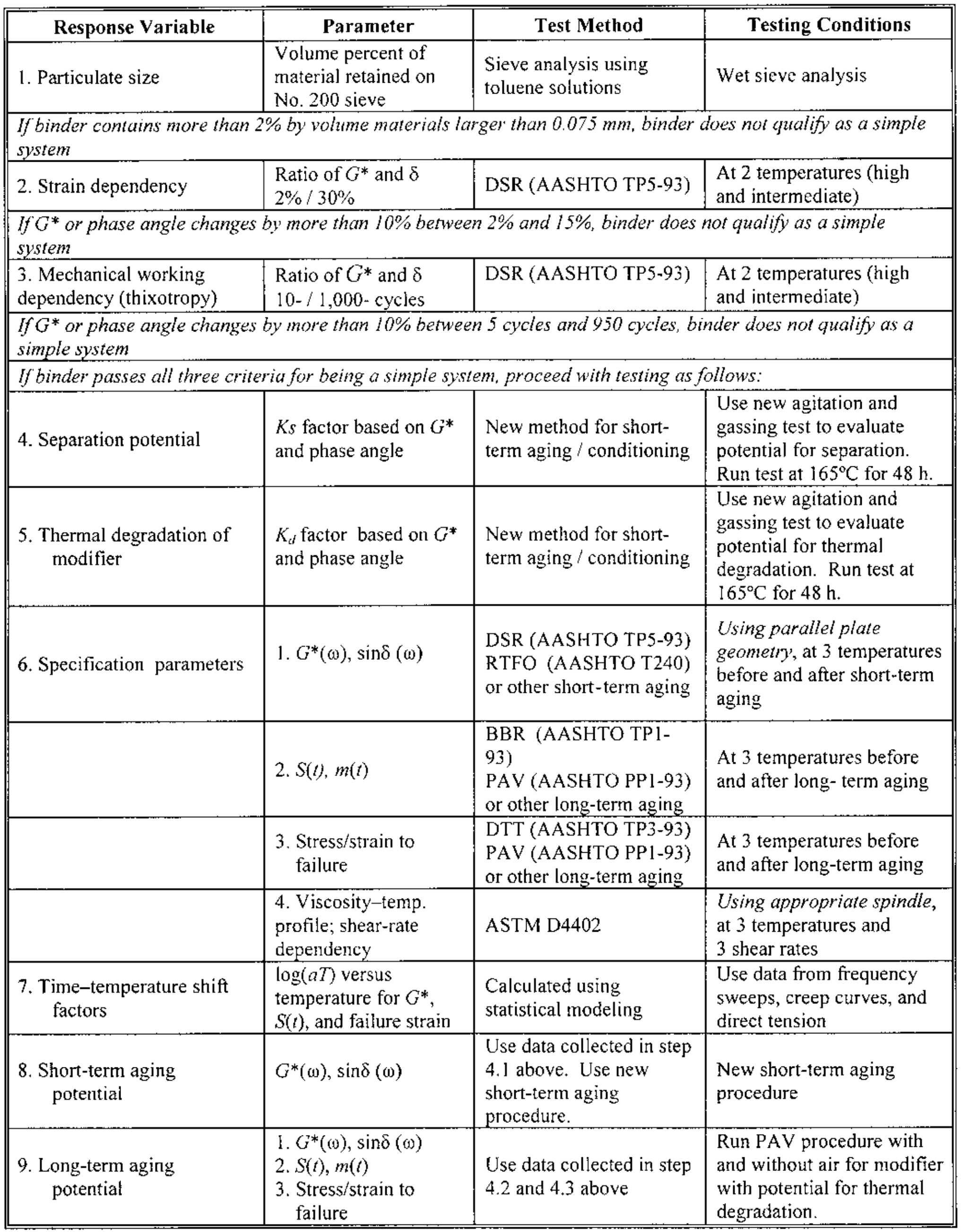
Temperature is an important variable for a study on paving materials. Three pavement temperatures are used in the studies

TABLE 1.1 Representative modified asphalt binders considered in the experimental design of Phase II

Asphalt Source	**Boscan – AAK**					**California Valley - AAG**				
Asphalt Grade	**70-X**		**58-28**	**52-40 or 46-40**		**70-X**		**58-28**	**52-40 or 46-40**	
Grade Improvement	6	12	6/-6	6/+6	12/+6	6	12	6/-6	6/+6	12/+6
Minimum in °C			or	or	or			or	or	or
High Temp / Low Temp			12/-6	Jun-00	Dec-00			12/-6	Jun-00	Dec-00
Modified Grade	76-X	82-X	64-34	58-34	64-34	76-X	82-X	64-34	58-34	64-34
(Concentration)			or	or	or			or	or	or
			70-34	52	58-40			70-34	52-40	58-40
Modifier Type	High Temp		Low Temp			High Temp		Low Temp		
1. Styrene-Butadiene-Styrene (SBS) Triblock										
2. SBS Radial										
3. Styrene-Butadiene (SB) Di-block										
4. Styrene-Butadiene rubber (SBR) LMW										
5. SBR HMW										
6. Ethylene vinyl acetate										
7. Ethylene Terpoly										
8. Polyethylene (PE) unstabilized										
9. PE stabilized										
10. Hydrated Lime										
11. Gelled asphalt cement (AC)										
12. Gilsonite										
13. Oxidized Back-blended (BB)										
14. Oxidized Straight run (SR)										
15. Steam Distilled										
16. Polyamines										
17. Amidoamines										

NOTE: AAK and AAG are asphalt codenames that were used in the original Strategic Highway Research Program asphalt research program.

TABLE 1.2 Testing protocol and response variables proposed for full characterization of modified binders



Response Variable	Parameter	Test Method	Testing Conditions
1. Particulate size	Volume percent of material retained on No. 200 sieve	Sieve analysis using toluene solutions	Wet sieve analysis
If binder contains more than 2% by volume materials larger than 0.075 mm, binder does not qualify as a simple system			
2. Strain dependency	Ratio of G^* and δ 2% / 30%	DSR (AASHTO TP5-93)	At 2 temperatures (high and intermediate)
If G^* or phase angle changes by more than 10% between 2% and 15%, binder does not qualify as a simple system			
3. Mechanical working dependency (thixotropy)	Ratio of G^* and δ 10- / 1,000- cycles	DSR (AASHTO TP5-93)	At 2 temperatures (high and intermediate)
If G^* or phase angle changes by more than 10% between 5 cycles and 950 cycles, binder does not qualify as a simple system			
If binder passes all three criteria for being a simple system, proceed with testing as follows:			
4. Separation potential	K_s factor based on G^* and phase angle	New method for short-term aging / conditioning	Use new agitation and gassing test to evaluate potential for separation. Run test at 165°C for 48 h.
5. Thermal degradation of modifier	K_d factor based on G^* and phase angle	New method for short-term aging / conditioning	Use new agitation and gassing test to evaluate potential for thermal degradation. Run test at 165°C for 48 h.
6. Specification parameters	1. $G^*(\omega)$, $\sin\delta$ (ω)	DSR (AASHTO TP5-93) RTFO (AASHTO T240) or other short-term aging	*Using parallel plate geometry*, at 3 temperatures before and after short-term aging
	2. $S(t)$, $m(t)$	BBR (AASHTO TP1-93) PAV (AASHTO PP1-93) or other long-term aging	At 3 temperatures before and after long- term aging
	3. Stress/strain to failure	DTT (AASHTO TP3-93) PAV (AASHTO PP1-93) or other long-term aging	At 3 temperatures before and after long-term aging
	4. Viscosity–temp. profile; shear-rate dependency	ASTM D4402	*Using appropriate spindle*, at 3 temperatures and 3 shear rates
7. Time–temperature shift factors	log(aT) versus temperature for G^*, $S(t)$, and failure strain	Calculated using statistical modeling	Use data from frequency sweeps, creep curves, and direct tension
8. Short-term aging potential	$G^*(\omega)$, $\sin\delta$ (ω)	Use data collected in step 4.1 above. Use new short-term aging procedure.	New short-term aging procedure
9. Long-term aging potential	1. $G^*(\omega)$, $\sin\delta$ (ω) 2. $S(t)$, $m(t)$ 3. Stress/strain to failure	Use data collected in step 4.2 and 4.3 above	Run PAV procedure with and without air for modifier with potential for thermal degradation.

NOTE: RTFO = rolling thin film oven; BBR = bending beam rheometer; DTT = direct tension test.

of this project: high temperature (HT), intermediate temperature (IT), and low temperature (LT). In this report, HT and LT are defined as they are in AASHTO M P1. IT is defined as the temperature at which $G^*\sin\delta = 5{,}000$ kPa at 10 rad/s (1.59 Hz) for PAV-aged binder.

1.4.3 Phase III

The objectives of Phase III of the project were to verify the findings from Phase II through Superpave performance testing of mixtures made with modified asphalt binders and to recommend any needed modifications to those procedures. The specific objectives by task were to

- Fully characterize a selected set of asphalt mixtures made with modified asphalt binders (Task 8);
- Revise mixing and compaction temperatures (Task 9);
- Recommend needed modifications to the existing Superpave AASHTO standards (Task 10); and
- Submit a final report (Task 11).

Based on the results of Phase II, nine modified binders were selected for the mixture testing in Phase III. These binders included three PG 82-22 binders, three PG 70-34 binders, and three PG 58-40 binders. Two aggregate sources and two gradations were included in the experimental designs. Testing included the use of the Superpave shear test (SST, AASHTO TP7) and the indirect tension test (IDT, AASHTO TP9). It also included limited testing, using the triaxial testing system proposed as a Superpave simple performance test and as an HMA characterization method suggested for the 2002 Pavement Design Guide (under development in NCHRP Project 1-37A, "Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures: Phase II").

The original plan for Tasks 8 and 9 was expanded to include more aggregate variables and to conduct more comprehensive testing at additional strain levels and temperatures. The original plan for Task 10 was also revised to include three subtasks focused on soliciting expert opinions, conducting limited field validation, and carrying out an expanded experiment on the direct tension and glass-transition behavior of selected binders. The following briefly describes the work plan for each task in Phase III.

1.4.3.1 Task 8

In Task 8, performance testing was conducted on 36 different mixtures produced from nine selected modified binders and four types of aggregates from two sources. The modified binders were chosen based on test results from Phase II. Each source of aggregates was used to produce a fine gradation and a coarse gradation.

Each of the 36 mixtures was tested at high, intermediate, and low temperatures using the Superpave recommended protocols. The testing plan was designed to characterize asphalt mixture properties at a range of temperatures from –20 to 70°C.

This task tested two hypotheses:

1. The relationships between binder and mixture rheological properties and between binder and mixture failure properties are independent of the composition of the binder or the type of additive used; and
2. The specification parameters used in the binder specification, in its current revised format, are good predictors of mixture performance for a given mixture design.

The controlled material variables chosen for Task 8 are shown in Table 1.3. Two aggregate sources were used in the project. One source was the Asphalt Institute's laboratory standard aggregate, a moderately low water absorption (less than 1.5 percent) central Kentucky limestone, representing the crushed aggregates. The other source was an aggregate from Alabama, representing the less angular, mostly uncrushed gravel source, which is commonly used by the National Center for Asphalt Technology (NCAT) as a laboratory standard.

A coarse aggregate gradation (i.e., below the restricted zone) and a fine aggregate gradation (i.e., above the restricted zone) were developed for each source. Both gradations have a nominal maximum aggregate size (NMAS) of 12.5 mm. This NMAS is commonly used for high-traffic-wearing course mixtures, in which modified asphalt binders are most often used. Its smaller size also made specimen preparation easier and reduced the material variability for advanced mixture testing. The complete mixture designs are presented in Appendix A.

Testing was conducted at the design asphalt content determined by Superpave volumetric mixture design procedures to yield 4.0 percent air voids at the design number of gyrations (96) using the PG 70-22 Boscan base asphalt. The mixture design, and subsequent determination of design asphalt content, was accomplished for each aggregate type.

The nine asphalt binders were classified into three groups: the first group represented binders that have superior HT properties. This group included one binder modified with an elastomeric modifier (i.e., styrene-butadiene-styrene [SBS] radial), one modified with a plastomeric modifier (i.e., polyethylene [PE] stabilized), and one modified by processing in the refinery (i.e., steam distilled).

The second group represented binders with superior IT properties and HT properties. This group included one binder modified with an elastomeric modifier (i.e., styrene-butadiene rubber (SBR) low molecular weight [LMW]), one modified with a plastomeric modifier (i.e., ethylene terpoly), and one modified by processing in the refinery (i.e., oxidized).

The third group represented binders with superior LT properties. This group included two binders modified with elastomeric modifiers (i.e., styrene-butadiene (SB) di-block and SBS linear) and one modified by refinery processing (i.e., oxidized-back blended). An attempt was made to keep the critical temperature for each group as close as possible to each

TABLE 1.3 Task 8 material control variables

Variable	Level
Aggregate Source/Angularity	2 (Crushed limestone and gravel)
Mixture Gradation	2 (12.5 mm coarse and 12.5 mm fine)
Asphalt Binder Content	1 (Design)
Asphalt Binder	9 (Based on Phase II results)

other. The performance grades of the nine binders and the critical temperatures at which they meet the grade are shown in Table 1.4.

Although the binders were divided into three groups, each binder was tested at all selected temperatures and analyzed for all properties. This experimental design allowed a critical evaluation of the effect of binder composition. It also permitted the studying of the effect of grade within each generic type of modifier.

The testing included three temperature ranges: high, intermediate, and low. At each range, test procedures that measure both moduli and failure (damage) properties were performed. The tests were conducted at multiple loading frequencies, loading patterns, and strain levels.

In the HT range, the repeated shear at constant height (RSCH) and the frequency sweep at constant height (FSCH) tests were conducted with the SST. Both tests were developed during SHRP as part of the A-003A contract at the University of California-Berkeley. Test procedures are described in AASHTO TP7.

The testing conditions were modified in order to evaluate some new ideas that the research team introduced. The modifications included multiple strain levels in the FSCH, a wider range of temperatures, and increasing stress levels and temperatures in the RSCH testing to better simulate actual failure conditions.

The FSCH test is a controlled strain test in which a selected shear strain is applied to the mixture specimen using a sinusoidal load. As the shear stress is applied, the specimen height tends to increase, and the axial stress increases to maintain a constant height. Testing is executed at multiple frequencies from 30 Hz to 0.01 Hz. The number of load cycles at each frequency varies from 150 cycles (30 Hz) to 4 cycles (0.01 Hz). Testing is commonly conducted at high and intermediate pavement temperatures, calculated from the project's climate data.

FSCH testing in Task 8 was conducted at 6, IT, 40, 46, and 52°C. FSCH testing was modified in two particulars from the procedure described in AASHTO TP7. First, an additional loading frequency (i.e., 30 Hz) was added at each temperature. Thus, the loading frequencies for each temperature were 30, 10, 5, 2, 1, 0.5, 0.2, 0.1, 0.05, 0.02, and 0.01 Hz. In addition, FSCH testing was conducted at four shear strain levels—0.01 (standard), 0.04, 0.07, and 0.1 percent—at each temperature.

The FSCH test was replicated for each combination of frequency, temperature, and strain. The average values of the complex shear moduli and phase angles were analyzed to assess their dependence on frequency, temperature, and strain level.

The output of the FSCH test provides data on the viscoelastic properties of the asphalt mixture. By using the complex shear modulus (G^*) and phase angle, the viscous (G'') and elastic (G') components of the modulus can be calculated. A relative comparison of HT mixture stiffness can be made among various asphalt mixtures by comparing their complex shear moduli values.

Data collected were used to evaluate the effects of binder grades and modification type on the rheological properties of mixtures produced with the four different aggregates. The effects of binders were compared with the effects of aggregate and temperature. The analysis was to

- Quantify the relationships between binder G^* values and mixture G^* values at selected conditions of temperature, frequency, and strain.
- Quantify the sensitivity of mixture to selected changes in temperature, frequency, and strain, and compare with the binder sensitivity to the same or similar changes in these factors.
- Develop master curves of binders and mixtures and describe the role of binder modification and grade in changing the overall rheological behavior of mixtures.

In the IT range, the FSCH test was conducted to measure the rheological properties at the intermediate grade temperature and at 6°C. For the damage characterization, the beam fatigue (AASHTO TP8) test was conducted at the intermediate grade temperature using a selected strain level.

In the LT range, the IDT was conducted at three temperatures, 0, −10 and −20°C, as is recommended in AASHTO TP9. The optional loading time of 1,000 s was used in all creep tests. Failure time was measured in the failure procedure in order to estimate the strain at failure.

At least two replicate specimens were tested for each response measured. In the case of beam fatigue testing, three

TABLE 1.4 Binder matrix used in the experimental design

Grade	PG 82+IT+LT	PG HT+23+LT	PG HT+IT−40
Modifier Type			
Elastomeric	PG 82-22 SBS Radial (84.4°C)	PG 82-22 SBR LMW (22.7°C)	PG 58-40 SB Di-block (−39.0°C)
Plastomeric	PG 82-22 PE Stabilized (83.6°C)	PG 76-22 Ethylene Terpoly (23.0°C)	PG 58-40 SBS Linear (−39.0°C)
Processed	PG 82-22 Steam Distilled (84.0°C)	PG 76-22 Oxidized (24.1°C)	PG 52-40 Oxidized (−43.3°C)

replicates were measured. The average values of the measurements were used in the analyses. All data were carefully reviewed to ensure the quality of data for meaningful analyses.

1.4.3.2 Task 9

The objective of Task 9 was to evaluate and revise the Superpave mix design laboratory mixing and compaction temperature requirements for modified asphalt binders, as well as to assess the effect of modified asphalt binders on the QC/QA practice developed in NCHRP Project 9-7.

The current standard method for preparing specimens using the Superpave gyratory compactor is AASHTO T308, “Standard Method for Preparing and Determining the Density of Hot-Mix Asphalt (HMA) by Means of the Superpave Gyratory Compactor.” This method specifies a mixing temperature range of viscosity for unaged asphalt binders of 0.17 ± 0.02 Pa · s (1.0 Pa · s ≈ 1000 mm^2/s kinematic viscosity for asphalt binders), and a compaction temperature range of 0.28 ± 0.03 Pa · s.

For many of the modified binders tested in this project, these viscosities cannot be achieved unless the binder has been heated to very high temperatures. Such heating may result in excessive aging and, sometimes, in degrading the modifier. In order to prevent this aging and degrading, the recommendations of the modified binder supplier are typically followed. For some polymer modified binders, viscosities in the range of 0.30 to 0.40 Pa · s for mixing and 1.00 to 1.20 Pa · s for compaction were recommended. These ranges are mostly based on experience rather than scientific results and are mostly recommended for compaction methods other than the Superpave gyratory compactor. It is therefore necessary to evaluate the effect of viscosity level on the Superpave compaction procedure.

Using the results of viscosity testing collected in Task 4A and Task 4B, an experimental plan was developed to measure the sensitivity of the densification of selected modified mixtures to the viscosity of asphalt binders, by testing two hypotheses:

1. Asphalt binders can effectively coat aggregates at a wide range of viscosity levels higher than 0.17 ± 0.02 Pa · s, which is currently required by the Superpave procedure and other mixture design procedures.
2 The Superpave gyratory compactor will effectively compact with the same compaction effort at viscosity levels higher than 0.28 ± 0.03 Pa · s, which is currently required by the Superpave procedure and other mixture design procedures.

The controlled variables and their levels in this experiment included

- Aggregate source: 2 levels (crushed limestone and gravel);
- Aggregate gradation: 2 levels (coarse and fine);
- Binder type: 7 levels (control, 2 elastomeric, 2 plastomeric, and 2 particulate modifiers);
- Temperature at mixing: 3 levels (control, control + 15°C, control + 30°C); and
- Temperature at compaction: 3 levels (control, control + 15°C, control + 30°C).

Because HT performance grades PG 82 and PG 76 are considered especially critical, the following seven binders were selected:

1. PG 70-22 Boscan base asphalt (control),
2. PG 82-22 SBS radial (elastomer 1),
3. PG 82-22 SBR (LMW) (elastomer 2),
4. PG 82-22 PE stabilized (plastomer 1),
5. PG 76-22 ethylene terpoly (plastomer 2),
6. PG 82-22 hydrated lime (particulate additive), and
7. PG 82-22 crumb rubber (particulate additive).

The total number of combinations is 168, including 7 binders, 2 aggregate sources, 2 aggregate gradations, 3 temperatures for mixing, and 3 temperatures for compaction.

The procedure described in ASTM D2489 for evaluating the coating of aggregates in field mixtures was used to evaluate the effect of modifiers on mixing temperatures. Instead of field samples, 1,200- to 2,000-g samples of laboratory-prepared mixture were tested using two types of laboratory mixers. Both a Hobart bakery mixing apparatus, which is widely used in asphalt laboratories, and a conventional bucket apparatus, recently introduced to mix large samples, were used. The time required for complete coating was recorded based on duplicate measurements for each of the mixers. The reference time was the time required for complete coating with the base asphalt at viscosity levels currently required.

The Superpave gyratory compactor standard procedure was used to evaluate the effect of modifiers on compaction temperature. The main response variables were volumetric properties (i.e., percent air voids at N_{ini}, N_{des}, and N_{max}). The number of gyrations to reach an air voids content of 4 percent was also included as a main response in the analysis.

1.4.3.3 Task 10

The objective of Task 10 was to recommend necessary modifications to the following AASHTO standards:

- The AASHTO Superpave asphalt binder tests, standards, and specifications.
- The AASHTO MP2, “Standard Specification for Superpave Volumetric (Level I) Design”; AASHTO PP28, “Standard Practice for Superpave Volumetric (Level I) Design for Hot-Mix Asphalt (HMA)”; and AASHTO PP5 “Standard Practice for Laboratory Evaluation of Modified Asphalt Binders.”

- The NCHRP Project 9-7 QC/QA practice as it relates to modified asphalt binders.
- Any additional protocols for Superpave asphalt binder tests (as necessary) for modified binders in AASHTO format.

In March 1999, the project panel approved a modification of the work plan for Task 10 that included three new work elements. A new subtask was established for each of the work elements. The Task 10 subtasks were as follows.

- **Task 10A: Direct Tension and Glass-Transition Temperature**

The objective of this task was to evaluate the applicability of the most recent protocol developed by the TRB Binder Expert Task Group for estimating the cracking temperature of modified asphalt binders. The failure properties of modified binders tested in Task 4 of the project were measured using the most recent direct tension test procedure developed by TRB. The dilatometric glass-transition temperature (T_g) was also measured with a testing procedure originally used in the SHRP A-002A project.

- **Task 10B: Limited Field Validation Study**

The objective of this task was to test binders from 8 to 10 field projects or full-scale pavement testing experiments in which modified asphalts have been used and for which performance data have been collected. The testing was conducted using the new testing protocols to validate the relation between the test results and actual field performance.

- **Task 10C: Internal Advisory Group**

During Phase II, it became apparent that, for this project to be successful, it was necessary to provide the asphalt industry with the means for a detailed, ongoing review of the research data. Based on discussions with the project panel members, the TRB Expert Task Group members, and the project officer, the research team requested the approval to form an internal advisory group to discuss the development of specifications, to gather practical feedback on the new concepts being introduced, and to provide the research team with guidance on the proposed changes to specifications and the necessary validation before implementation. Members of the Internal Advisory Group were selected from federal and state agencies (3), asphalt industry and consulting (6), and academia (2). The group had five one-day meetings at the NCHRP offices, during which significant discussions of the concepts and the new testing protocols took place. The names of the members of the group and the minutes of two of its important meetings are given in Appendix B.

The outcome expected from Task 10 was a recommendation for a standard practice for advanced characterizations of asphalt binders that describes the revisions necessary to apply the Superpave binder specification to modified binders. The new standard, designated AASHTO PP5, is expected to offer a set of standard test procedures and tentative criteria that state agencies and industry can collectively use in order to better select modified asphalt binders.

A recommended draft of (new) AASHTO PP5 is presented in Appendix C of this report. In the opinion of the research team, it provides a better alternative for the characterization of modified binders than the SHRP-Plus specification that some state agencies are currently using for modified binders.

This final report concentrates on the results and the findings from the work conducted in Tasks 8, 9, and 10 of the project. It describes the relationships between mixture properties and the grade and type of modification of the asphalt binders used in the mixture. Key results from the earlier tasks are also summarized.

CHAPTER 2

FINDINGS

2.1 TYPES OF ASPHALT MODIFIERS

Asphalt modifiers can be classified in several ways: on the mechanism by which the modifier alters the asphalt properties; on the composition and physical nature of the modifier; or on the target asphalt property that needs improvement or enhancement. Based on the review of published literature (*1–12*), and the survey information collected in Phase I of the project (*13*), a list of the types of modifiers used currently in the asphalt industry is given in Table 2.1. The modifiers are classified based on the nature of the modifier and the generic types of asphalt modifiers. The target distress shown in the table corresponds to the main distress the additive is expected, or claimed, to reduce. The information is based on an interpretation of the published information for brands of modifiers that belong to the modifier classes shown. In many cases the reported effects are based on limited data and should not be generalized to all asphalt sources.

The information in Table 2.1 indicates that asphalt modifiers vary in many respects. They can be particulate matter or additives that will disperse completely or dissolve in the asphalt. They range from organic to inorganic materials, some of which react with the asphalt, while others are added as inert fillers. The modifiers generically vary in their specific gravity as well as in other physical characteristics. They are expected to react differently to environmental conditions such as oxidation and moisture effects. With such diversity in asphalt modifiers, it is clear that the current AASHTO MP1 specification may be too simplistic to characterize all these varieties of modified asphalts.

The literature review was extended to cover the methods used in characterizing modified asphalts and the difficulties observed in applying the Superpave binder testing protocols to modified binders. Emphasis was placed on identifying the characteristics that are not considered by the Superpave method, along with their importance.

2.2 CHARACTERISTICS NOT CONSIDERED BY THE SUPERPAVE BINDER SPECIFICATION AND TEST PROTOCOLS

Based on review of the published SHRP reports (*5,14–16*) and the information published in various technical journals and conference proceedings (*10,17–27*), it was found that the criteria in current Superpave binder specification, although they were to some extent validated for unmodified asphalts, might not be suitable for asphalts modified with various additives. In addition, modified asphalts may have performance-related characteristics that are not considered in the current specification (*28*). The following characteristics of modified asphalts that need special attention were identified:

1. *Storage stability of modified asphalts:* Because most modified asphalts are multi-phase systems, possible phase separation and possible continued reaction under static or agitation conditions should be evaluated.
2. *Shear-rate dependency of viscosity:* Most modified binders are non-Newtonian fluids at temperatures in the range of mixing and compaction currently used in the field. The effect of shear rate on binder workability could be very important in selecting proper mixing and compaction temperatures.
3. *Strain dependency of rheological response:* Many modified binders show highly nonlinear behavior at strains and stresses that are well within the pavement application conditions. Therefore, relying on linear viscoelastic properties to estimate binder contribution to pavement performance could be misleading.
4. *Effect of mechanical working:* Asphalt binders in general are known to accumulate damage when subjected to repeated loading. Relying on rheological properties measured after few cycles of loading does not allow an evaluation of damage accumulation. Modifiers are used as reinforcements in asphalt binders. It is more important to evaluate their effectiveness in altering damage behavior over load repetition than their initial behavior.
5. *Loading-rate dependency and time–temperature equivalency:* The Superpave binder specification is founded on the assumption that all binders have similar sensitivity to loading rate and temperature. Grade shifting is used to account for traffic speed. Modified binders could vary significantly in their sensitivity to loading rates because of variation in basic microstructure. To estimate the contribution of binders to pavement performance under different traffic conditions, a direct measure of loading-rate dependency is necessary.

TABLE 2.1 Generic types of asphalt modifiers currently used for paving applications

Modifier Type	Class	Effect on Distress				
		PD[a]	FC[b]	LTC[c]	MD[d]	AG[e]
Fillers	Carbon black	x				x
	Mineral: Hydrated lime	x				x
	Fly ash	x				
	Portland cement	x				
	Baghouse fines	x				
Extenders	Sulphur	x	x	x		
	Wood lignin				x	
Polymers - Elastomers	Styrene butadiene di-block SB	x		x	x	
	Styrene butadiene triblock/radial block (SBS)	x	x	x		
	Styrene isoprene (SIS)	x				
	Styrene ethylbutylene (SEBS)					
	Styrene butadiene rubber latex SBR	x		x		
	Polychloroprene latex	x	x			
	Natural rubber	x				
	Acrylonite butadiene styrene (ABS)	x				
Polymers - Plastomers	Ethylene vinyl acetate (EVA)	x	x			
	Ethylene propylene diene monomer (EDPM)	x				
	Ethylene acrylate (EA)	x				
	Polyisobutylene	x				
	Polyethylene (low density and high density)	x		x		
	Polypropylene	x				
Crumb rubber	Different sizes, treatments, and processes	x	x	x		
Oxidants	Manganese compounds	x				
Hydrocarbons	Aromatics			x		
	Napthenics					
	Paraffinics/wax			x		
	Vacuum gas oil			x		
	Asphaltenes: ROSE process resins	x				
	SDA asphaltenes	x				
	Asphaltenes: DEMEX asphaltenes	x				
	Shale oil				x	x
	Tall oil					
	Natural asphalts: Trinidad	x	x	x	x	
	Gilsonite	x			x	
Antistrips	Amines: Amidoamines				x	
	Polyamines				x	
	Polyamides				x	
	Hydrated lime				x	
	Organo-metallics				x	
Process-based	Air blowing					
	Steam distillation					
	Propane de-asphalted (PPA)					
Fibers	Polypropylene	x	x	x		
	Polyester	x		x		
	Fiberglass					
	Steel	x	x	x		
	Reinforcement	x	x	x		
	Natural: Cellulose	x				
	Mineral	x				
Antioxidants	Carbamates: Lead			x		x
	Zinc			x		x
	Carbon black	x				x
	Calcium salts					x
	Hydrated lime				x	x
	Phenols					x
	Amines				x	x

[a] Permanent deformation	[d] Moisture damage
[b] Fatigue cracking	[e] Oxidative aging
[c] Low-temperature cracking	

2.3 SUMMARY OF FINDINGS FROM THE SURVEYS

Three types of questionnaires were used to collect information from state and provincial highway agencies, suppliers and contractors, and academia. A copy of each survey is attached in Appendix D. Detailed summaries of the questionnaires were published in the first interim report. The following findings are based on these summaries.

2.3.1 State/Provincial Agency Surveys

1. The majority of state and provincial agencies (32 out of 52) have specifications for modified asphalt binders. A recent report by Koch Materials showed that 28 of the 50 states have adopted specifications or provisions for modified binders (*29*). These specifications are based on a variety of test methods not necessarily related to Superpave.
2. The survey indicates that the majority of states and provinces (33 out of 52) are currently using the Superpave binder specifications routinely or experimentally.
3. The survey indicates that the majority of states and provinces (31 out of 52) are currently using the Superpave mixture specifications routinely or experimentally.
4. Eight agencies answered "No" to all three sections of question No. 1, indicating that they do not have provisions for modified binders and that they have not started using Superpave binder or mixture specifications. These agencies are Nebraska; North Dakota; Oregon; Puerto Rico; Rhode Island; Saskatchewan; Washington, D.C.; and West Virginia.
5. For binders, the areas that agencies reported as the most problematic include
   –Compatibility/separation (12 out of 35)
   –Short-term aging (8 out of 35)
   –Particle size (4 out of 35)
   –Long-term aging (PAV) (3 out of 35)
   Other areas either were not identified as a problem or were identified by no more than three agencies.
6. For mixtures, the areas that agencies reported as the most problematic are
   –Mixture/compaction temperature (15 out of 31)
   –Short-term aging (8 out of 31)
   –Compatibility/separation (5 out of 31)
   –Long-term aging (PAV) (3 out of 31)
   Other areas were not identified or were identified as a problem by no more than three agencies.
7. Table 2.2 lists the most frequently used types of asphalt modifiers as reported by state agencies.
   Other modifiers were identified by two or fewer agencies or were not identified at all.
8. Regarding future usage of modified asphalt binders, the survey results indicate the following:
   –*No* agency planned on using less,
   –*12* are going to use about the same, and
   –*35* agencies plan on using more.

TABLE 2.2 The types of asphalt modifiers most frequently used by state agencies

Type	Class	No. of Agencies	Target Distress/Property				
			PD^1	FC^2	LTC^3	MD^4	AR^5
Polymer - Elastomer	Styrene Butadiene Styrene (SBS)	28	18	8	10	3	6
	Styrene Butadiene SB	16	13	5	5	0	2
	Styrene Butadiene Rubber Latex SBR	17	10	4	4	1	2
	Tire Rubber	3	1		1		
Polymer - Plastomer	Ethyl Vinyl Acetate (EVA)	6	3				1
Anti-Stripping Agents	Fatty Amidoamines	8				4	
	Polyamines	6				4	
	Hydrated Lime	4				3	
	Others	7				3	
Hydrocarbons	Natural Asphalts	6	5				
Fibers	Cellulose	12	3		1		1
	Polypropylene	7	4		1		
	Polyester	6	4		1		1
	Mineral	3	1	1	1		
Processed-Based	Air Blowing	4	2				
Mineral Fillers	Lime	4				1	
Anti-Oxidants	Hydrated Lime	7				4	
Extenders	Sulphur	4					

^{1} Permanent deformation ^{2} Fatigue cracking ^{3} Low-temperature cracking ^{4} Moisture damage ^{5} Aging resistance

9. The justification for the use of modifiers was as follows:
   - –Rutting resistance = 39,
   - –Low-temperature cracking = 28,
   - –Fatigue cracking = 21,
   - –Aging = 14, and
   - –Moisture damage = 14.

2.3.2 Supplier and Contractor Surveys

1. The survey results reveal that a significant percentage of suppliers and contractors (29 out of 41 respondents) are currently using Superpave binder testing procedures.
2. The survey results show that most suppliers and contractors (31 out of 40) do not use Superpave mixture testing procedures.
3. Ten of the 41 supplier and contractor respondents indicated that their company uses neither Superpave binder nor mixture-testing methods.
4. For binders, the survey results indicate that the following are the areas in which suppliers and contractors have encountered problems:
   - –Short-term aging (12 out of 29)
   - –Compatibility/separation (10 out of 29)
   - –Long-term aging (10 out of 29)
   - –Time–temperature shift factors (10 out of 29)
   - –Strain dependency (linear) (9 out of 29)
   - –Solubility (6 out of 29)

   The other areas either were not identified or were identified as a problem by three or fewer suppliers or contractors.
5. For mixtures, the following areas were identified by suppliers and contractors as the most problematic:
   - –Mixing/compaction temperatures (5 out of 9)
   - –Short-term aging (3 out of 9)
   - –Compatibility/separation (3 out of 9)

   The other areas either were not identified or were identified as a problem by two or fewer suppliers or contractors.
6. Table 2.3 lists the most frequently used types of asphalt modifiers and the distresses they target. The other modifiers were identified by two or fewer agencies or were not identified at all.
7. Table 2.4 indicates the methods of incorporating the modifiers into the asphalt used or recommended by the suppliers and contractors.

2.3.3 Academia Surveys

1. The survey results indicate that approximately 50 percent of academic institutions surveyed (8 out of 14 respondents) are currently using Superpave binder testing procedures.
2. The survey results indicate that nearly 60 percent of academic (9 out of 14 respondents) are currently using Superpave mixture-testing procedures.
3. Six out of the 21 respondents indicated they use neither Superpave binder nor mixture-testing methods.
4. For binders, the areas that academic representatives reported as the most problematic are
   - –Short-term aging (4 out of 8)
   - –Compatibility/separation (3 out of 8)

TABLE 2.3 The most frequently used types of asphalt modifiers and the distresses they target

Type	Class	No. of Users	Target Distress/Property				
			PD[1]	FC[2]	LTC[3]	MD[4]	AR[5]
Polymer - Elastomer	Styrene Butadiene Styrene (SBS)	25	23	21	20	5	10
	Styrene Butadiene Rubber Latex SBR	18	14	11	14	2	8
	Styrene Butadiene SB	9	8	7	7	2	3
	Polychloroprene Latex	4	3	1	2		1
Polymer - Plastomer	Low-Density Polyethylene (LDPE)	5	4	2		1	
	Ethyl Vinyl Acetate (EVA)	5	5	3	2	1	2
	Polypropylene	3	2	2			1
Anti-Stripping Agents	Polyamines	12		2	2	10	
	Fatty Amidoamines	9				8	1
Hydrocarbons	Aromatics	7	1	2	3		2
	Napthenics	6			3		3
	Vacuum Gas Oil	5	1	1	4		
Fibers	Cellulose	5	3	3	3	2	2
	Polyester	3	2	3	2		1
Processed-Based	Air Blowing	4	4	1	2	2	1
	Propane De-asphalted	4	1				
Mineral Fillers	Hydrated Lime	3	1			2	
Extenders	Sulpher	3	3	1			

[1] Permanent deformation [2] Fatigue cracking [3] Low-temperature cracking [4] Moisture damage [5] Aging resistance

TABLE 2.4 The methods of incorporating the modifiers into the asphalt used or recommended by the suppliers and/or contractors

Type of Modifier	Suppliers/Contractors Responses to Method of Incorporation				
	Refinery	Terminal	Broker/ Blender	Mix Plant (Drum/Batch)	In-line Blending
Polymers – Elastomers	15	11	4	2	3
Polymers – Plastomers	8	4	3	2	
Polymers – Unspec.	2	1			
Antistripping Agents	6	2		2	2
Hydrocarbons	1	1	1		2
Fibers			1	4	2
Process-based	1				1
Mineral Fillers			1	1	1
Oxidants					1
Extenders				1	

–Long-term aging (3 out of 8)
–Test geometry (3 out of 8)
–Particle size (2 out of 8)

The other areas were not identified or were identified as a problem by only one respondent.

5. For mixtures, the areas reported as the most problematic are:
   –Mixing/compaction temperatures (4 out of 9)
   –Short-term aging (1 out of 9)
   –Compatibility/separation (1 out of 9)
   The other areas were not identified.

2.4 CLASSIFICATION OF MODIFIED BINDERS

The Superpave binder test protocols are based on two main assumptions:

1. Binder behavior is independent of film thickness and sample geometry; and
2. The binder is evaluated based on the properties within the linear viscoelastic range in which its behavior is independent of the strain or stress level.

The essence of these two assumptions is that the asphalt binder is a simple system that can be characterized using linear viscoelasticity and simple geometry within which stress and strain fields are simple to calculate. To apply the current Superpave binder protocols for modified binders, both of these conditions have to be satisfied. In other words, the modified binders must be rheologically "simple."

Modified binders not classified as simple are termed "complex" according to the following definitions:

- *Simple binders:* Asphalt binders with rheologically simple behavior that do not violate the assumptions upon which the PG system is based; these assumptions include independence of strain, non-thixotropy, isotropy, and independence of sample geometry.
- *Complex binders:* Asphalt binders that cannot be classified as simple binders because their behavior violates one or more of the PG-system assumptions.

It was hypothesized here that (1) simple binders can be evaluated using the existing Superpave binder protocols and can be classified as PG binders and (2) complex binders should be evaluated using Superpave mixture protocols. Although binder protocols can still be used to characterize certain aspects of the behavior of complex binders, they should not be a performance grade system without evaluation of mixture data.

The simple–complex classification was also considered as a replacement for the modified–unmodified classification. Based on the understanding of the existing literature, it was difficult to derive a definite set of criteria for classifying binders as modified or unmodified. Many modification processes involve no additives or liquid additives that are very difficult to detect in a binder. Using the performance-grade spread (the rule of a 90°C or larger difference between the high and low grades) is not adequate; production of binders with a PG spread of less than 90°C using modification with refinery by-products is starting to emerge in the markets.

The simple–complex classification tends to address the challenge of a "blind" specification directly. Because a reasonable specification could be used to blindly cover all "simple" binders, users do not need to know the composition of these simple binders because the performance-grading system would provide adequate characterization. This concept is an extension of the concept that users do not need to know the crude source used in the production of conventional asphalts. It could also be true that users need to know neither the process nor the type of additives used in modifying asphalts. What they need to know, however, is whether the testing protocols used to derive the performance grade of the binder are applicable. The only reason why such protocols would not apply is if the binder is too complex to be tested within the limited geometry and test conditions.

This concept was used to design several experiments, and extensive data were collected for simple and complex binders with the objective of defining specific criteria to differentiate simple from complex binders. The following criteria were initially defined to classify a binder as simple binder:

1. Contains no more than 2 percent by volume of additives retained on No. 200 sieve using the particulate additive test (PAT) with toluene.
2. Is stable during long-term (2 days or longer) storage at typical HMA mixing and compaction temperatures. The laboratory asphalt stability test (LAST) is used to measure stability.
3. Is not excessively sensitive to mixture volumetrics, or pavement structure, or both, as simulated by change in stress or strain ranges during rheological testing. Such sensitivity is commonly termed “nonlinearity” and is highly dependent on the overall structural capacity of pavement structure. It is measured by conducting a stress or a strain sweep using the dynamic shear rheometer (DSR).
4. Is not excessively sensitive to repeated or cyclic loading that simulates traffic volume. Such sensitivity is commonly called “cyclic” or “creep fatigue.” It is measured by conducting a time sweep using the DSR.

A binder is classified as a complex binder if it violates any of the above criteria. As more data were collected in the project, it became apparent that, although Criteria 1 and 2 could be easily implemented, it was very difficult to define what is excessive sensitivity to pavement structure or to traffic volume. It was also apparent that binders that contain particulate additives or that are not stable during storage are too risky to use and thus should not be included in any binder classification system.

The focus of the research then shifted from the classification of binders to screening binders for particulate additives and for storage stability to disqualify such “risky” binders. As to the sensitivity to pavement structure or traffic volume, the focus was shifted to integrating these important application conditions into the specification testing so that they are directly taken into consideration along with pavement temperature and traffic speed.

In summary, the classification of binders as simple or complex was found to be unnecessary. All binders should be screened for particulate additives and for storage stability to qualify them for inclusion in a performance-grading system. An enhanced grading system should consider the following application conditions:

- Pavement temperature,
- Traffic speed,
- Traffic volume, and
- Pavement structure.

Many experts consulted during the project indicated that, regardless of the terminology used, differentiating between binders with additives and without additives is necessary to protect against misusing a blind specification system. To achieve this requirement, the PAT includes two solvents: n-octane and toluene. All additives separate in n-octane; this indicates whether an additive is present in the binder. Toluene separates only additives that are not likely to be soluble in asphalt; this indicates the relative compatibility of the additive with the base asphalt.

With the application of PAT, the specification can be applied blindly to binders with no particulate additives, which are stable during storage, by including sensitivity to pavement structure and traffic volume into the testing protocol.

2.5 CHARACTERISTICS OF MODIFIED BINDERS

In Phase II of the project, extensive rheological and failure testing was conducted on a large number of modified binders with the Superpave binder protocols. To address the deficiencies in the current testing protocols, methods for determining the type and amount of additive (the PAT) and for determining the storage stability (the LAST) were developed. In addition, a modification of the current rolling thin film oven test (AASHTO T240) was developed. The results were summarized in the second interim report of the project (*30*); the main findings are discussed in the following subsection.

2.5.1 PAT

One of the alternatives to using microscopy to determine the nature of the asphalt additives is separation of the additive from asphalt. With separation, the general type of the additive and its characteristics can be determined. In a PAT, a diluted solution of the asphalt binder is passed through a sieve to separate particulate additives from the base asphalt. Particulate additives can result in potential separation or in interference with test sample geometry. In the current standard Superpave binder test methods, the particulate size is limited to 250 μm, selected arbitrarily as one-fourth (¼) of the minimum testing sample dimension. The PAT separates material larger than 75 μm using a No. 200 (0.075-mm) mesh. This size was selected because larger-size particulates are commonly considered part of the mineral aggregates in the asphalt mixture.

The schematic of the test set up is shown in Figure 2.1. In the test, the asphalt binder is heated to 135°C until it becomes soft enough to pour. Approximately 10 ml of sample is transferred into a 125-ml Erlenmeyer flask. While hot, the sample is diluted using 100 ml of solvent in small portions with continuous agitation until all lumps disappear and no undissolved sample adheres to the container. A metal, 50-mm diameter, No. 200 sieve disk is placed in the vacuum filtering apparatus, and the vacuum filtration is started. The container is washed

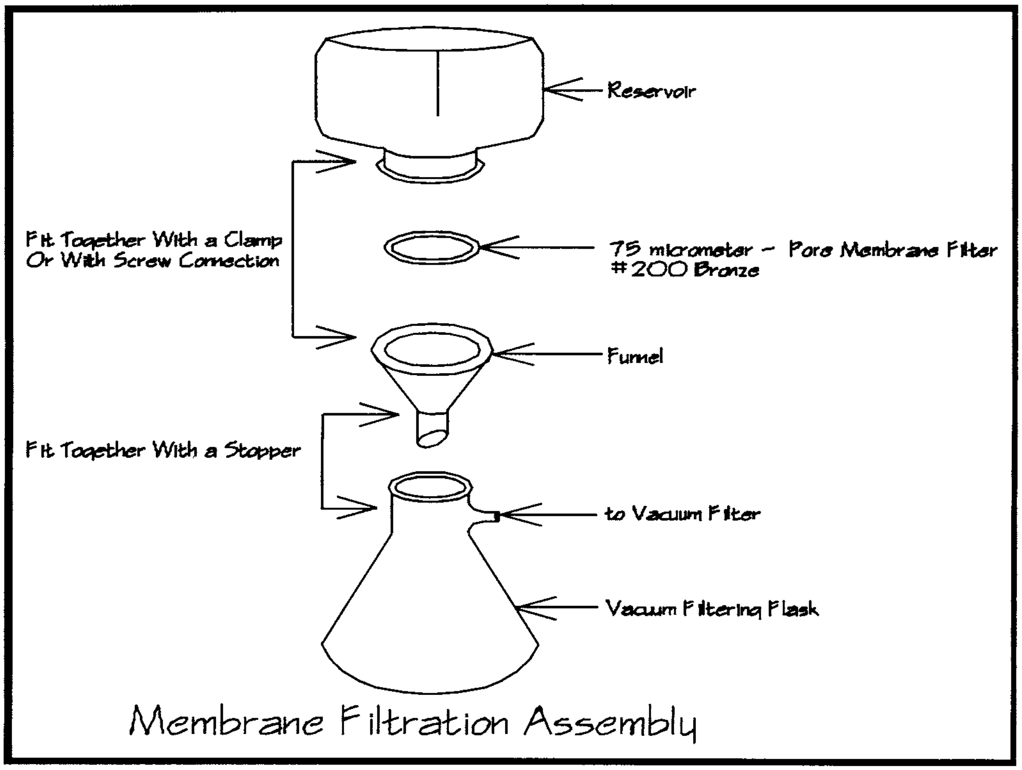


Figure 2.1 Schematic of the PAT.

with small amounts of solvent to facilitate filtering. Filtration is continued until the filtrate is substantially colorless, then suction is applied to remove the remaining distillate. The material retained on the filtering sieve is transferred to a centrifuge tube, and the volume is measured partially filled with the solvent. The tube is placed in a centrifuge apparatus for 30 min at approximately 3,000 rotations per minute (rpm). At the end of the centrifugation, the volume of material at the bottom of the tube is measured to the nearest 0.01 ml. Using the final volume of particulate and the initial volume of sample, the percentage of compacted volume of the particulates retained on a No. 200 sieve by volume of the asphalt is calculated. The conditions used for the protocol were selected based on several experiments.

Based on trial testing using various solvents, n-octane and toluene were selected for PAT. n-Octane is considered as an alternative because of its zero aromaticity, its suitability for dissolving asphalt, and its relatively high boiling point. The tests conducted have shown that n-octane can separate polymers that are not particulate in nature and so give an indication of the use of additive in a binder. Toluene is used because its solubility parameter is considered very similar to that of asphalt. The tests conducted showed it is a good solvent that can dissolve most additives that are believed to be compatible with asphalt.

Table 2.5 gives results collected using the PAT for a number of PG 82, PG 76, PG 70, PG 64, and PG 58 grades modified with different additives. The results are shown for one or more of the solvents used. The results reveal that n-octane can be used to detect the existence of any additive in the asphalts. The material retained on the sieve after treatment with n-octane can be further treated with toluene to determine its solubility.

2.5.2 LAST

The general requirements for a new test to evaluate the storage stability of modified asphalts were selected based on the review of research done in the past (*17,24,31–43*) and on an evaluation of typical storage tanks and conditions used to store such asphalts in the field. From previous research, it was clear that a new test should allow for an evaluation of the following factors:

- Effect of extended storage at HTs in the range of 160 to 180°C,
- Effect of mechanical agitation of the modified binders, and
- Performance-related properties measured at multiple temperatures and frequencies.

From the review of field practices and of the design of storage tanks typically used in the field, it was concluded that, in

TABLE 2.5 Results of testing selected modified asphalts using the PAT

		ORGANIC SOLVENT			
		Pet. Distillate	**N-Heptane**	**N-Octane**	**Toluene**
LABORATORY HAND-BLENDS		**VOL. FRACTION PARTICULATES > 0.075 mm**			
Binder	Filler / Polymer				
PG 76-22	Hydrated Lime (16% HL)	7.5	18	13.0	7.0
PG 58-34	SBS	2.1	16	13.0	0.0
PG 76-22	EVA	6	13	2.5	0.0
PG 58-28	Crumb Rubber (12% CR)	15	15	14.0	14.0
PG 70-22	Silica Quartz (10% SQ)	1.5	1	0.8	0.8
PG 82-XX	Gilsonite (8% Gil.)	1	2	1.5	1.0
PG 70-22	PG 70 Neat	0	0	0.0	0.0
MODIFIED BOSCAN ASPHALT					
Binder	Filler /Polymer				
PG 82-XX	PE Unstabilized	-	-	17.0	2.5
PG 58-40	PE Unstabilized	-	-	6.0	0.0
PG 64-34	Amidoamines	-	-	1.0	0.0
PG 64-34	Polyamides	-	-	0.9	0.0
PG 64-34	SBS Linear	-	-	25.0	0.0
PG 82-XX	SBR LMW	-	-	0.5	0.0
PG 82-XX	SBR HMW	-	-	3.0	1.0
PG 82-XX	Ethylene Terpoly	-	-	1.5	0.0
PG 82-XX	SBS Linear	-	-	19.9	0.0
PG 58-40	SBS Linear	-	-	16.0	0.0
PG 82-XX	SBS Radial	-	-	22.0	2.0
PG 70-34	SBS Radial	-	-	15.0	0.0
PG 58-40	SBS Radial	-	-	3.5	0.0
PG 52-40	SBS Radial	-	-	0.0	0.0
PG 82-XX	Steam Distilled	-	-	1.0	0.0
PG 82-XX	SB Di-block	-	-	18.0	0.0
PG 70-34	SB Di-block	-	-	15.0	0.0
PG 58-40	SB Di-block	-	-	3.0	0.0
PG 82-XX	Oxidized	-	-	2.5	0.0
PG 64-34	Oxidized	-	-	0.5	0.0
PG 58-40	Oxidized	-	-	0.0	0.0
PG 58-40	Ethylene Terpoly	-	-	0.0	0.0
PG 82-XX	PE Stabilized	-	-	0.0	0.0
PG 82-22	PE Stabilized	-	-	0.0	0.0
PG 58-40	Oxidized	-	-	0.0	0.0
PG 76-XX	SBS Linear	-	-	12.0	0.0
PG 58-40	SBS Linear	-	-	0.0	0.0
PG 64-34	Oxidized (BB)	-	-	2.5	0.0
PG 58-40	Oxidized (BB)	-	-	0.0	0.0
PG 58-34	SBR LMW	-	-	0.0	0.0
PG 58-34	SBR HMW	-	-	0.0	0.0
PG 82-10	Fibers (3%)	-	-	8.0	9.0
PG 58-XX	Fibers (2%)	-	-	6.0	7.5
PG 58-28	Neat West Texas	-	-	0.0	0.0
PG 64-22	Neat West Texas	-	-	0.0	0.0
PG 70-22	SB Di-block	-	-	0.0	0.0
PG 58-28	SBR HMW	-	-	0.0	0.0
PG 76-22	PE Stablized	-	-	0.0	0.0

almost all cases, asphalts are stored with some sort of continuous agitation to maintain a uniform temperature and the homogeneity of the material. Thermal history should not include a freezing step because it does not simulate field conditions. Two basic designs of tanks (i.e., horizontal and vertical) are used in the field. The vertical tanks that have double propellers are recommended to induce enough agitation to maintain uniformity of product. The general consensus is that horizontal tanks are less efficient in mixing than are the more recently designed vertical tanks.

Using the information gathered, it was decided to scale down a typical design of a vertical storage tank as manufactured by one of the principal U.S. suppliers. The selected design is shown in Figure 2.2. It features an internal heating element controlled by an electronic temperature-control feedback system to maintain isothermal conditions and a constant speed, double-propeller agitator centered in the middle of the cylindrical container. The dimensions are such that a sample of 400 ml is used and the sampling is done periodically using a pipette from the top and bottom of the container without stopping the conditioning. This device is called the LAST device. The storage stability of asphalt binders is evaluated using two ratios. A separation ratio, R_s, is calculated by dividing the response (G^*, δ, or a combination of parameters) of a sample taken from the upper part of the container by the response of another sample taken from the lower part of the container. A degradation ratio, R_d, is calculated by comparing the average of top and bottom responses at a given sampling time with the initial response. The ratios are measured at selected intervals. Detailed protocol proposed for LAST is written in Appendix C II of this report.

To evaluate the LAST design, experiments were performed to study the effects of several controlled factors:

- The source of heat: internal and external.
- The time of conditioning: 1, 3, 6, 12, 24, and 48 h.
- Agitation: none and slow.
- Testing frequency: 0.15 to 30 Hz.

Table 2.6 gives results collected using the LAST for a number of modified asphalts prepared from different base asphalts (B = Boscan; T = West Texas) and different additives. The results are shown for external heat with and without agitation. The values of the separation ratio (i.e., R_s) and the degradation ratio (i.e., R_d) are observed to vary by modifier type, base asphalt, rheological testing temperature, and rheological parameter measured. The following points summarize the findings:

- It does not appear that any one generic type of modifier as a whole, such as plastomers or elastomers, shows more potential for separation than others. The separation potential varies from modifier to modifier within the same generic type. Also, the level of separation does not appear to be related to the grade of the modified binder. These findings confirm that the separation potential is dependent on the specific nature of the modifier and cannot be predicted from information about the grade or generic type of modifier. The findings show that it may be necessary to measure the separation potential and to account for such phenomenon in selecting and using modified binders.

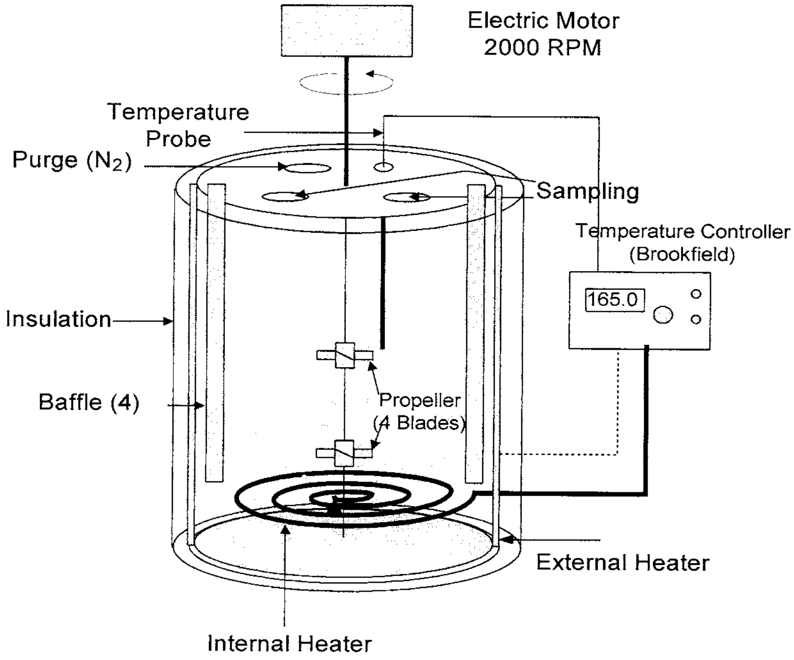


Figure 2.2 Schematic of the LAST apparatus.

TABLE 2.6 LAST results for modified asphalt *G**

Modifier	DSR Test	External with Agitation		External without Agitation	
		Rs	Rd	Rs	Rd
		(1.5Hz)	(1.5Hz)	(1.5Hz)	(1.5Hz)
PG 58-28 (B)	HT	-3.1	11.8	-1.5	3.2
Neat	6 C	-1.0	11.8	3.7	2.1
PG 82-22 (B)	HT	-0.3	19.0	-1.5	-2.7
SBS Radial	6 C	12.1	11.8	3.7	5.3
PG 82-22 (B)	HT	9.1	12.9	-11.1	8.4
PE stabilized	6 C	-4.8	9.3	1.4	-1.8
PG 76-22 (B)	HT	2.4	15.8	256.0	113.8
PE unstabilized	6 C	10.6	3.4	-2.2	30.2
PG 70-34 (B)	HT	-4.5	-13.6	31.4	12.0
SB Di-block	6 C	4.9	7.3	-55.0	-16.8
PG 58-34 (B)	HT	1.3	18.6	50.6	21.9
SBR LMW	6 C	6.9	19.5	-89.8	-12.4
PG 58-34 (B)	HT	-17.2	15.8	169.4	42.1
SBR HMW	6 C	-1.2	16.8	-68.6	0.3
PG 76-22 (T)	HT	-5.9	14.9	17.0	-4.5
EMA	6 C	-2.3	-6.7	-28.5	8.0
PG 76-22 (T) (B)	HT	-3.3	0.6	17.0	18.7
PE stabilized	6 C	0.9	-2.8	0.9	-1.6
PG 70-34 (T)	HT	-2.4	-0.7	4.0	1.1
SBS Di-block	6 C	2.8	6.0	0.5	5.8
PG 70-34 (T)	HT	-0.5	5.9	4.0	6.3
Ethylene Terpoly	6 C	0.7	4.3	-19.8	10.6
PG 64-34 (T)	HT	10.8	3.9	18.6	12.1
SBS Linear	6 C	-3.0	-2.0	-20.3	4.3
PG 58-34 (T)	HT	5.0	-31.1	-5.4	-6.2
SBR LMW	6 C	19.1	-12.2	-17.0	10.8
PG 58-34 (T)	HT	-10.3	-18.6	2.9	2.9
SBR HMW	6 C	-1.9	-0.8	-8.4	0.3

- There is significant evidence that agitation can prevent separation very successfully. For some binders (e.g., PE unstabilized), agitation keeps the binder from showing separation for extended periods of time. It is therefore necessary to acknowledge that separation under static conditions as presented in Table 2.6 should not be used as the basis for rejection of a modified binder. It is important to recognize that most modified binders are two-phase systems that are designed to stay as two-phase systems.
- Without agitation there appear to be only marginal effects of degradation for most binders. However, different trends are found when high-speed (2,000-rpm) agitation is used to simulate extreme agitation conditions. Several of the modified binders show a significant increase in *G** values and a few show reductions in the *G** values. This indicates that high-speed agitation can either cause more reaction or degrade the additive.
- The effect of agitation is significantly different when evaluated at HT compared with 6°C. It is clear that thermal agitation at HTs can result in changing the rheological type of a binder and thus needs to be carefully measured.

2.5.3 Modification to the Rolling Thin Film Oven Test

One of the main problems with the rolling thin film oven (RTFO) procedure for modified binders is that these asphalts, because of their high viscosity, will not roll inside the glass bottles during the test. Also, asphalts are capable of creeping out of the bottles during the test. To solve this problem, two modifications were considered: (1) using a number of steel spheres to create shearing forces to force the spreading of thin films and (2) using a steel rod to induce the same action. Initial evaluations indicated that the steel rod is more practical, simpler to use, and easier to clean. Several length and diameter combinations were tried with asphalts that varied in their viscosity at the testing temperature of 163°C between 0.5 and

3.0 Pa · s. The optimum conditions were achieved using steel rods that were 127 mm long by 6.35 mm in diameter.

The criteria for accepting the use of the steel rods in the RTFO included two items. First, the rods should not have major effects on aging of neat asphalts; otherwise, the relation between the test results and the field aging, which is accepted for the standard test without the rods, can be questioned. Second, the aging with the rods should not interfere with the stiffening effect of inert additives.

To evaluate the first criterion, neat asphalts were aged with and without rods, and the changes caused by aging were evaluated using the DSR. To evaluate the second criterion, Ottawa sand (considered to be an inert filler) was mixed with asphalt and aged in the RTFO with rods. The same amount of the sand was mixed after aging the neat asphalt in the RTFO with rods. The stiffening effect of the sand was evaluated with premixing and with post-mixing to determine the interference of the RTFO aging.

Typical results from both experiments are listed in Table 2.7. It can be seen that the effect on the aging of the neat asphalt is minimal. The effect on the filled asphalts, however, is significant and consistent. The stiffening of the asphalts by the filler (i.e., OS) appears to be the same with premixing and post-mixing when the rods are used, which supports the hypothesis that using the rods does not interfere with stiffening by the filler.

Several modified asphalts were tested in the modified RTFO. The results indicate that steel rods in the RTFO test can be used to alleviate the problem reported for aging modified binders in RTFO bottles. For all the tests conducted, it was observed that the rods inside of the bottles uniformly spread the asphalt binder. Creeping of material outside the bottles was observed for few of the highly modified asphalts. The problem was solved by tilting the oven slightly (2 degrees) to keep these asphalts from rolling out of the bottle.

2.5.4 Effect of Traffic Speed (Frequency Dependency)

To evaluate the procedure of changing grades to account for traffic speed, frequency sweep data were used to calculate the critical temperatures (i.e., the temperatures at which the specification requirement, e.g., $G^*\sin\delta = 5000$ kPa, is satisfied). The critical temperatures for the unaged conditions, RTFO-aged condition, and the PAV-aged condition are shown in Figure 2.3. The figure shows the change in critical temperature as a result of changing the loading rate from 1.5 to 0.15 Hz. The 1.5 Hz is assumed to represent normal traffic (50 to 60 mph), while the 0.15 Hz represents slow (e.g., urban) traffic speed (5 to 10 mph). The figure also shows the effect of changing the loading time on the critical temperatures calculated using the criteria $S(t) = 300$ MPa and $m(t) = 0.300$.

The following points summarize the trends observed:

- The effect of reducing or increasing frequency by one order of magnitude at HT is approximately 20°C. This effect is very significant and indicates that the grade must be shifted by at least 3°C at high pavement temperatures in order to represent the effect of reducing speed from 50 mph to 5 mph shifting.
- The change in critical temperature caused by loading rate is binder specific. It is therefore believed that selecting a single standard shift in temperature to represent the effect of speed is not appropriate. The standard deviations are in the range of 3°C and the range in ΔT is between 15 and 24°C, which is sufficiently high to make using one standard shift difficult.
- At the IT, the effect of changing the frequency is much lower than the effect calculated at HTs. The average effect is 7 to 10°C compared with the 20°C effect mentioned

TABLE 2.7 The effect of metal rods in the RTFO on neat and modified asphalts

	G*/sinδ RTFO Test w/ Rods	G*/sinδ RTFO Test w/o Rods	Diff. %
PG58-28 (Neat)	2784	2872	3.06
PG64-22 (Neat)	3711	3741	0.80
PG70-22 (Neat)	3326	3200	3.94
Pre-mixing			
PG70 + 15% OS	4133	3270	
	4173 (2^{nd} run)	3363 (2^{nd} run)	
PG64 + 20% SQ	4508	3578	25.9
	4617 (2^{nd} run)	3590 (2^{nd} run)	28.6
PG58 + 15% CR	3494	3081	13.5
	3555 (2^{nd} run)	2961(2^{nd} run)	20.1
Post-mixing			
PG70 +15% OS	4107	3823	7.4
	4142 (2^{nd} run)	3856 (2^{nd} run)	7.4

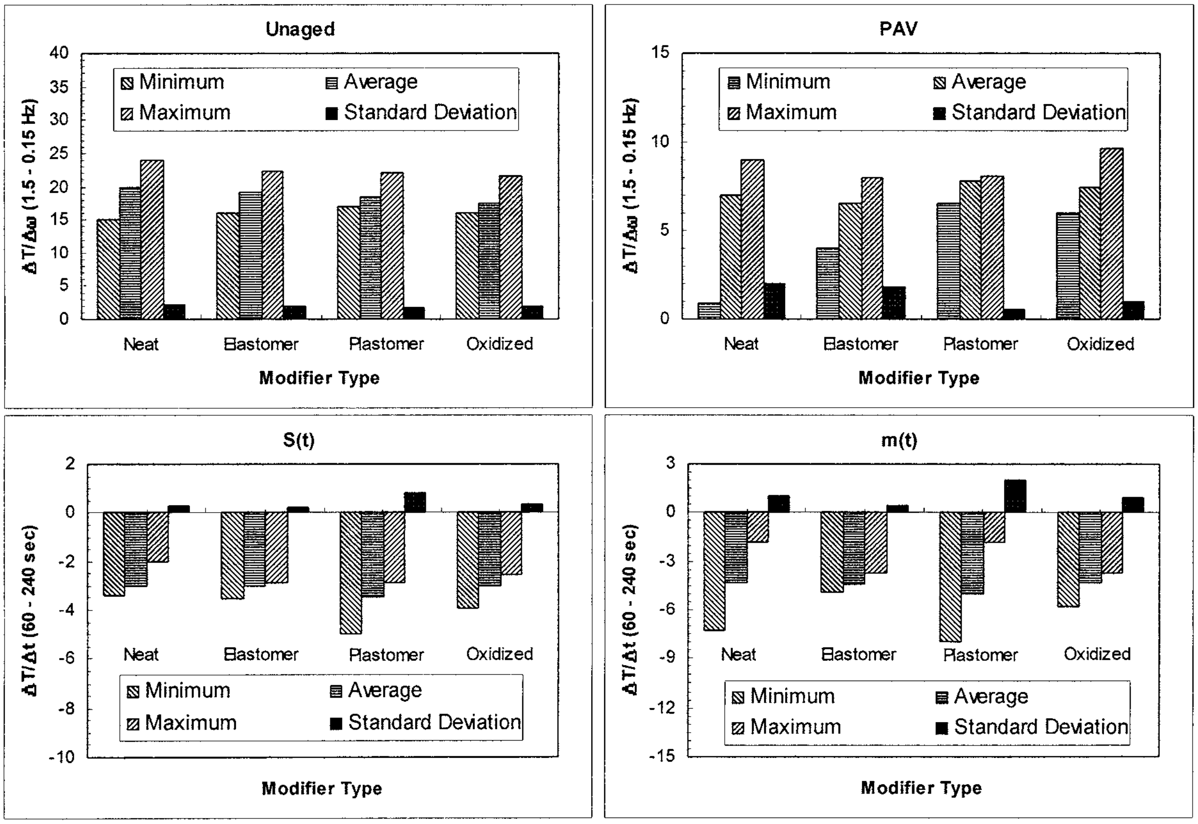


Figure 2.3 Changes of critical temperatures for G*/sinδ *(unaged),* G*sinδ *(PAV),* S(t), *and* m(t) *as a result of reducing testing frequency from 1.5 Hz to 0.15 Hz or changing creep loading time from 8 to 240 s for different modifiers.*

above for HT. It is also clear that the effect is highly variable and cannot be assumed similar for all binders.

- At LT, the effect of changing the loading time from 60 to 240 s is also significant. This effect represents the effect of changes in cooling cycles. Similar to the HT and IT results, the change in the critical temperature is not identical for all binders.
- For most binders, the effect on the critical temperature of the *m*-value is more significant than the effect on the critical temperature of the stiffness.

The results of the changes in critical temperatures caused by changes in frequency or loading time show that modified binders can vary significantly in their dependency on the rate of loading. Determination of a direct measure of the effect of traffic speed on the grading of the binders by conducting the testing at the specific simulated traffic speed is recommended.

2.5.5 Effect of Traffic Volume (Thixotropy/Fatigue)

It is important to prove the hypothesis that repeated loading is a factor to which modified asphalt responds differently. This is important because traffic application is cyclic in nature and because the morphology of modified asphalts can indeed play an important role in resisting fatigue and in showing a stable, non-thixotropic response to the traffic cyclic loading.

To test the hypothesis, the effect of cyclic loading on each of the binders was measured at the HT and the IT. For example, a PG 76 + 25–22 was tested at HT = 76°C and at IT = 25°C. Selected binders were tested for 5,000 to 10,000 cycles. Figure 2.4 summarizes the results using the ratio of *G** at 50 cycles to the *G** at 5,000 cycles. As shown in the figure, fatigue cycles at selected strain levels have a highly significant effect on *G**. For some modified binders, ratios of 5 to 7 were observed.

The effects appeared to be highly dependent on the composition of the binder. They were also observed to be highly dependent on the strain or stress level used in the testing. Because of the importance of the results, several base asphalts used in the experiment were included in the testing. The results as shown in Figure 2.4 clearly differentiated between base asphalts and modified asphalts. They also showed the variation in *G** ratios based on the generic modifier type. It can be seen that, for some neat asphalts, the ratio of *G** at 50 to *G** at 5,000 is as high as 23 compared with a maximum of 7 for the modified binders. It appears that modification in general significantly increased resistance to the *G** decay with

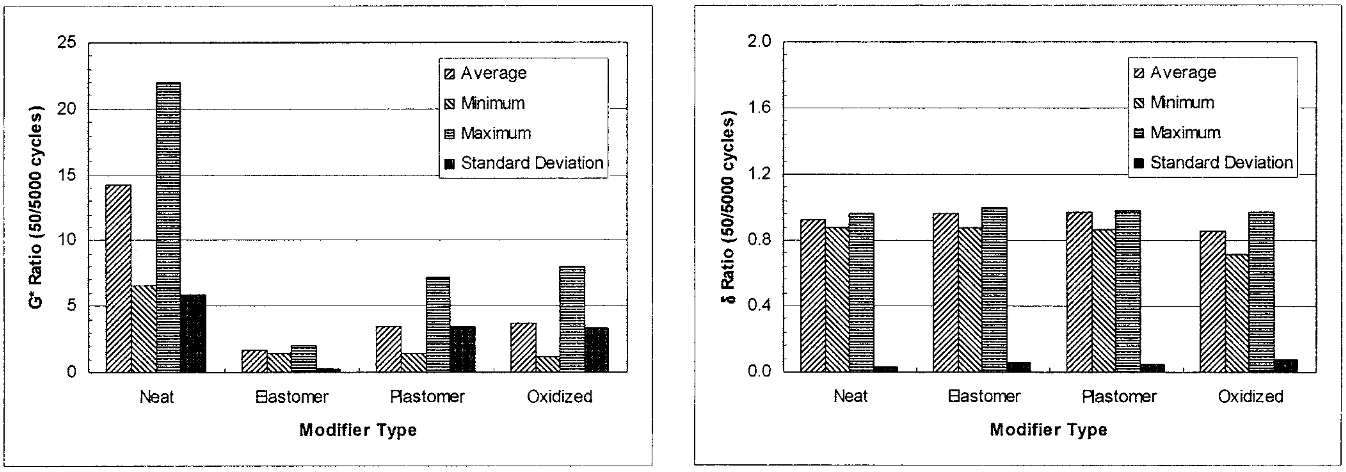


Figure 2.4 Effect of repeated loading at 20 percent strain on G* *and phase-angle ratios measured for 50 cycles relative to 5,000 cycles for different binders.*

repeated cycling. It also showed the importance of sensitivity to strain or stress nonlinearity.

The effects on the phase angle are shown in Figure 2.5. The results indicate that the effects on the phase angle are not as important as the effects on the *G** values. This finding suggests that relying on the phase angle to supplement the Superpave specification criteria for evaluating modified binders could be misleading.

Work was also done to evaluate the factors that control the highly significant effect on *G**. The results are presented in another publication (*44*), which focuses on the interaction between strain levels, temperature, frequency, type of modifier, and the fatigue performance. Figure 2.5 is a sample of the results, which show how increasing the strain level results in highly significant fatigue effects.

These observations are very important in understanding the fatigue behavior of asphalt mixtures. It is believed that binders within a mixture are subjected to relatively high strains and thus can be considered to play a major part in the fatigue behavior of asphalt mixtures. As will be shown in the following subsection, modified binders are observed to vary significantly in their strain dependency.

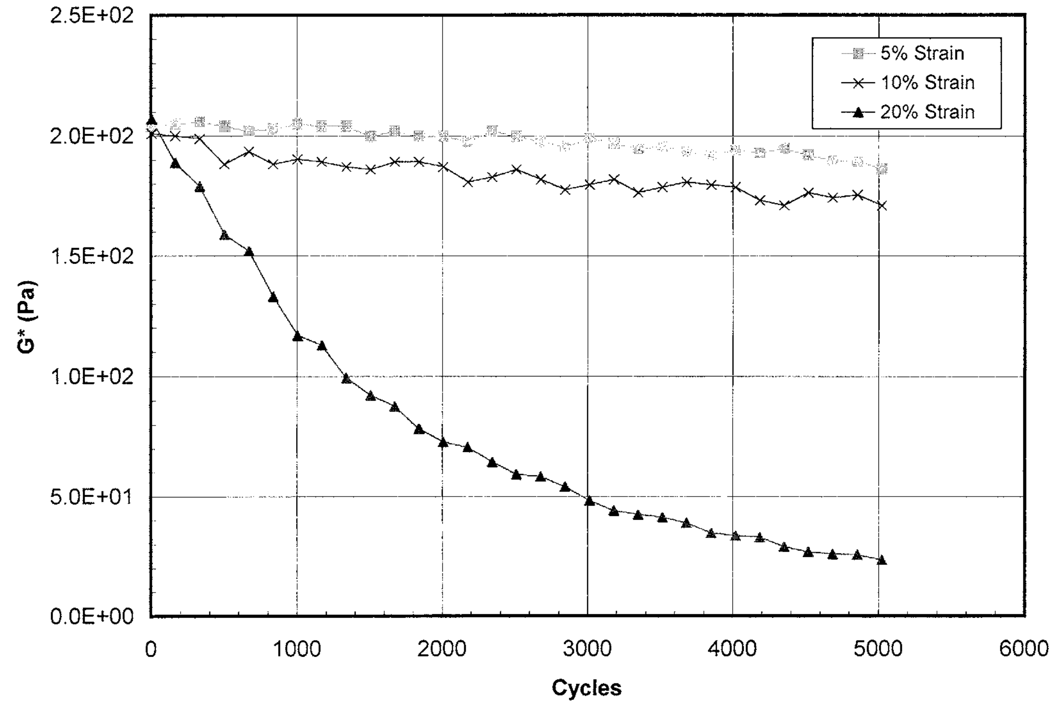


Figure 2.5 Typical interaction between strain and fatigue damage for a modified binder.

2.5.6 Effect of Pavement Structure (Strain Dependency)

To test the hypothesis that the contribution of modified binders to pavement performance depends on the pavement structure, it was necessary to test whether the current procedure of measuring properties at small strain (which is indicative of a strong pavement structure) can effectively estimate performance at high strain (which is indicative of a weak pavement structure).

The binders were tested for their strain dependency at HT and at IT. Strain sweeps at a constant frequency of 1.5 Hz were conducted for all binders. The results were summarized by calculating the ratio of the difference between the value at 2 percent strain and at 50 percent strain to the value at 2 percent strain. The ratios sorted by grade and by modifiers are shown in Figures 2.6 and 2.7 for HT and IT, respectively.

The following points summarize the findings:

- At HT, although few binders show high dependency on strain, the average values are within a 10-percent change for most grades and types of modifiers.
- There is no specific trend that could be observed between strain dependency and grade of binder or type of modifier at HT.
- At IT, the majority of binders show significant strain dependency. The averages of *G** ratios based on grades are close to 80 to 90 percent.
- The ratios of *G** at IT do not show a significant trend with grade. There is, however, a clear distinction between types of modifiers. The processed binders (i.e., binders modified by refinery processes with no additives) show significantly higher *G** ratios and lower δ ratios compared with the other modifiers (see Figure 2.7). The elastomers show the lowest *G** ratios, while the other group shows the lowest δ ratios.
- It appears that the limits for linearity (i.e., ratios of *G** and δ are very sensitive to temperature. It also appears that the sensitivity of the *G** ratio and δ ratio to temperature is not the same for all binders.

The above results indicate that the strain dependency is one of the important characteristics that will distinguish modified binders and that can possibly discriminate between well-performing and poor-performing binders. As shown in Figure 2.7, it is clear that processed binders are different from the other types of binders. Processed binders are known to have lower ductility, and a number of experts have expressed concern regarding the use of such binders. An increase in the phase angle similar to that observed in Figure 2.7 may indicate nonhomogeneity or breakdown of the molecular structure. It is observed that, for the majority of binders, nonlinear behavior starts at a low strain and is highly modifier specific.

2.5.7 Low-Temperature Creep and Glass-Transition Properties

In addition to the bending beam rheometer, the direct tension and a newly developed glass-transition test were used to characterize LT behavior of modified binders. The objective of the testing was to evaluate the applicability of the most recent protocol developed by the TRB Binder Expert Task Group for calculating low-temperature cracking temperature of modified asphalt binders. The results were used to compare the binder behavior with the mixture creep and failure behavior and to quantify the effects of modification on mixture behavior.

The failure properties of binders were measured using the latest direct tension test procedure developed by TRB as in AASHTO TP3-98. The procedure was expanded to include three strain rates at each of the three temperatures. Figure 2.8 presents an example of the data generated for the failure properties. The data represent failure stress and failure strain that

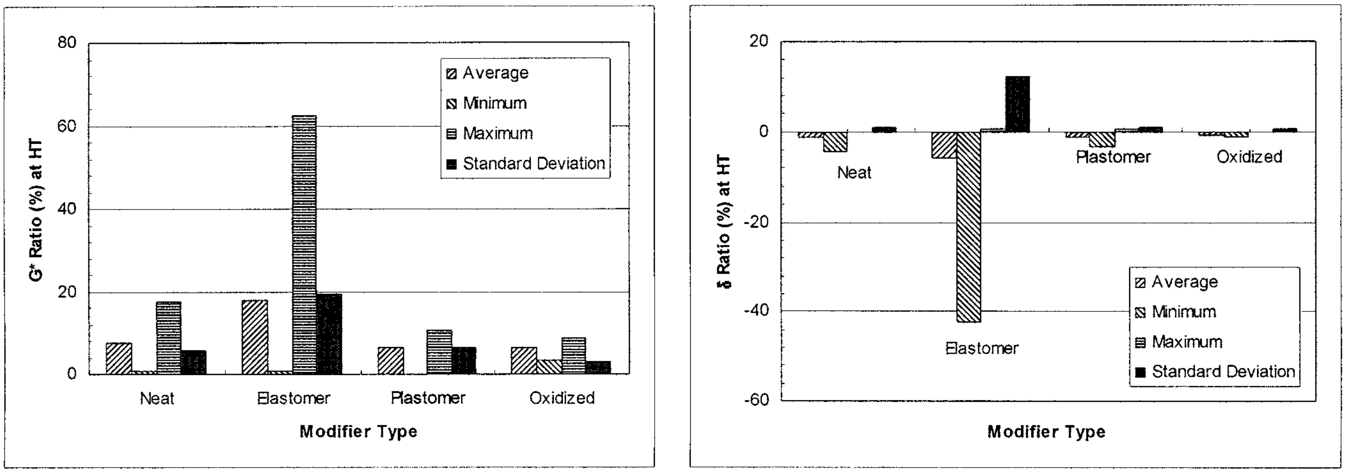


Figure 2.6 G* *and δ ratios measured from strain sweeps between 2 and 50 percent (strain dependency) at HT sorted by type of modifier and by grade.*

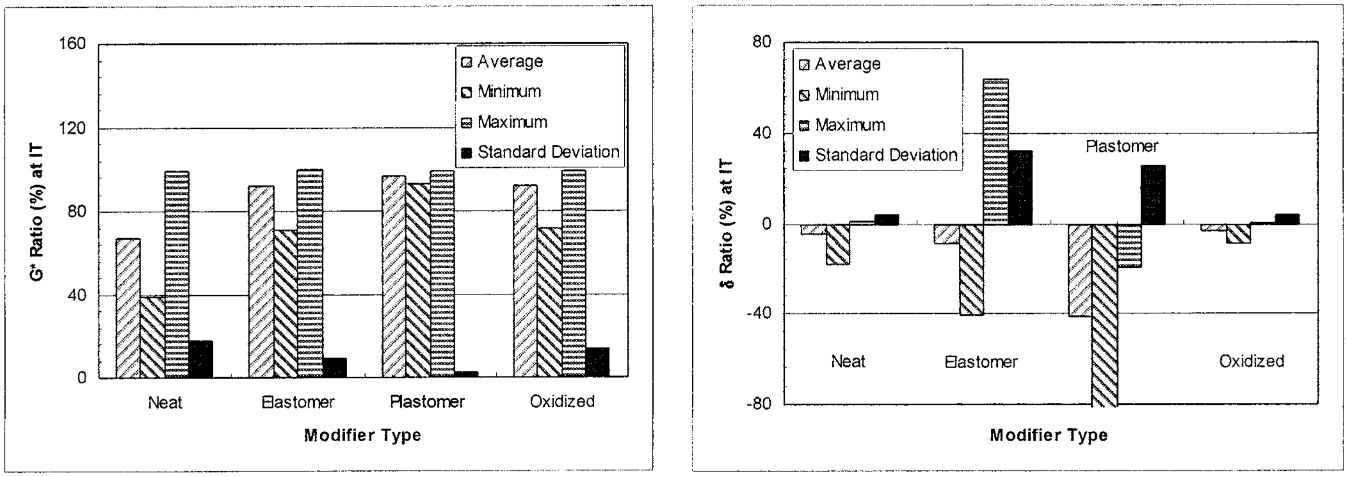


Figure 2.7 G and δ ratios measured from strain sweeps between 2 and 50 percent (strain dependency) at IT sorted by type of modifier and by grade.*

are shifted along the loading time scale (i.e., the horizontal scale) to produce failure master curves. The shifting is done with the time–temperature shift factors estimated from the bending beam rheometer. These master curves are necessary to estimate the critical cracking temperature based on the stress at failure and the strain at failure. The details of the models used to fit the data and the computational steps used to calculate the critical temperatures were published by the research team (*45*).

This approach, which is compatible with the recommendations of the Expert Task Group, extends the analysis to the strain at failure criterion, which is not considered in AASHTO MP1A. The approach also considers the effect of cooling rate and solves the problem of matching the rate of cooling in the development of stress build-up to that in estimating strength or strain at failure. In the analysis procedure, the thermal properties of binders including the glass-transition temperature are needed. Instead of making assumptions about these properties, as is recommended in AASHTO MP1A, the glass-transition device was used to measure these properties and incorporate them in the analysis.

The dilatometric glass-transition temperature (T_g) was measured using a testing procedure employed originally in the SHRP A-002A project (*14*). The T_g measurements include the change in volume as a function of temperature between

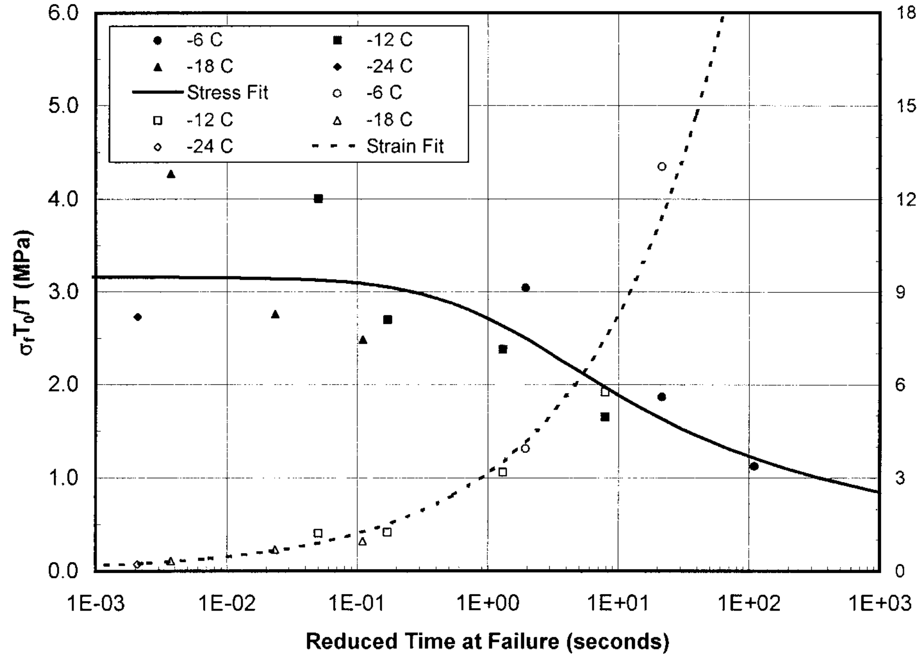


Figure 2.8 Strain at failure and stress at failure master curves for an asphalt binder.

+40 and −70°C. Figure 2.9 shows an example of the glass-transition measurements conducted in this project and the curve fitting used to estimate the parameters.

A computer program was developed to provide a better control of the glass-transition device and to fit the data and to calculate the parameters. In the program, the following model is used to estimate α_l, the coefficient of contraction above the glass-transition temperature; the glass-transition temperature, T_g, and α_g, the coefficient below T_g:

$$v = c_v + \alpha_g (T - T_g) + R(\alpha_l - \alpha_g) \cdot \ln\{1 + \exp[(T - T_g)/R]\} \tag{1}$$

Here, v is the specific volume change, c_v is a constant, and R is a regression constant related to the rate of the volume change at and near the glass-transition temperature, T_g. This model was used during the SHRP program for the same purpose. As shown in Figure 2.9, the system collects data from two samples simultaneously, and the data from both replicates are used to fit the model. The starting temperature is 40°C, and the minimum temperature is −70°C. The data generated are used to quantify the effects of modifiers on the thermal behavior of asphalt binders.

2.5.7.1 Failure Properties

The low-temperature failure properties (see Table 2.8) are represented by critical cracking temperatures. These are based on the concept that cracking will occur when thermal stress reaches the strength of the binder (shown in Table 2.8 under the title "Stress"). This critical temperature was calculated with a computer program developed for this purpose. The program can also calculate a critical temperature based on a selected conversion factor (shown in Table 2.8 under "×18 Stress" for a factor of 18) as well as a critical cracking temperature based on the concept that cracking can occur when thermal strain exceeds the failure strain (shown in Table 2.8 under "Strain"). This program follows the same principles used in the procedure for determining the critical cracking temperature recommended recently by the TRB Binder Expert Task Group as included in AASHTO MP1A.

The results in Table 2.8 clearly show, however, that the cracking temperature is highly dependent on the criterion used. The conversion factor can have an important effect that is not directly proportional to the value of the factor. The strain criterion can give a different ranking for the same binder performance grade.

The initial analysis also indicates that modification can have a significant effect on the cracking temperature and that this effect is also dependent on the criterion used. Figures 2.10 and 2.11 show examples of the changes in failure properties as a result of modification and other factors. In Figure 2.10 (a), the failure strain master curve is plotted for the base asphalt and three modified binders of the same low-temperature grade (PG XX-22). The failure strain behavior is significantly different for the three binders; the elastomer-modified binder shows the highest ductility. Furthermore, the ranking is temperature specific. In other words, if these binders are compared at −10°C, they will be ordered differently at 0°C. The entire strain curve is highly dependent on the modifier type.

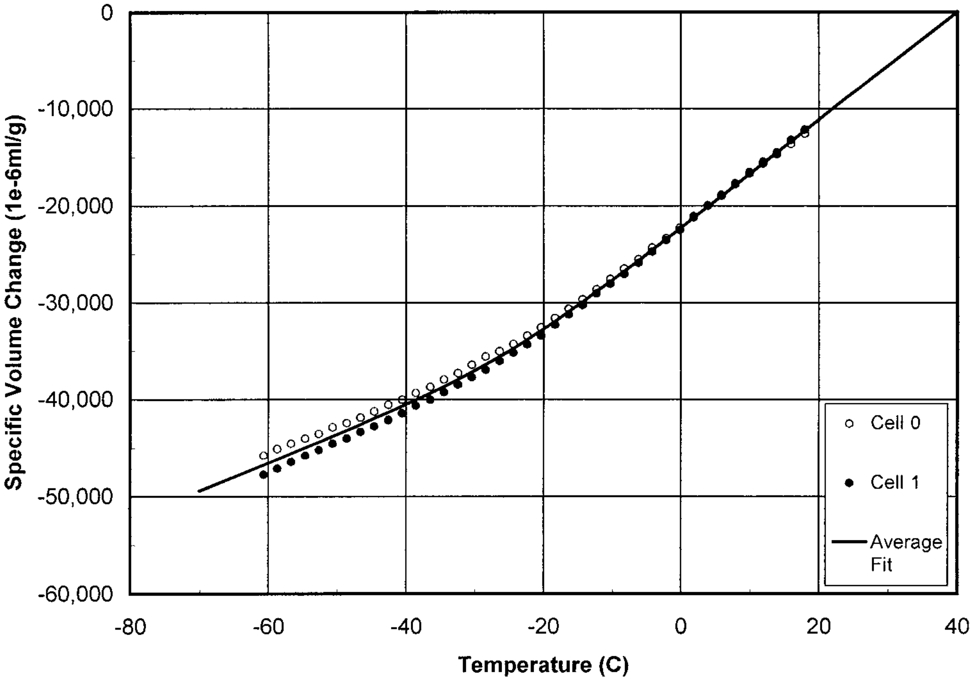


Figure 2.9 An example of the glass-transition measurements of asphalt binders.

TABLE 2.8 Summary of results of estimates of critical cracking temperature for modified binders

Modified Binders	Modification Category	Cooling Rate (°C/hr)	Critical Temperatures (°C)		
			Stress	×18 Stress	Strain
PG XX-22 Grades					
PG 70-22 Base	None	1	-52.1	-17.0	-34.7
PG 82-22 SBS Linear	Elastomer	1	-49.1	-14.5	-37
PG 82-22 SBS Radial	Elastomer	1	-52.3	-14.4	-36.5
PG 82-22 EMA	Plastomer	1	-49.2	-14.1	-33.7
PG 82-22 PE Stabilized	Plastomer	1	-52.7	-16.3	-35.6
PG 82-22 Steam Distilled	Oxidized	1	-55.4	-12.7	-34.3
PG 76-22 EMA	Plastomer	1	-54.5	-10.0	-37.4
PG 76-22 Ethylene Tripoly	Elastomer	1	-52.7	-15.8	-34.8
PG 76-22 PE Unstabilized	Plastomer	1	-47.1	-14.5	-33.3
PG 76-22 Oxidized (BB)	Oxidized	1	-50.6	-16.2	-34.7
PG 76-22 Oxidized (SR)	Oxidized	1	-53.9	-19.3	-34.7
PG XX-34 Grades					
PG 70-34 SBS Radial	Elastomer	1	-51.1	-15.8	-41.8
PG 70-34 SB Di-block	Elastomer	1	-59.2	-28.9	-41.7
PG 70-34 N.A.	N.A.	1	-67.1	-32.9	-39
PG 64-34 SBS Linear	Elastomer	1	-59.1	-22.9	-43
PG 64-34 Oxidized (SR)	Oxidized	1	-62.4	-26.9	-43.2
PG 64-34 Oxidized (BB)	Oxidized	1	-55.8	-27.6	-41
PG XX-40 Grades					
PG 58-40 SBS Linear	Elastomer	1	-72.7	-32.7	-54.3
PG 58-40 Ethylene Tripoly	Elastomer	1	-69.8	-31.8	-50.1
PG 58-40 Oxidized	Oxidized	1	-67.4	-38.2	-46.9
PAV-Aged Binders					
PG 82-22 (P) SBS Radial	Elastomer	1	-55.8	-16.6	-30.5
PG 76-22 (P) EMA	Plastomer	1	-53.5	-8.8	-32.8
PG 70-34 (P) SB Di-block	Elastomer	1	-45.4	-10.4	-39.3
PG 58-40 (P) Ethylene Tripoly	Elastomer	1	-78.8	-37.9	-46.8
Field-Related Binders					
PG XX-22 AC20	N.A.	1	-49.4	-6.2	-31.2
PG XX-22 MGAC	N.A.	1	-39.4	-10.5	-31.3
PG XX-22 Neoprene	N.A.	1	-53.3	-15.7	-35.2
PG XX-22 Novophalt	N.A.	1	-39	-8.3	-26.4
PG XX-22 PAC	N.A.	1	-56.6	-15.7	-38.3
PG 52-34 Canada	N.A.	1	-62.8	-13.6	-50
PG 52-40 Canada	N.A.	1	-66.8	-30.3	-43.3
PG XX-28 Polybilt	N.A.	1	-53.6	-19.0	-35.3
PG XX-28 Styrelf	N.A.	1	-74.2	-30.6	-43.2

NOTE: (P) denotes PAV-aged.

Figure 2.10 (b) shows the strength master curves for the same set of binders. The data in the figure indicate that the plastomer-modified binder shows higher strength values at all temperatures than do the elastomer-modified binders, which show strength characteristics similar to those of the unmodified binder. Also, it shows that the oxidized binders have higher strength values below −12°C than do all the rest of the binders.

Together, these data indicate that cooling rate and the overall position of the master failure curve will have a significant impact on the calculation of the cracking temperatures.

Figure 2.11 compares the failure strain master curves (Part a) and the strength master curves (Part b) for the same base asphalt with five modified binders that were modified with elastomers, but have different performance grades. The strain curves clearly show the effect of grade on the failure strain properties. As the low-temperature grade of the binder decreases, its failure strain curve rapidly shifts to lower temperatures. The strength master curves indicate that the PG 82-22 modified binder, the PG 76-22 modified binder, and the PG 70-22 binder have very similar strength characteristics. The other three binders, two PG XX-34 binders and one

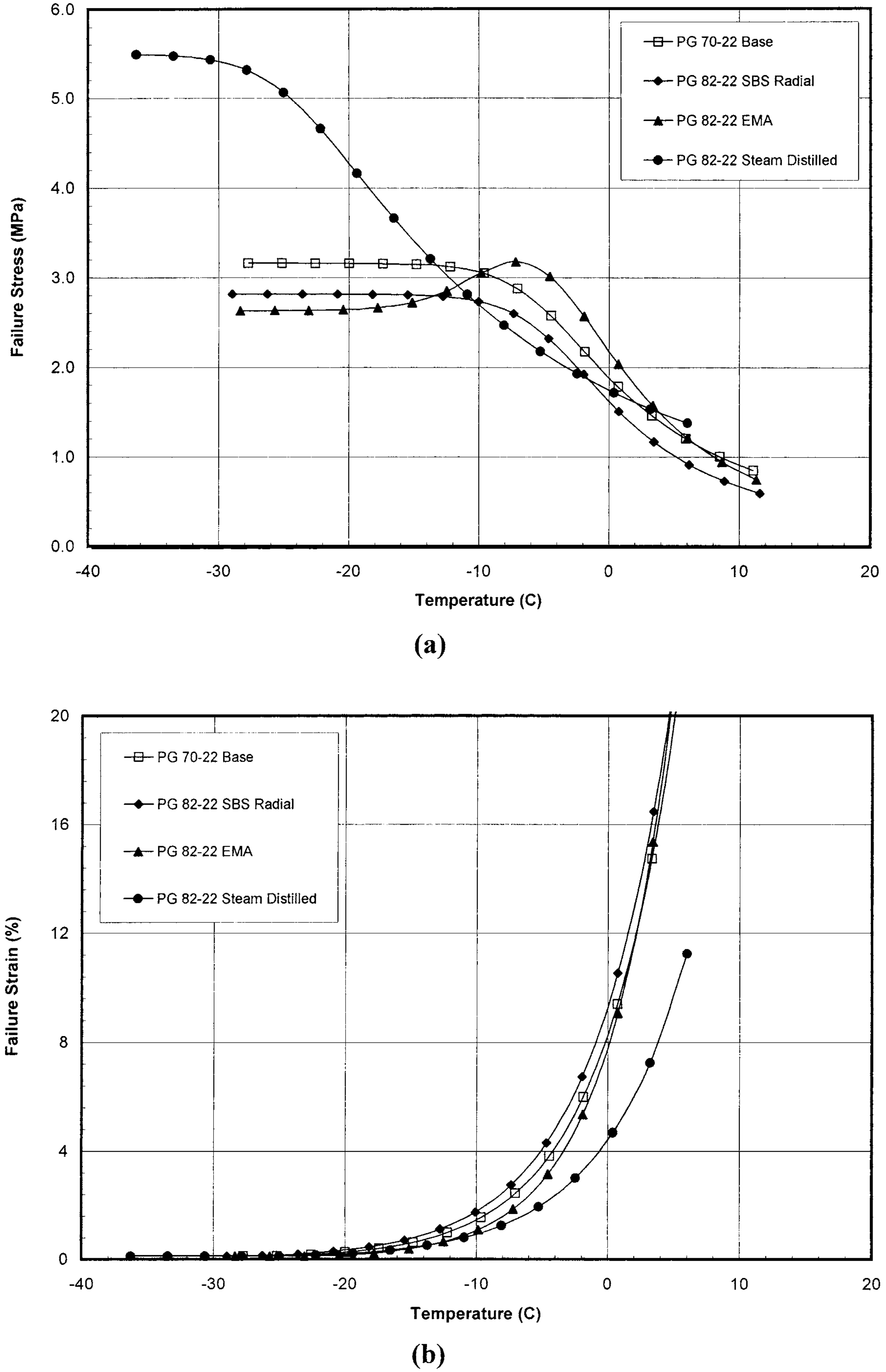
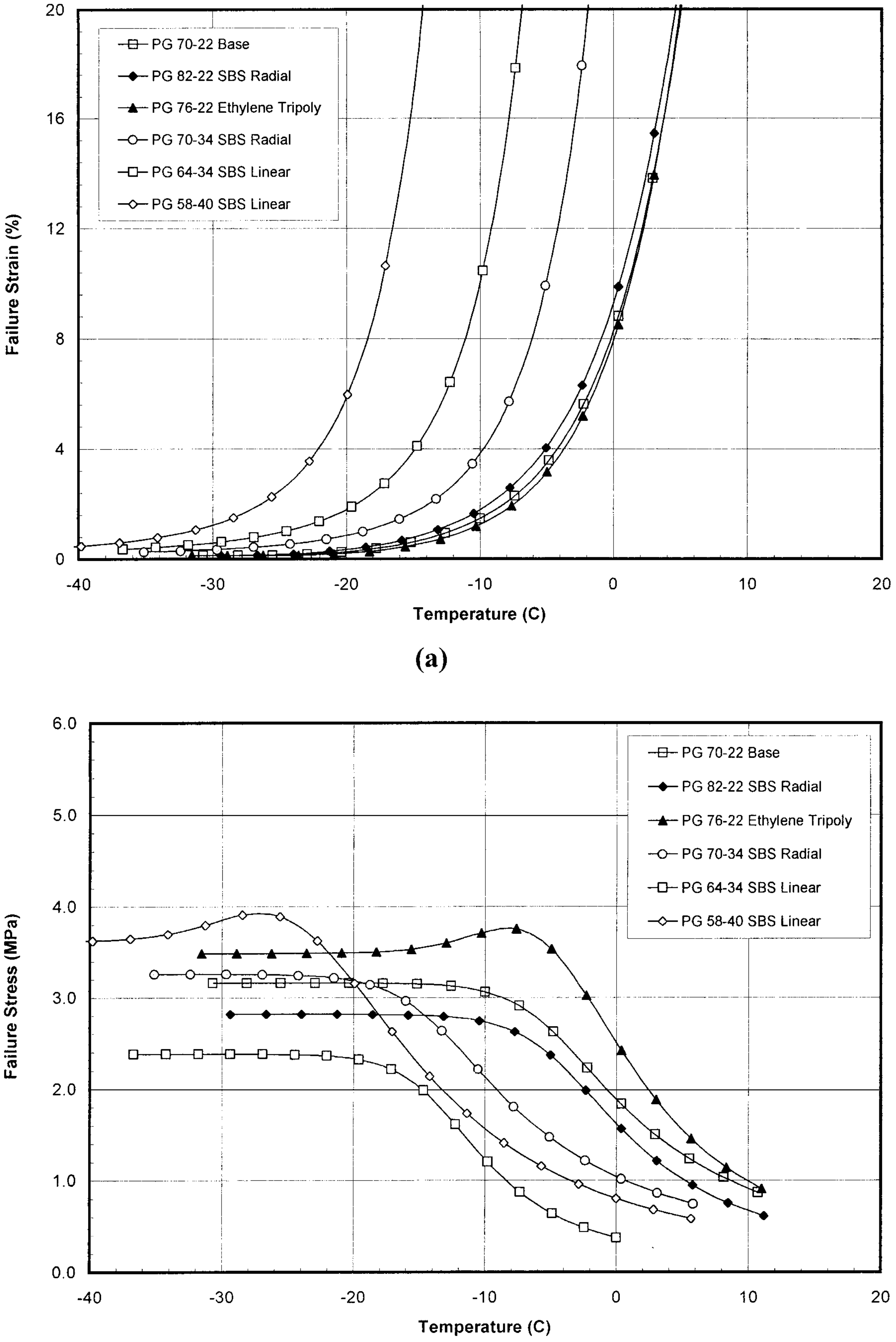


Figure 2.10 Effect of modifier on failure stress and failure strain (reference time being 10 s).



(a)

(b)

Figure 2.11 Effect of binder grade on failure stress (reference time being 10 s).

PG XX-40, show very different properties compared with these three binders.

These similarities and differences point out that the performance grading based on the creep-stiffness properties may not be sufficient to compare the failure characteristics in terms of failure strain or strength. It is observed that there is a similar shift in the master curve to lower temperatures with the grade of the binders. However, the shape of the curve and the peak value vary significantly and are not well related to the grade of the binder. It is also observed that a better understanding of failure properties of modified binders is realized by comparing the master curves of the stress at failure and the strain at failure.

Similar results were obtained to evaluate the effect of the PAV aging. The effect on the strength appears to be almost negligible. The effect of the PAV aging on failure strain values is, however, very important and results in a significant shift of the failure strain to higher temperatures, which is an indication of brittleness. This finding is important because it shows that aging will affect strength differently in comparison with failure strain. The data indicate that the effect of PAV aging on failure characteristics of modified binders could be very complicated and counterintuitive. These results confirm the importance of evaluating failure master curves and the danger of using a one-point measurement to estimate the role of binders in cracking.

2.5.7.2 Correlation Between Failure and Creep Properties

One of the concepts used to support the use of creep properties (i.e., stiffness and *m*-value at 60 s) in the current Superpave specification (AASHTO MP1) is the high correlation between creep and failure properties (*14*). The correlation between critical temperatures estimated from $S(60)$ and $m(60)$ with cracking temperatures based on thermal stress and thermal strain are shown in Figure 2.12. It is observed that, although the trends are logical, the correlation is not very high. The correlation coefficients, r^2, for the stress criterion are 0.49 with the $m(60)$ critical temperatures and 0.58 with the $S(60)$ critical temperatures. Moreover, there is a wide scatter in the data. The stress cracking temperature can vary by as much as 20°C for the same $S(60)$ or $m(60)$ critical temperature. Given the fact that the binder grading is based on 6°C intervals, this correlation is not acceptable. Similar trends can be observed in the case of failure strain cracking temperatures. The correlation coefficients are 0.63 with critical temperatures estimated based on $m(60)$ and 0.57 with critical temperatures estimated based on $S(60)$. The scatter around the correlation line is also in the range of 20°C. Thus, there is a logical relationship, but no strong correlation, between creep properties and failure properties for these modified binders. This indicates that these measures cannot be surrogates to each other. The failure properties will need to be measured and the cracking temperatures should be estimated based on a collective analysis of creep and failure data.

2.5.7.3 Effect of Modification on Glass-Transition Temperature

Figure 2.13 is a summary of the glass-transition temperatures (T_g) estimated for a set of binders of different grades and modification type. The base asphalt, a PG 70-22 prepared from the Boscan crude, is also shown as the first binder in the figure. The T_g values show that modification can have the significant effect of either increasing or decreasing the T_g of the base asphalt. The data also show that the PAV aging can result in a significant increase of the T_g value.

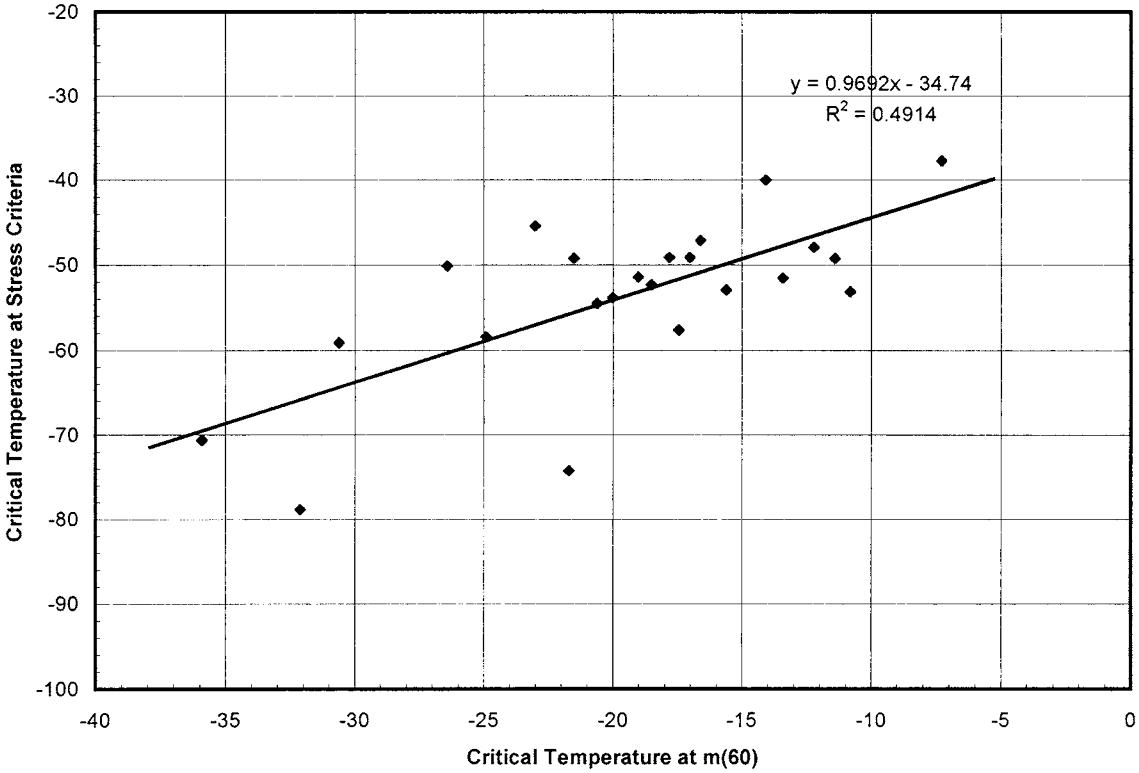
The glass-transition temperature ranges between a low value of –29°C for the PG 82 modified with ethyl-methyl-arylate (EMA) and a high value of –17°C for a PG 82 steam-distilled. The PG 76 grades show slightly lower T_g values, but also show a wide range of approximately 10°C. In the figure, there are five PG XX-34 grades and three PG XX-40 grades. There is a clear decrease in the T_g values for these binders, which indicates that improving the low-grade temperature is effective in reducing the T_g values. The reduction is, however, not consistent and is dependent on the modification technique.

It is also observed that the coefficients of contraction are affected by modification. Figure 2.14 depicts the values of the coefficients for the same set of binders. The values of the coefficient below T_g, α_g, are higher for all modified binders compared with the base asphalt. The coefficients above T_g, α_l, show mixed changes relative to the base asphalt. It is important to realize that the T_g values and the coefficients can have a major impact on the calculations of thermal stress and thermal strain in thermal analysis.

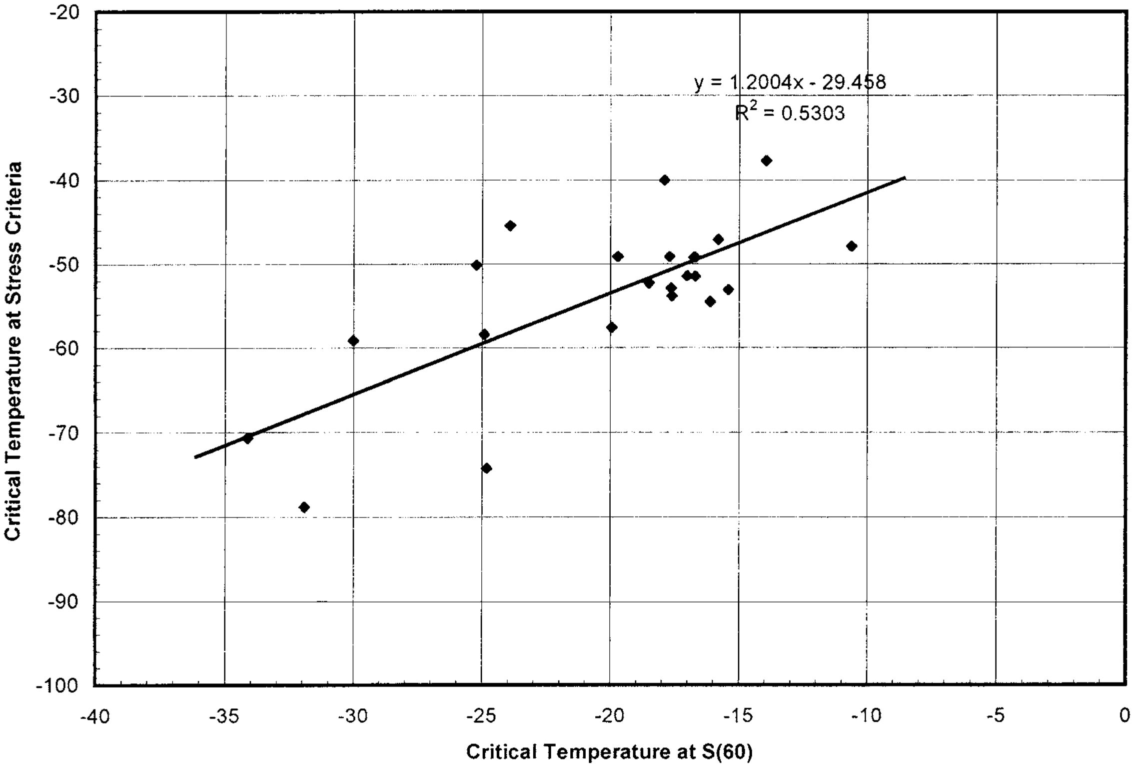
To assume that there is a universal value for the coefficient of contraction and that the effect of variation, T_g, is not important may result in serious errors. Unlike the T_g values, no clear distinction between grades or type of modifications could be seen in these data. What is important to notice is the wide range in these values. Many previous studies have indicated that, for unmodified binders, the coefficients vary within a narrow range. The results presented here show a change between low values for α_l of $525 \cdot 10^{-6}$/°C and $725 \cdot 10^{-6}$/°C. This is more than a 35-percent change. For α_g, the range is even higher and varies between a low value of $175 \cdot 10^{-6}$/°C and a maximum value of $475 \cdot 10^{-6}$/°C, which is more than a 100-percent change.

The analysis of results collected using the direct tension test (DTT) and the glass-transition measurement leads to the following findings:

- Modification can have a significant effect on cracking temperature, and the effect is dependent on the criterion used.
- Performance grading based on the creep-stiffness properties is not sufficient to evaluate the failure characteristics in terms of failure strain or strength.



(a) Correlation between stress and m*(60) critical temperatures.*



(b) Correlation between stress and S*(60) critical temperatures.*

Figure 2.12 Correlation between critical temperatures estimated from stress at failure and S*(60) or* m*(60).*

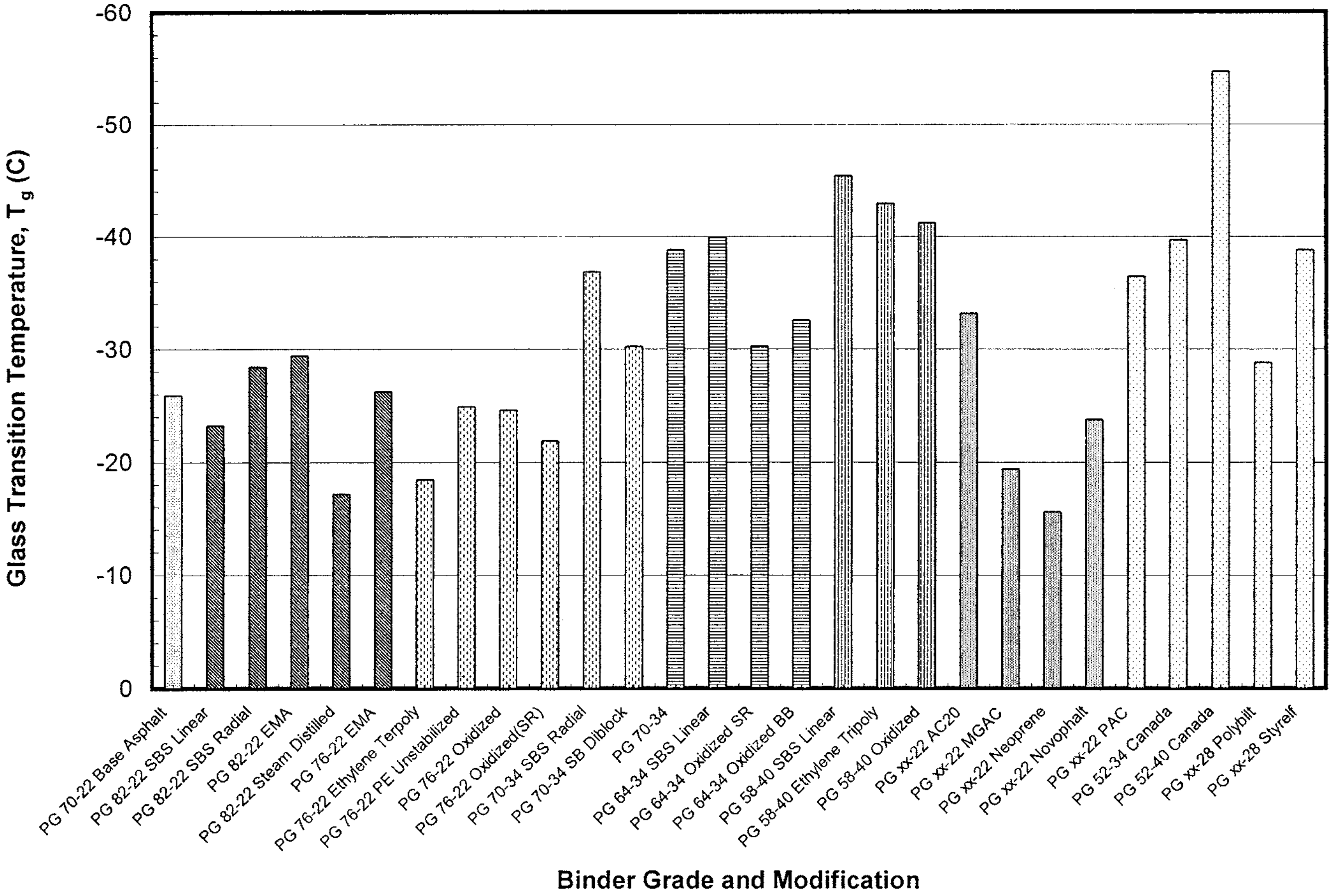


Figure 2.13 Effects of binder modification and PG grade on glass-transition temperature of asphalt binders.

- A better understanding of failure properties of modified binders is realized by comparing the master curves of the stress at failure and strain at failure. These master curves give more significant and informative information compared with calculating critical temperatures only.
- The effect of PAV aging on failure characteristics of modified binders is very complicated and counterintuitive. This confirms the importance of evaluating the failure master curves and the danger of using a one-point measurement to estimate the role of the binder in cracking.
- Comparing the critical cracking temperatures by criterion shows the variation in effects of the modifiers on different properties. It also shows that the approach of a one-point critical-temperature estimation could be very confusing and perhaps misleading.
- There is a need to carefully consider the cooling rate in the field and use failure stress and failure strain master curves to take into account the effect of cooling rate on failure properties.
- There is a logical relationship, but no strong correlation, between creep properties and failure properties for these modified binders. This indicates that the measures cannot be surrogates to each other. The failure properties will need to be measured and the cracking temperatures should be estimated based on a collective analysis of creep and failure data.
- The T_g values show that modification can have a significant effect by either increasing or decreasing the T_g of the base asphalt. It is also observed that the coefficients of contraction are affected by modification.
- The change in glass-transition temperature caused by aging varies significantly depending on the modifier type.

It is important to realize that the T_g values and the coefficients can have a major impact on the calculations of the thermal stress and thermal strain in the thermal analysis. To assume that there is a universal value of the coefficient of contraction and that the effect of variation T_g can be neglected may result in serious errors.

2.6 EFFECT OF MODIFIED ASPHALTS ON THE RHEOLOGICAL BEHAVIOR OF MIXTURES

The mixtures tested in this project were designed using the Superpave volumetric design procedure as defined in

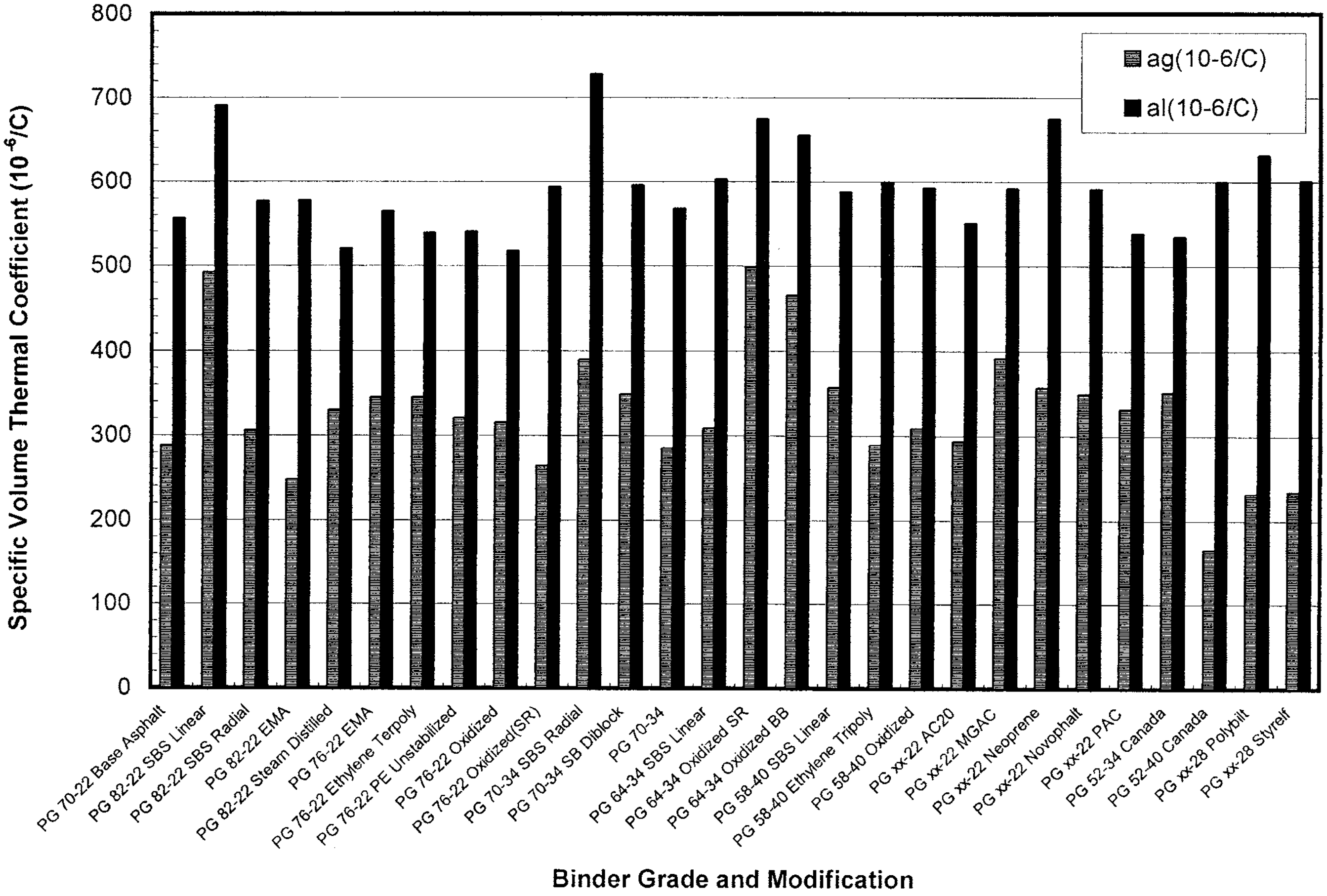


Figure 2.14 Effects of binder modification and PG grade on the thermal coefficients of contraction of asphalt binders.

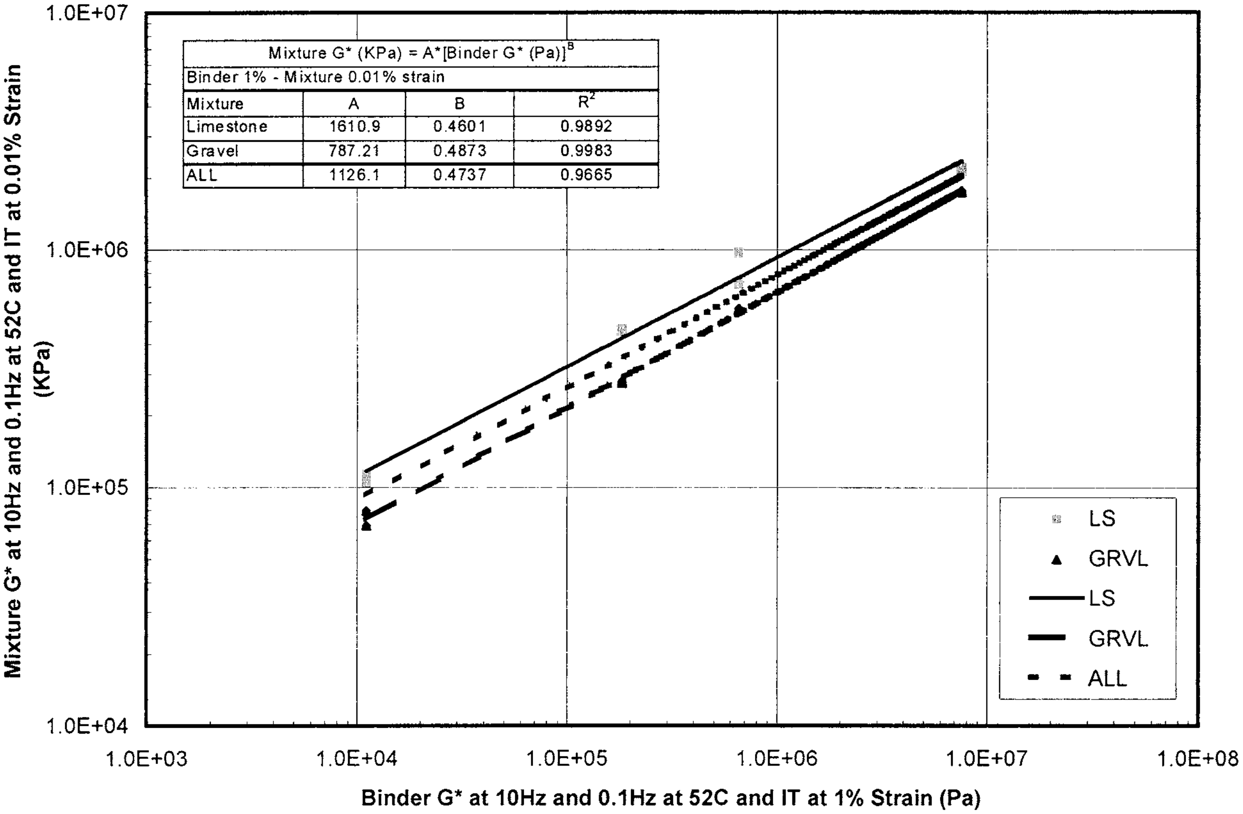
AASHTO MP2. In the mixture design process, the base (i.e., unmodified) asphalt was used to determine the asphalt content and to check the volumetric properties. The complete mixture design results are shown in Appendix A of this report. No attempt was made to characterize the mixture produced with the unmodified asphalt. The focus of the project was to evaluate the effect of modification type rather than to compare modified with unmodified mixtures.

Two performance specimens were prepared with the Superpave gyratory compactor at air voids of 3.5 ± 0.5 percent for each cell in the experiment. The mixtures were subjected to short-term oven aging (4 h at 135°C). The performance specimens were prepared according to the Superpave recommended protocols. Two test specimens, each 50 mm in height, were cut from the larger performance specimens prepared with the Superpave gyratory compactor.

2.6.1 Relationships Between Binder and Mixture *G** Values at Selected Conditions

To quantify the relationship between binder and mixture *G** values, all nine modified binders were aged in the RTFO and tested under temperature and frequency conditions comparable with those conditions used in the FSCH testing of mixtures. The binders were also tested at multiple strain levels to simulate the conditions of high and low strains used in the mixture testing. The *G** values of mixtures and binders, calculated from averaging two replicates, were plotted in the format of X–Y plots as shown in Figure 2.15. For each binder, four *G** values representing the measurements at 52°C and at the IT were used. At each temperature, frequencies of 0.1 Hz and 10 Hz were used. These conditions were selected to represent the HT, 52°C, at which rutting is believed to be important, and the IT, at which fatigue is important.

To take into account the effect of strain level during testing, two levels of strain were selected. The low-strain conditions were represented by the mixture results at 0.01 percent (100 microstrain) and the binder results at 1 percent (see Figure 2.15 [a]). To represent the high strain conditions, the mixture results at 0.1 percent (1,000 microstrain) and the binder results at 10 percent strain were used. The detailed data analysis was documented in a final topical report for Task 8. The following represents a summary of the main findings supported by sample data.

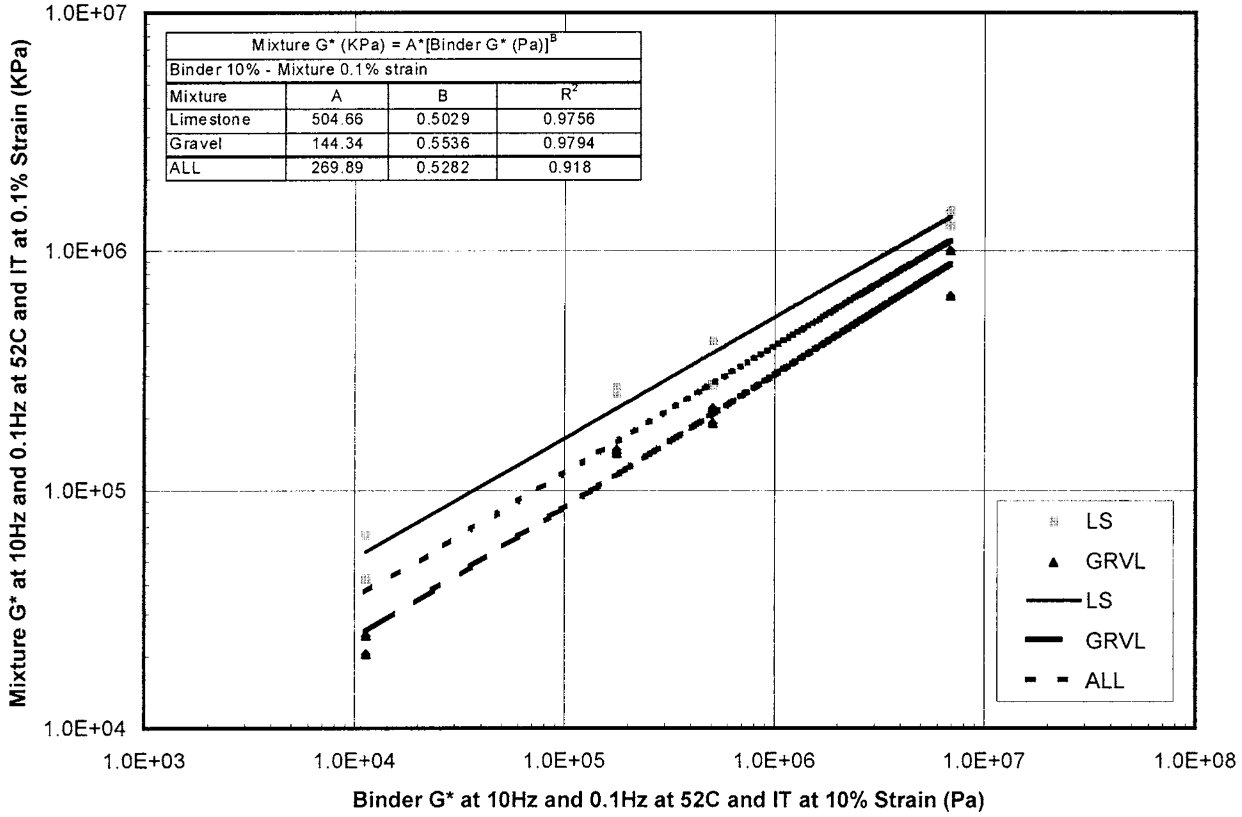


Mixture G* (KPa) = A*[Binder G* (Pa)]^B

Binder 1% - Mixture 0.01% strain

Mixture	A	B	R^2
Limestone	1610.9	0.4601	0.9892
Gravel	787.21	0.4873	0.9983
ALL	1126.1	0.4737	0.9665

(a) Low-strain conditions: mixture at 0.01% and binder at 1.0%.



Mixture G* (KPa) = A*[Binder G* (Pa)]^B

Binder 10% - Mixture 0.1% strain

Mixture	A	B	R^2
Limestone	504.66	0.5029	0.9756
Gravel	144.34	0.5536	0.9794
ALL	269.89	0.5282	0.918

(b) High-strain conditions: mixture at 0.1% and binder at 10%.

Figure 2.15 Relationship between binder and mixture G* *measured at same temperature and frequency for PG 82 SBSr binder and mixtures with four types of aggregates.*

2.6.1.1 *Effect of Aggregate Properties on Rheological Behavior*

Plotting the binder *G** versus mixture *G** indicated that the relationship approximately follows a power-law function. As shown in Figure 2.15, a log–log plot results in a simpler linear relationship. In the plots, the fitted power-law model parameters are shown for individual mixtures according to the binder used and the aggregate type. The power-law model used to fit the data is as follows:

$$G^*_{mix} = A(G^*_{bin})^B \tag{2}$$

where A and B are model parameters. Parameter A can be considered a hypothetical intercept representing the mixture G^* at a very low value of binder G^*.

The data in Figure 2.15 include four types of aggregates: limestone fine and coarse gradations and gravel fine and coarse gradations. The data shown are for a PG 82-22 binder modified with SBSr. Several important observations can be derived from this sample plot:

- The average slope in the log–log plots is approximately 0.50, which implies that a 10-fold increase in binder *G** translates into only a 3.16-fold increase in mixture *G**.
- The gradation does not appear to be an important factor, and, in fact, the coarse and fine gradations of each aggregate source were fitted to one curve with a high degree of correlation.
- Aggregate source is more important than gradation for both strain levels. The gravel mixtures have a lower overall *G** value compared with the limestone mixtures. The limestone mixtures have lower intercept values, but higher slope values compared with the gravel mixtures for the same type of binder and for a given range of binder *G**. This trend is reasonable because angular aggregates are expected to be more sensitive to binder properties.
- There is a significant effect of the strain level. For this binder, it appears that changing the strain from 100 to 1,000 microstrain in mixture testing resulted in greater effects than those effects contributed by the source and the gradation combined.
- The analysis of the full data set indicated the following important trends:
  - There is a significant effect of the modification type,
  - There is a significant effect of the strain level, and
  - There does not appear to be a major effect of aggregate gradation on the relationship between binder and mixture *G** values.

In order to present the details of the effects of modification type, the analysis was expanded to present the relationships for the nine binders with each of the mixtures and at each level of strain. These results are discussed in the next section.

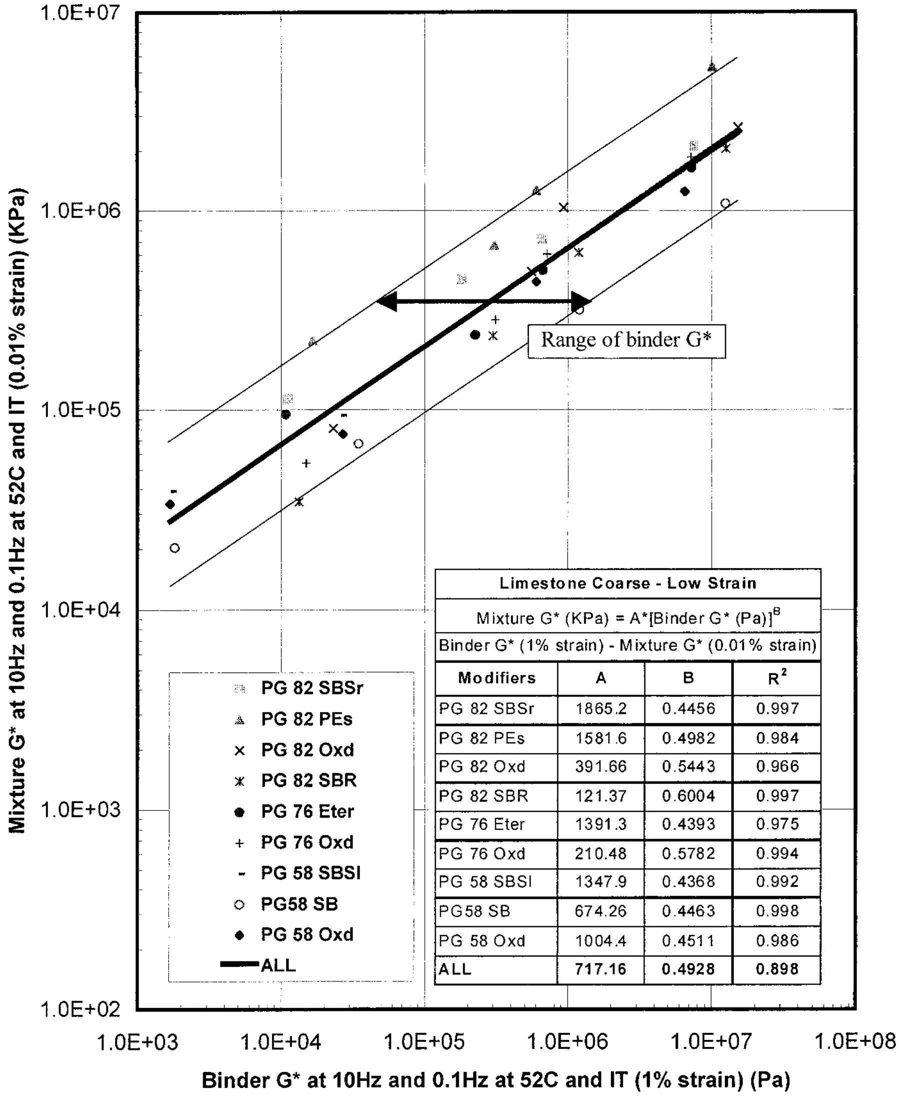
2.6.1.2 *Effect of Binder Modification on Mixture Performance*

Figure 2.16 presents the relationship between *G** for nine binders tested in this part of the project and the *G** of the limestone coarse mixtures made with these binders measured at the low strain level. The table embedded in the plot provides the intercept and slope for each of the binders as well as the values for the overall fit for all the data. The data sets were sorted by mixture type and by strain level and were plotted for other mixtures and strain levels. Figure 2.17 shows a similar plot comparing the averaged fitted model for low strain and high strain for the limestone and the latter with the gravel coarse at high strain. The full set of data and their analysis are presented in the final topical report for Task 8. The analysis led to the following findings:

- The *G** values of mixtures are a simple function of the binder *G** values. Using the power-law function, the mixture *G** can be plotted as a linear function of the binder *G** on a log–log plot. The plot defines the mixture *G** values in terms of an intercept parameter and a slope parameter.
- The intercept and the slope values are highly dependent on the modification type of the binder and the strain level of testing.
- The slope parameter of the log–log relationship is approximately 0.5, indicating that for every 10-fold change in binder *G**, the mixture *G** changes by a factor of 3.2.
- At higher strains, the mixture is more sensitive to binder *G**. Also, the variation in mixture *G** changes in modification type is significantly higher than the variation at low strains.
- Aggregate gradation shows only a minor effect on the intercept and the slope values compared with other factors. Aggregate source is more important than gradation, but has less effect in comparison with the modification type or strain level.

2.6.2 Sensitivity of Mixture Properties to Temperature and Frequency

Temperature, traffic speed, and strain are among the most important factors affecting asphalt pavement response. To quantify the sensitivity of mixture properties to selected changes in these factors, the fitted rheological models were used to calculate the ratios of the values of G^*, G', and G'' at 46°C to the corresponding values at 52°C for all the 36 mixtures tested. The ratios grouped by aggregate type were used to produce column charts for comparison as shown in Figure 2.18. In addition, the effect of frequency, which represents the effect of traffic speed, was measured by calculating the ratios of



Limestone Coarse - Low Strain

Mixture G* (KPa) = A*[Binder G* (Pa)]^B

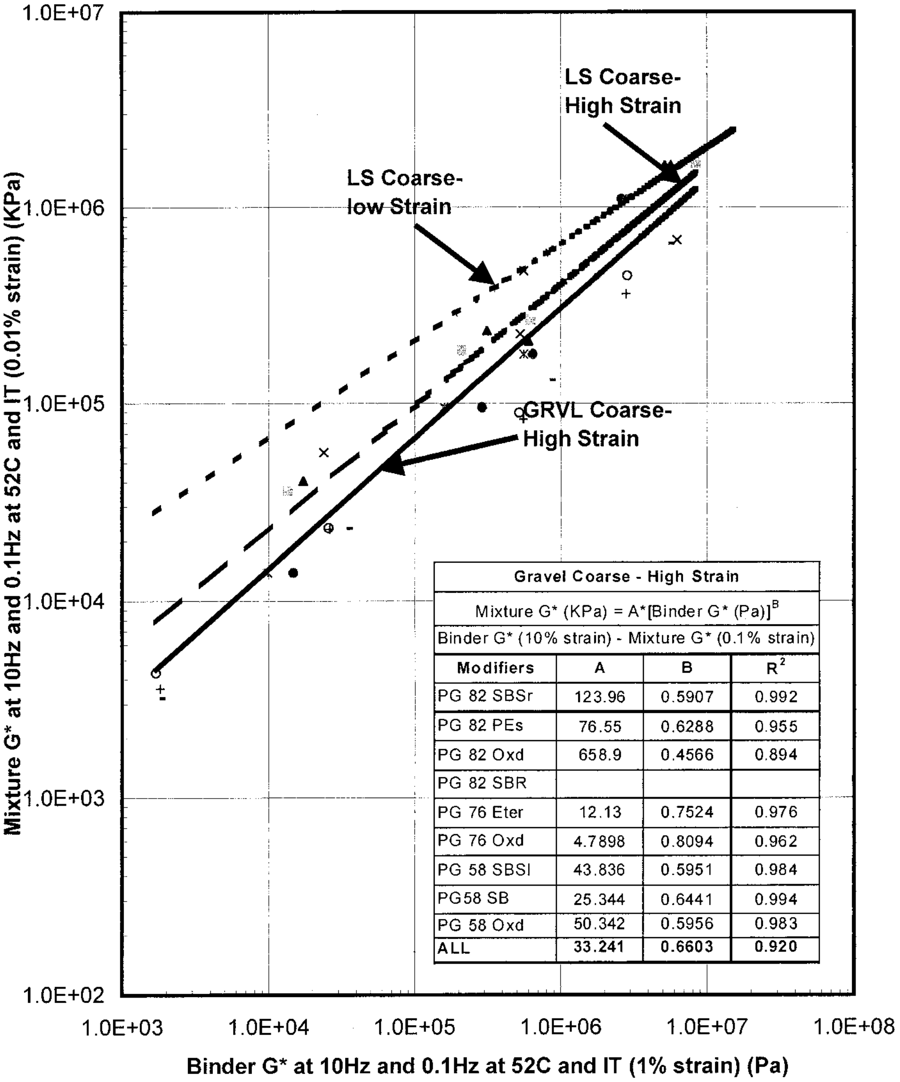
Binder G* (1% strain) - Mixture G* (0.01% strain)

Modifiers	**A**	**B**	**R^2**
PG 82 SBSr	1865.2	0.4456	0.997
PG 82 PEs	1581.6	0.4982	0.984
PG 82 Oxd	391.66	0.5443	0.966
PG 82 SBR	121.37	0.6004	0.997
PG 76 Eter	1391.3	0.4393	0.975
PG 76 Oxd	210.48	0.5782	0.994
PG 58 SBSI	1347.9	0.4368	0.992
PG58 SB	674.26	0.4463	0.998
PG 58 Oxd	1004.4	0.4511	0.986
ALL	**717.16**	**0.4928**	**0.898**

Figure 2.16 Effect of grade and modification type of binders on the rheological properties of mixture produced using the limestone coarse aggregate (low-strain conditions).

the values at 10 Hz to the values at 0.1 Hz at 52°C, as shown in Figure 2.19. The effect of the frequency is also evaluated at the intermediate grade temperature, as shown in Figure 2.20. In each of these figures, the binder ratios are also shown in order to compare relative roles of binders and aggregates in the changes in rheological properties of mixtures. The analysis of these results led to the following findings:

- Binders vary significantly in their sensitivity to temperature and frequency. This variation is carried over into the mixture properties.
- The sensitivity of mixtures to temperature is significantly lower than the sensitivity of binders. In addition, the effect of changing temperature by 6°C on mixtures ranges between a relative value from 1.1 to 1.6; for binders, it is in the range of 1.7 to 3.4.
- The sensitivity of mixtures to frequency is also significantly lower than the sensitivity of binders. The sensitivity is also highly dependent on the temperature range. At 52°C, the effect of changing frequency from 0.1 Hz to 10 Hz on mixtures is a relative change of 3 to 6; for binders, it is in range of 16 to 24.
- At ITs, the effect of frequency is not as high as at the HT range. The relative change for the same range of frequency is between 2 and 4 for mixtures and between 5 and 11 for binders.
- The viscoelastic properties of binders, in terms of separation of this response into the loss and storage com-



Gravel Coarse - High Strain			
Mixture G* (KPa) = A*[Binder G* (Pa)]^B			
Binder G* (10% strain) - Mixture G* (0.1% strain)			
Modifiers	**A**	**B**	**R^2**
PG 82 SBSr	123.96	0.5907	0.992
PG 82 PEs	76.55	0.6288	0.955
PG 82 Oxd	658.9	0.4566	0.894
PG 82 SBR			
PG 76 Eter	12.13	0.7524	0.976
PG 76 Oxd	4.7898	0.8094	0.962
PG 58 SBSl	43.836	0.5951	0.984
PG58 SB	25.344	0.6441	0.994
PG 58 Oxd	50.342	0.5956	0.983
ALL	**33.241**	**0.6603**	**0.920**

Figure 2.17 Effect of grade and modification type of binders on the rheological properties of mixture produced using the gravel coarse aggregates (high-strain conditions).

ponents, do not appear to have a significant impact on mixtures at HT. At 52°C, the relative changes of the storage (G') and loss (G'') moduli are very similar and not sensitive to the binder viscoelastic characteristics. At ITs, the storage moduli (G') of mixtures are more sensitive to the frequency than are the loss moduli (G'').

2.6.3 Mixture Rheological Master Curves

Master curves provide a fundamental rheological understanding of viscoelastic materials and allow an estimation of mechanical properties at wide ranges of temperature and frequency that could be realized in the field, but that are not practical to directly simulate in the laboratory. In this study, an attempt was made to develop a model for the master curves of mixtures as well as for binders. This model was used to describe the role of binder modification and grade in changing the overall rheological behavior of mixtures.

Many researchers have attempted to model the behavior of asphalt binder or asphalt mixture (*46–53*). However, a universal model for the characterization of both asphalt binder and mixture with one being the other's special case has not been found in the literature. A mathematical model that best characterizes asphalt mixture and binder was developed during this study. This model is composed of four equations for the complex modulus master curve, phase-angle master curve,

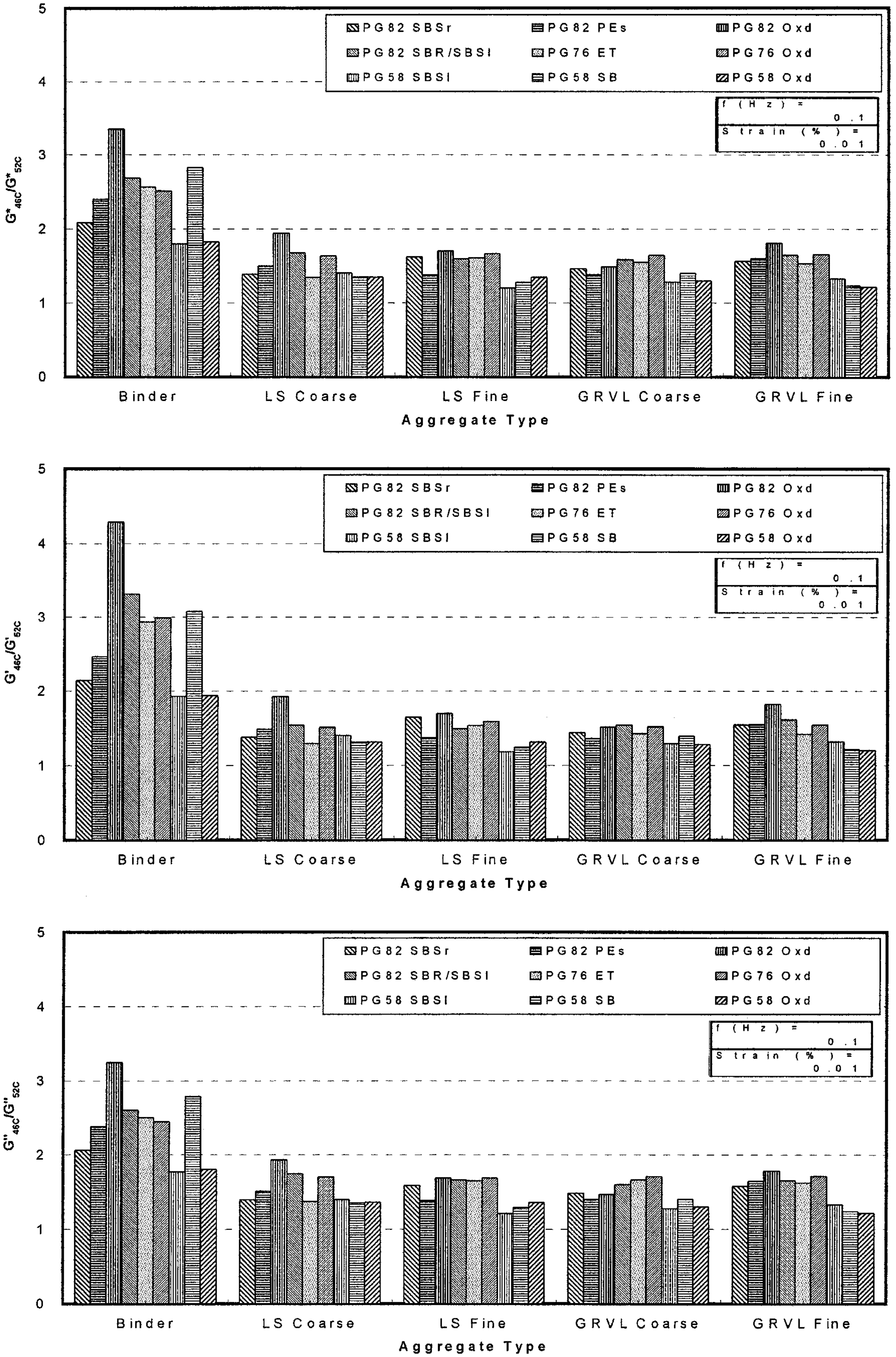


Figure 2.18 Ratios of G*, G′, and G″ *for mixtures measured at 46°C to the values measured at 52°C (0.01 percent strain, 0.1 Hz frequency).*

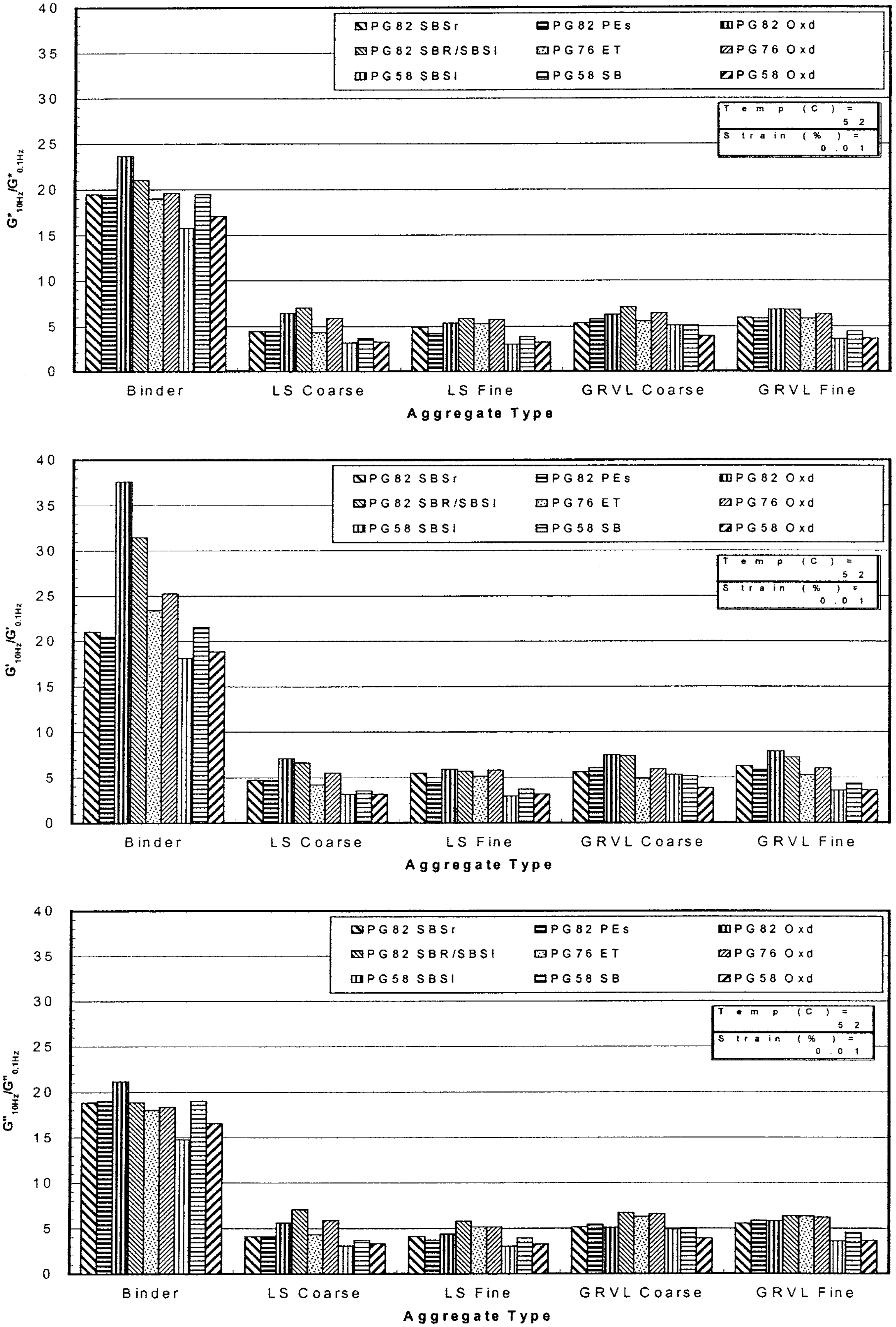


Figure 2.19 Ratios of G*, G′, *and* G″ *for mixtures measured at 10 Hz to the values measured at 0.1 Hz (0.01 percent strain, 52°C).*

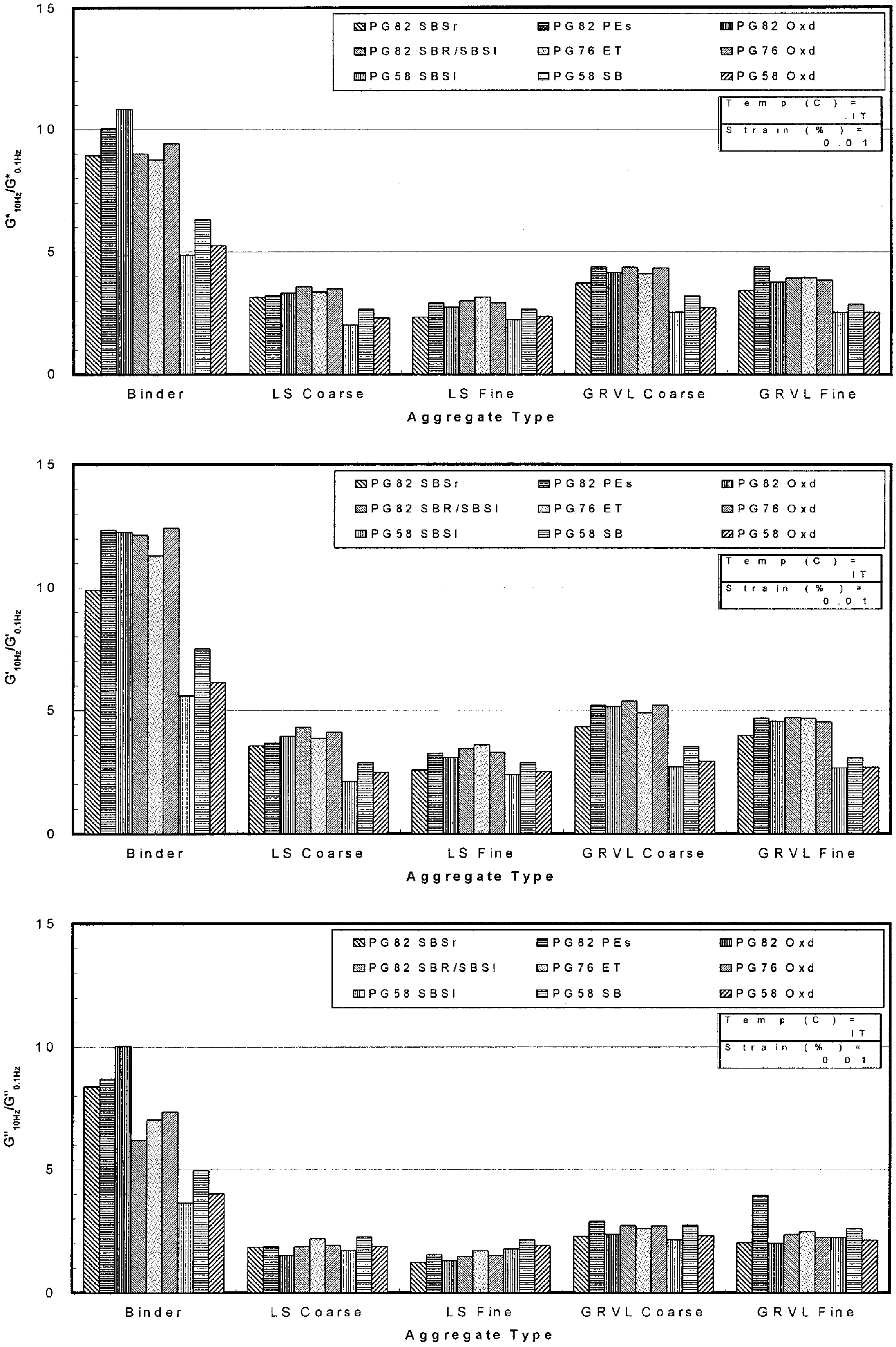


Figure 2.20 Ratios of G*, G′, *and* G″ *for mixtures measured at 10 Hz to the values measured at 0.1 Hz (0.01 percent strain, IT).*

temperature-shift factor, and strain-shift factor. The equation for complex modulus is given by

$$G^* = G_e^* + \frac{G_g^* - G_e^*}{\left[1 + (f_c/f')^k\right]^{m_e/k}} \tag{3}$$

where

$G_e^* = G^*\,(f \rightarrow 0)$, equilibrium complex modulus, $G_e^* = 0$ for binders and $G_e^* > 0$ for mixtures in shear;
$G_g^* = G^*\,(f \rightarrow \infty)$, glass complex modulus;
f_c = location parameter with dimensions of frequency;
f' = reduced frequency, function of both temperature and strain; and
k, m_e = shape parameters, dimensionless.

Figure 2.21 (a) illustrates the complex modulus master curve in Equation 2-3. It can be seen that G_g^* is the horizontal asymptote at $f \rightarrow \infty$, and G_e^* is the horizontal asymptote at $f \rightarrow 0$. The G_e^* asymptote is zero for binders. The third asymptote is the one with a slope of m_e.

The G_g^* and m_e asymptotes intercept at f_c. The G_e^* and m_e asymptotes intercept at

$$f_{c'} = f_c\left(\frac{G_e^*}{G_g^*}\right)^{1/m_e} \tag{4}$$

For binders, $f_{c'} = 0$.

The distance (one logarithmic decade being unity) between $G^*(f_c)$ and G_g^* for asphalt binders is given by

$$R = \log\frac{2^{m_e/k}}{1 + \left(2^{m_e/k} - 1\right)G_e^*/G_g^*} \tag{5}$$

For binders, $G_g^* = 0$, $R = m_e/k\log 2$.

The distance between $G^*(f_{c'})$ and G_e^* is given by

$$R' = \log\left\{1 + \left(\frac{G_g^*}{G_e^*} - 1\right)\left[1 + \left(\frac{G_g^*}{G_e^*}\right)^{k/m_e}\right]^{-m_e/k}\right\} \tag{6}$$

For binders, $R' = \log 2$.

The equation for phase angle of the model is as follows:

$$\delta = 90I - (90I - \delta_m)\left\{1 + \left[\frac{\log(f_d/f')}{R_d}\right]^2\right\}^{-m_d/2} \tag{7}$$

where

δ_m = phase-angle constant at f_d, the maximum value for asphalt mixtures, and the value at the inflexion for asphalt binders;
f' = reduced frequency;
f_d = location parameter with dimensions of frequency, at which δ_m occurs;
R_d, m_d = shape parameters; and

$$I = \begin{cases} 0 \text{ for mixtures} \\ \begin{cases} 0 \text{ if } f > f_d \\ 1 \text{ if } f \le f_d \end{cases} \text{ for binders} \end{cases}$$

Equation 2-7 satisfies the requirement that the phase angle should vary from 90 to 0 degrees when the frequency is increased from zero to infinity for asphalt binders as viscoelastic fluids. On the other hand, for asphalt mixtures in shear as viscoelastic solids, this equation satisfies the requirement that the phase angle increases from 0 to a peak value somewhere in the middle and returns to 0 degree when the frequency is increased from zero to infinity.

It is noted that, in the case of the asphalt binder, Equation 2-7 offers an inflexion at f_d in the relationship between logarithmic frequency and phase angle. Measurements on modified asphalt binder systems in this study (see the next section) and elsewhere do exhibit this trend (*54*). It is also noted that the parameters in Equation 2-7 are independent of those in Equation 2-3, thus allowing the diversity of material behavior. Using the same parameters for the phase angle and for the complex modulus implies that, for different materials, as long as complex moduli are the same, their phase angles will necessarily be the same under a given set of conditions, which might cover important differences among various materials between the extremes in the frequency domain.

The Williams–Landel–Ferry (WLF) formulation (*55*) is used in the model to express the temperature-shift factor in this study:

$$\log\frac{a_T(T)}{a_T(T_0)} = -\frac{c_1(T - T_0)}{c_2 + (T - T_0)} \tag{8}$$

where

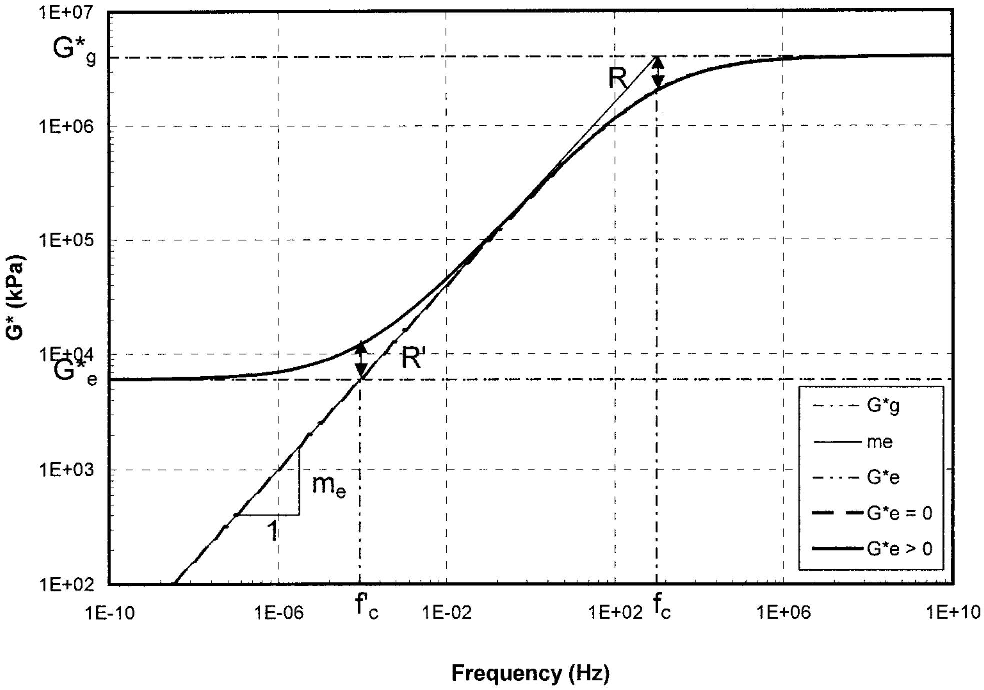
a_T = temperature-shift factor,
T_0 = reference temperature,
c_1 = constant, and
c_2 = temperature constant.

As for the strain dependency, the WLF equation is utilized to formulate the strain-shift factor, which has been indicated to be effective (*56*), as follows:

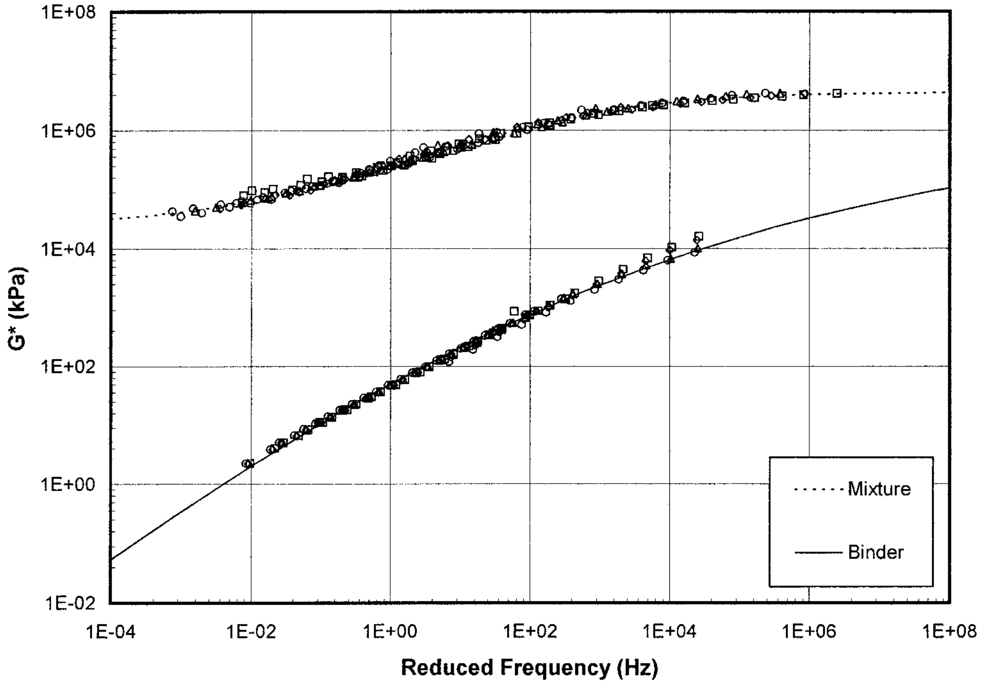
$$\log\frac{a_\gamma(\gamma)}{a_\gamma(\gamma_0)} = -\frac{d_1(\gamma - \gamma_0)}{d_2 + (\gamma - \gamma_0)} \tag{9}$$

where

a_γ = strain-shift factor,
γ_0 = reference strain,
d_1 = constant, and
d_2 = strain constant.



(a) Illustration.



(b) An example (PG 82 SBSr binder, limestone coarse aggregates).

Figure 2.21 Model for complex modulus

The reference temperature, T_0, and reference strain, γ_0, can be arbitrarily chosen for convenience. A reference temperature $T_0 = 52°C$ and a reference strain $\gamma_0 = 0$ are used in the analysis of the test data. Because the single complex or phase-angle curve accounts for both temperature and strain effects, it is termed "master–master curve" to differentiate it from traditional master curve that accounts for the temperature effect only. A spreadsheet program was used to calculate the model parameters and determine the shift factors. Figure 2.21(b) is an example of the complex modulus master–master curve for a binder and a mixture with the binder. Details of the modeling are found elsewhere (*57*). The following focuses on the complex modulus of the materials tested in this study.

Although the model developed is statistical (i.e., phenomenological), the parameters have important physical meaning:

- G_e^* is the equilibrium modulus representing the minimum modulus that a mixture can offer in shear. This asymptotic value is assumed to represent the ultimate interlock between aggregates when the contribution of the binder in a mixture is assumed to be negligible. It also represents the modulus at very low frequencies or HTs.
- G_g^* is the maximum asymptotic modulus in shear that represents response at very high frequencies or LTs, at which the binder in a mixture could contribute the most to the mixture modulus.
- k and m_e are shape parameters. The ratio of m_e to k multiplied by log2 gives a shape index called "*R*," which is shown in Figure 2.21 (a). This index is an indicator of the width of the relaxation spectrum. A higher value is an indication of more gradual transition from the elastic behavior to the viscous behavior. It indicates less sensitivity to frequency changes, generally lower G^* values, and higher phase angles within the intermediate range of frequency.
- The parameter f_c is a location parameter indicating the frequency at which the elastic component (G') is approximately equal to the viscous component (G''). A higher value of f_c is an indication of a higher phase angle and thus a greater overall viscous component in the behavior.

Figure 2.22 depicts three examples of the master curves determined for one of the aggregates, mixed with each of three binders. The model used and the model parameters are shown in the insert in the same figure. Initial analyses of the mixture rheological master curves revealed that they are significantly affected by aggregate, binder, temperature, frequency, and strain.

The objective of this analysis is to determine which of these parameters are mostly affected by the aggregate type binder properties and to identify the effects of binders and aggregates on the general and detailed response of asphalt mixtures. If this could be achieved, a better understanding of the role of binders in mixtures is possible.

Using this approach, the data for the 36 tested mixtures were analyzed; the results are shown in Table 2.9. Figures 2.23 and 2.24 compare m_e for the different binders mixed with the four different aggregates. The data presented in the table show

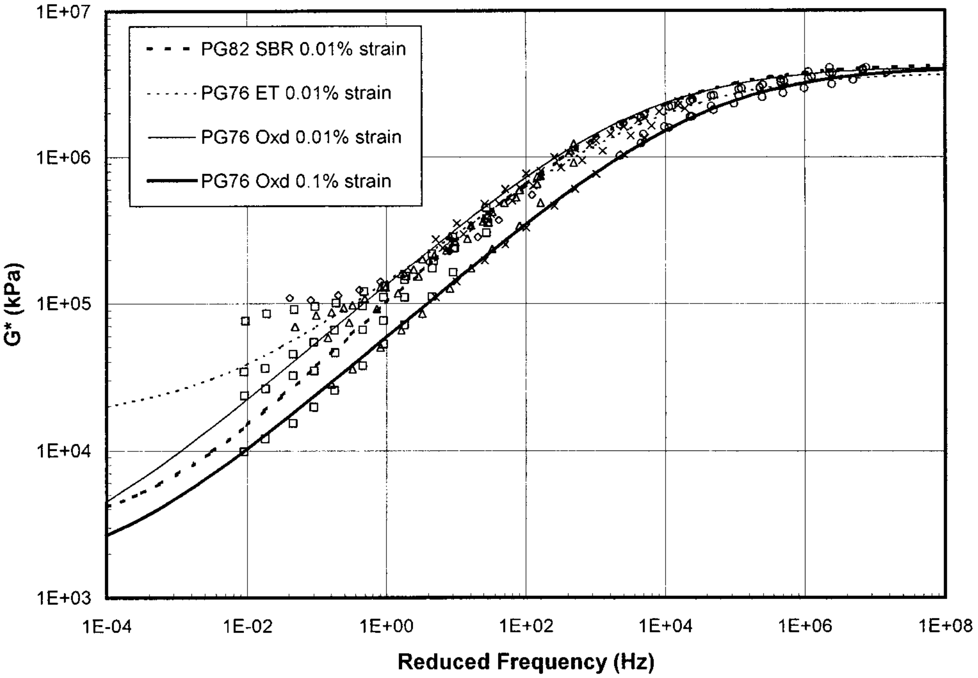


Figure 2.22 Master G* *curves for two binders at 0.01 percent strain and one binder at 0.01 and 0.10 percent strains (limestone coarse aggregate).*

TABLE 2.9 Model parameters estimated for the binders and mixtures tested

Binder	Aggregate	$G^* = G^*_e+(G^*_g-G^*_e)\{1+[f_c/(a_\gamma a_T f)]^k\}^{-me/k}$					R
		G^*_e (kPa)	G^*_g (kPa)	f_c (Hz)	k	m_e	
PG 82 SBSr	Binder	0.00E+00	1.00E+06	3.06E+02	0.105	1.000	2.878
	LS Coarse	2.58E+04	4.36E+06	1.38E+03	0.375	0.395	0.318
	LS Fine	6.41E+03	3.39E+06	3.26E+02	0.326	0.403	0.372
	GRVL Coarse	9.62E+03	3.94E+06	1.91E+03	0.414	0.411	0.299
	GRVL Fine	5.98E+03	4.57E+06	1.10E+03	0.276	0.463	0.505
PG 82 PEs	Binder	0.00E+00	1.00E+06	2.07E+02	0.112	1.000	2.694
	LS Coarse	1.46E+04	3.82E+06	1.90E+03	0.400	0.367	0.277
	LS Fine	3.18E+04	5.28E+06	4.48E+02	0.300	0.397	0.398
	GRVL Coarse	1.07E+04	3.89E+06	7.79E+02	0.381	0.442	0.349
	GRVL Fine	5.89E+03	4.51E+06	2.12E+03	0.369	0.426	0.348
PG 82 SD	Binder	0.00E+00	1.00E+06	4.18E+02	0.130	1.000	2.310
	LS Coarse	2.07E+04	4.68E+06	1.80E+02	0.348	0.527	0.455
	LS Fine	3.99E+04	4.29E+06	5.58E+01	0.324	0.534	0.497
	GRVL Coarse	3.60E+03	6.51E+06	1.73E+02	0.277	0.495	0.538
	GRVL Fine	4.75E+03	5.09E+06	1.66E+02	0.278	0.529	0.572
PG 82	Binder	0.00E+00	1.00E+06	3.99E+02	0.112	1.000	2.685
	LS Coarse	2.69E+03	4.33E+06	1.94E+03	0.364	0.464	0.383
	LS Fine	1.41E+03	3.94E+06	1.23E+03	0.391	0.411	0.316
	GRVL Coarse	4.69E+02	4.11E+06	4.09E+03	0.404	0.442	0.329
	GRVL Fine	2.68E+02	4.11E+06	4.30E+03	0.386	0.434	0.339
PG 76 ET	Binder	0.00E+00	1.00E+06	2.55E+02	0.103	1.000	2.921
	LS Coarse	1.59E+04	3.75E+06	3.61E+03	0.366	0.377	0.310
	LS Fine	7.85E+03	4.72E+06	3.41E+03	0.368	0.402	0.329
	GRVL Coarse	3.29E+03	3.86E+06	1.01E+04	0.444	0.395	0.268
	GRVL Fine	3.46E+03	3.98E+06	6.64E+03	0.345	0.421	0.368
PG 76 Oxd	Binder	0.00E+00	1.00E+06	2.58E+02	0.109	1.000	2.772
	LS Coarse	1.30E+03	4.14E+06	2.54E+03	0.382	0.408	0.322
	LS Fine	1.19E+03	4.95E+06	6.01E+02	0.289	0.438	0.457
	GRVL Coarse	6.98E+02	4.71E+06	8.58E+03	0.377	0.420	0.335
	GRVL Fine	9.53E+02	4.39E+06	3.84E+03	0.355	0.426	0.361
PG 58 SBSl	Binder	0.00E+00	1.00E+06	1.83E+02	0.077	1.000	3.903
	LS Coarse	1.35E+04	4.18E+06	4.50E+02	0.130	0.464	1.079
	LS Fine	1.48E+04	4.43E+06	7.28E+02	0.145	0.564	1.168
	GRVL Coarse	4.73E+03	4.50E+06	1.62E+01	0.130	0.753	1.745
	GRVL Fine	3.55E+03	4.44E+06	8.89E+05	0.172	0.364	0.637
PG 58 SB	Binder	0.00E+00	1.00E+06	9.64E+02	0.087	1.000	3.463
	LS Coarse	1.35E+04	4.18E+06	1.10E+02	0.154	0.667	1.302
	LS Fine	1.48E+04	4.43E+06	6.02E+02	0.176	0.583	0.999
	GRVL Coarse	4.73E+03	4.50E+06	3.02E+02	0.154	0.697	1.361
	GRVL Fine	3.55E+03	4.44E+06	8.27E+02	0.132	0.503	1.144
PG 58 Oxd	Binder	0.00E+00	1.00E+06	3.54E+02	0.081	1.000	3.718
	LS Coarse	1.35E+04	4.18E+06	4.23E+03	0.160	0.454	0.856
	LS Fine	1.48E+04	4.43E+06	1.74E+04	0.180	0.395	0.661
	GRVL Coarse	4.73E+03	4.50E+06	3.04E+05	0.212	0.376	0.534
	GRVL Fine	3.55E+03	4.44E+06	9.05E+05	0.214	0.324	0.455

that there is a significant binder-aggregate interaction effect on all model parameters.

The following are seen in the model parameters shown in Table 2.9 and Figures 2.23 and 2.24:

- There is a relatively significant interaction with aggregates characteristics. The coarse gradations show higher "*R*" values compared with the fine gradations for several of the binders. However, it is difficult to identify a specific trend that is related to the type of modification.
- The gravel aggregates tend to show higher values of the critical frequency "f_c" values compared with the limestone aggregates. The interaction between binders and aggregates is significant such that the binder f_c values are not readily reflected in the mixture f_c values.
- The glassy moduli G_g^* are generally not sensitive to binder type or aggregate characteristics. The values of the equilibrium modulus G_e^* show that mixtures vary within a wide range in this parameter. In general, the PG 82 grades result in a higher value of G_e^* for the mixtures. The trends

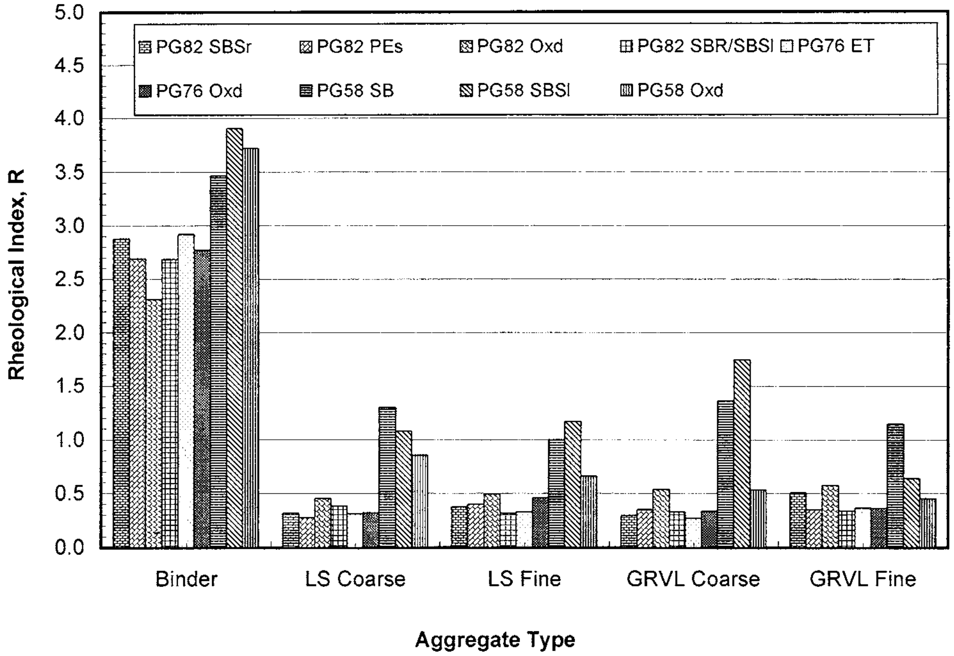


Figure 2.23 Effect of binder type and aggregate type on the value of the index R.

are, however, not consistent and it is difficult to identify a specific trend in terms of binder modification or aggregate characteristics.

- The analysis using the rheological model does not appear to be very useful in identifying the critical aspects of mixture behavior to which the binders contribute. It is apparent that the interaction between asphalts and aggregates is very complex and does not allow simple separation of the binder and aggregate effects.

The modeling, however, has shown that the temperature and strain effects can be assumed separable and can be accounted

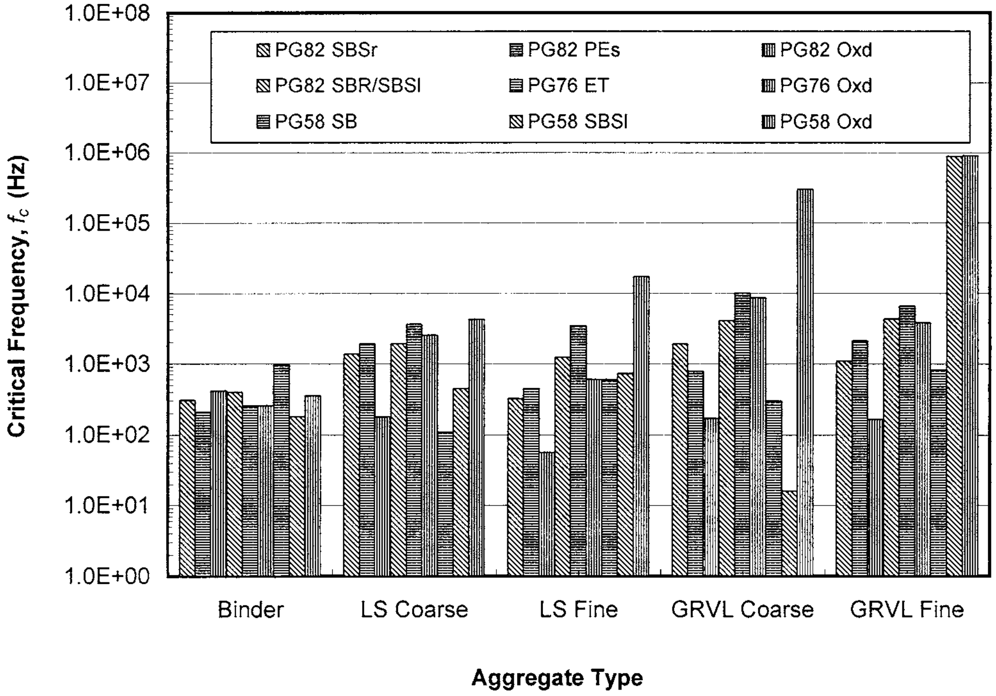


Figure 2.24 Effect of binder type and aggregate type on the value of the critical frequency f_c.