Summary Report

on

Asphalt Properties and Relationship to Pavement Performance

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Literature Review

(SHRP Task 1.4)

TM-UCB-A-003A-90-3
SR-ARE-A-003A-89-3

by

Fred N. Finn
Co-Principal Investigator

and

Margot T. Yapp
John S. Coplantz
Amer Z. Durrani

ARE Inc - Engineering Consultants
1 Victor Square, Suite G
Scotts Valley, California 95066

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SUMMARY REPORT

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ASPHALT PROPERTIES AND RELATIONSHIP TO PAVEMENT PERFORMANCE LITERATURE REVIEW

(SHRP Task 1.4)

Submitted to:

University of California - Berkeley
Institute of Transportation Studies
Asphalt Research Program
Richmond Field Station
Richmond, CA 94804

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This report has been prepared by ARE Inc., Engineering Consultants, as part of Project A-003A of the Strategic Highway Research Program (SHRP). SHRP is a unit of the National Research Council that was authorized by Section 128 of the Surface Transportation and Uniform Relocation Assistance Act of 1987. This project is entitled "Performance Related Testing & Measuring of Asphalt-Aggregate Interactions and Mixtures," and is being conducted by the Institute of Transportation Studies, University of California, Berkeley, with Carl L. Monismith as Principal Investigator. The support and encouragement of Dr. Ian Jamieson, SRHP Contract Manager, is gratefully acknowledged.

The draft of this report was reviewed by an Expert Task Group (ETG) and SHRP staff, who provided many valuable comments and will continue to provide guidance throughout the contract.

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1.0 INTRODUCTION

1.1 BACKGROUND

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

The highway community responded to the report with quick resolve and continuing commitment and action. Pre-implementation activities focused on the preparation of a final research program which culminated with the publication of the Strategic Highway Research Plans for each of the original six research areas (SHRP 1986). Part of SHRP's Asphalt Program is designed to investigate how the chemical and physical properties of asphalt binder relate to pavement performance. The results are expected to include improved characterization of materials, improved product testing methods, and improved construction procedures. Other parts of the asphalt program involve the development of accelerated tests for asphalt-aggregate mixtures to better predict the performance of the pavements. Improved understanding of asphalt-binder characteristics and the availability of new asphalt-aggregate tests would also permit the development of more performance-oriented specifications.

In order to accomplish these goals, a series of research contracts have been developed by SHRP to implement the research plans identified in the 1986 report. One such contract is SHRP Project A-003A entitled "Performance Related Testing and Measuring of Asphalt-Aggregate Interactions and Mixtures," of which this report forms a part.

SHRP Project A-003A seeks to:

1. Develop methods to analyze asphalt-aggregate interactions which significantly affect pavement performance,
2. Develop accelerated performance related tests for asphalt-aggregate systems that successfully model construction and service conditions, and

3. Develop a database derived from laboratory investigations that can be used to verify the asphalt chemical and physical characteristics significant to the performance of asphalt paving mixtures.

The project will focus on three of the many technical tasks which were identified in the 1986 Research Plan. These tasks include Research Plan (SHRP) Tasks 1.4, 2.2 and 2.3.

SHRP Task 1.4 (Relationships of Asphalt Chemical and Physical Properties to Pavement Performance) has two major objectives. The first objective is to assimilate information in the technical literature relating chemical and physical properties of asphalts to pavement performance and mixture properties and to accumulate test data for incorporation in the national data base. An effort will be made in this task (1.4) to evaluate and rank various asphalt properties as they affect pavement performance. Such evaluations will initially be based on information in the technical literature and, in later stages, will include additional information from unpublished information or on-going research including SHRP asphalt-related contracts. The second objective is to provide information which will be useful in planning and evaluating activities and results relevant to Task 2.2.

The major objective of SHRP Task 2.2 (Testing and Measuring for Asphalt-Aggregate Systems With and Without Asphalt Modification) will be to describe and standardize test methods for measuring those properties which characterize fatigue cracking, permanent deformation and low temperature cracking in asphalt-aggregate mixtures. A wide range of tests will be included in this part of the investigation including provisions for aging, moisture sensitivity, and temperature conditions representative of a range of field conditions.

The major objective of SHRP Task 2.3 (Relationship of Asphalt Chemical and Physical Properties to Asphalt-Aggregate Mixture Properties) will be to establish the relationship between asphalt chemical and physical properties, mixture properties and performance for a range of asphalts and aggregates including selected modified asphalts. An expanded test program with selected tests for fatigue, low temperature cracking, and permanent deformation will be included in this task.

1.2 OBJECTIVES AND SCOPE OF REPORT

This report documents the results of the initial literature review for SHRP Task 1.4. The objectives are to collect, summarize and analyze information in the technical literature (as of February 1990) which relates asphalt properties (both chemical and physical) to pavement performance. This review attempts to determine those properties of asphalt cement and asphalt concrete mixes which can significantly influence pavement performance with an emphasis on asphalt cement properties. The analysis consists of an examination and interpretation of available data, if required, to determine if any consensus exists in the literature regarding the above relationships. If possible, threshold values, regression or graphical relations are presented with as much specificity as needed. It is anticipated that the test methods developed by other project members will then be sensitive to the asphalt properties identified as important in this review. It should be noted that this report is not in final form; as more information becomes available, it will be reviewed and appropriate modifications will be made for the final report.

Both chemical and physical properties of the asphalt are considered pertinent to this review. The physical properties of greatest interest are generally penetration, viscosity, softening point, temperature susceptibility, binder stiffness, ductility and other rheological characteristics. The chemical properties include the chemical composition factors and functionality of asphalt. The final selection of specific physical and chemical properties is based on information provided by the literature.

The primary focus has been on information from test roads, particularly those where tests for asphalt cement properties were performed and the results related to pavement performance. Since the majority of such work and their results were published after 1950, the literature search generally concentrated on post-1950 publications and activities. However, any pertinent information of historical significance published before 1950 was also included. Asphalt modifiers have not been included for study in this report at this time.

1.3 ORGANIZATION OF REPORT

This report contains five chapters. The next chapter identifies the sources for the literature review and the general format of the review process. Chapter 2 also discusses the selection, sorting and prioritizing criteria used in the literature search to identify the most promising documents for careful review and analysis. Chapters 3 and 4 provide in-depth reviews of controlled and uncontrolled test road literature, respectively. Controlled test roads are planned experimental road trials which were constructed under controlled conditions. Uncontrolled road tests are those that were built with normal contract procedures, but where the performance was monitored and the asphalt properties were known or determined. Finally, Chapter 5 summarizes the results of the information reviewed to determine if any consensus exists in the published literature regarding the relationships between asphalt properties and pavement performance.

2.0 LITERATURE SEARCH

There exists a large body of information that is related to all aspects of pavement materials and performance. Much of the information has been accumulated during the past 40-50 years of research, particularly for studies relating to field performance. The information exists in both documented or published form as well as undocumented practice and experience. For this report, only published sources that relate asphalt properties to field performance have been examined. This chapter describes the sources of information examined and the selection criteria used to identify those sources which were candidates for more careful review.

2.1 SOURCES OF INFORMATION

A variety of sources were relied upon during the literature search. Library databases, previous literature reviews related to the subject, and inquiries to members of the research team resulted in an extensive list of references to be evaluated for review. Sources of information included the following:

1. TRIS (Transportation Research Information Services) - A library database produced by the Transportation Research Board which contains records and abstracts of published articles and reports in the transportation field.
2. COMPENDEX - A library database produced by Engineering Information Inc., which provides coverage of significant engineering and technological literature. Each record in COMPENDEX contains references to published articles and includes concise abstracts.
3. Melvyl - This database includes all items contained in the libraries of the University of California and California State University systems. Particular attention was given to references which are shelved at the Institute for Transportation Studies (ITS) Library on the University of California, Berkeley campus.
4. SHRP Research Plans: Final Report - This report contains 168 references on asphalt properties and proved to be an invaluable source of pertinent references.
5. Bibliography of Asphalt Research Studies, Volumes 1 and 2 - A list of references was compiled by the Asphalt Institute in July 1985 for a variety of subject headings related to asphalt technology. Many references were found to be pertinent to the investigation.
6. Previous summaries - State-of-the-art summaries prepared by other researchers (eg. Welborn 1979, Goodrich 1984, Petersen 1984, Halstead 1985) provided detailed, concise reviews of information in the literature existing prior to the completion of the reports.

7. Proceedings of the Association of Asphalt Paving Technologists (AAPT) - The titles of papers published after 1950 were reviewed to locate any important research that may not have been included in the above sources of information.
8. Proceedings of the International Conference on the Structural Design of Asphalt Pavements - Papers published from 1962 to the present were reviewed.
9. Transportation Research Board (TRB) Annual Meeting Program and Preprints - These documents for the 1989 and 1990 annual meetings were reviewed to include recent literature not yet published.
10. Inquiries to Research Team - Requests to other members of the research team provided information and findings of current ongoing projects as well as pertinent unpublished information. The Expert Task Group (ETG) and SHRP staff also suggested references for review.

The project team will continue to identify and collect pertinent sources of information as they appear, particularly as on-going research is completed.

2.2 SELECTION PROCESS

During the early stages of the literature review, criteria were established to identify those sources which specifically related to the project from the large amount of available information. References were arranged into one of three categories (Figure 2.1) depending on their relevance to the project.

Category 1 contains all of the references collected from each of the sources discussed in Section 2.1. These items resulted from key word searches of the computer databases and title searches of the other sources. This listing has grown to over 700 documents (as of February 1990), and it was used as a master list to develop the Category 2 and Category 3 reference lists.

Category 1 references passed to Category 2 if they had potential usefulness to the A-003A project. The following acceptance criteria controlled this selection:

1. The reference included information on pavement performance; specifically in the areas of fatigue cracking, rutting, low temperature cracking, aging and water sensitivity.
2. The reference included information relating asphalt cement properties (physical or chemical) to pavement performance.
3. Generally, references published after 1950.

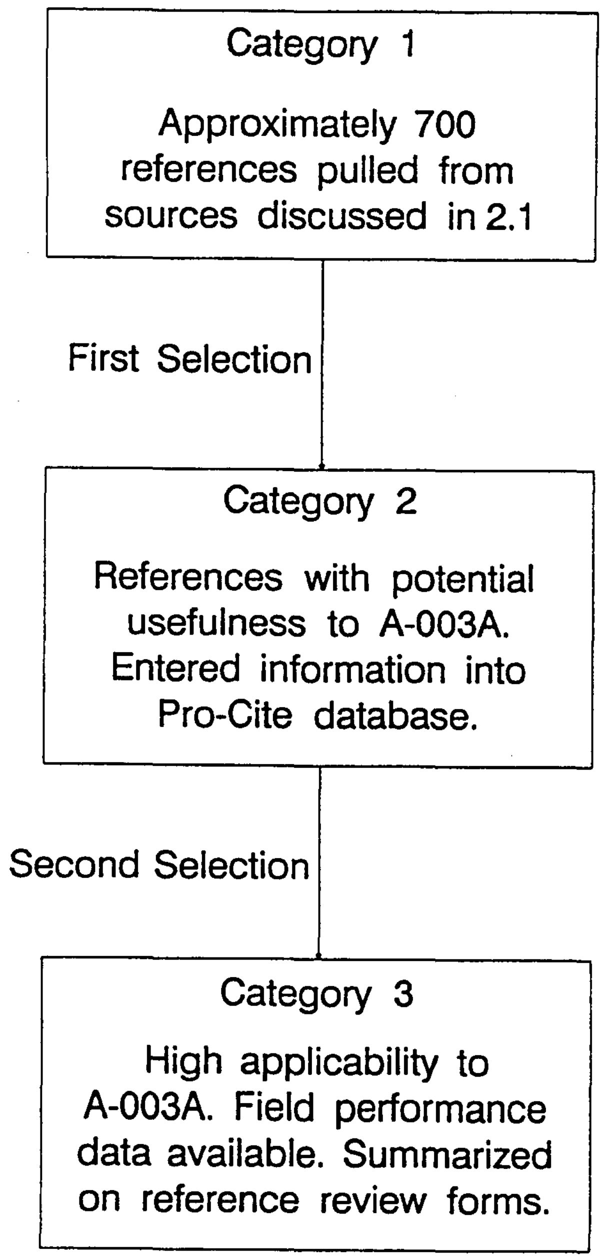

Figure 2.1 Selection criteria used for SHRP A-003A

4. Reference information specifically identified by the other project members was automatically classified as Category 2.

All Category 2 references were entered into Pro-Cite, a microcomputer-based bibliographic database. Then, the abstract, introduction and conclusions of all Category 2 references were examined. Two main criteria were used to determine which references should be subject to detailed review (Category 3). Those criteria included "Applicability to A-003A" and "Type of performance data." To be applicable to A-003A, the reference must have contained information that explicitly relates properties of asphalt to one or more of the five performance parameters; fatigue cracking, rutting, low temperature cracking, aging, or water sensitivity. Field and laboratory performance data on asphalt pavements or asphalt mixtures as related to properties of asphalt cement were considered most valuable. In particular, references describing test roads or field test sections were considered to be of highest priority. References in which asphalt properties were related to field performance data only were also of prime importance. References which did not include or make reference to field performance data were given a lower priority.

2.3 REVIEW FORMAT

The Category 3 references were reviewed and summarized on the Reference Review Form which is reproduced as Figure 2.2. The source, research objectives, asphalt properties measured, performance data (both laboratory and test road), conclusions, applicability to A-003A, and other comments were recorded on the forms and entered into Pro-Cite. This format allowed quick summarization of the reference to determine its relevance to the objectives of the A-003A investigation and also served as a starting point for in-depth review of the references.

2.4 PRIORITIZING CRITERIA

Only a limited number of the Category 3 references could be reviewed in great detail. A prioritization matrix was developed to insure that the references providing the best information to meet the objects of this project were subject to this careful review. The matrix is shown in Figure 2.3 with the lowest numbers of the cells corresponding to the highest priority. The prioritizing criteria included applicability to A-003A and type of performance data. A description of these criteria follows.

<u>Applicability to A-003A</u>

There were two categories for this criterion; "Applicable," and "Requires further evaluation." A review of the abstract, the conclusions, or possibly the report itself was necessary to confirm the

File No: ________

SHRP A-003A: Reference Review Form

Title: ______________________________

____

Authors: ______________________________

Source/Agency: ____________ Report/Journal: ____________

Date Published: ________ Report/Journal No: ____ Page No. ____

Research Objectives: ______________________________

____

A. Asphalt Properties Measured

Penetration ________ Durability Tests ________
Ductility ________ Stiffness ________
Viscosity ________ Chemical Properties ________
Softening Point ________ Others ________
Other Rheological Properties ________

B. Controlled Test Road ________ Uncontrolled ________ Lab ____

C. Pavement Performance

Fatigue ________ Rutting ________
Low Temperature ________ Aging ________
Water Sensitivity ________ Others ________

Overall Conclusions: ______________________________

____

____

Applicability to A-003A: ______________________________

____

Other Comments: ______________________________

____

____

Keywords: ______________________________

Ranking

Reviewer: ____________ Date Reviewed: ______ Page 1 of 2

Abstract:

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Figure 2.2 Reference review form for SHRP A-003A

	Applicability to A-003A:	
	Applicable	Requires further evaluation
Type of Performance Data: Field and Lab	1	3
Type of Performance Data: Field Only	2	4
Type of Performance Data: Lab Only	5	6

NOTE: Numbers in cells correspond to sorting code.

Figure 2.3 Sorting Matrix for Prioritizing Category 3 References applicability identified during the Category 2 review. If this information was readily determined from a quick review, the report was assigned as "Applicable." If applicability could not be determined without further review, it was categorized as "Requires further evaluation." In some cases, references with potential applicability in Category 2 were not confirmed and the references were not assigned a sorting code.

Type of Performance Data

This criterion had three categories; field and laboratory data, field data only, and laboratory data only. Laboratory testing of asphalt properties was included in all Category 3 reports regardless of the type of performance data reported. Examples of field data include but are not limited to fatigue or low temperature cracking observations, rut depth observations, or laboratory tests on field samples to determine aging or water sensitivity. Examples of laboratory data include performance test results on the asphalts or on asphalt mixtures prepared in the laboratory, such as creep or fatigue tests. Reports with both field performance observations together with laboratory performance data were assigned highest priority for this criterion. Reports with only field performance observations were ranked next, followed by those containing laboratory performance data only.

3.0 CONTROLLED TEST ROADS

Controlled test roads are planned experimental road trials using asphalts of different types and sources and constructed under controlled conditions. This definition is derived from Welborn (1979). These road tests are important since they provide field performance data as well as detailed asphalt properties obtained from laboratory testing. Test road projects considered most relevant to this investigation have been included in this report for review and analysis.

This chapter presents the results of the literature review (as of February 1990) on controlled test roads. For each test road, a description is provided, together with the asphalt properties measured and pavement performance results. The results, conclusions and relationships derived are also presented. Appendix A contains additional tables and figures which provide more details and which may be of interest to the researcher.

3.1 ZACA-WIGMORE

Description

The Zaca-Wigmore Asphalt Test Road (Hveem et al. 1959, Zube and Skog 1969, Skog 1959, 1981) was constructed in 1954-1955 in California near Santa Barbara to test new asphalt specifications developed by the then California Division of Highways. A series of test sections were built as part of a state highway contract. Ten asphalts in the 200-300 penetration grade were selected, of which 6 met the new specifications, 2 met the previous specifications and 2 asphalts which were not commercially available in California were also included to aid in the evaluation. The main difference in the new specifications was a change in flash point requirements and a reduction in weight loss and a higher retained percentage of original penetration after the standard loss on heat test. The test method for the flash point was changed from Cleveland Open Cup to the Pensky-Martens Closed Cup test. However, it is not clear from the studies if the new specifications were accepted, as current specifications use the Cleveland Open Cup. With one exception, all the asphalts represented different crude sources and methods of production. The asphalts were produced from three major areas broadly described as the Los Angeles Basin, the Kern Basin at the southern end of the San Joaquin Valley, and the Santa Maria field along the Coast. The exception was a mid-continent crude source asphalt refined in Arkansas (Smackover). Based on a recommended asphalt content range of 5.6 to 6%, the actual contents used were 5.5%, 5.6% and 5.8%. Asphalt I-2 from Arkansas was assigned an asphalt content of 5.8 and 6.3%. The physical properties of the asphalts used are summarized in Table A.1 of Appendix A.

The anticipated 10-year 5000-lb equivalent wheel load was 10.7 million. The project consisted of converting an existing two-lane highway into a four-lane divided expressway by constructing 2 new lanes and widening the existing pavement. The structural sections for both alignments are shown in Appendix A as Figure A.1. Construction was carried out in 3 periods between October 1954 to March 1955. The last construction period, from February to March 1955, was sufficiently different (rain storms) to conclude that the test sections built at this time could not be compared with the other 2 periods. Therefore, sections constructed during the February to March period and during the October to February period were analyzed as two separate projects.

The pavements were evaluated at various intervals for approximately 10 years using deflection measurements (Benkelman Beam) and cores as well as visual observation and crack surveys. The quantity of transverse, longitudinal and alligator cracks were measured in both travel and passing lanes. In addition, other distress types were noted including raveling and block cracking. The limiting performance criterion was set at 10% of the surface showing alligator cracking in the travel lane. Changes in the original properties of the asphalts during mixing and throughout pavement service life were also studied.

Results

Ten sections showed some amount of distress between 38 and 92 months of service with seven sections remaining in satisfactory (less than 10% cracking) condition after 9 years. All of the latter performance were from sections on old concrete. An interesting observation is that there is a rapid increase in alligator cracking after a certain period of time (Figures 3.1 and 3.2). This occurred after 1-3% of the travel lane area was affected. Prior to resurfacing the road in 1964, a number of badly alligator-cracked pavement areas were removed and it was observed that the fatigue cracking started at the bottom of the leveling course and progressed upward through the mix.

A straight line can be used to approximate the relationship between the amount of cracking and pavement age. This allows extrapolation of the curves to any service age. Using this method the percentage of cracking at 97 months of service life was determined. The penetration of the recovered asphalt at this time was also determined from cores. The results (Figure 3.3 and Table 3.1) indicate that as the percent of fatigue cracking increases, the penetration of the recovered asphalt decreases. (NB: The "Pave.Per" column in Table 3.1 refers to the Paving Period of each section.) For the Zaca-Wigmore project, the results indicated that a penetration of 30 is the approximate critical consistency for the failure criterion adopted. A similar correlation with viscosity at 77°F and 0.05 sec^{-1} shear rate was also found with a viscosity of approximately 20 megaPoises occurring at the failure criterion.

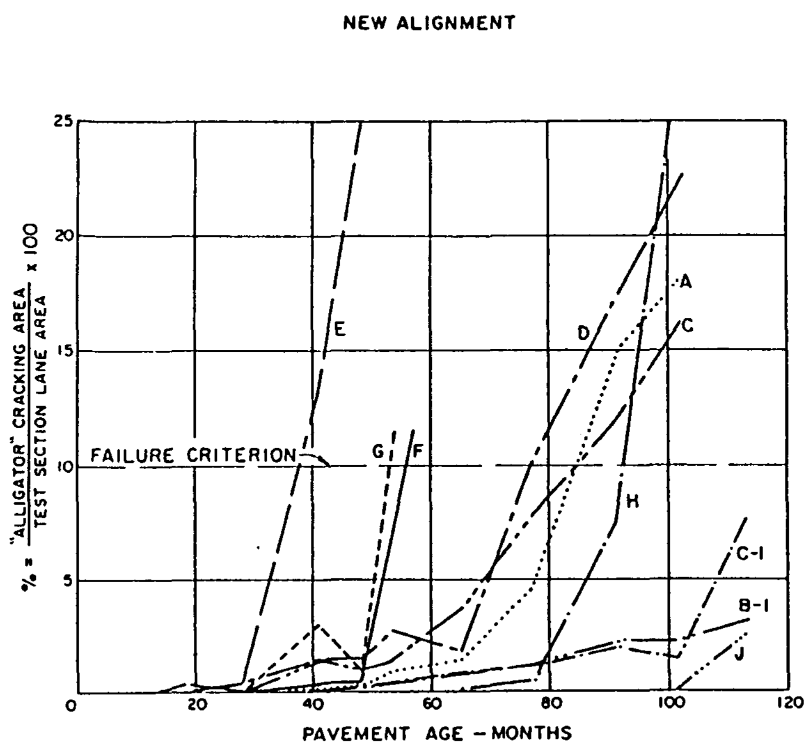


Figure 3.1 Alligator type cracking in the travel lane during service life for Zaca-Wigmore (new alignment) (Zube & Skog 1969)

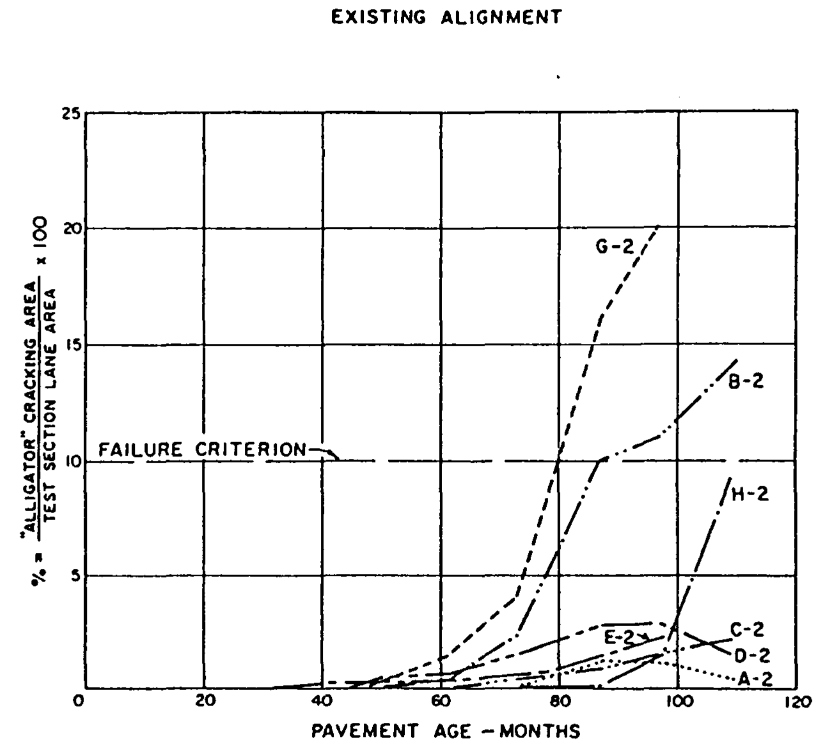


Figure 3.2 Alligator type cracking in the travel lane during service life for Zaca-Wigmore (existing alignment) (Zube & Skog 1969)

Table 3.1 Comparison of consistency of recovered asphalts and percentage of alligator cracking after 97 months of service life for Zaca-Wigmore (Zube & Skog 1969)

Asphalt	Pave. Per.	% "Alligator" Cracking Tr. Lane	Pen. 77°F	Visc. 77°F S.R.=.05Sec^{-1}
A	I	19	24	33
C	I	17	25	20
D	I	19	29	22
E	I	100	4	-
F	I	100	17	33
G	I	100	11	50
H	I	15	25	32
J	I	0	40	10
B-1	IA	2.2	76	2
C-1	IB	1.7	38	14
B-2	2	11	33	18
C-2	2	1.5	43	6
D-2	2	2.9	47	6
G-2	2	26	22	55
H-2	2	1.4	53	5
I-2 5.8%	2	-	29	23
I-2 6.3%	2	-	46	10

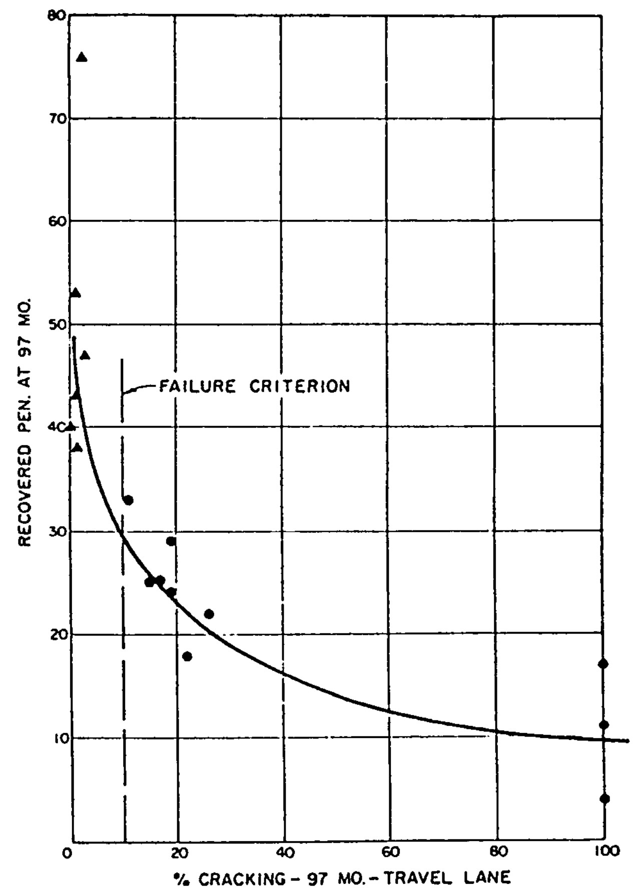


Figure 3.3 Comparison of consistency of recovered asphalt and percentage of alligator type cracking after 97 months of service life for Zaca-Wigmore (Zube & Skog 1969)

Hardening with age as measured by the penetration test is shown in Figure 3.4. The curves show a rapid increase in hardening (decrease in penetration) during the first 16 to 20 months, and a decrease in the hardening rate thereafter. In the case of asphalt I-2 where two asphalt contents (5.8% and 6.3%) were used, it was possible to evaluate the effect of asphalt content and air voids (Figures 3.5a and b). For Figure 3.5a, the lower asphalt content shows a lower penetration than for the higher asphalt content. The rate of hardening is also different after about 11 months of service.

Durability is generally defined in terms of changes in physical (rheological) properties of asphalt. An asphalt with good durability would exhibit a high resistance to change in such properties as penetration, ductility, viscosity, and temperature susceptibility. The durability of asphalts used at the Zaca Wigmore project were measured by the Shell microviscometer tests on the original and aged asphalts (Simpson et al. 1959). The results, for those asphalts tested, both from laboratory and field aged conditions, are shown in Figures 3.6a and 3.6b. The microfilm durability test was used to artificially age the bulk asphalt in the laboratory. The authors state that it is not the purpose of the microfilm durability test to predict the viscosity which an asphalt will attain in a road within a certain time as this will depend on many factors such as the mix design, compaction and climate. However, the microfilm durability test does predict the relative rate at which a number of asphalts will harden in the road if all are used under the same conditions.

Other general conclusions reported by Simpson et al. also indicated that asphalt viscosity decreased with increasing depth in the pavement indicating that hardening of the asphalt is greatest at the top and decreases with increasing depth in the pavement. In general, the asphalt in the top 0.25 inch has a higher viscosity than the rest of the pavement. It was also noted that the general level of viscosity of the recovered asphalt is related to the air void content in the compacted mix. Figure 3.7 shows the results of two cores with an age of 32 months taken from the same pavement section but with different air voids. At the surface and at a depth of 3+ inches, the effect of air voids on viscosity is not clear; however, between 1 inch and 3 inches from the surface there is a difference of 9:1 to 2:1 in viscosity in the range of 1 megapoise to 10 megapoises at 77°F. The high void contents would be expected to lead to more rapid hardening in use.

Davis and Petersen (1967) used inverse gas liquid chromatography (IGLC) to study the asphalts from the Zaca-Wigmore test road. The principle of the IGLC technique is to measure the retention behavior of selected test compounds that possess different functional groups such as phenol. Retention behavior (as described by Davis and Petersen) is a measure of functional group interactions between the test compound and the asphalt, and is thus related to the chemical composition of the asphalt. Studies were made to determine the relationship between the IGLC test

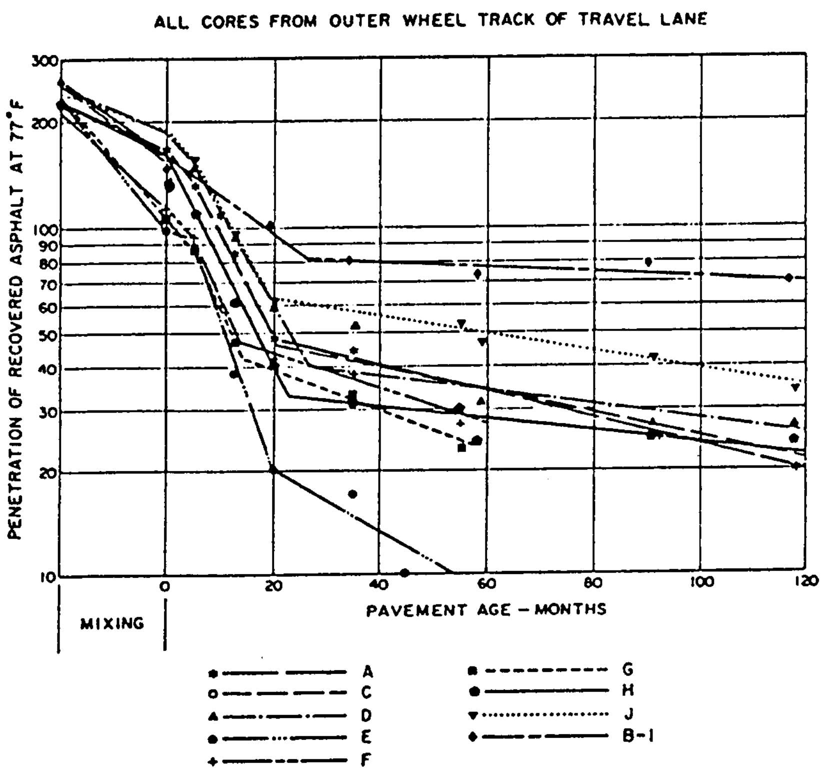


Figure 3.4 Hardening of 200-300 paving grade asphalt for Zaca-Wigmore, period 1 and 1A paving (Zube & Skog 1969)

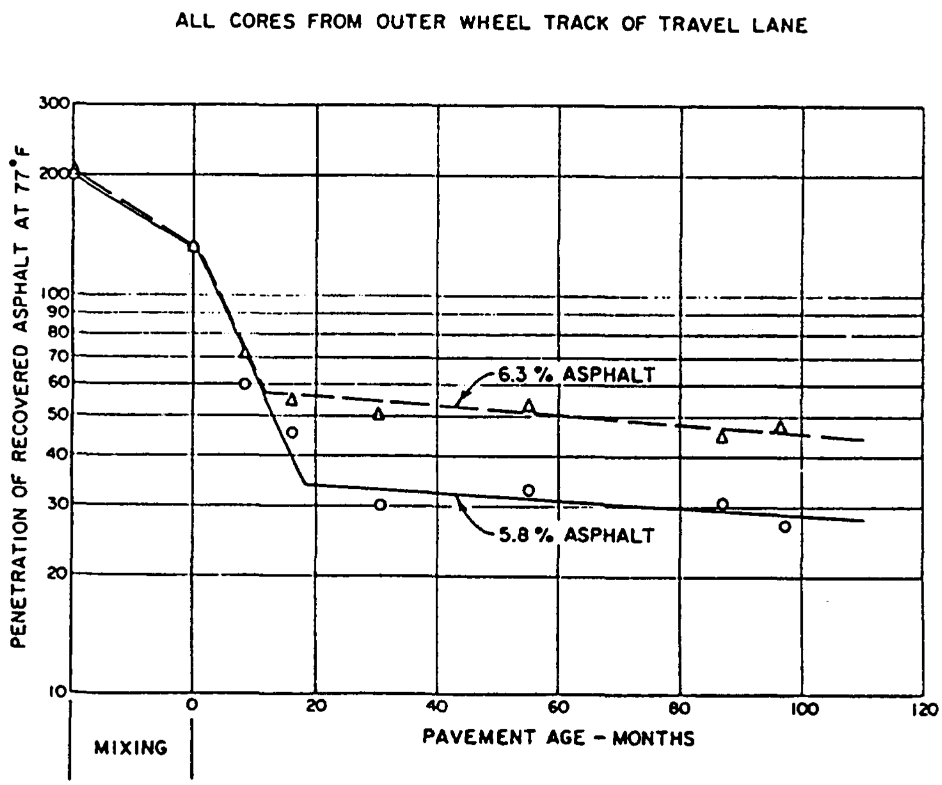


Figure 3.5a Change in penetration during mixing and service life for Zaca-Wigmore, asphalt I-2, period 2 paving (Zube & Skog, 1969)

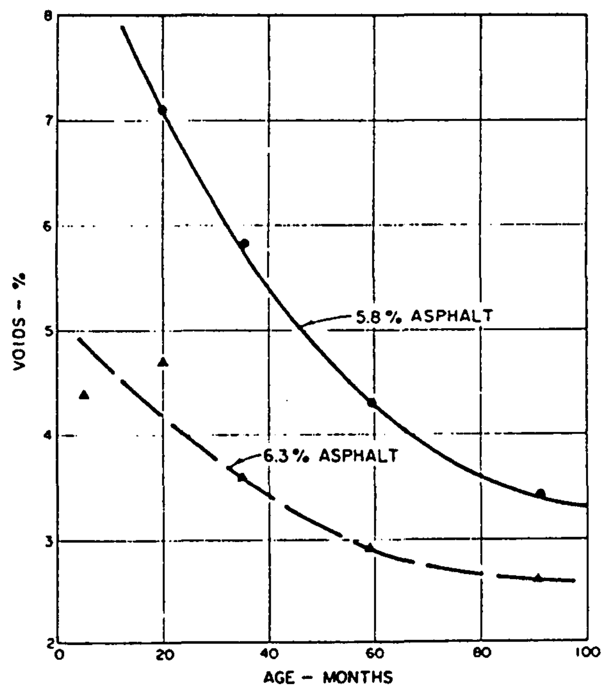


Figure 3.5b Change in void content during service life for test section containing 5.8% and 6.3% of I-2 asphalt at Zaca-Wigmore (Zube & Skog 1979)

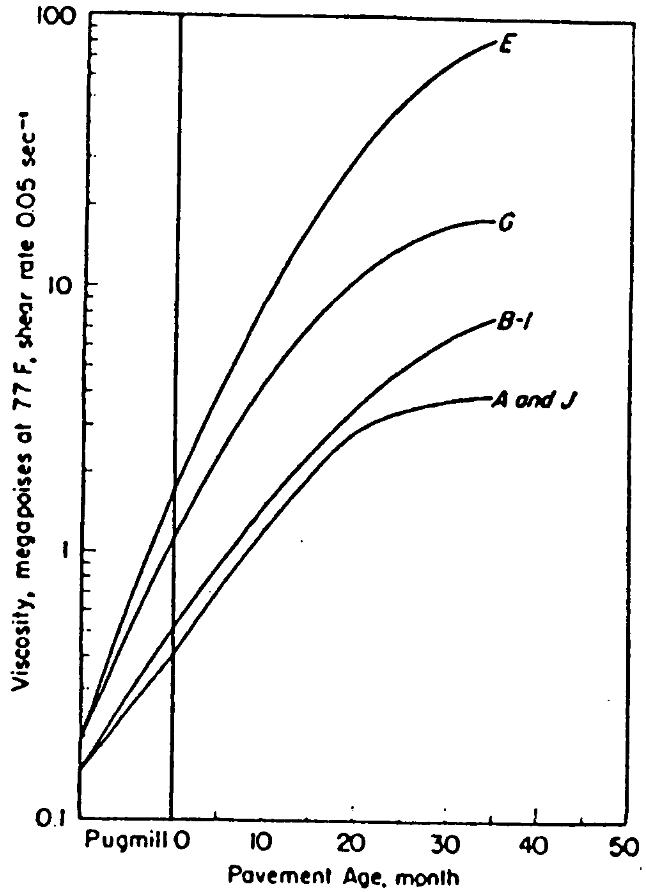


Figure 3.6a Increase of viscosity with time on the Zaca-Wigmore Road Test (Simpson et al. 1954)

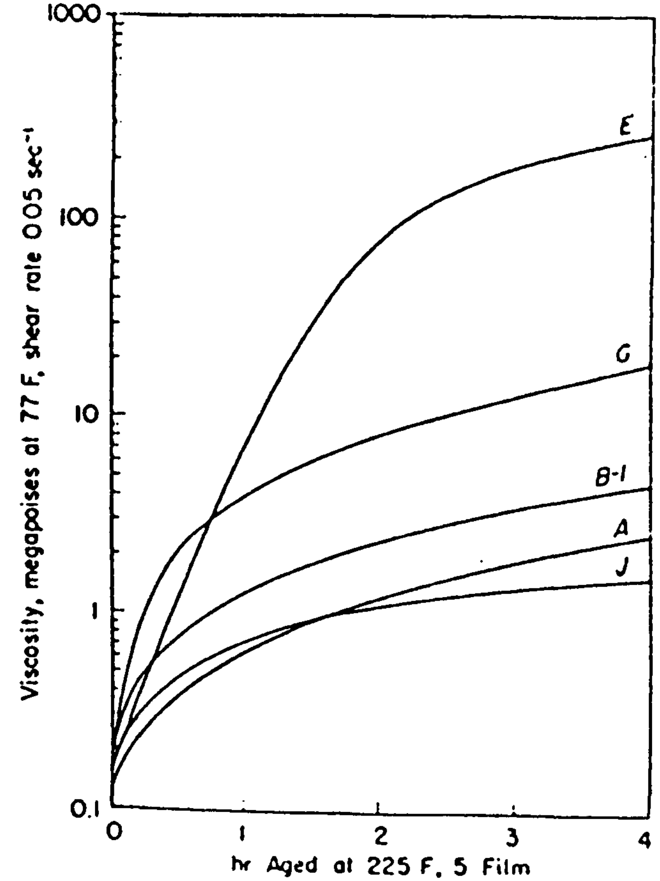


Figure 3.6b Increase of viscosity with time in microfilm durability test for the Zaca-Wigmore Road Test (Simpson et al. 1959)

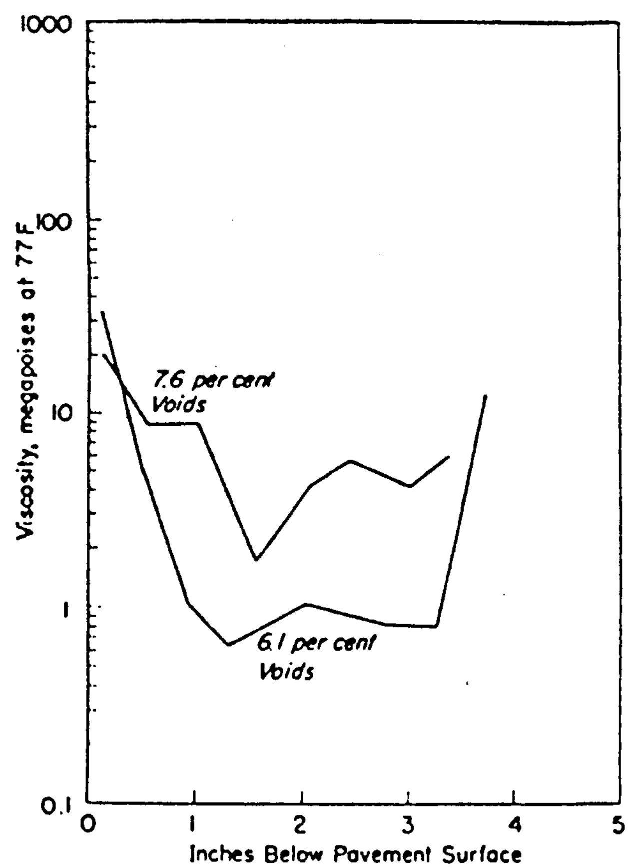


Figure 3.7 Influence of air void content on asphalt hardening for Zaca-Wigmore (Simpson et al. 1959)

results on the weathered asphalts and their in-service performance and the changes in the asphalts during the laboratory microfilm durability test. A good correlation was found between service life (as defined by the 10% cracking criterion) and the phenol interaction coefficient (I_p) as shown in Figure 3.8. (This figure is an update of that shown in the 1967 paper by Davis & Petersen, and was reported in the discussion following Zube & Skog's 1969 report. Both Test Sections B-1 and J had not failed as of 1969.) The phenol interaction coefficient is a measure of asphalt polarity. The asphalts that developed greater amounts of strongly interacting polar groups during aging had a shorter service life. Results from Paving Period 2 were not plotted as the majority had not failed after 113 months of service. Although a good correlation is shown between I_p and service life, Dr. Petersen cautions that these data may not necessarily match with performance data in other road tests. This may be due to variations in engineering and environmental variables, as well as different asphalts. There exists asphalts whose phenol I_p's on oxidation are not a good indicator because of a poor balance or incompatibility of molecular components. For the Zaca-Wigmore asphalts, it was found that an I_p less than 145 resulted in a service life of greater than 100 months.

In a later study, Schmidt and Santucci (1969) reported that three alternative microfilm durability tests, the Thin Film Plate Durability Tests (TFP), the Rolling Microfilm (RMF) and the Rolling Microfilm on Original Asphalt (RMFO) all correlate equally well with the pavement life obtained at the Zaca-Wigmore test road. These three tests are modifications of the Thin Film Durability Test (TFP) used experimentally by the California Division of Highways. The first modification measures the microviscosity at a constant stress level instead of constant shear rate. The other modifications expose the asphalt as a microfilm in the interior of a bottle instead of a glass plate. The authors state that although statistically the correlations are not significantly better than those of the Thin Film Oven (TFO) or the Rolling Thin Film (RTF) oven exposures, their tests are quite different in their ease of determination and precision. According to the authors, the RMFO test is the most convenient, economical and most reliable of these microfilm tests. Figure 3.9 illustrates the correlation for the RMFO residua viscosities with the Zaca-Wigmore pavement life. Note that Methods I and II refer to the method used to estimate the failure time of each test section. In Method I, failure is taken at the time where each best-fit line intercepts the 10% cracking level. Method II curves were drawn with slopes that are the average of all the slopes from Method I. Using Method I, a viscosity of 20 megaPoises (at 77°F, 167 g/cm^2 on RMFO residue) would result in a service life of approximately 80 months for Period I paving. The two curves in each graph relate to the two different paving periods.

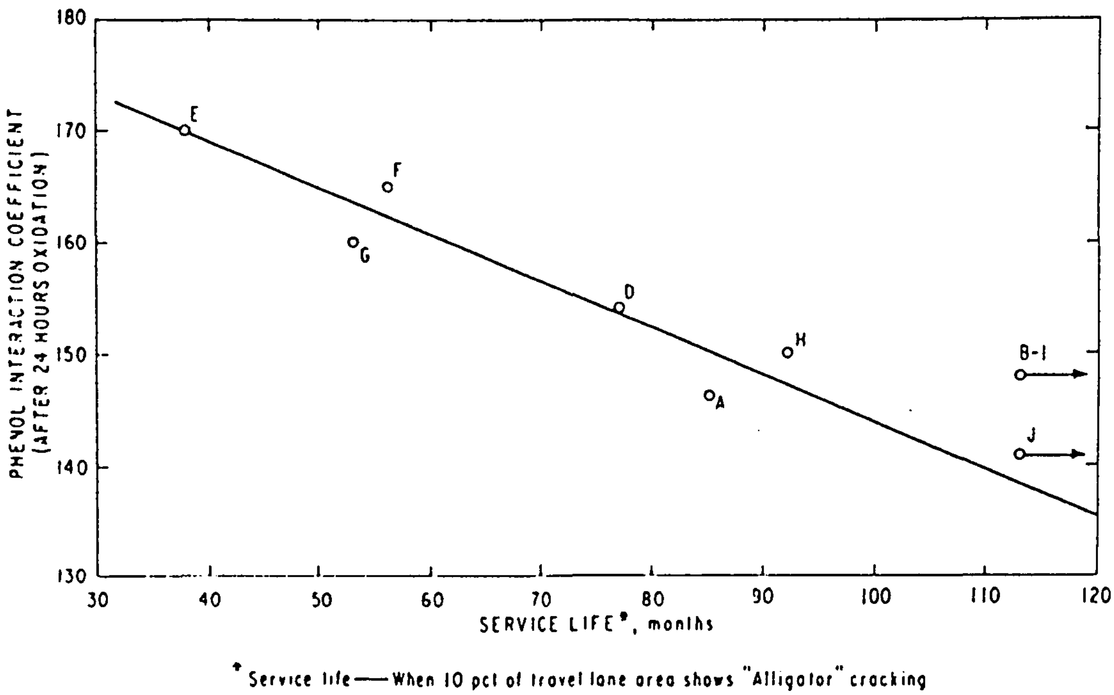


Figure 3.8 Relationship between phenol interaction coefficient and service life - paving periods 1 and 1A at Zaca-Wigmore (Petersen's Discussion, Zube & Skog 1969)

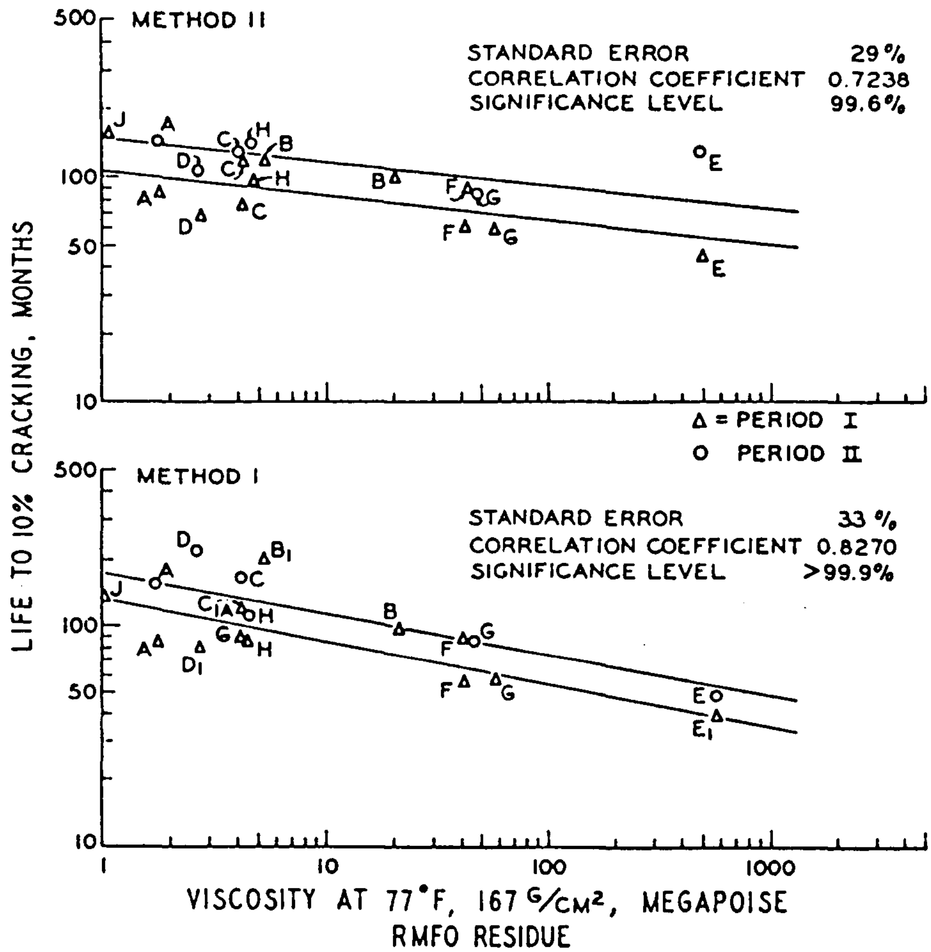


Figure 3.9 Correlation of Zaca-Wigmore pavement life with rolling microfilm original (RMFO) residua viscosities (Schmidt & Santucci 1969)

Conclusions

It should be noted that these results were obtained for pavement structures of 4-inch AC over a cement treated base and a 4-inch AC overlay over existing PCC pavement. From this review of the studies on the Zaca-Wigmore test road, the following conclusions were made.

1. There is a rapid increase in alligator cracking after a certain period of time. This occurs after 1 to 3% of the travel lane area has been affected by cracking.
2. The occurrence of fatigue (alligator) cracking was inversely related to the recovered penetration of the asphalt. The limiting value of penetration related to 10% fatigue cracking was 30. For viscosity at 77°F and 0.05 sec^{-1} shear rate, the limiting value was approximately 20 megaPoises.
3. There is a rapid increase in hardening the first 16 to 20 months and a decrease in hardening rate thereafter.
4. The Shell Modified Microfilm Durability test correlated well with changes in viscosity of recovered asphalt used on the Zaca-Wigmore project.
5. In general, higher air void contents tend to result in higher viscosities.
6. As the phenol interaction coefficient of an asphalt increases, the service life (10% of cracking) of the pavement decreases. At I_p's of less than 145, service life is greater than 100 months.
7. The Thin Film Plate Durability Test (TFP), the Rolling Microfilm (RMF) and the Rolling Microfilm on Original Asphalt (RMFO) correlate well with pavement life.
8. It is apparent that the different asphalt sources exhibited different performances, although it is not known which asphalts were used to construct a particular test section.

3.2 STE. ANNE

Description

The Ste. Anne Test Road, constructed in Manitoba in 1967, was built to study low temperature cracking of asphalt pavements (Burgess et al. 1971, 1972; Gaw et al. 1974; Deme & Young 1987). The study also included laboratory tests to examine the theoretical aspects of low temperature cracking and made comparisons with field performance.

The test road was all new construction and contained twenty-nine 400 foot pavement sections, 24 feet wide. One-half of the road was constructed on a heavy clay subgrade and the other half on a sand subgrade. The test location experienced some 3000 to 4000 degree days of freezing and ambient temperatures ranged from 100°F to -45°F. Annual precipitation is approximately 20 inches.

The road was new construction and carried an ADT of 1,250 of which some 10 percent was classified as truck traffic. Instrumentation was installed to measure temperatures within the road structure and to provide information on the initiation of transverse cracking. Cracking frequency and crack pattern data were obtained from visual surveys. It should be noted that the test road was designed for early failure.

Four different binders were used on the project; an SC-5 liquid asphalt made from a high viscosity base asphalt, a 150-200 penetration LVA (low viscosity asphalt with a relatively high temperature susceptibility, PI = -2.7), a 300-400 penetration LVA (PI = -2.9) refined from the same crude source, and a 150-200 penetration HVA (high viscosity asphalt with a relatively low temperature susceptibility, PI = -1.4). The low viscosity asphalts had waxy characteristics while the high viscosity asphalts did not (Gaw et al. 1974). It should be noted that the 150-200 HVA and the 300-400 LVA have similar viscosities at 39.2°F. Properties of the binders are shown in Table A.2. Three different asphalt concrete mixes were used:

(a) A blend of 80% limestone and 20% igneous material having 50% crushed particles retained on the No. 4 sieve, with 2.5% of the aggregate passing the No. 200 sieve,

(b) The same blend of 80% limestone and 20% igneous material, with approximately 3.0% additional cement filler, and

(c) 100% igneous aggregate with all sizes crushed and with 5.5% passing the No. 200 sieve.

An experimental factorial showing the various combinations of factors used for the test sections is shown in Figure 3.10. According to the factorial, each of the binders were from Western Canadian Crudes, although it is presumed that the sources may have varied between binders. Three different asphalt contents were used; Marshall optimum, 1.0% below optimum, and 0.5% above optimum. Three different structural sections were studied; asphalt concrete with granular base over both sand and clay subgrade, and full depth asphalt concrete over clay subgrade.

Results

Field observations during the winter of 1967-1968 (Burgess et al. 1971) indicated that transverse cracks were initiated at the surface of the pavement and propagated downward. The pavement surface temperature was lowest when the atmospheric temperature was close to the minimum for the day concerned. Most of the cracks did not penetrate into the binder course but were limited to the colder and stiffer surface courses. The pavements containing the 150-200 LVA cracked at a pavement surface temperature of approximately -34°C, whereas the pavement containing 300-400

	150-200 PENETRATION GRADE LOW VISCOSITY ASPHALT (WESTERN CANADIAN CRUDE)					150-200 PENETRATION GRADE HIGH VISCOSITY ASPHALT (WESTERN CANADIAN CRUDE)					300-400 PENETRATION GRADE LOW VISCOSITY ASPHALT (WESTERN CANADIAN CRUDE)				SC-5 ASPH. (W.C. CRUDE)
ROAD STRUCTURE	BELOW OPTIMUM ASPHALT CONTENT	BELOW OPTIMUM ASPHALT, CEMENT FILLER	OPTIMUM ASPHALT CONTENT	OPTIMUM ASPHALT CONTENT, CEMENT FILLER	ABOVE OPTIMUM ASPHALT CONTENT	BELOW OPTIMUM ASPHALT CONTENT	BELOW OPTIMUM ASPHALT, CEMENT FILLER	OPTIMUM ASPHALT CONTENT	OPTIMUM ASPHALT CONTENT, CEMENT FILLER	OPTIMUM ASPHALT CONTENT, 100% CRUSH IGNEOUS AGGREGATE	BELOW OPTIMUM ASPHALT CONTENT	BELOW OPTIMUM ASPHALT, CEMENT FILLER	OPTIMUM ASPHALT CONTENT	OPTIMUM ASPHALT CONTENT, CEMENT FILLER	OPTIMUM ASPHALT CONTENT
4 IN. PAVEMENT • 16 IN. BASE COURSE CLAY SUBGRADE	X	X	X	X		X		X	X	X	X	X	X	X	X
4 IN. PAVEMENT • 6 IN. BASE COURSE SAND SUBGRADE	X	X	X	X	X	X		X	X	X	X	X	X	X	X
10 IN. FULL DEPTH • ASPHALT PAVEMENT CLAY SUBGRADE			X					X							

• All aggregates in bituminous pavement mix processed from glacial drift deposits (20% igneous, 80% limestone; 50% crush) unless otherwise indicated.

Figure 3.10 Ste. Anne test road test section design variables (Burgess et al. 1972)

LVA on the clay subgrade cracked at approximately -37°C. Mixes containing SC-5 and 150-200 HVA did not crack at -38.3°C, the lowest pavement surface temperature recorded during the winter of 1967-1968. After 3.5 years of service, no signs of transverse cracking had yet appeared on the 150-200 HVA or SC-5 test section (Burgess et al. 1972). The other two test sections exhibited transverse cracking.

The Ste. Anne test road was monitored until 1975 when the then eight-year-old pavement was overlaid with 5 inches of asphalt concrete made with SC-5 liquid asphalt. Just before the pavement sections were overlaid, the test sections were surveyed again (Deme and Young 1987). It was observed that the softer 300-400 LVA pavements exhibited less transverse cracking than pavements containing the harder 150-200 LVA on clay subgrade and delayed cracking for five years on the sand subgrade. The relatively low temperature susceptible 150-200 HVA (PI = -1.4 vs -2.7 for 150-200 LVA) did not crack after eight years on the clay subgrade and delayed cracking for five years on the sand subgrade. The SC-5 liquid asphalt sections exhibited no cracking after eight years.

Pavement structure factors were also found to affect the degree of transverse cracking. The thicker 10-inch asphalt concrete pavement constructed with 150-200 LVA on clay subgrade exhibited only 40 percent of the transverse cracking of the pavement containing 4-inch asphalt concrete on 16 inches of crushed gravel base with the same binder and subgrade. More transverse cracking was found in the pavement sections constructed on sand subgrade as compared to clay subgrade. The interaction of subgrade type, traffic and asphalt affected the time of crack initiation and cracking frequency. A higher incidence of transverse cracking was observed in the traffic lanes for 300-400 LVA and 150-200 HVA binders than in the passing lanes on sand subgrade only.

Aggregate quality had a modest effect on cracking frequency with the sections containing 100 percent igneous aggregate and 150-200 HVA binder over sand subgrade exhibiting no cracking while sections with local 80 percent limestone and 20 percent igneous aggregate showed moderate transverse cracking. Asphalt content, within the range of 1 percent below optimum to 0.5 percent above optimum, and cement filler content did not show any significant effect on transverse cracking according to Deme and Young (1987).

Crack Temperature Prediction - Hills and Brien Procedure

A comparison of predicted cracking temperatures with the actual cracking temperature recorded in the field was performed. The predicted cracking temperature was determined using the procedure developed by Hills & Brien (1966). In this procedure, the thermal stress (σ_{th}) and breaking stress (σ_{br}) versus temperature curves are plotted for a given cooling rate and their intersection determines

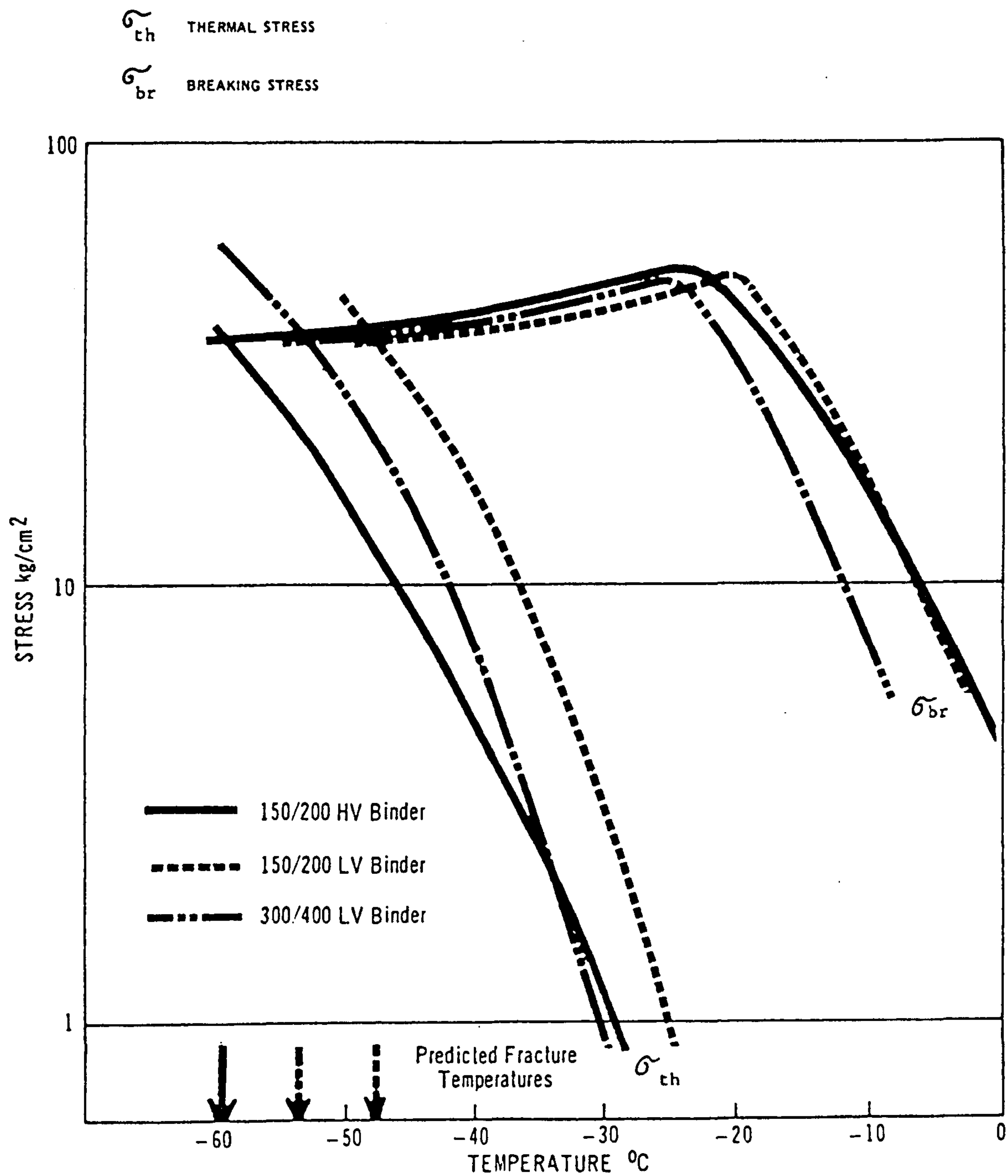


Figure 3.11 Calculation of predicted fracture temperatures based on field-aged Ste. Anne binders (Burgess et al. 1971)

the predicted fracture temperature. An example of the thermal and breaking stress calculation for field aged Ste. Anne 150-200 LVA binder is shown in Table A.3. Determination of fracture temperatures for field aged binders recovered after construction are shown on Figure 3.11. Predicted fracture temperatures for the laboratory and field aged binders and the binders recovered from actual pavement specimens after 3.5 years in service are given in Table 3.2 (Burgess et al. 1972). The less temperature susceptible 150-200 HVA binder resulted in lower fracture temperatures than the higher temperature susceptible 150-200 LVA binder.

Crack Temperature Prediction - Binder Stiffness

The results from further research connected with the Ste. Anne test road produced several methods for predicting the temperature at which low temperature cracking would occur. The primary influence variable for such predictions was the stiffness of the asphalt binder at low temperatures. To investigate this hypothesis, the stiffness moduli of the three field-aged binders recovered after construction at 10,000 seconds loading time as suggested by Krom and Dormon (1967) was calculated using Van der Poel's (1954) nomograph and plotted as a function of temperature in Figure 3.12 (Young et al. 1969).

Binder data used to enter the nomograph were obtained by plotting penetration at three different temperatures on the Bitumen Test Data Chart (Heukelom 1973) and obtaining the corrected softening points. These corrected softening points were used together with penetration at 25°C to calculate the Penetration Indices (PI). The corrected softening points and PI values were used to obtain binder stiffness values from the nomograph. In later work, Fromm and Phang (1971) determined from the relationship in Figure 3.12 that the temperatures at which initial cracking of the 150-200 LVA and 200-300 LVA binders occurred during the first winter corresponded to a binder stiffness of 20,000 psi at 10,000 seconds loading time.

In a second procedure, Burgess et al. (1972) also calculated stiffness moduli for the binders recovered after construction at one-half hour loading time and plotted as a function of temperature in Figure 3.13. Stiffness moduli were calculated using the same procedure stated above. The relationships were then combined with field performance data which showed that mixes containing 150-200 HVA did not crack when the pavement surface reached a low temperature of -38°C. From the curve in Figure 3.13 it was deduced that mixes containing this binder did not crack when the stiffness modulus of the binder reached 2,550 Kg/cm^2 (approximately 36,200 psi). Burgess et al. (1972) assumed this binder stiffness to be the maximum critical value at which pavements will tolerate without cracking under low temperature conditions. Based on this assumption, it was possible to calculate the minimum pavement surface temperatures at which pavements containing

Table 3.2 Predicted fracture temperatures for Ste. Anne (Burgess et al. 1972)

Material	Binders LV 150/200 °C.	LV 300/400 °C.	HV 150/200 °C.
Lab. Aged Ste. Anne Binders	-47	-52	-59
Field Aged Ste. Anne Binders	-48	-53	-59
Recovered Ste. Anne Binders(1)	-47	-50	-52
Temp. of Stat. Sig. Field Cracking	-34	-37	-
Temp. of Appearance of First Field Cracks	-29	-36	-

(1) After 3-1/2 years' field service.

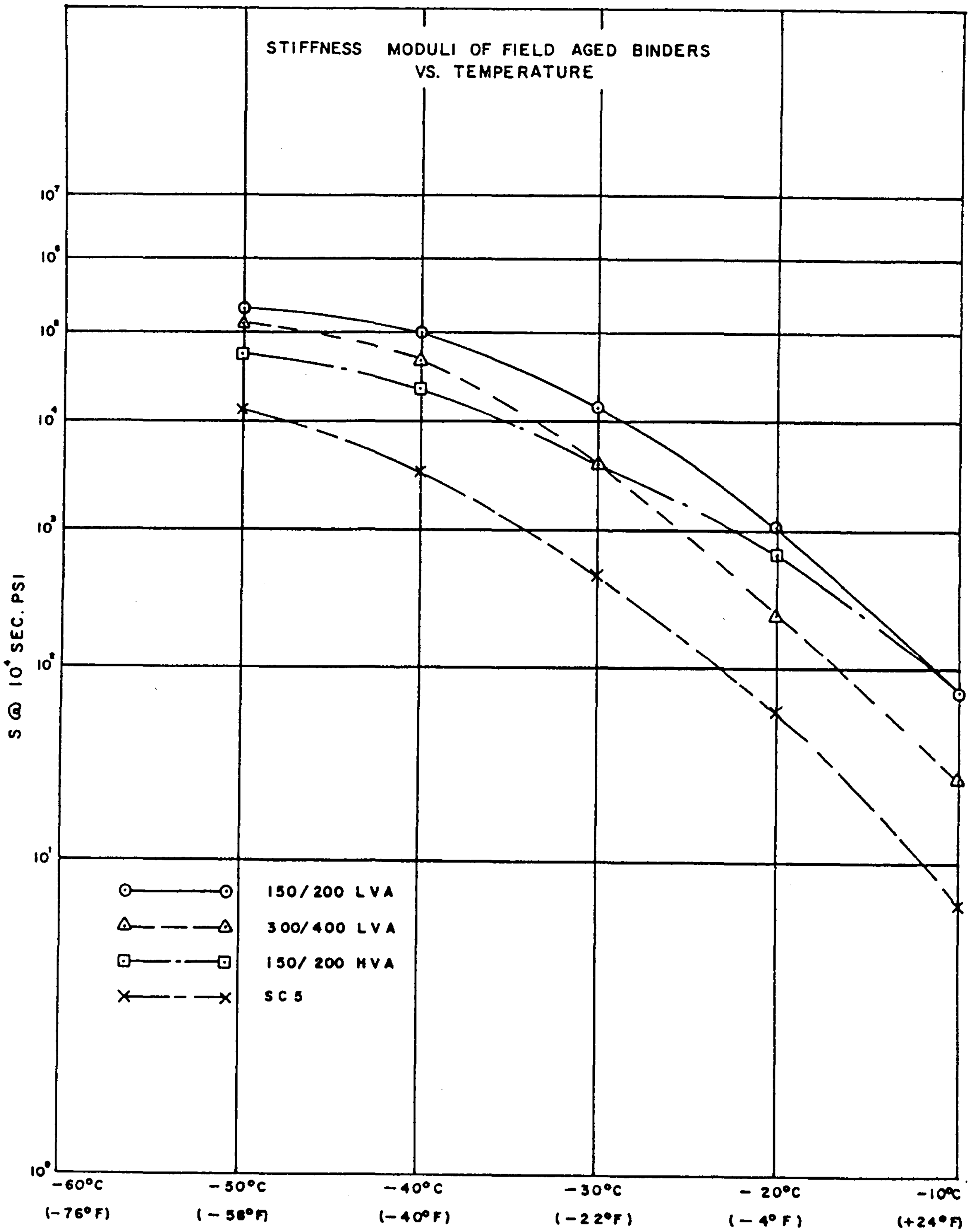


Figure 3.12 Stiffness vs. temperature at 10,000 seconds loading time for Ste. Anne field aged binders (Young et al. 1969)

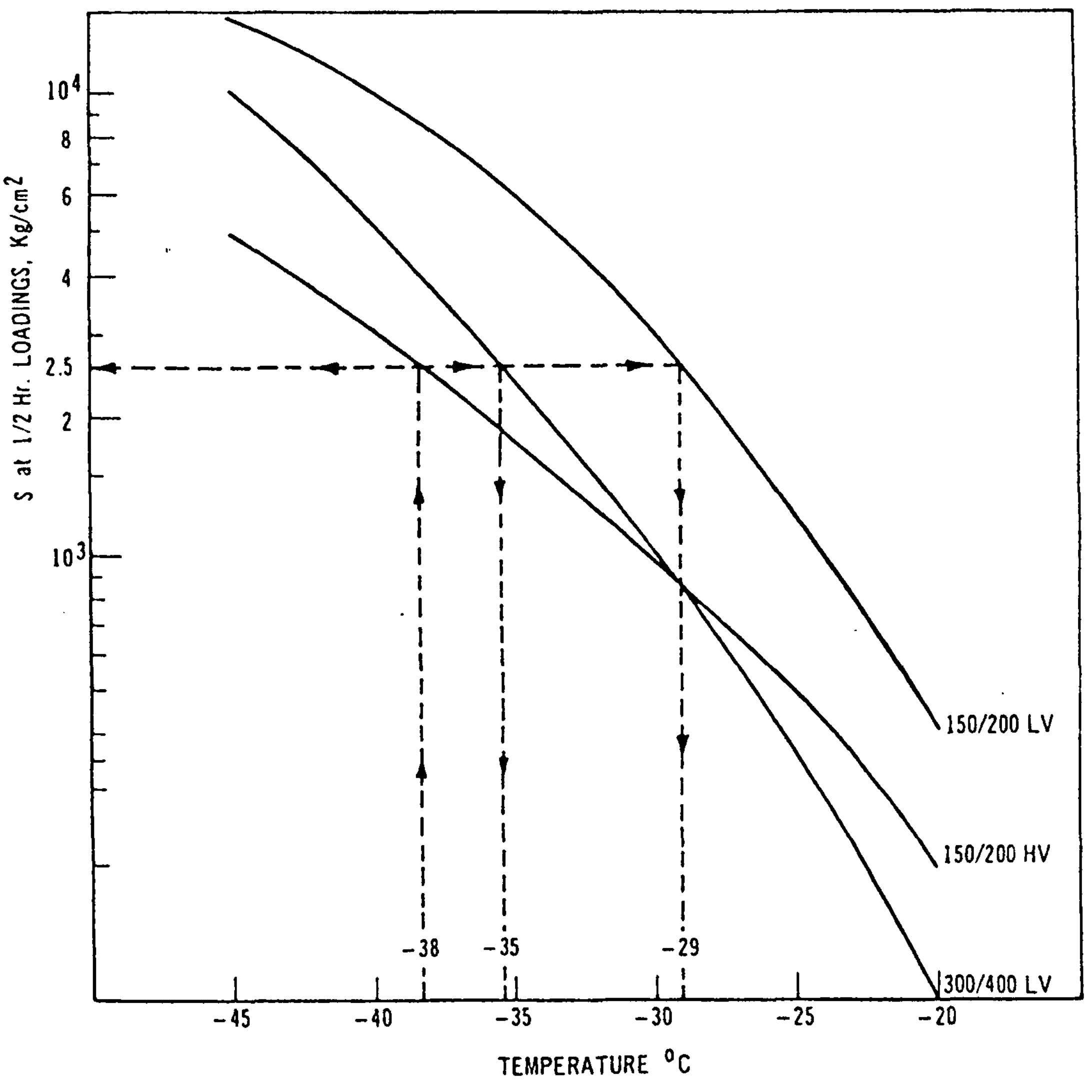


Figure 3.13 Minimum temperatures to avoid cracking for Ste. Anne field aged asphalts (Burgess et al. 1972)

the other two binders could tolerate without cracking. These temperatures were estimated to be -29°C for mixes containing 150-200 LVA and -35°C for mixes containing 300-400 LVA.

Similar treatment of the data for laboratory aged binders produced minimum non-fracture temperatures of -30°C and -35°C for mixes containing 150-200 LVA and 300-400 LVA respectively. Predicted minimum non-fracture temperatures for both methods are shown in Table 3.3 in comparison with those pavement surface temperatures at which initial cracking and "statistically significant" cracking occurred in the field. These results show the average non-fracture temperatures to be consistently higher than the temperature at which "statistically significant" cracking occurred, and almost identical to the temperatures at which the very first cracks appeared in the field.

Using similar plots of stiffness moduli as a function of temperature for the three binders recovered from the Ste. Anne text sections after 3.5 years of field service, minimum non-fracture temperatures of -31°C, -28°C and -32°C for mixes containing the 150-200 HVA, 150-200 LVA and 300-400 LVA binders, respectively, were calculated. These data indicate that the paraffinic low viscosity binders aged (stiffened) in the field over 3.5 years such that their predicted minimum non-fracture temperature increased by 2 to 3°C. The naphthionic 150-200 HVA, however, aged to the extent that the predicted minimum non-fracture temperature increased from -38°C to -31°C after 3.5 years. These data are illustrated in Figure 3.14.

Although the pavement containing the 150-200 HVA binder had not cracked after 3.5 years of service, the point was stressed that although a pavement containing a high viscosity type binder is generally considered more resistant to low temperature cracking than one containing a low viscosity type binder, the recorded data in Figure 3.14 indicate that after some 3 years of service, this is no longer the case (Burgess et al. 1972). If, for example, the minimum pavement surface temperature obtained during the 1967/68 winter were only -32°C, neither the LVA 300-400 mixes nor the HVA 150-200 mixes would have cracked. If a pavement surface temperature of -30°C were then to be reached during the 1971/72 winter, the HVA 150-200 mixes would crack whereas the LVA 300-400 mixes would not crack (Burgess et al. 1972).

Burgess et al. (1972) cautioned that the prediction procedure using binder stiffness only indicates the low temperature cracking tendency of new pavement construction. The temperature at which initial cracking may be expected will increase with the age of the pavement through stiffening of the binder. However, the authors felt that this increase in cracking tendency with

Table 3.3 Field cracking temperature vs estimated minimum non-fracture temperature based on binder stiffness moduli for Ste. Anne (Burgess et al. 1972)

	LV 150/200 °C.	LV 300/400 °C.
Estimated Minimum Non-fracture Temp.		
- Field Aged	-29	-35
- Lab. Aged	-30	-36
- Average	-29	-35
Temperature of Significant Initial Field Cracking	-34	-37
Temperature When First Field Cracks Appeared	-29	-36

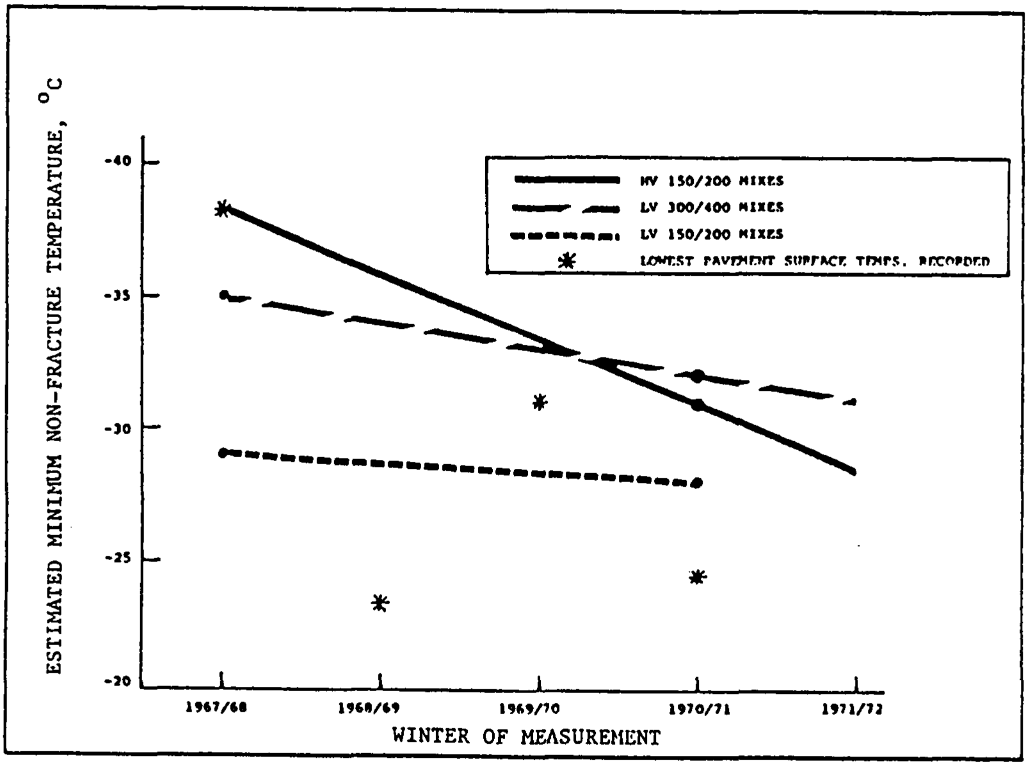


Figure 3.14 Effect of field service of test pavements on estimated minimum non-fracture temperature for Ste. Anne (Burgess et al. 1972)

pavement age could be predicted from a knowledge of the aging characteristics of the binder(s) involved during the service life of the pavement.

In a third procedure, Gaw et al. (1974) derived "empirically corrected predicted stiffness values" for the asphalts based on the relationship between predicted stiffness obtained from van der Poel's 1954 nomograph and stiffness measured directly with the Shell sliding plate rheometer using procedures described by Fenijn and Krooshof (1970). The "empirically corrected" stiffnesses were obtained by applying an adjustment factor to the predicted stiffnesses obtained from van der Poel's nomograph. This was based on the relationship between predicted stiffness and experimentally measured stiffness values at -10°C and by assuming that both methods would coincide at $3 \times 10^9 N/m^2$. The "empirically corrected" stiffnesses for the three laboratory aged binders are plotted as a function of temperature in Figure 3.15.

From these relationships, Gaw et al. (1974) determined that the temperatures at which initial cracking occurred during the first winter corresponded to a binder stiffness of 10^9 N/m^2 (1.4×10^5 psi) at 1/2 hour (1,800 seconds) loading time for both 150-200 LVA and 300-400 LVA binders. It is pertinent to note that this stiffness and loading time was accepted by the Asphalt Institute's Ad Hoc Committee (1981) as the limiting stiffness at which pavements crack.

Deme and Young (1987) compared the stiffnesses obtained from original, thin-film aged and recovered asphalt properties from the pavement after five years (Figure 3.16). They noted a progressive increase in binder stiffness modulus with time at warm temperatures but little change at low temperatures in the regime at which transverse crack initiation was observed. This did not substantiate field evidence which indicated a tendency for pavement cracking to increase with time. Deme and Young (1987) attributed the increase in cracking with time to "age hardening" or "structural hardening" of the binder. They concluded that binder stiffness modulus was an excellent criterion for predicting the initial critical cracking temperature. However, they expressed concern that this was an inadequate tool for predicting long-term change in pavement cracking susceptibility as the binder structure caused by structural hardening is altered in the binder recovery process.

Conclusions

The following conclusions are pertinent to the results and observations obtained from the Ste. Anne field and laboratory investigations that are outlined in this report:

1. Transverse cracking was initiated mainly at the pavement surface at a time when the surface temperature was close to the minimum on a given day.

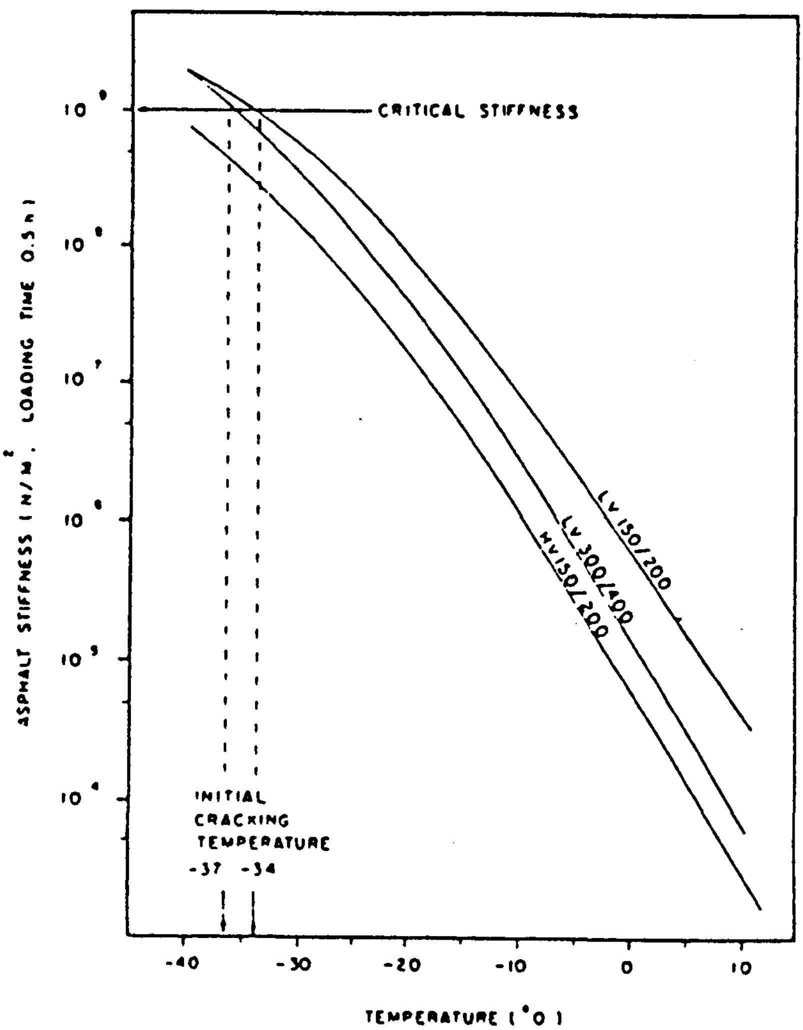


Figure 3.15 Critical binder stiffness at initiation of transverse cracking (after Gaw et al. 1974)

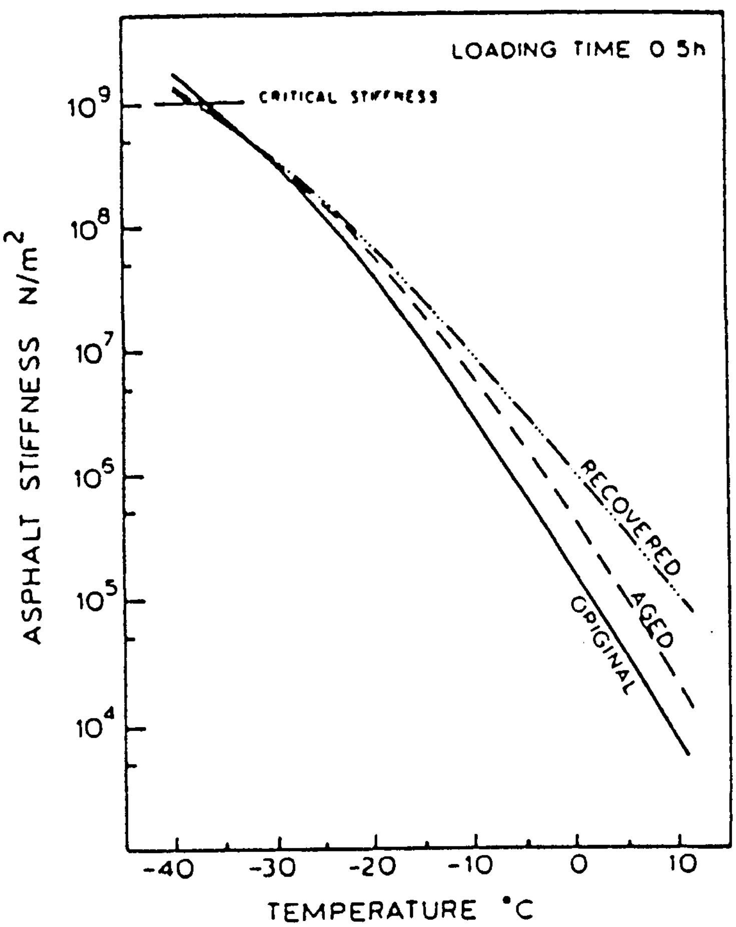


Figure 3.16 Recovered binder properties from 5-year old Ste. Anne pavement (Deme & Young 1987)

2. The asphalt grade had a significant effect on transverse cracking with the softer low viscosity 300-400 pen asphalt cement exhibiting less transverse cracking than the harder 150-200 pen asphalt cement from the same crude source on clay subgrade. In addition, it delayed cracking for five years on sand subgrade.
3. The temperature susceptibility of the asphalt had a significant effect on low temperature cracking with the less temperature susceptible (high viscosity) 150-200 pen grade asphalt cement exhibiting no transverse cracking after eight years on clay subgrade and delaying cracking for five years on sand subgrade as opposed to the more temperature susceptible (low viscosity) 150-200 pen asphalt which cracked during the first winter on both subgrades.
4. The SC-5 liquid asphalt, a high penetration binder made from a high viscosity base asphalt, exhibited no cracking after eight years.
5. The thickness of the asphalt pavement structure affected the amount of low temperature cracking. Forty percent less cracking occurred in the full-depth 10-inch asphalt concrete pavement on clay subgrade as compared to the thinner 4-inch asphalt concrete with 16-inch crushed gravel base pavement on clay subgrade.
6. The cumulative effects of traffic increased the frequency of transverse cracking on some of the pavements with the sand subgrade.
7. Less cracking for the same binder type (150-200 HVA) was found in the asphalt concrete mixes containing the 100% igneous aggregates as compared with mixes containing 80% limestone and 20% igneous aggregate.
8. Asphalt content did not influence cracking within the range of one percent below to one-half percent above Marshall optimum.
9. Mix filler content did not influence cracking within the range of 2% to 5.5% finer than 75μm (No. 200 sieve).
10. The critical stiffness moduli for the asphalt binder at the time of cracking was determined to be 10^9 N/m^2 (1.45 x 10^5 psi) at a loading time of 0.5 hour.
11. Pavements for which the stiffness moduli exceeded the limits in item 10 cracked during the first winter.
12. While recovered binder stiffness correlated with transverse cracking initially, its precision was found to diminish with time for predicting changes in susceptibility to cracking. This was attributed to the reversal of "structural hardening" during binder recovery from the pavement.

3.3 PENNSYLVANIA (1976)

Description

In 1976, six test pavements were constructed on Traffic Route 219 in Elk County, Pennsylvania using six AC-20 asphalt cements from five different sources (Kandhal, Mellott and Busso 1984). The objectives were to; (a) study the change in asphalt properties with age for in-service pavements, (b) determine the effect of rheological properties at 77°F (25°C) or lower temperatures on pavement performance and durability, and (c) develop suitable specifications for AC-20 asphalt cement to insure durable pavements.

The test road was built on a two-lane 20 foot wide highway with an ADT of 3700. Each test section was approximately 2000 feet long and consisted of a 1.5 inch resurfacing of an existing structurally sound pavement. The structure of the existing pavement was as follows:

1. Subgrade - consists of silty soil (AASHTO Classification A-4).
2. Base - 10 inch crushed aggregate base and 3 inch penetration Macadam (constructed 1948)
3. 3 inch binder and 1 inch coarse sand mix (constructed 1962) surface treatment (placed 1974)
4. 1.5 inch bituminous concrete wearing course (placed 1976)

Mix composition and compaction levels were held reasonably constant on all test pavements. The mix temperature for each test pavement was adjusted to obtain a mixing viscosity of 170 ± 20 centistokes to ensure more uniform construction. The only significant variable was the asphalt source. The crude sources, methods of refining and chemical compositions are given in Table 3.4. Asphalts T-1 and T-5 came from the same refinery. The properties of original asphalt cements and asphalts recovered just after construction are given in Tables A.4a and A.4b respectively and are plotted on Bitumen Test Data Charts developed by Heukelom (1973) in Figures A.2 through A.7.

Results

A very cold winter (1976-1977) followed construction of the test pavements with air temperatures reaching as low as -20°F (-29°C) and minimum pavement temperatures 2 inches below the pavement surface reaching -10°F (-23°C). Visual observations indicated that two pavements (T-1 and T-5) out of the original six developed low-temperature cracking. These two pavements contained asphalt from the same crude source and were both refined by vacuum distillation with propane deasphaltizing while the others were refined by steam or vacuum distillation. The results of crack surveys

Table 3.4 Study asphalts - crude sources, methods of refining and chemical composition for Pennsylvania (Kandhal et al. 1984)

Asphalt Type	Crude Sources	Method of Refining	Rostler Analysis[a], Percent					$\frac{A_1+N}{A_2+P}$
			A	N	A_1	A_2	P	
T-1	49% Sahara, 21% W. Texas, 21% Montana and 9% Kansas	Vacuum Distillation and propane de-asphalting	8.1	9.0	39.9	30.9	12.1	1.14
T-2	66-2/3% Texas Mid-Continent and 33-1/3% Arabian	Steam Distillation	22.4	17.4	24.4	24.4	11.3	1.17
T-3	85% Light Arabian and 15% Bachaquero	Vacuum Distillation	17.0	23.2	18.8	31.0	10.0	1.02
T-4	75% W. Texas Sour and 25% Texas and Louisiana Sour	Vacuum Distillation	19.4	23.1	17.0	27.7	12.8	0.99
T-5	Same as T-1	Same as T-1	15.9	28.7	18.2	27.7	9.4	1.26
T-6	Blend of Heavy Venezuelan and Middle East Crude	Vacuum Distillation	10.4	25.8	19.1	25.3	19.3	1.01

[a] A = Asphaltenes, N = Nitrogen Bases, A_1 = First Acidaffins, A_2 = Second Acidaffins and P = Paraffins

conducted within 3 years of construction are given in Table A.5. In this table, the cracking index as referenced by Fromm and Phang (1972) was used except 1/4 of part width (P) cracks of 2 to 8 feet were also added. The typical crack configuration for T-1 and T-5 pavements is shown in Figure 3.17.

After 6 years of service, the existing cracks on sections T-1 and T-5 had widened and more cracks developed with each successive winter while the other test pavements gradually started to develop cracking to different degrees (Kandhal, Mellot and Busso 1984) . Longitudinal cracking (as part of block cracking) was extensive in sections T-1 and T-5 as well. Performance ratings obtained 6 years after construction are presented in Table A.6. Each category was rated on a scale from 1 to 10. A perfect pavement would have a rating of 40. Results of a transverse crack survey conducted 5 years after construction is shown in Table A.7. Again, the cracking index shown in this table was modified to include 1/4 of part width (P) cracks.

The temperature susceptibility of the original asphalts and asphalts recovered immediately after construction was determined by three indirect methods: (a) the PI (pen/R & B) method (Pfeiffer and van Doormaal 1936), (b) the PI (pen/pen) method (Heukelom 1973), and (c) the PVN method (McLeod 1976). The results of these measurements are shown on Table 3.5. Asphalt T-1 was the most temperature susceptible according to all 3 methods followed closely by asphalt T-5. Kandhal et al. (1984) found that the Penetration Index numbers based on the Heukelom method, PI (pen/pen), were substantially lower than the Penetration Index numbers using the Pfeiffer and van Doormaal method (PI (pen/R&B)). It was also found that, of the three methods, the McLeod method (PVN) showed the least change in temperature susceptibility between original asphalts and asphalt recovered just after construction while the Penetration Index values obtained by the other methods showed greater change.

Stiffness moduli for the original asphalts were determined indirectly using three methods. For each method, a loading time of 20,000 seconds and temperatures of -20°F (-29°C) and -10°F (-23°C) were used. The three methods used are described below:

1. Original van der Poel Method (Method I) - The original method as described by van der Poel (1954), in the form of a nomograph. This method uses penetration at 77°F (25°C), softening point (R & B) and PI (pen/R & B).
2. Heukelom Modification (Method II) - This method (Heukelom 1973) uses penetration at two or three temperatures, "corrected" softening point (T_{800}) pen and PI (pen/pen). The original van der Poel nomograph is used for determining stiffness modulus.

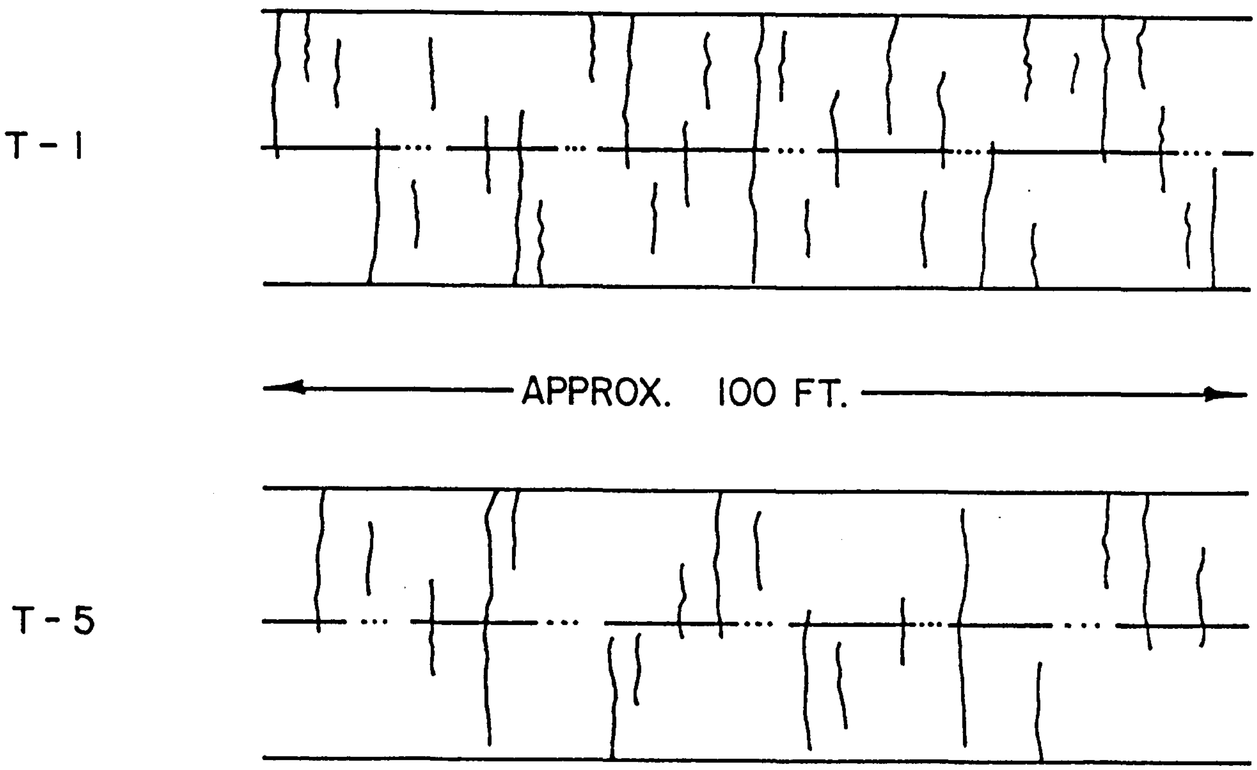


Figure 3.17 Typical cracking patterns in T-1 and T-5 sections for Pennsylvania (Kandhal et al. 1984)

Table 3.5 Temperature susceptibilities of original and aged asphalts for Pennsylvania (Kandhal et al. 1984)

Asphalt Type	PI (pen/R & B)		PI (pen/pen)		PVN	
	Original	Aged	Original	Aged	Original	Aged
T-1	-1.43	-1.18	-2.77	-2.24	-1.04	-1.13
T-2	-0.61	-0.91	-0.71	-0.80	-0.70	-0.68
T-3	-0.60	-0.61	-1.51	-0.99	-0.61	-0.72
T-4	-0.57	-1.24	-1.05	-0.65	-0.86	-1.03
T-5	-0.74	-1.30	-2.23	-2.03	-1.03	-1.16
T-6	-0.16	-0.32	-1.29	-0.64	-0.45	-0.47
Average	-0.68	-0.93	-1.59	-1.22	-0.78	-0.86

3. McLeod Method (Method III) - This method (McLeod 1976) uses penetration at 77°F (25°C) and viscosity at 275°F (135°C).

Results are shown on Table 3.6. Kandhal, Mellot & Busso (1984) observed that the modulus values obtained from the Heukelom method were significantly higher than the other methods for asphalts T-1, T-3, T-5 and T-6. However, all three methods showed that the two asphalts that developed excessive low-temperature cracking (T-1 and T-5) had the highest stiffness moduli.

The stiffness moduli of asphalts recovered immediately after construction also were determined by the same three methods at -10°F (-23°C) and -20°F (-29°C) for both 10,000 second and 20,000 second loading times (Table 3.7). Based on Ste. Anne Test Road data (Young et al. 1969), Fromm and Phang (1971) have suggested a limiting stiffness for field aged asphalt of 20,000 psi at a loading time of 10,000 seconds to eliminate cracking. This stiffness modulus was calculated using Heukelom's (1973) suggested procedure. The two asphalts that showed early cracking (T-1 and T-5) had stiffness moduli at -10°F and 10,000 seconds loading time above that limit.

In July 1977, a specification requirement for AC-20 asphalt cement for use in cold regions of Pennsylvania where the Air Freezing Index exceeds 1000 degree/days was implemented. The specification requirement allows a maximum permissible stiffness modulus of 275 kg/cm^2 (3905 psi) at -10°F and 20,000 seconds loading time based on McLeod's (1976) method. The 275 kg/cm^2 value was selected since it was midway between the highest asphalt stiffness for uncracked pavements (200 kg/cm^2 for asphalt T-4) and the lowest asphalt stiffness on the cracked pavements (375 kg/cm^2 for asphalt T-5). The specification was implemented by specifying minimal allowable PVN and viscosity at 275°F for various penetration values. The minimum allowable PVN and viscosity at 275°F requirements were set such that the stiffness would not exceed 275 kg/cm^2 (3905 psi) when calculated using McLeod's method.

Twenty months after construction, core samples were taken and asphalt cements were recovered. The recovered asphalt properties are presented on Table A.8. The PI (pen/pen) values of the original and aged asphalts showed large increases (decreased temperature susceptibility) after 20 months, whereas PVN values have changed very little (Tables A.4a, A.4b, and A.8). Due to the large changes in the PI (pen/pen) values, the stiffness moduli of most asphalts determined by the Heukelom method using PI (pen/pen) decreased after 20 months rather than increased.

Table 3.6 Stiffness moduli (psi) of original asphalt cements, load time 20,000 sec for Pennsylvania (Kandhal et al. 1984)

	ASPHALT TYPE					
	T-1	T-2	T-3	T-4	T-5	T-6
<u>At -20 F (-29 C)</u>						
1. van der Poel Method	36,250	7,250	5,800	7,250	10,875	3,335
2. Heukelom Method	159,500	7,250	13,050	7,975	50,750	9,425
3. McLeod Method	26,980	7,810	5,680	9,940	17,040	4,615
<u>At -10 F (-23 C)</u>						
1. van der Poel Method	10,150	2,102	1,740	2,102	3,625	870
2. Heukelom Method	29,000	2,030	2,900	2,175	10,585	2,175
3. McLeod Method	9,940	2,556	1,775	2,840	5,325	1,278

1 psi = 6896 Pa = 0.07 kg/cm^2

Table 3.7 Stiffness moduli (psi) of recovered asphalt cements for Pennsylvania (Kandhal et al. 1984)

	ASPHALT TYPE					
	T-1	T-2	T-3	T-4	T-5	T-6
At -20 F and 20,000 sec.						
1. van der Poel Method	58,000	21,025	14,500	37,700	43,500	9,860
2. Heukelom Method	145,000	17,400	15,950	17,400	116,000	10,875
3. McLeod Method	71,000	18,460	17,040	39,050	56,800	11,360
At -10 F and 20,000 sec.						
1. van der Poel Method	21,025	6,090	5,220	10,875	15,950	3,480
2. Heukelom Method	50,750	5,800	5,075	6,090	37,700	3,480
3. McLeod Method	26,980	6,390	5,680	14,200	21,300	3,905
At -20 F and 10,000 sec.						
1. van der Poel Method	72,500	24,650	18,850	46,400	58,000	12,325
2. Heukelom Method	174,000	23,200	21,750	24,650	145,000	13,050
3. McLeod Method	99,400	23,430	21,300	53,960	71,000	14,200
At -10 F and 10,000 sec.						
1. van der Poel Method	29,000	8,990	7,250	15,950	21,750	4,640
2. Heukelom Method	72,500	8,120	7,250	8,700	66,700	5,075
3. McLeod Method	32,660	9,940	7,100	19,880	24,850	5,396

1 psi = 6895 Pa = 0.07 kg/cm^2

Six years after construction, core samples were again taken and recovered asphalt current properties measured as shown in Table A.9. Asphalts T-1 and T-5, which exhibited the most transverse cracking, had the lowest penetrations at 15 and 22, respectively.

Conclusions

The following conclusions were drawn from this research project. It should be noted that these conclusions are based on a 1.5 inch overlay containing the test asphalts of an existing structurally sound pavement. Note also that the authors did not specifically relate Rostler's parameters to performance despite their measurement. It is likely that their initial hypothesis was to determine if some relationship could be established between the parameter and performance. The fact that no relationship was reported suggests that no correlation was found. Nevertheless, the authors contribute to the general body of knowledge as regards typical values for Rostler's parameters.

1. Temperature susceptibility of the original and lab aged asphalts as determined by PI (pen/pen) values (Heukelom 1973) are substantially lower (ie. more temperature susceptible) than PI (pen/R & B) values (Pfeiffer & van Doormaal 1936) and PVN values (McLeod 1976). PVN values are comparable to PI (pen/R & B) values and were within $\pm$0.4 with an average difference within 0.1.
2. PI (pen/pen) values (Heukelom) generally increased substantially (decreased temperature susceptibility) when the asphalt cements were aged, whereas changes in PVN values were minimal.
3. Stiffness moduli of the original asphalt cements, determined from van der Poel's (1954) nomographs using Pfeiffer and van Doormaal's (1936), Heukelom's (1973), and McLeod's (1976) methods were in agreement that asphalts T-1 and T-5 had the highest stiffness values, respectively. Pavements constructed with these asphalts suffered severe non-load associated transverse cracking. It is pertinent to note that these asphalts were refined by propane deasphalting and vacuum distillation while the other asphalts were refined by vacuum distillation and steam distillation.
4. The limiting stiffness criterion of 20,000 psi at 10,000 seconds loading time recommended by Fromm and Phang (1971) for asphalt recovered immediately after construction, using PI (pen/pen) and corrected softening point suggested by Heukelom as the basis of calculation, was verified. Asphalts recovered from sections T-1 and T-5 had stiffness moduli over 20,000 psi at -10°F (-23°C), recorded 2 inches below the pavement surface, at 10,000 seconds loading time.
5. From the limited data available, a maximum permissible stiffness modulus of 275 kg/cm^2(3906 psi) for the original asphalt cement (at a minimum pavement temperature

of -10°F (-23°C) and 20,000 seconds loading time), based on McLeod's work, was incorporated into AC-20 asphalt cement specifications for cold regions of Pennsylvania.

6. Viscosity at 140°F or 275°F of recovered asphalt after 6 years did not correlate with the pavement performance.
7. Penetration at 77°F of the recovered asphalt after 6 years indicates a general trend: lower penetration associated with poor performance and vice versa. Asphalts T-1 and T-5, used by the poorest performing pavements, had penetrations of 15 and 22, respectively. Asphalt T-3, used in the best performing pavement, had a penetration of 35.

3.4 EUROPEAN TEST ROADS - FRANCE AND GERMANY

Description

Two controlled test roads were constructed in 1963 and 1964 in France and Germany (Chipperfield & Welch 1967, Chipperfield et al. 1970) to study the behavior of eight 70-100 penetration bitumens in asphalt concrete wearing courses in hot/dry and cold/wet climates. The test road in France was laid out on Route Nationale 7 (RN7) between Nice and Antibes where maximum road temperatures of 140°F (60°C) are recorded to represent a hot climate. The test road in Germany was laid on Bundestrasse No. 2 (B2) near Augsburg where minimum road temperatures of -4°F (-20°C) are recorded to represent a cold climate. The French test sections were part of a three-lane highway carrying approximately 15,000 vehicles per day and the German test sections were placed on a two lane highway carrying about 8,000 vehicles per day. In both test roads, the sections were laid as 1.5 inch wearing courses for resurfacing existing roads of uniform construction.

For each location, 110 test sections were constructed. Two mixture designs, two aggregates and three binder contents were studied for each of the eight asphalts in a 2x2x3x8 experiment with 14 control sections containing the number 1 bitumen. The experiment design layout is shown in Figure 3.18. Original asphalt properties were measured for each bitumen on both road trials (Table A.10). The properties of the asphalts remained substantially unchanged between the France Test Road (RN7) constructed in 1963 and the Germany Test Road (B2) constructed in 1964 with the exception of Bitumen No. 1. This difference was thought to be due to an unavoidable small change in the basic feedstock between construction dates. Samples of bitumen were also shipped to the USA and additional testing was performed by Materials Research and Development, California Division of Highways, and the Bureau of Public Roads. The results are given in Tables A.11, A.12, A.13a and A.13b, respectively. It is interesting to note that the Rostler durability parameter rated all of the asphalts as having satisfactory durability or better.

Performance Measurements

Performance measurements were conducted visually by a panel of observers. Overall condition was evaluated in addition to assessment of disintegration, excessive binder, deformation, traffic laning (differences in surface texture due to variations in traffic intensity across the lanes), general variability and texture. Overall condition ratings were based on a six-point scale; VG (very good), G (good), FG (fairly good), F (fair), P (poor), and B (bad). The ratings were subjective and those reported were an average assessment agreed upon by the raters. Full details of the rating procedure are reported in Chipperfield and Welch (1967). The number of test sections, by bitumen type, falling into the various rating categories after 5 and 6 years for the B2 and R7 trials, respectively are shown in Table 3.8.

Effects of Aging

Chemical and rheological properties of the bitumens recovered after mixing, transport, laying, and during service were measured to determine aging effects. The procedures used for sampling and recovery are given in Appendix B of Chipperfield and Welch (1967). Viscosity measurements at 15°C and 35°C using a sliding plate microviscometer (Griffin et al. 1957) on a constant shear stress basis were reduced to a single master curve at 77°F (25°C) using principles of time/temperature superposition. Viscosities were measured on recovered asphalts from mixes sampled during construction and from the top 1/8 inch slice of field cores. Aging indices (recovered viscosity/original viscosity) versus mixing, laying and time in service is shown in Figure 3.19. Changes in broad chemical composition after mixing and infield service, as measured by a modification of the Corbett and Swarbrick procedure (Corbett and Swarbrick 1958), is presented on Table A.14 and Figures 3.20 through 3.23. In the modified procedure, the asphaltenes were precipitated by n-heptane and the remaining material was separated by chromatography to give saturates as an iso-octane eluate, cyclics as a toluene eluate and resins as a toluene/ethanol eluate.

The results of the above measurements showed that the major compositional changes occurred mostly in the mixing and laying operations. Results show a slight decrease in cyclics content and increases in the resins and asphaltene contents for all bitumens. The saturates contents of the B2 bitumens were essentially unchanged, whereas for the RN7 bitumens, the saturates contents increased slightly. This slight increase was thought to be caused by oil spillage contamination. Chipperfield et al. (1970) concluded that "...despite the fact that the bitumens were chosen from a

		Aggregate Gradation (Coarse)						Aggregate Gradation (Dense)					
		Aggregate: Basalt (RN7) or Diabase (B2)			Aggregate: Quartzite (RN7) or Granite (B2)			Aggregate: Basalt (RN7) or Diabase (B2)			Aggregate: Quartzite (RN7) or Granite (B2)		
	Bitumen	% AC opt - 0.3%	% AC opt.	% AC opt. + 0.3%	% AC opt - 0.3%	% AC opt.	% AC opt. + 0.3%	% AC opt - 0.3%	% AC opt.	% AC opt. + 0.3%	% AC opt - 0.3%	% AC opt.	% AC opt. + 0.3%
France (RN7)	1												
	2												
	3												
	4												
	5												
	7												
	8												
	9												
Germany (B2)	1												
	2												
	3												
	4												
	5												
	7												
	8												
	9												

Notes: 1. 14 additional control sections using Bitumen 1 were constructed to assess variations due to construction or service.

2. Bitumen Number 6 was not used.

Figure 3.18 Experiment design layout for European Test Roads.

Table 3.8 Summary of performance measurements for European test roads after 5 and 6 years service (after Chipperfield et al. 1970)

Rating	Bitumen ID Number 1	2	3	4	5	7	8	9
VG	12	6	6	6	6	6	6	6
G	29	12	14	15	15	15	15	13
FG	10	6	6	4	3	3	3	5
F	--	--	--	--	--	--	--	--
P	1	--	--	--	--	--	--	--
B	--	--	--	--	--	--	--	--

NOTES:

1. Values in table are the number of test sections corresponding to each performance rating separated by bitumen number.

2. Bitumen number 6 was not used.

3. VG--Very Good
   G--Good
   FG--Fairly Good
   F--Fair
   P--Poor
   B--Bad

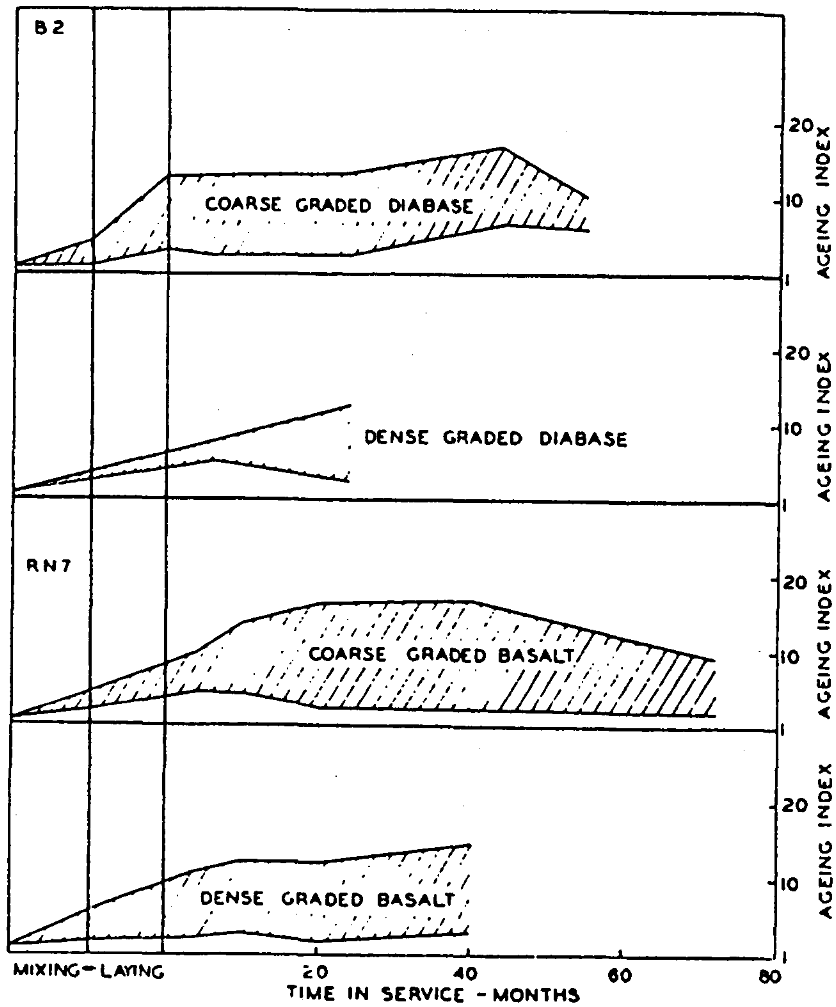


Figure 3.19 Range of aging indices (recovered viscosity/original viscosity) vs. time for European test roads (Chipperfield et al. 1970)

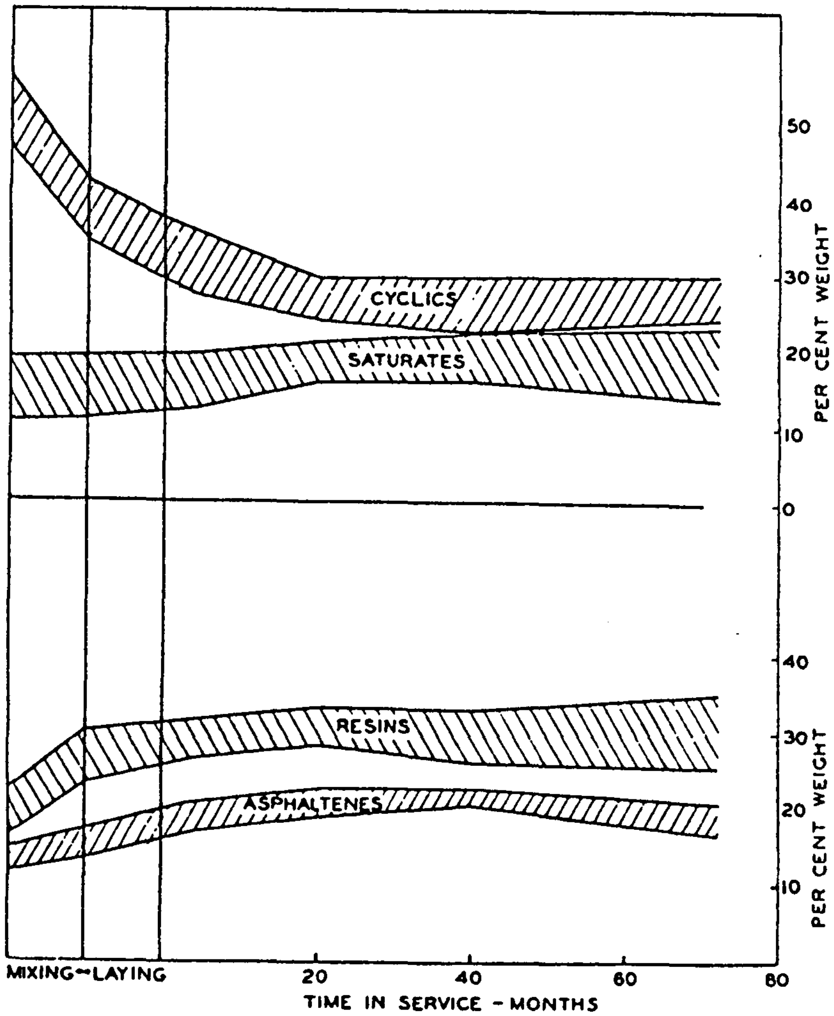


Figure 3.20 Change in chemical composition - RN7 basalt coarse graded mix for European test roads (Chipperfield et al. 1970)

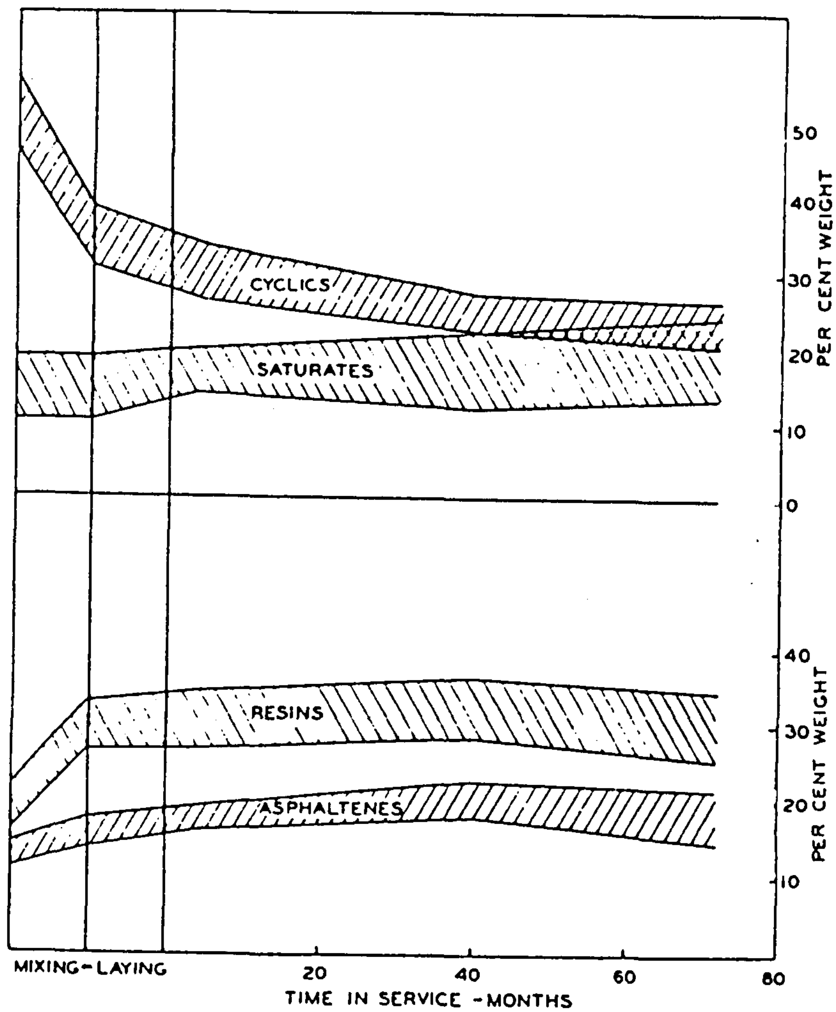


Figure 3.21 Change in chemical composition - RN7 quartzite coarse graded mix for European test roads (Chipperfield et al. 1970)

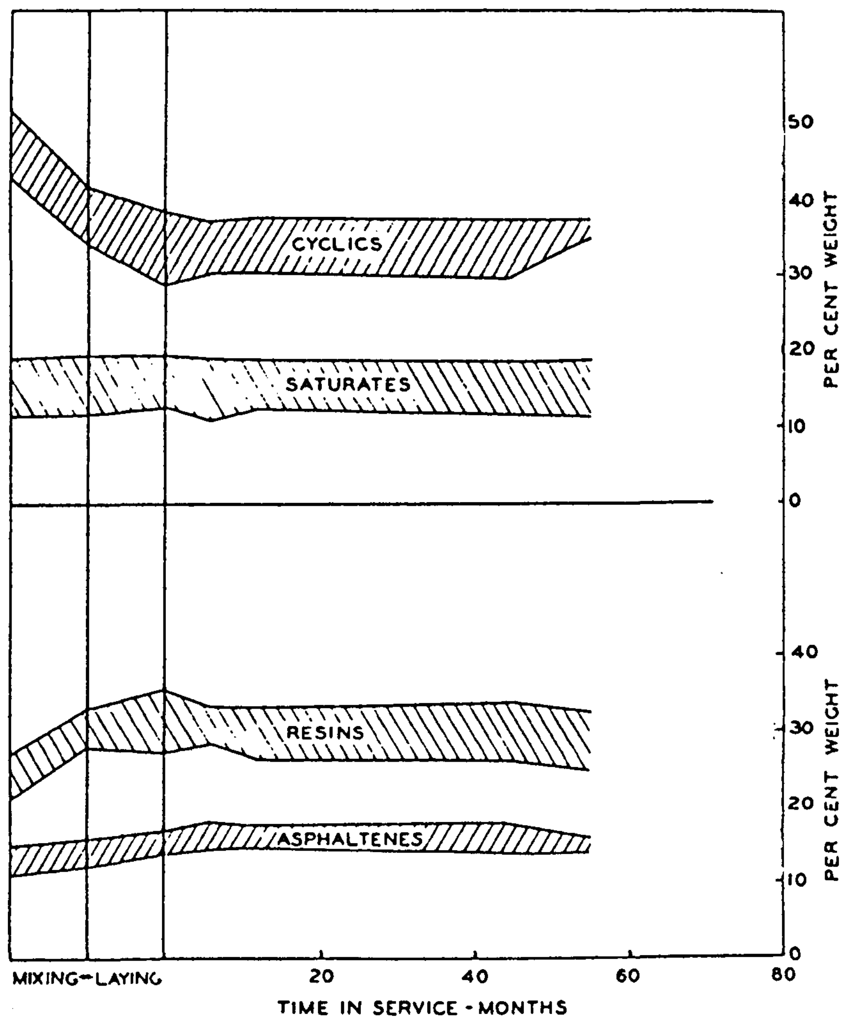


Figure 3.22 Change in chemical composition - B2 diabase coarse graded mix for European test road (Chipperfield et al. 1970)

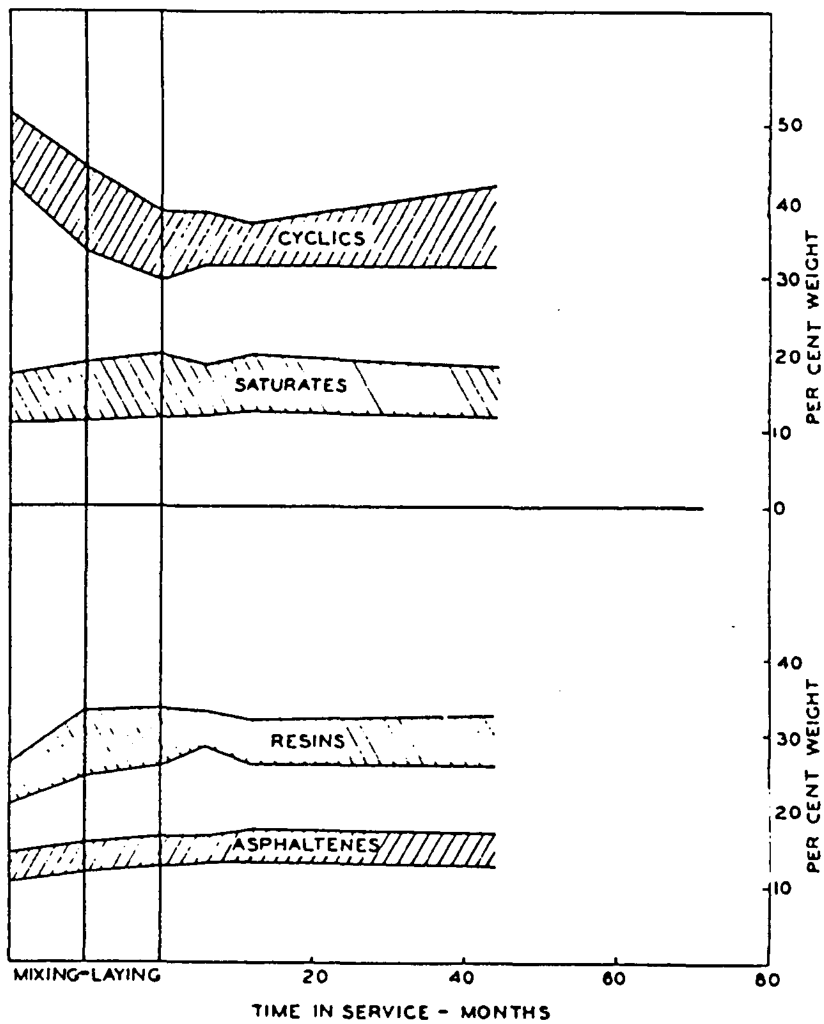


Figure 3.23 Change in chemical composition - B2 granite coarse graded mix (Chipperfield et al. 1970)

wide range of crude sources and prepared by different processing routes, the overall chemical changes of all the bitumens are remarkably similar for each test road".

Comparisons of Thin Film and Rolling Thin Film Oven Test (TFOT and RTFOT) results with field results for RN7 coarse graded basalt mixes are shown in Table A.15. The results showed that the tests indicated viscosity changes fairly well but were not in good agreement with broad chemical changes.

Conclusions

From the results of the studies, the following conclusions were drawn (Chipperfield et al. 1970). These conclusions were based on a 1.5 inch overlay containing the test asphalts of an existing pavement of uniform construction.

1. Despite differences in the initial characteristics of the asphalts, no large differences in pavement performance attributable to the asphalts were found. The differences observed in the various test sections after five and six years were accounted for by differences in aggregate grading and type, and defects in the road structure.
2. If the performance of the road trials proceeded in the same manner, the eventual failure of the test sections would unlikely be related to differences in bitumen properties.
3. Rheological measurements and broad chemical composition studies on samples of bitumen recovered at various stages during the construction and service of test sections have shown the bulk of the changes to occur during mixing and laying, with only slight changes in the 5 to 6 year service. Air void contents for cores extracted 4-6 months after construction generally ranged from 3 to 6 percent and after 20-24 months generally ranged from 2 to 5 percent.

During the discussion of the Chipperfield et al. (1970) paper at the Association of Asphalt Paving Technologists (AAPT) meeting, the point was raised by Welborn et al. that hardening of asphalt in service is largely affected by air voids content and asphalt content with asphalt characteristics of lesser importance. Because of the low air voids of the test roads, there was little chance for oxygen to enter the asphalt mixture and react with the asphalt. Vallerga commented that the results of a Bureau of Public Roads (now FHWA) study (Vallerga et al. 1970) indicated that hardening was directly related to air voids content and voids filled with asphalt. At less than 3 percent air voids, changes in asphalt properties were relatively insignificant. Over 3 percent, the changes were large.

Post-Study Observations

In France, the test sections were inspected and measurements taken after 11 years of service (Welborn 1979). Over-all, serviceability was still good, although there was a fair amount of longitudinal cracking on one side of the road which appeared to have been caused by pipe-trenching beside the road. Asphalt recovered from the upper 1/8 inch of core specimens showed only minor changes in chemical composition with time.

In Germany, the wearing course had suffered such severe damage from studded tires that the road required resurfacing and the study was terminated in 1971. The data obtained in 1970 and 1971 did not necessitate any modifications of the conclusions in the 1970 report.

3.5 MICHIGAN

Description

In 1954, Michigan constructed a controlled field experimental project consisting of six uniform test sections in which 60-70 penetration grade asphalts from six different sources were used (Parr et al. 1955, Parr & Serafin 1959, Serafin et al. 1967, Corbett & Merz 1975). The objectives were to correlate in-service pavement performance with asphalt properties in order to compare the performance of new asphalt sources becoming available in Michigan with asphalts that previously had been giving satisfactory service. The main differences in physical characteristics were in viscosity, temperature susceptibility, and resistance to hardening during hot plant mixing.

The test sections were built on a portion of US 10 with an ADT of 12,000 vehicles. Each test section was 2,400 feet long and 40 feet wide with no transitions and consisted of a 3-inch overlay on 23 year old reinforced concrete pavement with a granular subbase. The contractor was required to use the same sources of aggregates in producing the bituminous mixtures for each of the test sections. In addition, mixture proportions were maintained as uniform as possible in order to eliminate any effects such variations would have on pavement performance or physical properties of the asphalt or mixtures. The only significant variable was the asphalt source. The asphalt source for each section and composition of the mixtures is shown on Table 3.9. Samples of asphalt cements were obtained from tank car shipments and tested. The properties of the original asphalt cements are given in Tables A.16.

Pavement Performance

A 4-year coring and testing program, and visual observations gave little or no evidence of differences in performance of the pavement sections containing the 6 asphalts (Parr & Serafin 1959). Reflection cracking was present in all sections. It was noted that sections 2 and 6 had a rough

Table 3.9 Wearing course bituminous mixture materials and properties for Michigan (Serafin et al. 1970)

MATERIAL	SOURCE	TYPICAL BATCH PROPORTIONING, PERCENT
25A Coarse Aggregate	American Aggregate Corp., Green Oak, Mich.	55.0
38C Fine Aggregate	Blend of two local sands	34.5
3MF Limestone Mineral Filler	National Lime & Stone Co., Findlay, Ohio	5.0
60-70 Asphalt Cement	Section 1-Wyoming Crude, Refinery A Section 2-Venezuelan Crude Section 3-Wyoming Crude, Refinery B Section 4-West Texas Winkler Crude Section 5-Arkansas Smackover Crude Section 6-East Texas Talco Crude	5.5

surface texture. This condition had been noted since construction when mixes containing these high viscosity asphalts were difficult to roll, leaving a rougher textured surface.

After 12 years, differences in performance of the test sections were minimal. Some engineers inspecting the project felt that section 2 (Venezuelan crude) and section 6 (East Texas Talco crude) exhibited more cracking and pitting than the other sections. Penetration and ductility of recovered asphalts after 4 and 11 years are shown on Table A.17. The results show that asphalts 2 and 6 had the lowest penetrations with values of 26 and 29, respectively, after 11 years. Asphalt 6 showed the greatest decrease in ductility followed by asphalt 4 and asphalt 2. Serafin et al. (1967) cautioned, however, that there may have been variations in condition of the underlying concrete pavement and base material at the time of resurfacing which would make it difficult to draw any firm, significant conclusions regarding any differences in condition of the six test sections.

Rut depth measurements after 12 years of field service (Serafin et al. 1967) are summarized on Table A.18. There was very little difference between the various sections in results obtained for a given lane. Much more rutting developed in traffic lanes than passing lanes. Analysis of the data indicated no significant correlation between rutting and vertical alignment. In general, the greatest rutting occurred in the outermost lane around horizontal curves. More rutting occurred in the northbound traffic lane than the southbound, probably due to northbound truck traffic being more heavily loaded, because of numerous gravel pits at the south end of the test project. No correlation could be established between rut depth and asphalt viscosity. After 18 years, the performance of the sections was judged based largely on wear and weathering of the surface, with some consideration of edge raveling. In terms of relative performance, Sections 1, 3, and 5 were rated best, 2 and 6 as poorest, and 4 as intermediate.

Additional Studies

In 1975, Corbett and Merz reported the results of a study of the asphalt cements used on the Michigan test road. The study was undertaken to; (a) determine the extent of the change in chemical composition of the asphalt binders after 18 years of service, (b) relate the changes to the mechanism of binder hardening, and (c) relate, if possible, the compositional changes with respect to wear and weathering. Bulk samples of original asphalt stored in containers since construction were tested to determine original composition and viscosity at 140°F. Pavement cores were obtained from each of the six asphalt sections. The asphalt was extracted and recovered by the Abson method for the top 1/8-inch layer and the next 1/4-inch minus layer below the saw cut. Properties of the asphalt recovered from the cores are presented in Table A.19. The authors noted that for all asphalts, the viscosities at 140°F (60°C) and softening points increased and the penetrations and ductilities decreased. The changes were greater in the top 1/8-inch layer than in the 1/4-inch minus layer below it. This indicated that hardening is more pronounced at or near the pavement surface because of increased exposure to air, sunlight, etc. The binders from sections 2 and 6 had the highest recovered viscosities and binder 6 had the lowest recovered ductility.

Compositional analysis were performed using the Corbett (1969) method. These data are shown in Table A.20. The mechanism of change for each layer during aging is shown on Figure 3.24. The saturate content from recovered binders was virtually unchanged from that of the original binders; however, slight increases, thought to be attributable to drippage, were found in the top layer. The amount of naphthene aromatics decreased in all sections, more so in the top 1/8-inch layer than in the 1/4-inch minus layer. Asphaltenes consistently increased, especially in the top layer. Polar aromatics showed no distinct pattern. Both saturates and naphthene aromatics are low-viscosity components and Corbett believed that they function as plasticizers for the higher-viscosity components, ie. polar aromatics and asphaltenes. The compositional changes due to aging decrease in the liquid component (naphthene aromatics) and increase in the solid components (polar aromatics and asphaltenes) which would be expected based on the changes in physical properties.

<u>Conclusions</u>

The following conclusions were drawn from the Michigan Test Road.

1. All of the test sections showed considerable reflection cracking, wear, weathering and raveling; however, these distresses were more pronounced in sections 2 (Venezuelan) and 6 (East Texas Talco) asphalts.
2. Recovered binders from sections 2 and 6 had the lowest penetrations at 77°F with values of 31 and 34 after four years and 26 and 29 after 11 years. Recovered penetrations after 11 years for the remaining binders ranged from 30 to 35.
3. Recovered binders from sections 2 and 6 after 18 years had the highest viscosities at 140°F with values of 17,041 and 34,414 poise, respectively, compared with the range of 7,320 to 11,787 poise for the remaining binders.
4. Recovered binder from sections 2 and 6 had generally the lowest recovered ductility after 18 years with values of 0.5 and 2.5 cm at 60°F and 7 and 5 cm at 77°F.
5. Although sections 2 and 6 showed somewhat greater pitting and cracking, variation due to condition of the underlying concrete pavement and base material would make it difficult to draw any firm conclusions regarding cracking performance of the test sections.

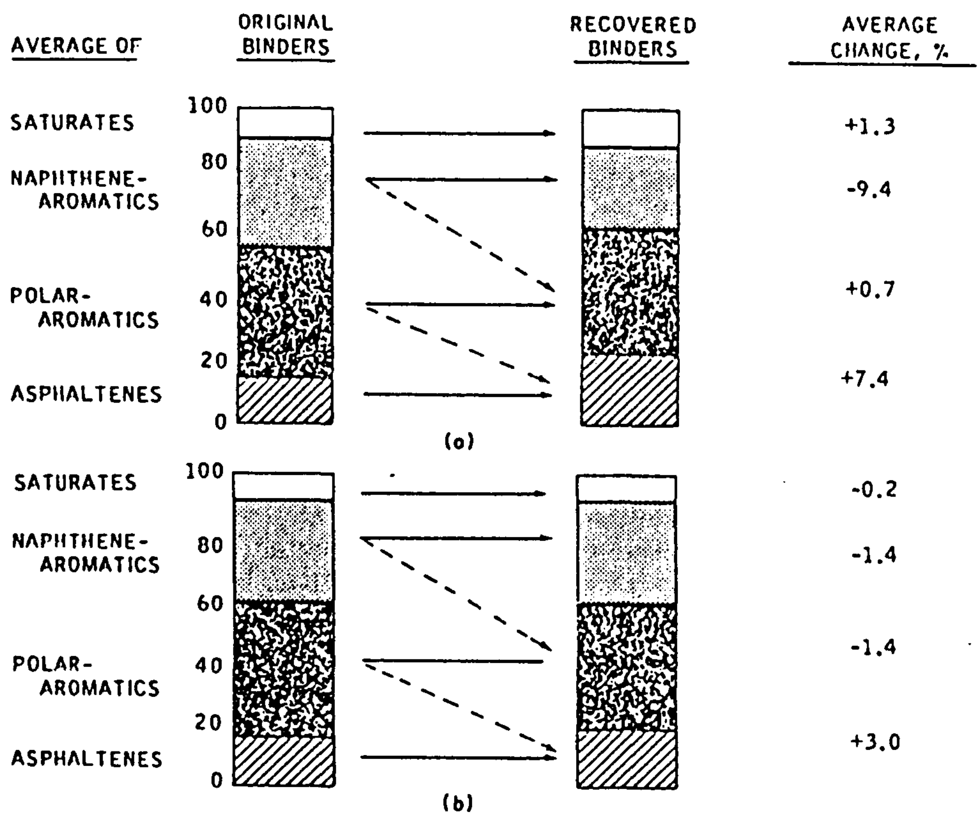


Figure 3.24 Mechanism of change for (a) top 1/8-inch layer and (b) 1/4-inch minus layer in Michigan (Corbett & Merz 1975)

6. Although a slight amount of rutting occurred in the test sections, no distinct relationship could be established between observed rut depth and asphalt viscosity.
7. Chemical composition on original and recovered binders revealed a reduction in naphthene aromatics and an increase in asphaltenes while polar aromatics and saturates showed no distinct trend.
8. Changes in binder consistency and chemical composition after 18 years were much greater in the top 1/8 inch layer than in the 1/4-inch minus layer of the wearing course, indicating that hardening is greater near the pavement surface because of increased exposure.

3.6 ONTARIO (McLeod)

Description

In 1960, the Ontario Ministry of Transportation and Communication constructed 3 test sections, each 6 miles long in Southwestern Ontario (McLeod 1972, 1987, 1988). Each test section was constructed with 85-100 penetration asphalt with low, medium or high temperature susceptibilities over loam to clay loam subgrades. The asphalt content for each road was 6.0 $\pm$.5%. All these roads contained approximately 3% air voids in the in-place condition. The major objective of these test roads was to determine the influence of asphalt temperature susceptibility on pavement performance. The asphalt properties of these sections are summarized in Table A.21 and the mix properties in Table A.22.

The 1969 daily traffic volumes on the three test roads ranged from 900 to 1,500 with 10 to 18% comprised of trucks. The roads were surveyed for transverse cracks after 8, 9, 10, 11 and 15 years of service. From the 1972 report, McLeod indicates that only Type 1 transverse cracks should be used in relating the cracks with other pavement or environment variables. Type I transverse cracks are those that extend across the full widths of a traffic lane. Cores were also taken to determine mix and binder properties (Tables A.23 through A.25).

Results

The temperature susceptibility of the recovered asphalt was initially determined using the penetration index (P.I.) as developed by Pfeiffer and van Doormaal based on penetration at 77°F (25°C) and the softening point of an asphalt cement. McLeod's analysis indicated that the number of Type 1 transverse cracks decreased with a decrease in the PI value. (Decreasing values of PI indicate increasing temperature susceptibility.) He therefore concluded that the "...PI values do not always provide a realistic measure of asphalt temperature susceptibility". One reason was that one of the asphalts was a Western Canadian waxy light crude. As a result of the wax content, the softening points was too high which gave a temperature susceptibility rating that was too low. A different method of providing a quantitative measure of temperature susceptibility was developed. This method is based on the penetration of an asphalt cement at 77°F and the viscosity at 275°F, hence the term "pen-vis number." McLeod indicated that "...because of the way [the penetration-viscosity number (PVN)] is derived, [it] is numerically similar to, and may be numerically identical with Penetration Index values for many paving asphalts."

From the data for the three test roads, McLeod concluded that the relationship between the amount of transverse cracking that occurred correlated very well with Penetration-Viscosity numbers instead of Penetration Index. Figure 3.25 illustrates that the number of transverse cracks increased with a decrease in the pen-vis number (increasing temperature susceptibility) of the asphalt cement. He also stated that the pen-vis numbers did not change for the life of the pavement (1987). He indicated this is not true for the PI values, which change with time and aging of the asphalt cement.

A chart (modified since 1972) was presented (Figure 3.26) was presented to demonstrate how the penetration at 77°F and the temperature susceptibility of an asphalt cement can be selected to avoid low temperature transverse pavement cracking within a range of minimum service temperatures (+10°F to -40°F) during the lifetime of a well designed and constructed asphalt pavement. For example, when the minimum service temperature anticipated is -10°F, only original paving asphalts with combinations of penetrations at 77°F and viscosities at 275°F that are on or to the right of the oblique line labelled -10°F, would avoid low temperature pavement cracking throughout their service lives. Asphalts to the left of this line are too hard and pavements containing them would be subject to thermal cracking at a minimum service temperature of -10°F. The penetration of the asphalt must be increased as the temperature susceptibility of the paving asphalt increases. Figure 3.26 also indicates that the modulus of stiffness of the paving mixture for each oblique temperature-labelled line representing a minimum service temperature, is constant throughout its length.

In the discussion after McLeod presented his 1972 paper, it was emphasized by Halstead that not only the grade but the temperature susceptibility of the asphalt should be considered in pavement design to mitigate low temperature cracking. His point in support of McLeod's paper was that regardless of whether penetration or viscosity grading is used, the temperature susceptibility is the important variable.

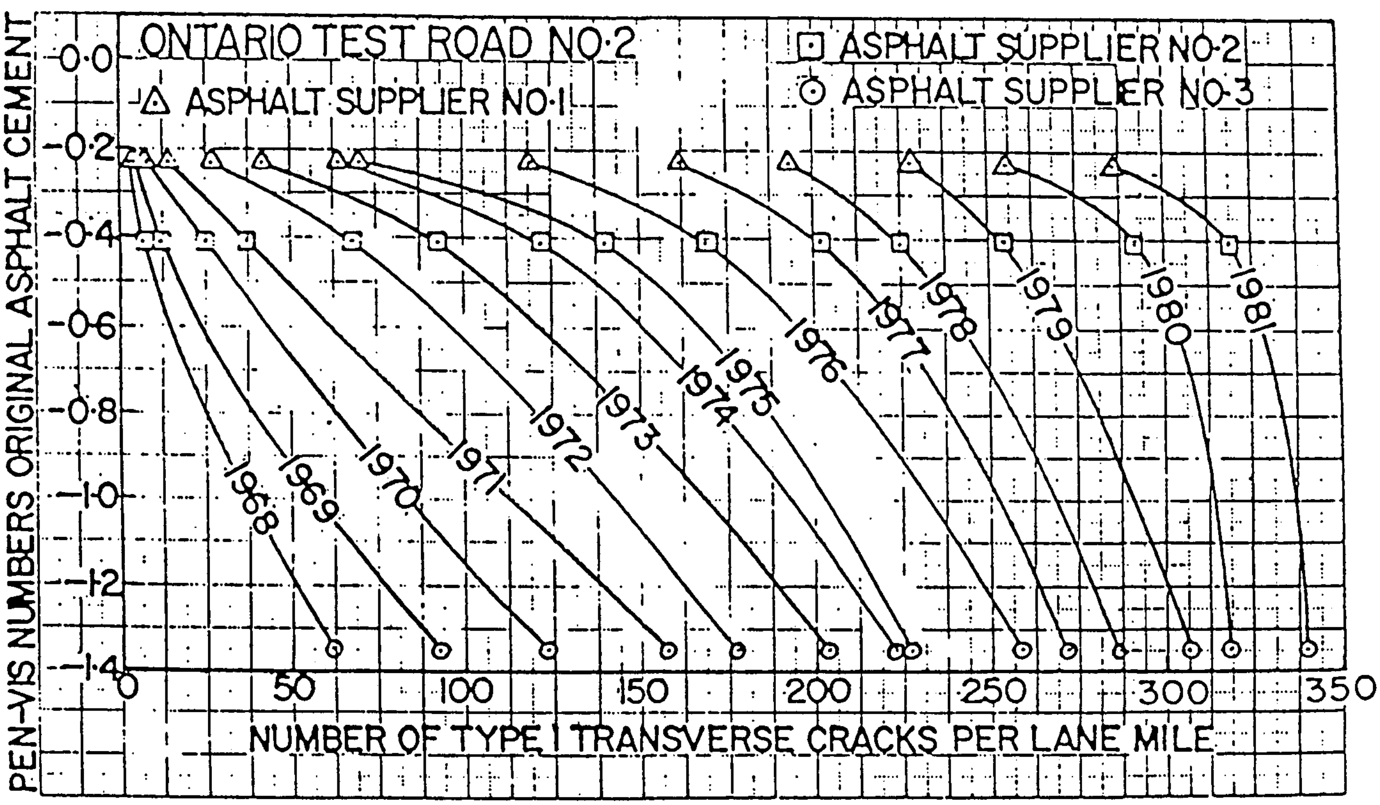


Figure 3.25 Influence of paving asphalt temperature susceptibilities on annual count of Type 1 low temperature transverse pavement cracks per lane mile for Ontario (McLeod 1987)

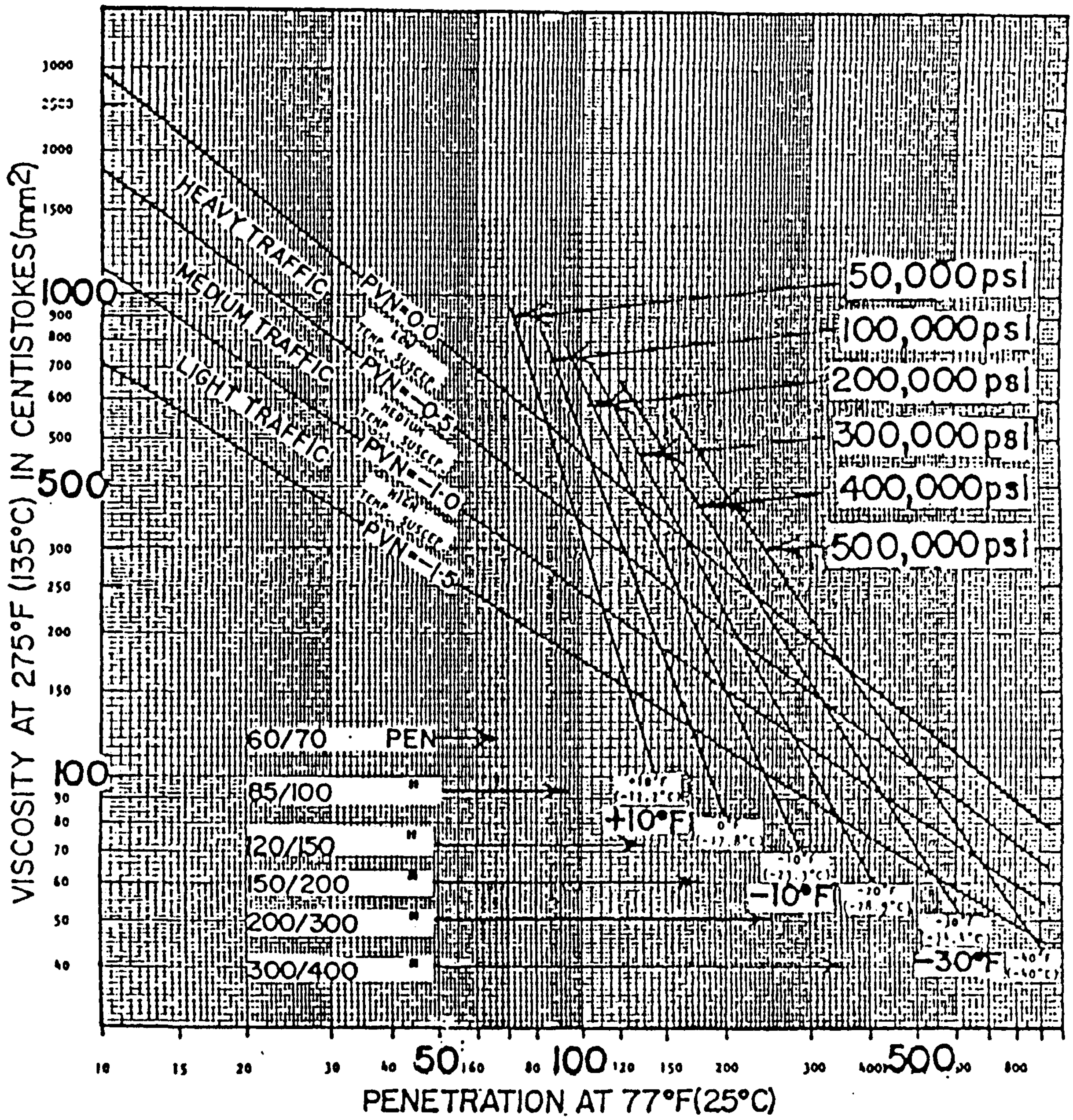


Figure 3.26 Chart for selecting paving asphalts with various combinations of temperature susceptibilities and penetrations at 77°F to avoid low temperature transverse cracking at selected minimum winter temperatures for Ontario (McLeod 1988)

The stiffness modulus of the various recovered asphalt cements was determined using Van der Poel's nomograph as shown in Figure 3.27. The PVN values are used in place of Pfeiffer & van Doormaal's Penetration Index numbers in the nomograph. Asphalt was extracted from pavement samples taken from each of the nine pavement test sections of the three Ontario test roads after 8 and 15 years of service. The average minimum temperature to which the pavements had been subjected during the 8 and 15 year periods were -3.2°F and -3.8°F respectively. In Figure 3.28, the stiffness for each recovered asphalt cement has been plotted against the number of corresponding Type I thermal cracks per lane miles that had occurred during the 8 and 15 year periods. It is clear that the number of low temperature transverse cracks increases with increasing asphalt stiffness. Also, the number of cracks after 15 years for any given test section is greater then after 8 years, as expected.

Figure 3.29 shows a similar graph, except that the modulus of stiffness of the nine paving mixtures are used instead. In both figures, the method of least squares was used to draw the fitted line. The author notes that the data points for the paving mixtures are more closely clustered around the least square line (no further statistical information was given) than for the data showing the corresponding recovered asphalts. For this reason, the author "...claims that the properties of the paving asphalt itself are responsible for ... 85 to 95% of low temperature thermal cracking." However, it was recognized that variables other than the asphalt cement in the mix can also influence low temperature transverse pavement cracking.

McLeod has concluded that low temperature transverse cracking is likely to occur when the modulus of stiffness of a pavement reaches 1 million psi at a pavement depth of 2 inches due to any critical combination of chilling to a low pavement temperature, hardness of the asphalt cement and other controlling factors. This value is based on Van der Poel's nomograph for a loading time of 20,000 seconds and applies to "well designed paving mixtures" with a C_v value of 0.88 (14.5% VMA and 3% air voids). McLeod bases this conclusion on the performance of the Ontario and Ste. Anne test roads, laboratory studies and his observations of the service behavior of asphalt pavements in Canada, the USA and Norway.

In the discussion after McLeod's paper at the CTAA (1978), it was pointed out by W.D. Robertson that "...in general, the PI and the PVN are not numerically equal. This is true for both vacuum reduced asphalts, on which the PVN correlation is based, and air blown products". Data was presented on 88 asphalt cements made from 19 different crude oils. The Penetration Index was calculated for each of these bitumens, based on penetration measurements at 25°, 10° and 4°C. Pen-vis numbers were calculated for the same materials. "The correlation between PI and PVN

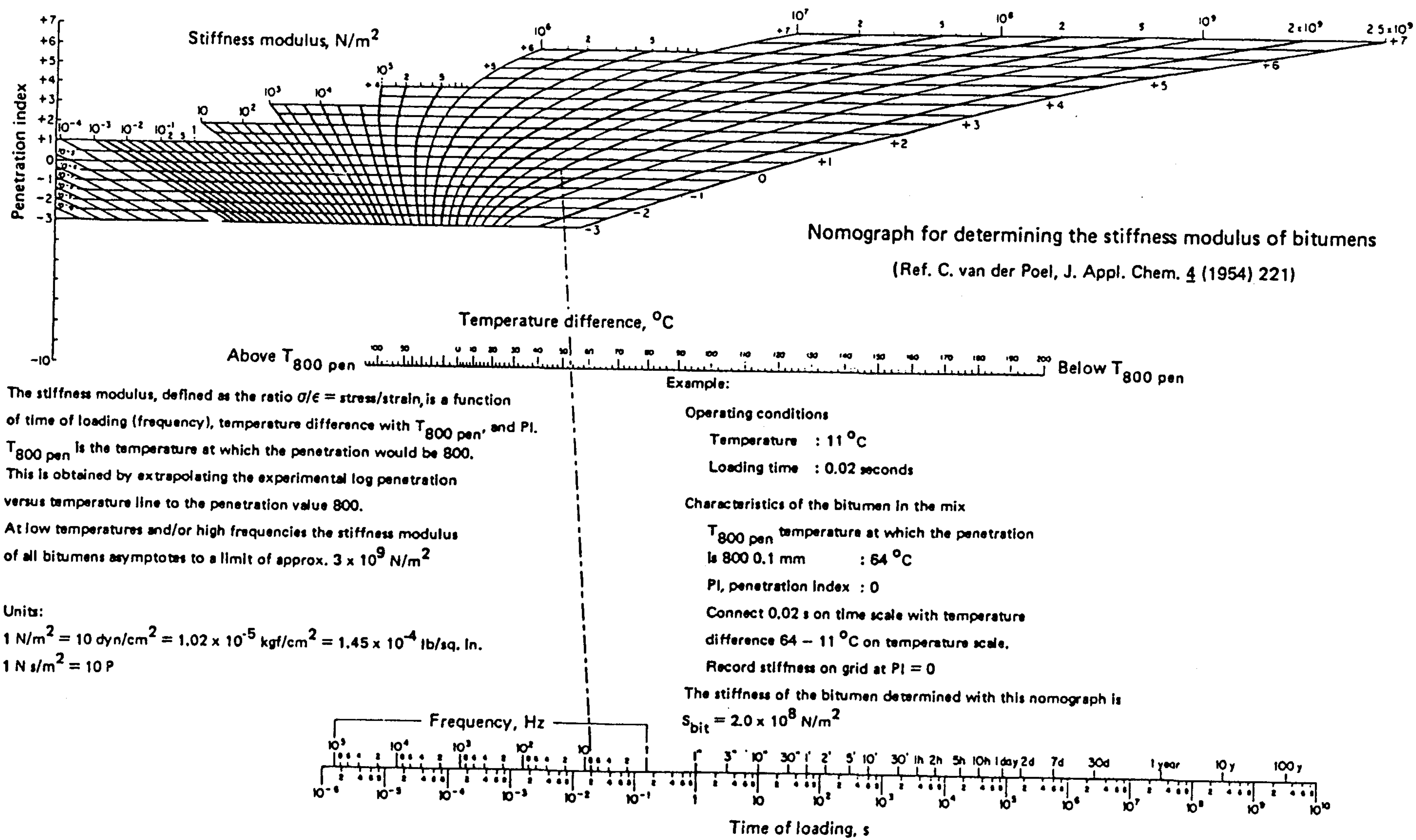


Figure 3.27 Van der Poel's nomograph for estimating stiffness modulus of bitumens (Van der Poel 1954)

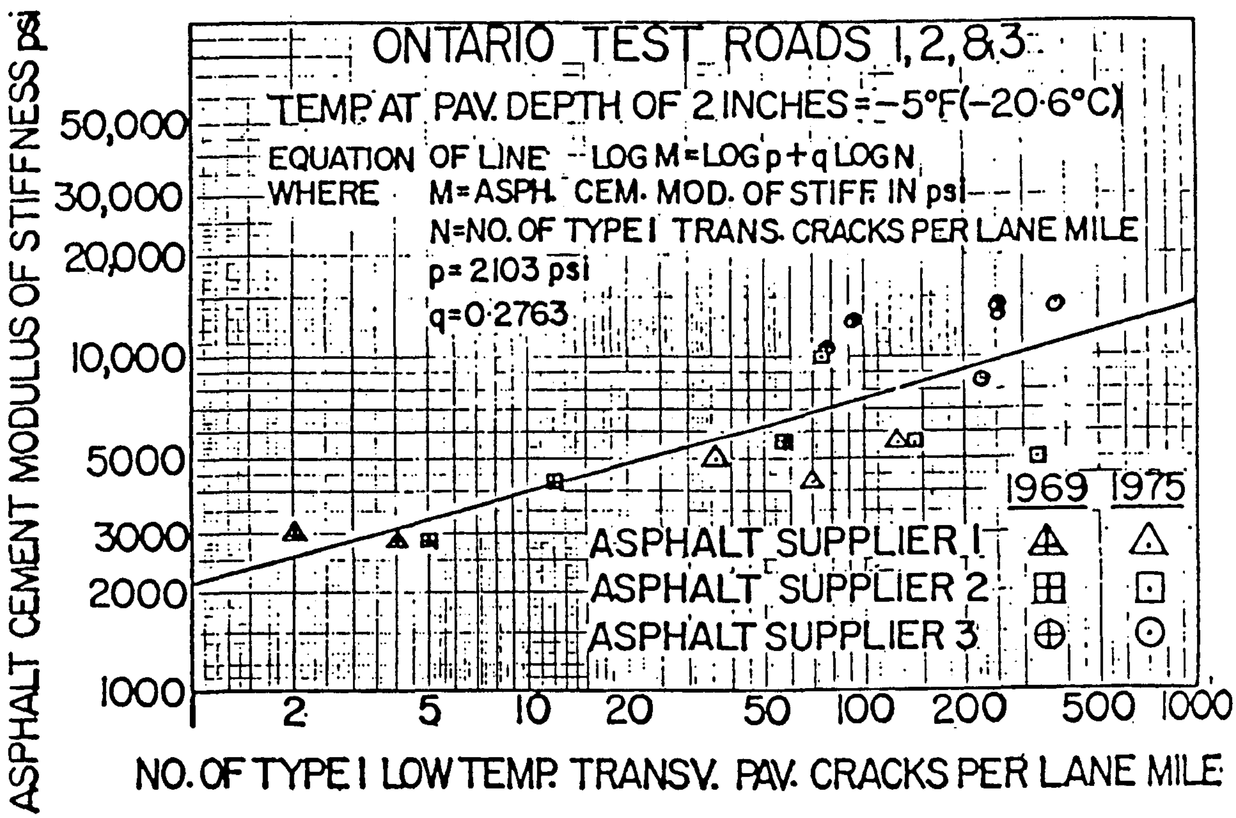


Figure 3.28 Asphalt cement modulus versus number of Type 1 transverse cracks per mile for Ontario (McLeod 1988)

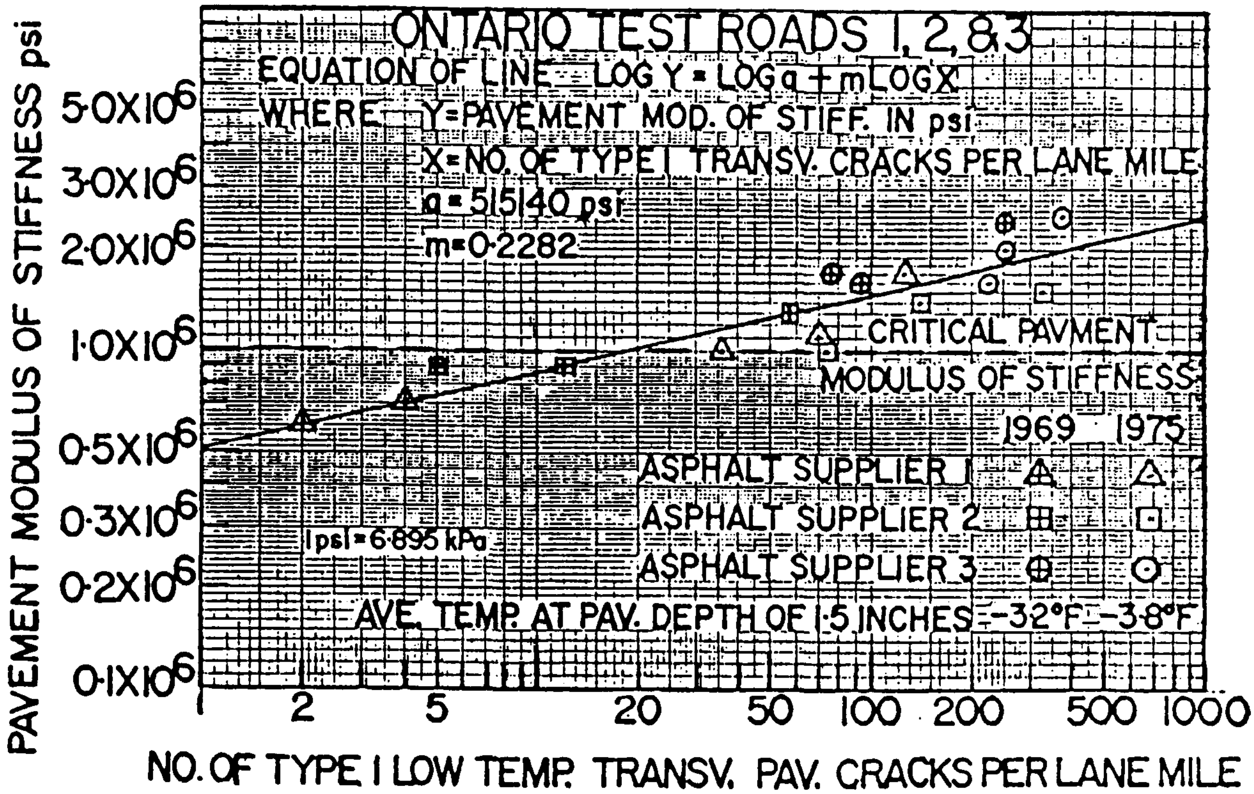


Figure 3.29 Low temperature pavement moduli of stiffness versus number of Type 1 transverse cracks per mile after 8 and 15 years of service for Ontario (McLeod 1988)

is poor (R^2 = 0.53), which indicates that there are other factors, not related to temperature susceptibility in the low temperature region, which affect the high temperature viscosity". Also, all of the factors which cause the temperature susceptibility in the high temperature region to differ from that in the low temperature region are not known; however, one is the presence of wax. Melting of crystalline components as the temperature is increased causes a drop in viscosity, and hence an artificially low PVN value.

McLeod's response was that the close relationship for PI and PVN is valid only for paving grades of asphalt (not air blown materials) produced by steam or vacuum reduction. He also commented that "...asphalt cements manufactured by steam or vacuum reduction from waxy crude oils have had high temperature susceptibilities and therefore low PVN numbers". It was his belief that pavements prepared with these waxy asphalts have developed more low temperature transverse pavement cracks than other asphalt pavements in the same area made with asphalt cements having lower temperature susceptibility -- higher PVN values. Based on this discussion, it was concluded that PVN can be affected by wax content in the asphalt.

Conclusions

From the series of papers presented on the Ontario test roads, the following conclusions can be drawn:

1. The PVN method was developed as a more "realistic" measure of asphalt temperature susceptibility.
2. A chart is presented to select an asphalt cement to avoid low temperature transverse cracking within a range of service temperature (+10°F to -40°F). This chart uses penetration at 77°F and PVN as the basis for selection.
3. As the PVN decreases (i.e. increasing temperature susceptibility) the number of low temperature transverse cracks increases.
4. As the stiffness of the recovered asphalt cement increases, the number of low temperature cracks also increases. For the criteria of not more than 20 transverse cracks/mile, the limiting stiffness is approximately 5,000 psi.
5. A similar relationship exists for the stiffness of paving mixtures. McLeod's limiting modulus of 1 million psi results in approximately 20 low temperature cracks per mile.
6. Wax content in asphalts can give artificially low PVN values.

3.7 UTAH

Description

The Utah study analyzed pavement performance data from two sources in an effort to develop relationships between asphalt properties and field performance (Anderson et al, 1976). The study included 20 controlled test sections where design, environmental factors, and construction were essentially the same. Also included were 108 uncontrolled sections that were evaluated subjectively (visually) over a period of 7 years. Of these, 39 were also evaluated objectively (measurements). The subjective rating included a 1-to-5 rating of transverse, longitudinal, and map cracking, bleeding, polishing, rutting, spalling, and roughness. A rating of 1 indicated severe distress, and a rating of 5 indicated no distress. The objective survey included actual measurements of the number of transverse cracks, length of longitudinal cracks, area of map cracking, rutting, and surface roughness as measured by a PCA roadmeter.

The report deals with asphalt cements from eight sources; American (Salt Lake), Phillips, American (Casper), Douglas, Arizona, Golden Bear, Husky, and Douglas (Santa Maria). The group includes viscosity grades of AC-5, 10, 15, 20, and 40. Objective evaluations were made to compare transverse cracking with (a) the properties of the original asphalt, (b) the properties of the asphalt after the rolling thin-film circulating oven test, and (c) the properties of asphalt recovered from the pavement sections. The ages of the cores ranged from 1.8 to 6.4 years (Figure A.8U).

Results

The following tests were performed on the asphalt cements (original, RTFC and asphalts recovered for selected cores):

1. Penetration at 25°C (AASHTO T-49)
2. Cannon core viscosity at 25°C
3. Absolute viscosity at 60°C (AASHTO T-202)
4. Kinematic viscosity at 135°C (AASHTO T-201)
5. Ductility at 4°C (AASHTO T-51)
6. Rolling Thin Film Circulating Oven Test (AASHTO T-179)
7. Force ductility at 4°F (developed by Utah DOT)
8. Chemical analysis (Rostler)

In addition, the temperature susceptibility was calculated using the following equation:

$$\text{Temp. Susceptibility} = \frac{\log\log \eta_1 - \log\log \eta_2}{\log T_2 - \log T_1} \qquad \text{(Eqn 3-1)}$$

where η_1 and η_2 are viscosity readings at temperature T_1 and T_2. Figure A.8 summarizes the asphalt properties compared to the performance of the pavement (transverse cracks) for the controlled test section. Figure A.9 is similar except that longitudinal cracks are used instead of transverse cracks to rate pavement performance.

Finally, a multiple regression analysis was performed which resulted in the relationships in Table 3.10 between transverse and longitudinal cracking and rutting and asphalt properties, load and age. Figure A.9 also illustrates the longitudinal cracking prediction equation. From Table 3.10, the confidence level indicates the significance of the results, despite the low correlation coefficients obtained. The authors caution that their equations are limited since they assume average conditions. They are based on the average climate in Utah and do not allow for extreme temperature or moisture. Also, subgrade and base variations are not considered although they affect performance. They are recommended as quick, approximate performance indicators.

Conclusions

1. The asphalt source used in a pavement plays a significant role in the performance of the pavement (Figure A.8A). Transverse cracks measured from 0 to 59 cracks per kilometer (0 to 95 per mile) for the different asphalt sources.
2. Percent air voids in an asphaltic mix has a significant effect on the hardening of the asphalt binders (Figure 3.30). Greater changes in asphalt properties such as increases in viscosity and decreases in ductility and penetration occur where the air void contents are high (7.1%). The changes in asphalt properties such as Cannon Cone viscosity for the low air voids level (2.6%) are not as severe (A.I. = Aging Index).
3. The aging indices calculated using the field and original viscosity values at 60°C correlate well with transverse cracking (Figure A.8D). An aging index less than 6.5 results in less than 10 transverse cracks per km.
4. Penetration values for both the original and field samples did not correlate well with the performance parameters (Figure A.8-E,F). Penetration values for the field asphalts were greater than 30 in all but one case, but the pavement performance ranged from excellent to poor.
5. Cannon Cone viscosity values at 25°C (77°F) did not correlate very well with the transverse cracking observed (Figure A.8-G,H).

Table 3.10 Regression equations developed for Utah (Anderson et al. 1976)

	Regression Equation		Correlation Coefficient (R)	Confidence Level
1.	TRANS =	-289 + 73.5 (TS)	0.51	0.1%
2.	LONG =	-2950 + 163 (%AC) + 524 (TS) + .00033 (ACC 18^k)	0.57	1%
3.	RUTS =	0.25 - 0.013 (N) + 0.036 (%AC) + 0.0030 (DUCT) + 0.13 X 10^{-6} (ACC 18^k)	0.54	5%

Where:

TRANS	=	# of transverse cracks per Km
TS	=	Temperature susceptibility measured at 25°C and 60°C of original asphalt
LONG	=	Longitudinal cracking in m/lane-Km
% AC	=	Asphalt content in percent
ACC 18^k	=	Accumulated 18^k axle loads
RUT	=	Rut depth, cm
N	=	Nitrogen bases content
DUCT	=	Ductility

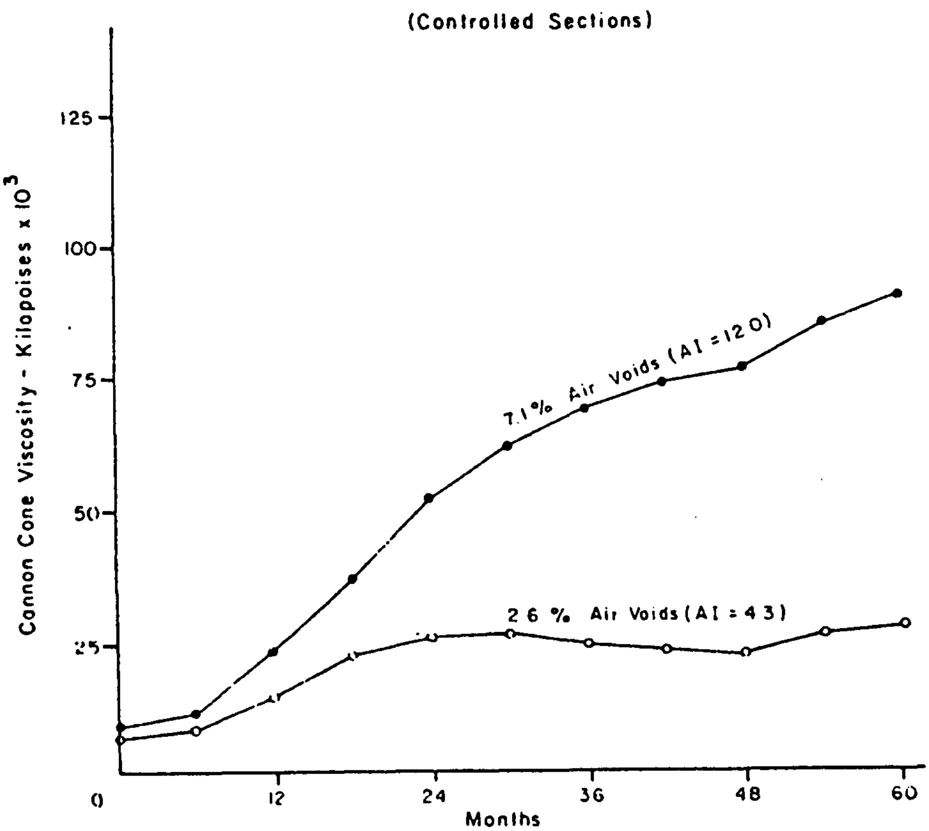


Figure 3.30 Effect of air voids on Cannon Cone Viscosity for Utah (Anderson et al. 1976)

6. The ductility test at 4°C (39.2°F) is not a good indicator of asphalt consistence at low temperatures, and thus low-temperature cracking. However, it was observed that asphalts which maintain a high ductility at a lower temperature generally perform better (Figure A.8-I,J).

7. The Force Ductility test is an improvement of the standard ductility test according to the authors. It increases the repeatability of the test, and measures the consistence in the first ten centimeters of elongation where the cross-sectional areas of the various asphalts are nearly the same. Also indefinite test results such as "100+" are eliminated, and fewer abnormal breakages occur. Nevertheless, the correlation with transverse cracking is poor (Figure A.8-M,N). However, it may be seen from Figure 3.31 that generally, less than 10 transverse cracks per kilometer are found when force ductility values are less than 7 lbs. for original cements.

8. The temperature susceptibility 25° -60°C (77° -140°F) of the asphalts has a poor correlation with transverse cracking (Figure 3.32). However, it may be observed that below a temperature susceptibility value of 4.2, less than 10 transverse cracks per kilometer were measured.

9. Similarly, the paraffin content of the asphalts used in Utah do not show a good correlation with transverse cracking. A paraffin content of less than 15% does, however, indicate that less than 10 transverse cracks/km were measured (Figure 3.33). The N/P ratio also has a fair correlation with transverse cracking and a N/P ratio greater than 2 results in less than 10 transverse cracks per kilometer (Figure 3.34).

10. An excellent correlation exists between the temperature susceptibility and paraffin content of original asphalts (Figure 3.35). Higher paraffins are indicative of higher temperature susceptibility values for the original asphalts. High paraffins are also related to low ductility values and higher Force Ductility parameters.

11. The viscosity values at the various temperatures are higher for the asphalts higher in asphaltenes content. This is true for both original and field samples. Increases in the asphaltenes contents with aging partially explains the increase in viscosity values with aging (Figure 3.36 to 3.39).

12. The source of crude is the major variable affecting the performance parameters, physical properties, and chemical makeup (Figure A.8A,A.9-A).

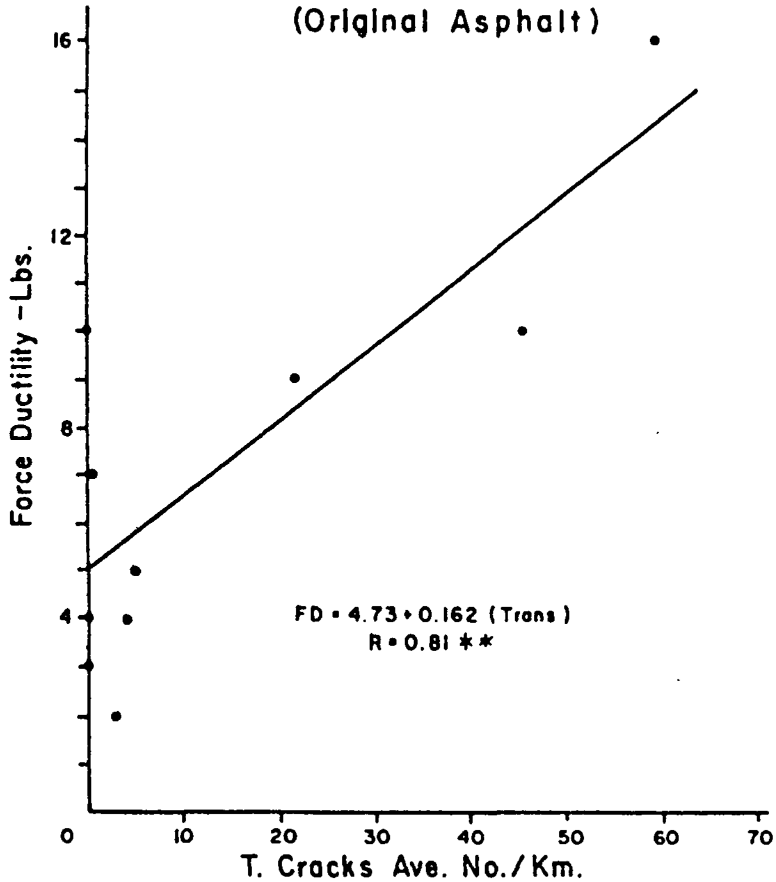


Figure 3.31 Transverse cracks versus force ductility at 4°C for Utah (Anderson et al. 1976)

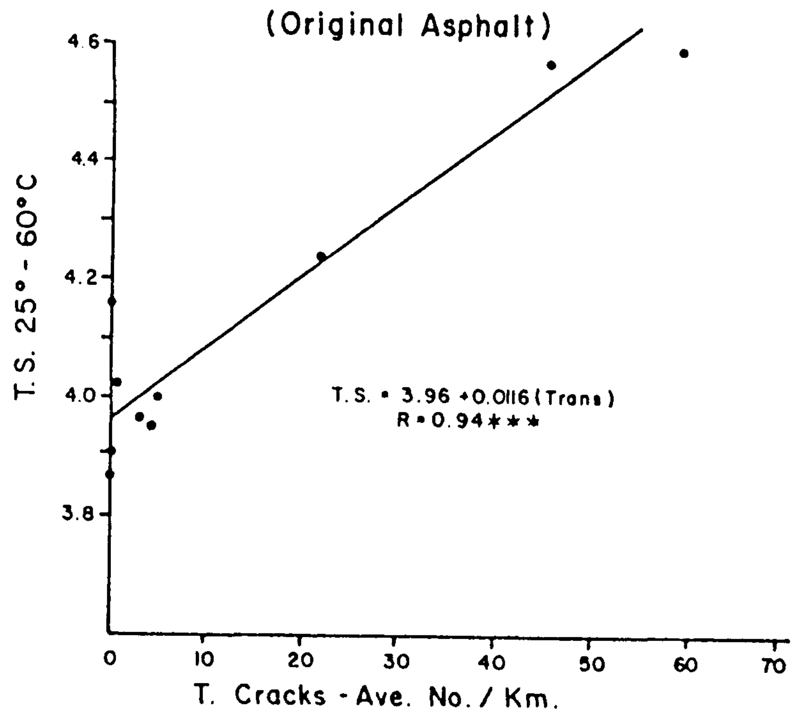


Figure 3.32 Transverse cracks versus temperature susceptibility 77° - 140°C for Utah (Anderson et al. 1976)

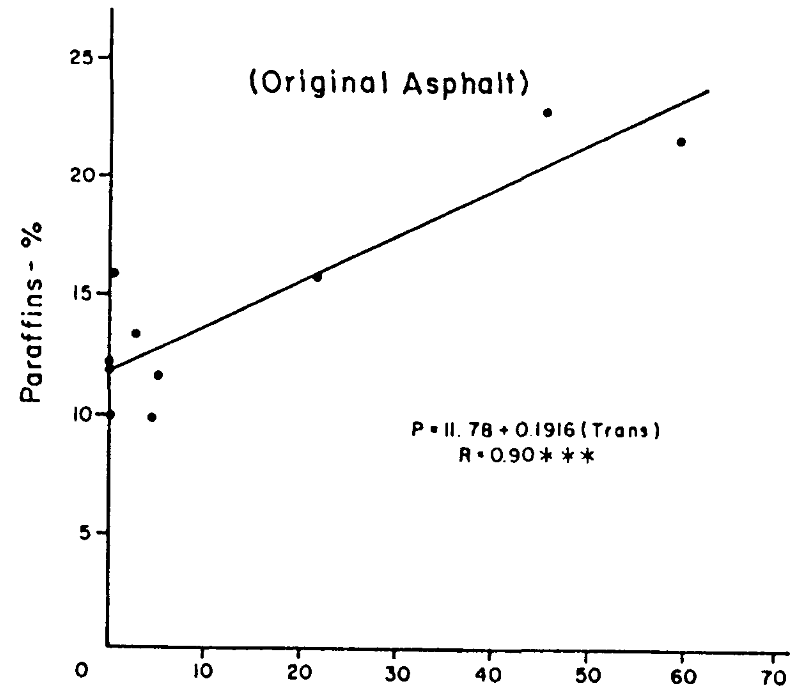


Figure 3.33 Transverse cracks versus paraffins content for Utah (Anderson et al. 1976)

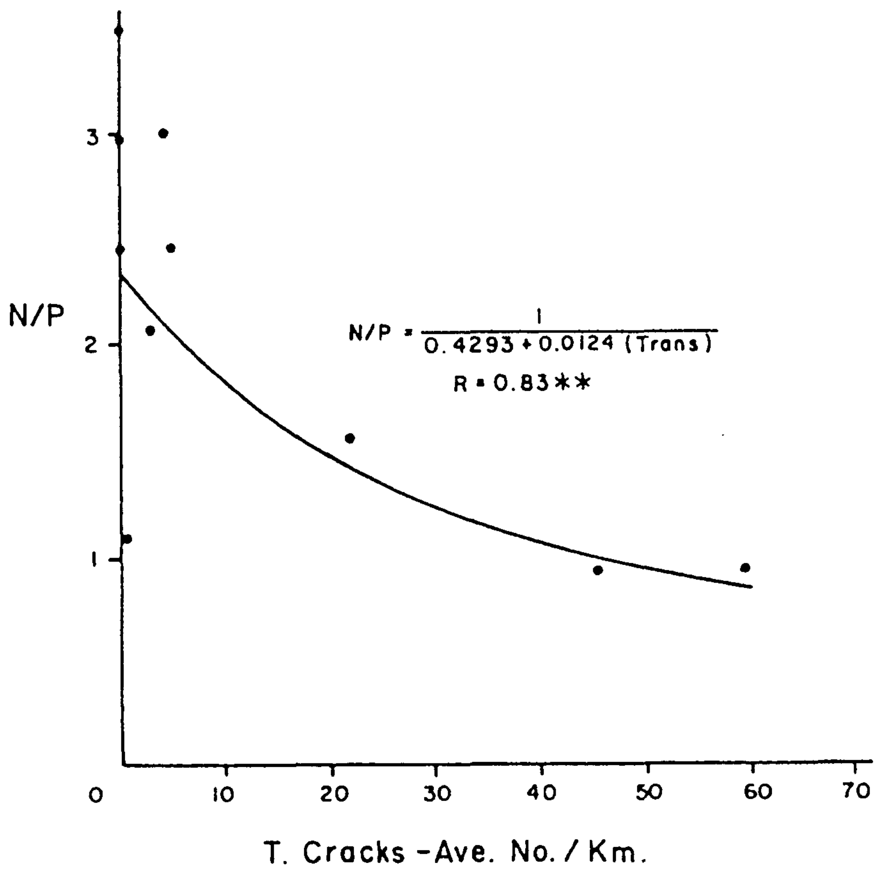


Figure 3.34 Transverse cracks vs nitrogen bases/paraffins for Utah (Anderson et al. 1976)

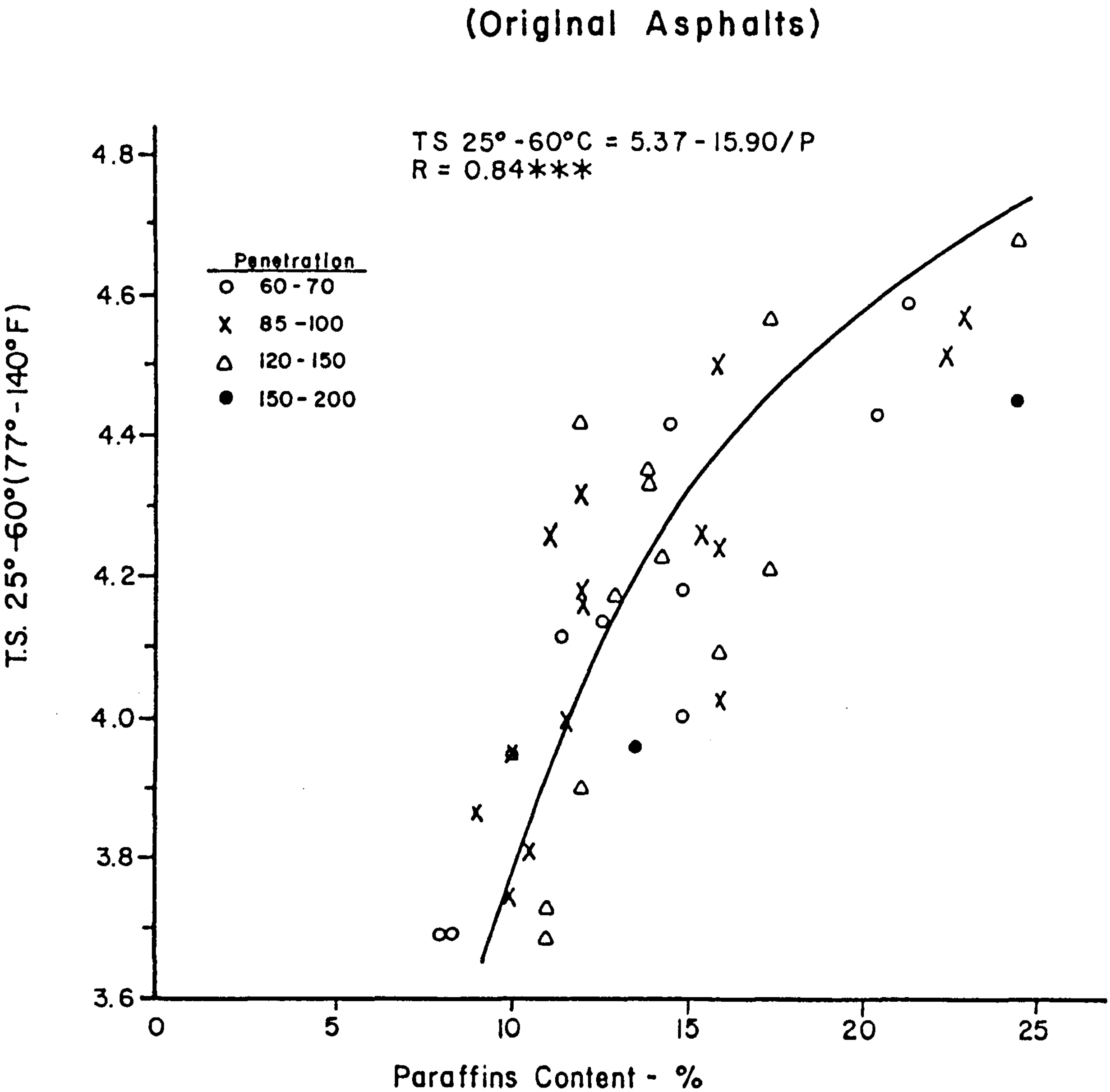


Figure 3.35 Temperature susceptibility vs paraffins content for Utah (Anderson et al. 1976)

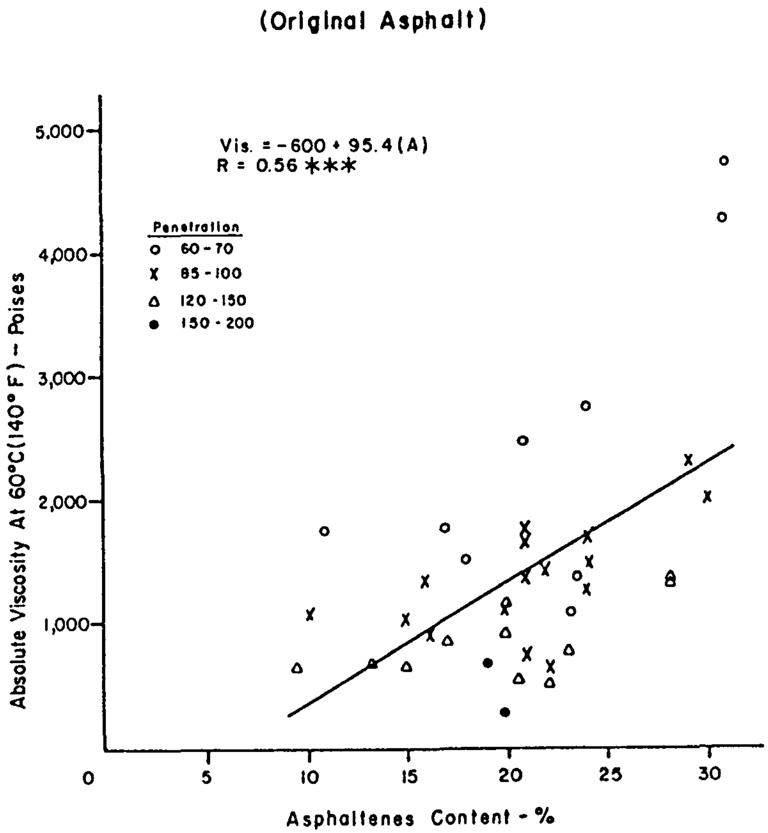


Figure 3.36 Absolute viscosity vs asphaltenes content for original asphalt in Utah (Anderson et al. 1976)

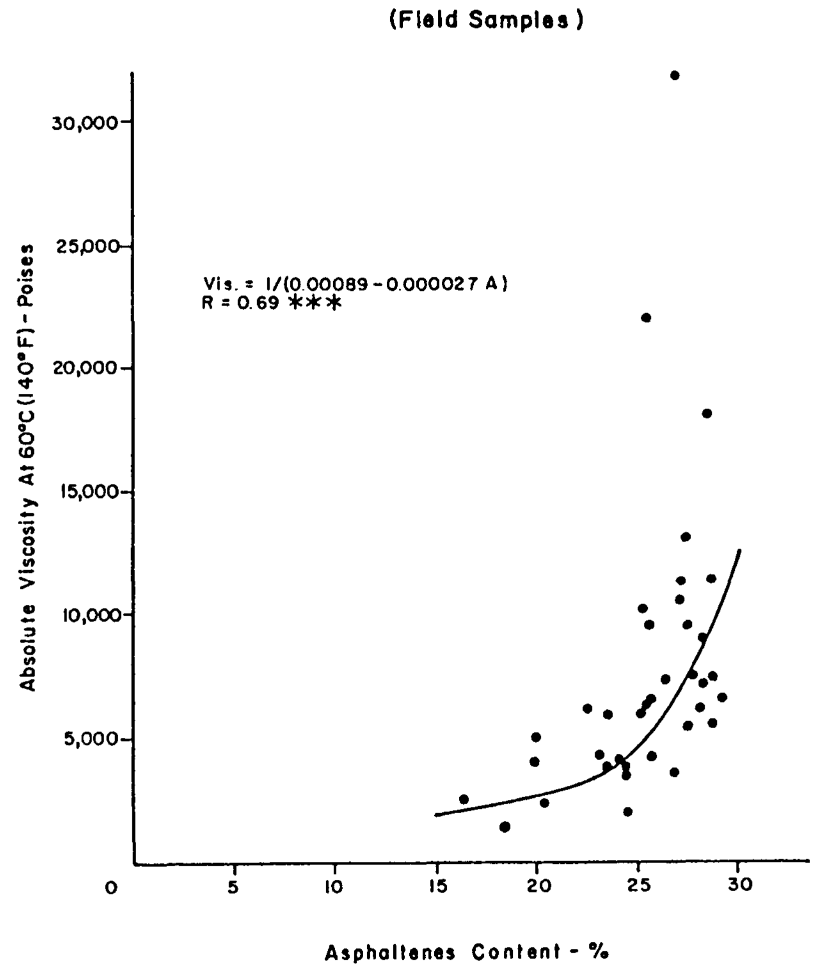


Figure 3.37 Absolute viscosity vs asphaltenes content for field samples in Utah (Anderson et al. 1976)

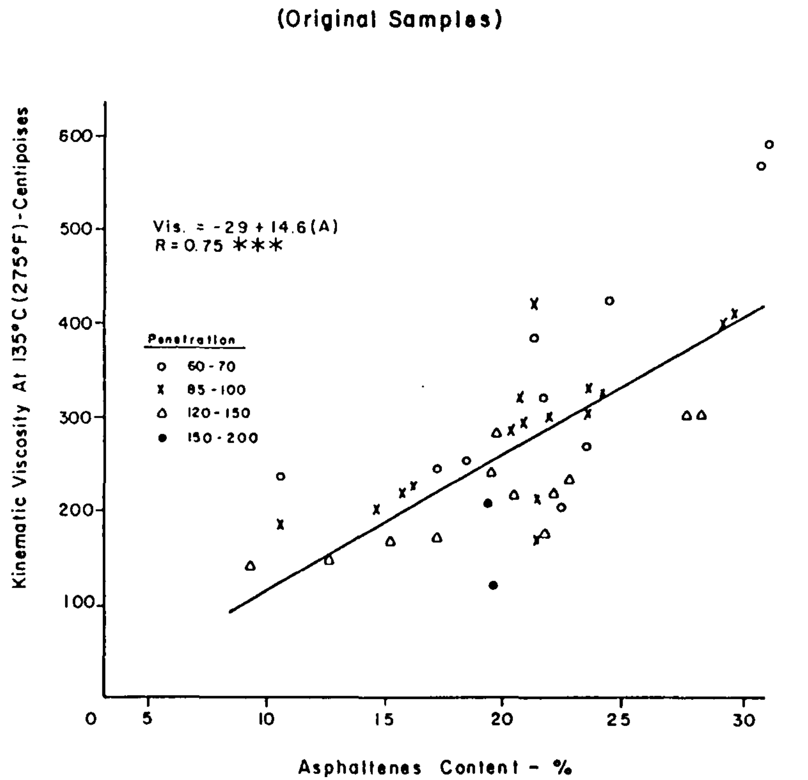


Figure 3.38 Kinematic viscosity vs asphaltene content for original asphalts in Utah (Anderson et al. 1976)

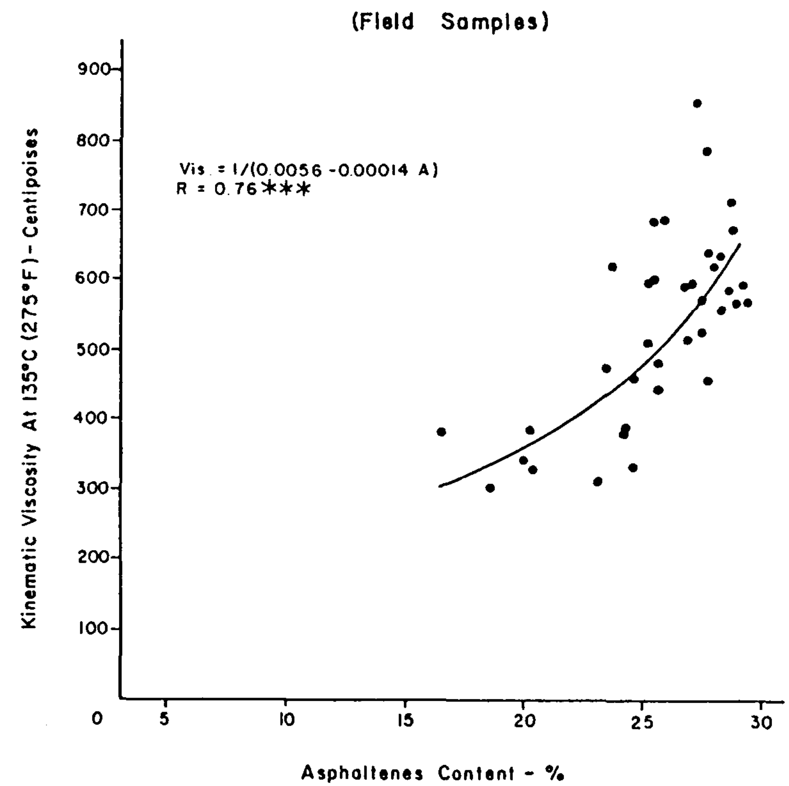


Figure 3.39 Kinematic viscosity vs asphaltene content for field samples in Utah (Anderson et al. 1976)

3.8 SASKATCHEWAN

Description

In 1963, Saskatchewan constructed full-scale in-service test sections in order to evaluate the effect of the asphalt source, the penetration grade of the asphalt cement, and the thickness of the base course and prime coat on the frequency of transverse cracking (Culley 1969). The various asphalts were used on the same construction project. The performance of the 1963 test sections showed a significant relationship between the frequency of cracks and the asphalt supplier. As a result of this research work, specifications for asphalt cement were changed to lower the allowable variation in penetration and viscosity for asphalts from different sources.

A second series of test roads was constructed in 1966 to determine if the new asphalts meeting these specifications showed a relationship between asphalt source and transverse crack frequency. Asphalts from four refineries, conforming to the new specifications, were used on five separate projects. Asphalt concrete thicknesses were 3 inches for both of the Refinery 1 sections and the Refinery 4 sections and 1.5 inches for the Refinery 3 and 5 sections. Hardening of the asphalts during various phases of construction and after 12 months of service was determined. The original and aged asphalt properties are shown in Tables A.26 through A.28.

Results

A comparison was made between the amount of transverse cracking that occurred on the roadway with the amount of hardening that took place in the asphalt cement; however, no definite relationships were found. A number of variables were investigated with regard to their effect on the amount of transverse cracking. Those variables included: (a) the average daily traffic, (b) the freezing index degree days, (c) the air void content of the mix, (d) the retained percentage of the original asphalt cement penetration, (e) the retained percentage of the original asphalt cement viscosity, (f) the temperature susceptibility of the binder, and (g) the shear susceptibility of the binder. The relationship between these variables and the amount of transverse cracks is shown on Table 3.11. It should be noted that only full width transverse cracks were counted. It was determined that none of the variables had a ranking order which completely matched the number of transverse cracks that occurred during the first year of service. According to Culley (1969), the variable that had the closest ranking was freezing index (environment).

It can be seen from the table that sections from Refineries 1 (9-9), 3 and 4 had high transverse crack frequencies grouped fairly close together. Refinery 1 (10-7) had a medium crack frequency and Refinery 5 had a relatively low crack frequency.

Table 3.11 Ranking order of several variables affecting transverse cracking for Saskatchewan (Culley 1969)

Refinery	Trans. Cracks per Mile	1967 ADT	1966-67 Freezing Index Degree Days	Field Air Voids %	% of Orig. Pen.	% of Orig. Visc. @ 60 F	Temp. Susc.	Shear Susc.
				A. After Mixing, R @ 0				
1 (9-9)	137 (4)			5.4 (2)	0.76 (1)	162 (2)	0.99 (4)	4.0 (2)
1 (10-7)	71 (2)			10.5 (5)	0.55 (5)	160 (1)	0.98 (3)	4.3 (3)
3	141 (5)			7.8 (3)	0.75 (2)	219 (3)	0.94 (1)	2.4 (1)
4	131 (3)			4.1 (1)	0.63 (4)	302 (4)	0.97 (2)	9.7 (5)
5	39 (1)			9.5 (4)	0.65 (3)	160 (1)	0.97 (2)	9.0 (4)
				B. After 12 Months Service, R @ 12				
1 (9-9)	137 (4)	1500 (4)	6294 (5)		0.49 (2)	339 (2)	0.90 (1)	10.8 (1)
1 (10-7)	71 (2)	1200 (2)	5165 (3)		0.46 (4)	356 (3)	0.93 (4)	19.0 (5)
3	141 (5)	900 (1)	6018 (4)		0.46 (4)	679 (5)	0.91 (2)	14.2 (2)
4	131 (3)	1200 (2)	4918 (2)		0.50 (1)	405 (4)	0.92 (3)	14.3 (3)
5	39 (1)	1300 (3)	4323 (1)		0.48 (3)	265 (1)	0.92 (3)	17.8 (4)

NOTE: Numbers in parentheses indicate rank in "best"-"worst" sequence of (1)-(5).

Conclusions

After one year of data, there was no definite relationship between the changes in asphalt properties and transverse cracking. Culley (1969) concluded that transverse cracking is probably more affected by the degree of compaction and by environmental conditions than by changes in a single variable such as penetration or viscosity during handling or mixing.

During the discussion of the paper after it was presented at AAPT, McLeod argued that, based on his observations in Ontario, transverse cracking is highly influenced by location and that it was virtually impossible to relate transverse cracking to asphalt characteristics only unless the test pavements are constructed at a single uniform test site. He added that the original asphalt properties (not recovered) were more closely related with observed performance in Ontario. McLeod stated that he "...was not surprised at the findings..." of the Saskatchewan study based on his experiences at Ontario.

3.9 IOWA

Description

In 1966, a research project was initiated in Iowa (Lee 1973) as a long-range comprehensive program to study asphalt durability. Its ultimate objective was to develop a simple and rapid laboratory durability test to select asphalts, to identify inferior asphalts and to reasonably predict the useful life of asphalts in pavements.

Eight asphalt pavements formed the basis for a study of changes in rheological and chemical properties for 48 months of service. These pavements were constructed in 1967-1968, and samples of asphalts, mixtures and pavement cores were taken during construction. Cores were removed from the wheel paths and from between wheel paths at 6-month intervals for 48 months. The sections were located on US 63, US 75, US 52 and State highways 327, 3 and 92. After 48 months, all of the pavements were in good condition except for some transverse and centerline cracks.

The 8 asphalts came from 4 sources:

1. Blend of Casper, Wyoming and Big Springs, Texas
2. Sugar Creek, Missouri
3. Esso Research and Engineering Company
4. Big Springs, Texas

Results

The asphalts were analyzed for penetration at 77°F (25°C), viscosity at 77°F and 140°F (60°C), microductility and softening point. They were also subjected to the Thin-Film Oven Test (TFOT), the Iowa Durability Test (IDT) and chemical analysis by the Rostler Method. Infrared spectra were obtained through the use of multiple internal reflection techniques. Tables A.29, A.30 and A.31 in Appendix A contain additional details on the project locations, mix characteristics and asphalt properties, respectively.

In the IDT, the residue from the 1/8-inch TFOT (from Bureau of Public Roads) was exposed to a pressure-oxygen treatment of 20 atm oxygen at 150°F (66°C) for up to 1,000 hours. The pressure-oxygen treatment was conducted in pressure vessels fabricated of 0.5 inch stainless steel. These vessels are capable of simultaneously treating 10 standard TFOT samples to a pressure of 450 psi. The changes in the physical and chemical properties of the asphalt were determined after 24, 48, 96, 240, 480 and 1,000 hours of service, and for the recovered asphalts at 6-month intervals. Changes in the penetration, viscosity at 77°F and 140°F with time are shown in Figures 3.40, 3.41 and 3.42 respectively. (The results are also tabulated in Tables A.32 - A.35). The "d" data points are for the lab-aged samples, and the "f" data points are field-aged.

As shown on these figures, the property changes followed the hyperbolic model (except for a few instances in ductility). That is to say, penetration and ductility decrease, viscosity and softening point increase with time, and finally approaching definite limiting values. This agrees with actual field hardening. Specifically, Lee found that the development of aging followed a hyperbolic model as suggested by Brown et al. (1957):

$$\Delta Y = \frac{T}{a + bT}$$

Where ΔY	=	change of property with time T or the difference between the zero-life value and the value at any subsequent time
a	=	constant; the value of the property at zero time
b	=	rate of change of the property
1/b	=	the ultimate change (limiting value) of the property

Figures 3.41 and 3.42 show high correlation coefficients between curves developed with this model and the observed data from the laboratory and field tests. Only in the case of ductility data was this model inadequate.

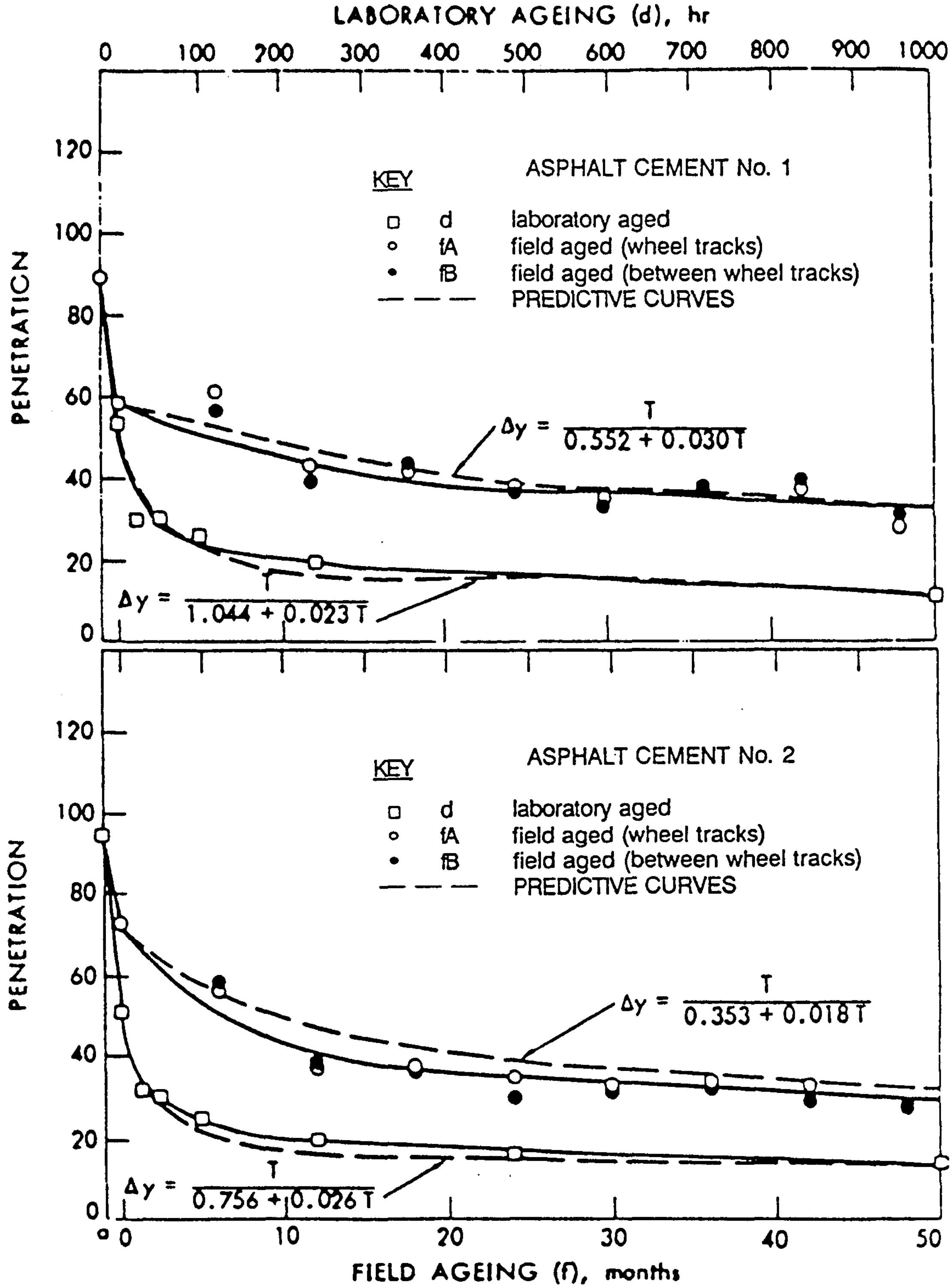


Figure 3.40 Penetration vs time of aging for Iowa (Lee 1973)

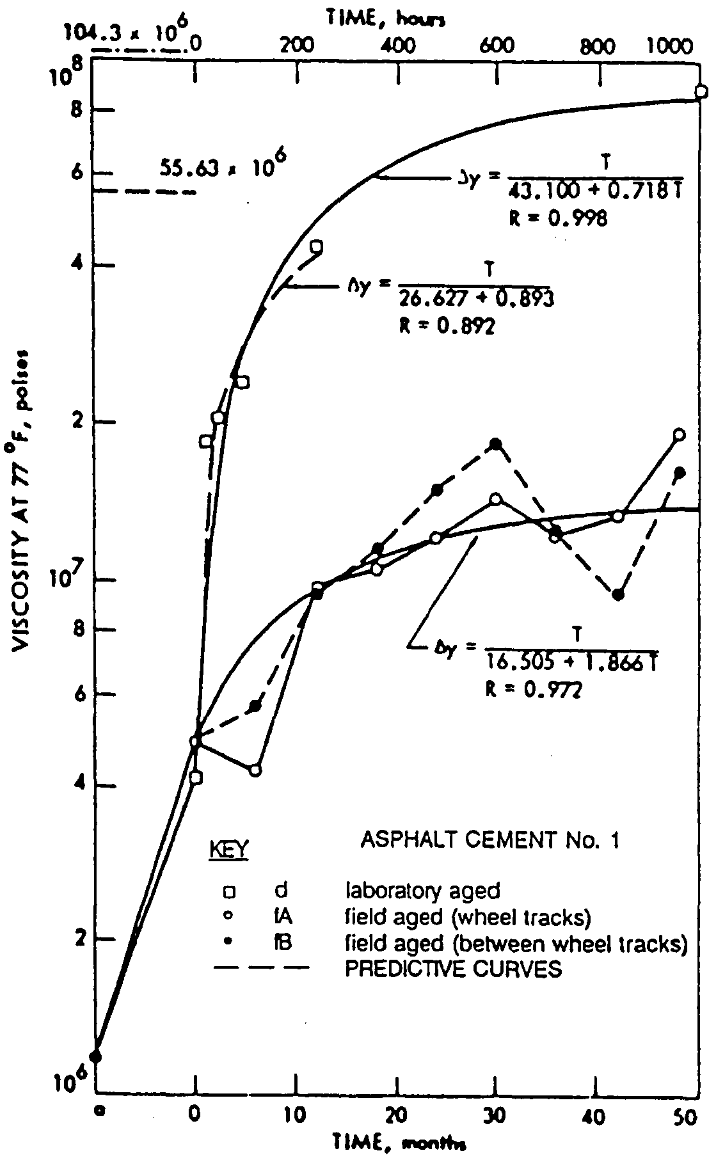


Figure 3.41 Viscosity at 77°F vs time of aging for Iowa (Lee 1973)

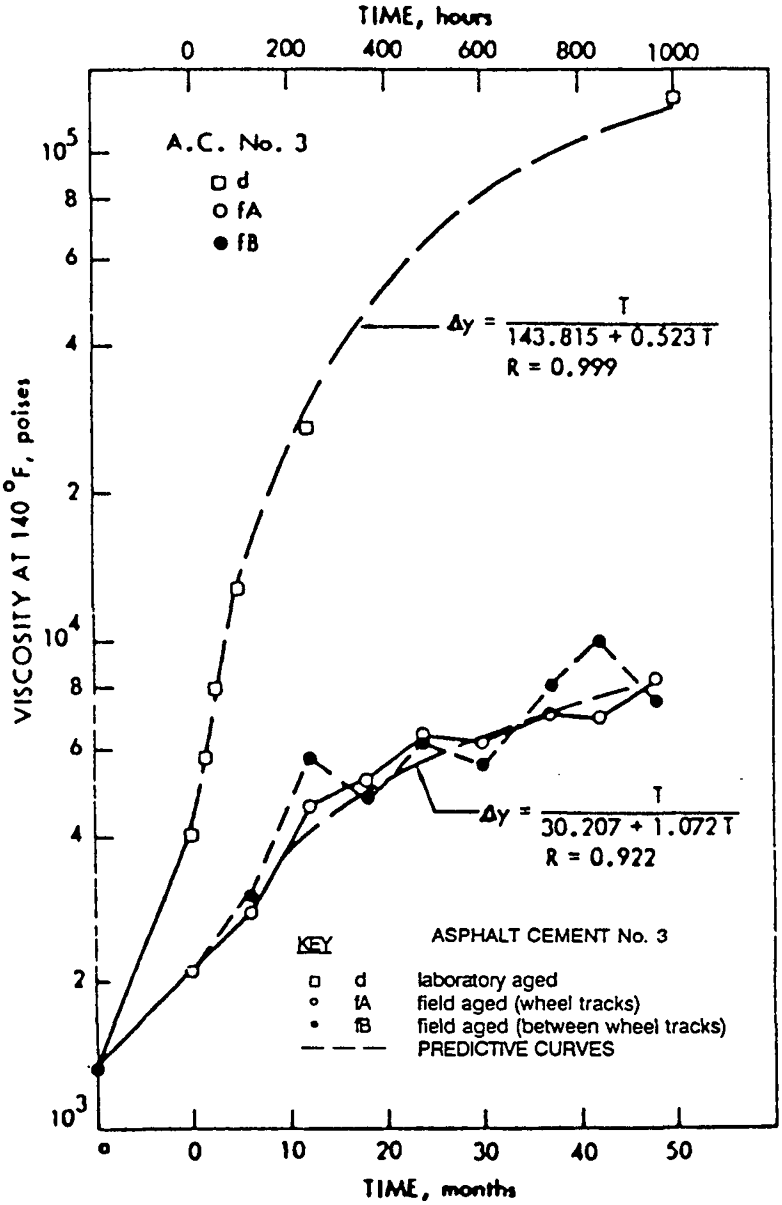


Figure 3.42 Viscosity at 140°F vs time of aging for Iowa (Lee 1973)

The Expert Task Group (ETG) has observed that there are several questions arising from these figures with regards to the location of zero and "a" on the abscissa. However, the author does not explain what these symbols are in his report. The author also suggests that the IDT is realistic and that the correlation between field hardening and hardening of asphalt in the IDT is possible.

From Rostler's method of chemical analysis, it was found that:

1. Aging is accompanied by an increase in asphaltene content.
2. Change in asphaltene content is a hyperbolic function of time.
3. Rates of asphaltene content formation are different for different asphalts.

The Rostler parameter $[(N + A_1)/(A_2 + P)]$ which has been correlated with asphalt durability in other studies (Rostler & White 1962, Halstead et al. 1966) was not borne out in this report. No pattern of change could be defined.

General correlation curves were developed for each asphalt property -penetration, softening point, viscosity at 77°F and 140°F, asphaltene content and microductility (Figure 3.43). During the study, it was also found that void content affects the hardening rate. As void content increases, longer laboratory IDT time is required to reach equivalent field-service hardening (Figure 3.44). A master time-equivalency curve was developed (shown as the curve between 3% voids or 4-7% voids) from regression analysis that combined all properties and asphalts. On the basis of this curve, Lee concludes that 46 hours of aging in the IDT will age asphalts to the equivalent of 60 months in Iowa pavements.

<u>Conclusions</u>

The following conclusions were made:

1. The aging process follows a hyperbolic function of time both in the field and in the IDT lab conditions, but at different rates.
2. Good correlations between field service aging in Iowa and lab aging during IDT have been obtained. A master time-equivalency curve between IDT in hours and pavement service life in months was developed. Correlation curves for different properties and different levels of air voids were also obtained.

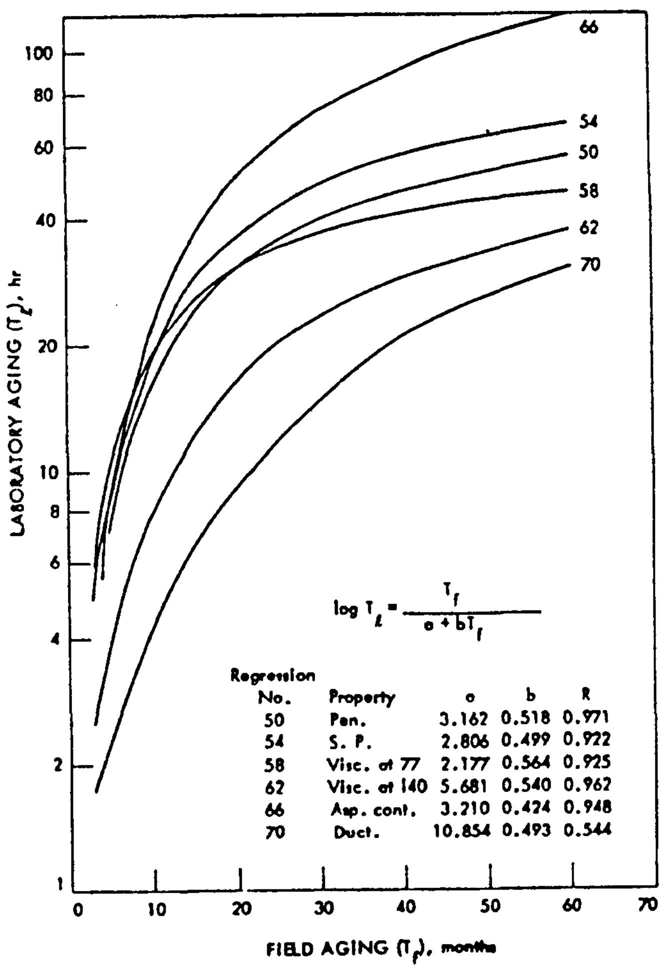


Figure 3.43 Time-equivalency correlation curves for various properties for Iowa (Lee 1973)

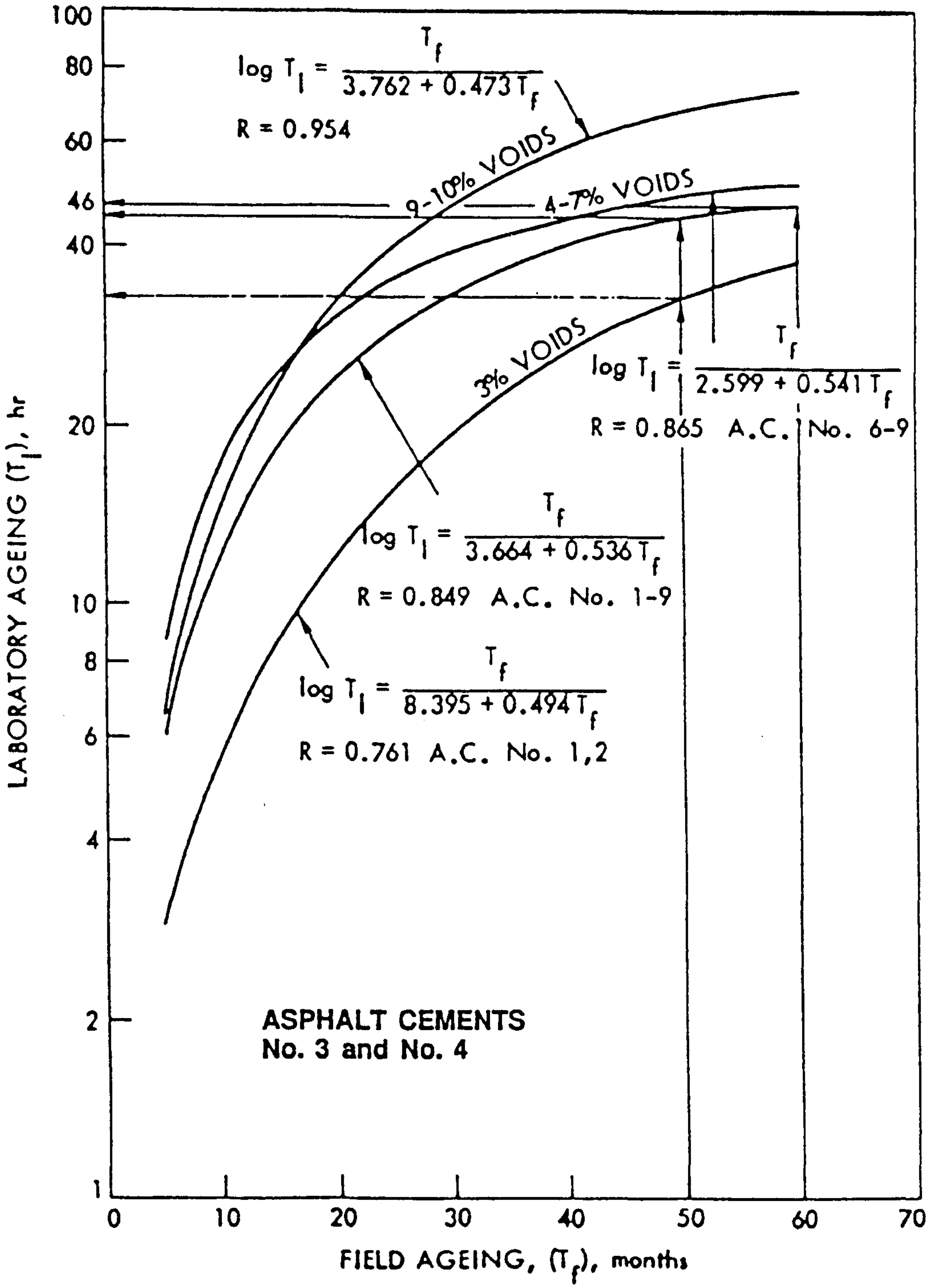


Figure 3.44 Time-equivalency correlation curves by voids level for Iowa (Lee 1973)

3. Aging is accompanied by an increase in asphaltene content. The Rostler parameter did not exhibit any pattern of change with age.

3.10 DELAWARE

Description

An experimental test section was constructed on FAS road 30 in Delaware in 1958 (Kenis 1962). It should be noted that both a rigid and a flexible base was present in the test section. The test sections were designed as asphalt overlays on rigid and flexible bases. The objective of the test road was to obtain data on asphalt test characteristics and the service behavior of asphalts from different sources. Three different laboratories were used to obtain these results: Bureau of Public Roads (BPR), the Asphalt Institute (AI), and Delaware State Highway Department (DEL).

Two asphalts of 60-70 penetration from 2 sources (Venezuela-Lagunillas and Middle East-Sataniya) were used. Both asphalts were vacuum-distilled. Properties of the original asphalts and aggregate characteristics were obtained at the time of construction and the results shown in Tables A.36 and A.37. From the results, it was apparent that there was little difference in the visual physical properties of the 2 asphalts. Both asphalts also had the same resistance to hardening in the thin-film test, although there was a difference in the ductility of the thin-film residue. The asphalt contents (Marshall method) were 5.1 to 5.5% for asphalt A and 5.2 to 5.5% for asphalt B.

Results

After 2 years of service, cores were taken and tested. The results are shown in Table A.38. The authors reported that the penetration of the recovered asphalt generally decreased with age (Figure 3.45). After 2 years, penetrations were less than 25 and yet the pavements exhibited no distress. In addition, the softening point increased with age (Figure 3.46). The three different lines in each graph relate to the three different laboratories used to measure asphalt properties.

Conclusions

Aside from the above, Kenis stated that "...at present no specific conclusions can be drawn, but the lab tests indicate that variances in behavior of the same asphalt at different locations...may be as great as or greater than variances in the asphalts from the different crude sources used in this study".

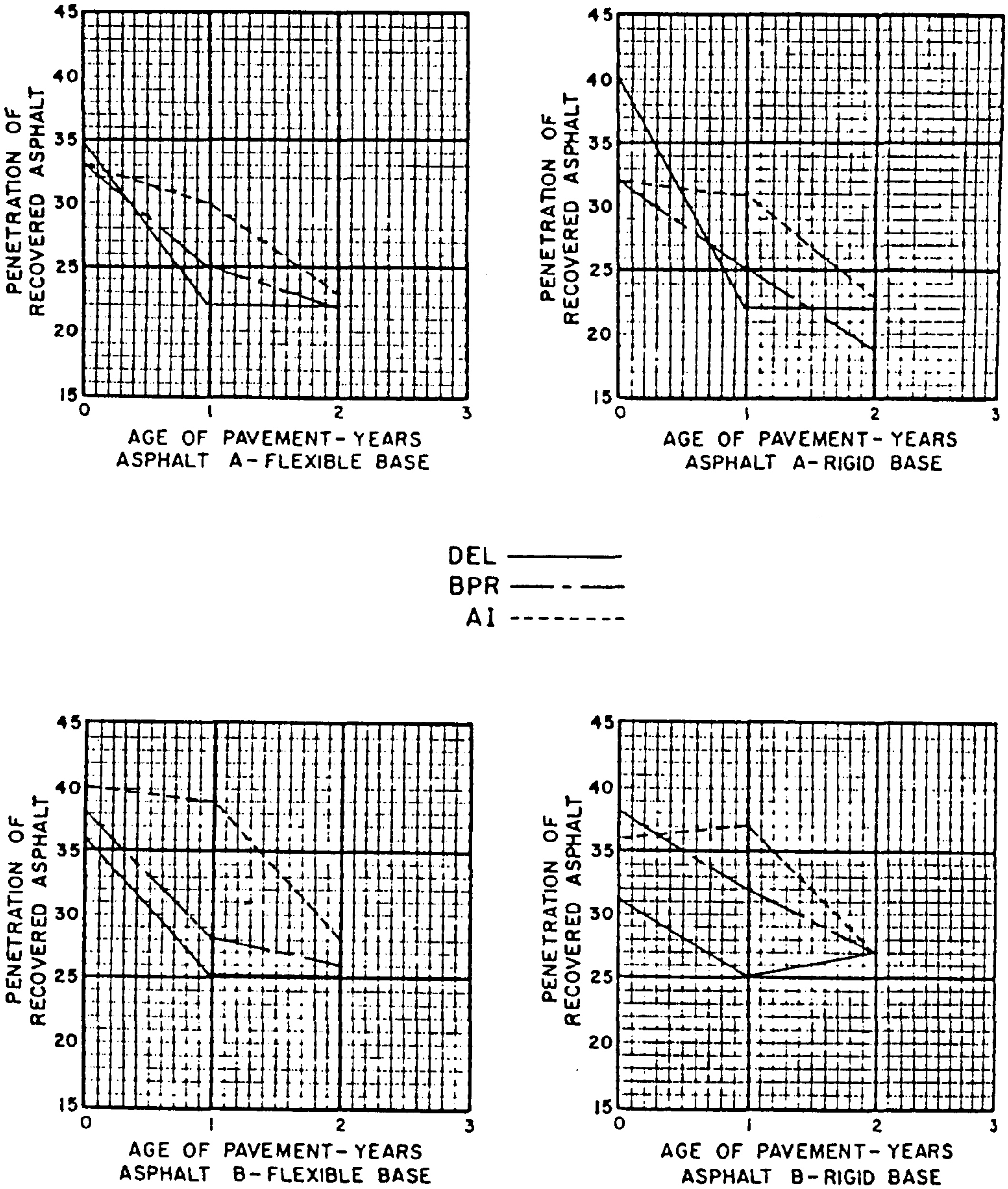


Figure 3.45 Penetration of recovered asphalt, with years of service for Delaware (Kenis 1962)

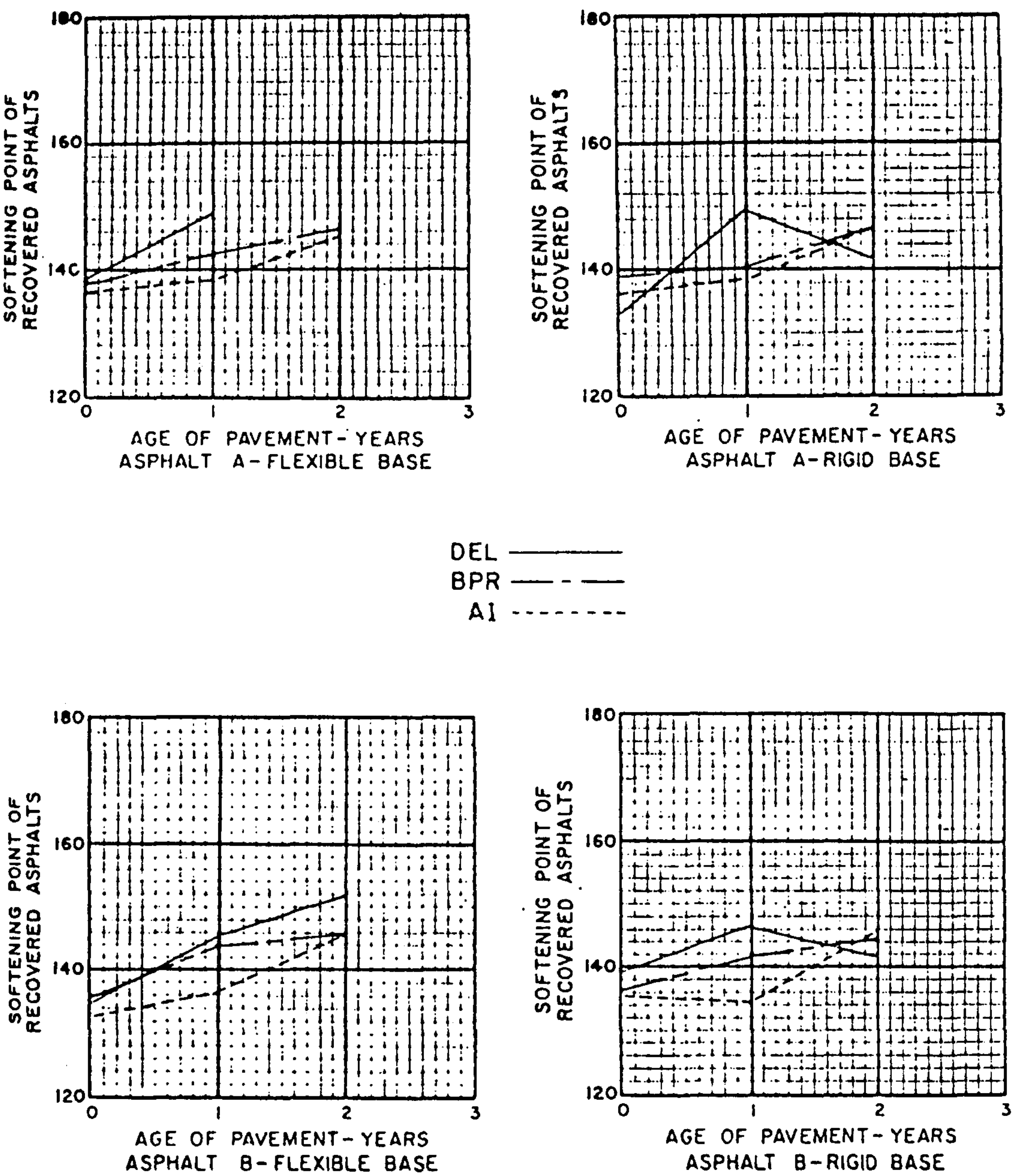


Figure 3.46 Softening point of recovered asphalt, with years of service for Delaware (Kenis 1962)

In addition, no deterioration in the pavement was noted. Welborn (1979) reports that the study continued for 103 months, at which time the pavements were still performing satisfactorily. The recovered asphalts were tested and the results found to support Kenis' statement.

3.11 MONTANA BIG TIMBER TEST ROAD

Description

In 1983, Montana constructed a series of 20 test sections on Interstate 90 in the south-central plains of Montana to compare the performance of asphalt cements from each of the states four refineries (Jennings et al. 1988, Bruce 1987). Three of the refineries were in the Billings area in south-central Montana and the fourth was in Great Falls in the north-central part of the state. It was noted by Jennings et al. (1988) that no two of these refineries used the same crude blend, although parts of the crude slates of the Billings area refineries may overlap.

Two of the refineries used a propane deasphalting (PDA) process for all or part of their asphalt production. The other manufacturers made asphalt to grade from the vacuum tower, although both may occasionally blend to achieve a desired grade. The refineries in the study are identified as refiners A, B, C and D; however, no relationship between refinery ID and refining process was given. The asphalt grade, with only three exceptions, was 120-150 pen asphalt which met Montana's standard specifications. The exceptions were an 85-100 asphalt section from refiner B, a section with 200-300 pen asphalt from refiner B containing ChemCrete and a 200-300 pen asphalt from refiner B containing microfil 8 (carbon black).

Details regarding the construction of the test sections is described by Bruce (1987). The pavement sections were all new construction, consisting of 0.4 feet of asphalt concrete surface placed in two lifts over 1.35 feet of crushed base course. The subgrade was prepared as uniformly as possible over the length of the site and had an average R-value of 25. This site was long enough to permit the construction of 20 test sections, each 1,250 feet long, separated by 500-foot transition zones. Ten sections were placed on the eastbound lanes and ten for the westbound lanes. Traffic on the sections is similar in both directions. The traffic (one direction) in 1985 was given as 700 equivalent 18,000 lb. axle loads annually (ADT of 4800). Temperatures during mixing were controlled to produce an asphalt viscosity of 170 ± 20 centistokes.

The aggregate type and gradation remained constant and was tightly controlled throughout the project. Test sections containing 1.5 percent fly ash, 1.5 percent hydrated lime, and 0.5 percent liquid anti-strip additive (ACRA) were constructed from each asphalt source in addition to control sections with no additives. A test section containing a specially blended asphalt composed of 75 percent asphalt B and 25 percent asphalt D, and thought to have an ideal high-performance gel permeation chromatography (HP-GPC) trace was also included. Each test section received an individual mix design (Marshall method) in an effort to achieve constant stability and voids content in the pavement throughout the sections. The mixture variables for each test section are shown in Table A.39. Cores were obtained immediately after construction and the associated recovered asphalt properties and mixture properties are shown on Table A.40. More complete data on mixtures is summarized by Bruce (1987).

Results

During the first winter, severe transverse cracks developed on Sections 5, 6, 7, 14 and 15. The following summer, some of these cracks "healed" under the influence of traffic and the sun. Nevertheless, these sections remained more severely cracked than the others. Performance data obtained 4 years after construction are shown on Tables A.41 and A.42. Test section 19, which included the ChemCrete additive, does not appear on the table since it was overlaid due to extreme rutting caused by design and construction problems. Test section 18, the only section containing 85-100 pen asphalt, contained more cracking and less rutting than the 120-150 pen asphalt from the same source. Table 3.12 gives the penetration-viscosity numbers (PVNs) of both the original asphalts and the asphalts recovered from postconstruction cores along with the number of cracks and depth of ruts in the corresponding sections with no additives.

Corbett fractionations were conducted on the original asphalts. The fraction percentages are given in Table 3.13 in relation to the number of cracks and depth of ruts (inches) in the corresponding test section without additives. The authors cautioned, however, that the asphalt content has a fairly strong and perhaps overriding influence on rutting relationships.

A major variable to be considered in these experimental sections was asphalt source. In Figure 3.47, the rut depth in inches is plotted against the components of the test section. It may be generalized that relative resistance to rutting increases in the order A < B << C < D. The authors noted that asphalts C and D were associated with higher asphaltene content but cautioned that the relationships may be complicated by the influence of asphalt content. A similar plot of total number of transverse cracks versus components of the test section is shown in Figure 3.48. In general, it may be seen that the resistance to cracking of the asphalts increases in the order B < A < D << C. Here the authors note that asphalt C contained relatively lower concentrations of napthene aromatics. The special blend was found to resist cracking, although it was composed of 75 percent B and 25 percent D, both of which are materials that tend to crack when used alone.

Table 3.12 Relationship of PVN and performance for Montana (Jennings et al. 1988)

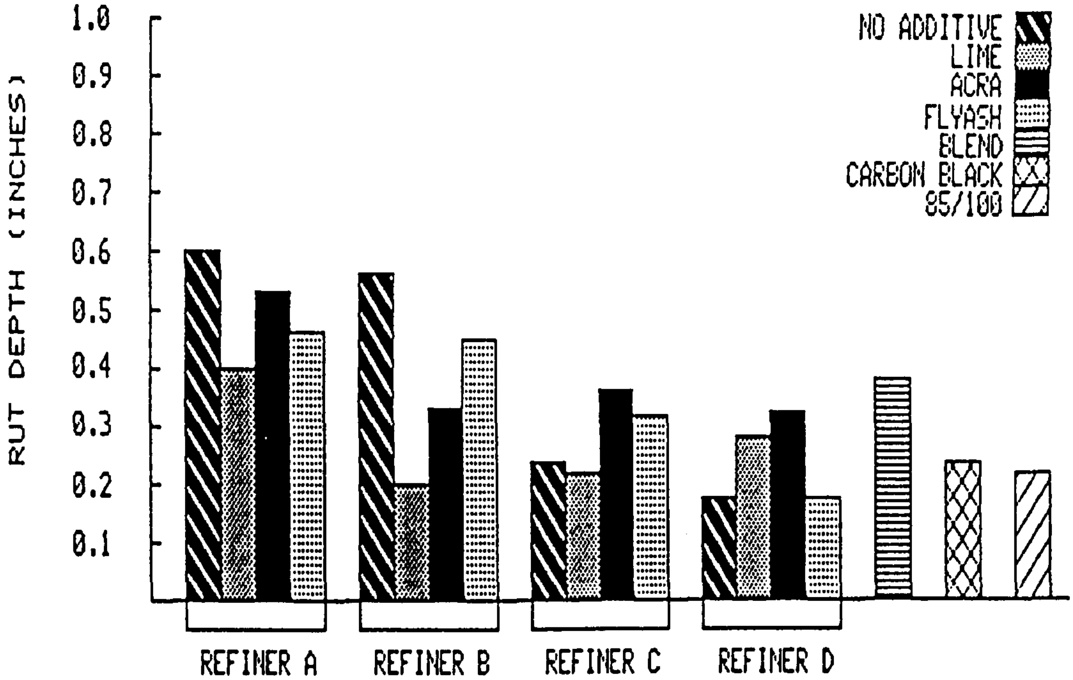
PVN				
Original	Recovered	Asphalt	Cracks[a]	Ruts[a]
–0.65	–0.5	D	19	0.16
–0.71	–0.65	C	1	0.25
–0.72	–0.74	A	18	0.62
–0.93	–1.06	B	23	0.56
–[b]	–0.99	B	0	0.43

[a]In sections with no additive.
[b]Not available.

Table 3.13 Relation of Corbett fraction to performance in Montana (Jennings et al. 1988)

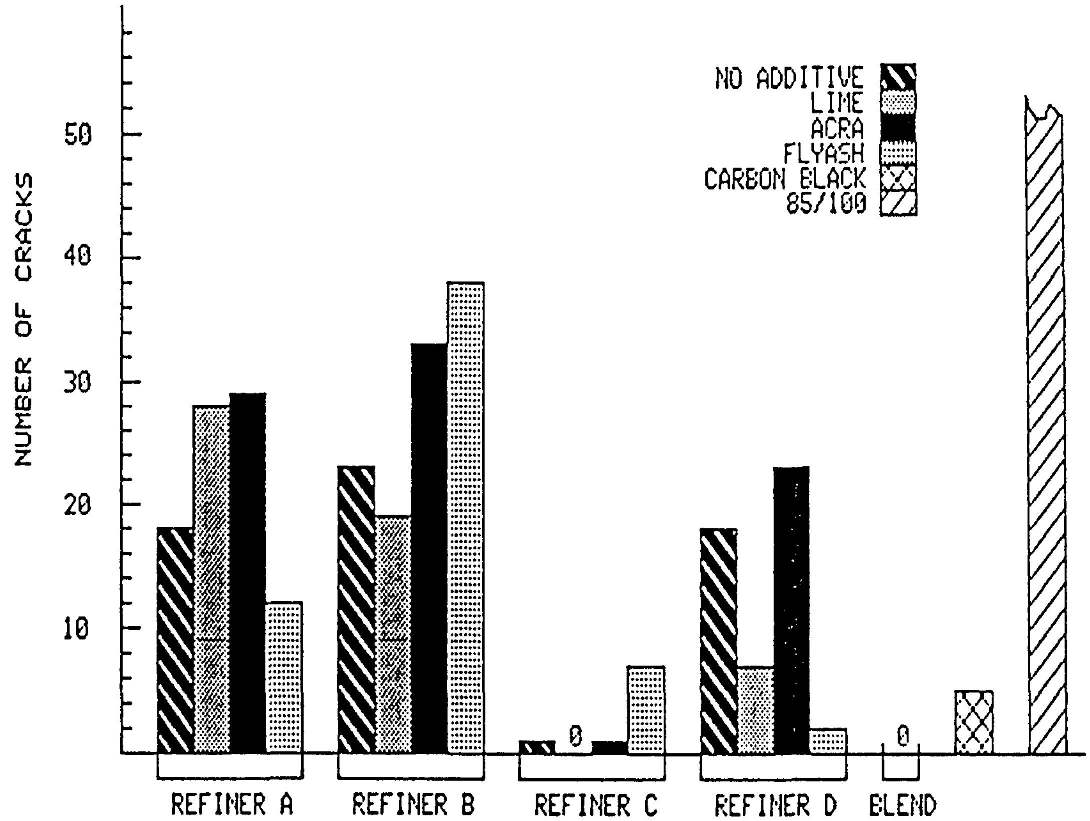
	Original Asphalt				
	ASP (%)	PA (%)	NA (%)	SAT (%)	Asphalt
Cracks					
1	18	28	34	20	C
18	12	29	46	13	A
19	16	25	42	17	D
23	11	27	46	16	B
Rutting (in.)					
0.16	16	25	42	17	D
0.25	18	28	34	20	C
0.56	11	27	46	16	B
0.62	12	29	46	13	A

NOTE: ASP = asphaltenes, PA = polar aromatics, NA = naphthene aromatics, and SAT = saturates.



Notes: 1. ACRA is a liquid anti-stripping additive.
2. Special asphalt blend composed of 75% Refiner B, 25% Refiner D.
3. Carbon Black section constructed with 200-300 pen. asphalt from Refiner B.
4. 85/100 pen asphalt is from Refiner B.

Figure 3.47 Rutting in test section pavements in Montana (Jennings et al. 1988)



Notes: 1. ACRA is a liquid anti-stripping additive.

2. Special asphalt blend composed of 75% Refiner B, 25% Refiner D.

3. Carbon Black section constructed with 200-300 pen. asphalt from Refiner B.

4. 85/100 pen asphalt is from Refiner B.

Figure 3.48 Cracking in test section pavements in Montana (Jennings et al. 1988)

HP-GPC chromatogram of the virgin 120-150 asphalts and of the special blend are compared in Figure 3.49. The theory that an excess of large molecular size (LMS) material over the optimum amount is associated with cracking is supported with the exception of asphalt B. The authors state that Asphalt B is one of a small number of asphalts, found around the country, that are characterized by relatively high temperature susceptibility, relatively low asphaltene content, relatively low LMS content, and a tendency to crack severely quite early in their service lives. These particular asphalts are also products of propane deasphalting units and the authors caution that interpretation of results with these asphalts should be guarded, especially with regard to cracking.

Conclusions

A summary of the conclusions stated by the authors includes:

1. Penetration grade may help to predict the relative performance of products from one refinery or crude source.
2. Considering all 4 refineries, there is no apparent correlation between transverse cracks or rutting to penetration and viscosity.
3. High PVN characterizes one of two asphalts that show early and severe cracking. It also characterizes one of three asphalts that have cracked to date, as well as one of two asphalts that have shown serious rutting tendencies.
4. Lower asphaltene content from the Corbett separation characterizes the two asphalts that are rutting most severely. Resistance to cracking is associated with relatively lower concentrations of naphthene aromatics.
5. Within the 120-150 grade, both rutting and cracking differ with source. Performance is modified by additives.
6. Asphalts that contain more large molecular sizes (LMS) than the theoretical ideal demonstrate a tendency to crack. A good correlation is observed when Asphalt B, a known exception, is removed from consideration. Molecular size distributions vary with asphalt source.

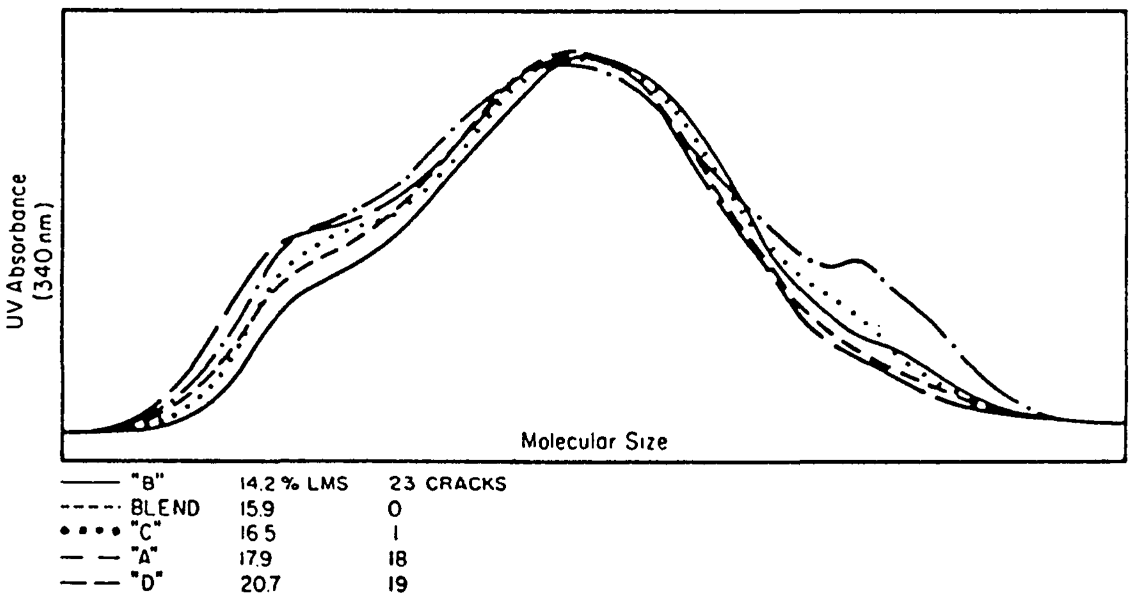


Figure 3.49 HP-GPC chromatogram for original asphalts in Montana (Jennings et al. 1988)

3.12 SASKATCHEWAN "SOFT" ASPHALT STUDY

Description

In 1973, a test road was constructed to compare the performance of a full depth asphalt concrete pavement made with SC-6 "soft" asphalt with that of one made with standard AC-6 asphalt (Culley 1978). Table A.43 summarizes asphalt and mix property data for the two test pavements. Both were constructed with low temperature susceptible asphalts.

Results

Premature failure of both structures negated direct comparison of performance. Traffic was much heavier than design estimates and early shear failure resulted. The research aspect of the test section was thus abandoned at the end of 1977 after 48 months in service. At that time, the SC-6 section showed no transverse cracking whereas the AC-6 section had 20 cracks. This was the only significant difference in performance between the two sections.

3.13 SASKATCHEWAN AIR-BLOWN ASPHALT STUDY

Description

In 1973, a full-scale test section was constructed in central Saskatchewan to evaluate the performance of air-blown asphalts used to reduce thermal cracking (Clark and Culley 1976). The pavement sections were 7.5 inches of full-depth asphalt concrete over oxidized till subgrade with Saskatchewan-type crushed aggregate having a maximum size of 5/8 inch.

Results

The properties of the mixtures and asphalts before and after 6, 12, and 24 months were determined. Stiffness values of the asphalts were determined by Van der Poel's nomograph at -40°C (-40°F) and by the sliding plate rheometer at 0°C (32°F) according to the method developed by Fenijn and Krooshof (1970). This data is given in Table A.44.

Conclusion

After 24 months, the lowest temperature recorded was -40°C (-40°F). Cracking was minimal and, except for an anomalous value for the AC-5, the binder stiffness did not reach the critical stiffness of 20,000 psi (138 MPa) that was suggested by Fromm and Phang (1971) for a loading time of 10,000 seconds. A summary of the performance of the test sections and stiffness relationships of the original asphalt and the asphalt after 24 months of service is shown in Figure 3.50.

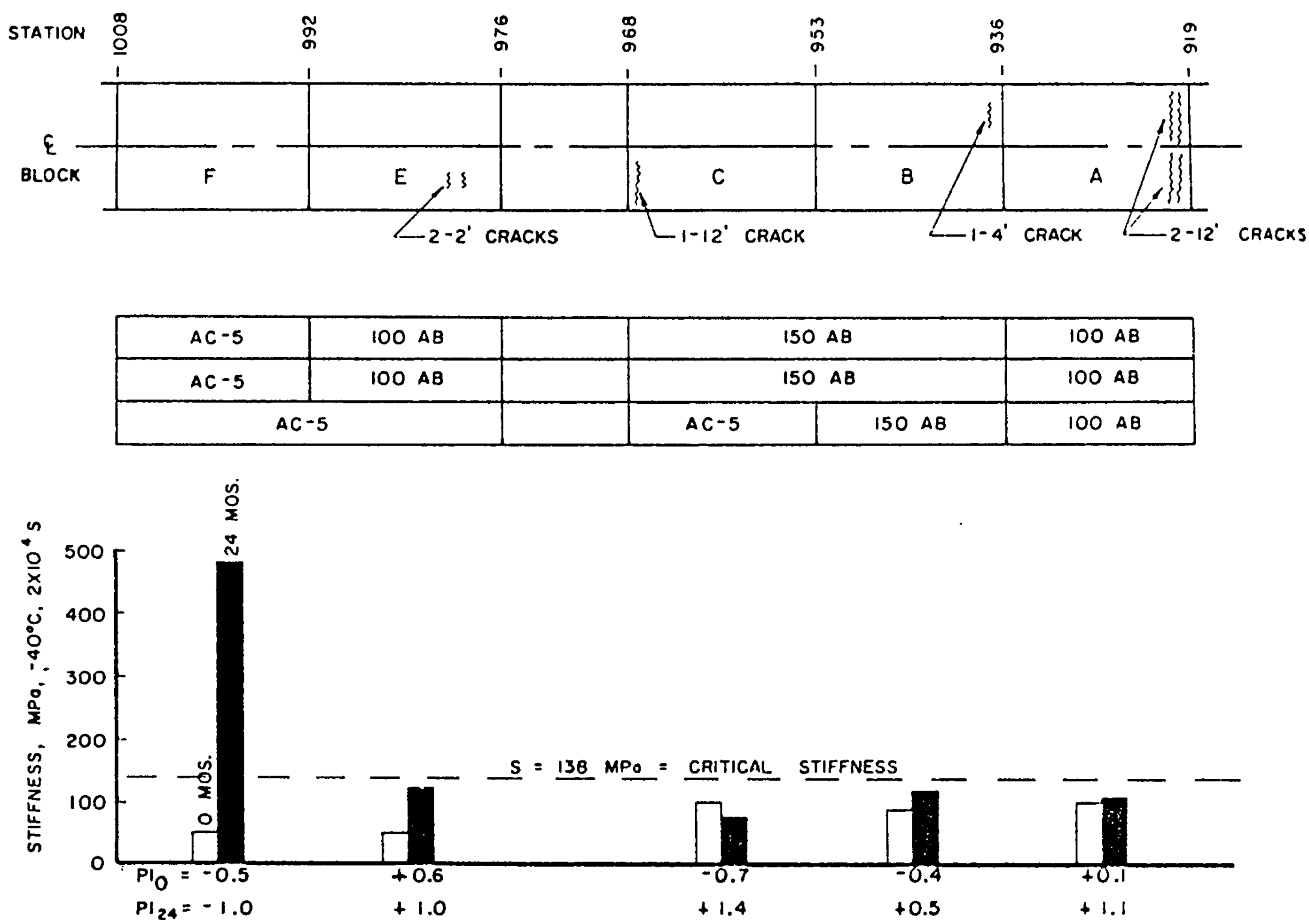


Figure 3.50 Transverse cracking-stiffness relationships at 24 months in Saskatchewan (Clark & Culley 1976)

3.14 MANITOBA AND ONTARIO AIR-BLOWN ASPHALT STUDIES

Description

Test roads were constructed on the Trans-Canada Highway in Manitoba in 1971 and in Northern Ontario in 1973 to evaluate the performance of air-blown asphalt cements produced from waxy crudes (Gaw et al. 1976). The test road in Manitoba (Richer) was constructed using blown low viscosity BLV 100/150 and BLV 150/200 asphalts in a 2-inch surface lift over 2 inches of conventional SC5 asphalt concrete mixture. The blown asphalts were from the same crude source as the 150/200 LVA (high temperature-susceptibility) asphalt used on the Ste. Anne Road Test, which cracked extensively. In addition, pavement sections containing high viscosity 200/300 HVA and 300/400 HVA asphalts and SC5 asphalt were constructed simultaneously; however, detailed analysis of these binders were not performed. The test road in Ontario (Shabandowan) was constructed as two 1.5 inch lifts of asphalt concrete mixture on top of 12 inches of granular material over an existing cracked road. The test sections contained blown low viscosity BLV 150/200 and BLV 85/100 asphalts and a high viscosity 150/200 penetration asphalt. The authors stated that reflection cracking was expected but it was judged that the 12 inches of granular material would delay the onset of reflection cracking sufficiently to permit evaluation.

Typical properties of the air-blown asphalts used in the Manitoba Test Road are shown on Table A.45. Properties for the flashed HV and blown LV asphalts used in Ontario are presented in Table A.46. Recovered asphalt properties after 4 years of service in Manitoba and 6 months and 2 years of service in Ontario are shown on Tables A.47 and A.48, respectively. The stiffness values shown in the tables were obtained with a sliding plate rheometer in accordance with the method described by Fenijn and Krooshof (1970).

Results

The performance of both test roads is summarized along with binder type in Table 3.15. In Manitoba, no cracking was observed in any of the test sections after the first two years of service. The minimum air temperature was -41°C. During the third year, temperatures dropped to -44°C. Extensive hairline cracking was observed in the BLV 100/150 section, and a few less obvious cracks developed in the BLV 150-200 section. During the fourth winter, the minimum temperature was -36°C, and cracking in both the BLV sections became more distinct. There was a predominance of transverse cracking, but longitudinal wheel path cracking was also evident. No cracking was observed in the HV 200/300, HV 300-400, and SC5 sections after 4 years. The crackihg in the BLV test sections was not typical of the transverse thermal cracking usually observed. However, the authors indicate that the cracking in the BLV was similar to one of the composite sections placed

Type of Binder	Amount of Cracking After: 3 years	4 years
Manitoba		
BLV 150-200	Few cracks	Some transverse and longitudinal cracking
BLV 100-150	Extensive* hairline cracking	Extensive transverse and longitudinal cracking
HV 200-300	No cracking	No cracking
HV 300-400	No cracking	No cracking
SC-5	No cracking	No cracking

*Cracking was not the typical transverse thermal cracking that generally had been observed in other test roads

Ontario		
BLV 150-200	Not significant	
BLV 85-100	Not significant	
HV 150-200	Moderate	

Table 3.15 Performance of air-blown asphalt test sections after 3 and 4 years in Saskatchewan (Gaw et al. 1976)

on the Ste. Anne Road Test. At that time, the cracking was thought to be associated with thermally induced stress.

In Ontario, minimum air temperatures reached -39°C during the first two years, with no cracking in either the blown asphalt sections or the control sections. During the third winter, with minimum temperatures of -38°C, moderate transverse cracking occurred in the HV 150-200 (control) section. No significant cracking developed in the low-viscosity 150-200 and the 85/100 test sections.

Conclusions

From the study, the authors concluded that:

1. The laboratory and field study confirmed that good low-temperature performance of blown, low-viscosity asphalts can be expected.
2. Air blowing improves the temperature susceptibility, and hence the low temperature cracking resistance, of waxy low viscosity asphalts.
3. The changes in the asphalts in service are primarily in the form of improvements in the temperature-susceptibility characteristics of the low-viscosity waxy asphalts.
4. No unusual amount of stone ravelling or stone stripping occurred with the blown waxy asphalts, suggesting that the presence of wax, per se, is not detrimental to good performance.

3.15 ALBERTA

Description

The high occurrence of transverse cracking in Alberta prompted pavement studies to determine the contribution of the environment, materials, and construction practices to the problem (Anderson et al. 1966). Initially, observations were made on 20 in-service (uncontrolled) pavements, and laboratory tests were made on representative samples from each of the projects.

The recovered asphalt properties for these uncontrolled projects are presented in Table A.49. Transverse crack information is presented in Table A.50. The asphalts for these pavements came from 10 suppliers. It was found that a higher frequency of cracking appeared to be associated with certain asphalt sources with some exceptions. However, examination of the mixtures failed to reveal any particular design or field properties that were clearly associated with the observed cracking frequency. Also, the temperature-viscosity relationships at various shear rates failed to provide a satisfactory means of predicting low-temperature cracking. Ductility testing proved inconclusive and was abandoned early in the program.

Therefore, the Alberta Department of Highways decided to construct and study experimental test sections. In 1966, experimental test sections using three 200/300 penetration asphalt cements of low, medium, and high viscosities at 140°F were constructed (Anderson and Shields 1971). The pavement structure consisted of 4 inches of asphaltic concrete placed in 2 equal lifts over 2 inches of asphalt-bound base and 12 inches of compacted granular base over uniform subgrade. The asphalt concrete was prepared from a single aggregate source within one contract; the sole major variable was the source of the asphalt cement.

Three separate Marshall mix designs were prepared with the single source of aggregate to be used on the project. The design mixes were generally of medium stability with somewhat low flow characteristics and low percentages of voids filled at optimum bitumen content. Properties of the original asphalts are summarized in Table A.51. Recovered asphalt properties after construction and after 12, 24 and 34 months of service are summarized in Table A.52.

Results

The study showed that the pavement section containing the highest temperature-susceptible asphalt developed transverse cracks earliest, and the density of that mixture increased the most under traffic. A summary of the number of transverse cracks per mile is shown on Table 3.16. The authors stated that the marked increase in cracking from all 3 suppliers during the third winter was caused by the most severe winter experienced in that portion of Alberta in 75 years, while the previous two winters were considered to be average.

Conclusions

A specification incorporating a minimum penetration of 250 at 77°F and a minimum viscosity of 275 poises at 140°F was adopted in 1967. It was reported that under normal conditions, this has resulted in a more uniform product, a reduction of early low-temperature transverse cracking, and the elimination of tender mixes.

3.16 PENNSYLVANIA (1964)

Description

Six test pavements were constructed in 1964 to evaluate the functions of penetration and viscosity during construction and the durability of various asphalt cements under field conditions

Table 3.16 Summary of tensile splitting test information on highway section 2-D-2/1 and 2/2 at test temperature 0•F in Alberta (Anderson & Shields, 1971)

SUPPLIER	CONDITION	PAVEMENT PERFORMANCE (CRACKS/MILE)
1	After first winter	4
	After second winter	87
	After third winter	187
2	After first winter	nil
	After second winter	nil
	After third winter	126
3	After first winter	nil
	After second winter	4
	After third winter	84

(Sandvig & Kofalt 1968, Kandhal et al. 1972, Kandhal et al. 1973, Kandhal & Wenger 1975). The pavements were located in Clinton County on Legislative Route 219 (US-220). The original pavement consisted of 8 inches of reinforced concrete 18 to 20 feet wide. This pavement was resurfaced with a 2 inch binder course and a 1 inch wearing course. A different asphalt was used for each wearing course. Average daily traffic at the time of construction was 4,200.

Properties of the six asphalts used in the project are given in Table A.53. During construction of the test sections in each project, mixture and construction variables were maintained as uniformly as possible. Mixing temperatures were controlled by viscosity, which varied between approximately 140 and 300 centistokes. The only significant variable was the asphalt type. Since construction of the pavements, core samples have been obtained periodically to determine the percentage of air voids in the pavements and rheological properties of the aged asphalt. The last core sampling was done in March 1974, 113 months after construction.

Results

No differences in texture or color tones were observed between the asphalts when the pavements were visually inspected just after construction and after one year of service. Visual evaluation after 30 months of service indicated that the entire road surface was good with the exception of some raveling in the Asphalt 1 section which was not adequately compacted and had a higher void content than sections where other asphalts were used.

The rating method suggested by Olsen et al. (1969) was used as a guideline to accomplish the visual pavement condition survey for evaluating the effects of asphalt aging. Subjective visual evaluation included ride quality, raveling, spalling, loss of matrix, rutting, cracking (transverse, longitudinal, and alligator but not reflection cracking), and surface texture. A team of five engineers evaluated these sections after 80 months of service (Kandhal et al. 1972). The last performance evaluation was conducted after 113 months of service by eight evaluators (Kandhal & Wenger 1975). Brief details of the pavement condition after 113 months are given in Table A.54. Pavement performance ratings and viscosity, shear susceptibility, slopes of viscosity vs. shear susceptibility, and ductility are given in Table 3.17. An ideal pavement according to this performance evaluation would have a performance rating of 72.

Test Pavement	Performance Rating	Viscosity 77 F (25 C) 0.05 sec^{-1} Rating	Shear Suscep-tibility Rating	Slope m of Shear-Viscosity Curves at 77F (25 C)	Ductility 39.2 F, 1 cm/min Original Asphalt (cm)	After Mixing (cm)	Ductility 60 F 5 cm/min (cm)
1 (poorest)	51.1	1	1	1.174	14	4.1	0
6	59.8	6	4	1.354	21.9	7.3	8
4	60.1	5	6	1.431	23.5	7.5	7
2	60.4	4	5	1.738	53.3	11.9	19
5	61.2	2	3	2.859	68.3	24.3	19
3 (best)	61.5	3	2	3.166	101.0	42.2	49

Note: Tests are on asphalt recovered after 113 months of service.

Table 3.17 Relation of pavement performance to properties of recovered asphalt for Pennsylvania 1964 (Kandhal & Wenger 1975)

Conclusions

The authors concluded that:

1. The slope of the relationship of shear susceptibility and viscosity proved a better correlation with pavement performance than either shear susceptibility or viscosity alone. Shear susceptibility (or shear index as defined in this study), is the tangent of the angle of log shear rate (X axis) versus log viscosity (Y axis) determined from viscosity measurements at 77°F using the microviscometer (Kandhal et al. 1973),
2. Ductility test values of the asphalts at 39.2°F, before and after pug mill mixing, are consistent with the pavement performance ratings. Higher ductility values are associated with better pavement performance. Asphalt 1 with the lowest ductility shows the poorest performance.

3.17 TEXAS

Description

In an attempt to correlate standard laboratory test data to field performance, three experimental test roads were constructed in different environments in Texas using different asphalt cements (Adams & Holmgreen 1986). Another objective of the study was to determine the feasibility of using gel permeation chromatography not only in research but also in the quality monitoring of asphalts. Much of the focus of the study is on mix properties.

Ten asphalt cements (5 sources, 2 refineries) were used to construct pavement sites at Dickens, Dumas & Lufkin in 1982-1983. The asphalts were all of the AC-10 or AC-20 grade. Original and recovered asphalts were tested for penetration at 39.2°F, 77°F and viscosity at 77°F, 140°F and 275°F. In addition, Thin Film Oven residue was tested for penetration at 77°F and viscosity at 140°F. The recovered asphalts came from cores sampled after one and two years of service. Tables A.55 to A.71 summarize the results of the physical tests that were performed on these asphalts. In addition, the chemical composition of the asphalts were determined using both the Rostler-Sternberg & Corbett procedures (Tables A.72 and A.73). The results indicate that there are wide differences in chemical composition of these asphalts. The temperature susceptibilities of the asphalts were computed using different methods and the results shown in Table A.74.

Gel Permeation Chromatography (GPC) was performed on asphalts extracted from field cores as well as on the original asphalts. The major objective of the GPC study was to evaluate its potential in quality monitoring applications. One factor considered was the effect of asphalt grade on chromatograms produced for the same refinery. The results showed that, for the most part, different asphalts can be recognized by their respective chromatogram, although there was an exception during this study.

Results

After one year, the pavements were inspected. At Dickens, the pavement surface appeared dry but no major distress was otherwise noted. The road was fog-sealed the following year. At Dumas, the sections with asphalts B and C began to ravel severely within two months. By the end of the first year, the sites had to be removed and replaced. However the authors conclude that there is no indication that the asphalt at Dumas was the primary cause of the failure since air voids reached 15% at some places. The other sites were performing well. No distress was noted at Lufkin.

Conclusions

Due to the lack of any long term field results, no correlation with asphalt properties were presented in this report. However, a follow-up project is currently underway, and the results will be reported as they become available. Some conclusions on mix properties were made, but they have not been included in this report. Some relevant conclusions on the asphalt cement included the following:

1. Based on the field performance data to date, the different asphalts in this study exhibited equivalent performance.
2. Asphalt temperature susceptibility can adversely affect compaction of hot mixed asphalt concrete.
3. Gel Permeation Chromatography will produce chromatograms that will allow identification of different asphalt cements. However, there is no direct correlation between the asphalt chromatogram shape and aging as observed by change in viscosity, but the shape does show change with age.
4. GPC will produce similar chromatograms for asphalt grade AC-10 and AC-20 from the same producer. It will indicate the presence of an additive or some other foreign material in the asphalt cement in a comparative analysis of the chromatogram of the virgin asphalt cement. Finally, it provides a means for monitoring the continuing consistency of asphalts.

4.0 UNCONTROLLED TEST ROADS

Uncontrolled road tests are those that are not planned experimental road trials. The investigations represent an effort to study in-service pavements to determine if in-service asphalt properties can be related to pavement performance. As defined by Welborn (1979), this category includes the following:

1. Pavements constructed under normal contract procedures, with performance surveys and tests on representative pavement samples to determine the properties of the asphalt after one or more periods of service. Generally, only results of specification tests on the asphalts were available.
2. Pavements constructed under normal contract procedures, with performance and tests on the original and recovered asphalts where known or determined.

This chapter presents the results of the literature review (as of August 1989) on uncontrolled test roads. Appendix B contains additional tables and figures with detailed information pertinent to each investigation.

4.1 OREGON

Description

Between 1984 and 1988, the Oregon Department of Transportation (Thenoux 1987, Thenoux et al. 1988) conducted a study to evaluate and implement an analytical chemical procedure that could be used to characterize asphalt pavement materials. Eight highway projects that represented a range of performance and highway environments throughout Oregon were selected for the study. Physical and fractional properties of original and recovered asphalts were obtained together with mix properties. The relationships between the fractional components and physical properties were examined as well as relationships between fractional components and temperature susceptibility parameters. Table B.1 summarizes the mix and core properties from the test sites.

Cores were taken from the travel lanes on each project, as well as some from shoulders and non-trafficked path areas. The cores were cut into 2 halves and analyzed separately so as to differentiate between the environmental effect on the exposed (top half) and unexposed (bottom half) portions of the pavements. A modified, small-scale version of the Corbett-Swarbrick procedure (currently ASTM D4124 with modifications under consideration by Committee D04.47) was used to perform the asphalt composition analysis. The Corbett-Swarbrick chemical analysis yields 4 district fractions: asphaltenes, saturates, naphthene-aromatics and polar-aromatics. Tables B.2a and B.2b show the chemical composition for original samples before and after RTFO, and for recovered asphalts, respectively. The results shown are the average of two separate and independent tests.

Results

Four groups of tests were performed on all 8 road projects:

1. Physical properties of original samples and after RTFO (Rolling Thin Film Oven test).
2. Physical properties of core-recovered asphalts.
3. Asphalt chemical fractionation test results.
4. Fraass test results before and after pressure oxygen vessel (POV) aging test.

The original asphalts used in the test sites had been stored in sealed cans since construction of the projects. Physical tests on these asphalts had been performed before the storage period and after to determine if any changes had occurred during storage. The stored asphalts were artificially aged in the RTFO and tested. Table B.3 summarizes the results obtained for original asphalts and after RTFO together with the results available at the time of construction. It was concluded that asphalt samples stored in sealed cans did not show significant variations in their physical properties. The minor variations found were attributed to the reproducibility of the test results rather than changes due to aging.

Asphalt from cores was extracted and recovered using combinations of the following two extraction and recovery methods.

1. Extraction - Reflux hot extraction
   - Cold vacuum extraction
2. Recovery - Modified Abson
   - Roto-Evaporator

Tables B.4a and B.4b show the results of the physical tests performed on recovered asphalts. The authors conclude that the recovered asphalt did not have the same fractional composition as the RTFO samples. This indicates that recovered asphalt, after going through the extraction and recovery procedure, may be chemically altered and no longer represents the in-place asphalt and/or that the RTFO aging test may not duplicate the changes made by the asphalt under natural weathering and in contact with mineral aggregates. The authors did not attempt to predict these changes. Figure 4.1 illustrates the different trends for the recovered asphalts and the RTFO samples with regard to asphaltenes and naphthene aromatics.

Four physical properties were measured; penetration at 4°C and 25°C, absolute viscosity at 60°C and kinematic viscosity at 135°C. Generally better correlations were found at higher temperatures than at lower temperatures. The results for penetration at both temperatures were similar and indicated that it was independent of the percentage of saturates, napthene aromatics and polar aromatics (Figure 4.2). The asphaltene fraction has an impact on penetration but the scatter in the data suggests that some other physiochemical property of the asphaltenes may be more important. The authors also conclude that the test for penetration, in general, may not be sensitive enough. The relationships for viscosity are similar to those for penetration but have more noticeable trends, particularly for asphaltenes. The higher the asphaltene content, the higher the viscosity.

A certain level of generalization of rheological and chemical behavior on original and aged asphalt was possible by studying a relatively small group of asphalts. However, analysis of individual asphalts showed that different asphalts do behave and age differently. The different types of behavior shown by all samples when changing from original to aged materials suggest that more than one aging condition could be studied. For example, asphalt samples should be aged at three or four different RTFO conditions and the rate of changes in measured properties compared among the different asphalts. Measuring absolute changes of asphalt properties based on one aging condition may not reflect the overall aging behavior.

The POV test (conducted at 100 psi oxygen pressure and 60°C for 2 and 5 days) was used to produce accelerated aging of asphalt binders. The POV device was used in conjunction with the Fraass test to evaluate oxidative aging. It should be noted that the POV test ages asphalt in an oxygen-rich environment in an attempt to simulate long-term aging whereas the RTFO uses high temperature and simulates short-term construction effects.

The Fraass Brittle Point Test, which is not a standard ASTM or AASHTO test, was utilized to assess the changes in fractional components of the asphalt after the POV aging test. Changes in composition were compared with changes in Fraass temperature and with changes after the RTFO. Figure 4.3 shows the relationships between Fraass temperature and the asphaltene component for results from three projects before and after aging. The asphaltenes were the only fraction that had a relationship with Fraass temperature. The authors concluded that the POV 5-day test was the most severe aging test, and caused much greater change in composition than did the RTFO test. Also,

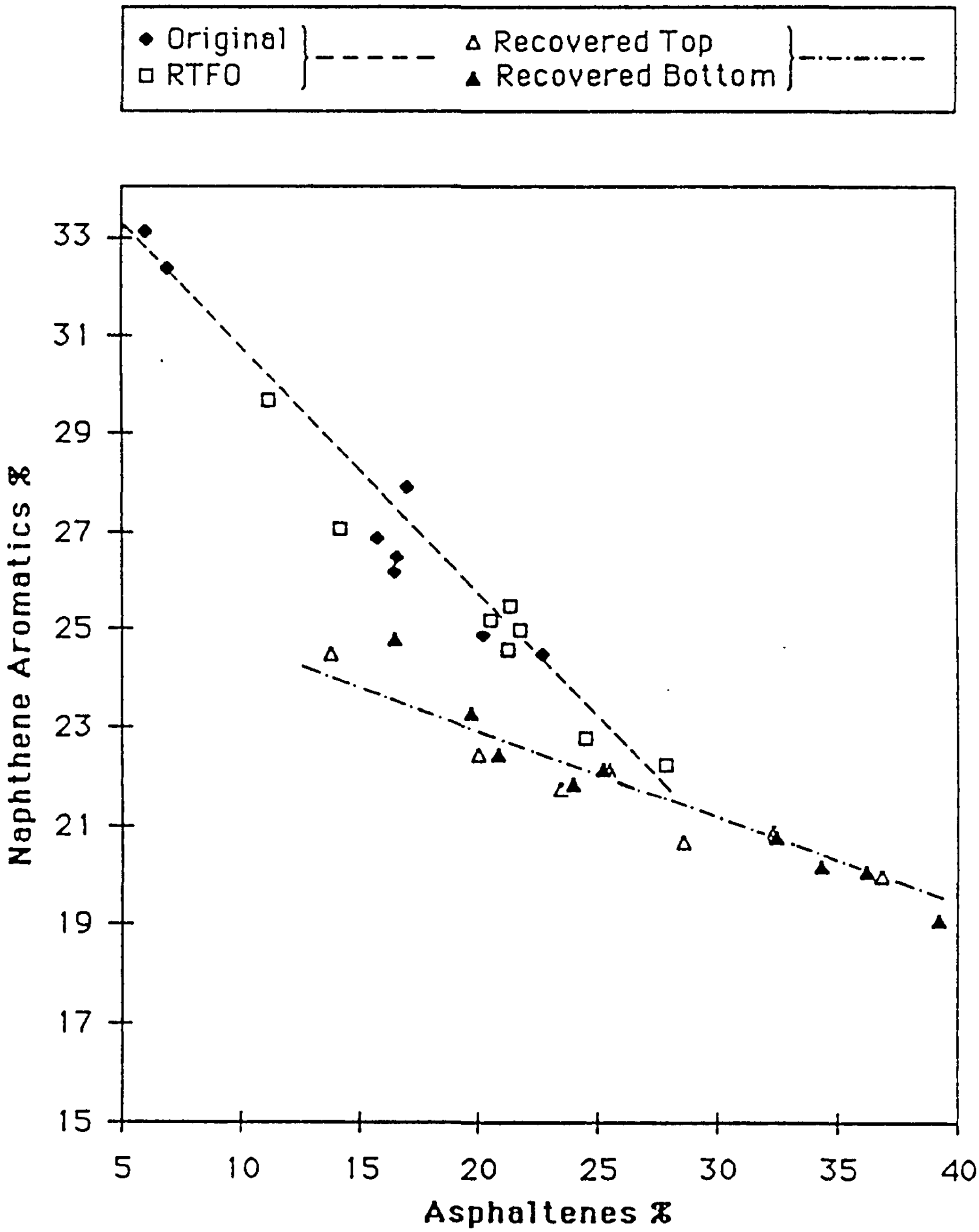


Figure 4.1 Naphthene aromatics vs asphaltenes for Oregon (Thenoux 1987)

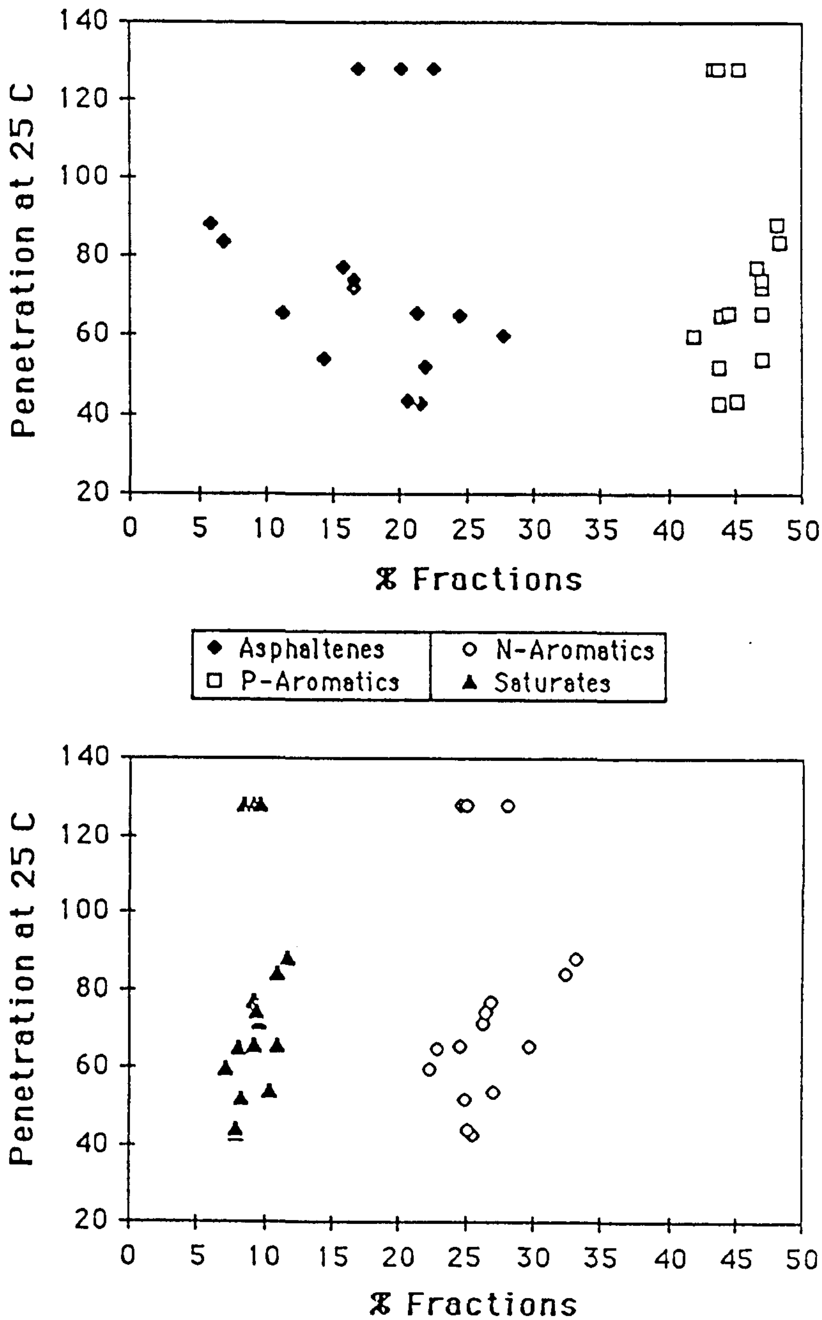


Figure 4.2 Penetration at 25°C vs. chemical fractions, original and RTFO samples for Oregon (Thenoux 1987)

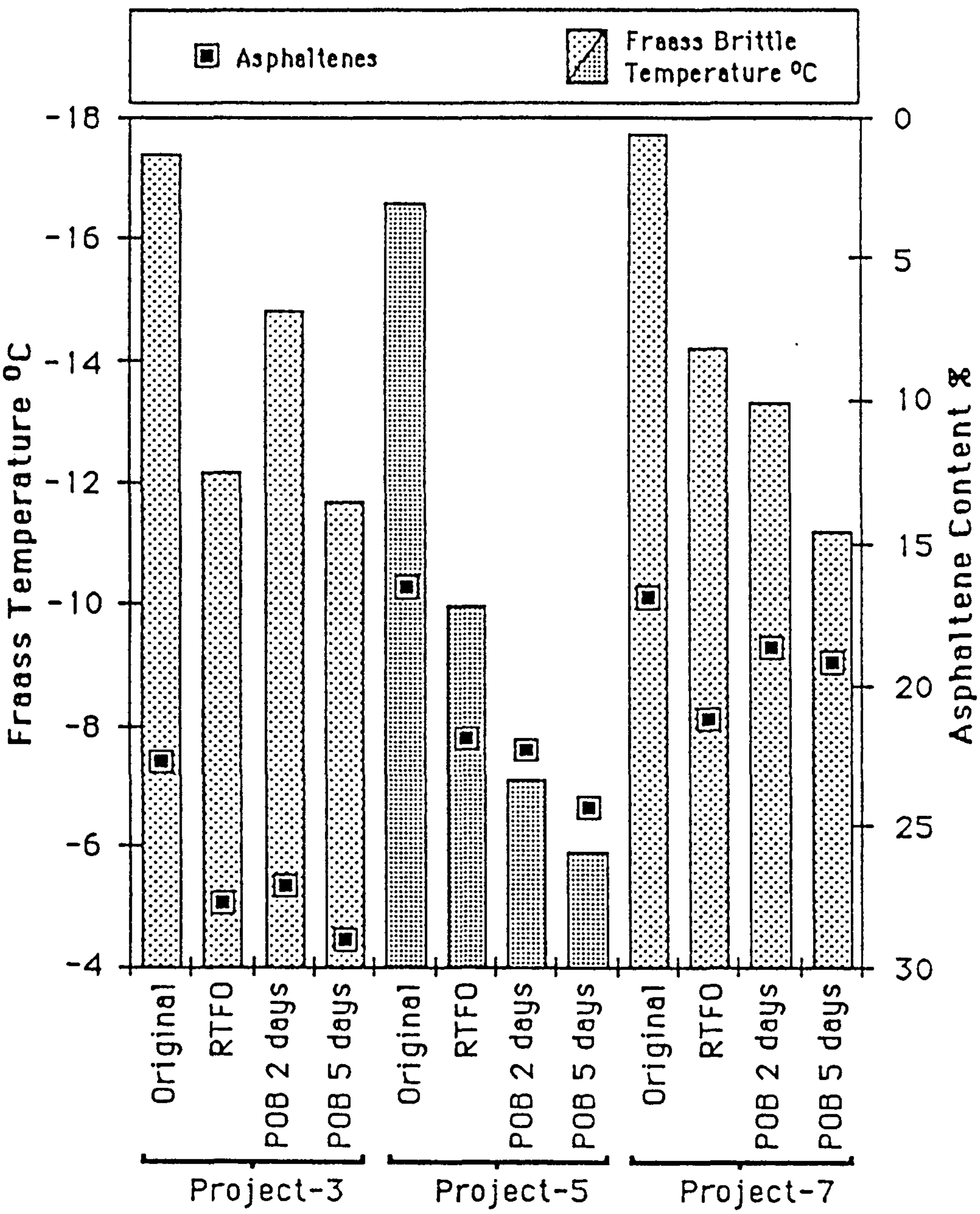


Figure 4.3 Comparison of results of POV test for Oregon (Thenoux 1987)

asphaltene content increases with aging but original asphaltene content is not related to the total amount of aging after RTFO and POV.

Finally, four indices were calculated to correlate chemical composition analysis with temperature susceptibility: Penetration Index (PI), Viscosity Temperature Susceptibility (VTS), Penetration-Viscosity Number (PVN) and Penetration Ratio (PR). The PI was calculated using Pfeiffer's and van Doormaal's procedure and the PVN using McLeod's (1972) procedure. The VTS may be calculated using the following equation:

$$VTS = [\log \log V_2 - \log \log V_1] / [\log T_1 - \log T_2] \qquad \text{(Eqn. 4.1)}$$

where

VTS = Viscosity temperature susceptibility

T_1 & T_2 = 60°C and 135°C, respectively

V_1 = absolute viscosity at 60°C, Poises

V_2 = kinematic viscosity at 135°C, Poises

The penetration ratio is obtained using the following equation:

$$PR = (\text{Pen @ } 4°C, 200g, 60 \text{ sec}) / (\text{Pen @ } 25°C, 100g, 5 \text{ sec}) \qquad \text{(Eqn. 4.2)}$$

A lower PR would indicate greater temperature susceptibility.

Relatively good relations were found between fractional composition and temperature susceptibility. Better correlations were found for the PR and PI indices (penetration ratio and penetration index, low temperature range) than for VTS and PVN (viscosity temperature susceptibility and penetration viscosity number, high temperature range). Figure 4.4 illustrates the data obtained for the PI relationships. It appears that the % asphaltenes has a marked effect on PI on original asphalt. The higher the asphaltene content, the higher the PI (lower temperature susceptibility). Regression analyses showed that the four indices used were distinctly different and that fractional composition had entirely different effects on all four.

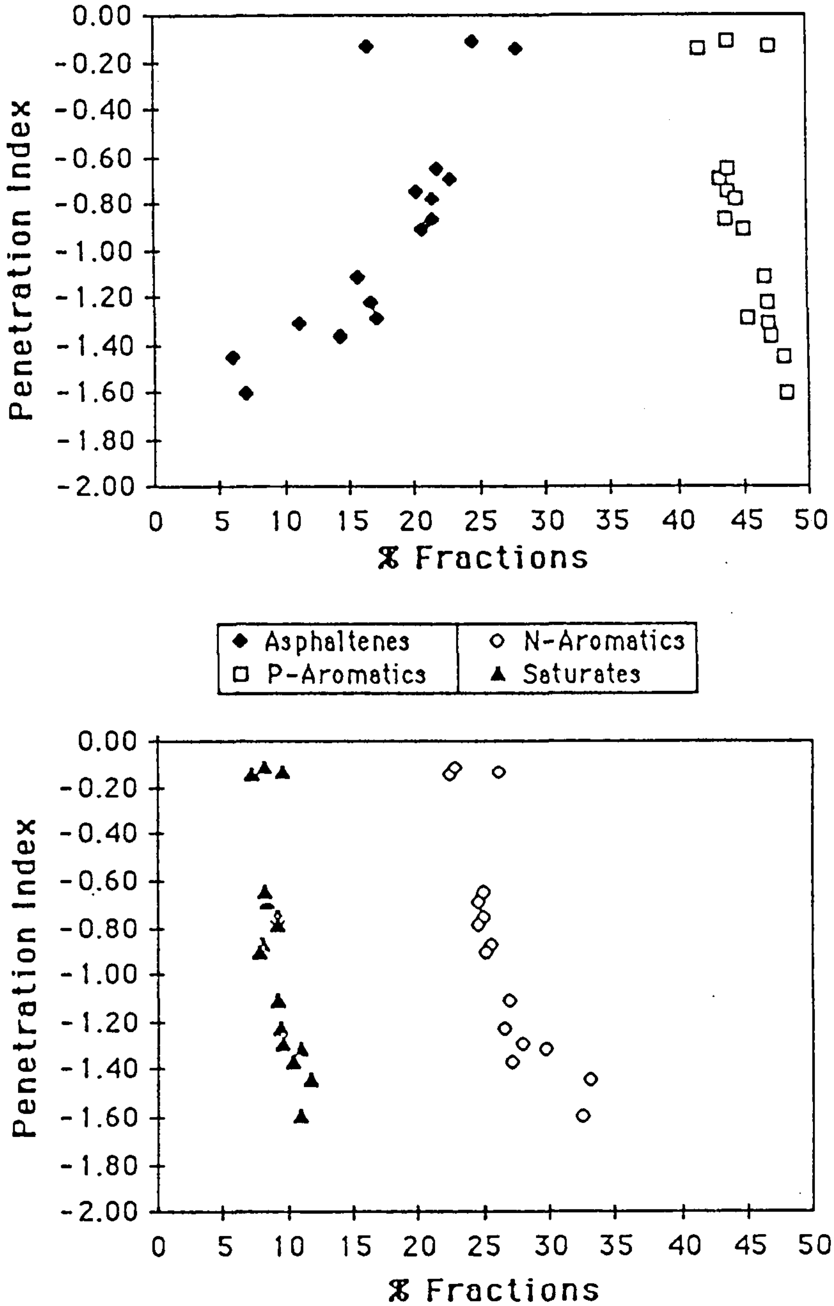


Figure 4.4 Penetration index versus chemical fractions original and RTFO samples for Oregon (Thenoux 1987)

Conclusions

The authors made the following conclusions:

1. Asphalt samples stored in sealed cans at room temperature for periods of up to 12 years did not show significant variations in their physical properties.
2. Due to differences in the fractional composition between original and RTFO asphalts, the extraction and recovery procedure may chemically alter the asphalt such that it no longer represents the in-place asphalt. Another possibility is that the RTFO aging test does not duplicate changes due to weathering.
3. Physical properties of original and RTF samples generally showed better correlation with all four fractions (asphaltenes, saturates, polar and napthene aromatics) at higher temperatures (viscosity) than at lower temperature (penetration), i.e. viscosity at 60°C and 135°C correlated better with chemical fractions that did penetration at 4 and 25°C. The higher the asphaltene content, the higher the viscosity.
4. Good relationships were found between PR and PI indices and fractional composition of original and RTFO samples in the low temperature range. Less favorable correlations were found with VTS and PVN.
5. The four methods used to extract and recover asphalts gave different results.
6. In general, with the few data available, Fraass brittle temperature does not depend on asphalt composition. However, asphaltene content increases with aging similar to the increase in Fraass temperatures.
7. The POV test in general did not simulate field aging in terms of percent change in asphaltenes. It did cause greater change in composition than the RTFO test.

4.2 FEDERAL HIGHWAY ADMINISTRATION

Description

The Federal Highway Administration (FHWA) began a comprehensive study in 1954 - 1956 on asphalts in relation to the performance of pavements (Vallerga et al. 1970, Vallerga & Halstead 1971, Welborn 1979). Asphalts representing current production then in the U.S. were collected and analyzed. The asphalts represented those used in 285 identifiable construction projects in 37 states. In 1967, a continuation study was initiated to survey the pavements where the known asphalts were used and to sample a preselected number of in-service and out-of-service pavements. The objectives of the study were to determine the over-all changes in the fundamental physical and chemical properties of the recovered asphalts after 11 to 13 years of service, to relate the changes to the fundamental properties of the asphalts before and after laboratory aging, and to relate the changes to pavement performance.

To accomplish this objective, 53 pavements in 19 states were selected for evaluation. Fifty were constructed with 85-100 penetration asphalt and the remaining 3 with 60-70 penetration asphalt. In 1967, 34 of the projects were in service and 19 had been resurfaced. The pavements were visually examined and rated for distresses such as cracking, raveling and rutting. Samples were shipped to the Materials Division of the FHWA, where fundamental and conventional tests were performed on the mixtures and recovered asphalts. Tables B.5 to B.6 summarize these properties. Chemical analysis by the Rostler method was performed by Materials R & D (Vallerga & Halstead 1971).

Results

The authors reported that none of the measured properties on the original asphalts was, by itself, a determining factor as to whether the pavements survived in service. The differences found in the penetrations, viscosities, Rostler parameter $[(N+A_1)/(P+A_2)]$ and pellet abrasion tests were found to be in the predicted direction and were indicative of aging, but they concluded that the differences were not considered to be governing. No significant differences were found in any of the properties of the asphalt concretes as of the aggregate between the surviving and unassuming pavements. However, differences were found in the ductility measurements which support the hypothesis that asphalts with a higher ductility are more likely to service longer.

Of greater interest in this study was the effect of air voids. The authors conclude that the relationships sought between mix and environmental factors and changes in asphalt properties due to field aging were greatly overshadowed by the void content of the mix.

Four laboratory aging procedures were also examined; the modified thin-film oven test, California weathering oven, Thin-film oven test and the MR & D infrared oven. Generally, it was possible to relate the relative severity of field and laboratory aging of the asphalt to void content.

a. Modified thin-film oven test - the 7-hour test is approximately equivalent to 11 to 13 years of field aging at void contents of 3 to 5%. For voids of 4 to 6% a 9-hour test was used.
b. California weathering oven - A 200 hour weathering period is more severe than 11 to 13 years of field weathering for void contents less that 2%. For void contents greater than 5%, 1,000 hours is required.
c. Thin-film oven test - For all air voids, this test was less severe than the proceeding tests.

d. MR & D infrared oven - A 7-day weathering period produces changes in chemical composition similar to those produced in 11 to 13 years of field aging in asphalt concrete pavements but with significant differences in individual components. Asphaltenes increases more when air voids > 4%; and nitrogen bases are relatively independent of voids. Paraffins and acidaffins generally decrease as air voids increases although the data do not give very clear trends.

Conclusions

The authors concluded that:

1. The most important factor in hardening (measured by ductility) of the asphalt cement in a pavement is the void content. In pavements of below 2% voids, field aging during a service life of 11 to 13 years appears to be negligible. Above this level, hardening increased with air voids (Figure 4.5). At 2% voids, the ductility at 60°F is approximately 15.
2. Changes in chemical composition were closely associated with the hardening of the binder, although there is considerable scatter in the data as shown in Figure 4.6. The increase in asphaltenes and decrease in acidaffins were also found to be generally related to the void content. The scatter of the data did not allow the authors to draw any well-substantiated conclusions. However, Figures 4.7 (penetration) and 4.8 (viscosity) show that:
   a. For void contents less than 2%, hardening was directly related to the Rostler parameter $(N+A_1)/(P+A_2)$;
   b. For void contents exceeding 2%, the relationship of penetration and viscosity to Rostler's parameter is not proportional. An increase parameter value above 1.4 resulted in a sharp increase in hardening. However, it was also indicated that increase in hardening may occur with low parameter values. The lowest amount of hardening occurs in a Rostler parameter range of between 1.0 to 1.5.
3. No overall correlations were found between pavement evaluation ratings and the properties of the asphalts. However, some trends were noted:
   a. Severe raveling was found when the penetration of the recovered asphalt was less than 10 and the ductility at 60°F less than 3.
   b. A similar relationship was found for spalling but it was less pronounced.
4. However, no numerical correlation was developed between void content and pavement ratings, despite voids having an effect on hardening.

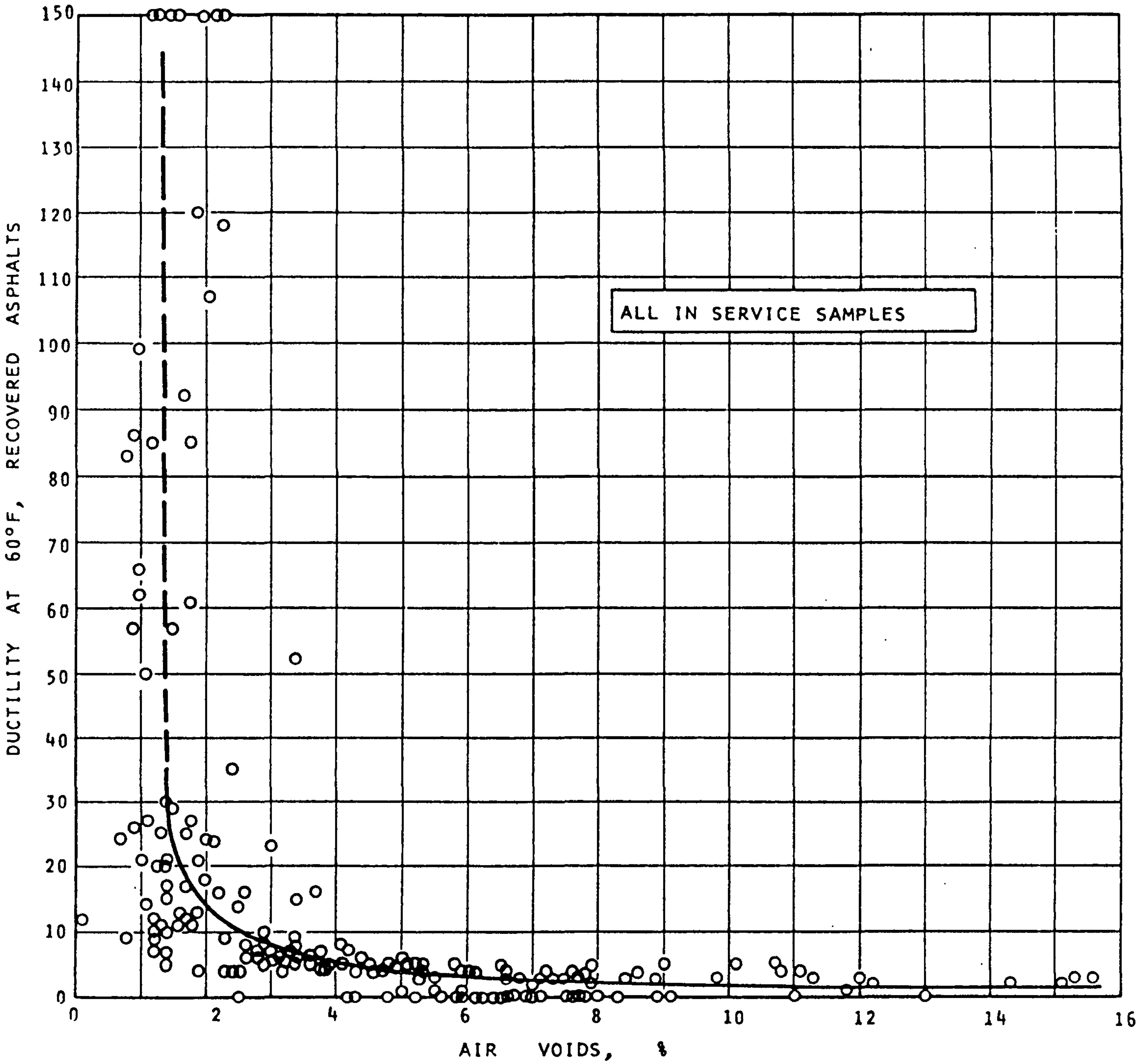


Figure 4.5 Relation of ductility at 60°F after field aging to air voids for FHWA study (Vallerga et al. 1970)

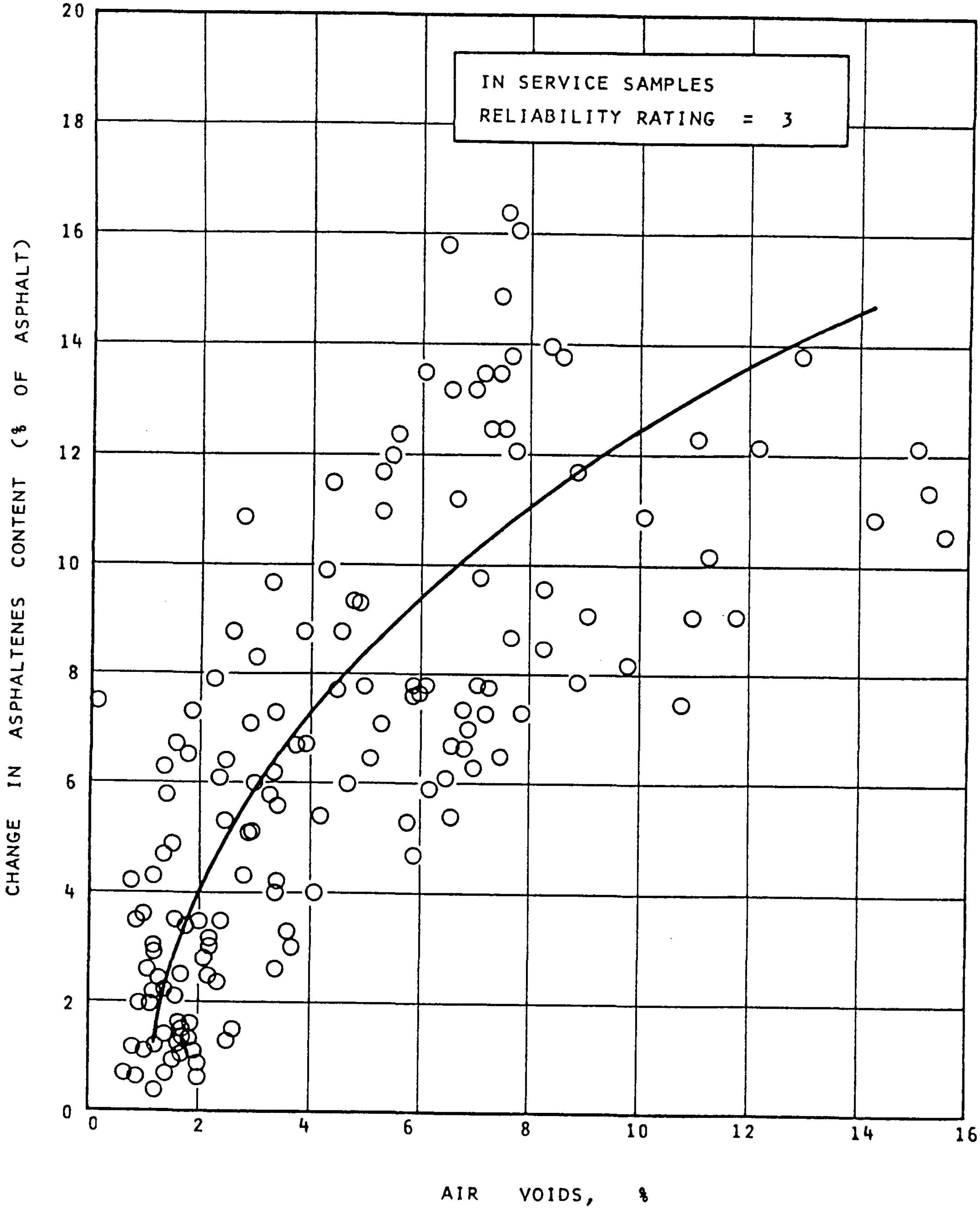


Figure 4.6 Relation of change in asphaltene content during field aging to air voids for FHWA study (Vallerga et al. 1970)

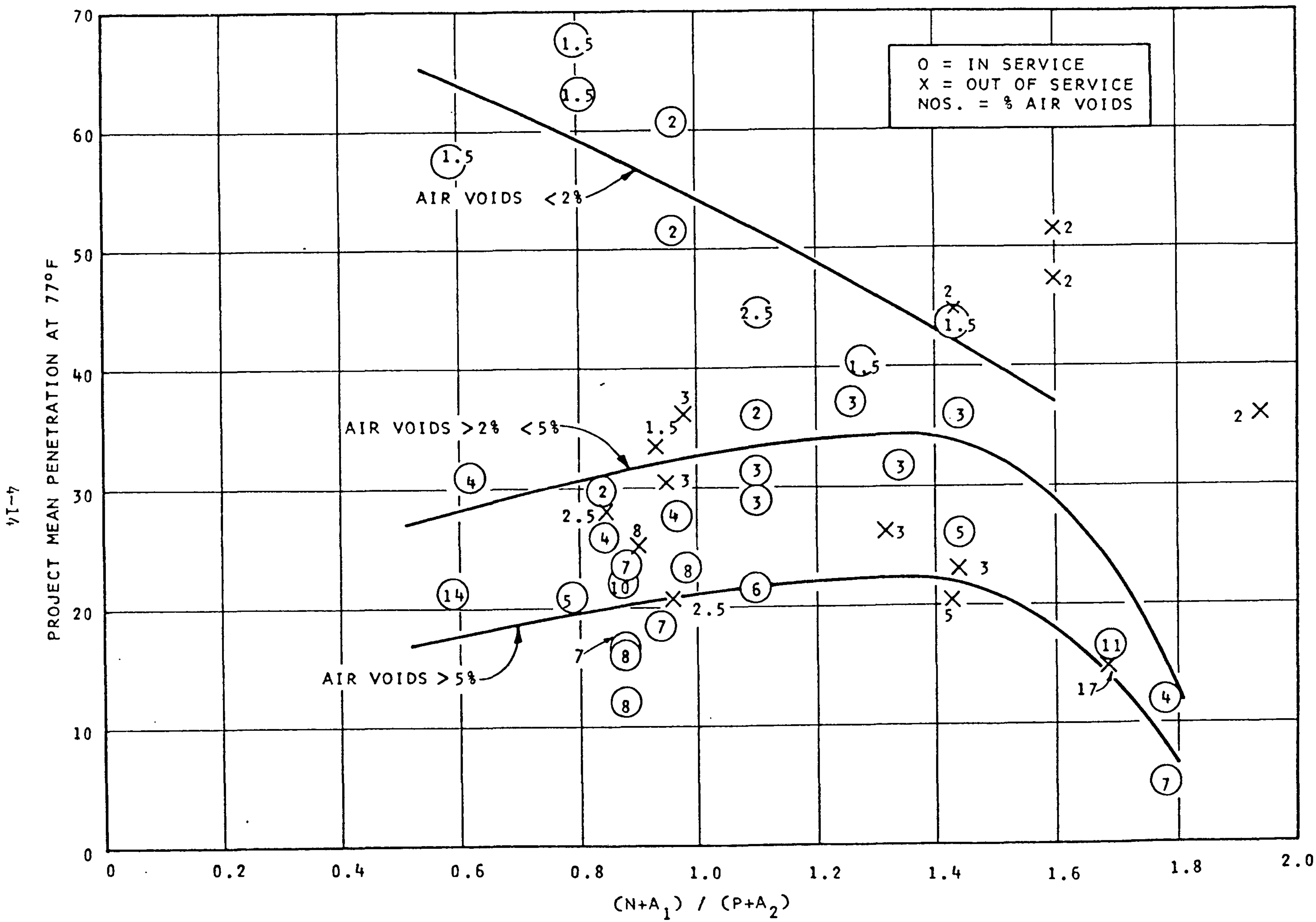


Figure 4.7 Relation of retained penetration after field aging to Rostler parameter of original asphalt for FHWA study (Vallerga et al. 1970)

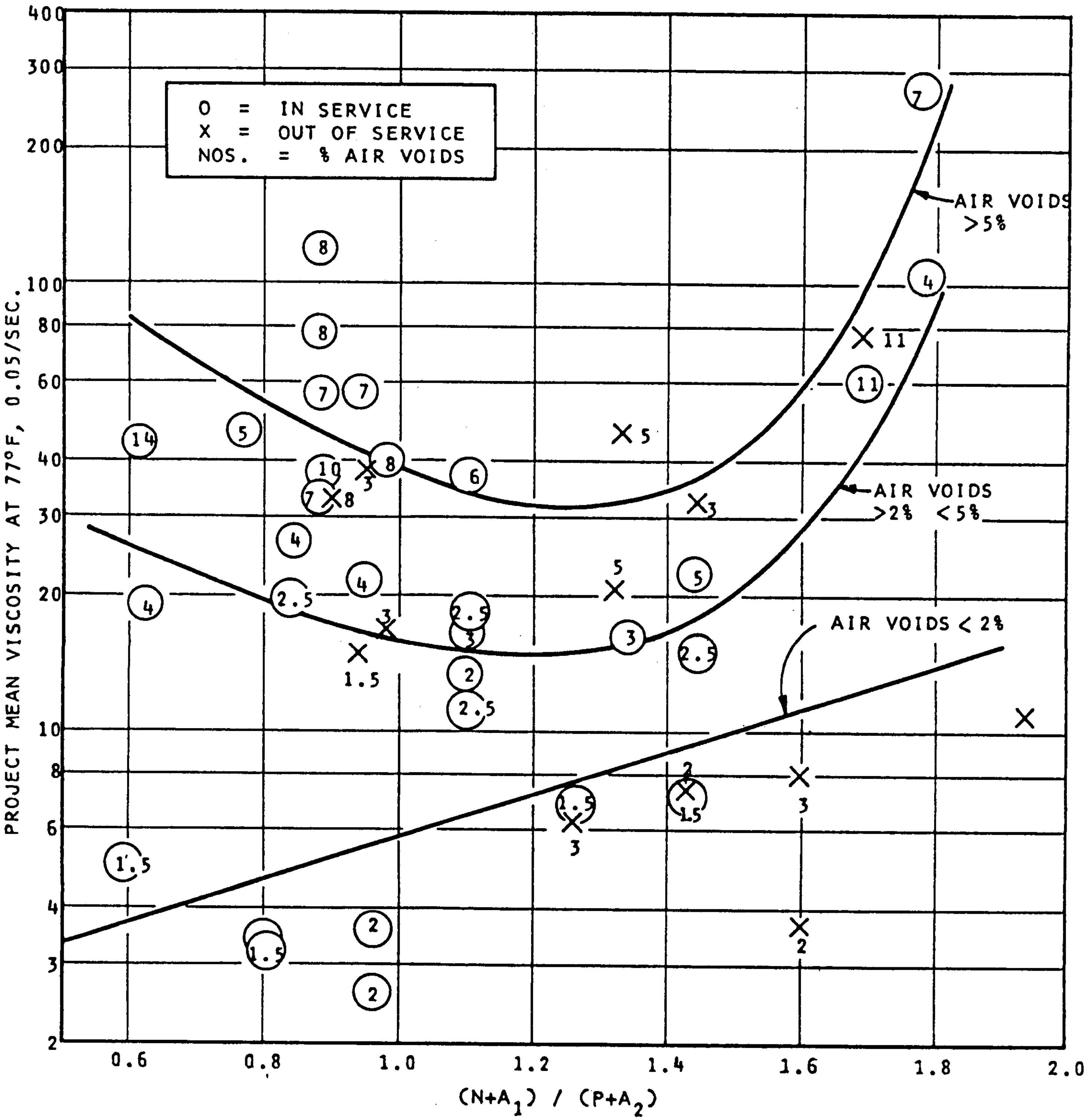


Figure 4.8 Relation of viscosity at 77°F after field aging to Rostler parameter for FHWA study (Vallerga et al. 1970)

5. Comparisons of field aging with laboratory aging tests show that both the California weathering oven aging (1,000 hours) and the FHWA modified thin-film oven aging (7 hours) resulted in hardening of the asphalt to a degree comparable to 11 to 13 years of field aging in a pavement with approximately 4% air voids.
6. The type of chemical change occurring during long-term field aging differs from that taking place in short-term, laboratory aging. Although formation of asphaltenes occurs in both cases, the disappearance of second acidaffins and increase in nitrogen bases occurs in a greater extent in field aging than in accelerated laboratory aging.

Further Studies

In a later study, Zenewitz and Welborn (1975) made detailed computerized linear regression analyses of the data. Included were linear regression equations, analysis of variance, coefficient of correlation, and standard error. Rostler's analysis was used to relate penetration and viscosity of original asphalts, thin-film residues, and recovered asphalts to the void contents of pavement samples and to chemical fractions. Coefficients of correlation ranged from -0.84 to - 0.93. The more important findings are as follows:

1. The consistency of the thin-film residue provided a better reference point than did the consistency of the original asphalt.
2. Of the chemical fractions, the second-acidaffin and paraffin content (P) of the original asphalt related best to the ultimate hardening level of the asphalt after service. The higher amounts of each of these fractions were associated with a lesser degree of hardening, assuming all other factors were constant.
3. Regression analysis also showed that relatively high content of first acidaffins (A_1) correlated with greater hardening of the binder.
4. Long-term hardening in the pavement generally can be estimated from a knowledge of the asphalt content and volume of air voids.
5. The data from this study are available for studying the effect of temperature and shear susceptibility on pavement performance.

A study of variability of the properties of the pavement samples within and between projects indicated the following:

1. Pavements should be sampled at several random sites so that meaningful estimates of mixture and asphalt properties can be obtained.

2. Significantly lower sampling and testing variability of the bulk specific gravity measurements were obtained in 8-inch core samples than in 6-inch and 4-inch samples. Whenever possible, 8-inch cores should be used.
3. Based on the findings of this study, the authors recommend that average mixture and pavement properties should include standard deviations and coefficients of variation.

Although not included in this synthesis, several correlations of variability were made on properties of pavement and mixture samples from surviving and non-surviving pavements.

4.3 OKLAHOMA

Description

Nine test sites in Oklahoma that had various degrees of transverse cracking were studied and reported by Noureldin and Manke (1978A). The objective of the study was to determine the nature and extent of transverse cracks on selected pavements and investigate the causes of this form of distress. Four sections were located on US-177 and five sections on I-35 and I-40. Two of the interstate sites had almost no cracking and were chosen for comparative purposes. The original construction for the sections was 4.5 inches of asphalt concrete with 9 to 12 inches of stabilized base. A 1.5 inch overlay was placed on all sections except 5 and 8. At each test site, the pavement was surveyed in detail and transverse cracks (multiple, full width, and half width) were counted. The cracking index (C.I.) was then computed based on the procedure referenced by Fromm and Phang (1972). Crack depths were measured during coring operations.

Random core samples of 6 inches and 4 inches in diameter were obtained from each test site. The larger cores were used to study crack formation, and the smaller cores were used to determine low-temperature tensile properties. The cores were taken approximately 5 years after the overlay was placed. The asphalt was recovered from the core specimens, and stiffness modulus was determined using McLeod's (1972) pen-vis number (PVN). A temperature of -10°F (-23.3°C) at a pavement depth of 2 inches and a loading time of 20,000 sec. was assumed for the calculation after a study of climatological data for Oklahoma. The recovered asphalt properties and calculated binder stiffnesses are presented in Table B.7 and the mixture composition and calculated mixture stiffnesses are shown in Table B.8.

Results

The low temperature stiffness moduli of the recovered asphalt cements were related to the cracking index as shown on Figure 4.9. With the exception of site 7 on I-40, pavement sections with

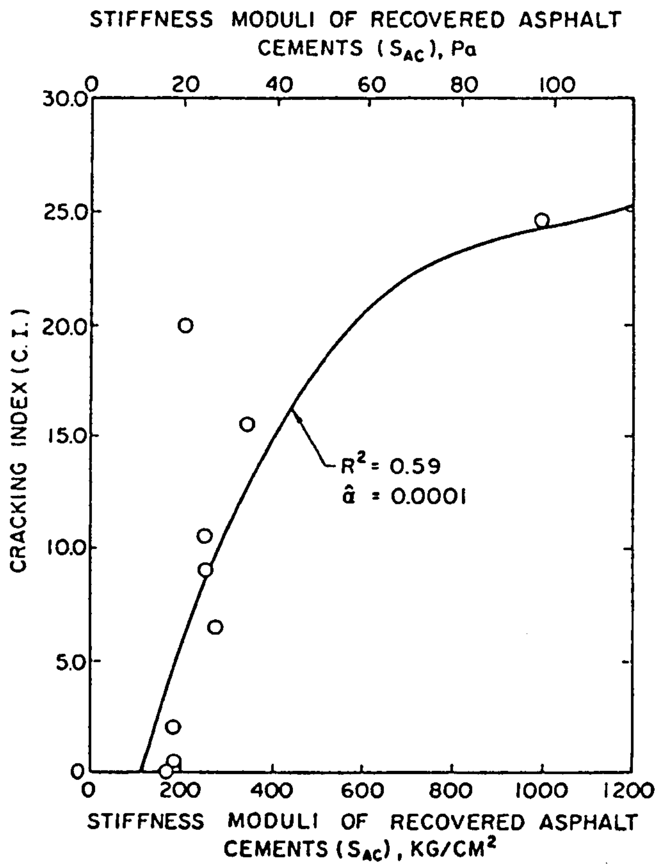


Figure 4.9 Relationship between stiffness moduli of recovered asphalt cement at -10°F and cracking index for Oklahoma (Noureldin & Manke 1978A)

a high degree of cracking were those that had the stiffer asphalt cements. The cracking index of site 7 was 20.0, but the stiffness modulus was relatively low. It is possible that the high frequency of cracking at this site was associated with a subgrade problem or other related factors. If the data for this site are disregarded, the coefficient of determination (R2) is 0.82. Noureldin and Manke (1978B) stated that "... this is a remarkably high value considering the variables involved in this generalized relation."

Conclusions

Two conclusions of the study in relating asphalt properties to transverse cracking (durability) were noted, as follows.

1. Cracks originated in the surface and appeared to have been caused by cold-temperature contraction of the asphaltic concrete surface layer.
2. The stiffness moduli of recovered asphalts, determined at the lowest minimum temperature in central Oklahoma using McLeod's (1972) PVN method, were correlated with the cracking indexes of the pavement test sites. The stiffer or harder the asphalt cement in a pavement, the greater was the degree of transverse cracking.

4.4 ONTARIO (Fromm & Phang)

Description

This investigation was designed to survey existing pavements where contrasting types of transverse cracking occurred within close proximity to each other, either on the same or different paving contracts. The objective was to identify those factors which differed between the projects and caused the cracking (Fromm and Phang 1972).

A field study was conducted during the summers of 1966 and 1967 to identify the causes of transverse cracking of asphalt concrete pavements in Ontario. A total of 33 paving projects were investigated. Crack counts were made and samples taken of all pavement layers and the subgrade soil. In addition, a laboratory analysis was made to determine the properties of the paving materials. Table B.9 summarizes the asphalt cement properties that were measured.

Results

The type and amount of cracking which was found on the various projects was quite varied. Four categories were used to differentiate between the types of transverse cracks found: (a) multiple transverse, (b) full transverse, (c) half transverse, and (d) part transverse. As a measure of the severity of the cracking, a Crack Index value was developed. This index was determined by adding the number of multiple and full transverse cracks with one half of the number of half transverse cracks occurring in a 500 foot stretch of two lane pavement.

The Cracking Index was used as the independent variable in a regression analysis of the factors that contribute to the low temperature cracking frequency and severity. Test results indicating the consistency of the asphalt cement binder materials were reduced to a single input value - the penetration of the recovered asphalt cement at 77°F. Further, the temperature susceptibility of the recovered binder was represented by a ratio of the viscosity at 60°F to the viscosity at 275°F.

The stepwise regression analysis included some 40 variables in the first run of the program. Additional runs resulted in the dropping of many variables that had low correlation coefficients. The final list of variables used in the analysis included: (a) viscosity ratio, (b) freezing index - degree days, (c) critical pavement temperature, (d) air void content, (e) stripping rating, (f) penetration of the recovered asphalt cement at 77°F, (g) percent asphaltenes, (h) amount of material passing the No. 200 sieve in the granular base course material, and (i) the amount of material passing the No. 200 sieve in the asphalt concrete aggregate.

For the general model, which includes all of Ontario, the correlation coefficient R (not R^2) was 0.636. The model for the northern part of the province had an R-value of 0.622 and for the southern portion of the province, R increased somewhat to 0.704. These equations are found in Table 4.1.

<u>Conclusions</u> (reported by Fromm & Phang)

The following conclusions were drawn from the results of the study:

1. Transverse cracking is largely a temperature phenomenon. The cracking is more severe in areas of high Freezing Index.
2. The transverse cracking of bituminous pavements can be reduced or retarded by using softer asphalts or asphalts of lower temperature sensitivity.
3. The stiffness modulus of the bituminous concrete at low winter temperatures is the major factor governing transverse cracking. The gradation of the base and subbase material has a small effect, but the major effect is concentrated in the bituminous pavement.

Table 4.1 Regression Equations for Ontario
(Fromm and Phang, 1972)

General Model:

$$Y = 52.22x_1 + 0.0007093x_2 + 0.4529x_3 - 1.348x_4 + 0.4687x_5 - 0.07903x_6 - 0.4887x_7 - 0.1258x_8 - 0.1961x_9$$

Multiple Correlation Coefficient R = 0.6357

Northern Model:

$$Y = 30.30x_1 + 0.00602x_2 + 0.5253x_3 - 1.280x_4 + 0.5190x_5 - 0.02563x_6 - 0.0844x_7 - 1.496x_8 + 0.225x_9 + 3.1043x_{10} + 0.097x_{11}$$

Multiple Correlation Coefficient R = 0.6222

Southern Model:

$$Y = 64.74x_1 + 0.008279x_2 + 0.3935x_3 - 1.491x_4 + 0.3246x_5 - 0.0001481x_6 - 0.6069x_7 - 0.8071x_8 - 0.6567x_9$$

Multiple Correlation Coefficient R = 0.7038

where:

- x_1 = viscosity ratio
- x_2 = freezing index
- x_3 = critical temperature
- x_4 = pavement voids
- x_5 = stripping rating
- x_6 = recovered asphalt penetration at 77°F
- x_7 = percent asphaltenes
- x_8 = granular base, pass #200
- x_9 = asphalt concrete, pass #200
- x_{10} = granular base, clay
- x_{11} = granular subbase, pass #4

4.5 ARIZONA

Description

Research projects in Arizona were begun in 1971 and 1972 to relate laboratory and field test data with field performance and the data reported in 1978 (Way 1978). A series of test locations and projects were identified and investigated including old road mixes built in the 1940's to modern mixes built in the 1970's. Table B.10 gives construction dates and additional information on mixture characteristics.

Results

Asphalt grades included 40/50 to 100/300 penetration asphalt, as well as cutbacks. One project, Minnetonka-East, contained all penetration grades of asphalt as part of a thin overlay. Various aggregate sources and types were also represented. Table B.11 gives considerable information about the as-constructed properties of the various asphalt cements. The oldest projects (Cutter and Show Low) built in 1940 and 1946 were road mixes with cutback asphalt used as the binder. Subsequent projects built from 1956 until 1975 were mixed in a hot plant, with paving grade asphalt. In 1976, four inch diameter cores were obtained and returned to the laboratory. These properties are shown in Tables B.12 and B.13.

Pavement performance measurements included Mays ride meter, average rut depth and average percent of cracking. The type of cracking (load or non load associated) was not distinguished. The percent of cracking was determined by comparison of photos of the test section with standard photos showing various levels of percent cracking. The rut depth measurements are shown in Table B.14 and the percent cracking estimates are shown on Table B.15.

Conclusions

No conclusions could be made regarding the relation between rutting or cracking and recovered asphalt properties. Rather, comparisons between the actual performance to predicted performance using the PDMAP performance prediction model described by Finn et al. (1977) were made instead.

4.6 WYOMING (Low Temperature Cracking)

Description

Fifteen pavement sites in Wyoming (Gietz & Lamb 1968) of varying ages and performance were sampled and the recovered asphalts and aggregates analyzed. These investigations were undertaken to examine the hardening of asphalt in service and the relationship of this hardening to the occurrence of lateral cracking. Temperature variant properties that had the most effect on pavement performances with respect to lateral cracking were examined.

The pavement sites were divided into 2 groups; Progressive and Performance. In the Progressive group, asphalt cement and uncompacted plant mix were obtained at the time of construction. In addition, samples from the compacted pavement were taken immediately after construction and at intervals thereafter for 2 years. The Performance group contained existing roads with service periods up to 8.5 years. The test results of the recovered asphalts of both samples are shown in Tables B.16 and B.17, respectively.

Results

In all samples, there was a considerable increase in softening point and a decrease in penetration with time. A greater decrease in penetration was noted on sites with a high void content. Pavements with frequent lateral cracking showed higher void contents in conjunction with greater hardening and oxidation as shown by penetration and the IGLC test. The Inverse Gas-Liquid Chromatography (IGLC) test is a procedure to measure the degree of interaction of the asphalt with various compounds. A higher numerical value for the Interaction Coefficient (I_p) indicates a greater degree of interaction with the asphalt. The results from this test are summarized in Table B.18. Davis & Petersen (1967) have shown that a high phenol I_p is associated with poor pavement performance in the Zaca-Wigmore test road (see Section 3.1). From the test sites in Wyoming, however, the results of IGLC test did not demonstrate any consistent variation of I_p values with sample age. Also, since projects on which cracking were noted were all cracked at the beginning of the project, it was not possible to correlate any actual level of aging with the inception of cracking.

Conclusions

The following conclusions were made:

1. Softening Point increases and penetration decreases with age.
2. Mixes with a higher void contents show a greater decrease in penetration.
3. IGLC test did not show any correlation with age.
4. No correlations were possible between aging and inception of cracking.

4.7 WEST TEXAS

Two studies were performed on pavements in West Texas. The first was by Benson (1976) which evaluated the relationship between asphalt hardening, thermal forces and low temperature transverse cracking. The second was performed by Anderson & Epps (1983) and was primarily concerned with the role of asphalt concrete in low temperature cracking. The focus of this second study was on mix properties.

Description (Benson 1976)

In the first study (Benson 1976), cores were taken periodically from over 50 highway projects, and a hardening model developed from these data. Controlled laboratory studies of asphalt specimens were also made. The effect of factors such as asphalt content, compactive effort, solar radiation and special addition on age hardening was then assessed.

Results (Benson 1976)

Out of the 50 highway projects, nine were eventually selected due to the fact that some sections had been sealed or overlaid or displayed no transverse cracking. These nine sites ranged in age from six to 10 years old. During the field surveys, the amount of transverse, longitudinal and alligator cracking was measured. In addition, Mays meter readings were also taken and serviceability indices calculated. Asphalt properties for the nine selected sites are summarized (cores were taken) on Table B-19. Also, Spearman's Rank correlation coefficient (r), used to test for the significance of the correlation between crack frequency and the asphalt properties, is included. From this table, it is apparent that viscosity at 77°F, hardening index at 77°F and penetration at 77°F all correlate significantly with transverse crack frequency. In addition, binder stiffness (but not mix modulus) and percent air voids also correlate significantly.

Other variables examined in this study included environmental factors such as precipitation, temperatures, freeze-thaw cycles and traffic. Surprisingly, none of these factors correlated with transverse crack frequency, although it was noted that different materials were used at each test site so there were varying tolerances to environmental conditions. However, it did indicate that material properties were more significant than environmental.

The laboratory specimens were included in the study to achieve a more precise control of all factors. Specimens were 17 inches in diameter by 2 inches thick, and compacted by a large gyratory compactor and were referred to as "pizzas." They were then placed outdoors for 3 years before they were cored and the recovered asphalts tested. Table B.20 summarizes these results which included penetration, viscosity, air voids and stiffness measurements. However, no clear pattern emerged from the data. Also, various hardening susceptibility indicators (Actinic light, Thin Film Oven Test, Aging Index and Vanadium content) were also studied and none were significant statistically (Table B.21).

Finally, a hardening model was developed to predict asphalt hardness for original properties. Two general models were developed of which the penetration model proved to have more practical benefit and was therefore selected for use.

1. Penetration model: $P = a + b \ln(t)$
2. Viscosity model: $V = at^b$

where:

P = Penetration at 77°F

V = Viscosity at 77°F (Megapoises, shear rate = 0.05 sec^{-1} sliding plate viscometer)

t = time from lay-down, months

a,b = non-dimensional coefficients derived from least square regression analysis

For penetration, the coefficient "a" and "b" represent indices for short and long term hardening, respectively. "a" is a function of original asphalt properties, mixing conditions and mix hardening susceptibility, while "b" is related to environmental factors and long-term chemical reactions. "a" is approximately one-half the original penetration, while a more reliable technique to predict "b" is required. Figure 4.10 illustrates the hardening curve for one site. The general models had correlations coefficient greater than 0.95, and generally follow experimental results reasonably well. It was also noted that the experimental errors for the lab results were much higher than for the field studies, primarily because of non-uniformity during the fabrication of the lab specimens and the use of less experienced personnel in the recovery and testing of the asphalt.

Description (Anderson & Epps 1983)

The second study (Anderson & Epps 1983), as mentioned earlier, focused primarily on mix properties. However, one of the objectives was to determine the contribution to cracking made by the binder, and to investigate the use of existing low temperature crack prediction models used in West Texas.

Six sections in West Texas were selected for detailed study, all were located on highways. All surfaces were constructed as overlays, using AC-20 with the exception of one site which used AC-10. All asphalt cements came from the same refinery. At each site, slabs were obtained and tested

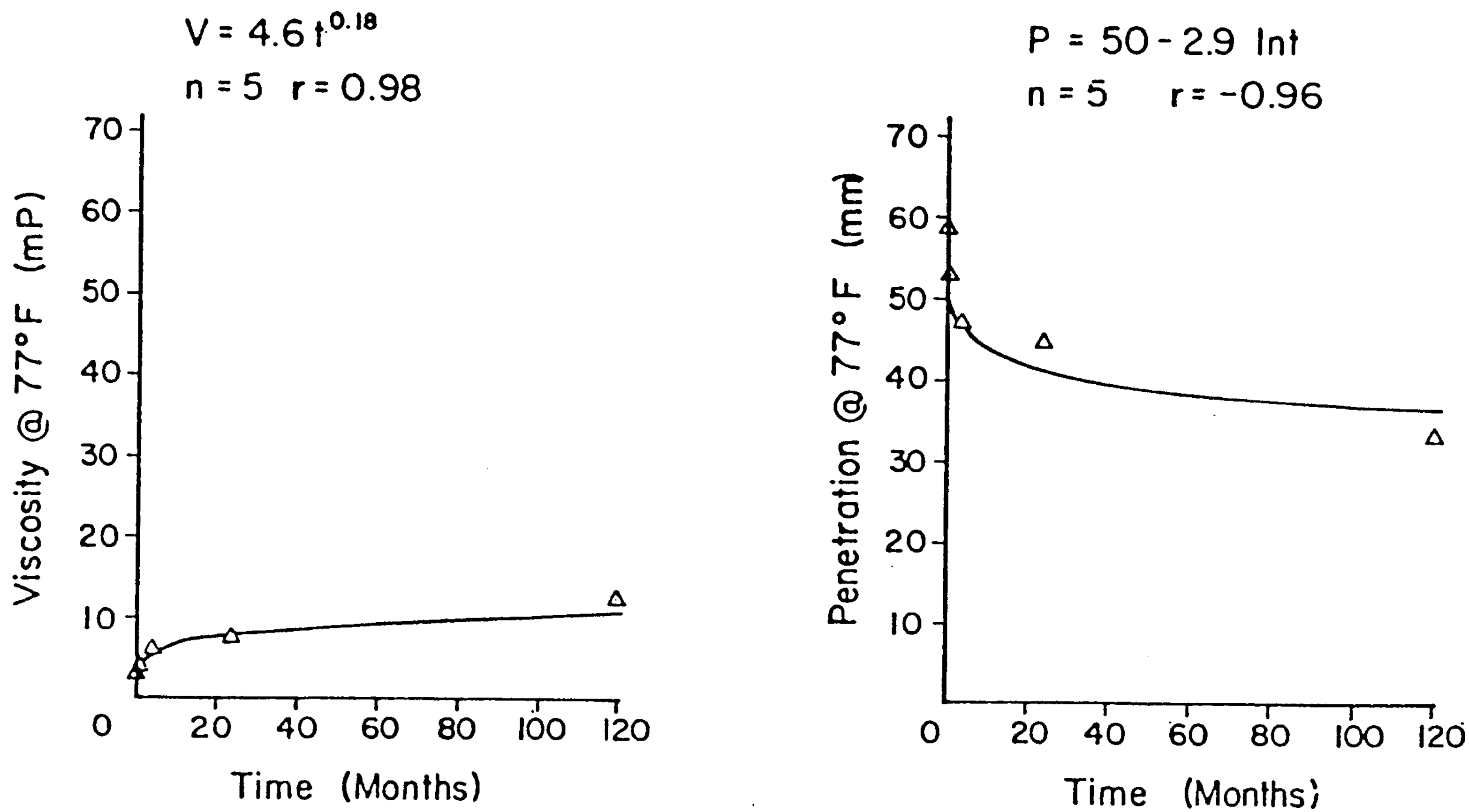


Figure 4.10 Hardening curves for Site 38 in West Texas (Benson 1976)

in the laboratory. The recovered asphalts were tested to determine penetration at 25°C and 4°C, viscosities at 25°C and 135°C and ring and ball softening point. Table B.22 summarizes the physical properties of these asphalt cements.

Results (Anderson & Epps 1983)

From the results, it was interesting to note that the AC-10 site (I-10) had the hardest asphalt rather than the softest. This could be explained by it being one or 2 years older and having lower asphalt content and high air voids. The results for each sample were plotted as Bitumen Test Data charts (Figure B.1) and there was very little variation in the slopes. From this, it was inferred that the temperature susceptibilities of all the recovered asphalts do not differ greatly from one another (using PI per Heukelom's method) and fell within the range expected for most U.S. produced asphalts.

All test sites were visually inspected and the serviceability index obtained from Mays meter runs. In addition, crack maps were drawn and Dynaflect measurements taken. These results are summarized in Table B.23a. Pavement ages at the time of inspection were 5 and 7 years, and 2 sections exhibited no cracking, while 2 others had extensive cracking. After 13 years, 3 sections had extensive transverse and block cracking while the same 2 sections still exhibited no cracking.

The Van der Poel nomograph was used to calculate the predicted cracking temperatures for each section as presented in Table B.23b. The predicted fracture temperatures were determined by calculating the temperature difference from the R & B softening point using a PI of -1.0, a loading time of 1/2 hour, and a critical stiffness of 1 x 10^9 N/m^2, and subtracting this value (90•C) from the actual softening point of the asphalt. Although the predicted cracking temperatures are all below the minimum expected pavement temperature of 8•F to 10•F, the section with the most cracking (I-10) comes within 7•F. The two sections that were uncracked (SH-18 and US-285 (186)) had the lowest predicted cracking temperatures.

For two test sections (I-10 and US-285 (186)), the low temperature cracking prediction model known as COLD (Finn et al. 1977) was run. Field observations showed that the section on I-20 contained cracking while US-285 (186) did not. The COLD program output for critical low temperature periods indicated that the induced stress due to temperature exceeded the tensile strength of the pavement on I-20 (i.e., predicted cracking) but the tensile strength was not exceeded on US-285 (186).

Conclusions

The authors concluded that while the predicted cracking temperature using the critical stiffness method does not yield precise and definitive results, it does demonstrate that asphalt cements recovered from pavements in West Texas are at least approaching the range of "critical stiffness" reported by other investigators at field temperatures. They also hypothesized that the more rapid cooling rates experienced in West Texas may increase the predicted cracking temperatures by justifying a shorter loading time.

In addition, they concluded that the COLD program produced predictions of low temperature cracking that agree with the observations of two field sections. They stated that time did not permit analysis of all six test sections; however, the validity of the approach appeared to have been demonstrated. Finally, the original counts had hardened to varying degrees to the range of 15 to 50 penetration at 25°C. Indicators of temperature susceptibility are inconclusive, although the use of PI or PVN's would appear to be reasonable.

4.8 TEXAS (Traxler)

Description

In 1964, 13 paving projects were located throughout the state of Texas (Traxler 1967a, 1967b) to determine the changes that occur in the asphaltic binder during the preparation, laying and service life of the pavement. Asphalt cements of 85-100 penetration from nine producers were studied. (Note: in Traxler's TTI report, 17 sites and asphalts from 10 producers were studied; however, four sites were removed from the AAPT report with no indication of the reasons). The pavements were constructed using normal construction procedures.

Results

Samples of the mixture at construction were taken, and slabs cut from the pavement after one day, 2 weeks, 4 months, 1 year and 2 years of service. Both original and recovered asphalts were tested for viscosity at 77°, 95°, 140° and 275°F. Ductility was measured using the California micro-susceptibility machine at 77°F. In addition, the original asphalts were tested for susceptibility to hardening by heat and oxidation (Dark air oven for 2 hours at 225°F). Asphaltene contents were determined after 1 and 2 years of service, and recovered petroleum tested for viscosity at 77°F. Tables B.24 - B.27 summarize these results.

A relative viscosity for each recovered asphalt was calculated to compare the hardening of the different asphalts. This relative viscosity (or aging index) is the ratio of the viscosity of the aged asphalt to the viscosity of the original asphalt. Table B.28 summarizes these results. It was found that the combined mixing cycle resulted in a 1.55 to 2.80 fold increase in viscosity. After 3 years of service, the pavements had ratios ranging from 13.5 to 41.5. The relative viscosity of the lab-aged asphalts ranged from 2.55 to 6.35 which is comparable to the 2.2 to 7.6 increase shown after 2 weeks of service.

Conclusions

The following conclusions were obtained from these tests:

1. A small increase in viscosity occurs during the preparation of the mix, laying and the first 2 weeks of service. Between the 2 weeks and 2 years of service, there is a greater increase in viscosity. However, the increase in viscosity varies among the different asphalts combined with different aggregates under various service conditions.
2. The results of the chemical analysis showed that asphaltene contents generally increased by 1.4 - 8.3% after 1 year. After 2 years, three of the asphalts had a reduced amount of asphaltenes, 1 - 1.4%; however, the remaining eight continued to increase in asphaltenes.
3. The viscosity of the petrolenes after 2 years were 1 to 20 times higher than the original asphalts. The 20 fold increase occurred on the site where limestone aggregates had been used. It was concluded that low viscosity oil had probably been drawn into the capillaries of the limestone.
4. Nuclear activation analyses indicate that oxygen content was higher after 4 months. After 1 year, it decreased slightly then increased or remained the same after 2 years.

4.9 SOUTH AFRICA

Description (Jamieson and Hattingh 1970)

Jamieson and Hattingh (1970) reported the results of comparisons between physical and chemical bitumen properties associated with performance after six years under South Africa coastal conditions (temperate to warm and humid climate). Eighteen bitumens were studied. Details of the test asphalts are shown on Table B.29.

Results (Jamieson and Hattingh 1970)

A comparison of physical and Rostler-Sternberg chemical precipitation test data in relation to field performance is shown on Table 4.2. The authors showed that under the South African coastal

Table 4.2 Chemical and physical test data for predicting road performance and actual performance of premix surfacings for South Africa (Jamieson & Hattingh 1970)

Sample No.	Pen. Grade	Details of Bitumen Type	Premix Type	Experimental Site	Service Performance Assessed by Road Inspection Panel* (After 6 years)	CACULATED DATA FOR ASSESSING DURABILITY — Chemical Composition: Component Ratio $\frac{N+A_1}{A_2+P}$	Chemical Composition: Modified Component Ratio $\frac{A_1+A_2+N}{A+P}$	Physical Properties: Pen./log Ductility on T.F.O.T. Residues	Physical Properties: Hardening on the Road as % Retained Pen. (After 22 Months)	Physical Properties: Increase in %age of Asphaltenes After 22 Months Service
734	150/200	Straight-run M.E.	Continuously-graded (Grading No. 5)	Urban	FG+d_1	1.40	2.48	<48	32.6†	4.0†
736	80/100	Straight-run M.E.			FG+d_1	1.57	2.14	<25	56.0	5.4
744	60/70	Straight-run M.E.			FG+vc_1	1.30	2.41	<20	51.6	6.6
752	80/100	Air-rectified M.E. (exptl.)			P—c_3‡	0.95	1.11	53	39.6	8.3
758	80/100	Air-rectified M.E. with cat. (exptl.)			P—c_5‡	0.54	0.99	42	33.7	8.5
754	60/70	Air-rectified M.E. (exptl.)			F—c_3d_1	0.93	1.26	44	37.8	6.6
762	80/100	Trinidad petroleum			F—c_5d_1	1.90	3.26	<31	28.7	7.5
765	60/70	Trinidad petroleum			FG—c_1d_1	1.39	.1.35	<18	26.6	8.3
778	80/100	50% TLA+50% Straight-run M.E.	Continuously-graded (Grading No. 6)	Urban	FG—vc_1	1.68	1.48	<30	45.5	0.0
777		25% TLA+75% Straight-run M.E.			G—vc_1d_1	1.39	1.45	<30	53.5	2.9
779		Thermally "cracked"			B—c_5‡	1.95	2.82	<18	10.8†	17.6†
883	80/100	Straight-run M.E. (Refinery "M")	Continuously-graded (Grading A)	Rural	Gv	1.33	2.54	<25	(after 3½ years) 53	—
885		Air-rectified M.E. (Refinery "M")			G—	1.27	2.25	<22	63.5	—
884		Air-rectified M.E. (Refinery "S")			Gvc_1	1.19	1.94	<25	52.5	—
749	40/50	Straight-run M.E.	Gap-graded (BS 594, 55% stone content)	Urban	G+v	1.49	2.38	<14	(after22 months) 72.5	9.3
756		Air-rectified M.E. (exptl.)			FG—c_1	1.00	1.03	57	94.0	6.1
760		Air-rectified M.E. with cat. (exptl.)			F—c_3	0.64	0.91	75	75.0	2.8
767		Trinidad petroleum			G—vc_1	1.65	1.81	<15	40.9	12.8

† Recovered after 36 months on the road

‡ Assessment after 5 years when the section required resurfacing

* Explanation of Symbols

† VG — Very good
G — Good
FG — Fairly good
} Satisfactory condition

F — Fair
P — Poor
B — Bad
} Unsatisfactory condition

Subscript signs indicate the following:
Minus sign — loss of stone
Plus sign — excess binder
"v" — general variability
"c" — cracking (degree of severity denoted by a numerical scale, 1, 3 or 5)
"d_1" — slight deformation

conditions, where these experiments were laid, the results of chemical analyses on the bitumens expressed as a ratio of the more reactive to the less reactive components (component ratio) should be 1.0 to 1.7 for optimum performance and not 0.8 to 1.5, as suggested by Halstead, Rostler and White (1966).

Description (Hattingh 1984)

Hattingh (1984) reported the results of another study in South Africa in which the performance of roads constructed with four locally produced 150/200 pen asphalts were compared to ductility measurements and high pressure liquid chromatography (HPLC) and Gel Permeation chromatography (GPC) determinations. The asphalt properties are shown on Table B.30. All asphalts met the South Africa specifications with the exception of a marginally high penetration for Asphalt A.

Results (Hattingh 1984)

Immediately after construction, the roads constructed with Asphalts A and B bled profusely while those constructed with Asphalts C and D were still in satisfactory condition after 9 and 3 years, respectively. Performance ratings in relation to the asphalt properties are given in Table 4.3. Figure 4.11 shows the molecular mass profiles after GPC analysis for asphalts C and D are compared with those of asphalts A and B. There is a difference in the concentration of the high molecular mass constituents at a retention time of approximately 18.0 minutes. The author concluded that the setting problems exhibited by asphalts A and B were probably due to:

a. Their low asphaltene contents, and
b. Insufficient high molecular mass components in the asphaltenes.

It should be noted that the report did not state whether the bleeding may have been related to factors other than asphalt characteristics, such as air voids, traffic, etc.

Conclusions

1. More fractionation data from asphalts that have been used in road construction are required before any conclusions can be drawn as to whether a relationship exists between the chemical fractionation results and in-service performance.
2. A certain percentage of asphaltenes is necessary to ensure that an asphalt performs well on the road. Too high a percentage of asphaltenes leads to cracking and too low a percentage causes setting problems.

Table 4.3 Asphaltene contents ductility measurements, extended TFOT residues, and road performance assessments of four 150/200 pen original asphalts for South Africa (Hattingh 1984)

	Asphalt A		Asphalt B		Asphalt C		Asphalt D	
	Orig.	ETFOT 32½ h	Orig.	ETFOT 32½ h	Orig.	ETFOT 32½ h	Orig.	ETFOT 32½ h
Ductility (mm)	+1400	+1400	+1400	+1400	+1400	65	+1400	870
Performance	Bad	-	Bad		Good	-	Good	-
Asphaltenes (%)	13.7	24.0	15.3	26.3	20.7	32.0	17.9	26.9
Increase in Asphaltenes after ETFOT	-	10.3	-	11.0	-	11.3	-	9.0

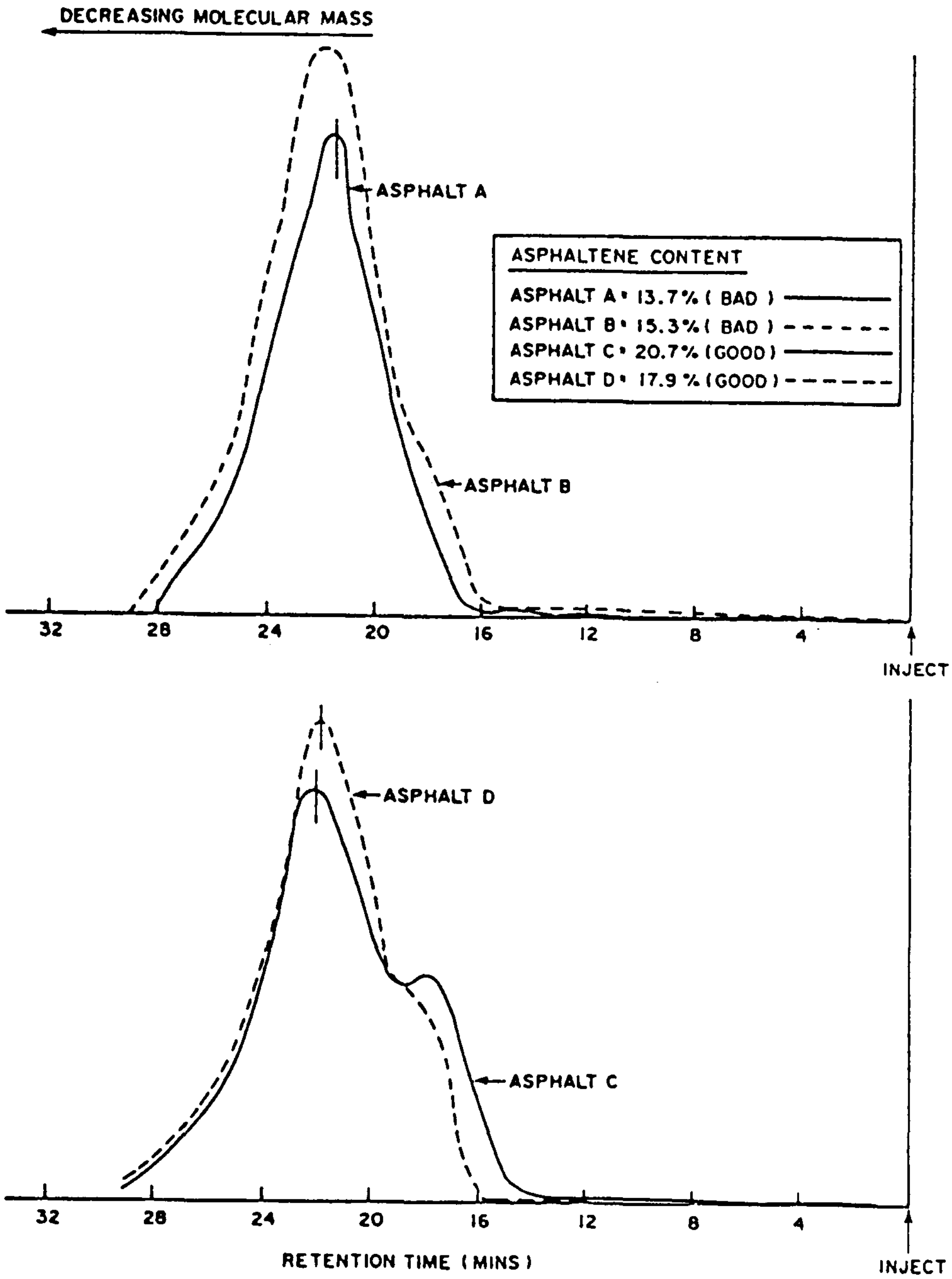


Figure 4.11 Molecular mass profiles of four 150/200 pen whole asphalts for South Africa (Hattingh 1984)

3. Asphaltene content alone does not provide sufficient data for the evaluation of the quality of an asphalt; it must be used in conjunction with the molecular mass distribution, i.e., the size and amount of the large molecular size material present, to give an indication of the road performance that can be expected from a particular asphalt.
4. Ductility measurements carried out on unweathered and artificially weathered asphalts seemed to differentiate between asphalts giving a satisfactory or unsatisfactory performance on the road.

4.10 PLUMMER AND ZIMMERMAN

Description

A study to determine if molecular size distribution and/or hardness results correlated with observed pavement cracking in Michigan and Indiana was reported by Plummer and Zimmerman (1984). Eleven roads were studied in Michigan and eight Indiana roads were studied. The age range of the cracked Michigan roads was six to twenty years and the range for uncracked Michigan roads was nine to twenty-two years. The age of both the cracked and uncracked Indiana roads ranged from one to five years. Tables B.31 and B.32 provide construction information on the roads studied.

Results

Tables 4.4 and 4.5 provide results of penetration and high performance liquid chromatography (HPLC) data for each set of roads. Larger asphalt MMSF values (ie. larger mean molecular sizes) were obtained from cracked roads than from uncracked roads. However, the MMSF value differentiating cracked from uncracked roads varied between Michigan and Indiana. The authors concluded that the difference is in agreement with the fact that harder asphalts are usually used for more southern locations.

Conclusions

Based on the HPLC results, the authors concluded that the observed cracking in both states was related to asphalts with lower penetration (average of 61 vs. 48 for Michigan) and higher asphalt mean molecular size (approximately 1 percent increase).

These conclusions are statistically supported at the 95 percent confidence level. The authors stated that the asphalts supplied to the Michigan roads which eventually cracked also might have had lower penetration and higher mean molecular size than those supplied to the roads which did not crack. However, this conclusion is statistically supported at only a 76 percent confidence level.

Table 4.4 High performance liquid chromatography (HPLC) results for asphalts extracted from Michigan roads (Plummer & Zimmerman 1984)

Sample No.	Road Data No.	Road Data County	Road Data Year	Asphalt Data Penetration(1) Supplied	Asphalt Data Penetration(1) Road	Asphalt HPLC Results(2) D_l	D_s	D_l/D_s	MMSF
Roads Exhibiting No Significant Cracking									
1	US-31	Muskegon	1963	66	53	30.1	22.3	1.35	228.0
2	I-96	Ottawa	1961	64	46	27.5	20.2	1.36	231.0
8	State 52	Ingham	1972	97	75	29.6	21.6	1.37	230.0
9	State 52	Ingham	1972	85	62	28.8	20.9	1.38	230.2
10	State 52	Ingham	1972	93	68	28.8	21.4	1.35	230.3
	Average			83	62	29.0	21.3	1.36	229.9
Roads Exhibiting Cracking									
3	US-27	Clare	1961	66	46	28.5	20.6	1.39	231.2
4	US-27	Clare	1961	64	43	28.2	20.7	1.36	230.8
5	US-27	Clare	1961	66	47	28.7	20.7	1.39	230.7
6	US-27	Roscomon	1961	93	66	29.3	21.6	1.35	231.3
6A	US-27(3)	Roscomon	1976	64	43	30.6	21.6	1.42	231.3
7	US-27	Roscomon	1961	64	43	28.8	21.3	1.35	231.3
	Average			70	48	29.0	21.1	1.38	231.1
Confidence Level On Cracking Predictions				76%	95%	None	31%	66%	96%

Comments

(1)Penetrations (77°F) were measured during road construction.

(2)MMSF = mean molecular size factor. D_l and D_s are, respectively, the lognormal Gaussian standard deviations for molecular sizes larger and smaller than the MMSF.

(3)Road 6A is an overlay on road 6.

Table 4.5 High performance liquid chromatography (HPLC) results for asphalts extracted from Indiana roads (Plummer & Zimmerman 1984)

Sample	Road Data			Asphalt HPLC Results[1]			
No.	No	County	Year	D_l	D_s	D_l/D_s	MMSF
Roads Exhibiting No Significant Cracking							
1	I-64	Crawford	1976	27.9	22.2	1.26	233.7
2	I-64	Crawford	1976	30.1	23.8	1.27	236.0
3	State 37	Perry	1979	29.9	23.2	1.29	234.0
4	State 37	Perry	1979	30.7	23.7	1.29	234.9
	Average			29.6	23.2	1.28	234.6
Roads Exhibiting Cracking							
5	I-64	Crawford	1976	33.7	26.4	1.28	238.5
6	State 37	Hamilton	1980	31.3	24.6	1.27	237.1
7	State 37	Perry	1979	33.1	25.6	1.29	237.3
8	State 37	Perry	1980	30.6	24.1	1.27	234.7
	Average			32.2	25.2	1.28	236.9
Confidence Level On Cracking Predictions				94%	96%	None	93%

Comments

[1]MMSF = mean molecular size factor. D_l and D_s are, respectively, the lognormal Gaussian standard deviations for molecular sizes larger and smaller than the MMSF.

It is also possible that the decreased penetration and increased mean molecular size in asphalts from cracked Michigan roads could have occurred from severe mix plant operations or use of different aggregate types. In Indiana, only the asphalt supplied prior to mixing with the aggregate was sampled. The penetration measurements obtained on the supplied asphalts did not show any significant variation between cracked and uncracked roads.

4.11 PENNSYLVANIA (1961-62)

Description

Pennsylvania constructed experimental test roads in 1961 and 1962 in order to evaluate the durability of asphalt surfaces constructed with various asphalt cements (Kandhal 1976, Kandhal & Koehler 1984).

In October 1961, two test pavements were constructed in Lycoming and Beaver counties; and in June 1962, two were constructed in Washington and Lebanon counties. These four test pavements were asphaltic concrete overlays, consisting of 2-inch binder and 1-inch wearing course, placed on 9-inch portland cement concrete pavements. Each wearing surface contained a different type of asphalt. Slab aggregate was used in Beaver and Washington counties, whereas limestone aggregate was used in Lycoming and Lebanon counties. All mixtures were designed according to the Marshall method. The construction methods used on the four jobs were basically similar. Average daily traffic varied from a low of 2850 in Washington county to a high of 6600 in Lycoming county.

Results

Asphalt properties for the 1961-1962 test pavements are summarized on Table B.33. Since construction of the test pavements, periodical core samples were obtained to determine the percentage of air voids in the pavements and the rheological properties of the aged asphalts. When constructed in October 1961 and June 1962 and visually inspected during November 1963, all four test pavements appeared satisfactory. Visual evaluation included riding quality, loss of fines, raveling, and cracking. Pavement condition surveys were conducted annually thereafter. The ranking orders of pavement performance after 10 years service, percentage of air voids, penetration at 77°F, viscosity at 140°F, and ductility at 60°F, are given in Table 4.6. Pavements in Beaver, Lycoming, Washington, and Lebanon counties are numbered 1, 2, 3, and 4, respectively.

Table 4.6 Ranking orders at Pennsylvania (1961-1962 pavements) (Kandhal 1976)

Pavement Performance	Pavement Air Voids	Penetration at 77°F	Viscosity at 140°F	Ductility at 60°F, 5 cm/min
3 (poorest)	3 (highest)	3 (lowest)	3 (highest)	3 (lowest)
2	4	4	2	2
1	1	2	1	1
4 (best)	2 (lowest)	1 (highest)	4 (lowest)	4 (highest)

Conclusions

The authors suggested that the asphalt ductility value obtained at 60°F is a good indicator of pavement performance. In addition, it was found that the viscosity also related to performance; however, the authors cautioned that later studies did not always confirm this relationship. It was noted that the pavement condition was satisfactory when ductility at 60°F was maintained above 10 cm. However, load-associated cracking began to develop when the ductility value fell in the approximate range of 3 to 5 cm.

4.12 SISKO AND BRUNSTRUM

Description

Viscoelastic properties of twelve asphalt cement samples used in road construction for a number of different states were measured to try to relate to observed pavement performance (Sisko and Brunstrum 1968 & 1969). Pavements were observed after 11 years service with the exception of Illinois which was observed after 3 years. The roads chosen were from a survey of pavements constructed in 1954 and 1955 by the Bureau of Public Roads.

Properties of the asphalts unaged and after TFOT are shown in Table B.34. The asphalts were 70-80 and 85-100 penetration graded and represented a variety of crude and manufacturing sources. Table B.35 gives physical properties of recovered asphalts extracted from the roads. Table B.36 provides information relative to the composition of the asphalt as determined by a combined solubility and chromatographic procedure. A summary of the road evaluations is presented in Table B.37.

Results

A Weissenberg rheogoniometer, modified to improve temperature control and accuracy of strain measurements, was used to measure viscoelastic properties on retained unaged and aged asphalts after 11 years service and on residues from the TFOT. The frequency of stress application ranged from 4×10^{-7} to 10^{2} cycles/second, and the test temperature range from 0 to 80°F. Measurements at various frequencies and temperatures were reduced to master curves at a 20°F reference temperature and 10^{-8} cycles per second frequency (Sisko and Brunstrum 1969). The relationship of complex modulus and pavement performance is shown on Table 4.7.

Conclusions

Conclusions from the study that concern the relationship of asphalt cement properties to pavement performance follow:

Table 4.7 Relationship of asphalt complex modulus and pavement performance (Sisko and Brunstrum 1969)

BPR NO. OR STATE	COMPLEX MODULUS X 10^{-4} (DYNES/CM ^{2}) n				ROAD EVALUATION
	UNAGED	TFOT	MARSHALL	ROAD	
California	0.32	2.1	---	33	Moderate cracking
Kentucky	0.60	6.0	---	410	Severe cracking
Illinois	2.4	17	---	22	No change
Maryland	0.55	3.9	---	130	Severe cracking
Massachusetts	0.42	3.4	---	34	Severe cracking
Nebraska	0.96	6.2	---	7.0	Mod. plastic deformation, slight cracking
Oklahoma	0.40	1.9	---	2.4	Mod. plastic deformation
Oregon	0.55	7.2	---	18	Slight cracking
Tennessee	0.23	2.1	---	140	Slight cracking, large plastic deformation
Texas	2.4	7.8	---	120	No change
Wisconsin	1.3	8.2	---	210	Moderate cracking
Wyoming	0.35	1.9	---	9.6	Slight cracking, slight plastic deformation

*At 10^{-8} cps, 20°F.

1. Large increases, induced by aging, in the hardness of the asphalt binder (as by complex modulus) are associated with road-cracking.
2. The large differences in viscoelastic properties that can develop during road-aging are accumulated ESAL's, fractured faces, air voids, VMA, due partly to intrinsic differences in aging resistance and partly to external factors.
3. The age-hardening of asphalts in the road does not correlate directly with the amount the asphalts hardened in the TFOT.

4.13 SASKATCHEWAN (Rutting)

Description

This study was an attempt to associate pavement rutting to mixture properties on highway segments in Saskatchewan (Huber & Heinman 1987). The type of rutting investigated was plastic deformation of the asphalt concrete layers. Structural design of the pavement was not addressed.

Field sites chosen for evaluation included both good and poor performing sections, sections with fast and slow moving traffic, sections with varying asphalt cement consistency, and both old and recently constructed sections. A list of sites chosen is shown in Table B.38.

A testing program was performed to characterize the asphalt concrete mixtures in various stages. A summary of the tests used is shown in Table B.39. It should be noted that the only asphalt cement properties measured in this study were recovered penetration and viscosity. These data are shown, along with additional data obtained during field sampling, in Table B.40. The study concentrated more heavily on the influence of asphalt mixture properties as opposed to asphalt cement properties.

Mix design information was obtained from historical records and using construction records. Post-construction was defined as the condition of the material immediately after construction. Since actual post-construction information available from the field quality control testing lacked coincidence between test locations and study sites, cores were obtained between the wheelpaths to identify post-construction parameters.

The present condition of the asphalt materials was defined as the condition at the time of testing. Cores were obtained in the outer wheelpath to identify these properties.

Results

A regression analysis was performed to identify if any of the laboratory measured properties correlated to rutting performance. Two methods quantifying rutting performance were used. The first was rut depth expressed in millimeters; the second was rutting rate expressed as millimeters per million equivalent axle passes. Each of these performance measures was correlated to both the post construction condition and the present condition. The individual parameters regressed to rutting performance include accumulated ESAL's, fractured faces, air voids, VMA, Hveem stability, Marshall stability, asphalt content, asphalt penetration and viscosity, percent voids filled and Marshall flow. A summary showing the results is shown in Table 4.8.

Rut depth was not found to correlate well with any of the asphalt or mixture properties. The rate of rutting was found to correlate well to air voids, voids filled, asphalt content and Hveem stability. Marshall stability, flow, penetration and viscosity of the asphalt showed little correlation with rutting.

A threshold analysis was also performed for post-construction data which indicated a significant correlation in the regression analysis; namely air voids, VMA, asphalt content, voids filled, fractured faces, Marshall stability and Hveem stability. Penetration and viscosity were not included due to the poor relationships found during regression analysis. Results are presented in Table 4.9. The results indicated that air voids and voids filled (which is a function of air voids) appear to be the most dominant parameter in defining rutting performance.

Conclusions

Among the conclusions gained from this study are the following:

1. Asphalt content and voids filled are the most basic parameters which affect rutting. Voids filled includes the effects of both air voids and voids in the mineral aggregate.
2. Marshall stability and flow do not show any independent effect on rutting performance.
3. Penetration and viscosity of the asphalt do not demonstrate a significant effect on rutting rate.
4. Mature pavements begin to exhibit plastic deformation when the air voids and the voids filled exceed the identified threshold values.

4.14 CALIFORNIA

Description

The California Transportation Laboratory performed an asphalt durability study involving the weathering of carefully controlled and fabricated briquettes in 4 different field environments in the late

Table 4.8 Summary of regression analysis using average parameter data for Saskatchewan rutting study (Huber & Heiman 1987)

	POST-CONSTRUCTION				PRESENT CONDITION			
	RUT DEPTH, mm		RUTTING RATE, mm/10^6 ESAL		RUT DEPTH, mm		RUTTING RATE, mm/10^6 ESAL	
PARAMETER	CORRELATION r^2	PROBA-BILITY	CORRELATION r^2	PROBA-BILITY	CORRELATION r^2	PROBA-BILITY	CORRELATION r^2	PROBA-BILITY
Accumulated ESAL's	0.099	0.410[1]	-	-	0.099	0.410	-	-
AC Content	0.134	0.332	0.673	0.007	0.082	0.456	0.667	0.007
Fracture	0.269	0.153	0.229	0.193	0.074	0.516	0.004	0.881
Air Voids	0.221	0.202	0.483	0.038	0.182	0.252	0.234	0.187
Voids in the Mineral Aggregate	0.111	0.379	0.124	0.352	0.001	0.927	0.262	0.159
Hveem Stability	0.211	0.213	0.490	0.037	0.279	0.144	0.489	0.036
Marshall Stability	0.111	0.380	0.056	0.540	0.019	0.723	0.227	0.194
Penetration	-	-	-	-	0.004	0.879	0.187	0.245
Viscosity	-	-	-	-	0.042	0.599	0.082	0.456
Voids Filled	-	-	0.530	0.026	0.211	0.213	0.282	0.141
Flow	-	-	0.027	0.301	-	-	-	-

1 - Probability that the parameter does not effect on the rutting performance. eg, 0.410 means 41% chance exists that accumulated ESAL's does not effect rut depth.

Table 4.9 Threshold analysis results for Saskatchewan rutting study (Huber & Heinman 1987)

PARAMETER	THRESHOLD VALUE
Air Voids*	4% minimum
Voids in the Mineral Aggregate	13.5% minimum
Asphalt Content	5.1% maximum
Voids Filled	70% maximum
Fractured Faces	60% minimum
Marshall Stability	---
Hveem Stability	37% minimum

*Note: Authors report this as a minimum rather than a maximum.

1970's (Kemp & Predoehl 1981). The purpose of the study was to compare the effects of different field environments on asphalts in briquettes to the effects produced on the same asphalts by various laboratory accelerated weathering procedures. In addition, asphalt test road in Calipatria was studied. The Calipatria test road used a study asphalt from the desert weathering site in Indio. The briquettes were aged and tested at 1, 2 and 4 years of age. The Abson recovery procedure (AASHTO T170) was used and the asphalts tested for penetration, viscosity and shear susceptibility. Not all the results from these tests have been included as only the desert site had corresponding field data. The asphalts used came from Kern Valley, LA Basin and Santa Maria. Four weathering sites with distinctive climates that represented major portions of California were selected for the briquettes. The Mountain climate was represented by South Lake Tahoe, Valley by Sacramento, Coastal by Ft. Bragg and Desert by Indio.

In November 1977, thick AC overlays (0.20 feet and 0.35 feet) were placed over a road at Calipatria, which is south of Indio. The LA Basin asphalt was used in the paving, and samples of the asphalt and mix were obtained during paving. In addition, cores were taken 2 years later. The test results of the asphalts recovered from these cores were then compared to the 2-year Indio test results to verify the authenticity of the field weathering procedure used in the durability study. Tables 4.10 and 4.11 show the results for the test section as well as on the briquettes. Note that only the top 2 inches of the cores were used to recover the asphalts so as to minimize the effects of underlying layers and the tack coat.

Results (Reported by Caltrans)

Since Indio and Calipatria have approximately the same climate, a comparison of rate of hardening results for the Indio briquette and Calipatria overlay was accomplished. Table 4.12 contains a comparison of the road results with 24-month Indio results. It can be seen that there is a fairly good correlation between the results. The average results from California are slightly softer, indicating that the briquette weathering is slightly more severe than that of an actual overlay. The data suggests that 24 months of briquette weathering time would approximately represent 32 months of road weathering time. The authors believe that while this correlation is only for the LA Basin asphalt, the other asphalts would also behave similarly.

In addition to the above, several laboratory procedures for predicting asphalt handling were studied. Compositional tests included the Rostler fingerprinting procedure (to obtain the Rostler ratio), the Heithaus method to obtain the state of peptization and vanadium content determination. Accelerated weathering procedures included the (1) Rolling Thin Film Durability Test (5 hours at 325°F), (2) Rolling Microfilm Circulating Oven Durability Test (RMF-C at 210°F for 48 hours), (3)

Table 4.10 Original asphalt test results for Calipatria test section (sampled 11-30-77) (Kemp & Predoehl 1981)

Asphalt Source L.A. Basin
Grade AR4000

Tests on Original Asphalt	AASHTO Designation	Section 5 R4940	Section 4 R4941	Section 6 R4942	Section 7 R4943
Penetration at 77F (25C) (0.1mm)	T 49	52	52	53	53
at 39.2F (4C)(0.1mm)	T 49	12	12	11	11
Penetration Ratio $\frac{39.2F Pen.}{77F Pen} \times 100$		23.1	23.1	20.8	20.8
Flash Point C.O.C. (°F)	T 48	545	535	535	540
Softening Point (°F)	T 53	123	123	124	124
Solubilityin Trichloroethylene(%)	T 44	99.98	99.99	99.99	99.99
Spot Test (heptane-Xylene Equivalent)	T102	20-30	20-30	20-30	20-30
Specific Gravity at 77F	T228	1.0199	1.0205	1.0201	1.0206
at 60F	T228	1.0259	1.0265	1.0261	1.0267
Absolute Viscosity at 140F(60C)(Poise)	T202	2166	2089	2056	2091
Thin Film Loss (325F,5hr.)(% Loss)	T179	0.117	0.13	0.128	0.116
Rolling Thin Film Test (325F,85min)	T240				
Tests on RTF Residue					
Absolute Viscosity at 140F(60C)(poise)	T202	4375	4166	4164	4220
Kinematic Viscosity at 275F(135C) cSt	T201	363	348	363	383
Penetration at 77F(25C)(0.1mm)	T 49	34	34	33	32
% original penetration at 77F.	T 49	65.4	65.4	62.3	60.4
Ductility at 77F, 4cm/min. (cm)	T 51	100+	100+	100+	100+
at 39.2F, 1 cm/min. (cm)	T 51	0.25	0.25	0.25	0.5

Tests on AC Mix Samples from Road		Section 4	Section 5	Section 6	Section 7
Penetration at 77F (25C) (0.1mm)	T 49	28	29	28	26
Softening Point, (°F)	T 53	136	137	136	137
Ductility at 77°F (25C)5cm/min. (cm)	T 51	100+	100+	100+	100+
Absolute Viscosity at 140°F(60C)(poise)	T202	5926	6097	5280	5872
Kinematic Viscosity at 275°F(135C)(cSt)	T201	453	465	468	468

Table 4.11 Comparison of original properties of L.A. Basin asphalt used in durability study and L.A. Basin asphalt from Calipatria test section (Kemp & Predoehl 1981)

(Average Results)

Common Tests	AASHTO Method	Durability Study	Calipatria Test Section
Test on Original Asphalt			
Penetration at 77F (25C)(0.1mm)	T 49	58	53
Flash Point, COC (°F)	T 48	525	539
Solubility in Trichloroethylene (%)	T 44	99.96	99.99
Specific Gravity (77F/77F)	T228	1.019	1.0203
Absolute Viscosity at 140F(60C)(poise)	T202	1816	2100
Rolling Thin Film Procedure	T240		
Tests on RTFC Residue			
Absolute Viscosity at 140F(60C)(poise)	T202	3509	4231
Kinematic Viscosity at 275F(135C)(cSt)	T201	339	364
Penetration at 77F (25C)(0.1mm)	T 49	41	33
% Original Penetration at 77F (%)	T 49	71	63
Ductility at 77F (25C)(5cm/min)(cm)	T 51	150+	100+
Test on Residue from "California Tilt-oven Durability Test."			
Absolute Viscosity at 140°F(60°C) (Kilopoise)	T202	50.2	56.6
Kinematic Viscosity 275°F(135C)(cSt)	T201	1038	1044
Penetration at 77°F(25°C)(0.1mm)	T 49	11	10
Ductility at 77°F(25C)5cm/min.(cm)	T 51	12	12

Table 4.12 Average results from cores of Calipatria test sections (Kemp & Predoehl 1981)

Test Site Calipatria

Weathering Period (Months)	Asphalt Source	SAMPLE No	AASHTO Test Method					California Test Method No.			
			Pen. at 77°F (0.1 mm)	S.P. °F	Ductility at 77°F (cm)	A. Viscosity at 140°F (Kilopoise)	Kinematic Viscosity at 275°F (CST)	Micro-Viscosity (megapoise) at 0.05 sec^{-1} Shear rate	Micro-Viscosity (megapoise) at 0.001 sec^{-1} Shear rate	Shear Suscep-tibility (Slope)	Micro-Ductility at 77°F (mm)
ABSON RECOVERED AASHTO T 170			T 49	T 53	T 51	T 202	T 201	348		348	349
26	LA Basin	80260	12	148	96	31.5	974	59.8	108.0	.15	16
26	LA Basin	80261	11	148	55	31.4	890	79.5	124.0	.11	0
26	LA Basin	80262	11	146	30	39.7	1006	120.0	163.0	.08	4
26	LA Basin	80263	13	145	82	29.9	926	64.5	107.0	.13	16
26	LA Basin	80264	13	147	95	28.6	357	76.0	103.0	.08	11
26	LA Basin	80265	11	149	68	34.8	936	103.0	166.0	.12	4
26	LA Basin	80266	10	150	31	39.1	1075	118.0	177.0	.10	0
AVERAGE			12	148	65	33.6	952	88.7	135.0	.11	7
* 24 Mo. Briq. AVG. from Charts			11			39		120			

*For Indio location.

Ottawa sand mix weathering in planetary oven at 140°F for 400-800 hours, (4) weathering plate durability test (24 hours at 210°F), and (5) Actinic Light Weathering Test (95°F, 18 hours, 100 Mw/cm^2 of actinic radiation).

After completing the testing of the 2 year old briquettes, it was apparent that none of the procedures above adequately predicted the effect of asphalt weathering. Therefore, a new procedure, the California Tilt-Oven Asphalt Durability Test was developed. The results indicated that this test predicted the change in asphalt properties occurring during the 24 month test period at Indio West. This is particularly true for the penetration results. It appears that the California Tilt-Oven procedure can be used successfully to control asphalt hardening in areas where thermal hardening is the most significant weathering factor (i.e. high temperature areas).

Conclusions

The following conclusions were determined from this study:

1. High average air temperature (thermal oxidation) is the most significant factor affecting rate and amount of asphalt hardening in hot climates. Voids and aggregate porosity are also contributing factors but depend on the susceptibility of the asphalt to these factors.
2. The California test road indicates that briquette weathering per unit time is slightly more severe than actual road climate in the Indio climate. Twenty four months of briquette weathering is approximately equal to 32 months of road weathering.
3. The California Tilt-Oven Asphalt Durability Test can be used to predict asphalt hardening caused in 2 years at the Indio site and it can be used to control asphalt hardening in a hot climate.
4. In addition to an improved asphalt from a durability procedure, the authors believe that the following will improve asphalt durability: a) Reduce voids, b) Select asphalts suited to quality of aggregate, c) Avoid absorbent aggregates in hot areas, and d) use softest grade of asphalt consistent with curing and stability constraints.

4.15 WASHINGTON

Description

Four projects on Interstate 90 in eastern Washington where asphalt concrete was used as base, leveling and surface course were studied and reported by LeClerc and Walter (1976). The projects studied were such that sections of good and poor performing pavements of similar construction and traffic levels were located adjacent to each other.

Project numbers 8101 and 8200 both consisted of a surface and leveling course with a combined thickness of 0.35 feet of asphalt concrete (5/8 inch minus aggregate), 0.45 feet of asphalt concrete base (1-1/4 inch minus aggregate) and 0.25 feet of crushed aggregate over a river gravel embankment subgrade. The traffic counts for these projects showed an ADT of 10,700 with 15 percent trucks. Project numbers 8086 and 8138 both consisted of the same pavement sections as the other two projects but the subgrade was fragmented rock. The climate conditions for all four projects were rather severe with less than 11 inches of annual rainfall and a temperature range of -7°F to 101°F.

Results

After 5 to 6 years of service, one pavement developed extensive wheel path cracking, two pavement developed slight cracking, and one pavement showed no cracking.

Void contents and extraction and recovery tests were made on each of the three layers. Penetration at 77°F and ductility at 45°F and 1 cm/min, were determined on the recovered asphalt. The pavements were rated according to the Washington method (LeClerc & Marshall 1969) which combines ride and distress. Pavement ratings and recovered asphalt data are presented in Table 4.13. The pavement that was rated lowest, project 8086, had extensive longitudinal cracking in the wheelpaths and some transverse cracking. Two of the pavements, project 8101 and project 8200) had some wheelpath cracking. In all cases, the wheelpath cracking penetrated only the wearing and leveling course but not the base course, leading the authors to believe that failure was in the asphalt concrete, not the base. Project 8138, showed no distress. The asphalt in this project had relatively high retained penetration values and high relative ductility compared with the three sections exhibiting distress.

Conclusions

The authors concluded that penetration measurements alone could not identify asphalt properties related to pavement performance. They found, however, that a modified ductility test of 45°F, where a release agent was applied to the first inner surfaces of the mold end pieces, showed some promise in identifying asphalts which were related to in-service performance.

Table 4.13 Pavement ratings compared to properties of recovered asphalt for Washington after (LeClerc & Walter 1976)

PROJECT NUMBER	PROJECT	RATING	PENETRATION, 77°F (25°C)			DUCTILITY, 45°F (7°C) 1 CM/MIN		
			WEARING	LEVELING	BASE	WEARING	LEVELING	BASE
8086	Renslow to Ryegrass	40 (very poor)	18	22	24	1-	1.5	2.5
8101	Yakima Rd. to W. Ellensburg	52 (poor)	12	22	49	1-	5	51
8200	W. Ellensburg to Bull Rd.	59 (fair)	15	35	46	1-	11	72
8138	Ryegrass to Vantage	62 (good)	24	50	48	5.5	100+	100+

4.16 LOUISIANA

Description

This study was conducted in an attempt to provide comparisons between viscosity-graded and penetration-graded asphalt cements. In particular, the aging characteristics of these asphalt and associated relations to pavement durability were studied (Shah 1978). Towards the end of 1970, test sections were constructed on a 5-mile stretch of portland cement concrete pavement on LA-1 near Baton Rouge.

The test sections were constructed using different asphalt crudes (Hawk, Mexico, Arabian, Light Arkansas and Smackover). Each source provided a penetration-graded asphalt cement (60 to 70 pen) and a viscosity-graded asphalt cement. The viscosity-graded cements were controlled for consistency by absolute viscosity at 60°C (140°F). These grades were specifically prepared by the suppliers for this study. Figure 4.12 shows the layout of the various test sections.

Samples from the test sections were taken at 1 day, 36 days, 110 days, 1, 3 and 5 years after construction to evaluate aging characteristics. To control additional hardening of asphalt cements in the mix after sampling, the samples were stored in deep freeze until ready for extraction. The Asphalt Institute Laboratory in Maryland conducted the extraction, recovery and testing of the recovered asphalts. Pavement performance was also evaluated in terms of ride quality (Mays road meter), rutting, block and alligator cracking and ravelling after 60 months of service. Table 4.14 summarizes these results. A scale of 0 to 3 was used with 0 being poor performance and 3 indicating the absence of any defects defined in this study. Traffic volume has remained stable at an ADT of 3100.

Results

The change in the penetration properties appears to fit a hyperbolic function with time as shown in Figure 4.13. Those sections with relatively low asphalt viscosity, such as sections 9 and 10, show less pavement distress. However, an argument against softer asphalts is the early manifestation of rutting, and this is clearly shown in Table 4.14 where these two sections have slightly deeper ruts than the remaining sections. However, it should be kept in mind that the differences in rut depths are in the order of 1 to 3 mm and section 1 with a viscosity comparable to an average for all ten test sections had the highest amount of rutting.

Figure 4.13 indicates that for both types of asphalts, there is a rapid rate of hardening during the first 12 months and a decreasing rate thereafter. The rate of hardening with time is not

TEST CORES

SECTION NO.	1	2	3	4	5	6	7	8	9	10
ASPHALT SOURCE	A	A	B	B	C	C	D	D	A	B
ASPHALT CRUDE	HAWK	HAWK	MEX	MEX	LIGHT ARK	SMACK OVER	HAWK, ARAB	HAWK, ARAB	HAWK	MEX
ASPHALT GRADE	PEN	VISC	PEN	VISC	PEN	VISC	PEN	VISC	VISC	VISC

Figure 4.12 Layout and identification of pavement sections for Louisiana (Shah 1978)

Table 4.14 Pavement condition rating for Louisiana (Shah 1978)

	Section									
Criterion	1	2	3	4	5	6	7	8	9	10
Riding	1.31	1.53	1.63	1.67	1.80	1.78	2.01	1.93	1.89	2.06
Raveling	2.08	1.52	1.79	1.81	2.21	1.62	2.03	2.21	2.28	2.51
Loss of matrix	1.95	1.41	1.70	1.76	2.09	1.64	2.12	2.34	2.37	2.47
Cracking										
Block and alligator	1.52	2.36	2.30	2.03	2.27	2.16	2.10	2.06	2.48	2.52
Transverse and longitudinal	1.51	1.98	1.93	1.67	1.91	1.95	1.95	2.04	2.18	2.45
Overall subjective rating	1.67	1.76	1.87	1.79	2.06	1.83	2.04	2.12	2.24	2.40
Rutting, mm	7.1	4.4	4.1	3.4	4.6	3.8	4.6	4.6	5.3	6.6
Mays roughness, in/mile	144	131	123	123	111	108	96	85	125	85
Dynaflect deflection, $^1/_{1000}$ in	1.64	1.40	1.35	1.27	1.33	0.95	1.03	0.89	0.91	1.32
Block and alligator cracking, $ft^2/1000\ ft^2$	42.76	9.22	0.36	3.33	20.26	13.91	3.70	6.98	0.68	0.00

Note: 1 mm = 0.039 in.

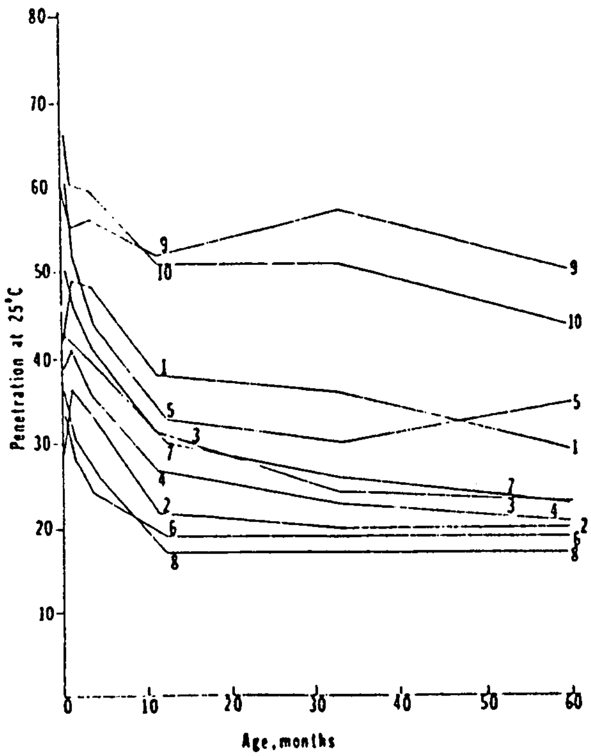


Figure 4.13 Penetration at 25°C (77°f) vs time of aging for Louisiana (Shah 1978)

consistent although the rate for viscosity-graded asphalts is slower than for penetration-graded asphalts. Figure 4.14 shows a plot of the aging index and section 3 had a 13-fold increase in viscosity after 60 months. This section also had the highest original and thin-film residue viscosity. However, this section did not have the highest amount of cracking, in fact it had less cracking than eight of the ten sections. The slopes of these curves can be used as indicators of the relative durability of these asphalts, with the flatter slopes implying a more durable asphalt. Accordingly,based on 60-month date, all viscosity-graded asphalts would be classified as more durable than corresponding penetration-graded asphalts. Sections 9 and 10, which have the flattest slope, are therefore the most durable asphalts, and section 3 the least durable but not the poorest performing.

The ductility values typically provide some measure of asphalt. However, the data collected indicate inconsistencies in the rate of change in this property. A close association between ductility values and shear index was observed: in several, asphalts with shear indices less than 0.40 had ductilities greater than 100 cm (39 inches). It is generally recognized that the degree of initial and final compaction or void content or both in the pavement has an effect on the rate of hardening of asphalts. More specifically, the higher the initial void content is, the greater the rate of hardening. However, the data were too scattered to indicate any association of hardening rate with air void content in pavement. This disassociation should not be construed to mean that the magnitude of air void content does not affect the hardening rate of asphalt binder. What it does indicate is the fact that the air void variability in pavement is so pronounced that it overshadows the resulting effect on the hardening process.

Conclusions

The primary intent of the study reported here was to make a comparative evaluation of the durability and performance of penetration- and viscosity-graded asphalt cements by means of field evaluation of asphaltic concrete mixtures. The principal findings summarized below are applicable within the constraints of the environment, materials, construction, and traffic that existed at the test site:

1. Hardening of asphalt cements, regardless of how they are graded, is a hyperbolic function of time although the rate or curvature may vary with specific asphalts.
2. For a given asphalt source, the difference in durability between the two types of asphalts was not significant. Likewise, no significant difference was evident in their performance in the field.

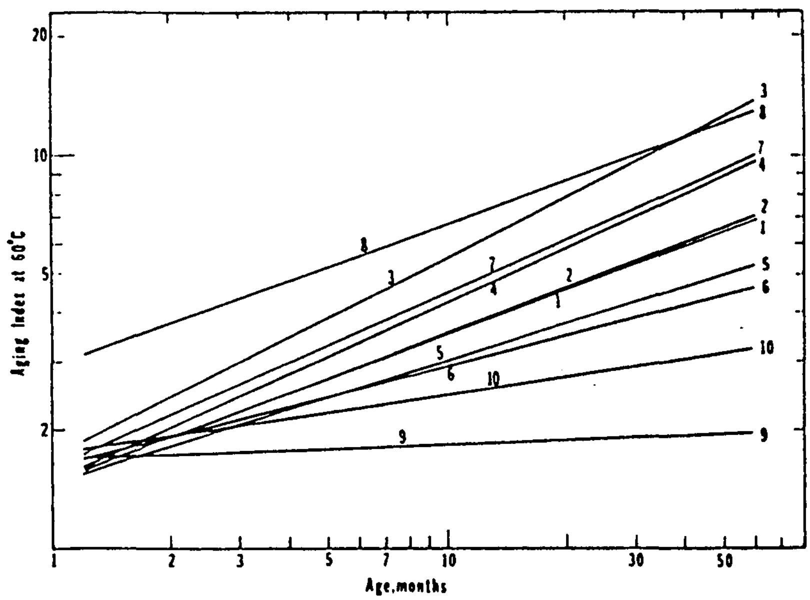


Figure 4.14 Viscosity index at 60°C (140°F) vs time of aging for Louisiana (Shah 1978)

3. Viscosity-graded asphalts were more susceptible to temperature than corresponding penetration-graded asphalts, but there was no correlation between this characteristic and pavement distress.
4. There was no association between voids in the pavement and rate of hardening.

4.17 QUEBEC

Description

In 1980, data were collected from test road sections in Quebec, Canada to evaluate the effects of materials and in-situ conditions on pavement performance (Keyser & Ruth 1984). A total of 23 test sections were used and the majority had been in service for 7 to 9 years under medium traffic conditions. Freezing index values ranged between 1500 to 3000 degree-days. Typical cross-sections contained 3 to 5.5 inches of asphalt concrete, 6-12 inches of base, 8-12 inches of subbase and 12-24 inches of clean sand as a frost protection layer.

In-situ measurements of rutting, cracking, deflection and ride quality were taken, together with core samples to determine the properties of the recovered asphalts as well as mix properties. Penetration and viscosity data for the original bitumen were also included. Environmental conditions were noted. A series of statistical methods were then used to determine which variable had a high degree of interaction and to what degree of significance the variables were related to a specific material, performance or response parameter.

Results

Regression relationships were developed between the 3 consistency measurements (viscosity, penetration and softening point). In general, there were fair to poor correlations with single variable correlations. Multiple variable correlations provided reasonably good correlations as shown in Table 4.15.

Other relationships were developed between asphalt consistency and mix parameters. In particular, it was found that air voids affected consistency values, and that both parameters influenced the tensile strength of the core.

Analysis of the transverse cracking data resulted in some representative equations (Table 4.15). Equations 20 to 22 have extremely low correlation coefficients, but suggest that penetration, traffic, and air voids affect the degree of pavement cracking. Figure 4.15 shows that the data should be segregated according to traffic levels. Figure 4.16 illustrates the effects of penetration, asphalt

Table 4.15 Relationships for transverse cracking for Quebec (Keyser & Ruth 1984)

Eqn. No.	Regression Equations for Different Parameters[a]	Std. Error	R^2
	Single Variable Correlations:		
20	$TC = 25.326 + 0.396\left(\frac{120}{PEN}\right)^{3.3}$	37.8	0.311
21	$TC = -131.251 + 247.84(TRA)^{-0.3}$	33.8	0.449
22	$TC = 4.8471 + 10.035\ (VV)$	40.4	0.213
23	$TC = 5.15 + 0.235\left(\frac{120}{PEN}\right)^{3.3}$	----	0.933
	(Light to medium traffic, Levels 6 to 3)		
24	$TC = 21.58\left(\frac{120}{PEN}\right)^{1.206}$	----	0.855
	(Medium to heavy traffic, Levels 3 to 1)		
	Multiple Variable Correlations		
25	$TC = -129.37 + 0.323\left(\frac{120}{PEN}\right)^{3.3} + 218.57(TRA)^{-0.3}$	27.6	0.650
26	$TC = -239.91 + 0.321\left(\frac{120}{PEN}\right)^{3.3} + 407.54(TRA)^{-0.3}$		
	$- \frac{17.05\ THICK}{(TRA)^{-0.3}}$	27.2	0.679
27	$TC = -97.619 + 0.0682(VIS) + 263.96(TRA)^{-0.3} - 0.0228(FI)$		
	$- 0.0426\ (THICK)(ITS)$	27.9	0.682

Note: Presented in this table are mathematical models developed by statistical regression analyses to define the relationships between different parameters and transverse cracking.

[a] TC = total simple, multiple, and transverse cracks; PEN = penetration @ 25°C; TRA = traffic (1 = heavy, 6 = light); THICK = thickness of asphalt concrete pavement (in.); FI = freezing index (° days); ITS = indirect tensile strength at 0°C.

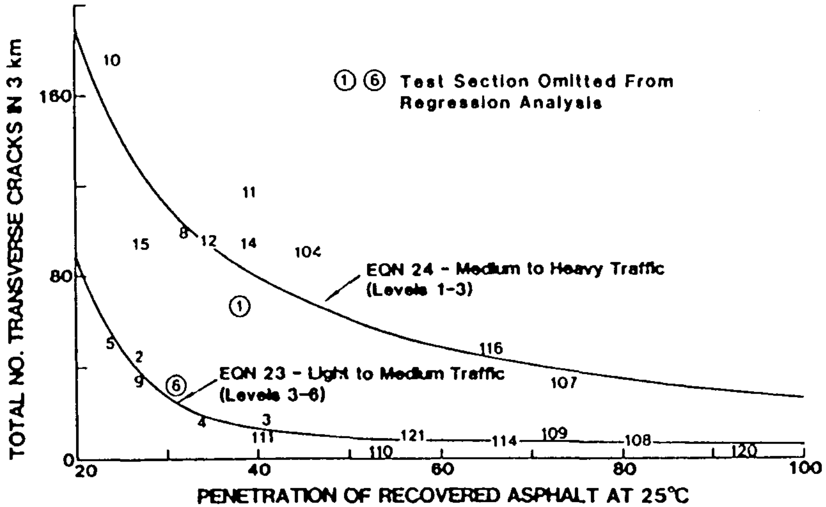


Figure 4.15 Effect of binder penetration and traffic on transverse cracking for Quebec (Keyser & Ruth 1984)

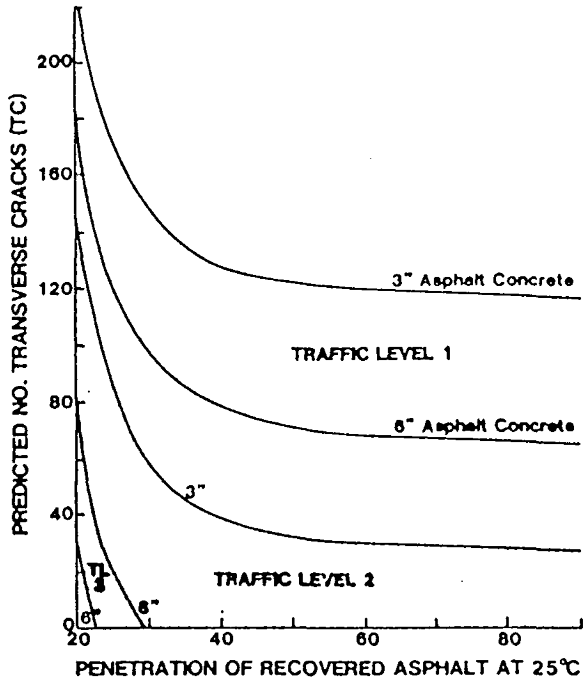


Figure 4.16 Effect of variables on transverse cracking for Quebec (Keyser & Ruth 1984)

concrete thickness and traffic on transverse cracking. From this figure, it can be seen that 3 inches of asphalt concrete with a pen of 50-60 is essentially equivalent to 6 inches of asphalt concrete with a 25 pen binder. The addition of other variables such as viscosity, traffic, freezing index, thickness, and indirect tensile strength parameter did not indicate any improvement over those equations containing only penetration. Equation 27 defines the best multiple variable correlation obtained using the viscosity. This equation indicates that cracking increases with viscosity and traffic, decreases with an increase in freezing index and decreases with increasing thickness and tensile strength.

Two models were selected for evaluation and use in the prediction of transverse cracking and cracking temperatures. Both the Haas model and the Asphalt Institute procedure showed no correlation between the number of transverse cracking and the predicted differential cracking temperature. The poor results may be due to insufficient range in the data or other reasons. Ride quality was also measured and regression analyses developed with freezing index, transverse cracks and other subsurface variables as independent variables. Attempts to develop rational relationships between rut depth and the available data were unsuccessful. This could be partly due to the insufficient range in rut depth values, as only 5 of the 23 test sections exhibited rutting in excess of 0.5 inches.

Conclusions

The results of analyses performed on data collected from 23 test road sections in the Province of Quebec, Canada, provided information on the effects that certain parameters have on the properties and performance of asphalt concrete.

1. The most significant finding was that consistency of recovered asphalts and traffic level defined to a high degree of significance the amount of transverse cracking on 6 to 9 year-old pavements.
2. The use of two categories of traffic level and the penetration (25°C) of recovered asphalt provided a relationship with the number of transverse cracks.
3. Relationships developed between penetration, viscosity, and ring and ball softening point of recovered asphalts demonstrate that these consistency measurements are related to such a high degree of significance that only one of these consistency parameters is needed to characterize the asphalt. However, viscosity-penetration relationships for original asphalts are not uniquely related and form the basis for selection of asphalts to minimize low temperature cracking potential.

4. Several existing models were used to predict transverse cracking and cracking temperature. Analysis of these values and the actual number of observed transverse cracks provided absolutely no correlation between predicted and actual cracking.
5. The depth of rutting was insignificant, except for five sections that had ruts in excess of 0.5 inches. No relationships were found to define the amount of rutting.
6. The effects of air void content on asphalt hardening and indirect tensile strength at 0°C (32°F) were identified. A tentative procedure for prediction of in-service hardening of bitumens has been suggested for use in determining the effects of air void content and age on the indirect tensile strength and consistency (penetration or viscosity) of the asphalt binder.

4.18 FLORIDA

Description

Since 1983, asphalts with abnormally high light fractions have been encountered in Florida (Page et al., 1986). In particular, operational problems were encountered in both drum mix and batch plants because of the loss of the light fractions during the mixing process. The effect of these losses were a visually dry appearance of the mix after paving, and subsequent compaction and ravelling problems in some projects.

The concern over the eventual availability of asphalts with a high volatile fraction on state paving projects led to the initiation of a testing program by the Florida DOT. The primary objectives were to determine if asphalts with high percentages of light fractions could be related to any change in mix properties and pavement performance when compared to a paving mixture with a more typical light fraction asphalt cement. In addition, the differences in batch vs. drum mix plants were also evaluated.

A recycling project on SR 16 in Clay County was selected--the pavement was originally constructed in 1942 and consisted of a natural subgrade, 6 inches of limerock base, and a 3/4 inch surface treatment. In 1958, the pavement was overlaid with 2.5 inches of an asphalt concrete surface course. The test pavement project involved the milling of 2 inches of the existing surface course and placement of a 1.25 inch layer of a recycled Type S-III concrete structural mix followed by 1.25 inches of a conventional Type S-III surface course mixture in the traffic lane of the test section area. A 1.25 inch surface course layer of the recycled Type S-III was placed in the passing lane and at all other locations outside the test section area.

The most significant differences between the two types of mixtures are air void content and asphalt content (higher in conventional mix). Two AC-20 grade asphalts were used, one with a high light fraction and one with a typical light fraction. A total of eight test sections were constructed (2 asphalts and 2 plants, with and without steam distillation).

The asphalt cements to be used in each of the test sections were sampled upon delivery to the asphalt plant and submitted to the Central Laboratory (FDOT-Bureau of Materials and Research) for testing. The following tests were performed to determine the physical properties of the asphalts before and after Thin Film Oven Test (TFOT):

1. Viscosity @ 77•F, 140•F, and 275•F (25, 60, 135•C).
2. Penetration @ 77•F (25•C).
3. Ductility @ 60•F (15.6•C) and 77•F (25•C).
4. TFOT-percent loss.
5. Flash Point (COC).
6. Smoke Point (COC).
7. Solubility--Trichloroethane.

These results for the asphalt cement are summarized in Tables 4.16 to 4.19. Tables 4.20 to 4.21 summarize the asphalt mix properties. In addition, the asphalt cements were pre-processed using steam distillation to remove the light fractions from the asphalt cements. However, after 6 days of testing, this process was terminated as there was no significant change in TFOT loss. The steam distillation results are not discussed any further herein.

Monitoring of the completed test sections was performed at three, six and nine months after construction. Cores were taken, rut depths measured, crack surveys taken, as well as Dynaflect, Mays Meter and friction number measurements. Table 4-22 summarizes these measurements. The performance data show no indication of early failure or inferior behavior of any of the test sections.

Results

Nine-month aging trends for viscosity @ 140•F are shown in Figure 4.17. Sections 1 and 2 show the same rate of hardening when compared to Sections 3 and 4, but the high light fraction asphalts in Sections 3 and 4 have a lower viscosity than 1 and 2. The close correspondence in the slopes suggest that viscosity tests on TFOT residues obtained at different heating times could be used to establish the in-service binder hardening rate. The dashed lines indicate that the viscosity

Table 4.16 Asphalt Cement Test Data--Low Light Fractions, Batch Plants (T.S. 1) (Page et al. 1986)

Description: Asphalt Cement - Typical Light Fractions
Plant: Drum Mix

TEST SECTION No. 2

Test	Original	After TFOT	2 wk	3 mo	6 mo	Recovered from Roadway 9 mo	1 yr.	2 yr.	3 yr.
Viscosity 275F	462	774	695 (1240)	819 (1365)	928 (1561)	1153 (2396)			
Viscosity 140F	2273	7724	4711 (18090)	7322 (21956)	9585 (31978)	15755 (77085)			
Viscosity 77F	1.28E6	2.52E6	1.49E6 (3.74E6)	2.29E6 (4.86E6)	3.08E6 (8.08E6)	6.43E6 1.38E7			
Penetration 77F	82	51	63 (40)	49 (36)	47 (34)	37 (25)			
Ductility 77F	150+	125	150+ (32)	150+ (50)	150+ (16)	94 (10)			
Ductility 60F	145	15	40 (8)	14 (6)	16 (8)	7 (5)			
Loss (%)	-	0.28	0.94	0.62	0.87	1.27			
Flash Point COC F	525	-	550	555	540	550			
Smoke Point COC F	240	-	340	330	330	350			
Solubility-Trichloroethene	99.98	-	99.86	99.97	99.94	99.96			

NOTE: Values shown in parentheses are after Thin Film Oven Test

Table 4.17 Asphalt Cement Test Data--Low Light Fractions, Drum Mix Plants (T.S. 2) (Page et al. 1986)

Description: Asphalt Cement - Typical Light Fractions
Plant: Batch

TEST SECTION No. 1

Test	Original	After TFOT	2 wk	3 mo	6 mo	Recovered from Roadway 9 mo	1 yr.	2 yr.	3 yr.
Viscosity 275F	462	774	861 (1440)	968 (1537)	909 (1679)	1275 (2174)			
Viscosity 140F	2273	7724	7613 (23,715)	11,040 (29,339)	10,343 (37,509)	19,735 (72,701)			
Viscosity 77F	1.28E6	2.52E6	2.16E6 4.06E6	3.50E6 (6.11E6)	2.96E6 (7.10E6)	8.20E6 1.37E7			
Penetration 77F	82	51	54 (36)	44 (34)	46 (32)	34 (26)			
Ductility 77F	150+	125	115 (18)	128 (21)	150+ (18)	66 (9)			
Ductility 60F	145	15	10 (7)	9 (6)	10 (5)	5 (5)			
Loss (%)	-	0.28	0.86	0.70	1.01	0.94			
Flash Point COC F	525	-	535	545	530	550			
Smoke Point COC F	240	-	335	325	345	350			
Solubility-Trichloroethene	99.98	-	99.98	100.0	99.93	99.94			

NOTE: Values shown in parentheses are after the Film Oven Test

Table 4.18 Asphalt Cement Test Data--High Light Fractions, Batch Plants (T.S. 3) (Page et al. 1986)

Description: Asphalt Cement - High Light Fractions
Plant: Batch

TEST SECTION No. 3

Test	Original	After TFOT	2 wk	3 mo	6 mo	Recovered from Roadway 9 mo	1 yr.	2 yr.	3 yr.
Viscosity 275F	483	802	683 (1113)	734 (1216)	913 (1559)	1116 (2693)			
Viscosity 140F	2101	5630	4000 (11727)	4703 (13652)	7692 (23528)	10216 (58126)			
Viscosity 77F	0.98E6	2.47E6	1.38E6 (3.52E6)	1.63E6 (4.78E6)	2.64E6 (6.57E6)	4.31E6 1.44E7			
Penetration 77F	95	58	70 (48)	65 (42)	54 (37)	52 (28)			
Ductility 77F	150+	150+	150+ (110)	150+ (90)	150+ (24)	86 (19)			
Ductility 60F	150+	62	86 (14)	74 (10)	27 (8)	19 (10)			
Loss (%)	-	0.65	0.99	1.05	1.15	1.96			
Flash Point COC F	485	-	520	535	535	545			
Smoke Point COC F	280	-	330	315	350	310			
Solubility-Trichloroethene	99.98	-	99.10	99.98	99.97	99.94			

NOTE: Values shown in parenthesis are after Thin Film Oven Test

Table 4.19 Asphalt Cement Test Data--High Light Fractions, Drum Mix Plants (T.S. 4) (Page et al. 1986)

Description: Asphalt Cement - High Light Fractions
Plant: Drum Mix

TEST SECTION No. 4

Test	Original	After TFOT	2 wk	3 mo	6 mo	Recovered from Roadway 9 mo	1 yr.	2 yr.	3 yr.
Viscosity 275F	483	802	780 (1278)	819 (1334)	1116 (1781)	1190 (1957)			
Viscosity 140F	2101	5630	5305 (16725)	5802 (17032)	10971 (29945)	13551 (42484)			
Viscosity 77F	0.98E6	2.47E6	1.67E6 (4.48E6)	2.07E6 (5.56E6)	- -	1.11E7 1.14E7			
Penetration 77F	95	58	64 (40)	59 (38)	45 (37)	38 (28)			
Ductility 77F	150+	150+	150+ (86)	150+ (71)	150+ (16)	92 (29)			
Ductility 60F	150+	62	74 (9)	57 (8)	102 (7)	8 (5)			
Loss (%)	-	0.65	1.01	1.00	0.85	0.85			
Flash Point COC F	485	-	540	540	550	545			
Smoke Point COC F	280	-	320	300	340	315			
Solubility-Trichloroethene	99.98	-	99.89	99.97	99.97	99.97			

NOTE: Values shown in parentheses are after Thin Film Oven Test

Table 4.20 Asphalt Concrete Mixture Data; T.S. 1 and T.S. 2 (Page et al. 1986)

Description: Asphalt Cement - Typical Light Fractions — TEST SECTION NO. 1
Plant: Batch

Test	Mix Design	Plant Mix	As Const.[a]	Roadway Samples[b] 3 mo.	6 mo.	9 mo.
Density (pcf)	131.0	132.7	126.9	121.6/125.5	125.6/124.8	131.7/138.9
Air Voids (%)	6.7	5.5	9.6	15.4/14.8	12.6/13.1	8.3/10.3
Marshall Stability (lbs)	1580	1783	-	-	-	-
Marshall Flow (0.01 inch)	9	9	-	-	-	-
Tensile Strength $(psi)_{wp}$	-	124	42	56	75	133.0

Description: Asphalt Cement - Typical Light Fractions — TEST SECTION NO. 2
Plant: Drum Mix

Test	Mix Design	Plant Mix	As Const.[a]	Roadway Samples[b] 3 mo.	6 mo.	9 mo.
Density (pcf)	131.0	131.7	125.6	125.5/124.9	127.1/127.4	129.2/126.7
Air Voids (%)	6.7	6.2	10.5	12.2/12.5	10.9/10.7	9.4/11.3
Marshall Stability (lbs)	1580	1766	-	-	-	-
Marshall Flow (0.01 inch)	9	9	-	-	-	-
Tensile Strength $(psi)_{wp}$	-	125	62	65	88	128.0

[a] As Const - Average of 10 nuclear density measurements (corrected)
[b] Between Wheel Paths (BWP)/In Wheel Path (WP)

Table 4.21 Asphalt Concrete Mixture Data; T.S. 3 and T.S. 4 (Page et al. 1986)

Description: Asphalt Cement - High Light Fractions — TEST SECTION NO. 3
Plant: Batch

Test	Mix Design	Plant Mix	As Const.[a]	Roadway Samples[b] 3 mo.	6 mo.	9 mo.
Density (pcf)	131.0	133.5	128.6	127.9/128.4	128.1/127.5	130.4/125.4
Air Voids (%)	6.7	5.0	8.4	11.4/10.3	10.6/11.1	8.9/12.4
Marshall Stability (lbs)	1580	1378	-	-	-	-
Marshall Flow (0.01 inch)	9	9	-	-	-	-
Tensile Strength $(psi)_{wp}$	-	113	51	55	60	122

Description: Asphalt Cement - High Light Fractions — TEST SECTION NO. 4
Plant: Drum Mix

Test	Mix Design	Plant Mix	As Const.[a]	Roadway Samples[b] 3 mo.	6 mo.	9 mo.
Density (pcf)	131.0	128.5	124.7	124.7/125.6	126.1/126.6	127.3/122.9
Air Voids (%)	6.7	8.5	11.2	13.5/13.0	12.5/12.2	12.1/14.8
Marshall Stability (lbs)	1580	1713	-	-	-	-
Marshall Flow (0.01 inch)	9	9	-	-	-	-
Tensile Strength $(psi)_{wp}$	-	111	56	65	89	106

[a] As Const. - Average of 10 nuclear density measurements (corrected)
[b] Between Wheel Paths (BWP)/In Wheel Path (WP)

Table 4.22 Pavement Performance Data; T.S. 1-4 (Page et al. 1986)

TEST SECTION NO 1

Description: Asphalt Cement - Typical Light Fractions
Plant - Batch

Test	As Constructed	3 months	6 months	9 months
Rut Depth, inch	.00	.04	.00	.00
cracking, sq. ft./1,000 sq. ft.	0	0	0	0
Dynamic Deflection (No. 1 Geophone)	0.80	0.71	0.79	0.70
Present Serviceability Index PSI sv, Mays Ride Meter	4.22	4.37	4.25	4.38
Friction Number @ 40 mph	46.50	60.30	50.60	57.50

TEST SECTION NO 2

Description: Asphalt Cement - Typical Light Fractions
Plant - Drum Mix

Test	As Constructed	3 months	6 months	9 months
Rut Depth, inch	.00	.02	.01	.00
cracking, sq. ft./1,000 sq. ft.	0	0	0	0
Dynamic Deflection, (No. 1 Geophone)	0.86	0.74	0.72	0.76
Present Serviceability Index PSI sv, Mays Ride Meter	4.30	4.51	4.36	4.49
Friction Number @ 40 mph	45.90	59.90	50.60	60.5

TEST SECTION NO 3

Description: Asphalt Cement - High Light Fractions
Plant - Batch

Test	As Constructed	3 months	6 months	9 months
Rut Depth, inch	.00	.04	.03	.06
cracking, sq. ft./1,000 sq. ft.	0	0	0	0
Dynamic Deflection (No. 1 Geophone)	1.06	0.91	1.19	1.02
Present Serviceability Index PSI sv, Mays Ride Meter	4.12	4.22	4.16	4.42
Friction Number @ 40 mph	41.50	57.20	61.50	57.30

TEST SECTION NO 4

Description: Asphalt Cement - High Light Fractions
Plant - Drum Mix

Test	As Constructed	3 months	6 months	9 months
Rut Depth, inch	.00	.03	.04	.05
cracking, sq. ft./1,000 sq. ft.	0	0	0	0
Dynamic Deflection, (No. 1 Geophone)	0.85	0.79	0.86	0.83
Present Serviceability Index PSI sv, Mays Ride Meter	4.15	4.26	4.22	4.40
Friction Number @ 40 mph	47.10	65.40	63.90	63.40

Table 4.23 Fatigue Test Data (Page et al. 1986)

Identification Section and Sample No.		Extreme Fiber Stress psi	Extreme Fiber Strain @ 200 Cycles	Fracture Life Nf
Sec. 1 368	MC-1	100.0	6.401×10^{-4}	2,465
Typical Lights	3	90.7	5.928×10^{-4}	4,789
Undistilled	2	82.0	4.138×10^{-4}	16,376
Batch Plant	5	75.0	3.296×10^{-4}	18,278
	4	79.0	3.780×10^{-4}	25,129
Sec. 2 368	MC-6	82.0	6.065×10^{-4}	4,689
Typical Lights	9	79.8	3.952×10^{-4}	17,583
Undistilled	7	77.2	3.470×10^{-4}	22,192
Drum Mixer	8	52.0	4.051×10^{-4}	52,610
Sec. 3 368	MC-11	86.5	8.882×10^{-4}	2,619
High % Lights	14	80.0	5.223×10^{-4}	13,908
Undistilled	13	70.0	3.904×10^{-4}	39,520
Batch Plant	12	60.0	3.828×10^{-4}	60,016
Sec. 4 368	MC-16	79.6	7.108×10^{-4}	1,859
High % Lights	17	70.7	4.196×10^{-4}	5,170
Undistilled	18	60.3	3.297×10^{-4}	15,473
Drum Mixer	19	57.1	3.939×10^{-4}	22,814
	19A	52.3	2.398×10^{-4}	52,060
Sec. 5 368	MC-24	118.6	5.889×10^{-4}	7,907
Typical Lights	23	100.7	5.379×10^{-4}	11,384
Distilled	20	78.0	5.104×10^{-4}	11,992
Batch Plant	21	73.5	4.151×10^{-4}	20,392
	22	71.6	3.644×10^{-4}	58,016
Sec. 6 368	MC			
Typical Lights	26	67.4	3.978×10^{-4}	6,222
Distilled	27	65.7	3.448×10^{-4}	6,618
Drum Mixer	29	70.6	2.544×10^{-4}	16,485
	28	56.2	2.414×10^{-4}	24,755
Sec. 7 368	MC-33	143.2	5.019×10^{-4}	5,812
High % Lights	30	80.6	4.453×10^{-4}	17,817
Distilled	32	98.3	4.138×10^{-4}	30,505
Batch Plant	31	75.1	3.866×10^{-4}	59,818
Sec. 8 368	MC-38	144.2	5.700×10^{-4}	6,766
High % Lights	37	121.3	5.087×10^{-4}	14,328
Distilled	36	84.5	4.396×10^{-4}	36,035
Drum Mixer	35	78.5	4.773×10^{-4}	59,233

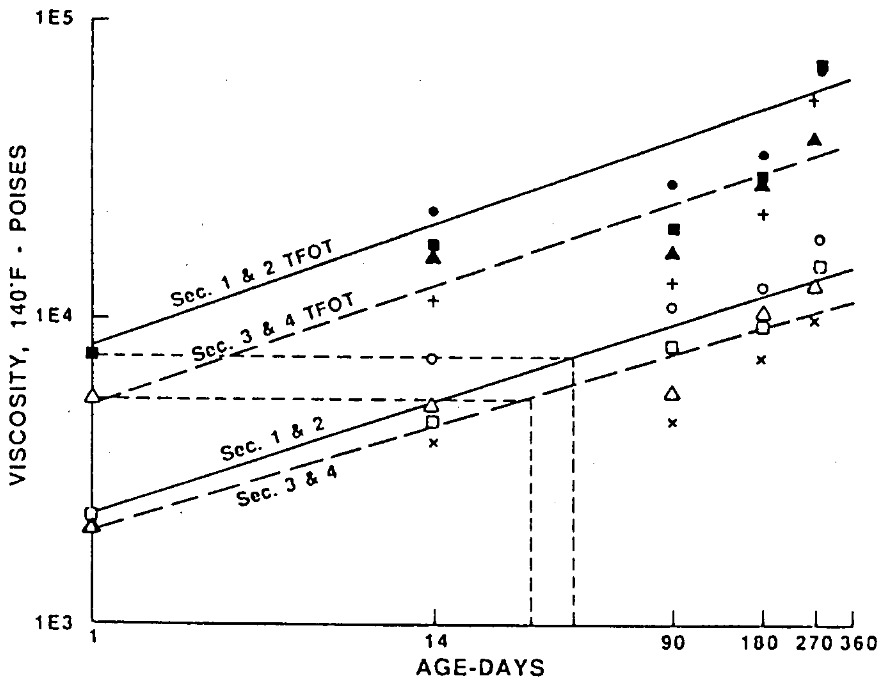


Figure 4.17 Age Hardening of Recovered Asphalts--140•F Visco (Page et al. 1986)

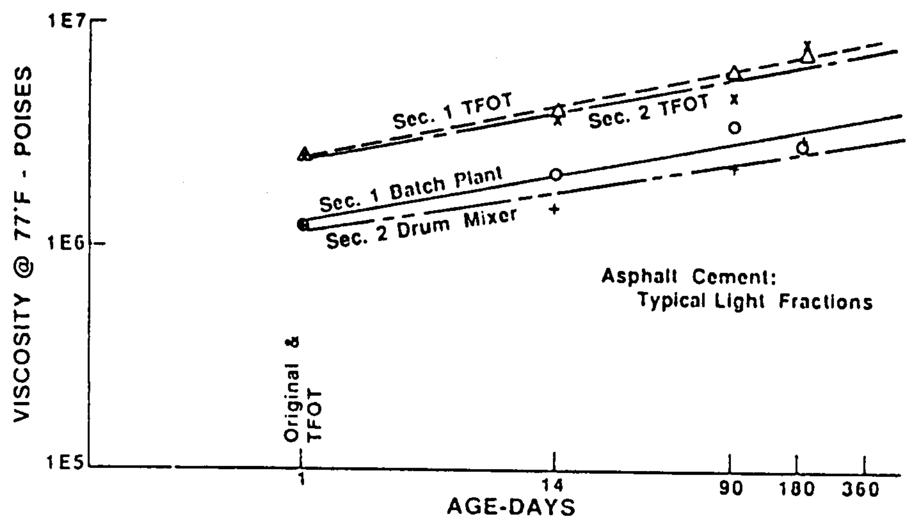


Figure 4.18 Age Hardening of Typical Light Fractions Asphalt--77•F Viscosity (Page et al. 1986)

of the TFOT residue from the original asphalt with high light functions is equivalent in viscosity to the in-service binder at 30 days.

The viscosity @ 77•F results in Figures 4.18, 4.19 and 4.20 show that viscosity increases with age. They also illustrate that viscosity data provides greater sensitivity than penetration in the analysis of data trends.

Finally, flexural fatigue tests were conducted @ 77•F and analyzed. It was found that the asphalts with high light fractions (Sections 3 & 4) gave fatigue lives similar to those for Section 1. Table 4.23 summarizes these results. The authors (Page et al., 1986) conclude that the results for Section 2 are erroneous and therefore do not include them in the analysis.

Conclusions

The authors (Page et al., 1986) report the following conclusions:

1. Asphalts with high light fractions produce operational problems at the plants.

2. The asphalt with higher light fractions has a higher viscosity than the asphalt with typical light fractions.

3. Fatigue testing indicated no major differences between the mixtures used in the test sections.

4. The viscosity tests on TFOT residues indicate the same trend as the asphalts recovered from the field, i.e., viscosity increases with time.

5. Penetration does not show as great a sensitivity to hardening as does viscosity.

4.19 CLARK COUNTY, NEVADA

Description

In 1983, Finn et al. (1983) reported the results of an investigation into mix design procedures using the Marshall test and the Asphalt Institute procedures. Two examples were illustrated where mixes designed with the above procedures resulted in early rutting distress or in a mix that would have been considered unsuitable for a highway pavement using the California method of mix design. In addition, stabilometer and creep tests were used to modify the mixture designs. The authors also

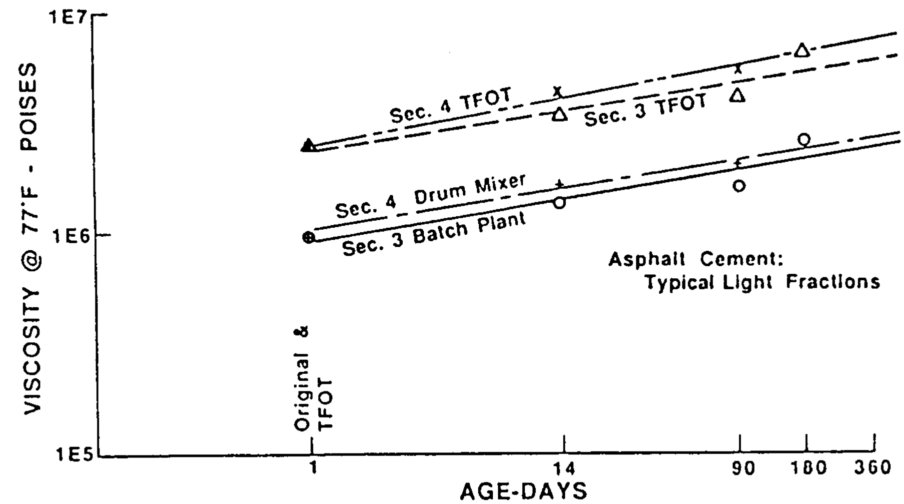


Figure 4.19 Age Hardening of Recovered High Light Fractions Asphalt--77•F Viscosity (Page et al. 1986)

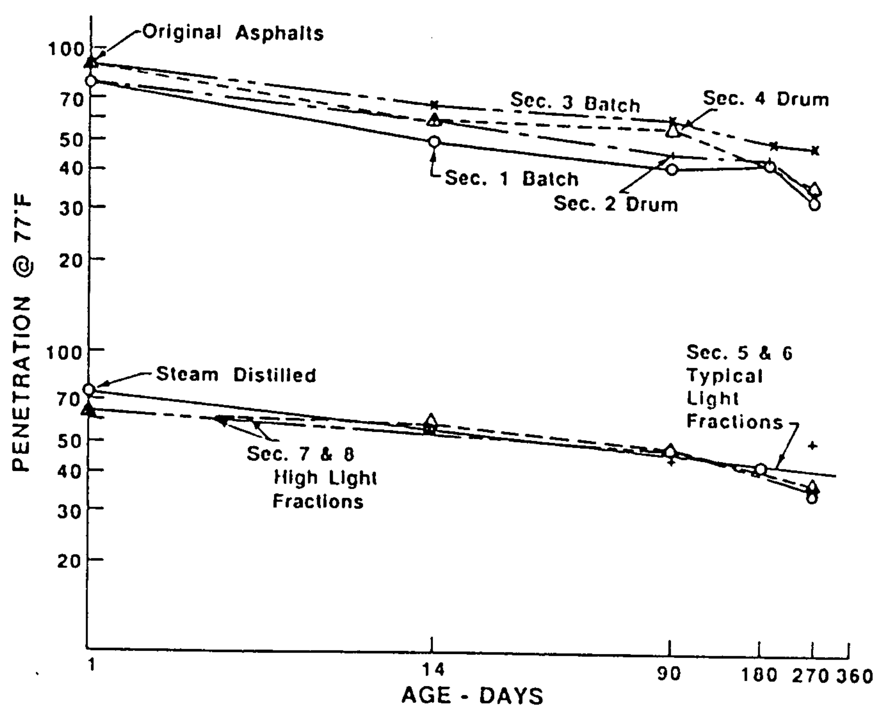


Figure 4.20 Penetration Changes of Recovered Asphalt (Page et al. 1986)

assessed the efficacy of the creep test as an additional design test, particularly for pavements subjected to heavy loading conditions in hot environments.

Two case studies are presented in which the Marshall test procedure for both airfield and highway pavements has required additional tests to assist in the mix design process. In both cases, the stabilometer and creep tests were utilized. The creep test was used as an indication of the relative amounts of rutting under repetitive traffic loading which occurs in mixes at different asphalt contents.

In 1979, an investigation was initiated in Clark County, Nevada to define the cause or causes of premature cracking (mostly block cracking) in recently constructed asphalt pavements. Because of the presence of volcanic (absorptive) aggregates, asphalt contents were subsequently increased. A field investigation in the spring of 1982 reported that some bleeding and rutting were found in mixes subjected to slow-moving, heavy traffic. The major incidence of these distresses were located in the approach lanes of major, signalized intersections.

Cores were taken from four projects in these areas, and they were rated as acceptable, unacceptable, and non-trafficked. Tests on the cores included determinations of unit weight, asphalt content, percent passing No. 200 sieve, Marshall stability and flow (remolded), stabilometer "S" value (both on cores and same materials remolded) and creep stiffness.

Results

Table 4.24 summarizes the recovered asphalt properties. The asphalt in the mix for Project 1, Phase 1, has a relatively high viscosity when compared with the other projects. The average recovered viscosity @ 140°F (60°C) for the acceptable areas is significantly different from that for the unacceptable areas. The average PVN was -1.1 for all the projects, with a range of -0.3 to -1.8. Thus, temperature susceptibility may be considered high but not beyond the normal range reported in the literature.

Air voids (Table 4.25) indicate that the mixes have been compacted to less than 3% (overall average 2% for unacceptable areas, 4% for acceptable areas). The low values correspond with the observed performance, i.e., bleeding. Table 4.26 summarizes the stability data, and it may be concluded that:

1. High Marshall stability values do not assure adequate rutting resistance.
2. Stabilometer "S" values decrease significantly as air voids are reduced below 4%.

Table 4.24 Recovered Asphalt Properties (Finn et al. 1983)

Project	Performance	Asphalt Characteristics			
		Recovered Properties			
		Original Grade	Viscosity at 140 F - poises	Pen. at 77 F 100 gm. 5 sec.	Pen-Vis Number
1	Acceptable	AR-8000	25,100	16	-0.6
	Unacceptable	AR-4000	3,055	45	-1.8
2	Acceptable	AR-4000	3,570	44	-1.5
	Unacceptable	AR-4000	1,870	66	-1.8
3	Acceptable	AR-4000	15,400	19	-0.8
	Unacceptable	AR-4000	1,930	67	-1.6
4	Acceptable	AR-4000	12,000	19	-0.8
	Unacceptable	AR-4000	2,830	52	-1.5

Table 4.25 Void Content Data--In Situ Mixes (Finn et al. 1983)

Project	Performance	VMA - percent			Air Void Content of Cores - Percent*	
		Specifications	Job Mix Formula	Cores	Location 1	Location 2
1	Acceptable	14	15	9.8	4.2	4.8
	Unacceptable	12.5/13.0	12.4	9.7	1.2	1.9
	Nontrafficked			11.1	5.3	4.5
2	Acceptable	12.5/13.0	13.2	8.9	3.7	-
	Unacceptable	12.5/13.0	13.2	8.7	2.6	1.5
	Nontrafficked	12.5/13.0	13.2	7.3	-	-
3	Acceptable	12.5/13.0	14.5	11.8	3.4	4.1
	Unacceptable	12.5/13.0	14.5	10.9	2.6	2.5
	Nontrafficked	12.5/13.0	14.5	11.1	2.5	
4	Acceptable	12.5/13.0		14.6	5.6	2.6
	Unacceptjable	12.5/13.0	-	12.7	1.5	2.3
	Nontrafficked	12.5/13.0		13.8	2.8	

*Average values for cores taken from respective projects.

Table 4.26 Stability Data--In Situ Mixes (Finn et al. 1983)

Project	Performance	Marshall Test Data		Stabilometer "S" Value	
		Stability lb.	Flow (0.01 in.)	Core	Remolded Specimen
1	Acceptable	5,480	10	31	20
	Unacceptable	4,570	12.5	7	10
	Nontrafficked (I)				29
	Nontrafficked (II)				18
2	Acceptable	4,590	16	29	12
	Unacceptable	4,100	17	1	5
	Nontrafficked			17	8
3	Acceptable	4,540	10	45	22
	Unacceptable	3,595	15	6	2
	Nontrafficked			42	21
4	Acceptable	3,435	11	22	16
	Unacceptable	2,625	23	3	3
	Nontrafficked			24	7

Table 4.27 Creep Moduli and Stabilometer "S" Values Associated with Different Levels of Performance (Finn et al. 1983)

Performance	Creep Modulus - psi			Stabilometer	
	Mean	Standard Deviation	Number of Specimens	Mean "S" Value	Number of Specimens
Acceptable	29,000	7,900	8	31.6	16
Unacceptable	13,400	5,300	9	9.0	14
Nontrafficked					
Upper Section	32,600	11,600	6	28.3	6
Lower Section	20,900	4,800	6	----	-

3. Stabilometer values are generally less than the values recommended for highway design (i.e. S = 35 min.).
4. Flow values from Marshall tests are high, ranging from 15 to 23 for four of the eight conditions shown.

Creep tests were also performed at 100•F (38•C) on cure specimens. Only the results obtained after one hour of load application are reported (Table 4.27). In general, it will be noted that as the "S" value increases, the creep modulus also increases. A creep modulus of 34,500 psi would correspond to an "S" value of 35.

The second example was for an airfield pavement in the Middle East. A test section was built with a binder course containing 4.2% asphalt content and a surface course at two asphalt contents: 4.9% and 5.4%. Proof rolling was conducted during two periods: 1) within one week after placement of the asphalt concrete, and 2) during periods of high ambient temperature five months later. A 60-ton pneumatic tired roller with 30,000 lbs on each tire was used to apply the test loading.

Table 4.28 summarizes the resulting deformations--it can be seen that the difference in rutting potential of the two trial surface course mixes are insignificant. However, in two of the three test periods, the deformation increased with an increase in asphalt content.

Creep tests were performed on lab specimens with asphalt contents of 4.5% and 6%. Each specimen was tested at stress levels of 14.7 psi (101 kPa), 30 psi (207 kPa) and 70 psi (483 kPa) and the stress maintained for one hour. Figure 4.21 illustrates the creep moduli as a function of loading time. Extrapolations beyond 36,000 sec. are shown as dashed lines. Note that two different temperatures were used: 80•F for 4.5% asphalts, and 70•F for 6% asphalts. Rut depths were then predicted using the ELSYM5 layered elastic program and the procedure developed by Shell (Van de Loo, 1976). From this, it was observed that rutting range from 4 to 10 times for the 6% asphalt mix compared to the 4.5% asphalt mix.

In addition, for an average "S" value of 46 (4.5% asphalt), the creep modulus at 100•F and one-hour loading time is 37,000 psi, whereas for an "S" value of 1 (6.0% asphalt), the creep modulus is 2,000 psi. This compares well with the Clark County data.

Conclusions

From this investigation, Finn et al. (1983) concluded that:

1. Higher asphalt contents were obtained with volcanic aggregate with the Asphalt Institute criteria for intersections of major arterials in a hot climate than would be indicated by

Table 4.28 Measured Deformations in Proof Rolling Test (Finn et al. 1983)

Time	Coverages	Cumulative Deformation - in.			
		Average		Maximum	
		4.9 Percent	5.4 Percent	4.9 Percent	5.4 Percent
March 1981	1,150	0.05	0.08	---	---
August 1981	1,224	0.05	0.04	0.09	0.12

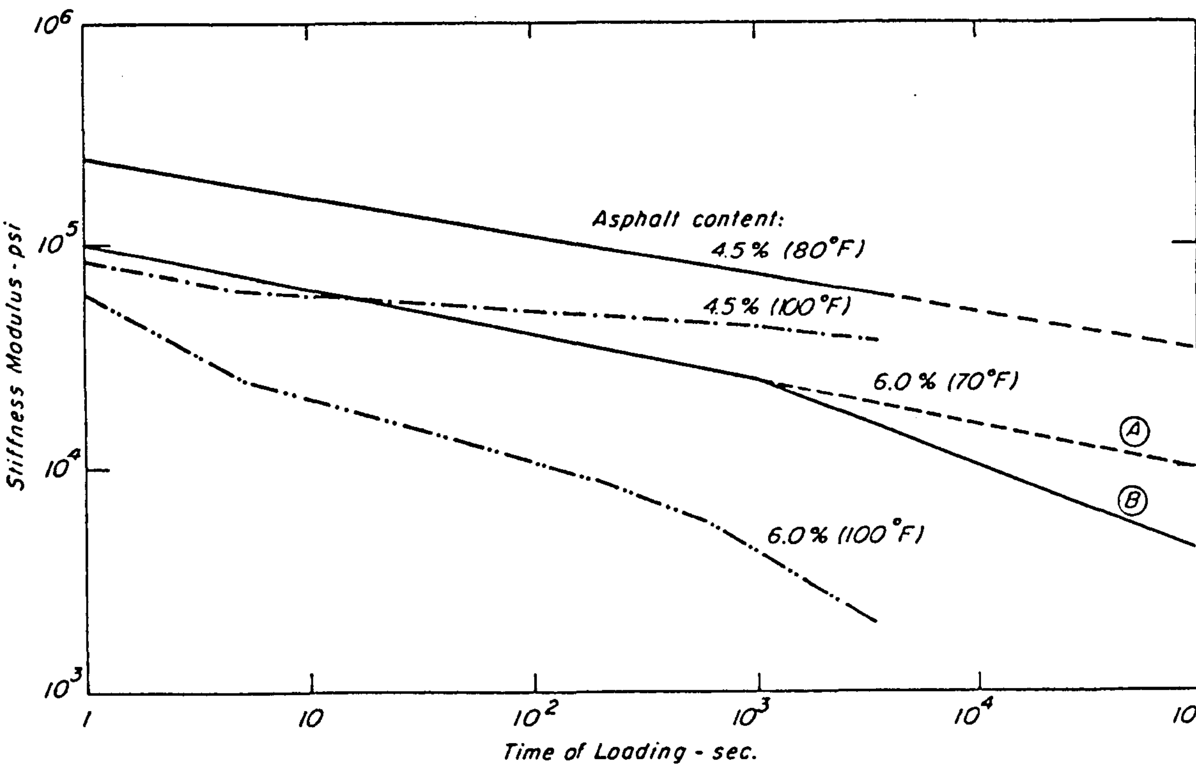


Figure 4.21 Average Creep Curves for Surface Course Mix at Asphalt Contents of 4.5 and 6.0 Percent (Finn et al. 1983)

the California Stabilometer. The mixes so designed exhibited rutting in a relatively short time period.

2. High Marshall stability values do not assure rutting resistance.
3. For Clark County, a creep modulus of 34,500 psi at 100°F, 36,000 sec. loading time, is correlated with a stabilometer "S" value of 35. This is the minimum recommended to preclude rutting.
4. For the Middle East airport, the average "S" value of 46 (4.5% asphalt) corresponded to a creep modulus of 37,000 psi. However, no significant rutting was observed in this study.
5. The creep test appears to be a useful supplement to mixture design.

5.0 SUMMARY

This survey of published literature was designed to provide information which could relate in-situ asphalt properties and the characteristics of asphalt-aggregate systems to the performance of in-service pavements. In reviewing the literature, specific types of information were stipulated to be of major interest, namely, (i) chemical properties such as fractional composition, which is generally related to the chemical activity of asphalt, and (ii) physical properties such as rheological characteristics of asphalt which can be related to properties of asphalt-aggregate mixtures. Specific objectives for the literature review were as follows:

1. Provide an exhaustive compilation of references (through December 1990) related to the SHRP asphalt research program and which would be useful to future investigators interested in investigating asphalt and asphaltic mixtures;

2. To identify those asphalt properties which are related to pavement performance and which can be used as a basis for applying a "test of reasonableness" to test results under investigation by the A-003A contractor. For example, if low temperature cracking studies produce results contrary to field experience an intensive re-evaluation of the test method or the interpretation of measured mix properties would be required;

3. To identify those asphalt properties to which test results should be sensitive. For example, if field data indicates that asphalt consistency and temperature susceptibility significantly influence performance of asphalt-aggregate mixtures, it will be important to include in all of the studies, asphalts with a wide range of physical properties in order to determine how well the test will reflect differences in these asphalt properties;

4. To identify problems with past research which create difficulties when attempting to pool data from widely spaced (time and location) field projects; and

5. Provide a basis for ranking asphalt properties from good to poor or from acceptable to unacceptable.

For purposes of this review, performance has been defined in terms of (i) fatigue cracking, (ii) low temperature cracking, (iii) permanent deformation, (iv) moisture sensitivity of asphalt and asphalt-aggregate systems, and (v) aging of asphalt and asphalt-aggregate systems.

Figure 5.1 provides some indication of the associations used to guide the review of the literature. Task B (SHRP Task 1.4) indicates the role of the literature review in helping to meet SHRP's objectives in the asphalt research program.

5.1 CONSIDERATIONS FOR EVALUATING DATA FROM CONTROLLED AND UNCONTROLLED FIELD PROJECTS

Before summarizing overall results from the review of the literature, it is important to note some of the problems which adversely affected the reviewer's ability to develop any consensus from the large number of publications included in this survey.

1. Confounding factors related to controlled field investigations - As defined, a controlled study would be one that was planned with a specific objective identified. Good examples would be the Zaca-Wigmore, Ste. Anne or the European trials in France and Germany. However, even these studies were confounded in so far as the ability to develop specific conclusions or performance information required for SHRP. For example, the Zaca-Wigmore project was constructed partially over a cement-treated base which in some cases was over an old portland cement concrete pavement; both factors would influence the occurrence of cracking in the asphalt concrete. The project was constructed in two different periods involving different weather conditions. The Ste. Anne Test Road included a limited number of asphalts (four), one of which was a SC-5 liquid asphalt. Although the Ste. Anne project included 29 test sections, there were a number of variables included with the study; i.e., four asphalts, three types of aggregates in the mixture, two subgrade types, three asphalt contents and three structural sections or 216 possible combinations. Nevertheless, this project produced some very useful information for low temperature cracking and represents one of the most definitive with regard to asphalt properties, mixture characteristics and pavement performance. The European study was one of the largest and was well planned. Asphalt testing, both chemical and physical, was extensive; however, the authors concluded that differences in performance were due to differences in aggregate gradation and defects in the road structure. No conclusions were reported relative to the role of the asphalt even though that was the primary objective of this large (110 test sections) controlled study.

   Similar problems can be associated with most of the controlled studies, e.g., investigations based on the properties of overlays or lack of information concerning

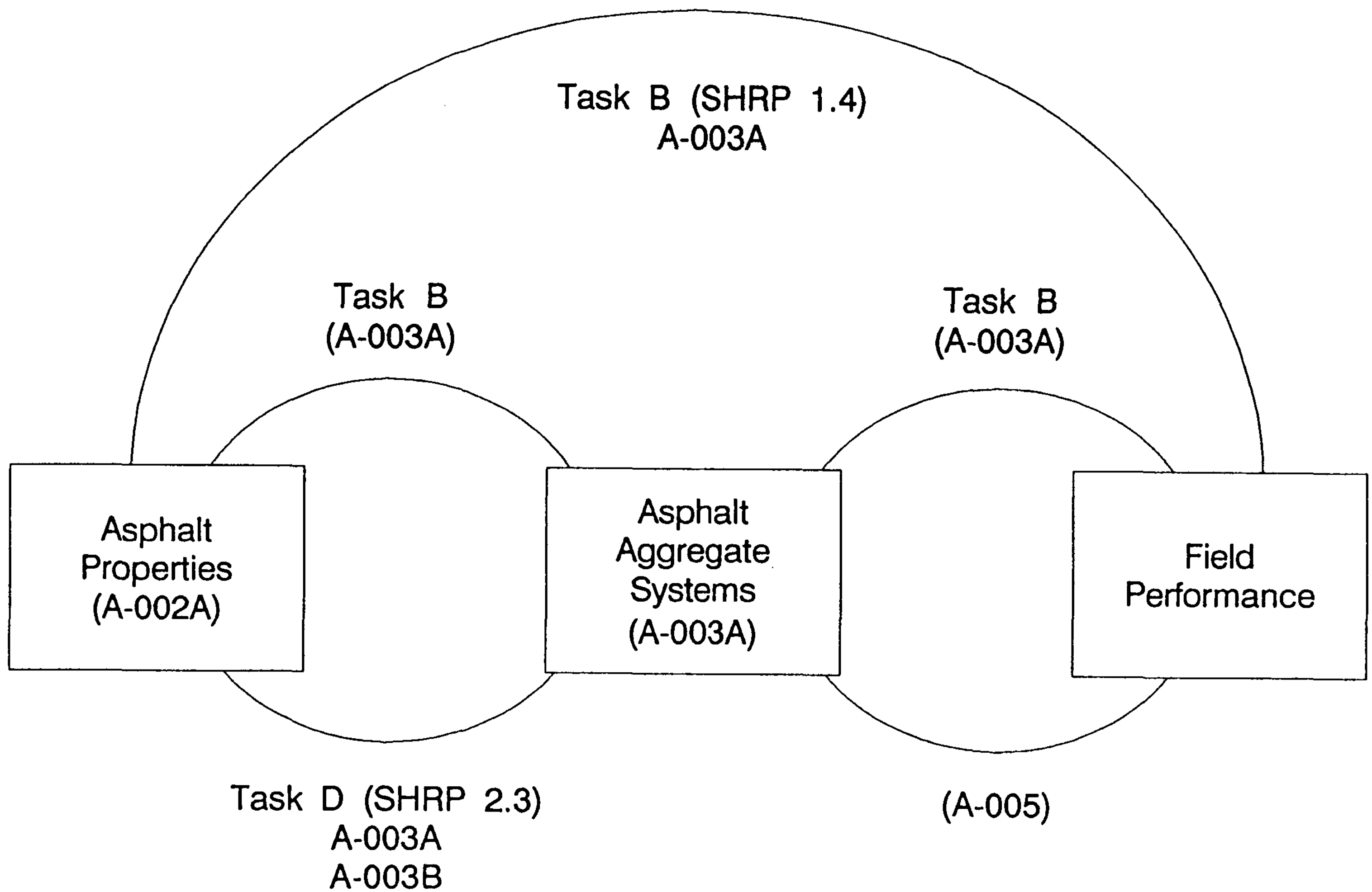


Figure 5.1. Relationship of A-003A Task B to relate asphalt properties to field performance.

traffic or structure. These confounding factors make it difficult but not impossible to draw some conclusions if a consensus evolves from a consolidation of findings.

2. Confounding factors related to uncontrolled investigations - As defined, an uncontrolled study involved studies of existing in-service pavement which were sampled and tested without benefit of a well-formulated experiment design but which were planned to obtain information relative to the influence of asphalt properties to performance. The very nature of such studies sometimes makes it difficult to isolate specific properties and relationships; however, most of the studies included in this category were well planned with specified objectives and extensive testing. The results of these were considered useful for the sponsoring agency but were not especially productive in developing quantitative information which could be extended to other areas or materials.

3. Properties of asphalt and asphalt-aggregate systems - The majority of projects evaluated asphalt properties using traditional tests to measure consistency of asphalt and asphalt-aggregate systems. Specifically, most tests included only measures of penetration, viscosity and ductility for asphalt and Marshall or Hveem stability for mixtures. While these properties have proven useful in the past, and may be used in the future, the emphasis in SHRP is on new and innovative testing designed to be more strongly related to pavement performance.

   The types of testing for physical properties of asphalt are likely to include dynamic viscoelastic properties such as described by Ishai, Brule, Vaniscote and Ramond (1988), Sisko and Brunstrum (1969), and Goodrich (1988). This type of testing is not necessarily new, having been reported by such prominent researchers as van der Poel, Heukelom dating back to 1958. Such tests will produce information such as complex shear modulus, dynamic viscosity, storage (elastic) modulus and loss tangent (ratio of viscous modulus to elastic modulus) as reported by Goodrich (1988). It is expected that these properties will provide improved correlations with the mechanical properties of the asphalt-aggregate system.

   The types of tests for asphalt-aggregate systems are likely to include measurements that identify thermal coefficients, elastic, viscoelastic, shear, fatigue properties and would, in most respects, reflect comparable characteristics to those used with asphalt binders. The tests will be designed for use with analytical models to simulate stress, strain and deformation under in-service conditions.

Unfortunately, measurements of the type desired by SHRP were limited largely to laboratory investigations and rarely included field studies as reported in the published literature.

4. Test methods used for asphalt - A variety of test methods were used to measure penetration, viscosity and ductility of asphalt. Penetration was normally measured under standard ASTM procedures; however, variations on such procedures were sometimes used to measure penetrations at temperatures other than 25°C (77°F). Viscosity measurements were made with a range of viscometers and rheometers at various shear rates. The use of different instruments complicates comparative results between field projects.

   The estimates of temperature susceptibility and asphalt durability have a large number of variations, again making comparisons difficult.

   There are no baseline tests to identify chemical composition or fractions. Similarities in fraction identification do exist; however, comparisons are difficult when evaluating reported properties since the types of information vary depending on test methods. However, the purpose of this report is not to make comparisons in methods. This report is primarily interested in identifying possible associations between chemical fractions and performance. Thus, each method is evaluated on its own merits and comparisons are not important.

   It is recommended that SHRP develop a set of standard test methods and instruments which can be used as a baseline for comparisons of asphalt properties.

5. Variations in measurements of performance (distress) - There is no consistency between field projects as regards methods used to measure or evaluate performance. For example, some projects used a simple numerical scale, i.e., 0 for poor to 5 for good, based on visual observations; others measured the area of cracking or amount of rutting; still others simply only referred to good, fair and poor. The most consistent standards are those used for low temperature cracking in which full-width or half-width cracks are combined into a single index.

   As with testing, it will be important that SHRP establish a baseline for condition surveys that can be used for all future investigations related to pavement performance.

6. Lack of a strong consensus relative to conclusions - Findings from one investigation do not always concur with those from another investigation. The best example of this type of problem can be illustrated by findings reported for the influence of in-service asphalt penetrations on fatigue cracking. The Zaca-Wigmore study concluded that standard penetration values of 30 or less were associated with fatigue cracking. However, a West Texas study with a recovered asphalt penetration of 5 evidenced no cracking; likewise studies in Delaware and Louisiana reported recovered penetration values less than 30 with little or no cracking and definitely less cracking than comparable sections with higher penetration values. Similar comparisons can be made for viscosity and ductility. Nevertheless, there is some overall indication, based on the published literature, that asphalts with lower penetration values, higher viscosity or low temperature ductility values contribute to reduced service life in terms of fatigue or low temperature cracking. It should be noted that many of these studies are based on thin overlays and most do not involve asphalt concrete thicknesses in excess of 5 to 6 inches for new construction.

   A similar lack of consensus exists with regard to chemical composition. Unfortunately most reports which did include information on chemical properties were inconclusive. For example, the Michigan and European studies were unable to relate chemical fractions to performance. However, Rostler and White (1959) and Jamieson and Hattingh (1970) have shown correlations between chemical fractions and pavement performance. Likewise Jennings et al. (1982) and Hattingh (1984) have shown relationships between HP-GLC characterization and performance.

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

5.2 INTERPRETATION OF RELATIONSHIP BETWEEN ASPHALT PROPERTIES AND PAVEMENT PERFORMANCE

An effort has been made to evaluate the influence of asphalt properties on performance. Two approaches have been used to determine if and how well both quantitative and qualitative relationships can be established.

Qualitative Relationships

Tables 5.1 through 5.6 provide information pertinent to qualitative relationship. Table 5.1 shows by arrows the qualitative relationships; e.g., for low temperature cracking, increasing values of recovered asphalt viscosity was associated with increasing amounts of cracking. The table reference; i.e., Table 5.2, provides sources of information for ready reference. The codes within each cell indicate the number of sources (8 for low temperature cracking), the recommendation of threshold values, not necessarily the same values (Y for yes, N for no), and finally, an indication of contradictions to the general finding (0 for no, 1 for yes).

From Table 5.1, it is clear that asphalt properties can influence pavement performance. Although the amount of information is limited, there is little doubt that relationships do exist. Except for rutting, the effects of aging tend to have adverse effects on performance. It should be noted, however, that according to some investigations (Finn et al. 1978), asphalt aging may not be detrimental for fatigue cracking, provided such materials are incorporated in relatively thick layers; i.e., 8 inches or more, of asphalt concrete in the pavement structure.

Quantitative Relationships

In order to evaluate and quantify possible relationships between asphalt properties and performance, the following procedure was used:

1. Select results from a single investigation; establish relationships and compare with other projects to determine how well predictions compare with actual performance.

   Due to the large amount of variability between projects, data from more than one project was not pooled, although such an approach would have some obvious benefits in attempting to achieve a consensus in findings.

2. Look for simple correlations between all variables for which there is some basis for expecting an association; e.g., summary of qualitative relationships in Table 5.1.

3. Examine the data, using graphs to look for outliers and trends.

4. Start with simple (single-factor) models with what is believed to be the strongest variable.

5. Investigate the possible use of multiple regressions in order to improve the correlations.

Table 5.1 Summary of results: asphalt and mix properties and relationships to pavement performance

ASPHALT PROPERTIES / PERFORMANCE VARIABLES	PHYSICAL										CHEMICAL									
	VISCOSITY	PENETRATION	DUCTILITY	TEMPERATURE SUSCEPTIBILITY	MIX STIFFNESS	ASPHALT STIFFNESS	ASPHALT SOURCE	AGING INDEX	LMS	SOFTENING POINT	INTERACTION COEFFICIENT	PARAFFIN CONTENT	N/P RATIO	ASPHALTENES	NAPHTENE AROMATICS	POLAR AROMATICS	FIRST ACIDAFFINS	SECOND ACIDAFFINS	ROSTLER PARAMETER	AIR VOIDS
LOW TEMPERATURE CRACKING ↑ (TABLE 5.2)	↑ 8 Y0	↓ 9 Y0	↓ 3 Y1	↑ 5 Y0	↑ 1 Y0	↑ 6 Y0	▬ 4 N0	↑ 1 Y0	↑ 2	—	—	↑ 1 Y0	↓ 1 Y0	—	↑ 1 N0	—	—	—	—	—
FATIGUE CRACKING ↑ (TABLE 5.3)	↑ 1 Y0	↓ 2 Y0	↓ 2 Y0	—	—	—	▬ 1 N0	—	—	—	↑ 1 Y0	—	—	—	—	—	—	—	—	—
RAVELLING ↑ (TABLE 5.6)	↓ 1 Y0	—	↓ 1 Y0	—	—	—	—	—	—	—	—	—	—	↑ 1 Y0	↓ 1 Y0	—	—	—	—	—
RUTTING ↑ (TABLE 5.4)	▬ 1 NO	▬ 1 N0	—	↓ 1 N0	—	—	▬ 1 N0	—	—	—	—	—	—	↓ 1 NO	—	—	—	—	—	↑ 1 Y0
AGING ↑ (TABLE 5.5)	↑ 5 Y0	↓ 5 Y0	↓ 3 Y0	↓ 1 N0	—	—	—	↑ 1 Y0	—	↑ 4 N0	▬ 1 N0	↑ 1 N0	—	↑ 6 Y0	↓ 1 N0	▬ 1 N0	↑ 1 N0	↓ 1N0	↑ 2 N0	↑ 3 Y0

Note: An ↑ arrow indicates that as this property increased, the preformance variable also increased, eg as viscosity increased, low temperature cracking increased. ▬ indicates that no trends were reported. The 3-digit codes refer to the number of reports reviewed, whether a threshold or limiting value was reported, and if any reports contradicting the trends were reported: eg 8 Y0 would mean 8 reports where reviewed, limiting values were reported, and no contradictory reports to the ↑ trend was found.

Table 5.2 Summary of results for low temperature cracking

ASPHALT PROPERTY	TEST ROAD	RESULTS/COMMENTS
Viscosity	Pennsylvania (1976)(Sect. 3.3)	Performance did not correlate with viscosity @ 140•F & 275•F of recovered asphalt after six years.
	Utah (Section 3.7)	Transverse cracking did not correlate well with Cannon Cone Viscosity @ 77•F .
	Montana Big Timber (Section 3.11)	No apparent correlation.
	Manitoba & Ontario Air-Blown (Section 3.14)	Air-blown, low viscosity asphalts exhibited little low temperature cracking after four years.
	Alberta (Section 3.15)	Minimum viscosity of 275 poises @ 140•F was adopted as a result of this study.
	Pennsylvania (1964) (Section 3.16)	The slope of the relationship of shear susceptibility and viscosity (@ 77•F) had a better correlation than either property alone.
	West Texas (Section 4.7)	Transverse crack frequency increases with viscosity @ 77•F. However, there was no relationship developed for viscosity @ 275•F.
	Quebec (Section 4.17)	Transverse cracking on 6-9 year old pavements increases with viscosity @ 77•F of recovered asphalts.
Penetration	Ste. Anne (Section 3.2)	300-400 penetration (100 g., 5 secs., 77•F) asphalts exhibited less transverse cracking than 150-200 penetration asphalts.
	Pennsylvania (1976)(Sect. 3.3)	Penetration @ 77•F of the recovered asphalt after six years indicated that poor performance correlated with a low penetration. The worst pavements had penetrations of 15 & 22, the best at 35.
	Utah (Section 3.7)	Transverse cracking did not correlate well with penetration.

Table 5.2 Summary of results for low temperature cracking (continued)

ASPHALT PROPERTY	TEST ROAD	RESULTS/COMMENTS
Penetration (Continued)	Montana Big Timber (Section 3.11)	No apparent correlation.
	Alberta (Section 3.15)	Minimum penetration of 250 @ 77•F was adopted to reduce low-temperature cracking.
	Wyoming (LTC) (Section 4.6)	Low-temperature cracking increases as penetration decreases.
	West Texas (Section 4.7)	Transverse crack frequency increased as penetration @ 77•F and 32•F decreased.
	Plummer & Zimmerman (Section 4.10)	Cracking increased with a lower penetration (100 g., 5 secs., 77•F).
	Quebec (Section 4.17)	The number of transverse cracks increased as the penetration @ 25•C decreased. At a penetration of 30, this results in 10 transverse cracks/km for light to medium traffic.
Ductility	Utah (Section 3.7)	Force ductility values of original cements @ 4•C. A critical value of 7 lbs. or less resulted in less than 10 transverse cracks per kilometer. However correlations were poor.
	Pennsylvania (1964)(Sect. 3.16)	High ductility @ 39.2•F before and after mixing are associated with better pavement performance after 113 months based on subjective rating suggested by Olsen et al. (1969).
	Pennsylvania (1961-62) (Section 4.11)	Good pavement condition was noted when ductility @ 60•F was maintained above 10 cm. Load-associated cracking developed when ductility value fell to 3-5 cm.
Temperature Susceptility	Ste. Anne (Section 3.2)	Less temperature susceptible (150-200 pen HVA) asphalts had no cracking after eight years and more temperature susceptible (150-200 pen LVA) asphalt cracked during the first winter.

Table 5.2 Summary of results for low temperature cracking (continued)

ASPHALT PROPERTY	TEST ROAD	RESULTS/COMMENTS
Temperature Susceptility (continued)	Ontario (McLeod) (Section 3.6)	Low temperature cracking increases as PVN decreases.
	Utah (Section 3.7)	Temperature susceptibility (77•F-140•F) of the asphalt calculated using: $$\text{Temperature Susceptibility} = \frac{\log\log\eta_1 - \log\log\eta_2}{\log T_2 - \log T_1}$$ where η_1 and η_2 are viscosity readings @ T_1 and T_2, respectively. Below a temperature susceptibility of 4.2, less than 10 transverse cracks per kilometer were measured. However, correlations are poor.
	Montana Big Timber (Section 3.11)	High PVN (low temperature susceptibility) characterizes cracking tendencies.
	Alberta (Section 3.15)	Highest temperature susceptibility asphalts showed tendency to crack earliest.
Mix Stiffness	Ontario (McLeod) (Section 3.6)	Pavement Modulus of Stiffness value is based on Van der Poel's nomograph for a loading time of 20,00 seconds and applies to "well-designed paving mixtures," with a C_v of 0.88 (14.5% VMA and 3% air voids). For limiting the number of transverse cracks to 20 per mile, the limiting pavement modulus is 1 million psi.
Asphalt Stiffness	Ste. Anne (Section 3.2)	Critical stiffness moduli for asphalt binder at the time of cracking was determined to be 10^9 N/m^2 (1.45 x 10^5 psi) at a time of loading of 1800 sec.

Table 5.2 Summary of results for low temperature cracking (continued)

ASPHALT PROPERTY	TEST ROAD	RESULTS/COMMENTS
Asphalt Stiffness (continued)	Pennsylvania (1976)(Sect. 3.3)	The higher the stiffness values (determined from Van der Poel's, McLeod's and Heukelom's methods), the greater the transverse cracking found. Limiting stiffness criteria of 20,000 psi @ 10,000 sec. (Fromm & Phang) for recovered asphalts immediately after construction was verified.
	Ontario (McLeod) (Section 3.6)	Asphalt stiffness calculated using Van der Poel's nomograph. For not more than 20 transverse cracks/lane mile, the limiting stiffness is 5000 psi.
	Oklahoma (Section 4.3)	Stiffness modulus of recovered asphalts (determined using McLeod's method) correlates with cracking indices. The stiffer the asphalt, the greater the degree of transverse cracking.
	Saskatchewan Air-Blown (Section 3.13)	Stiffness did <u>not</u> reach critical value of 20,000 psi at loading time of 10,000 sec. despite low temperature of -40·F after 24 months.
	Ontario (Fromm & Phang) (Section 4.4)	Stiffness moduli for asphalt cement is the major factor governing transverse cracking.
Asphalt Source	Utah (Section 3.7)	Asphalt source is the major variable affecting performance parameters (transverse cracking), physical properties and chemical makeup.
	Saskatchewan (Section 3.8)	Roads constructed from five refineries had different transverse cracking results. However, Culley (1986) concludes that this is due to compaction densities and environmental results than any one variable.
	Montana Big Timber (Section 3.11)	The number of cracks ranged from 1 to 23 cracks after four years. However, the authors caution that asphalt content may have an over-riding significance.

Table 5.2 Summary of results for low temperature cracking (continued)

ASPHALT PROPERTY	TEST ROAD	RESULTS/COMMENTS
Aging Index	Utah (Section 3.7)	The Aging Index is calculated as: $\text{Aging Index} = \dfrac{\text{Field Viscosity @ 60}\cdot\text{C}}{\text{Original Viscosity @ 60}\cdot\text{C}}$ An Aging Index less than 6.5 results in less than 10 transverse cracks/km.
LMS	Montana Big Timber (Section 3.11)	Large molecular size content (using High Pressure Gel Permeation Chromatography (HP-GPC) greater than the theoretical ideal demonstrate a tendency to crack.
	Plummer & Zimmerman (Section 4.10)	Larger mean molecular size fraction (MMSF) values measured using high performance liquid chromatography (HPLC) were obtained from cracked roads than from uncracked roads.
Paraffin Content	Utah (Section 3.7)	Paraffin Content as measured with Rostler's method. A paraffin content of less than 15% indicates that less than 10 transverse cracks/km were measured. However, correlations were poor.
N/P Ratio	Utah (Section 3.7)	A nitrogen to paraffin ratio greater than 2 resulted in less than 10 transverse cracks/km.
Naphthene Aromatics	Montana Big Timber (Section 3.11)	Lower concentrations associated with resistance to cracking.

Table 5.3 Summary of results for fatigue cracking

ASPHALT PROPERTY	TEST ROAD	RESULTS/COMMENTS
Viscosity	Zaca-Wigmore (Section 3.1)	Viscosity @ 77•F and 0.05 sec^{-1} shear rate using Shell microviscometer. Limiting value is 20 megapoises for 10% fatigue cracking of travel lane.
Penetration	Zaca-Wigmore (Section 3.1)	Penetration of recovered asphalt @ 77•F. Limiting value of recovered penetration after 97 months was 30 based on 10% alligator cracking of travel lane as failure criterion.
	Washington (Section 4.15)	Penetration of recovered asphalt @ 77•F alone could not be related to pavement performance (measured as cracking, including fatigue).
Ductility	Pennsylvania (Section 4.11)	Ductility @ 60•F, 5 cm/min–Authors note that pavement condition was satisfactory when ductility @ 60•F was maintained above 10 cm; Load-associated cracking began to develop when the ductility value fell in the approximate range of 3 to 5 cm.
	Washington (Section 4.15)	Ductility @ 45•F (1 cm/min) increased with good pavement performance (measured as cracking, including fatigue).
Asphalt Source	Zaca-Wigmore (Section 3.1)	Different asphalt sources were used to construct the test section which exhibited different fatigue cracking behavior. Some performed better than others.
Interaction Coefficient	Zaca-Wigmore (Section 3.1)	Phenol Interaction Coefficient measured as one of the parameters available from Inverse Gas Liquid Chromatography (IGLC). The phenol interaction coefficient is a measure of asphalt polarity. The asphalts that developed greater amounts of strongly interacting polar groups during aging had a shorter service life. Limiting value is 145, based on 10% allowable fatigue cracking of the travel lane.
Rostler Parameter	South Africa (Section 4.9)	The Rostler Parameter should be 1.0 to 1.7 for optimum performance, not 0.8 to 1.5 as suggested by Halstead et al. (1986)

Table 5.4 Summary of results for rutting

ASPHALT PROPERTY	TEST ROAD	RESULTS/COMMENTS
—	Utah (Section 3.7)	Regression equations were developed relating rutting to asphalt content, traffic, ductility and the Nitrogen base content. However, no data were presented in Anderson et al.'s (1976) report.
—	Quebec (Section 4.17)	Insufficient rutting data was collected for a correlation.
Viscosity	Saskatchewan (Rutting) (Section 4.13)	No relationship was found.
Penetration	Saskatchewan (Rutting) (Section 4.13)	No relationship was found.
Temperature Susceptibility	Montana Big Timber (Section 3.11)	High PVN shows rutting tendencies.
Asphalt Source	Montana Big Timber (Section 3.11)	Rut depths vary from 0.2 to 0.6 inches after four years. However, asphalt content may have an over-riding influence.
Asphaltenes	Montana Big Timber (Section 3.11)	Lower asphaltene content characterizes asphalts that are rutting most severely.
Air Voids	Saskatchewan (Rutting) (Section 4.13)	A 4% maximum air voids was found to be the threshold value, as interpreted by authors of this literature review. In general, rutting correlated well with VMA, voids filled, asphalt content and Hveem stability.
*Creep Modulus	Clark County, Nevada (Section 4.19)	A minimum creep modulus of 34,500 psi @ 100•F, 36,000 sec. loading time is recommended to preclude rutting. This corresponds to a stabilometer "S" value of 35.

*This property is not included in Table 5.1.

Table 5.5 Summary of results for aging

ASPHALT PROPERTY	TEST ROAD	RESULTS/COMMENTS
Viscosity	Zaca-Wigmore (Section 3.1)	Viscosity @ 77•F, shear rate of 0.05 sec^{-1} (Shell microviscometer) indicate rapid hardening during first 16-20 months, and a decrease in hardening rate thereafter. At 35 months, viscosities ranged from 4 to 60 megapoises. Top 1/4" has highest viscosity.
	European Test Roads (Section 3.4)	Viscosity @ 15•C & 35•C using a sliding plate microviscometer (constant shear/stress) showed an increase during mixing and laying, with a slight decrease in the hardening rate (measured as an aging index) thereafter.
	Michigan (Section 3.5)	Viscosity @ 140•F on 18-year old cores range from 17,041 to 34,414 poises.
	Iowa (Section 3.9)	Viscosity @ 77•F increases with time and reaches a limiting value of 17 to 20 megapoises at 50 months.
		Viscosity @ 140•F increases with time and reaches a limiting value of 7500 to 8200 poises for 50-month old cores.
	Texas (Traxler) (Section 4.8)	Viscosity @ 77•F, 95•F, 140•F and 275•F increases with time, with the greatest increase occurring after mixing.
	Florida (Section 4.18)	Viscosity @ 77•F, 140•F increases with age.
Penetration	Michigan (Section 3.5)	Penetration (100g., 5 secs., 77•F) on recovered cores 4-11 years old showed a decrease with time.
	Iowa (Section 3.9)	Penetration (100g., 5 secs., 77•F) decreases to approximately 30-32 as the limiting value in 50 months.
	Delaware (Section 3.10)	Penetration (100g., 5 secs., 77•F) decreases with age. After 2 years, penetrations were less than 25.
	Wyoming (LTC) (Section 4.6)	Penetration decreases with age.

Table 5.5 Summary of results for aging (continued)

ASPHALT PROPERTY	TEST ROAD	RESULTS/COMMENTS
Penetration (continued)	West Texas (Section 4.7)	Penetration @ 25•C of original cements had decreased to 15-50 with time (13 years).
	Florida (Section 4.18)	Penetration @ 25•C decreases with age.
Ductility	Michigan (Section 3.5)	Ductility @ 60•F decreases to 0.5 to 2.5 cm. @ 18 years for top 1/8" AC. Ductility @ 77•F decreases to 5 to 7 cm. @ 18 years for top 1/8" AC.
	Iowa (Section 3.9)	Microductility @ 77•F, cms decreases with time to a limiting value (not given).
	Federal Highway Administration (Section 4.2)	Ductility @ 60•F decreases with age. At 2% air voids, the ductility is 15 after 11-13 years.
Temperature Susceptibility	Pennsylvania (1976)(Sect. 3.3)	As asphalt cements aged, PI (pen/pen & Heukelom) values increased; i.e., decreasing temperature susceptibility. However, PVN changes were minimal.
Aging Index	Utah (Section 3.7)	Aging Index (calculated using field & orig. viscosity @ 60•F) less than 6.5 results in less than 10 transverse cracks/km.
Softening Point	Michigan (Section 3.5)	Ring & Ball softening point (•F) increases with age.
	Iowa (Section 3.9)	Ring & ball softening point (•F) increases with time and approaches limiting values (not given).
	Delaware (Section 3.10)	Ring & Ball softening point (•F) increases with age to 142•-152•F at the end of two years. There was no pavement distress.
	Wyoming (LTC) (Section 4.6)	Softening point increases with age.
Interaction Coefficient	Wyoming (LTC) (Section 4.6)	There was no consistent variation in the interaction coefficient with age.

Table 5.5 Summary of results for aging (continued)

ASPHALT PROPERTY	TEST ROAD	RESULTS/COMMENTS
Paraffin Content	Federal Highway Administration (Section 4.2)	High paraffin contents as measured with Rostler's methods indicated relatively high amount of hardening.
Asphaltenes	European Test Roads (Sect. 3.4)	Increase in asphaltenes occurred mostly during mixing and laying, with slight changes thereafter.
	Michigan (Section 3.5)	Results reported based on top 1/8" AC layer on 18 year old recovered asphalts. Percent asphaltenes increased.
	Iowa (Section 3.9)	Percent asphaltenes as measured by Rostler's Analysis increases with time, and is a hyperbolic function. The rate of change is different for different asphalts.
	Oregon (Section 4.1)	Asphaltene content increases with aging.
	Federal Highway Administration (Section 4.2)	Percent asphaltenes as measured by Rostler's Analysis increased with time.
	Texas (Traxler) (Section 4.8)	Asphaltenes increased by 1.4-8.3% within one year.
Naphthene Aromatics	Michigan (Section 3.5)	Results reported based on top 3/8" AC layer on 18 year old recovered asphalt. Percent Napthene Aromatics decreased.
Polar Aromatics	Michigan (Section 3.5)	Results reported based on top 3/8" AC layer on 18 year old recovered asphalt. Percent Polar Aromatics showed no distinct trend.
First Acidaffins	FHWA (Section 4.2)	High first acidaffin content resulted in greater hardening of the binder.

Table 5.5 Summary of results for aging (continued)

ASPHALT PROPERTY	TEST ROAD	RESULTS/COMMENTS
Second Acidaffins	Federal Highway Administration (Section 4.2)	High second acidaffin content as measured with Rostler's Methods indicated a lesser degree of hardening.
Rostler Parameter	Iowa (Section 3.9)	No correlation was found with time.
	FHWA (Section 4.2)	The amount of hardening increased with increasing values of the Rostler parameter when air voids were less than 2%. For voids larger than 2%, the relationship is not proportional.
Air Voids	Utah (Section 3.7)	Higher air voids (7.1%) affect hardening to a greater extent than low voids (2.6%). Greater changes in viscosity (increases), ductility & penetration (decreases) occur at high voids.
	FHWA (Section 4.2)	In pavements with voids below 2%, field aging (11-13 years) is negligible. Above 2%, hardening increases.
	Louisiana (Section 4.16)	There was no correlation found between voids and rate of hardening.

Table 5.6 Summary of results on raveling

ASPHALT PROPERTY	TEST ROAD	RESULTS/COMMENTS
Penetration	FHWA (Section 4.2)	Severe raveling was found when the penetration (@ 77°F) of the recovered asphalt was less than 10.
Ductility	FHWA (Section 4.2)	Severe raveling was found when the ductility @ 60°F was less than 3.

6. Recognize that the relatively low values of determination (R^2) may be useful indicators of the influence of asphalt properties.

For the investigation, Sisko and Brunstrum (1968, 1969) was used as a baseline for establishing correlations. This project was selected because it included extensive information concerning asphalt properties and involved projects from twelve states, which would automatically include a variety of asphalt sources. The pavements, with one exception, had been in-service for eleven years at the time visual condition surveys and samples were obtained. The one exception was a pavement that was three years old. Information was also available for the original asphalts. Two deficiencies in the information were: 1) Likely differences in traffic on different projects, and 2) performance was rated only as none, slight, moderate, and severe. For purposes of analysis, the descriptors of performance were changed to numerical values with four intervals or zones of performance.

The simple correlations included the relationships between penetration at 77•F, viscosity at 140•F and 275•F, and percent asphaltenes with cracking and rutting. Figures 5.2 through 5.5 illustrate the simple relationships with cracking. The highest coefficient of determination R^2 was obtained with viscosity at 275•F ($R^2 = 0.716$). It should also be noted that the asphalt properties obtained after the aging in the thin film oven test did not correlate with cracking.

An effort was made to improve the correlations by using multiple variables incorporating physical and functional properties of the asphalt. The R^2 values did increase; however, two problems were encountered which were sufficiently compelling to discard the results. First, there were high correlations between the independent variables included in the regression, and second, the algebraic signs associated with some of the variables did not correspond to the qualitative relationships shown in Table 5.1.

Table 5.7 summarizes asphalt properties associated with acceptable and unacceptable levels of performance.

Plots of asphalt consistency versus ranking according to levels of rutting indicated poor relationships and correlations are not reported. As shown in Figure 5.6, there was a fair correlation

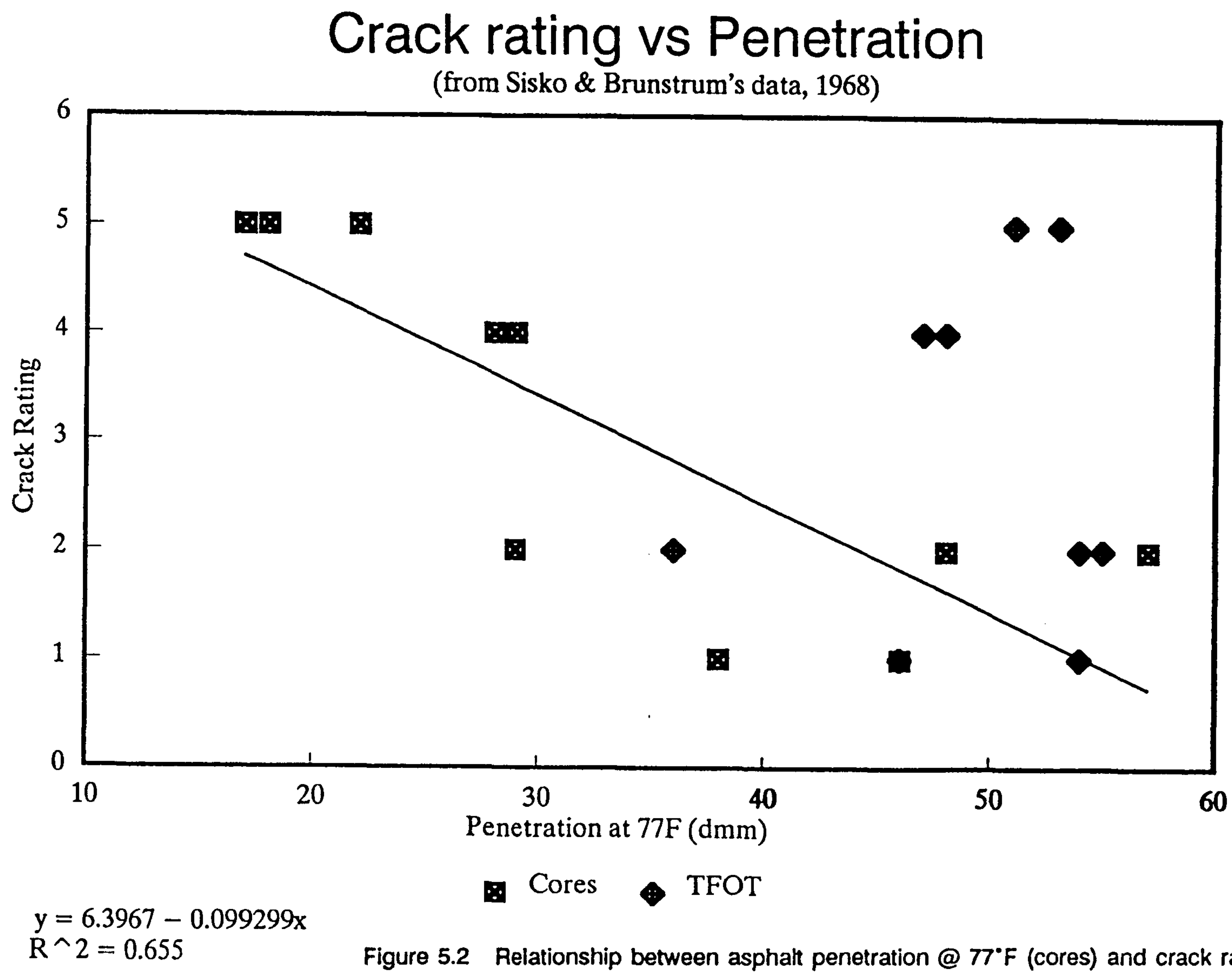


Figure 5.2 Relationship between asphalt penetration @ 77°F (cores) and crack rating

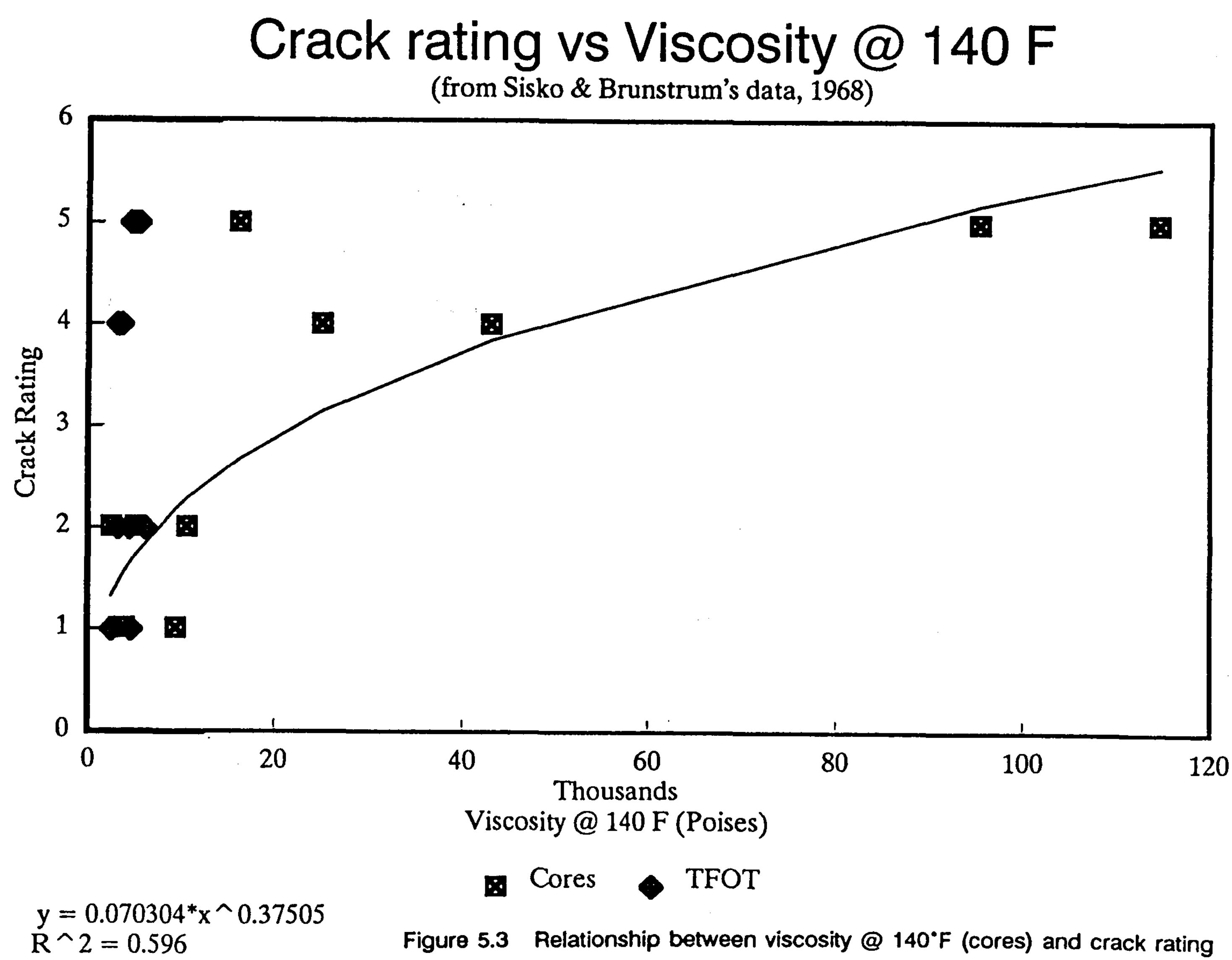


Figure 5.3 Relationship between viscosity @ 140°F (cores) and crack rating

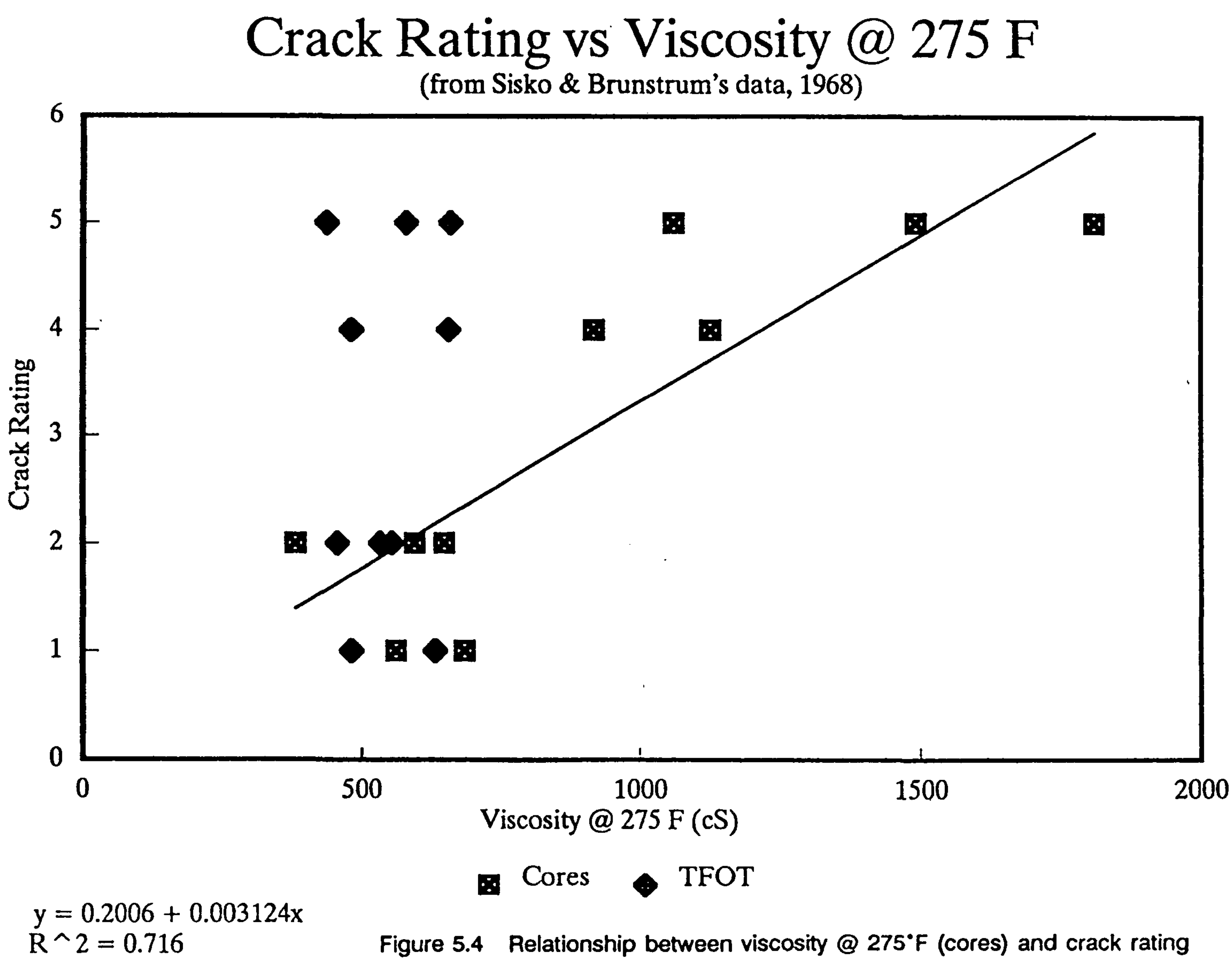


Figure 5.4 Relationship between viscosity @ 275°F (cores) and crack rating

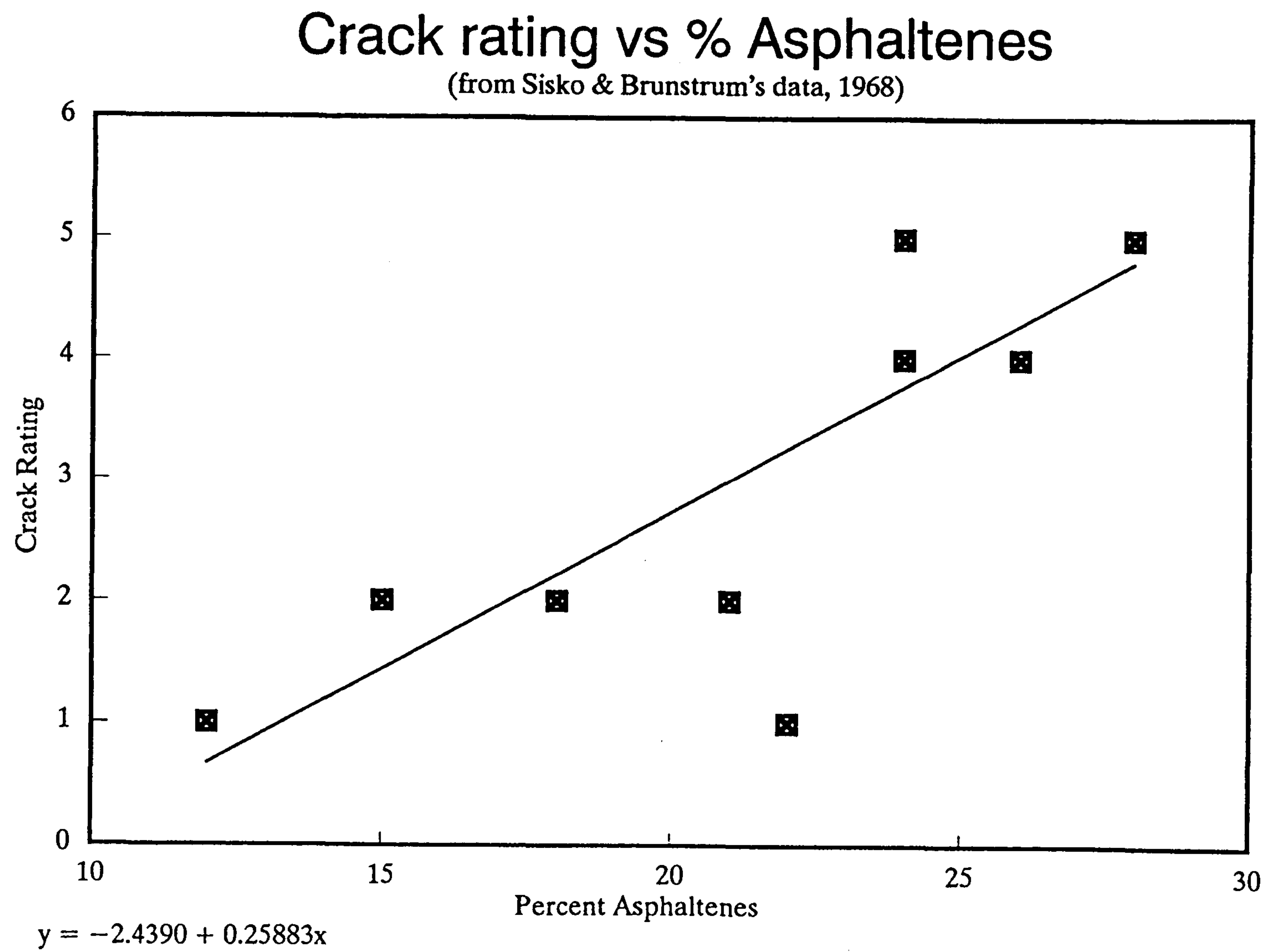


Figure 5.5 Relationship between percent asphaltenes (cores) and crack rating

Table 5.7 Asphalt properties associated with acceptable and unacceptable performance (Sisko & Brunstrum, 1968, 1969)

PROPERTY	CRACK RATING		
	ACCEPTABLE		UNACCEPTABLE
	2	3	4
Penetration @ 77•F	44	34	24
Viscosity @ 140•F Poises	7,533	22,206	47,818
Viscosity @ 275•F cS	576	896	1,216
Asphaltenes, %	17	21	25

Table 5.8 Percent asphaltenes associated with acceptable and unacceptable performance (Sisko & Brunstrum, 1968, 1969)

PROPERTY	RUT DEPTH RATING		
	ACCEPTABLE		UNACCEPTABLE
	2	3	4
Asphaltenes, %	21	16	11

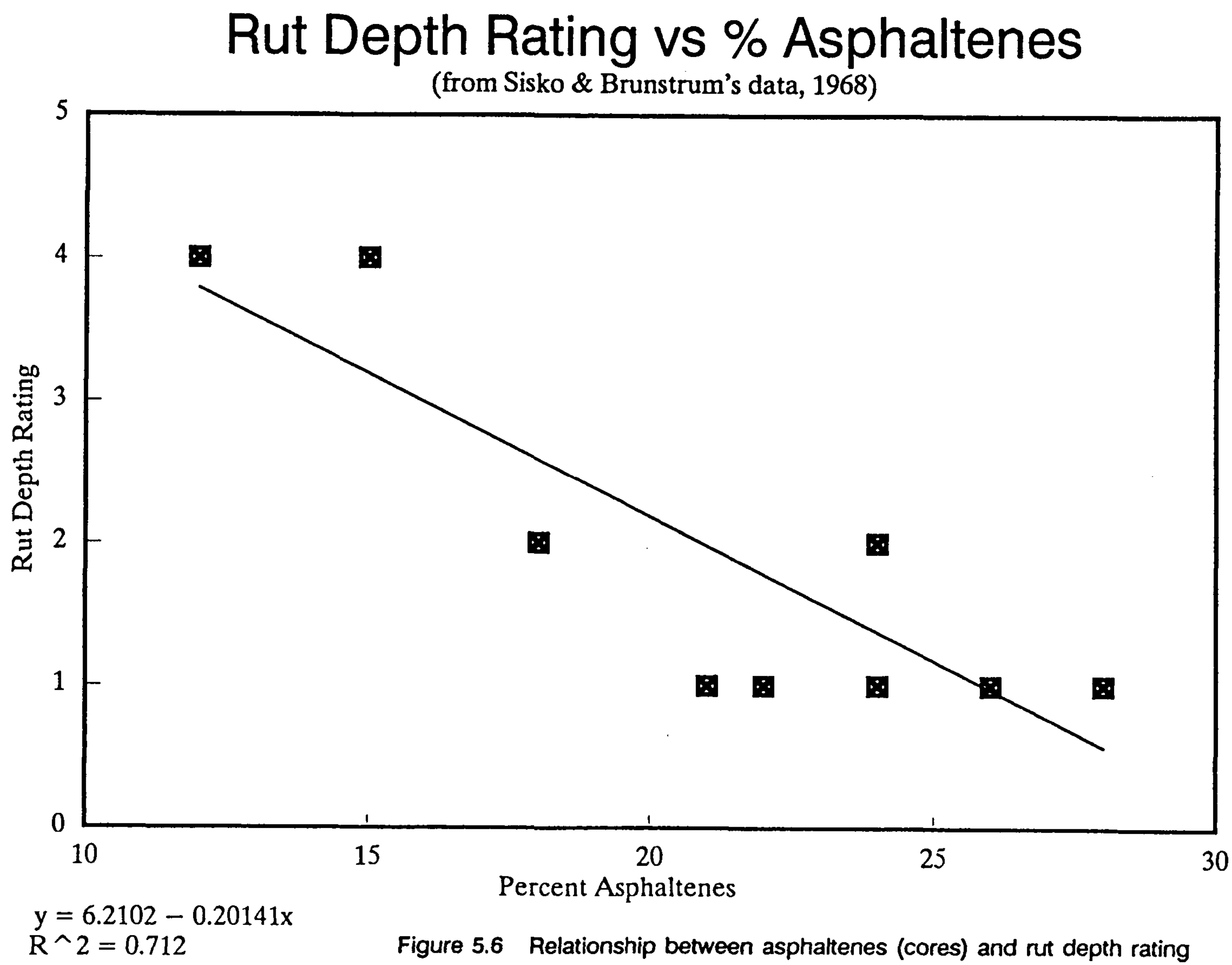


Figure 5.6 Relationship between asphaltenes (cores) and rut depth rating

between rut depth rankings and percent asphaltenes ($R^2 = 0.712$). Threshold values are shown in Table 5.8.

In order to verify the applicability of information summarized in Table 5.7, an effort was made to apply such associations to other projects which had similar information. Hugo and Kennedy (1985) summarized information from field trials in South Africa based on reports by Dickinson (1982). An interpretation of information reported by Hugo and Kennedy indicated that sections with cracking contained asphalt with an average viscosity of 5.17 log viscosity at 122°F expressed in Pa.S and with a standard deviation of 0.75. These values would correspond to an average viscosity in poises of 1,479,108 at 122°F and 0.05 sec^{-1} rate of strain with plus or minus one standard deviation ranging from approximately 263,027 poises to 8,317,638 poises, a very large difference. These viscosities would be at a level indicative of a high potential for cracking when compared with findings from other field investigations (such as Sisko and Brunstrum, 1968).

Jennings et al. (1980), as part of their recommendations to the Montana Department of Highways, and based on a wide range of field studies, recommended asphaltene contents of 12.5 to 16.5 percent in order to minimize cracking. This range compares favorably with data shown on Figure 5.5.

Jennings recommended asphalt penetration values higher than 120 to minimize cracking. These values are believed to be associated with low temperature cracking which may not have been considered in the Sisko & Brunstrum ranking.

Based on information reported by Jennings et al. (1988), it was possible to develop a relationship between rut depth and percent asphaltenes as follows:

$$\text{Rut depth (inches)} = 1.267 - 0.0610\ (\%\ \text{asphaltenes}) \qquad R^2 = 0.791$$

Accepting 0.375 inches as an allowable rut depth, the asphaltene content should be 15 percent, which would be within the range indicated from the Sisko & Brunstrum data.

While not directly related to the work of Sisko & Brunstrum, it can also be reported that Anderson et al. (1983) and Kandhal related low temperature cracking with low values of penetration; i.e., less than 43 by Anderson and less than 29 by Kandhal.

Some anomalies in comparisons of asphalt properties can be found in comparing results from Sisko & Brunstrum to information provided by Way (1978). Penetration values in nine sections out of 10, with no cracking, ranged from 5 to 23 and there was no rutting in one section with a penetration of 75. Poor comparisons were also indicated for viscosities at 140•F; e.g., no cracking with viscosity of 123,000 and over 1 million poises.

In evaluating the results of quantitative analysis, as reported herein, it must be noted that specific values for thresholds or ranges associated with performance, are based on best-fit regressions. The actual variation in values, associated with specific correlations, can be very large, as reported by Hugo and Kennedy. Further, the standard error of estimate for the various correlations reported from the Sisko & Brunstrum data is approximately 1.0. Thus, if one were to select threshold values it would be necessary to select those values related to a crack or rut depth rating of 2 rather than from 2 to 3. Therefore, considerable caution must be exercised when trying to apply or interpret results from the analysis of field data.

5.3 INTERPRETATION OF RELATIONSHIPS BETWEEN THE PROPERTIES OF THE ASPHALT-AGGREGATE SYSTEM AND PAVEMENT PERFORMANCE.

Information in the literature on asphalt-aggregate systems (mixtures) is primarily identified with either the Marshall or Hveem methods used to measure mixture properties. Since the SHRP research is being directed toward more fundamental properties, no effort has been made in the literature review to evaluate the relevancy of current specifications or to attempt to associate resulting physical properties to performance.

For purposes of this report, the properties of asphalt-aggregate systems considered relevant to the SHRP research objectives would include measures of stiffness at various temperature and times of loading, tensile strength, fatigue properties, and shear strength. For both types of measurements, there are currently in the research literature, a variety of procedures which can be used to measure specific properties; e.g., complex modulus, dynamic creep, shear modulus, etc.

Qualitative Relationships

The research literature, based primarily on laboratory studies, would suggest the following relationships between asphalt-aggregate stiffness properties and predicted pavement performance.

Fatigue Cracking - increasing damage potential with increasing stiffness for pavements with 7 inches or less of asphalt concrete and decreasing damage potential for pavements with more than 7 inches of asphalt concrete.

Rutting - decreasing damage potential with increasing stiffness.

Low Temperature Cracking - increasing damage with increasing stiffness.

No qualitative relationships have been reported regarding chemical fractions except that these properties can influence the properties of the asphalt and consequently, influence the properties of the asphalt-aggregate system to some degree.

Quantitative Relationships

In order to develop quantitative relationships between asphalt aggregate systems and pavement performance, an effort has been made to locate information from field trials. Table 5.9 from Monismith et al. (1987) summarizes information reported by investigators in the United States, Canada, France and South Africa. The specific values of stiffness associated with the various types of distress are reported to illustrate that such properties can be related to pavement performance. The confidence interval associated with each value is unknown and can be expected to be relatively large.

5.4 CONCLUSIONS

1. The source of crude used to produce asphalt does influence the in-service performance and durability of asphalt and asphaltic mixtures. While not as definitive, the method of refining, particularly by propane deasphalting, produces asphalts with different properties when compared with vacuum distillation.

2. Chemical fractions influence the aging characteristics of asphalt and the performance of asphalt-aggregate systems; however, at the present time the association is more qualitative than quantitative and depends on the method of characterization. Further, the effect on aging can be confounded by other factors such as air voids and possibly asphalt content.

3. The in-service rheological properties of asphalt influence the performance of asphalt-concrete systems; however, at the present time the association is more qualitative than quantitative.

Table 5.9 Mix design considerations based on expected performance (Monismith et al. 1987)

Mix Property	Distress Mode			
	Fatigue	Rutting	Thermal Cracking (Fracture)	Raveling
Stiffness	mix stiffness, S_{mix} at t = 0.01s, 20•C (68•F) 1. Thin asphalt bound layer 700 to 1400 MPa (100,000 to 200,000 psi) 2. Thick asphalt-bound layer>3500 MPa (500,000 psi) mix stiffness, S_{mix} at t = 300 s, 0•C, used by LCPC as one of the parameters to estimate fatigue response (Bonnot, 1986)	mix stiffness, S_{mix} at 40•C (104•F) 1. $S_{mix} \geq$ 80 MPa (12,000 psi) at t = 100 min and σ_0 = 0.2 MPa (30 psi) (Viljoen et al. 1981) 2. $S_{mix} \geq$ 50 to 65 MPa (7500 to 10,000 psi) at t = 60 min and σ_0 = 0.1 MPa (Kronfuss et al. 1984) 3. $S_{mix} \geq$ 135 MPa (20,000 psi) at t = 60 min and σ_0 = 0.2 MPa (30 psi) (Finn et al. 1983) 4. S_{mix} correspon-ding to Hveem "S" value of 35 estima-ted at 34,500 psi (Finn et al. 1983)	mix stiffness, S_{mix}, at lowest temperature expected in-situ 1. S_{mix} < 20 GPa (3 x 10^6 psi) at t = 30 min (Gaw et al. 1974)	1. asphalt stiffness, S_{asp}, at $t = 10^{-3}$ s less than some value for low road temperature. 2. strain at break for mixture $(\epsilon_b)_{mix}$ at t = 0.01 s larger than some value (for example, 10^{-1}) for expected low pave-ment temperature (Dormon, 1969)

Additional relationships are reported in Chapter 5 herein.

Asphalt properties, as presently measured, do not lend themselves easily to analytical models for use in predicting performance.

4. There is a need to use standard or baseline test procedures to measure asphalt properties as part of field investigations to evaluate the role of asphalt to pavement performance.

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

6. Information relating asphalt properties to fatigue cracking and rutting is inconclusive; however, the association with low temperature cracking is more conclusive indicating that asphalt stiffness and asphalt-aggregate mixture stiffness is related to the occurrence of this type of cracking.

7. A number of analytical methods have been proposed to predict low temperature cracking; however, the reliability of these programs has not been verified and, in fact, efforts to do so have not resulted in any consensus as to their applicability.

8. Most of the controlled and uncontrolled projects have included both laboratory and field aging studies. From the field studies aging can generally be divided into three categories: (i) during construction, (ii) up to 2 years beyond construction and (iii) after 2 years. The greatest change in consistency occurs in the construction (mixing) period and decreases to a relatively slow rate after 2 years.

9. The ability to reproduce in-situ field aging in the laboratory has generally been poor, particularly as regards the ability to predict long-term aging. Some reasonably good qualitative comparisons have been reported; however, a shift factor would be required to achieve a quantitative comparison and, in all probability, such a factor would be dependent on a particular asphalt, air void content and environment.

10. Based on interpretations of test data from the Ste. Anne Test Road and the Ontario studies, the critical binder stiffness for low temperature cracking could be 14,000 psi at 1,800 seconds loading time or 2,000 psi at 20,000 seconds loading time. This wide range is indicative, at least in part, of the lack of a standard procedure for testing. It is also affected by the method of analysis used by researchers.

11. The Michigan studies, based on 12 years of observations, concluded that cracking would be

reduced if the asphalt viscosity at 140°F was less than 12,000 poises. California (Zaca-Wigmore) indicated a value of 20 megapoise at 77°F and 0.05 sec^{-1} for 10 percent cracking. This type of difference in reporting illustrates the problems associated with not having a baseline for testing asphalt or reporting performance.

12. The P.I. (Pfeiffer and van Doormaal) of asphalts at the Ste. Anne Test Road were -1.5 (acceptable performance) and -2.7 (unacceptable performance). However, based on the report by Puzinauskas (1979), very few asphalts produced in the United States or Canada would have P.I. values approaching -2.7 (9 of 68 tested).

13. McLeod has concluded that low temperature transverse cracking is likely to occur when the modulus of stiffness of a pavement reaches 1 million psi at a pavement depth of 2 inches due to any critical combination of chilling to a low pavement temperature, hardness of the asphalt cement and other controlling factors. This value is based on Van der Poel's nomograph for a loading time of 20,000 seconds and applies to "well designed paving mixtures" with a C_v value of 0.88 (14.5% VMA and 3% air voids). McLeod bases this conclusion on the performance of the Ontario and Ste. Anne test roads, laboratory studies and his observations of the service behavior of asphalt pavements in Canada, the USA and Norway.

14. One uncontrolled study (Sisko & Brunstrum 1968) reported a limiting complex modulus, of 20 x 10^{-4} (dynes/cm^2) associated with slight cracking and 34 x 10^{-4} to 410 x 10^{-4} dynes/cm^2 for severe cracking. Values were normalized to 10^{-8} cps at 20°F and tests were made using a Weissenberg rheogoniometer. This work by Sisko and Brunstrum was not generally appreciated at the time of publication. However, these investigators were unique in that they attempted to relate complex modulus of asphalt to pavement performance.

15. Isolating the role of asphalt or asphalt-aggregate systems is especially difficult for fatigue cracking because of the strong inter-relationships between this particular type of distress and pavement thickness, asphalt concrete thickness, asphalt concrete stiffness, temperature cycles and potentially the water sensitivity of the asphalt concrete. Laboratory studies and mechanistic analysis have been used to relate these factors. Of particular interest to A-003A is the indication that fatigue cracking is related to the stiffness (representative of a time and temperature dependent elastic modulus) of the asphaltic mixture and of the asphalt binder. This relationship is also dependent on the method used to measure fatigue characteristics, i.e., stress controlled or strain controlled.

Finn et al. (1978) summarized a series of case studies designed to evaluate the comparison between predicted fatigue cracking and observed cracking. The actual investigation was a joint effort by Monismith and Caltrans (Monismith et al., 1967). The results of this study indicated that there was a correspondence between observed and predicted performance, although a shift factor would be required. Similar results were reported by Finn et al. (1977).

Using these concepts, no critical or threshold value could be identified. By using analytical methods, the designer would be able to predict the design life for a particular section or to adjust the design to be compatible with asphalt stiffness.

16. Monismith and Tayebali (1988) summarize suggested limiting values of mixture stiffness at temperatures on the order of 100°F to 104°F (38°C to 40°C) as follows:

Researchers	Limiting Stiffness
Viljoen and Meadows (National Institute for Transport and Road Research 1981)	80 MPa (12,000 psi) at t = 100 min. with confining pressure of 0.2 MPa (30 psi)
Kronfuss et al. 1984	50 - 60 MPa (7,500-10,000 psi) at t = 60 min. with confining pressure of 0.1 MPa (15 psi)
Monismith et al. 1987	135 MPa (20,000 psi) at t = 60 min. with confining pressure of 0.2 MPa (30 psi)

Considering differences in loading time and confining pressure these values, which were developed independently, are considered to be comparable.

17. Information developed from both controlled and uncontrolled studies have produced valuable and useful information relative to the influence of asphalt properties to pavement performance. Individually, these projects have contributed to development of new specifications; e.g., Zaca-Wigmore and Pennsylvania, and to a better understanding of factors influencing performance, e.g., Ste. Anne, Montana, Zaca-Wigmore, Pennsylvania, etc. Every project, in a way related to specific project objectives, has contributed to a better understanding of pavement performance. However, in order to translate results from field studies to a more national or international basis, there will need to be baseline tests which can be used for comparison.

18. To date, it was not considered feasible to develop a ranking system for asphalt beyond that shown in Table 5.1.

An overall summary of the information would suggest that: (i) asphalt properties do influence pavement performance, and (ii) the ability to arrive at a reliable quantitative and consistent association between asphalt properties, asphalt-aggregate properties and pavement performance has not as yet been established through field studies.

6.0 BIBLIOGRAPHY

Adams, C. K. & Holmgreen, R. J. (1986). "Asphalt properties and pavement performance." Federal Highway Administration Report FHWA/TX-86-287-4F.

Altgelt, K. H. (1965). "Fractionation of asphaltenes by gel permeation chromatography." Journal of Applied Polymer Science 9:3389-3393.

Altgelt, K. H. (1970). "GPC in structural research of asphalt." Bitumen, Teere, Asphalte, Peche 20:475.

Altgelt, K. H. & Gouw, T. H. (1975). "Chromatography of heavy petroleum fractions." Advances in Chromatography (ed. J.C. Giddings, et al.) Marcel Dekker, Inc., New York, NY :71-175.

Altgelt, K. H. & Harle, O. L. (1975). "The effect of asphaltenes on asphalt viscosity." Industrial and Engineering Chemistry - Prod. Research and Development 14(4):240-256.

Altgelt, K. H. & Hirsch, E. (1970). "GPC separation and integrated structural analysis of petroleum heavy ends." Separation Science 5(6):855.

Anderson, D. A. & Dukatz, E. L. (1980). "Asphalt properties and composition: 1950-1980." Association of Asphalt Paving Technologists 49:1-29.

Anderson, D. A. & Dukatz, E. L. (1985). "Fingerprinting versus field performance of paving grade asphalts." Pennsylvania Transportation Institute Report FHWA-RD-84-095:132.

Anderson, D. A., Petersen, J. C. & Christensen, D. W. (1986). "Variations in asphalt cements and their effects on performance of asphalt concrete mixtures." Association of Asphalt Paving Technologists 55:250-268.

Anderson, D. I., Peterson, D. E. & Wiley, M. L. (1976). "Characteristics of asphalts as related to the performance of flexible pavements." Utah Department of Transportation Report UDOT-MR-76-6.

Anderson, D. I., Peterson, D. E. & Wiley, M. L. (1976). "Evaluation of viscosity-graded asphalt cements in Utah." Transportation Research Record 595:9-17.

Anderson, K. O. (1982). "Use of field observations and distress prediction techniques to improve low temperature pavement cracking performance." Proc., Paving in Cold Areas, TP-3869 1:79-98.

Anderson, K. O. & Epps, J. A. (1983). "Asphalt concrete factors related to pavement cracking in West Texas." Association of Asphalt Paving Technologists 52:151-197.

Anderson, K. O. & Hahn, W. P. (1968). "Design and evaluation of asphalt concrete with respect to thermal cracking." Association of Asphalt Paving Technologists 37:1-31.

Anderson, K. O., Shields, B. P. & Dacyszyn, J. M. (1966). "Cracking of asphalt pavements due to thermal effects." Association of Asphalt Paving Technologists 35:247-262.

Anderson, K. O. & Shields, B. P. (1971). "Some Alberta experience with penetration-graded asphalt cements having differing viscosities at 140°F. "Highway Research Record 350:15-25.

The Asphalt Institute. (1981). "Cause and prevention of stripping in asphalt pavements." Asphalt Institute Educational Series, College Park, Maryland.

Asphalt Institute Ad Hoc Committee. (1981). "Design techniques to minimize low-temperature asphalt pavement transverse cracking." Asphalt Institute Research Report Report RR-81-1.

ASTM. (1966). "Grading of paving asphalts by viscosity at 140°F versus penetration at 77°F." ASTM Special Technical Publication 424.

Baladi, G. Y. & Harichandran, R. S. (1987). "Asphalt mix design and the indirect test: A new horizon. "ASTM Special Technical Publication, 1041:86-105.

Bardet, J., Nahas, N. C., Eckmann, B. & Siano, D. B. (1990). "Polymer modified asphalts for high performance hot mix pavement binders." AAPT Preprint :1-8.

Benson, P. E. (1976). "Low temperature transverse cracking of asphalt concrete pavements in Central and West Texas." Federal Highway Administration Report FHWA/RD/M-0352.

Blanchard, L. & Maachar, M. A. (1983). "A comparative study of bitumens originating from different sources." Proc., Canadian Technical Asphalt Association :260-281.

Boduszynski, M. M. (1981). "Asphaltenes in petroleum asphalts, composition and formation. "Chemistry of Asphaltenes (ed. J.W. Bunger, N.C. Li), American Chemical Society, Washington, D.C. Advances in Chemical Series Volume 195, Chapter 7.

Boduszynski, M. M. (1984). "Relationship between solvent-derived and compound-class fractions in coal-derived distillates and vacuum still bottoms." Fuel 63:93-100.

Boduszynski, M. M., McCoy, J. F. & Latham, D. R. (1980). "Asphaltenes, Where are you?" Association of Asphalt Paving Technologists 49:123-143.

Bohn, A. O., Stubstad, R. N., Sorensen, A. & Simonson, P. (1977). "Rheological properties of road materials and their effect on the behavior of a pavement section tested in a climate controlled linear track road testing machine." Association of Asphalt Paving Technologists 46:105-131.

Bonnot, J. (1986). "Asphalt aggregate mixtures." Transportation Research Record 1096:42-51.

Brown, A. B. (1958). "Possible effects of hardening on pavement life." Association of Asphalt Paving Technologists 27:171-176.

Brown, A. B., Sparks, J. W. & Larsen, O. (1957). "Rate of change of softening point, penetration, and ductility of asphalt in bituminous pavement." Association of Asphalt Paving Technologists 26:66-81.

Brown, A. B., Sparks, J. W. & Smith, F. M. (1957). "Steric hardening of asphalts." Association of Asphalt Paving Technologists 26:486-494.

Brown, A. B. & Sparks, J. W. (1958). "Viscoelastic properties of a penetration grade paving asphalt at winter temperatures." Association of Asphalt Paving Technologists 27:35-51.

Brown, S. F. (1989). "Improving asphalt technology for roads." Municipal Engineering 6.

Brown, S. F., Cooper, K. E. & Preston, J. N. (1989). "Development of a new procedure for bituminous mix design." Prepared for the 4th Eurobitume Symposium, Madrid, Spain.

Brown, S. F. & Cooper, K. E. (1984). "The mechanical properties of bituminous materials for road bases and base courses." Association of Asphalt Paving Technologists 53:415-439.

Bruce, B. (1987). "Asphalt as a variable - Big Timber Test Sections." Association of Asphalt Paving Technologists 56:688-710.

Brule, B., Ramond, G. & Such, C. (1986). "Relationships between composition, structure and properties of road asphalts: State of research at the French Public Works Central Laboratory. "Transportation Research Record 1096:22-34.

Brunstrum, L. C., Ott, L. E., Sisko, A. W., Speer, T. L., Wilke, R. A. & Evans, J. V. (1965). "Relation of asphalt pavement variables to performance." Highway Research Record 104:1-17.

Bullin, J. A., Glover, C. J., Button, J. W., Davison, R. R., Donaldson, G. R., Hilavinka, M. W. & Philip, C. V. (1987). "Characterization of asphalts using gel permeation chromatography and other methods." Texas Transportation Institute.

Burgess, R. A., Kopvillem, O. & Young, F. D. (1971). "Ste. Anne Test Road - Relationships between predicted fracture temperatures and low temperature pavement performance." Association of Asphalt Paving Technologists 40:148-193.

Burgess, R. A., Kopvillem, O. & Young, F. D. (1972). "Ste. Anne Test Road - Flexible pavement design to resist low temperature cracking." Proc., Third International Conference on Structural Design of Asphalt Pavements, London, England 3:27-40.

Busching, H. W., Burati, J. L. Jr. & Amirkhanian, S. N. (1986). "An investigation of stripping in asphalt concrete in South Carolina." Federal Highway Administration Report FHWA-SC-86-02.

Bussow, C. (1936). "Notes on a method of recovering bitumen from paving materials." Association of Asphalt Paving Technologists 5:160-164.

Button, J. W. & Epps, J. A. (1985). "Identifying tender asphalt mixtures in the laboratory." Transportation Research Record 1034:20-26.

Button, J. W., Epps, J. A. & Little, D. N. (1980). "Influence of asphalt temperature susceptibility on pavement construction and performance - Interim report." Texas Transportation Institute Project 1-20.

Button, J. W., Epps, J. A. & Little, D. N. (1981). "Variability in temperature susceptibility of asphalt cement." National Cooperative Highway Reseach Program - Research Results Digest 130.

Button, J. W., Little, D. N., Gallaway, B. M. & Epps, J. A. (1983). "Influence of asphalt temperature susceptability on pavement construction and performance." National Cooperative Highway Research Program Report 268.

Button, J. W., Little, D. N., Kim, Y. & Ahmed, J. (1987). "Mechanistic evaluation of selected asphalt additives." Association of Asphalt Paving Technologists 56:62-90.

Button, J. W. & Perdomo, D. (1990). "Influence of aggregate on rutting in asphalt concrete pavements." Preprint Transportation Research Board 69th Annual Meeting.

Bynum, D. & Traxler, R. N. (1970). "Gel permeation chromatography data on asphalts before and after service in pavements." Association of Asphalt Paving Technologists (See discussion by J.C. Petersen) 39:683 - 702.

Carpenter, S. H. & VanDam, T. (1985). "Evaluation of low-temperature performance of asphalt cements." Cold Regions Research & Engineering Laboratory.

Chaiken, B., Halstead, W. J. & Olsen, R. E. (1962). "Application of infrared spectroscopy to bituminous mineral filler evaluation." Highway Research Board Bulletin 329:38-47.

Chari, C. T., Ruth, B. E., Tia, M. & Page, G. C. (1990). "Evaluation of age hardening on asphalts and mixtures." Association of Asphalt Paving Technologists Preprint :1-72.

Charles, R. (1987). "Asphalt concrete mix design in the Caribbean." ASTM Special Technical Publication 1041:115-136.

Chipperfield, E. H., Duthie, J. L. & Girdler, R. B. (1970). "Asphalt characteristics in relation to road performance." Association of Asphalt Paving Technologists 39:575-613.

Chipperfield, E. H. & Welch, T. R. (1967). "Studies on the relationships between properties of road bitumens and their service performance." Association of Asphalt Paving Technologists 36:421-488.

Chollar, B. H., Boone, J. G., Cuff, W. E. & Bailey, E. F. (1985). "Chemical and physical characterization of binder materials." Public Roads 49(1):7-12.

Christison, J. T., Murray, D. W. & Anderson, K. O. (1972). "Stress prediction and low temperature fracture susceptibility of asphalt concrete pavements." Association of Asphalt Paving Technologists 41:494-523.

Clark, M. F. & Culley, R. W. (1976). "Evaluation of air-blown asphalts to reduce thermal cracking of asphalt pavements." Association of Asphalt Paving Technologists 45:530-545.

Clark, M. F. & Culley, R. W. (1974). "Use of air-blown asphalt cements to reduce thermal cracking." Proc., Canadian Technical Asphalt Association 19:5-23.

Clark, R. G. (1958). "Practical results of asphalt hardening on pavement life." Association of Asphalt Paving Technologists 27:196- 208.

Cold Regions Research & Engineering Laboratory. (1988). "Summary report on low temperature cracking of asphalt pavements." Cold Regions Research & Engineering Laboratory, Hanover, New Hampshire.

Coons, R. F. & Wright, P. H. (1968). "An investigation of the hardening of asphalt recovered from pavements of various ages." Association of Asphalt Paving Technologists 37:510-528.

Cooper, K. E. & Brown, S. F. (Unknown). "Development of simple apparatus for the measurement of the mechanical properties of asphalt mixes." University of Nottingham, England.

Coplantz, J. S., Epps, J. A. & Quilici, L. (1987). "Antistrip additives: Background for a field performance study." Transportation Research Record 1115:1-11.

Corbett, L. W. (1969). "Composition of asphalt based on generic fractionation using solvent deasphalting, elution-adsorption chromatography and densimetric characterization." Annual of Chemistry 41:576-579.

Corbett, L. W. & Merz, R. E. (1975). "Asphalt binder hardening in the Michigan test road after 18 years of service." Transportation Research Record 544:27-34.

Corbett, L. W. & Swarbrick, R. E. (1958). "Clues to asphalt composition." Association of Asphalt Paving Technologists 27:107-122.

Craus, J., Ishai, I. & Sides, A. (1978). "Some physico-chemical aspects of the effect and the role of the filler in bituminous paving mixtures." Technion-Israel Institute of Technology, Transportation Research Institute Report 78-10.

Crawford, R. A. & Anderson, D. W. (1968). "Pavement cracking as related to asphalt hardness." South Dakota Department of Highways.

Csanyi, L. H. (1962). "Functions of fillers in bituminous mixes." Highway Research Board Bulletin 329:1-5.

Culley, R. W. (1978). "Performance of full-depth asphalt concrete with "soft" asphalt." Saskatchewan Highways and Transportation Technical Report 27.

Culley, R. W. (1969). "Relationship between hardening of asphalt cements and transverse cracking of pavements in Saskatchewan." Association of Asphalt Paving Technologists 38:629-645.

Davis, T. C. & Petersen, J. C. (1967). "An inverse GLC study of asphalts used in the Zaca-Wigmore experimental road test." Association of Asphalt Paving Technologists 36:1-15.

Dawley, C. B., Hogewiede, B. L. & Anderson, K. O. (1990). "Mitigation of instability rutting of asphalt concrete pavements in Lethbridge, Alberta, Canada." Association of Asphalt Paving Technologists Preprint :1-19.

Deme, I. J. & Young, F. (1987). "Ste. Anne test road revisited 20 years later." Proc., Canadian Technical Asphalt Association :254-283.

Dempsey, B. J., Ingersoll, J., Johnson, J. C. & Shahin, M. Y. (1980). "Asphalt concrete for cold regions." Cold Regions Research & Engineering Laboratory Report 80-5.

DeYoung, C. E. (1968). "Variability in asphalt paving materials." Association of Asphalt Paving Technologists 37:322-340.

Dhalaan, M. A., Khan, S. S., Farouki, T. A. & Farwana, T. A. (1990). "A comprehensive approach to the rutting problem in the Kingdom of Saudi Arabia." Association of Asphalt Paving Technologists Preprint :1-46.

Dickinson, E. J. (1982). "The performance of thin bituminous pavement surfacings in Australia." Proc. 11th Australian Road Research Board Conference 11(3):35-51.

Dillard, J. H. (1955). "Comparison of density of Marshall specimens and pavement cores." Association of Asphalt Paving Technologists 24:178-232.

Dormon, G. M. (1969). "Some observations on the properties of bitumen and their relation to performance in practice and specifications." Proc., First Conference on Asphalt Pavements for Southern Africa, Durban, South Africa.

Dow, A. W. (1936). "The Wash test in differentiating between aggregates or Colprovia paving mixtures." Association of Asphalt Paving Technologists 5:75-78.

Dunning, R. L. & Meeks, J. L. (1968). "Comparison between air blowing and accelerated aging." Association of Asphalt Paving Technologists 37:529-541.

Epps, J. A., Button, J. W. & Gallaway, B. M. (1983). "Paving with asphalt cements produced in the 1980's." National Cooperative Highway Research Program Report 269:1-28.

Fenijn, J. & Krooshof, R. C. (1970). "The sliding plate rheometer - A simple instrument for meausring the visco-elastic behavior of bitumens and related substances in absolute units." Proc. Canadian Technical Asphalt Association.

Fink, D. F. (1958). "Research studies and procedures." Association of Asphalt Paving Technologists 27:176-183.

Finn, F. N. (1973). "Relation between cracking and performance." Highway Research Board Special Report 140:93-97.

Finn, F. N. & Epps, J. A. (1981). "Selection of asphalt grade to optimize performance of asphalt surfaced pavements." Texas Transportation Institute Report 214-27.

Finn, F. N., Monismith, C. L. & Vallerga, B. A. (1967). "Factors involved in the design of asphaltic pavement surfaces." National Cooperative Highway Reseach Program Report 39.

Finn, F. N., Monismith, C. L. & Markevich, N. J. (1983). "Pavement performance and asphalt concrete mix design." Association of Asphalt Paving Technologists 52:121-150.

Finn, F. N., Nair, K. & Hilliard, J. (1976). "Minimizing cracking of asphalt concrete pavement." Association of Asphalt Paving Technologists 45:492-526.

Finn, F. N., Nair, K. & Hilliard, J. M. (1978). "Minimizing premature cracking in asphaltic concrete pavement." National Cooperative Highway Research Program Report 195.

Finn, F. N., Saraf, C., Kulkarni, K., Nair, K., Smith, W. & Abdullah, A. (1977). "The use of distress prediction subsystems for the design of pavement structures." Proc., Fourth International Conference on Structural Design of Asphalt Pavements I:3-38.

Ford, M. C., Jr. (1988). "Pavement desification related to asphalt mix characteristics." Transportation Research Record 1178:9-15.

Fromm, H. J. (1974). "The mechanisms of asphalt stripping from aggregate surfaces." Association of Asphalt Paving Technologists 43:191-223.

Fromm, H. J. & Phang, W. A. (1972). "A study of transverse cracking of bituminous pavements." Association of Asphalt Paving Technologists 41:383-423.

Fromm, H. J. & Phang, W. A. (1971). "Temperature susceptibility control in asphalt cement specifications." Transportation Research Record 350:30-45.

Fukuda, T., Murai, S. & Takahashi, H. (1984). "Study on reflection cracks by photoelastic experiments." Proceedings Paving in Cold Areas 2136:383-400.

Gallaway, B. M. (1959). "Factors relating chemical composition and rheological properties of paving asphalts with durability." Association of Asphalt Paving Technologists 28:280-293.

Garrick, N. W. & Biskur, R. R. (1990). "A classification system for interpreting GPC profiles of asphalts." Association of Asphalt Paving Technologists Preprint.

Garrick, N. W. & Biskur, R. R. (1990). "The effects of asphalt composition and physical properties on the indirect tensile strength of asphalt concrete mixes." Preprint Transportation Research Board 69th Annual Meeting.

Garrick, N. W. & Wood, L. E. (1986). "The relationship between HP-GPC data and the rheological properties of asphalts." Transportation Research Board 65th Annual Meeting.

Garrison, W. A. (1966). "Three-year evaluation of Shell Avenue test road." Highway Research Record 117:71-97.

Gaw, W. J. (Unknown). "Design techniques to minimize low temperature pavement transverse cracking." Douglas Oil Co. of California.

Gaw, W. J. (1976). "Mesurement and prediction of asphalt stiffness and their use in developing specifications to control low temperature transverse cracking." ASTM Special Technical Publication 628:57-67.

Gaw, W. J., Burgess, R. A., Young, F. D. & Fromm, H. J. (1976). "A Laboratory and field evaluation of air blown, low viscosity waxy asphalts from western Canada crudes." Proc.,Twenty-First Annual Conference of Canadian Technical Asphalt Association :193-223.

Gaw, W. J., Burgess, R. A. & Young, F. D. (1974). "Ste. Anne test road - Road performance after five years and laboratory predictions of road performance." Proc., Canadian Technical Asphalt Association XIX.

Gietz, R. H. & Lamb, D. R. (1968). "Age-hardening of asphalt cement and its relationship to lateral cracking of asphaltic concete." Association of Asphalt Paving Technologists 37:141-158.

Glover, C. L., Bullin, J. A., Button, J. W., Davison, R. R., Donaldson, G. R., Hlavinka, M. W. & Philip, C. V. (1987). "Characterization of asphalt using gel permeation chromatography and other methods." Texas Transportation Institute Report FHWA/TX- 87/53+419-1F.

Glover, C. J., Davison, R. R., Bullin, J. A., Button, J. W. & Donaldson, G. R. (1988). "Asphalt cement chemical characterization and performance related properties." Transportation Research Board Preprint.

Glover, C. J., Davison, R. R., Bullin, J. A., Button, J. W. & Donaldson, G. R. (1988). "Chemical characterization of asphalt cement and performance-related properties." Transportation Research Record 1171:71-81.

Goetz, W. H. (1987). "The evolution of asphalt concrete mix design." ASTM Special Technical Publication 1041:5-14.

Goodrich, J. L. (1988). "Asphalt and polymer modified asphalt properties related to the performance of asphalt concrete mixes." Association of Asphalt Paving Technologists 57:116-175. Goodrich, J. L. (1984). "Studies relating asphalt composition to road performance. "Chevron Research, Asphalt Division, U.S.A.

Goodrich, J. L., Goodrich, J. E. & Kari, W. J. (1984). "Asphalt composition tests: Their application and relationship to field performance." Chevron Research Company.

Gotolski, W. H., Ciesielski. S.K., Smith, R. W. & Kofalt, J. A. (1968). "Study of physical factors affecting the durability of asphaltic pavements." Highway Research Record 231:1-23.

Gouw, T. H. (ed.). (1972). "Guide to modern methods of instumental analysis." John Wiley & Sons, Inc., New York, NT :130.

Griffin, R. L., Miles, T. K. & Penther, C. J. (1955). "Microfilm durability test for asphalt." Association of Asphalt Paving Technologists 24:31-62.

Griffin, R. L., Miles, T. K., Penther, C. J. & Simpson, W. C. (1957). "Sliding plate microviscometer for rapid measurement of asphalt viscosity in absolute units." ASTM Special Technical Publication 212.

Haas, R. C. G. (1973). "A method for designing asphalt pavements to minimize low-temperature shrinkage." Asphalt Institute Research Report Report RR-73-1.

Haas, R., Thompson, E., Meyer, F. & Tessier, G. R. (1982). "Summary report on the study of additives and extenders for asphalt cements." Proc., Paving in Cold Areas, TP-3869 2:345-374.

Halstead, W. J. (1984). "Properties of asphalt cemetn used in virginia and their effects on pavement performance, Task 1: Evaluation of asphalt characteristics." Virginia Highway and Transportation Research Council Report VHTRC 84-R48.

Halstead, W. J. (1963). "Relation of asphalt ductility to pavement performance." Public Roads 32(10):221-232.

Halstead, W. J. (1985). "Relation of asphalt chemistry to physical properties and specifications." Virginia Highway and Transportation Research Council Report VHTRC-85-RP2.

Halstead, W. J. (1985). "Relation of asphalt chemistry to physical properties and specifications." Association of Asphalt Paving Technologists 54:91-117.

Halstead, W. J., Rostler, F. S. & White, R. M. (1966). "Properties of highway asphalts - Part III, Influence of chemical composition." Association of Asphalt Paving Technologists 35:91-126.

Hattingh, M. M. (1984). "The fractionation of asphalt." Association of Asphalt Paving Technologists 53:197-215.

Heithaus, J. J. (1960). "Measurement and significance of asphaltene peptization." Symposium on fundamentals of asphalt, Petroleum Division preprints 5:A23-A37.

Heithaus, J. J. & Johnson, R. W. (1958). "A microviscometer study of road asphalt hardening in the field and laboratory." Association of Asphalt Paving Technologists 27:17-34.

Her Majesty's Stationary Office. (1962). "Bituminous materials in road construction." Dept. of Scientific and Industrial Research, Her Majesty's Stationary Office, London, England.

Heukelom, W. (1973). "An improved method of characterizing asphalt bitumens with the aid of their mechanical properties." Association of Asphalt Paving Technologists 42:67-94.

Hicks, R. G., Martin, K. L., Wilson, J. E. & Allen, D. (1987). "Evaluation of asphalt additives: Lava Butte road - Fremont Highway Junction." Prepared for presentation at the 1987 Annual Meeting of the Transportation Research Board.

Hicks, R. G., Martin, K. L. & Wilson, J. E. (1987). "Evaluation of asphalt additives: Lava Butte Road - Fremont Highway Junction." Transportation Research Board 1115:75-88.

Hills, J. F. & Brien, D. (1966). "The fracture bitumens and asphalt mixes by temperature induced stresses." Association of Asphalt Paving Technologists 35:292-309.

Horiguchi, T. & Kasahara, A. (1984). "Applicability of fracture mechanics to reflection cracking." Proceedings Paving in Cold Areas 2136:353-367.

Howard, J. J. (1974). "Asphalt pavement mixture performance evaluation." Idaho Department of Highways Research Project Report 70:52.

Hoyt, D. M., Lytton, R. L. & Roberts, F. L. (1988). "Performance prediction and cost-effectiveness of asphalt rubber concrete in airports." Texas Transportation Institute Report 870399.

Huber, C. F. & Thompson, P. F. (1955). "The function and chemistry of asphalt additives." Association of Asphalt Paving Technologists 24:374-391.

Huber, G. A. & Heiman, G. H. (1987). "Effect of asphalt concrete parameters on rutting performance: A field investigation." Association of Asphalt Paving Technologists 56:33-61.

Hudson, S. B. & Vokac, R. (1962). "Effect of fillers on the Marshall stability of bituminous mixtures." Highway Research Board Bulletin 329:30-37.

Hughes, E. C. & Hardman, H. F. (1951). "Some correlations of asphalt composition with physical properties." Association of Asphalt Paving Technologists 20:1-22.

Hugo, F. & Kennedy, T. W. (1985). "Surface cracking of asphalt mixtures in Southern Africa." Association of Asphalt Paving Technologists 54:454-501.

Hveem, F. N., Zube, E. & Skog, J. (1959). "Progress report on Zaca- Wigmore experimental asphalt test project." ASTM Special Technical Publication 277:3-45.

Ishai, I., Brule, B., Vaniscote, J. C. & Ramond, G. (1988). "Some rheological and physico-chemical aspects of long-term asphalt durability." Association of Asphalt Paving Technologists 57:65-93.

Jacobs, F. A. (1981). "Hot rolled asphalt: Effect of binder properties on resistance to deformation." Transport and Road Research Laboratory Report 1003, United Kingdom.

Jamieson, I. L. & Hattingh, M. M. (1970). "The correlation of chemical and physical properties of bitumens with their road performance." Proc., Australian Road Research Board 5(5):293-324.

Janoo, V. C. (1988). "Performance of soft grade asphalt pavements in cold regions (Draft)." Cold Regions Research & Engineering Laboratory.

Jennings. P.W., Pribanic, J. A., Dawson, K. R. & Bricca, C. E. (1981). "Use of HPLC and NMR spectroscopy to characterize asphaltic materials." Preprints, Division of Petroleum Chemistry, American Chemical Society 26:915-922.

Jennings, P. W., et al. (1977). "Chemical composition of commercial asphalt cement as determined by high pressure liquid chromatography." Federal Highway Administration Report FHWA-MT-7929.

Jennings, P. W., et al. (1980). "High pressure liquid chromatography as a method of measuring asphalt composition." Federal Highway Administration Report FHWA-MT-7930.

Jennings, P. W., Pribanic, J. A. & Dawson, K. R. (1984). "Big timber test section report." Department of Chemistry, Montana State University.

Jennings, P. W., Pribanic, J. A., Dawson, K. R., Koontz, S. B. & Smith, J. A. (1985). "The expanded Montana asphalt quality study using high presure liquid chromatography." Montana Department of Highways Research Program Research Report No. FHWA/MT-85/001.

Jennings, P. W., Pribanic, J. A. S., Smith, J. & Mendes, T. M. (1988). "Predicting the performance of Montana test sections by physical and chemical testing." Transportation Research Record 1171:59-65.

Jennings, P. W., Pribanic, J. A. S. & Dawson, K. R. (1982). "Use of HPLC to determine the effects of various additives and fillers on the characteristics of asphalt." Federal Highway Administration Report FHWA/MT-82/001.

Jennings, P. W., Pribanic, J. A., Dawson, K. R., Koontz, S. B., Spittler. T. & Shane, S. (1985). "Uses of high performance gel permeation chromatography for asphalt analysis." For presentation at the 64th Annual Meeting of the Transportation Research Board.

Jimenez, R. A. (1990). "Methods and treatments to control debonding." Association of Asphalt Paving Technologists Preprint :1-60.

Jones, G. M., Darter, M. I. & Littlefield, G. (1968). "Thermal expansion - construction of asphaltic concrete." Association of Asphalt Paving Technologists 37:56-100.

Kallas, B. F., Puzinauskas, V. P. & Krieger, H. C. (1962). "Mineral fillers in asphalt paving mixtures." Highway Research Board Bulletin 329:6-29.

Kallas. B.F. (1982). "Low-temperature mechanical properties of asphalt concrete." Asphalt Institute Research Report Report RR-82-3.

Kandhal, P. S. (1990). "Design of large stone asphalt mixes to minimize rutting." Preprint Transportation Research Board 69th Annual Meeting.

Kandhal, P. S. (1990). "Large stone asphalt mixes: Design and construction." Association of Asphalt Paving Technologists Preprint :1-31.

Kandhal, P. S. (1976). "Low-temperature ductility in relation to pavement performance." ASTM Special Technical Publication 628:95-106.

Kandhal, P. S., Button, J. W., Davis, R. L., Eusley, E. K., Khosla, N. P. & Puzinouskas, V. P. (1988). "Low temperature properties of paving asphalt cements." Transportation Research Board State of the Art Report 7.

Kandhal, P. S. & Koehler, W. C. (1984). "Significant studies on asphalt durability: Pennsylvania experience." Transportation Research Record 999:41-50.

Kandhal, P. S., Mellott, P. B. & Busso Jr., H. R. (1984). "Durability study of viscosity graded AC-20 asphalts in Pennsylvania - Final Report." Federal Highway Administration Report FHWA/PA-84-009.

Kandhal, P. S., Sandvig, L. D., Koehler, W. C. & Wenger, M. E. (1972). "Asphalt viscosity-related properties of in-service pavements in Pennsylvania." ASTM Special Technical Publication 532:66-78.

Kandhal, P. S., Sandvig, L. D. & Wenger, M. E. (1973). "Shear susceptability of asphalts in relation to pavement performance." Association of Asphalt Paving Technologists 42:99-111.

Kandhal, P. S. & Wenger, M. E. (1975). "Asphalt properties in relation to pavement performance." Transportation Research Record 544:1-13.

Kandhal, P. S. & Wenger, M. E. (1975). "Evaluation of properties of AC-20 asphalt cements." Transportation Research Record 544:56-64.

Kasahara, A. (1984). "Failure behavior of notched asphalt concrete." Proceedings Paving in Cold Areas 2136:369-381.

Kemp, G. R. (1972). "Asphalt durability tests and their relationship to field hardening." ASTM Special Technical Publication 532:100-122.

Kemp, G. R. & Predoehl, N. H. (1981). "A comparison of field and laboratory environments on asphalt durability." Association of Asphalt Paving Technologists 50:492-533.

Kenis Sr., W. J. (1962). "Progress report on changes in asphaltic concrete in service." Highway Research Board Bulletin 333:39-65.

Keyser, J. H. & Ruth, B. E. (1984). "Analysis of asphalt concrete test road sections in the Province of Quebec, Canada." Transportation Research Record 968:54-65.

Khosla, N. P. (1978). "Effect of binder characteristics on tensile properties of bituminous mixtures." Indiana State Highway Commission Joint Highway Research Program Report JHRP-78-16.

Kim, Y. R., Little, D. N. & Benson, F. C. (1990). "Chemical and mechanical evaluation on healing mechanism of asphalt concrete." Association of Asphalt Paving Technologists Preprints :1-43.

Kinnison, R. R. (1986). "Statistical analysis of the expanded Montana asphalt quality study." Desert Research Institute, University of Nevada FHWA Grant No. DTFH 68-86-P-70099.

Koninklijke/Shell Laboratorium. (1972). "The computer programs PONOS and POEL."

Krchma, L. C. (1958). "Relationship of mix design to hardening." Association of Asphalt Paving Technologists 27:186-196.

Krom, C. J. & Dorman, G. M. (1967). "Consideration of a general functional specification for road bitumens on the basis of performance requirements under critical circumstances." Seventh World Petroleum Congress.

Kronfuss, F., Krzemien, R., Nievelt, G. & Putz, P. (1984). "Verformungsfestigkjeit von Asphalten Ermittlung im Kriechtest." Bundesmininsterium fur Bauten and Technik, Strassenforschung, Heft 240, Wien, Austria.

Krutz, N. C. & Stroup-Gardiner, M. (1990). "A relationship between permanent deformation of asphalt concrete and moisture sensitivity." Preprint Transportation Research Board 69th Annual Meeting :1-17.

Kudo, H., Kumagai, S. & Oguri, M. (1984). "Performance and reparation of thermal crackings in asphalt pavements." Proc., Paving in Cold Areas 2136:747-777.

Laforte, M. A. & Keyser, J. H. (1982). "Low temperature cracking of asphalt pavements - A reconsideration." Proceedings, Paving in Cold Areas, Volume 1 TP-3869:45-76.

Langlois, R. (1984). "Reflective cracking in bituminous overlays in Quebec." Proceedings Paving in Cold Areas 2136:321-351.

LeClerc, R. V. & Marshall, T. R. (1969). "A pavement condition rating system and its use." Association of Asphalt Paving Technologists 38:280-295.

LeClerc, R. V. & Walter, J. P. (1976). "Design and performance of asphalt concrete bases in the state of Washington." Transportation Research Circular 177:4-13.

Lee, D. (1973). "Asphalt durability correlation in Iowa." Highway Research Record 468:43-60.

Lee, T., Terrell, R. L. & Mahoney, J. P. (1983). "Measurement of mixing efficiency in pavements recycling." Association of Asphalt Paving Technologists 52:61-87.

Lefebvre, L. M., Dion, A., Langlois, R., Champagne, D. & McLeod, N. W. (1978). "Paving the 385 mile James Bay Access Road with 300/400 penetration asphalt cement." Association of Asphalt Paving Technologists 47:302-343.

Leung, S. C. & Anderson, K. O. (1987). "Evaluation of asphalt cements for low temperature performance." Transportation Research Record 1115:23-32.

Little, D. N., Button, J. W., White, R. M., Ensley, E. K., Kim, Y. & Ahmed, S. J. (1987). "Investigation of asphalt additives (Final report)." Federal Highway Administration Report FHWA-RD-87/001.

Lottman, R. P. (1978). "Predicting moisture-induced damage to asphaltic concrete." National Cooperative Highway Research Program Report 192.

Lottman, R. P. (1982). "Predicting moisture-induced damage to asphaltic concrete: Field evaluation." National Cooperative Highway Research Program 246.

Lottman, R. P. & Breje, S. (1990). "Moisture damage cutoff rations (Specifications) for asphalt concrete." Preprint Transportation Research Board Annual Meeting, 1990 :2-17.

Lucas, A. G. (1977). "Modified bitumens for aspahlt wearing courses." Shell Bitumen Review 55.

Mack, C. (1958). "Physical aspects of hardening of paving asphalts." Association of Asphalt Paving Technologists 27:183-185.

Mahboub, K. & Little, D. N. Jr. (1990). "An improved asphalt concrete mix design procedure." Association of Asphalt Paving Technologists Preprint :1-54.

Mahboub, K. & Little, D. N. (1990). "Overview of a rational asphalt concrete mix design." Preprint Transportation Research Board :2-41.

Majidzadeh, K. & Brovold, F. N. (1968). "State of the Art: Effect of water on bitumen-aggregate mixtures." Highway Research Board Special Report 98.

Marks, V. J. & Huisman, C. L. (1985). "Reducing the adverse effects of transverse cracking." Transportation Research Record 1034:80-86.

Martin, K. L., Davison, R. R., Glover, C. J. & Bullin, J. A. (1990). "Asphalt aging in Texas test sections." Preprint Transportation Research Board 69th Annual Meeting :1-39.

Marvillet, J. (1975). "Influence of asphalt composition on its rheological behavior." Association of Asphalt Paving Technologists 44:416-443.

McHattie, R. L. (1981). "Asphalt concrete properties and performance in Alaska." Federal Highway Administration Report FHWA-AK-RD-82-2.

McLeod, N.W. & Robertson, W. D. (1978). "Test data from three Ontario test roads after 15 years of service." Proceedings of the Canadian Technical Asphalt Associaton 23.

McLeod, N. W. (1972). "A 4 year survey of low temperature transverse pavement cracking on three Ontario test roads." Association of Asphalt Paving Technologists 41:424-493.

McLeod, N. W. (1976). "Asphalt cements: Pen-Vis number and its application to moduli of stiffness." ASTM Journal of Testing and Evaluation Vol. 4, No. 4.

McLeod, N. W. (1988). "Discussion. " Association of Asphalt Paving Technologists 57:304-319.

McLeod, N. W. (1987). "Employing paving asphalt temperature susceptibility in the structural design of asphalt pavements." Proc., 6th International Conference on the Structural Design of Asphalt Pavements Vol. I:248-271.

McLeod, N. W. (1970). "Influence of hardness of asphalt concrete on low temperature transverse pavement cracking." McAsphalt Engineering Services, Canada.

Monin, J. C. & Vignat, A. (1984). "Evidence of asphaltene associations via hydrogen bonding (was provided) by Staric Exclusion Chromatograph (SEC)." Rev. Inst. Fr. Pet. 39(6):821-827.

Monismith, C. L. (1959). "Effect of temperature on the flexibility characteristics of asphaltic paving mixtures." ASTM Special Technical Publication 277:89-108.

Monismith, C. L. (1958). "Flexibility characteristics of asphaltic paving mixtures." Association of Asphalt Paving Technologists 27:74-104.

Monismith, C. L., Finn, F. N. & Vallerga, B. A. (1987). "A comprehensive asphalt concrete mixture design system." ASTM Special Technical Publication 1041:39-71.

Monismith, C. L., Finn, F. N., Ahlborn, G. & Markevich, N. J. (1987). "A general analytically based approach to the design of asphalt concrete pavements." Proc., 6th International Conference on Structural Design of Asphalt Pavements I:344-365.

Monismith, C. L., Kasianchuk, D. A. & Epps, J. A. (1967). "Asphalt mixture behaviour in repeated flexure: A study of an in-service pavement near Morro Bay, California." Report No. TE67-4, Dept. of Civil Engineering, Inst. of Transportation and Traffic Engineering, University of California, Berkeley.

Monismith, C. L & Tayebali, A. A. (1988). "Permanent deformation (Rutting) considerations in asphalt concrete pavement sections." Association of Asphalt Paving Technologists 57:414-463.

Montana State Department of Highways special provision. (1983). "Requirements for the gel permeation mode (HP-GPC) of asphalt cement."

NCHRP. (1981). "Variablility in temperature susceptibility of asphalt cement." NCHRP Research Results Digest 130:5.

Noureldin, M. S. & Manke, P. G. (1978A). "Study of transverse cracking in flexible highway pavements in Oklahoma." Transportation Research Record 695:28-33.

Noureldin, M. S. & Manke, P. G. (1978B). "Application of the stiffness concept to transverse pavament cracking in Oklahoma." Association of Asphalt Paving Technologists 47:270-280.

Oliensis, G. L. (1957). "The Oliensis spot test as a quality test." Association of Asphalt Paving Technologists 26:82-125.

Olsen, R. E., Welborn, J. Y. & Vallerga, B. A. (1969). "Pavement evaluation - To evaluate the effect of asphalt aging on condition." Association of Asphalt Paving Technologists 38:317-327.

Organization of Economic Cooperation and Development. (1984). "Road Binders and Energy Savings." Prepared by The Organization for Economic Cooperation and Development, Scientific Expert Group, Road Transport Research Programme, Paris, France.

Page, G. C., Murphy, K. H. & Ruth, B. E. (1985). "Asphalt binder hardening - Causes and effects." Association of Asphalt Paving Technologists 54:140-167.

Page, G., Murphy, K. H. & Ruth, B. E. (1986). "Asphalt concrete mixture production problems related to light constituents in asphalts." Association of Asphalt Paving Technologists 55:277-313.

Pagen, C. A. (1968). "Size and themorheological relationships of asphaltic concrete." Association of Asphalt Paving Technologists 37:228-257.

Parr, W. K. & Serafin, P. J. (1959). "Four years test trends on the Michigan bituminous experimental road." Association of Asphalt Paving Technologists 28:295-315.

Parr, W. K., Serafin, P. J. & Humphries, T. (1955). "Michigan state highway experimental bituminous concrete construction project." Association of Asphalt Paving Technologists 24:125-177.

Pauls, J. T., & Halstead, W. J. (1958). "Progressive alterations in a sheet asphalt pavement over a long period of service." Association of Asphalt Paving Technologists 27:123-154.

Peters, R. J. (1975). "Compositional considerations of asphalt for durability improvement." Transportation Research Record 544:46-55.

Petersen, J. C. (1984). "Chemical composition of asphalt as related to asphalt durability - state of the art." Transportation Research Record 999:13-30.

Petersen, J. C. (1984). "Chemical composition of asphalt as related to asphalt durability - state of the art." Federal Highway Administration Report RD-84/047.

Petersen, J. C. (1974). "Effects of chemical composition on the properties of asphalt and asphalt-aggregate mixtures." Western Research Institute, Laramie, Wyoming.

Pfeiffer, J. P. H. (editor). (1950). "The properties of asphaltic bitumen." Koninklijke/Shell Laboratorium.

Pfeiffer, J. P. & van Doormaal, P. M. (1936). "The rheological properties of asphalt bitumen." Journal of the Institute of Petroleum Technology 22:414.

Phang W.A & Stott, G. M. (1981). "Pavement condition and performance observations - Brampton test road." Association of Asphalt Paving Technologists 50:345-378.

Plancher, H., Green, E. L. & Petersen, J. C. (1977). "Paving asphalts: Reduction of oxidative hardening of asphalts by treatment with hydrated lime - A mechanistic study (Interim report)." Federal Highway Administration Report FHWA-RD-77-147.

Plummer, M. A. & Zimmerman, C. C. (1984). "Asphalt quality and yield predictions from crude oil analyses." Association of Asphalt Paving Technologists 53:138-159.

Potts, C. F., Schweyer, H. E. & Smith, L. L. (1973). "An analysis of certain variables related to field performance of asphaltic pavements." Proceedings of the Association of Asphalt Paving Technologists 73:564-588.

Price, R. P. & Burati, J. L. Jr. (1990). "Predicting laboratory results for modified asphalts using HP-GPC." Association of Asphalt Paving Technologists Preprint :1-33.

Puzinauskas, V. P. (Unknown). "Modification of asphalt cement and asphalt concrete with polyethylene additive - Novaphalt process." The Asphalt Institute.

Puzinauskas, V. P. (1979). "Properties of asphalt cements." Association of Asphalt Paving Technologists 48:646-710.

Rader, L. F. (1936). "Correlation of low temperature tests with resistance to cracking of sheet asphalt pavements." Association of Asphalt Paving Technologists 5:29-43.

Roberts, J. R. & Gotloski, W. H. (1975). "Paving asphalt properties and pavement durability." Transportation Research Board 544:14-26.

Rostler, F. S. & White, R. M. (1962). "Composition and changes in composition of highway asphalts, 85-100 penetration grade." Association of Asphalt Paving Technologists 31:35-89.

Rostler, F. S. & White, R. M. (1959). "Influence of chemical composition of asphalts on performance particularly durability." ASTM Special Technical Publication 277:64-88.

Rust, F. C. & Hugo, F. (1990). "Towards performance-related design criteria and specifications for modified binders." AAPT Preprint :1-7.

Ruth, B. E., Lia, M. & Badu-Tweneboah, K. (1987). "The role of asphalts in rational mix design and pavement performance." ASTM Special Technical Publication 1041:72-85.

Sandvig, L. D. & Kofalt, J. A. (1968). "Penetration and viscosity functions in bituminous concrete." Association of Asphalt Paving Technologists 37:584-621.

Sato, K., Fukute, T. & Hachiya, Y. (1982). "Thermally induced cracking due to hardening of asphalt concrete in airport asphalt pavements." Proc., Paving in Cold Areas, TP-3869 1:101-125.

Scherocman, J. A. (1987). "International state-of-the-art colloquium on low temperature asphalt pavement cracking." Cold Regions Research & Engineering Laboratory.

Schmidt, R. J. (1975). "Use of ASTM tests to predict low temperature stiffness of asphalt mixes." Transportation Research Record 544:35-45.

Schmidt, R. J. & Graf, P. D. (1972). "The effect of water on the resilient modulus of asphalt treated mixes." Association of Asphalt Paving Technologists 41:118-162.

Schmidt, R. J., Painter, L. J., Skog, J. B. & Puzinauskas, V. P. (1968). "The precision of tests made on asphalt residua from thin film exposures - A cooperative study." Association of Asphalt Paving Technologists 37:476-509.

Schmidt, R. J. & Santucci, L. E. (1969). "The effect of asphalt properties on the fatigue cracking of asphalt concrete on the Zaca-Wigmore test project." Association of Asphalt Paving Technologists 38:39-64.

Serafin, P. J. (1958). "Laboratory and field control to minimize hardening of paving asphalts." Association of Asphalt Paving Technologists 27:209-216.

Serafin, P. J., Kole, L. L. & Chiritz, A. P. (1967). "Michigan bituminous experimental road - Final Report." Association of Asphalt Paving Technologists 36:582-614.

Shah, S. C. (1978). "Asphalt cement consistency." Transportation Research Record 695:1-7.

Shahin, M. Y. & McCullough, B. E. (1973). "Prediction of low-temperature and thermal-fatigue cracking in flexible pavements." Center for Highway Research.

Sherman, G. (1982). "Minimizing reflection cracking of pavement overlays." National Cooperative Highway Research Program Synthesis Report 92.

Shook, J. F. (1974). "San Diego County experimental base project." Asphalt Institute Research Report RR-74-1.

Simpson, W. C., Griffin, R. L. & Miles, T. K. (1959). "Correlation of the microfilm durability test with field hardening observed in the Zaca-Wigmore experimental project." ASTM Special Technical Publication 277:52-63.

Sisko, A. W. & Brunstrum, L. C. (1969). "Relation of asphalt rheological properties to pavement durability." National Cooperative Highway Research Program Report 67.

Sisko, A. W. & Brunstrum, L. C. (1968). "The rheological properties of asphalts in relation to durability and pavement performance." Association of Asphalt Paving Technologists 37:448-475.

Skog, J. B. (1981). "Effect of changes in asphalt properties on pavement performance: Zaca-Wigmore test road." Association of Asphalt Paving Technologists 50:538-560.

Skog, J. (1959). "Results of co-operative test series on asphalts from the Zaca-Wigmore experimental project." ASTM Special Technical Publication 277:46-51.

Skog, J. (1967). "'Setting' and durability studies on paving grade asphalts." Association of Asphalt Paving Technologists 36:387-420.

Skog, J., Beaton, J. L. & Zube, E. (1966). "Durability of paving asphalt, Part II." State of Califonia Transportation Agnecy Research Report M&R 633134-2.

Skog, J. B., Matthews, J. A., Mann. G.W. & Roberts, D. V. (1978). "Statewide flexible pavement performance and deflection study." Federal Highway Administration Report FHWA-CA-TL-78-28.

Snyder, L. R. & Kirkland, J. J. (1979). "Introduction to modern liquid chromatography." Second Edition, John Wiley & Sons, Inc., New York, NY.

Speer, et al. (1963). "Asphalt viscosity as related to pavement performance." Association of Asphalt Paving Technologists 32:236-247.

Speer, T. L. & Kietzman, J. H. (1962). "Control of asphalt pavement rutting with asbestos fiber." Highway Research Board Bulletin 329:64-89.

Stanley, A. F. (1974). "Asphalt quality evaluation." Idaho Transportation Department Research Project 62:52.

Stock, A. F. & Button, J. (1987). "Relationship among parameters derived from high performance liquid chromotography, physical properties, and pavement performance in Texas." Transportation Research Record 1115:33-45.

Strategic Highway Research Program. (1986). "Strategic Highway Research Program - Research Plans - Final Report." Transportation Research Board, Washington, D.C.

Stroup-Gardiner, M. & Epps, J. A. (1987). "Four variables that affect the performance of lime in asphalt aggregate mixtures." Transportation Research Record 1115:12-22.

Such, C., Baluja-Santos, C. & Brule, B. (1979). "Characterization of road asphalt by chromatographic techniques (GPC and HPLC)." Journal of Liquid Chromatography 2:437-453.

Sugawara, T., Kubo, H. & Moriyoshi, A. (1982). "Low temperature cracking of asphalt pavements." Proc., Paving in Cold Areas, TP-3869 1:3-42.

Sugawara, T. & Moriyoshi, A. (1984). "Thermal fracture of bituminous mixtures." Proceedings Paving in Cold Areas 2136:289-320.

Tada, H., Makita, M., Iijima, T. & Sakamoto, H. (1982). "Modified asphalts in Japan and research into the development of new rubberized asphalts." Proc., Paving in Cold Areas, TP-3869 2:307-342.

Takallou, H. B. (1984). "Stripping of asphalt pavements: State-of-the-art." Oregon State University, Transportation Research Institute Report TRR 84-6.

Taylor, M. A. & Khosla, N. P. (1983). "Stripping of asphalt pavements: State of the art." Transportation Research Record 911:150-158.

Thelen, E. (1958). "Surface energy and adhesion properties in asphalt-aggregate systems." Highway Research Board Bulletin 192:63-74.

Thenoux, G. (1987). "Chemical characterization of asphalt cements using chromatographic techniques: Relative to rheological properties and field performance." Oregon State University, Transportation Research Institute TRR 87-21.

Thenoux, G., Bell, C. A., Wilson, J. E., Eakin, D. & Schroeder, M. (1988). "Evaluation of asphalt properties and their realtionship to pavement performance." Federal Highway Administration Report FHWA-OR-RD-88-02-1.

Thenoux, G., Bell, C. A. & Wilson, J. E. (1988). "Evaluation of physical and fractional properties of asphalt and their interrelationship." Transportation Research Record 1171:82-97.

Thenoux, G., Bell, C. A., Wilson, J. E., Eakin, D. & Schroeder, M. (1988). "Experiences with the Corbett-Swarbrick procedure for separation of asphalt into four generic fractions." Transportation Research Record 1171:66-70.

Tons, E. & Henault G. (1962). "Evaluation of microaggregates by Smith triaxial test." Highway Reseach Board Bulletin 329:48-63.

"Transverse cracking of bituminous pavements - Final Report" (1976). South Dakota Department of Transportation.

Traxler, R. N. (1967a). "Changes in asphalt cements during preparation, laying and service of bituminous pavements." Association of Asphalt Paving Technologists 36:541-561.

Traxler, R. N. (1967b). "Hardening of asphalt cements during service in pavements, 1963-66 program." Texas Transportation Institute Progress Report 11, 2-8-59-9.

Tseng, K. & Lytton, R. L. (1990). "Fatigue damage properties of asphaltic concrete pavements." Preprint Transportation Research Board 69th Annual Meeting 890201:1-43.

Tuffour, Y. A. & Ishai. (1990). "The diffusion model and asphalt age-hardening." Association of Asphalt Paving Technologists Preprint :1-24.

Tunnicliff, P. G. & Root, R. E. (1984). "Use of antistripping additives in asphalt concrete mixtures." National Cooperative Highway Research Program Report 274.

Valkering, C. P., Lancon, D. J., de Hilster, E. & Stoker, D. A. (1990). "Rutting resistance of asphalt mixes containing non-coventional and polymer-modified binders." Association of Asphalt Paving Technologists Preprint.

Vallerga, B. A. (1955). "On asphalt pavement performance." Association of Asphalt Paving Technologists 24:79-102.

Vallerga, B. A. & Halstead, W. J. (1971). "Effects of field aging on fundamental properties of paving asphalts." Highway Research Record 361:71-92.

Vallerga, B. A., White, R. M. & Rostler, K. S. (1970). "Changes in fundamental properties of asphalts during service in pavements - Final report." US Bureau of Public Roads.

Van de Loo, P. J. (1974). "Creep testing, a simple tool to judge asphalt mix stability." Association of Asphalt Paving Technologists 43:253-284.

Van der Poel, C. (1954). "A general system describing the viscoelastic properties of bitumens and its relationship to routine test data." Journal of Applied Chemistry 4:Unknown.

Venable, R. L., Petersen, J. C., Robertson, R. E. & Plancher, H. (1983). "Investigation of factors affecting asphalt pavement recycling and asphalt compatibility." Laramie Energy Technology Center report DOE/LC/RI-83-4.

Viljoen, A. W. & Meadows, K. (1981). "The Creep Test - A mix design tool to rank asphalt mixes in terms of their resistance to permanent deformation under heavy traffic." Technical Note: TP/36/81, National Institute for Transport and Road Research, Pretoria, South Africa.

Von Quintus, H. L., Scherocman, J. & Hughes, C. (1987). "Asphalt aggregate mixtures analysis system : Philosophy of the concept." ASTM Special Technical Publication 1041:15-38.

Watanabe, T. & Himeno, T. (1984). "Estimation of fatigue life for asphalt pavements." Proceedings Paving in Cold Areas Technical Memorandum of PWRI No. 2136:61-82.

Way, G. (1978). "Asphalt properties and their relationship to pavement performance in Arizona." Association of Asphalt Paving Technologists 47:49-72.

Welborn, J. Y. (1984). "Physical properties as related to asphalt durability: State of the art." Transportation Research Record 999:31-36.

Welborn, J. Y. (1979). "Relationship of asphalt cement properties to pavement durability." National Cooperative Highway Research Program Synthesis 59.

Welborn, J. Y., Halstead, W. J., Vallerga, B. A. & Puzinauskas, V. P. (1970). Association of Asphalt Paving Technologists Discussion 39:611-612.

Welborn, J. Y., Oglio, E. R. & Zenewitz, J. A. (1966). "A study of viscosity-graded asphalt cements." Procedings of the Association of Asphalt Paving Technologists 35:19-60.

Williams, L. (1964). "Regionalization of freeze-thaw activity." Ann. Assoc. Am. Geographers 54:597.

Yandell, W. O. & Smith, R. B. (1987). "Toward maximum performance mix design for each situation." ASTM Special Technical Publication 1041:106-114.

Young, F. D., Deme, I., Burgess, R. A. & Kopvillem. O. (1969). "Ste. Anne test road - Construction summary and road performance after two years service." Proc., Canadian Technical Asphalt Association XIV.

Zenewitz, J. A. & Tran, K. T. (1987). "A further statistical treatment of the expanded Montana asphalt quality study." Public Roads Vol. 51, No. 3:72-81.

Zenewitz, J. A. & Welborn, J. Y. (1975). "Predicting asphalt performance." Federal Highway Administration Report FHWA-RD-7133.

Zube, E. & Skog, J. (1969). "Final report on the Zaca-Wigmore asphalt test road." Association of Asphalt Paving Technologists 38:1-39.

APPENDIX A

CONTROLLED TEST ROADS

This appendix contains supplementary tables and figures for the test roads discussed in Chapter 3. They are organized in the same order as the test roads in Chapter 3.

LIST OF TABLES

LIST OF FIGURES

Table A.1 Physical properties of paving grade asphalts at Zaca-Wigmore. (Hveem et al. 1959)

Asphalt	Specification	Paving Period	Cleveland Open Cup Flash Point	Pensky-Martins Closed Flash Point	Original Penetration	Penetration Ratio	Viscosity 275 F	Xylene Equivalent	Ductility, 77 F	Softening Point	Solubility CCl_4	Standard Loss		Thin Film*	
												Loss per cent	Per cent of Original Penetration	Loss, per cent	Per cent of Original Penetration
A	Special provision	1	505	450	222	26	61	10–15	100+	100	99.9	0.14	92.8	0.65	56.3
A-2		2	500	435	217	25	60	10–15	100+	97	99.9	0.20	88.5	0.70	52.5
B-1	Special provision	1A	470	405	269	31	78	25–30	92	97	90.9	0.38	91.8	1.35	41.2
B-2		2	430	410	221	33	110	25–30	100+	103	99.9	0.37	81.2	2.10	36.2
C	1954 standard specification	1	480	430	252	24	60	10–15	100+	98	99.9	0.36	89.3	0.75	50.0
C-1		1A	480	430	240	26	64	15–20	100+	93	99.9	0.18	90.0	0.80	49.6
C-2		2	475	395	242	26	65	15–20	100+	98	99.9	0.21	89.7	1.10	42.6
D	Special provision	1	480	350	243	31	61	15–20	100+	98	99.8	0.27	92.2	1.40	48.5
D-2		2	485	420	220	28	60	10–15	100+	100	99.8	0.32	89.6	0.85	50.9
E	1954 standard specification	1	300	325	231	43	117	30–35	100+	99	99.9	0.98	81.0	4.45	27.7
E-2		2	350	200	235	43	125	25–30	100+	100	99.8	1.79	62.5	5.65	18.7
F	Special provision	1	435	300	225	33	102	25–30	100+	102	90.9	0.47	84.8	2.25	34.7
G	Special provision	1	470	410	242	37	93	30–35	100+	101	90.8	0.28	89.2	1.60	35.1
G-2		2	450	400	210	36	103	30–35	100+	101	99.9	0.43	88.1	2.20	39.7
H	Special provision	1	485	430	227	20	73	25–30	100+	101	90.9	0.26	90.4	0.70	48.5
H-2		2	480	415	211	29	73	25–30	100+	100	99.9	0.29	83.5	1.05	45.1
I-2	No specification	2	470	440	204	35	65	35–40	100+	102	90.9	0.25	81.5	1.00	39.5
J	Special provision	1	600	405	245	32	119	25–30	100+	101	90.9	0.11	92.0	0.35	52.3

* CDH Materials Manual, Vol. I, Test Method 337.

Table A.2 Characteristics of asphalt binders used in Ste. Anne mixes. (Burgess et al. 1971)

Characteristics	LVA 150-200	HVA 150-200	LVA 300-400
PENETRATION			
At 77 F, 100 g, 5 sec	192	159	313
At 39.2 F, 100 g, 5 sec	10	14	14
At 39.2 F, 200 g, 60 sec	38	55	52
VISCOSITY			
At 275 F, cs	110	225	86
At 140 F, Poises	253	591	141
At 60 F, Poises at 0.05 sec^{-1}	3.0×10^6	3.0×10^6	1.5×10^6
At 39.2 F, Poises at 0.05 sec^{-1}	1.4×10^8	6.2×10^7	5.9×10^7
Softening Point, F, R & B	119	109	119
Ductility at 77 F, cm	44	150+	27
Lewis Thin Film			
Loss on Heating, % Wt	0.073	0.40	0.10
% Retained Pen., at 77 F	44.1	47.0	44.3
Ductility at 77 F, cm	88	150+	53

Table A.3 Calculation of thermally induced stress values for 150-200 LVA field aged Ste. Anne Binder. (Burgess et al. 1971)

T, C	ΔT, C	S_{bit} kg/cm²	$\Delta\epsilon_{TH} \times 10^{-3}$	$\Delta\sigma_{TH}$ kg/cm²	$\Sigma\sigma_{TH}$ kg/cm²	σ_{Br} kg/cm²
0						
	5	3	1	.003		
-5					.003	8
	5	1.2×10^1	1	.012		
-10					.015	15
	5	4.3×10^1	1	.043		
-15					.058	33
	5	1.9×10^2	1	.19		
-20					.248	52
	5	7.0×10^2	1	.7		
-25					.948	47
	5	1.8×10^3	1	1.8		
-30					2.748	41
	5	4.2×10^3	1	4.2		
-35					6.948	39
	5	8.5×10^3	1	8.5		
-40					15.448	38
	5	13.0×10^3	1	13		
-45					28.448	37
	5					
-50		15.0×10^3	1	15	43.448	36

S_{bit} = Obtained by Nomograph, Reference (9), at 1/2-hour loading time.
$\Delta\epsilon_{TH} = \Delta T \times \alpha = 5 \times 2 \times 10^{-4} = 1 \times 10^{-3}$
$\Delta\sigma_{TH} = \Delta\epsilon_{TH} \times S_{bit}$
σ_{Br} = After Heukelom, Reference (11)
Cooling rate 10 C/Hour.

Table A.4a Properties of original AC-20 cements in Pennsylvania. (Kandhal et al. 1984)

	Asphalt Type					
	T-1	T-2	T-3	T-4	T-5	T-6
Penetration @ 39.2 F (4 C) 100 g, 5 sec.	2.0	7.4	6.2	6.7	3.4	7.5
Penetration @ 60 F (15.6 C) 100 g., 5 sec.	11.2	25.0	24.5	23.0	16.0	29.0
Penetration @ 77 F (25 C) 100 g, 5 sec.	42	64	72	65	54	80
Viscosity @ 140 F (60 C), poises	2,710	2,284	1,764	1,705	1,759	1,982
Viscosity @ 275 F (135 C), centistokes	420	402	393	355	356	406
Ductility @ 60 F (15.6 C), 5 cm/min, cm.	150+	29	150+	117	150+	150+
Softening Point (R & B), C	50.6	50.0	48.9	50.0	51.1	49.4
TFO Residue						
Penetration @ 77 F (25 C), 100 g, 5 sec.	26	38	45	38	37	44
Viscosity @ 140 F (60 C), poises	5,501	6,835	3,982	4,694	3,248	5,721
Viscosity @ 275 F (135 C), centistokes	563	569	556	527	464	575
Ductility @ 39.2 F (4 C), 1 cm/min, cm.	3.5	3.5	4.6	5.2	8.6	12.4
Ductility @ 60 F (15.6 C), 5 cm/min, cm.	11.6	7.0	95.2	12.8	90.6	33.0

Table A.4b Properties of AC-20 cements recovered just after construction. (Kandhal et al. 1984).

TEST	ASPHALT TYPE					
	T-1	T-2	T-3	T-4	T-5	T-6
Penetration @ 4 C (39.2 F) 100 g., 5 sec.	1.5	4.5	4.5	4.0	2.0	5.8
Penetration @ 15.6 C (60 F) 100 g., 5 sec.	7	17	16	13	9	20
Penetration @ 25 C (77 F) 100 g., 5 sec.	24	40	43	34	29	49
Viscosity @ 60 C (140 F), pascal-seconds	552.6	572.9	378.9	382.9	401.9	461.1
Viscosity @ 135 C (275 F), mm^2/s	565	569	526	487	488	576
Softening Point (R & B), C	56.7	53.3	53.9	53.3	54.4	53.9
Ductility @ 4 C (39.2 F) 1 cm/min, cm	0.2	4.6	13.9	5.9	0.6	14.9
Ductility @ 15.6 C (60 F), 5 cm/min, cm	8.3	7.2	48.5	10.0	15.5	34.0
Ductility @ 25 C (77 F), 5 cm/min, cm	150+	80	150+	150+	150+	150+
PI (pen/pen)	-2.24	-0.80	-0.99	-0.65	-2.03	-0.64
PVN (Pen-Vis Number)	-1.13	-0.68	-0.72	-1.03	-1.16	-0.47

Note: 1 Pa.s = 10 poises, and 1 mm^2/s = 1 centistoke.

Table A.5 Transverse cracking survey in Pennsylvania, pavements T-1 and T-5. (Kandhal et al. 1984)

Pavement	Sta. to Sta.	Month & Year	No. of Transverse Cracks Full (F)	Half (H)	Part (P)	Cracking Index (500 ft.) $(F+\frac{1}{2}H+\frac{1}{4}P)$
T-1	205+00 to 215+00	Oct. 1977	5	41	102	51
		May 1978	6	44	164	69
		May 1979	7	45	184	76
T-5	125+00 to 135+00	Oct. 1977	11	26	58	38
		May 1978	14	28	88	50
		May 1979	21	35	64	54

Table A.6 **Pavement performance evaluation in Pennsylvania after six years. (Kandhal et al. 1984)**

OBSERVATIONS	ASPHALT TYPE					
	T-1	T-2	T-3	T-4	T-5	T-6
Loss of Fines (matrix)	slight to moderate	slight	slight	slight	slight	none to slight
Ravelling (loss of particle ¼ inch or larger)	moderate	slight	none to slight	slight	slight to moderate	slight
Transverse Cracking	very severe	slight	none	slight to moderate	very severe	slight to moderate
Longitudinal Cracking	very severe[a]	moderate	none to slight	slight to moderate	severe[a]	slight
Overall Rating Number	18.4	29.4	36.2	30.5	20.8	31.7

[a] mostly block cracking resulting from low temperature shrinkage.

Table A.7 Transverse cracking survey in Pennsylvania, August 1981. (Kandhal et al. 1984)

TEST PAVEMENT	Sta. to Sta.	NO. of Transverse Cracks			Cracking Index (500 ft) ($F+\frac{1}{2}H+\frac{1}{4}P$)
		Full (F)	Half (H)	Part (P)	
T-1	205+00 to 215+00	5	67	215	92
T-2	185+00 to 190+00	0	2	32	9
T-3	165+00 to 170+00	0	0	0	0
T-4	145+00 to 150+00	0	0	49	12
T-5	125+00 to 135+00	24	36	90	64
T-6	105+00 to 110+00	0	5	18	7

Table A.8 Properties of recovered AC-20 asphalt cements in Pennsylvania after 20 months. (Kandhal et al. 1984)

TEST	ASPHALT TYPE T-1	T-2	T-3	T-4	T-5	T-6
Penetration @ 4C (39.2F) 100 g, 5 sec.	3.0	5.0	5.0	5.7	2.7	5.0
Penetration @ 15.6C (60F) 100 g, 5 sec.	7.0	13.0	14.1	13.0	8.0	13.0
Penetration @ 25C (77F) 100 g, 5 sec.	19.0	25.2	36.0	31.0	20.6	29.3
Viscosity @ 60C (140F), pascal-seconds	847.4	2,273.8	557.0	799.8	816.6	1,487.1
Viscosity @ 135C (275F) mm^2/s	690	892	625	641	655	890
Softening Point R&B, C	58.9	64.7	57.1	61.7	59.4	62.2
Ductility @ 4C (39.2F) ½ cm/min, cm	1.5	1.0	5.0	5.5	4.1	4.8
Ductility @ 15.6C (60F) 5 cm/min, cm	1.5	3.4	12.2	4.5	4.4	5.6
Ductility @ 25C (77F) 5 cm/min, cm	150+	76	150+	47.5	125.1	42.6
PI (pen/pen)	+0.36	+1.22	-0.12	+0.93	-0.32	+0.60
PVN (Pen-Vis Number)	-1.07	-0.54	-0.65	-0.76	-1.07	-0.4

Note: 1 Pa.s = 10 poises, and 1 mm^2/s = 1 centistoke.

Table A.9 Properties of recovered asphalts in Pennsylvania after six years, 1976. (Kandhal et al. 1984)

TEST	ASPHALT TYPE					
	T-1	T-2	T-3	T-4	T-5	T-5
Penetration at 77 F, 100 g, 5 sec	15	26	35	25	22	35
Viscosity at 140 F, poises	13,339	20,556	7,422	14,418	6,495	11,263
Viscosity at 275 F, centistokes	815	858	721	781	583	815
Ductility at 60 F 5 cm/min, cm	1.2	4.5	14.0	5.0	4.0	11.2

NOTE: 1 poise = 0.1 Pa·S, and 1 centistoke - 1 mm^2/s

Table A.10 Standard data on bitumens used in the French (RN7) and German (B2) trials. (Chipperfield & Welch 1967)

Bitumen	No. 1		No. 2		No. 3		No. 4		No. 5		No. 7		No. 8		No. 9	
Road Trial	RN7	B2	RN7	B2	RN7	B2	RN7	B2	RN7	B2	RN7	B2	RN7	B2	RN7	B2
Kinematic viscosity @ 300°F cSt	301	252	164	173	276	259	206	205	198	184	198	210	173	163	141	163
Kinematic viscosity @ 250°F cSt	1,220	922	641	684	1,080	1,020	811	794	789	705	782	794	672	614	518	614
Kinematic viscosity @ 210°F cSt	5,550	4,010	2,790	3,090	4,670	4,570	3,550	3,470	3,440	3,120	3,460	3,610	2,560	2,640	2,170	2,600
Viscosity @ 70°C (158 F) cSt	93,500	52,800	40,200	46,300	69,500	64,100	51,300	48,000	53,500	52,000	56,200	53,300	48,800	39,900	29,200	36,200
Viscosity @ 60°C (140 F) cSt	344,700	183,300	156,100	172,700	290,600	228,900	224,000	171,100	214,300	164,400	225,200	194,600	196,800	143,000	109,700	136,200
Penetration																
@ 25°C, 100g, 5 sec mm/10	82	82	96	81	86	87	85	80	85	82	74	79	74	80	85	77
@ 15°C, 100g, 5 sec mm/10	34	27	32	26	31	31	32	26	32	29	27	27	29	26	29	26
@ 10°C, 100g, 5 sec mm/10	22	16	18	15	18	18	19	15	19	17	16	16	18	15	17	15
Softening Point ASTM °C	50.5	49.0	45.5	50.0	49.0	50.0	48.0	50.0	48.0	47.0	49.0	48.0	49.0	48.5	48.0	49.5
Apparent Viscosity																
@ 35°C and .05/sec x10,000 p	0.25	0.21	0.10	0.13	0.20	0.21	0.18	0.18	0.18	0.19	0.34	0.23	0.20	0.15	0.21	0.18
@ 25°C and .05/sec x10,000 p	2.15	1.60	0.91	1.05	1.28	1.42	1.90	1.87	1.56	1.58	2.31	2.10	1.70	1.60	1.94	1.78
@ 15°C and .05/sec x10,000 p	14.50	18.30	15.80	16.10	18.20	18.00	21.90	18.40	15.30	14.70	20.10	19.40	18.80	17.30	15.80	20.70
Fraass breaking point C	-20	-19	-19	-17	-17	-21	-20	-17	-17	-19	-19	-18	-16.5	-14	-19.5	-17
Ductility @ 25°C (15 cm/min) cm	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100
Ductility @ 4°C (15 cm/min) cm	6.0	5.0	9.0	8.0	5.0	7.0	5.0	6.0	5.0	5.0	5.0	5.0	5.0	5.0	4.0	4.0
Ductility @ 4°C (1 cm/min) cm	11.0	10.5	>100	>100	11.0	12.5	10.0	9.0	9.0	8.5	7.0	7.0	8.0	7.0	6.0	6.0
Thin Film Oven Test (0.25" film)																
Loss % weight	0.10	<0.05	0.10	<0.05	0.20	0.05	<0.05	nil	0.20	0.10	<0.05	nil	0.25	0.20	<0.05	nil
Retained penetration @ 25°C mm/10	52	50	67	52	55	49	50	43	54	53	43	44	45	43	52	42
Ductility @ 25°C cm	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100
Flash Point (COC) F	520	610	535	560	560	575	570	590	520	570	560	620	510	565	580	605
Solubility in carbon disulfide,% wt	99.95	99.95	99.95	99.95	100.0	99.95	100.0	100.0	100.0	100.0	99.95	100.0	99.90	99.80	99.90	99.90
Specific Gravity @ 25°C	1.025	1.025	1.025	1.025	1.027	1.027	0.124	1.024	1.034	1.033	1.017	1.016	1.029	1.026	1.014	1.015
nC_7 Asphaltenes (1P 143) % wt	15.9	10.2	12.1	10.5	11.9	12.5	13.2	11.6	15.5	14.5	12.3	11.5	14.6	13.3	11.7	9.9
Wax (DIN 1995) % wt	1.4	1.8	0.5	0.6	1.5	1.6	2.0	1.6	1.7	1.8	1.8	1.7	1.7	1.8	2.2	2.2
Acidity (1P 213B) mg KOH g	0.4	0.3	1.6	2.5	0.6	0.4	0.6	0.4	0.5	0.3	0.5	0.3	0.4	0.2	0.3	0.2

Table A.11 Oakland research data on bitumens used in the French trial (RN7). (Chipperfield & Welch 1967)

BP Road Trial Bitumen No.		1	2	3	4	5	7	8	9
Composition									
A	% wt	26·1	18·6	23·0	22·5	26·0	22·6	25·2	21·7
N	% wt	16·6	23·9	19·0	20·0	17·2	21·4	17·7	20·9
A_1	% wt	22·7	22·6	25·3	20·6	19·9	18·5	18·8	16·7
A_2	% wt	25·4	23·0	23·7	22·9	24·9	22·8	25·5	23·7
P	% wt	9·2	11·9	9·0	14·0	12·0	14·7	12·8	17·0
$\frac{N+A_1}{P+A_2}$		1·14	1·33	1·35	1·10	1·01	1·06	0·95	0·92
Durability rating		II	III	III	II	II	II	I	I
Wax		+ RT*	+ ice	+ RT	+ RT	+ RT	÷ RT	+ RT	+ RT
Refractive index 25°C (saturated hydrocarbon)		1·4809	1·4853	1·4802	1·4805	1·4804	1·4805	1·4812	1·479
Molecular weight (Fraction A)		4,080	3,100	4,650	4,330	3,930	4,080	3,820	3,630
Pellet Abrasion at 77°F									
Mix	Loss % wt	0·5	1·1	4·9	3·5	1·0	2·8	0·7	0·3
Aged for 3 days	Loss % wt	0·6	3·0	15·0	4·7	5·0	20·0	2·4	0·3
Aged for 7 days	Loss % wt	4·0	7·0	24·0	7·0	6·0	20·0	4·4	0·7
Average (mix and 7 days)	Loss % wt	2·2	4·1	14·0	5·3	3·5	11·0	2·6	0·5

* RT=room temperature

Table A.12 California Division of Highways data on bitumens used in the French trial (RN7). (Chipperfield & Welch 1967)

BP Road Trial Bitumen No.		Proposed Specification	1	2	3	4	5	7	8	9
Penetration at 25°C	mm/10	—	78	90	71	80	78	68	76	86
Flash point PMCT	°F	475 minimum	495	525	415	485	520	535	490	540
Stain No.		10 maximum	5	2	5	6	5	6	6	6
Solubility in CCl_4	% wt	99·9 minimum	99·9	99·9	99·9	99·9	99·9	99·9	99·9	99·9
Rolling Thin Film Test (325°F for 75 minutes)										
Residue Tests										
Ductility at 25°C	cm	75 minimum	100+	100+	100−	100+	100+	100+	100+	100+
Viscosity at 140°F	poise	4,000—6,000	7,792	2,920	7,325	5,509	4,198	7,509	4,164	2,768
,, ,, 275°F	cSt	425—800	834	420	582	834	495	600	426	354
Durability Test (RTFT+24 hours at 100°C)										
Residue Tests										
Viscosity at 77°F and 5×10^{-2} sec^{-1}	$\times 10^6$p	25 maximum	14	16	20	21	12	25	18	13
,, ,, 77°F and 10^{-3} sec^{-1}	$\times 10^6$p	60 maximum	41	26	59	83	36	104	53	40
Microductility	mm	10 minimum	8	50	6	6	9	3	6	7

Table A.13a Characteristics of bitumens used in the French trial (RN7)-Bureau of Public Roads data. (Chipperfield & Welch 1967)

Road Trial Bitumen No.	BPR No.	Penetration 100 g 5 sec			Ductility				Viscosity			Specific Gravity 77°/77°F	Weight Loss Thin Film Oven Test
					1 cm/min	5 cm/min			$0{\cdot}05\ sec^{-1}$				
		45°F	60°F	77°F	45°F	45°F	60°F	77°F	77°F	140°F	275°F		
					cm	cm	cm	cm	MP	poise	cS		%
1	B-3267	13	29	80	24	9	98	171	1·77	2936	570	1·010	0·22
2	B-3268	11	28	91	250+	135	250+	209	1·06	1428	291	1·021	0·05
3	B-3269	10	24	70	31	7	139	211	1·84	2464	575	1·023	0·15
4	B-3270	10	25	79	32	7	198	202	1·40	1666	395	1·018	0·02
5	B-3271	10	26	76	19	7	144	182	1·39	2182	375	1·032	0·19
7	B-3272	9	21	67	15	6	66	229	2·18	1902	394	1·013	0·12
8	B-3273	10	26	73	24	6	115	194	2·18	1629	322	1·026	0·22
9	B-3274	10	25	81	15	10	55	141	0·802	1062	257	1·013	0·03

Table A.13b Characteristics of thin film oven test residues from bitumens used on the French trial (RN7), Bureau of Public Roads data. (Chipperfield & Welch 1967)

Road Trial Bitumen No.	BPR No.	Penetration 100 g. 5 sec.			Ductility 5 cm/min.			Viscosity 0·05 sec⁻¹ rate of shear			Viscosity		Shear Susceptibility*	
		45°F	60°F	77°F	45°F	60°F	77°F	45°F	60°F	77°F	140°F	275°F	45°F	60°F
					cm	cm	cm	MP	MP	MP	poise	cS		
1	B-3267	11	22	53	5	22	94	87·9	29·3	5·10	8107	884	0·448	0·327
2	B-3268	11	21	59	14	150+	225+	141	22·5	2·70	2785	411	0·368	0·087
3	B-3269	9	19	45	5	32	161	141	32·5	5·15	6362	802	0·434	0·239
4	B-3270	10	19	49	4	21	225+	147	33·5	6·04	4627	569	0·517	0·305
5	B-3271	10	22	50	5	40	216	110	30·7	3·96	3939	516	0·442	0·248
7	B-3272	8	17	42	5	10	94	154	50·4	9·21	6706	603	0·509	0·398
8	B-3273	10	19	48	4	19	171	117	—	5·05	4397	449	0·490	—
9	B-3274	10	21	51	4	11	122	111	—	4·07	2927	359	0·507	—

* Tangent of log viscosity-log rate of shear curve between 0·05 sec^{-1} and 0·10 sec^{-1} rates of shear.

Table A.14 Broad chemical composition of bitumens recovered from the French RN7 road trial. (Chipperfield et al. 1970)

Bitumen No.	Construction/Service Stage at which Bitumen Sample was Recovered	Basalt Coarse Graded Mix — Broad Chemical Composition				Quartzite Coarse Graded Mix — Broad Chemical Composition			
		Asphaltenes % wt	Resins % wt	Cyclics % wt	Saturates % wt	Asphaltenes % wt	Resins % wt	Cyclics % wt	Saturates % wt
1	Original bitumen	14.7	17.9	54.6	11.0	14.7	17.9	54.6	11.0
	After mixing	17.1	25.8	41.1	11.2	18.0	30.5	36.9	11.2
	4 Months core sample	20.6	28.9	35.2	12.5	16.7	34.6	30.3	14.7
	20 Months core sample	22.2	29.9	28.8	16.0	-	-	-	-
	40 Months core sample	22.8	30.6	24.7	18.8	22.8 (19.3)	33.0 (32.3)	24.3 (34.9)	16.0 (11.3)
	72 Months core sample	20.4 (19.5)	29.0 (28.3)	27.6 (35.7)	17.7 (11.6)	21.7 (19.3)	28.4 (30.5)	22.1 (32.8)	19.1 (11.2)
2	Original bitumen	11.3	20.8	54.0	13.8	11.3	20.8	54.0	13.8
	After mixing	13.3	29.7	38.2	14.4	14.5	33.4	34.8	14.0
	4 Months core sample	19.6	31.3	31.7	14.0	16.7	34.6	30.3	14.7
	20 Months core sample	19.4	31.4	28.8	16.9	-	-	-	-
	40 Months core sample	20.1	31.5	29.1	16.1	19.1 (13.6)	34.6 (31.5)	26.3 (38.0)	16.6 (14.5)
	72 Months core sample	17.0 (13.0)	30.6 (26.2)	28.5 (41.5)	17.7 (14.8)	19.1 (13.5)	31.0 (29.0)	25.6 (38.6)	18.6 (14.4)
3	Original bitumen	11.6	18.8	55.8	12.0	11.6	18.8	55.8	12.0
	After mixing	13.3	28.3	42.6	11.6	14.7	31.3	39.0	11.6
	4 Months core sample	16.8	31.3	35.3	13.7	-	-	-	-
	20 Months core sample	19.8	31.8	27.8	17.2	-	-	-	-
	40 Months coressample	20.2	33.1	26.5	17.0	20.2 (14.5)	36.8 (35.2)	27.0 (36.4)	12.1 (11.4)
	72 Months core sample	16.9 (15.6)	35.0 (30.0)	29.8 (37.4)	13.8 (12.0)	19.5 (13.6)	34.8 (30.1)	26.9 (39.6)	13.7 (11.3)
4	Original bitumen	12.4	19.2	50.4	15.8	12.4	19.2	50.4	15.8
	After mixing	14.5	25.8	39.1	15.5	15.9	30.4	35.4	15.1
	4 Months core sample	17.7	30.8	31.1	17.9	17.6	32.8	29.0	17.4
	20 Months core sample	19.9	30.6	25.5	21.0	-	-	-	-
	40 Months core sample	21.0	28.8	24.9	22.4	21.4 (14.2)	31.7 (36.2)	23.4 (33.0)	19.9 (14.0)
	72 Months core sample	17.8 (16.8)	27.8 (27.5)	25.0 (25.6)	23.3 (16.2)	- (13.6)	- (29.4)	- (35.8)	- (16.1)
5	Original bitumen	14.6	16.3	54.1	13.3	14.6	16.3	54.1	13.3
	After mixing	16.3	23.1	41.7	14.3	16.7	27.0	38.9	14.2
	4 Months core sample	19.2	26.7	36.0	15.1	19.5	27.1	34.6	14.9
	20 Months core sample	21.0	29.0	29.8	17.2	-	-	-	-
	40 Months core sample	22.7	26.0	30.4	17.8	21.5 (16.8)	28.5 (27.7)	26.5 (39.7)	18.2 (14.4)
	72 Months core sample	18.8 (15.7)	25.3 (22.7)	30.3 (41.4)	18.6 (14.2)	21.3 (16.0)	25.6 (22.6)	25.2 (40.6)	20.1 (14.2)
7	Original bitumen	11.9	22.4	46.9	17.9	11.9	22.4	46.9	17.9
	After mixing	13.9	30.4	34.4	16.9	15.1	33.0	31.2	17.3
	4 Months core sample	19.6	31.9	27.6	17.3	16.9	34.8	27.0	18.0
	20 Months core sample	19.9	32.3	23.9	21.4	-	-	-	-
	40 Months core sample	21.4	29.6	25.0	21.1	19.2 (16.4)	33.8 (33.4)	22.7 (30.9)	20.8 (17.3)
	72 Months core sample	17.7 (15.1)	28.6 (29.6)	24.3 (31.9)	22.6 (17.2)	19.1 (15.6)	30.7 (30.6)	20.6 (30.5)	21.8 (17.2)
8	Original bitumen	13.5	17.9	52.3	15.1	13.5	17.9	52.3	15.1
	After mixing	15.5	25.9	39.8	14.8	16.7	27.3	37.4	15.0
	4 Months core sample	18.5	29.5	31.6	17.3	18.6	33.1	29.8	16.1
	20 Months core sample	20.3	28.5	28.1	19.9	-	-	-	-
	40 Months core sample	22.4	27.8	25.4	21.4	19.1 (16.0)	30.3 (29.3)	27.7 (37.9)	19.0 (15.4)
	72 Months core sample	18.2 (16.6)	26.0 (26.1)	28.6 (37.0)	21.0 (15.4)	19.6 (16.0)	26.4 (25.0)	26.5 (37.8)	20.5 (15.2)
9	Original bitumen	11.4	19.8	48.5	19.4	11.4	19.8	48.5	19.4
	After mixing	13.7	26.9	35.6	19.5	13.9	31.1	33.5	19.1
	4 Months core sample	16.9	30.7	30.7	19.6	16.3	31.2	28.8	20.4
	20 Months core sample	18.8	30.0	25.7	21.4	-	-	-	-
	40 Months core sample	20.2	30.5	23.8	21.8	18.2 (13.3)	31.9 (28.2)	24.2 (37.3)	22.6 (19.5)
	72 Months core sample	16.5 (13.8)	29.9 (26.1)	24.7 (34.8)	22.4 (19.3)	16.6 (13.3)	27.5 (26.1)	25.8 (36.8)	24.9 (19.8)

Note: Bitumens extracted from optimum binder content test sections - top 1/8 inch of cores
Values in brackets refer to third 1/8 inch of cores

Table A.15 Comparison of laboratory aging with field values measured on the RN7 coarse graded basalt mixes. (Chipperfield & Duthie 1970)

Bitumen No.	Sample	Broad Chemical Composition - % wt				Ageing Index
		Asphaltenes	Resins	Cyclics	Saturates	
1	Original bitumen	14.7	17.9	54.6	11.0	1
	After mixing	17.1	25.8	41.1	11.2	3.2
	4 Months core sample	20.6	28.9	35.2	12.5	9.1
	TFOT residue	16.6	23.6	46.6	11.0	3.3
	Californian Rolling TFOT residue	16.2	21.9	47.1	11.4	2.4
2	Original bitumen	11.3	20.8	54.0	13.8	1
	After mixing	13.3	29.7	38.2	14.4	2.5
	4 Months core sample	19.6	31.3	31.7	14.0	4.9
	TFOT residue	12.5	28.9	42.2	13.8	3.3
	Californian Rolling TFOT residue	11.1	25.2	43.8	14.1	2.8
3	Original bitumen	11.6	18.8	55.8	12.0	1
	After mixing	13.3	28.3	42.6	11.6	4.6
	4 Months core sample	16.8	31.3	35.3	13.7	7.9
	TFOT residue	13.3	25.8	45.2	11.3	6.9
	Californian Rolling TFOT residue	12.6	22.2	49.7	12.2	4.0
4	Original bitumen	12.4	19.2	50.4	15.8	1
	After mixing	14.5	25.8	39.1	15.5	4.3
	4 Months core sample	17.7	30.8	31.1	17.9	7.4
	TFOT residue	15.0	26.1	39.5	15.9	4.6
	Californian Rolling TFOT residue	14.0	24.0	43.1	16.3	3.6
5	Original bitumen	14.6	16.3	54.1	13.3	1
	After mixing	16.3	23.1	41.7	14.3	2.2
	4 Months core sample	19.2	26.7	36.0	15.1	6.0
	TFOT residue	15.9	23.0	45.1	13.9	3.6
	Californian Rolling TFOT residue	16.0	20.0	47.6	14.6	3.6
7	Original bitumen	11.9	22.4	46.9	17.9	1
	After mixing	13.9	30.4	34.4	16.9	3.7
	4 Months core sample	19.6	31.9	27.6	17.3	10.0
	TFOT residue	14.5	27.4	37.3	17.0	8.3
	Californian Rolling TFOT residue	13.7	23.7	42.6	18.4	5.2
8	Original bitumen	13.5	17.9	52.3	15.1	1
	After mixing	15.5	25.9	39.8	14.8	3.2
	4 Months core sample	18.5	29.5	31.6	17.3	6.5
	TFOT residue	16.5	25.2	41.5	14.8	6.3
	Californian Rolling TFOT residue	15.8	23.7	43.3	15.4	4.0
9	Original bitumen	11.4	19.8	48.5	19.4	1
	After mixing	13.7	26.9	35.6	19.5	2.2
	4 Months core sample	16.9	30.7	30.7	19.6	6.2
	TFOT residue	13.2	27.5	37.0	19.6	4.0
	Californian Rolling TFOT residue	12.8	25.5	38.4	20.6	3.0

Table A.16 Asphalt cement tests and calculated temperature susceptabilities of original asphalts in the Michigan test road. (Parr et al. 1955)

Section Source of crude	1 Wyoming A	2 Venezuelan	3 Wyoming B	4 Texas-Winkler	5 Arkansas Smackover	6 East Texas Talco
Laboratory No. 548	1716	1640	1641	1485	1443	1299
Course	Top	Top	Top	Top	Top	Top
Specific Gravity, 25/25 C	1.027	1.036	1.027	1.012	1.022	1.032
Penetration						
@ 15 C, 100g, 5 sec., dmm	22	21	22	23	21	23
@ 25 C, 100g, 5 sec., dmm	63	60	67	60	61	65
@ 35 C, 100g, 5 sec., dmm	173	154	186	151	144	141
@ 0 C, 200g, 60 sec., dmm	16	16	13	15	14	19
@ 4 C, 200g, 60 sec., dmm	21	21	20	24	21	26
Viscosity, Saybolt Furol @ 275 F, sec.	192.0	313	197.4	217	276	337
Viscosity, Saybolt Furol @ 300 F, sec.	101.4	169.6	105.0	128.0	150.0	172.0
Viscosity, Saybolt Furol @ 325 F, sec.	61.0	100.0	61.0	83.8	91.0	93.0
Ductility @ 4 C (5 cm/min) cm	0	1	0	5	5	5
Ductility @ 10 C (5 cm/min) cm	18	25	14	10	12	8
Ductility @ 15.6 C (5 cm/min) cm	150+	150+	150+	150+	150+	77
Ductility @ 20 C (5 cm/min) cm	150+	150+	150+	150+	150+	150+
Ductility @ 25 C (5 cm/min) cm	150+	150+	150+	150+	150+	150+
Ductility @ 30 C (5 cm/min) cm	140	150+	142	150+	150+	150+
Penetration Index (pen/pen)	-0.90	-0.52	-0.89	-0.59	-0.99	-0.26
(pen/R&B)	-0.93	-0.32	-0.99	-0.27	-0.67	0.42
Flash Point, Cleveland Open Cup, deg. C	320	282	326	316	356	300
Loss on Heating, 163 C, 5 hr., 50g, %	0.018	0.002	0.110	0.042	+0.004	0.018
Penetration of residue 25 C, 100g, 5s, dmm	56	54	62	56	56	59
Softening Point R & B, deg. C	48.9	51.8	48.1	50.7	49.0	51.2
Solubility in carbon tetratchloride, %	99.99	99.97	99.95	99.82	99.88	99.99
Oliensis Spot Test	Neg.	Neg.	Neg.	Neg.	Neg.	Neg.
Matter insoluble in hexane, %	17.25	21.44	17.43	15.50	15.33	21.94
0.125" Thin film method.						
Loss on heating @ 163 C, 1 hr, 50g, %	0.019	0.006	0.082	0.054	+0.021	0.009
Pen. of residue @ 25 C, 100g, 5 sec, dmm	54	54	55	52	54	55
Ductility of residue @ 25 C, 5 cm/min, cm	150+	150+	150+	150+	150+	150+
0.125" Thin film method.						
Loss on heating @ 163 C, 3 hrs, 50g, %	+0.010	0.057	0.073	0.052	+0.056	+0.004
Pen. of residue @ 25 C, 100g, 5 sec, dmm	47	45	45	46	45	53
Ductility of residue @ 25 C, 5 cm/min, cm	150+	150+	150+	150+	150+	120
0.125" Thin film method.						
Loss on heating @ 163 C, 5 hrs, 50g, %	0.022	0.132	0.079	0.091	+0.092	0.016
Pen. of residue @ 25 C, 100g, 5 sec, dmm	36	38	40	42	39	45
Ductility of residue @ 25 C, 5 cm/min, cm	150+	150+	150+	150+	150+	77
0.125" Thin film method.						
Loss on heating @ 163 C, 7 hrs, 50g, %	0.026	0.142	0.025	0.050	+0.105	0.054
Pen. of residue @ 25 C, 100g, 5 sec, dmm	31	34	34	38	35	37
Ductility of residue @ 25 C, 5 cm/min, cm	150+	150+	150+	150+	111	33
0.125" Thin film method.						
Loss on heating @ 163 C, 24 hrs, 50g, %	0.048	1.541	+0.046	0.114	+0.239	0.151
Pen. of residue @ 25 C, 100g, 5 sec, dmm	16	16	17	22	21	22
Ductility of residue @ 25 C, 5 cm/min, cm	6	4	6	7	6	4
Heat Stability Test, 100g, 500 F, 2 hrs						
Penetration @ 25 C, 100g, 5 sec, dmm	58	57	66	63	56	61
Viscosity, Saybolt Furol @ 275 F, sec	215	337	205	192.8	284	333
Viscosity, Saybolt Furol @ 325 F, sec	68.0	108.0	61.4	58.6	88.2	109.0
Heat Stability Test, 100g, 600 F, 2 hrs						
Penetration @ 25 C, 100g, 5 sec, dmm	62	65	70	98	65	61
Viscosity, Saybolt Furol @ 275 F, sec	206	321	186.2	151.6	266	316
Viscosity, Saybolt Furol @ 325 F, sec	70.0	103.0	57.6	50.0	71.5	95.0
Shattuck Test						
Penetration @ 25 C, 100g, 5 sec, dmm	29	28	29	38	34	36
Ductility @ 25 C, 5 cm/min, cm	136+	69	150+	150+	137	26

Table A.17 Asphalt cement tests and calculated temperature susceptibilities of recovered asphalts in the Michigan test road. (Serafin et al. 1967)

Section Source of crude	1 Wyoming A	2 Venezuelan	3 Wyoming B	4 Texas-Winkler	5 Arkansas Smackover	6 East Texas Talco
After 4 years:						
Penetration						
@ 25 C, 100g, 5 sec., dmm	40	31	36	38	41	34
Ductility						
@ 25 C(5cm/min)cm	150+	150+	150+	142	150+	65
After 11 years:						
Penetration						
@ 25 C, 100g, 5 sec., dmm	31	26	30	32	35	29
Ductility						
@ 25 C(5cm/min)cm	150+	136	150+	108	138	33

Table A.18 Summary of rut depth measurements in the Michigan test. (Serafin et al. 1967)

Lane	Wheel Track	Average Rut Depth for Subsection Indicated, in.											
		1A	1B	2A	2B	3A	3B	4A	4B	5A	5B	6A	6B
1	Right	0.16	0.08	0.14	0.16	0.20	0.21	0.14	0.18	0.19	0.15	0.13	0.18
	Left	0.11	0.05	0.15	0.16	0.20	0.15	0.22	0.15	0.10	0.12	0.11	0.11
2	Right	0.03	0.02	0.05	0.06	0.03	0.05	0.09	0.06	0.03	0.06	0.05	0.03
	Left	0.07	0.04	0.09	0.03	0.04	0.08	0.12	0.09	0.07	0.09	0.09	0.06
3	Left	0.01	0.03	0.03	0.05	0.04	0.06	0.04	0.04	0.04	0.05	0.05	0.03
	Right	0.02	0.01	0.03	0.03	0.07	0.05	0.05	0.02	0.03	0.06	0.03	0.06
4	Left	0.16	0.21	0.13	0.11	0.07	0.17	0.28	0.15	0.18	0.38	0.16	0.23
	Right	0.19	0.19	0.28	0.22	0.19	0.27	0.36	0.21	0.23	0.24	0.17	0.22

Table A.19 Physical properties of original and recovered samples in Michigan. (Corbett & Merz 1975)

Sample	Property	Section 1	2	3	4	5	6
Original	Softening point, deg F	126	124	124	125	125	126
	Penetration at 77 F	46	52	48	49	51	49
	Ductility at 60 F, cm	150+	150+	150+	67	16	27
	Viscosity at 140 F*, poises	3,880	5,310	3,290	4,480	6,410	7,740
Recovered, top ¼-in.	Softening point, deg F	136	150	140	147	147	165
	Penetration at 77 F	26	19	23	23	23	19
	Ductility at 77 F, cm	150+	7	150+	6	7	5
	Ductility at 60 F, cm	5	0.5	4.5	—	3	2.5
Recovered, ¼-in. minus	Softening point, deg F	129	136	133	135	138	147
	Penetration at 77 F	37	34	36	35	36	32
	Ductility at 77 F, cm	150+	150+	150+	150+	150+	40
	Ductility at 60 F, cm	8	8.5	8.5	6.5	6	4.5
	Absolute viscosity at 140 F, poises	7,320	17,041	7,752	9,705	11,787	34,414

Note: 1 in. = 25 mm; 1 F = 1.8 C + 32; 1 poise = 0.1 Pa·s.

*1965 data.

Table A.20 Compositional analysis of original and recovered asphalts in Michigan. (Corbett & Merz 1975)

Sample	Fraction	Section 1	2	3	4	5	6
Original	Saturates	9.8	6.0	8.6	13.9	7.9	8.6
	Naphthene aromatics	32.5	28.8	32.6	31.3	42.0	38.7
	Polar aromatics	41.7	45.1	46.7	40.9	36.5	32.4
	Asphaltenes	16.0	19.2	12.0	12.8	13.3	19.7
Recovered, top ¼-in.	Saturates	9.8	7.1	9.7	15.7	9.6	9.9
	Naphthene aromatics	25.9	20.7	25.9	22.7	28.4	24.2
	Polar aromatics	43.9	43.8	41.2	40.5	40.5	35.2
	Asphaltenes	19.3	27.7	19.4	20.3	20.7	28.8
Recovered, ¼-in. minus	Saturates	8.9	5.7	8.6	13.7	7.8	8.7
	Naphthene aromatics	34.6	28.9	35.5	30.4	32.7	35.0
	Polar aromatics	39.5	40.4	40.1	40.3	43.4	31.5
	Asphaltenes	16.9	22.8	15.6	15.0	15.9	24.7

Note. 1 in. = 25 mm.

Table A.21 Inspection data on original 85/100 penetration asphalt cements used for Ontario's three 1960 test roads. (McLeod 1972)

Supplier Number	1	2	3
Flash Point COC F.	585	525	615
Softening Point R and B, F.	115	115	119
Penetration 100 gr. 5 sec. 77 F.	83	96	87
200 gr. 60 sec. 39.2 F.	25	36	22
200 gr. 60 sec. 32 F.	22	26	19
Penetration Ratio	30.2	37.5	25.3
Ductility at 77 F., 5 cm/min	150+	150+	128
Viscosity Centistokes at 275 F.	460	365	210
Centistokes at 210 F.	3953	2763	1472
Thin Film Oven Test			
% loss by weight	0.1	0.3	0.0
Residue			
% Original Penetration at 77 F.	67.5	60.4	61.0
Ductility at 77 F., 5 cm/min	150+	110	115
Solubility in n-hexane			
% asphaltenes	19.7	24.7	18.8
Penetration Index			
(Pfeiffer and Van Doormaal)	-1.00	-0.57	-0.21
Pen-vis number	-0.19	-0.36	-1.34

Table A.22 Analysis of surface course pavement samples from the three pavement sections in each of Ontario's three test roads. (McLeod 1972)

Test Road		1			2			2	
Asphalt Supplier	1	2	3	1	2	3	1	2	3
Recovered Aggregate									
Passing Sieve 3/4 inch	100	100	100	100	100	100	100	100	100
1/2 inch	97.7	97.3	98.2	99.6	98.1	98.6	98.2	97.2	96.1
3/8 inch	78.7	80.9	81.3	86.1	85.0	83.3	79.8	80.4	79.3
No. 4	55.9	59.3	57.6	57.9	59.3	59.1	57.0	56.1	57.6
No. 8	47.6	44.8	49.2	47.5	47.0	46.3	47.6	46.1	47.9
No. 16	41.4	39.9	43.5	40.8	39.2	37.0	38.7	37.0	39.8
No. 30	32.7	31.1	36.1	31.3	28.9	27.1	25.1	24.4	27.6
No. 50	170	17.2	20.7	14.0	13.0	11.9	10.0	10.9	12.0
No. 100	73	7.1	8.7	5.8	5.8	5.3	5.7	6.0	5.4
No. 200	4.9	4.5	5.7	3.7	4.0	3.7	4.3	4.4	3.6
Specific Gravity Aggregate									
ASTM Bulk	2.652	2.653	2.652	2.681	2.680	2.682	2.627	2.627	2.627
ASTM Apparent	2.746	2.764	2.762	2.775	2.775	2.775	2.728	2.727	2.728
Virtual	2.712	2.721	2.684	2.725	2.706	2.697	2.680	2.685	2.681
Asphalt Absorption	0.84	0.94	0.46	0.60	0.36	0.23	0.75	0.82	0.77
Water Absorption, Wt. %	1.3	1.5	1.5	1.3	1.27	1.26	1.4	1.4	1.4
Characteristics									
Asphalt Content % by wt. of mix	5.63	5.91	5.60	6.45	6.15	6.0	6.17	5.63	6.02
Maximum Specific Gravity	2.474	2.470	2.464	2.452	2.449	2.448	2.428	2.452	2.435
Bulk Specific Gravity, Slabs	2.394	2.406	2.384	2.421	2.380	2.400	2.335	2.392	2.381
Bulk Specific Gravity, Recompacted	2.438	2.421	2.434	2.420	2.433	2.423	2.387	2.393	2.411
Percentage of Laboratory Density	98.2	99.4	97.9	100.0	97.8	99.1	97.8	100.0	98.8
Air Voids, Slabs, %	3.2	2.6	3.2	1.3	2.8	2.0	3.8	2.4	2.2
Air Voids, Recompacted, %	1.5	2.0	1.2	1.3	0.6	0.9	1.8	2.4	1.0
Voids, Mineral Aggregate, Recompacted %	13.2	14.1	13.3	15.6	14.8	15.1	14.9	14.0	13.7
Marshall Stability, Lb. at 140 F.	3400	3300	3150	2185	2800	2500	2475	2475	2625
Marshall Flow Index	14	11	13	13	14	11	11	11	11.5

Table A.23 Analysis of asphalt binders recovered from surface course of Ontario test road 1. (McLeod 1972)

Asphalt Supplier	1			2			3		
Description	Top 1/4 in.	Minus Top 1/4 in.	Full Thickness	Top 1/4 in.	Minus Top 1/4 in.	Full Thickness	Top 1/4 in.	Minus Top 1/4 in.	Full Thickness
Penetration									
100 g., 5 sec., 77 F.	22	30	-	23	34	30	26	31	28.5
200 g., 60 sec., 39.2 F.	10.5	13.5	-	14	21	17.5	13	15.5	14
Penetration Ratio	47.7	45.0	-	60.9	61.8	58.3	50.0	50.0	49.1
Ductility, 5 cm/min/77 F.	148	150-	-	21	71	51	10	20	15
Viscosity									
Poises at 140 F.	20,757	10,235	-	29,004	9,144	15,369	13,705	7,252	8,452
Centistokes at 275 F.	1,092	770	-	1,036	654	799	504	409	431

Table A.24 Analysis of asphalt binders recovered from surface course of Ontario test road 2. (McLeod 1972)

Asphalt Supplier	1			2			3		
Description	Top 1/4 in.	Less Top 7/16 in.	Full Thickness	Top 1/4 in.	Less Top 7/16 in.	Full Thickness	Top 1/4 in.	Less Top 7/16 in.	Full Thickness
Penetration									
100 g., 5 sec., 77 F.	30	51	44.5	37	37	34	33	40.5	35
200 g., 60 sec., 39.2 F.	16	23	20.5	14	18	16	17.5	20	18
Penetration Ratio	53.3	45.1	46.1	51.9	48.6	47.0	53.0	49.4	51.4
Ductility, 5 cm/min/77 F.	150+	150+	150+	37	99	107	23	69	51
Viscosity									
Poises at 140 F.	9,791	3,809	4,446	19,666	7,988	11,368	6,947	3,341	5,303
Centistokes at 275 F.	815	589	631	949	659	733	435	334	370

Table A.25 Analysis of asphalt binders recovered from surface course Ontario test road 3. (McLeod 1972)

Asphalt Supplier	1			2			3		
Description	Top 1/4 in.	Minus Top 1/4 in.	Full Thickness	Top 1/4 in.	Minus Top 1/4 in.	Full Thickness	Top 1/4 in.	Minus Top 1/4 in.	Full Thickness
Penetration									
100 g., 5 sec., 77 F.	32	43	38.5	26	52	40	30	37	35
200 g., 60 sec., 39.2 F.	15	18.5	16.5	15	26	22	14	16.5	15
Penetration Ratio	46.9	43.0	42.9	57.7	50.0	55.0	46.7	44.6	42.9
Ductility, 5 cm/min/77 F.	150+	150+	150+	42	150+	141	17	46	36
Viscosity									
Poises at 140 F.	8,945	4,838	6,165	17,673	3,652	8,081	9,691	4,059	4,646
Centistokes at 275 F.	788	597	695	884	481	640	439	339	371

Table A.26 Average penetration at 77°F, 100g., 5 sec. at Saskatchewan test road. (Culley 1969)

Refinery	Tank	Line	Truck	Paver	Road @ 0	Road @ 12	Truck / Tank	Road @ 12 / Road @ 0	Road @ 12 / Tank
1 (9-9)	147	145	117	109	112	72	0.80	0.64	0.49
1 (10-7)	157	152	110	99	86	73	0.70	0.85	0.46
3	177	---	124	130	132	82	0.70	0.62	0.46
4	149	147	86	92	94	94	0.58	0.79	0.50
5	138	139	89	94	90	66	0.64	0.73	0.48
Average Penetration at 39.2 F, 200 g., 60 sec.									
1 (9-9)	42	42	34	32	33	24	0.81	0.73	0.57
1 (10-7)	41	41	34	30	28	18	0.83	0.64	0.44
3	60	--	42	44	44	29	0.70	0.66	0.48
4	44	43	30	31	33	20	0.68	0.61	0.45
5	50	41	28	29	28	22	0.56	0.79	0.44

Table A.27 Average viscosities at 60°F (Poises) in Saskatchewan. (Culley 1969)

Refinery	Tank	Line	Truck	Paver	Road @ 0	Road @ 12	Truck / Tank	Road @ 12 / Road @ 0	Road @ 12 / Tank
1 (9-9)	4.7×10^6	4.1×10^6	7.3×10^6	8.4×10^6	7.6×10^6	15.9×10^6	1.55	2.09	3.39
1 (10-7)	5.3	5.2	7.4	8.2	8.5	18.9	1.40	2.22	3.56
3	2.6	---	6.4	6.2	5.7	17.6	2.46	3.09	6.79
4	3.9	4.5	11.8	13.6	11.8	15.8	2.81	1.34	4.05
5	7.3	5.4	15.2	10.7	11.7	19.3	2.37	1.65	2.65
				Average Viscosities at 140 F					
1 (9-9)	614	621	951	975	945	2232	1.55	2.29	3.62
1 (10-7)	640	673	1021	1016	1019	2007	1.60	1.98	3.14
3	731	---	1297	1173	1153	2291	1.76	1.98	3.12
4	641	617	1487	1326	1315	2005	2.31	1.53	3.12
5	754	728	1335	1319	1270	2135	1.77	1.68	2.82
				Average Viscosities at 275 F					
1 (9-9)	2.30	2.31	2.72	2.77	2.77	---	1.18	---	---
1 (10-7)	2.29	2.30	2.77	2.77	2.83	---	1.21	---	---
3	2.60	---	3.15	3.06	3.02	---	1.21	---	---
4	2.26	2.35	3.25	3.11	3.15	---	1.44	---	---
5	2.33	2.31	3.06	2.89	2.97	---	1.31	---	---

Table A.28 Average temperature susceptibility and shear susceptibility data for Saskatchewan test road. (Culley 1969)

Refinery	Tank	Truck	Road @ 0	Road @ 12
		$\frac{\text{Pen @ 39.2 F}}{\text{Pen @ 77 F}} \times 100$		
1 (9-9)	29	30	29	34
1 (10-7)	27	31	34	25
3	34	34	33	35
4	29	34	34	28
5	36	32	32	33
		$\frac{\text{Change in Viscosity}}{\text{Change in Temperature}}$		
1 (9-9)	1.03	0.98	0.99	0.90
1 (19-7)	1.03	0.97	0.98	0.93
3	0.95	0.93	0.94	0.91
4	1.01	0.95	0.97	0.92
5	1.02	0.98	0.97	0.92
		Change in Viscosity with Change in Shear Rate from 0.01 sec^{-1} to 0.10 sec^{-1}		
1 (9-9)	0.8×10^6 poises	4.9×10^6	4.0×10^6	10.8×10^6
1 (10-7)	1.6	3.1	4.3	19.0
3	0.3	2.2	2.4	14.2
4	1.0	10.4	9.7	14.3
5	3.5	15.3	9.0	17.8

Table A.29 Pavement project locations in Iowa. (Lee 1973)

Number	County	Location	Project Number	Date Laid
1	Chickasaw	On US-63 north of New Hampton	FN-63-8(1)-20-19	Nov. 1967
2	Dickinson	On Iowa 327 from Iowa 276 east and north	FN-327-1(1)-21-30	Oct. 1967
3	Harrison	On US-75 out of Mo. Valley, north into Mondamin	FN-75-2(3)-21-43	Nov. 1967
4	Story-Polk	On US-69 between Huxley and Ankeny	FN-69-5(2)	Oct. 1967
6	Monona	On US-75 from Harrison Co. line north into Ottawa 11 mi.	FN-3(2)-21-67	April 1968
7	Bremer	On Iowa 3	FN-3-6(5)-21-09	May 1968
8	Keokuk	On Iowa 92 from Sigorney east	FN-92-8(2)-21-54	May 1968
9	Jackson	On US-52 north of Maquoketa	FN-52-1(3)-21-49	June 1968

Table A.30 Characteristics of field pavement mixtures in Iowa. (Lee 1973)

	Asphalt							
Characteristic	1	2	3	4	6	7	8	9
Gradation, percent passing sieve								
3/4 in.					100			100
1/2 in.					94			
3/8 in.	100	100	100	100	77	100	100	78
No. 4	81	89	92	78	60	85	85	62
No. 8	62	67	58	59	43	67	63	49
No. 16	44	—	—	—	—	—	—	—
No. 30	30	34	34	31	25	35	35	23
No. 50	16	21	23	18	19	35	25	—
No. 100	12	13	13	13	9	—	13	—
No. 200	9.3	9.9	9.4	9.4	7.1	8.4	8.9	6.7
Percentage of bitumen	7.5	6.3	6.3	7.5	5.0	7.0	5.3	5.5
Specific surface[a], ft^2/lb	40.59	45.92	44.18	41.86	33.44	51.06	44.34	32.66
Bitumen index[b], $\times 10^{-3}$	1.996	1.459	1.517	1.935	1.585	1.469	1.263	1.778
Film thickness[c], μ	9.72	7.11	7.39	9.24	7.12	7.15	6.15	8.65
Laboratory design voids, percent	6.8	6.1	6.6	9.3	6.8	6.1	3.5	5.8
Hveem side pressure[d], psi	60	46	63	48	55	61	55	29
Asphalt concrete temperature, deg F	295	300	295	260	267	260	275	300
Aggregate temperature, deg F	300	320	310	310	340	310	375	305
Mix temperature, deg F	290	295	310	310	310	305	306	298
Initial field voids, percent	8.7	9.8	11.5	12.3	5.5	7.1	6.7	5.1
ADT, 1970	2,100	330	590	3,000	500	2,000	1,500	2,400
Condition after 48 months	Transverse cracks	Excellent	Excellent	Severe transverse and longitudinal centerline cracks	Good	Good, some cracks	Good to excellent	Excellent

[a]Mix Design Methods for Asphalt Concrete. The Asphalt Institute, MS-2, 1962.
[b]Percent bitumen (aggregate basic)/specific surface.
[c]Bitumen index x 4.870.
[d]At 400 psi vertical load.

Table A.31 Properties of asphalts studied in Iowa (Lee 1973)

CHARACTERISTIC	ASPHALT								
	1	2	3	4	5	6	7	8	9
Penetration, 77/100/5	89	94	91	90	84	87	95	90	92
Specific gravity, 77/77	1.017	1.026	1.042	1.011	1.017	1.019	1.042	1.003	0.999
Viscosity									
77°F, megapoises	1.16	1.23	1.58	1.10	1.14	1.70	1.15	1.10	1.22
140°F, poises	1,356	1,086	1,316	1,106	1,781	1,455	1,316	1,922	2,060
Softening point, deg F	119.0	116.5	115.5	114.5	113.0	118.0	116.0	118.0	119.5
Flash point, COC, deg F	600	595	630	625	690	615	625	655	655
Fire point, COC, deg F	680	690	705	705	730	670	685	735	735
Microductility at 77°F, cm	63	71	66	77	82	55	68	51	63
TFOT									
Residue penetration	53	51	57	56	67	55	59	60	62
Weight loss, percent	0.02	0.16	▲0.01	0.00	♦0.01	0.07	0.07	0.24	0.16
Spot test	Negative	Negative	Negative	Negative	Negative	Negative	Negative	Negative	Negative
Source*	1	1	2	2	3	4	4	2	2

*1 = blend of asphalts from Texaco at Casper, Wyoming, and American Petroline, Big Springs, Texas. 2 = American Oil, Sugar Creek, Missouri. 3 = Esso Research and Engineering Co. 4 = American Petroline, Big Springs, Texas.

Table A.32 Changes in viscosity at 77°F for Iowa test road. (Lee 1973)

Project Sample	Asphalt Viscosity (megapoises)								
	1	2	3	4	5	6	7	8	9
Original	1.16	1.23	1.58	1.10	1.14	1.70	1.15	1.10	1.22
TFOT	4.22	5.35	3.70	3.09	1.68	3.80	3.90	4.15	2.86
Laboratory aging (hours)									
d-24	18.4	11.7	9.50	9.0*	3.50*	—	7.00*	8.30*	10.30
d-48	20.5	19.3	14.2	15.7	5.60	14.60	12.50*	12.60	12.50
d-96	23.9	24.6	19.2	22.6	7.40	21.00	22.00*	14.80	15.00
d-240	44.0	36.2	47.0	36.9	13.50	33.00	51.00*	38.00	27.50*
d-480	64.5*	38.5	76.0*	52.0*	14.70*	37.00	78.50	53.50*	45.00*
d-1000	89.0	—	118.0	64.0	17.30	72.00	99.20	64.00	69.00*
Field aging (months)									
Plant	4.30	4.70	3.40	3.09	—	2.81	2.07	3.42	2.29
f-0									
A[b]	4.87	4.60	3.39	3.01	—	—	2.25	2.14	2.61
B[c]	4.87	4.60	3.39	3.01	—	—	—	—	—
f-6									
A	4.30	5.98	3.40	3.56	—	8.58	8.30	6.40	4.90
B	5.70	5.35	3.78	3.50	—	8.70	8.74	6.40	3.90
f-12									
A	9.80	8.90	9.66	11.2	—	10.60	9.80	14.50	11.50
B	9.54	9.20	—	12.2	—	11.90	9.80	12.50	8.90
f-18									
A	12.7	8.33	12.4	13.9	—	12.10	12.80	11.50	10.50
B	11.5	10.28	12.0	15.5	—	13.80	12.00	10.90	10.50
f-24									
A	12.2	—	12.6	21.0	—	12.70	14.50	9.80	12.00
B	15.0	15.0	11.9	18.6	—	7.80	12.20	8.90	8.40
f-30									
A	14.5	18.5	17.0	20.5	—	13.50	12.00	13.50	9.68
B	18.5	19.2	13.2	21.6	—	17.50	12.00	14.00	9.10
f-36									
A	12.3	13.8	21.0	16.2	—	—	13.80	—	11.50
B	12.5	16.0	19.6	17.7	—	—	—	—	—
f-42									
A	13.4	17.8	18.8	16.2	—	17.50	—	12.50	—
B	9.5	16.5	14.0	22.2	—	—	—	—	—
f-48									
A	19.2	19.6	20.7	21.8	—	—	—	—	—
B	16.4	20.9	16.1	22.0	—	—	—	—	—

*Interpolated value. [b]A = in wheel tracks. [c]B = between wheel tracks.

Table A.33 Changes in viscosity at 140°F for Iowa test road. (Lee 1973)

	Asphalt Viscosity (poises)								
Project Sample	1	2	3	4	5	6	7	8	9
Original	1,356	1,086	1,316	1,106	1,781	1,350	1,316	1,922	2,837
TFOT	2,368	3,448	4,041	3,556	2,341	3,556	3,538	4,376	4,412
Laboratory aging (hours)									
d-24	12,094	10,849	5,812	—	—	—	7,400[a]	10,500[a]	12,558
d-48	15,248	17,022	7,979	11,830	4,891	12,457	9,000[a]	20,199	13,826
d-96	23,276	34,249	12,772	13,930	4,459	19,550	12,800[a]	24,472	23,497
d-240	81,254	89,639	28,062	23,479	7,727	33,431	26,900[a]	44,731	—
d-480	—	—	—	—	16,500[a]	52,000[a]	53,500[a]	66,000	—
d-1000	200,000	—	130,000	71,560	37,000	80,000	130,000	120,000	—
Field aging (months)									
Plant	2,182	2,045	1,913	2,136	—	2,096	2,070	2,634	3,428
f-0									
A[b]	2,294	2,931	2,122	1,781	—	—	2,130	2,456	4,001
B[c]	2,294	2,931	2,122	1,781	—	—	—	—	—
f-6									
A	2,716	3,735	2,820	2,582	—	7,546	5,506	7,762	4,824
B	3,555	3,412	3,038	2,525	—	6,227	5,737	9,292	5,852
f-12									
A	5,814	8,165	4,634	5,984	—	5,167	4,634	9,717	4,593
B	6,722	8,098	5,829	5,964	—	8,181	4,597	9,005	5,358
f-18									
A	4,733	6,964	5,331	5,077	—	9,436	6,753	13,468	6,663
B	3,717	7,806	4,799	6,178	—	—	—	—	8,109
f-24									
A	5,158	9,742	6,437	7,672	—	7,797	5,950	9,676	8,653
B	6,675	10,204	6,217	8,526	—	—	—	—	—
f-30									
A	6,710	9,341	6,229	7,960	—	9,250	6,644	13,092	9,392
B	8,506	9,588	5,593	8,605	—	8,567	—	—	—
f-36									
A	5,747	9,991	7,218	7,777	—	—	7,270	10,532	13,940
B	7,031	12,792	8,130	7,484	—	—	—	—	—
f-42									
A	6,418	12,173	6,951	7,598	—	12,384	—	12,925	—
B	4,954	10,054	9,969	—	—	—	—	—	—
f-48									
A	11,044	13,773	8,305	8,490	—	—	—	—	—
B	9,473	14,082	7,549	8,647	—	—	—	—	—

[a]Interpolated value [b]A = in wheel tracks. [c]B = between wheel tracks.

Table A.34 Changes in softening point during weathering at Iowa test road. (Lee 1973)

Project Sample	Asphalt Softening Point, Ring and Ball (deg F)								
	1	2	3	4	5	6	7	8	9
Original	119.0	116.5	115.5	114.5	113.0	118.0	116.0	118.0	119.5
TFOT	127.5	131.0	123.0	126.0	119.0	127.0	123.0	129.0	119.5
Laboratory aging (hours)									
d-24	141.0	134.0	131.0	133.5[a]	121.5[a]	—	125.5[a]	136.0[a]	132.5
d-48	140.5	143.0	138.0	138.0	126.5	140.0	128.5[a]	142.5	136.0
d-96	143.5	149.0	140.5	141.0	126.0	143.0	134.0[a]	144.5	145.5
d-240	159.0	152.5	145.0	148.5	134.0	152.0	146.6[a]	150.0	151.5[a]
d-480	168.0[a]	167.0	154.5[a]	154.5[a]	143.8[a]	163.7[a]	160.5[a]	164.0[a]	—
d-1000	175.0	170.0[a]	171.0	173.5	154.0	174.0	171.0	173.0	—
Field aging (months)									
Plant	130.5	129.0	127.5	122.5	—	118.0	120.0	124.0	116.0
f-0									
A[b]	127.5	125.0	125.0	119.0	—	—	118.0	132.0	124.0
B[c]	127.5	123.0	125.0	119.0	—	—	—	—	—
f-6									
A	124.0	128.0	122.5	120.5	—	133.5	132.0	136.0	132.0
B	125.0	120.5	125.5	120.5	—	133.0	130.0	136.0	126.0
f-12									
A	129.0	135.0	130.0	130.0	—	134.0	129.5	134.0	138.0
B	129.5	137.5	131.0	131.0	—	138.5	130.0	126.0	132.0
f-18									
A	130.0	137.0	—	134.0	—	135.0	132.8	136.4	132.0
B	132.0	136.0	128.0	134.0	—	140.0	132.8	136.0	130.0
f-24									
A	129.5	134.5	136.5	136.5	—	129.0	138.0	138.0	136.0
B	136.0	138.0	136.5	136.5	—	—	—	—	—
f-30									
A	137.0	140.0	136.5	135.5	—	140.0	137.0	141.0	136.0
B	138.0	137.0	—	136.0	—	—	—	—	—
f-36									
A	138.0	143.5	138.0	134.5	—	—	137.0	—	139.0
B	140.0	142.0	138.0	136.5	—	—	—	—	—
f-42									
A	133.0	138.0	140.0	134.5	—	142.0	—	144.0	—
B	129.0	138.2	141.0	134.6	—	—	—	—	—
f-48									
A	134.5	136.5	136.5	136.5	—	—	—	—	—
B	137.3	136.4	134.6	136.4	—	—	—	—	—

[a] Interpolated value. [b] A = in wheel tracks. [c] B = between wheel tracks.

Table A.35 Changes in microductility at 77°F for Iowa test road. (Lee 1973)

Project Sample	Asphalt (cm)								
	1	2	3	4	5	6	7	8	9
Original	63	71	72	67	81.4	53.0	68.4	51.0	63.0
TFOT	46	36	82	89	91.0	57.0	84.0	60.0	55.0
Laboratory aging (hours)									
d-24	9.2	5.4	59	82	101.0*	16.0*	70.0*	20.0*	11.0
d-48	5.0	3.0	50	41	102.6	5.1	56.0*	7.0	5.5
d-96	2.6	2.0	20	23	122.0	3.0	29.0*	4.0	6.7
d-240	2.0	1.5	5.2	4.7	115.0	0.5	8.0*	2.0	—
d-480	—	—	—	3.8	79.0*	1.0*	4.0*	1.5*	—
d-1000	0.5	—	0.4	1.8	-0.3	1.1	1.3	1.0	—
Field aging (months)									
Plant	48	62	87	93	—	75.0	68.0	67.0	65.0
f-0									
A[b]	69	72	78	81	—	—	60.0	67.0	79.0
B[c]	69	72	78	102	—	—	—	—	68.0
f-6									
A	87	62	102	102	—	23.0	73.0	15.0	37.0
B	87	63	95	101	—	23.0	64.5	14.6	28.0
f-12									
A	63	27	44	58	—	24.4	76.0	25.0	57.0
B	70	—	82	56	—	31.0	101.0	24.0	47.0
f-18									
A	70	33	88	84	—	—	55.5	8.5	16.5
B	76	35	79	85	—	—	80.5	11.0	16.5
f-24									
A	65	14	21	69	—	23.0	57.8	—	—
B	16	9.0	43	55	—	—	—	—	—
f-30									
A	16	7.7	84	65	—	7.2	—	5.9	—
B	17	8.0	69	74	—	—	—	—	—
f-36									
A	15	8.3	35	26	—	—	50.7	—	—
B	17	6.3	35	37	—	—	—	—	—
f-42									
A	16.5	10.0	43	37	—	4.7	—	5.0	—
B	14.0	4.0	34	12	—	—	—	—	—
f-48									
A	4.8	3.8	28	28	—	—	—	—	—
B	6.0	—	29	15	—	—	—	—	—

*Interpolated value. [b]A = in wheel tracks. [c]B = between wheel tracks.

Table A.36 Properties of original asphalts in Delaware. (Kenis 1962)

	Asphalt A			Asphalt B		
	Del.	BPR	AI	Del.	BPR	AI
Specific gravity, 77 F	1.023	1.023	1.024	1.030	1.030	1.030
Penetration, 100 g, 5 sec, 77 F	60	62	61	65	66	66
Softening point, R&B (°F)	118	123	125	118	124	124
Ductility, 5 cm/min, 77 F (cm)	100	250	150	100	250	150
Flash point (°F):						
C.O.C.	585	580	610	610	570	595
P.M.	-	505	-	-	-	-
Solubility in CCl_4 (%)	99.9	99.9	99.2	99.9	99.8	99.9
Inorganic matter insol (%)	-	0.10	0.03	-	0.22	0.02
Furol viscosity at 275 F (sec)	-	279	-	-	285	-
Oliensis spot test	Neg.	Neg.	-	Neg.	Neg.	-
Loss on heating at 325 F (%)	0.04	0.02	0.03	0.03	0.02	0.03
Pen residue, 100 g, 5 sec, 77 F (%)	56	55	55	60	59	60
Orig. pen (%)	93	89	90	92	89	91
Film oven test, 1/8-in. film, 5 hr, 325 F:						
Loss (%)	-	0.09	0.08	-	0.08	0.11
Pen residue, 100 g, 5 sec, 77 F (%)	-	40	40	-	42	41
Orig. pen (%)	-	65	66	-	64	62
Soft point of residue (°F)	-	133	132	-	135	134
Duct. res., 77 F, 5 cm/min (cm)	-	250	150	-	140	150

Table A.37 Aggregate characteristics at Delaware test road. (Kenis 1962)

Characteristic	BPR	AI	Del.
Los Angeles percent of wear	24.7[a]	21.6[a]	27.5[b]
Sodium sulfate soundness loss (%)	4.6	-	-
Specific gravity:			
Coarse aggregate:			
Apparent	-	2.823	-
Bulk	-	2.870	-
Absorption (%)	-	0.540	-
Fine aggregate:			
Apparent	-	2.808	-
Bulk	-	2.795	-
Absorption (%)	-	0.150	-
Combined:			
Apparent	2.81	2.814	2.79
Bulk	2.76	2.789	2.77
Absorption (%)	0.60	0.30	0.30
Sand equivalent	86	83	-

[a]Based on grading C, AASHO Method T96.
[b]Based on grading A, AASHO Method T96.

Table A.38 Properties of asphalts recovered from wearing course in Delaware. (Kenis 1962)

Sample	Immediately After Construction									After One Year (1959)									After Two Years (1960)								
	Pen. 100 g, 5 sec, 77 F			Softening Point (°F)			Ductility 77 F, 5 cm/min			Pen. 100 g, 5 sec, 77 F			Softening Point (°F)			Ductility 77 F, 5 cm/min			Pen. 100 g 5 sec, 77 F			Softening Point (°F)			Ductility 77 F, 5 cm/min		
	Del.	AI	BPR	Del.	AI	BPR	Del.	AI	BPR	Del.	AI	BPR	Del.	AI	BPR	Del.	AI	BPR	Del.	AI	BPR	Del.	AI	BPR	Del.	AI	BPR
AF-1	34	-	34	133	-	137	17	-	250+	19	-	27	153	-	142	10	-	79	22	24	24	-	145	146	-	42	78
AF-2	38	-	32	138	-	139	26	-	226	25	-	24	-	-	145	-	-	85	23	22	21	-	146	146	-	38	71
AF-3	36	-	33	140	-	137	31	-	250+	25	-	22	145	-	142	19	-	129	21	22	22	-	145	147	-	42	55
AF-4	30	-	34	142	-	137	19	-	160	19	-	26	147	-	140	22	-	152	21	22	23	-	145	144	-	45	88
Avg.	35	33	33	138	136	137	23	150+	208+	22	30	25	148	138	142	17	150+	111	22	23	22	-	145	146	-	42	73
AR-1	30	-	33	136	-	137	78	-	242	-	-	24	-	-	142	-	-	93	21	21	22	144	146	145	23	50	69
AR-2	40	-	31	133	-	140	68	-	250+	22	-	25	145	-	136	15	-	96	21	23	17	136	148	147	20	27	28
AR-3	46	-	30	131	-	140	91	-	180	20	-	24	153	-	142	10	-	62	22	22	19	143	146	145	20	38	55
AR-4	42	-	32	133	-	138	82	-	250+	24	-	27	149	-	140	20	-	149	22	24	18	140	146	147	26	55	58
Avg.	40	32	32	133	136	139	80	150+	220+	22	31	25	149	138	140	15	150+	100	22	23	19	141	146	146	22	43	52
BF-1	38	-	36	135	-	136	91	-	143	22	-	25	145	-	146	20	-	41	26	27	27	152	146	144	39	45	62
BF-2	34	-	38	135	-	136	85	-	250+	27	-	29	144	-	141	22	-	65	24	25	28	154	148	142	27	26	79
BF-3	42	-	39	131	-	134	100	-	214	27	-	28	147	-	144	15	-	97	25	-	24	153	-	147	23	-	33
BF-4	31	-	38	135	-	135	42	-	150	25	-	30	141	-	141	40	-	95	26	31	25	144	142	145	35	85	36
Avg.	36	40	38	134	132	135	80	150+	188+	25	39	28	145	136	143	24	150+	75	25	28	26	151	145	145	31	52	52
BR-1	34	-	-	135	-	-	51	-	-	25	-	30	149	-	143	21	-	76	23	27	26	148	145	146	22	43	44
BR-2	30	-	36	140	-	138	23	-	173	23	-	28	149	-	144	11	-	46	26	25	23	144	147	146	23	28	45
BR-3	29	-	38	145	-	136	14	-	191	24	-	33	144	-	140	32	-	72	28	28	25	140	145	146	44	39	34
BR-4	32	-	41	136	-	135	34	-	250+	27	-	36	142	-	137	33	-	181	30	29	33	132	145	139	39	41	131
Avg.	31	36	38	139	135	136	31	150+	205+	25	37	32	146	134	141	24	150+	94	27	27	27	141	145	144	32	38	63

Table A.39 Components of test sections in Montana. (Jennings et al. 1988)

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

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

Table A.40 Physical test data for Montana test road. (Jennings et al. 1988)

Test Section	Components (asphalt/ additive)	AC (%)		Additive (%)		Marshall Stability, Field	Marshall Flow, Field	Voids In-Place (%)	Penetration, 77°F (recovered)	Viscosity, 275°F (cSt)	Ductility, 40°F (cm)
		Design	In-Place	Design	Constructed						
1	C	6.6	6.2			1,637	8	8.9	60	437	9
2	C/lime	6.2	6.5	1.5	1.5	1,876	9	9.5	47	441	7
3	C/antistrip	6.6	6.9	1.0	1.0	1,952	8	9.3	50	440	7
4	C/fly ash	6.2	6.0	1.5	1.9	1,762	8	8.7	63	410	10
5	A/fly ash	6.3	7.2	1.5	1.8	1,801	8	6.2	77	344	26
6	A	6.8	7.6			1,419	8	6.1	69	368	17
7	A/antistrip	6.8	8.0	1.0	0.9	1,502	9	5.8	75	340	24
8	A/lime	6.6	7.2	1.5	1.6	1,967	9	8.5	54	421	8
9	D/lime	6.3	6.7	1.5	1.5	1,874	8	7.6	64	463	7
10	D	6.6	6.6			1,661	9	9.0	78	378	6
11	D/antistrip	6.5	7.1	1.0	0.9	1,432	9	8.9	54	409	6
12	D/fly ash	6.3	6.9	1.5	1.9	1,727	8	7.8	54	437	6
13	Blend	6.5	6.8			1,638	7	8.5	89	263	39
14	B/fly ash	6.2	7.5	1.5	1.8	1,763	8	7.7	62	318	10
15	B	6.8	6.8			1,314	9	6.4	59	361	7
16	B/lime	6.2	6.5	1.5	1.6	1,602	9	10.8	61	328	8
17	B/antistrip	6.8	7.0	1.0	0.9	1,551	9	6.8	84	249	28
18	85/100	6.5	6.4			1,777	8	8.9	52	396	7
20	200-300 Microfil 8	6.2	6.9	1.0	0.9	1,576	8	9.5	57	317	7

Table A.41 Rutting measurements in Montana, March 1987. (Jennings et al. 1988)

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

Table A.42 Crack counts in Montana, March 1987. (Jennings et al. 1988)

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

Table A.43 Summary of aggregate gradations and mix properties in Saskatchewan soft asphalt study. (Culley 1978)

	AC-6 MIX		SC-6 MIX	
	DESIGN	CONSTRUCTION	DESIGN	CONSTRUCTION
Sieve Analysis				
% Passing				
18.0 mm	100	100	100	100
12.5	87.8	89.5	88.9	87.1
5.0	58.2	60.5	59.9	57.2
2.0	36.9	36.9	38.1	36.4
900 μm	22.5	21.6	22.9	19.7
400	16.4	15.0	16.5	13.3
160	9.8	9.1	9.9	9.3
71	7.0	6.8	7.3	8.4
Sand Equiv. %	28.7	34.8	--	34.8
Filler, %	5	5	5	5
Marshall Properties (50B)				
density, kg/m^3	2374	2397	2382	2237
stability, N	5026	--	4648	--
v.m.a., %	15.5	14.4	14.6	14.4
air voids, %	4.6	3.9	5.6	5.8
flow, mm	2	--	2	--
ret. stability, %	78.7	--	--	--
asphalt content, %	5.75	5.70	5.00	4.68
Asphalt Properties				
penetration, 25°C, mm^{-1}		180		451
viscosity, 60°C, Pa.s		73		23
penetration index (McLeod Method)		-0.3		+0.3

Table A.44 Summary of changes in asphalt rheology in Saskatchewan air blown asphalt study. (Clark & Culley 1976)

ASPHALT TYPE	A-100AB			B-150AB			C-150AB			E-100AB			F-AC-5		
ASPHALT AGE	PRE PUGMILL	POST PUGMILL	24 MONTHS	PRE PUGMILL	POST PUGMILL	24 MONTHS	PRE PUGMILL	POST PUGMILL	24 MONTHS	PRE PUGMILL	POST PUGMILL	24 MONTHS	PRE PUGMILL	POST PUGMILL	24 MONTHS
PENETRATION, mm^{-1}															
$4^{o}C$, 200g, 60s	53	21	28	71	31	35	71	29	35	53	18	27	61	27	28
$4^{o}C$, 100g, 5s	13	-	9	17	-	10	16	-	12	14	-	8	11	-	7
$25^{o}C$, 100g, 5s	88	48	46	133	85	62	139	73	58	82	43	42	207	117	67
VISCOSITY, Pa.s															
$15^{o}C, 5x10^{-2}\ s^{-1}$	$2.3x10^6$	$3.5x10^6$	$6.3x10^6$	$1.2x10^6$	$2.2x10^6$	$2.9x10^6$	$1.2x10^6$	$2.0x10^6$	$3.6x10^6$	$2.0x10^6$	$2.9x10^6$	$3.9x10^6$	$1.0x10^6$	$2.3x10^6$	$4.4x10^6$
$38^{o}C, 5x10^{-2}\ s^{-1}$	$9.3x10^4$	$4.9x10^5$	$2.8x10^6$	$4.2x10^4$	$1.1x10^5$	$1.3x10^5$	$4.2x10^4$	$1.3x10^5$	$1.0x10^5$	$1.4x10^5$	$4.5x10^5$	$2.6x10^5$	$2.3x10^4$	$4.0x10^4$	$5.8x10^4$
$60^{o}C$	$0.3x10^3$	$2.9x10^3$	$2.2x10^3$	$0.1x10^3$	$0.4x10^3$	$0.7x10^3$	$0.1x10^3$	$0.6x10^3$	$1.0x10^3$	$0.4x10^3$	$3.0x10^3$	$3.3x10^3$	$0.5x10^2$	$0.1x10^3$	$0.2x10^3$
$135^{o}C$	0.5	1.1	1.2	0.4	0.5	0.7	0.3	0.6	0.8	0.6	1.3	1.2	0.2	0.3	0.3
SOFTENING PT, ^{o}C	43	60	61	39	47	54	38	50	57	46	60	60	36	41	44
DUCTILITY, cm															
$4^{o}C$, 1cm/60s	17	4	3	50+	17	7	50+	9	6	17	4	2	50+	50+	50+
$25^{o}C$, 5cm/60s	150+	33	42	150+	150+	150+	150+	150+	150+	131	25	18	112	150+	150+
STIFFNESS, kPa															
$0^{o}C, 2x10^4 s$ (R)	28		110	6		50	7		42	13		115	9		65
$-40^{o}C, 2x10^4 s$ (N)	97 800		107 600	88 100		117 400	97 800		78 300	48 900		117 400	48 900		479 400

NOTES: 1. All values are averages of 3 tests
2. Stiffness (R) is rheometer value, (N) is nomograph value

Table A.45 Test properties of blown low viscosity asphalts from Western Canadian crude Ex. Manitoba (Richer) trial. (Gaw et al. 1976)

ASPHALT TYPE	BLOWN LOW VISCOSITY (BLV) 150/200 (M)			BLOWN LOW VISCOSITY (BLV) 100/150 (M)		
TEST PROPERTIES	ORIGINAL	AGED ASTM D1754	RECOVERED*	ORIGINAL	AGED ASTM D1754	RECOVERED*
Visc. @ 135°C (275°F), cSt	102	134	114	135	182	153
Visc. @ 60°C (140°F), p	177	433	198	389	993	434
Pen. @ 25°C (77°F)	182	105	160	113	72	100
Pen. @ 5°C (41°F)	19	14	17	15.5	12	14
Wt. Loss on Aging, % w		0.3			0.1	
Retained Pen, % w		58	88		64	89
Ductility @ 25°C (77°F)		76			71	
Solubility in Trichloro, % w	98.8			98.5		
Wax Content, % w	10			10		
% w Asphalt Recovered			4.1			4.5
EXPERIMENTAL STIFFNESS, N/m^2						
@ Temp. 10°C, 0.5h	3.6×10^4			6.6×10^4		
0°C, "	4.2×10^5			7.2×10^5		
-10°C, "	3.5×10^6			5.6×10^6		
-20°C, "	4.0×10^7			3.4×10^7		
T(800 pen), °C	38.3	45.0	39.3	44.5	51.5	46.0
Penetration Index	-1.3	-0.6	-1.2	-0.5	-0.1	-0.4
Nomographic Cracking Temp., °C (CTAA 1974, Nomograph)	-43	-43	-42	-44	-44	-43
Calculated Cracking Temp., °C (calculated from experimental S)	-38			-38		

*Asphalt binder recovered from uncompacted mixes.

Table A.46 Test properties of flashed HV and blown LV asphalts used in northern Ontario (Shebandowan) trial. (Gaw et al. 1976)

TEST PROPERTIES	BLOWN, LOW VISCOSITY 150/200(O)		BLOWN, LOW VISCOSITY 85/100(O)		HIGH VISCOSITY (CONTROL) 150/200(O)		M.T.C. ONTARIO SPECIFICATIONS	
	ORIG.	AGED*	ORIG.	AGED*	ORIG.	AGED*	150/200 Penetration	85/100 Penetration
Visc. @ 135°C (275°F), cSt	104	134	154	211	222	286	190 min.	280 min.
Visc. @ 60°C (140°F), p	201	458	553	1,710	579	1,250		
Pen. @ 25°C (77°F),	188	114	107	71	165	93	150-200	85-100
Pen. @ 5°C (41°F)	21	18	17	14	14	10.5		
Wt. Loss on Aging, % w.		0		0		0.1	1.3 max.	0.85 max.
Retained Pen., %		61		66		56	42 min.	47 min.
Ductility @ 25°C (77°F)		73		30		150*	100 min.**	75 min.**
Ductility @ 4°C (39.2°F)	15		6		50+		10 min.	6 min.
Flash Point (COC), °C	348		327		343		218 min.	232 min.
Solubility in Trichloro, % wt.	98.2		99.1		99.9		99.5 min.	99.5 min.
Experimental Stiffness, N/m^2 @ Temp. -10°C, 0.5h	2.1×10^6	4.2×10^6	3.7×10^6	6.5×10^6	1.2×10^6	2.7×10^6		
$T_{(800\ pen)}$, °C	38.3	45.9	46.5	54.0	38.0	45.1		
Penetration Index	-1.0	0.0	-0.1	-0.7	-1.7	-0.9		
Nomographic Cracking Temp., °C (CTAA 1974 Nomograph)	-45	-47	-47	-48	-39	-38		
Calculated Cracking Temp., °C (calculated from experimental S)	-42		-42		-38			

*Aged according to ASTM D-1754. **Test on aged residue.

Table A.47 Evaluation of recovered asphalts and road mixes from Manitoba (Richer) trial after four years. (Gaw et al. 1976)

ASPHALT TYPE	BLOWN LOW VISCOSITY 150/200 (M)	BLOWN LOW VISCOSITY 100/150 (M)	HIGH VISCOSITY 200/300	HIGH VISCOSITY 300/400	SC-5
<u>ASPHALT PROPERTIES</u>					
Visc. @ 135°C (275°F), cSt	135	242	302	221	111
Visc. @ 60°C (140°F), p	365	2,550	1,540	775	178
Pen. @ 25°C (77°F)	130	56	84	144	460
Pen. @ 5°C (41°F)	16.5	10.5	11	17	45
% w Asphalt Recovered	4.4	4.2	4.6	4.5	3.8
T(800 pen), °C	42.5	56.8	47.1	41.0	29.8
Penetration Index	-0.7	0.6	-0.6	-1.0	-1.5
Nomographic Cracking Temp., °C (CTAA 1974, Nomograph)	-44	-43	-40	-43	-50

*Standard deviation $\sigma = \sqrt{\Sigma (x - \bar{x})^2/(n-1)}$ where $\bar{x}$ is average value and n is number of tests.

Table A.48 Test properties of recovered asphalts from northern Ontario (Shebandowan) trial. (Gaw et al. 1976)

	BLOWN LOW VISCOSITY 150/200 (O)			BLOWN LOW VISCOSITY 85/100 (O)			HIGH VISCOSITY (CONTROL) 150/200 (O)		
	Uncompact. Mix	After 6 months	After 2 years	Uncompact. Mix	After 6 months	After 2 years	Uncompact. Mix	After 6 months	After 2 years
Visc. @ 135°C (275°F), cSt	144	147	156	210	221	248	310	345	402
Visc. @ 60°C (140°F), p	404	471	760	1,570	1,660	3,490	1,330	1,930	3,210
Pen. @ 25°C (77°F)	146	127	98	91	81	63	92	70	57
Pen. @ 5°C (41°F)	20	17.5	16	16	14	13	10.5	8	6.5
Retained Pen, % original	77	68	52	85	76	59	56	42	35
% w Asphalt Recovered	5.0	5.0	4.9	5.0	5.0	4.8	5.2	5.2	5.2
Experimental Stiffness, N/m^2 @ Temp. -10°C, 0.5h	1.6×10^6	1.8×10^6	4.0×10^6	2.5×10^6	2.9×10^6	5.6×10^6	2.5×10^6	3.3×10^6	6.6×10^5
T(800 pen), °C	42.3	43.5	48.2	50.2	51.0	57.0	45.0	47.4	49.4
Penetration Index	-0.4	-0.5	0.1	0.5	0.3	1.0	-1.0	-1.0	-1.0
Nomographic Cracking Temp., °C (CTAA 1974 Nomograph)	-47	-45	-47	-48	-46	-47	-38	-36	-34
Calculated Cracking Temp., °C (Calculated from experiment S)	-	-42	-	-	-43	-	-	-35	-

Table A.49 Properties of recovered asphalts in Alberta. (Anderson et al. 1966)

Code	Penetration			Ductility		Absolute Viscosity—Poise								
						140 F. (× 10^3)			77 F. (× 10^5)			39.2 F. (× 10^8)		
	77 F.(a)	39.2 F.(b)	R(c) %	77 F. cm.(d)	39.2 F. cm.	10^{-3} sec.$^{-1}$	10^{-1} sec.	"c"	10^{-3} sec.$^{-1}$	10^{-1}	"c"	10^{-3} sec.$^{-1}$	10^{-1}	"c"
A-1	94	34	36	-	-	8.98	2.63	0.78	13.3	9.60	0.96	5.30	1.91	0.80
2	126	32	25	-	-	2.56	0.92	0.84	11.1	4.10	0.75	3.77	1.61	0.81
3	112	27	24	-	-	2.72	1.15	0.81	25.6	5.94	0.69	4.92	1.21	0.70
B-1	64	35	55	150+	51 (e)	20.3	6.02	0.74	39.1	40.1	1.00	13.7	3.45	0.70
2	65	35	54	150+	52 (e)	18.8	5.24	0.72	31.4	24.4	0.94	10.2	3.50	0.77
C-1	111	44	40	-	-	9.36	2.67	0.83	46.5	7.40	0.80	3.36	1.29	0.80
2	82	32	39	-	-	27.1	3.90	0.77	20.6	14.1	0.90	7.80	2.10	0.78
D-1	95	27	29	-	-	1.41	0.84	0.95	11.8	6.99	0.93	9.93	1.24	0.79
E-1	53	23	43	-	-	6.45	4.81	0.93	81.1	41.6	0.84	17.1	3.00	0.56
2	77	23	30	-	-	3.33	1.59	0.92	24.5	11.0	0.87	7.68	2.77	0.60
3	94	38	40	-	-	3.10	2.13	0.98	16.9	11.9	0.93	5.13	1.97	0.79
F-1	85	27	32	150+	8 (f)	0.72	0.67	0.99	12.3	10.8	0.98	10.7	1.40	0.56
2	70	30	43	150+	9 (f)	1.18	1.12	0.99	15.3	14.8	0.98	11.3	1.06	0.44
3	90	36	40	150+	12 (f)	0.73	1.31	-	9.20	9.20	1.00	8.59	0.84	0.49
G-2	134	36	27	-	-	12.0	2.20	0.63	21.7	4.30	0.60	5.58	3.08	0.91
4	150	32	21	-	-	25.0	4.60	0.62	24.0	3.30	0.65	3.60	2.61	0.94
H-1	94	27	29	120	3 (e)	6.25	1.52	0.61	7.28	4.55	0.91	7.62	1.10	0.60
2	80	42	52	150+	7 (e)	6.68	3.48	0.88	16.6	13.6	0.92	6.20	1.81	0.74
J-2	93	29	31	-	-	24.2	3.40	0.65	13.0	10.0	0.93	7.30	4.80	0.88
6	95	34	36	-	-	61.5	6.60	0.50	13.0	9.30	0.91	7.80	3.90	0.86
K-1	102	39	38	-	-	6.47	2.67	0.87	12.2	8.54	0.93	4.39	1.25	0.72
2	85	33	39	-	-	5.33	2.86	0.86	42.3	21.4	0.90	17.9	1.37	0.62
3	62	29	47	-	-	15.3	7.55	0.72	57.6	28.0	0.83	11.6	3.33	0.70
L-1	94	22	23	-	-	1.74	1.10	0.95	36.1	12.3	0.80	10.0	1.80	0.63
2	89	34	38	-	-	3.75	2.50	0.84	28.9	16.4	0.91	4.68	1.80	0.80
M-1	120	27	22	-	-	14.55	1.58	0.61	15.4	6.20	0.74	12.3	5.09	0.74
2	121	29	24	-	-	8.22	1.91	0.65	17.3	7.09	0.85	14.9	6.46	0.77

(a) ASTM-D5-52.
(b) 39.2 F., 200 gms., 60 sec., 0.1 mm.
(c) Ratio: (Pen. @ 39.2 F. ÷ Pen. @ 77 F.) × 100.
(d) ASTM: D113-44.
(e) 1 cm./min. elongation.
(f) 5 cm./min. elongation.

(RCA-HD 4.2.66)

Table A.50 Transverse cracking investigation in Alberta; field sampling areas. (Anderson et al. 1966)

Route	Test Area Mi. to Mi.		Cracks per Mile Max.	Av.	Min.	Thickness (ins.) Surf.	Base(a)	Total	Age of Surface(b)	Asphalt Ref.(c)	Pen. Gr.	Code
2-D-	0.00	8.13	11	8	0	3.7	13.0	16.7	6 mos.	3	150/200	A-1
1/1, 1/2, 1/3	8.13	17.31	187	89	40	4.2	12.7	16.9	6 mos.	1	150/200	2
	17.31	24.97	27	7	0	4.4	12.6	17.0	6 mos.	1	150/200	3
2-G-4	219.5	220.0	-	0	-	4.0	9.9	13.9	6 yrs.	7	150/200	B-1
	225.0	225.5	-	80	-	4.5	9.9	14.4	6 yrs.	7	150/200	2
12-B-2	47.4	63.7	-		-	5.2	10.3	15.5	7 yrs.	8	150/200	C-1
	63.7	77.4	142	75	2	4.2	8.3	12.5	5 yrs.	4	150/200	2
15-A-2	30.0	31.0	-	310	-	4.0	14.0	18.0(d)	5 yrs.	1	150/200	D-1
18-A-1	5.0	6.0	-	80	-	3.0	6.0	9.0(d)	7 yrs.	8	150/200	E-1
	11.0	12.0	-	240	-	4.0	12.0	16.0(d)	3 yrs.	1	150/200	2
	25.0	26.0	-	0	-	4.0	14.0	18.0(d)	2 yrs.	5	150/200	3
21-A-1	24.21	26.13	-	229	-	4.9	9.3	14.1				F-1
	26.13	26.34	-	0	-	4.5	10.0	14.5	3 yrs.	10	150/200	2
	29.00	29.38	-	72	-	5.1	9.5	14.6				3
34-A-1	9.0	10.0	-	540	-	4.4	10.2	14.6	8 mos.	1(f)	200/300	G-2
	20.0	21.0	-	45	-	5.0	11.6	16.6				4
34-A-2	45.0	45.5	-	158	-	4.4	12.9	17.3	3 yrs.	1	150/200	H-1
	58.0	58.5	-	5	-	4.1	11.9	16.0	3 yrs.	5	150/200	2
36-D-1	6.0	7.0	-	131	-	5.2	7.1	12.3	8 mos.	3	150/200	J-2
	17.0	18.0	-	0	-	4.6	7.4	12.0				6
39-A-1	1.0	2.0	-	55	-	4.0	14.0	18.0(d)	8 yrs.	8	150/200	K-1
	9.0	10.0	-	475	-	4.0	16.0	20.0(d)				2
	19.0	20.0	-	15	-	4.0	14.0	18.0(d)	4 yrs.	2	150/200	3
57-A-1/2	12.55	14.54	-	348	-	4.3	6.7	11.0	4 yrs.	1	150/200	L-1
	14.55	16.55	-	0	-	4.1	8.7	12.8	3 yrs.	5	150/200	2
62-A	14.0	15.0	-	20	-	2.5	7.6(e)	10.1	8 mos.	10	150/200	M-1
	20.0	21.0	-	10	-	2.1	7.7(e)	9.8				2

(a) Includes nominal 2" coldmix, asphalt bound base.
(b) Surface age at time of survey.
(c) Refinery source.
(d) Nominal thickness.
(e) No asphalt bound base.
(f) 2" Surface Course. 2" Binder course HPMS (5-150/200) 1962. Uncracked prior to surfacing.

(RCA-HD 1.2.66.)

Table A.51 Summary of asphalt cement quality tests, Alberta, 1966. (Anderson & Shields, 1971)

Supplier	Condition	Penetration (1/10 mm) 77 F	39.2 F	32 F	Softening Point[a] (deg F)	Viscosity 275 F (centistoke)	140 F[b] (poise x 10^{0})	77 F[c] (poise x 10^{5})	39.2 F[c] (poise x 10^{8})	Penetration Index	Penetration Retained (percent)
1	Supply	265	68	40	103	139	237	3.0	1.2	+1.0	22.3
	σ	(28)	(8)	(4)	(7)	(8)	(26)				
	TFOT	177	43	22	108	–	358	8.6	2.5	+0.2	24.3
	σ	(12)	(7)	(2)	(1)		(60)				
	Mix	138	34	23	109	204	445	5.9	3.8	-1.0	25.3
	σ	(16)	(3)	(2)	(2)	(15)	(63)				
2	Supply	215	62	38	101	217	610	2.8	1.0	-1.0	31.2
	σ	(8)	(3)	(2)	(2)	(3)	(81)				
	TFOT	125	41	25	110	–	1,111	7.8	2.4	-0.8	32.8
	σ	(3)	(2)	(1)	(4)		(119)				
	Mix	113	41	27	111	309	1,415	13.4	3.4	-1.0	37.8
	σ	(9)	(4)	(2)	(3)	(20)	(350)				
3	Supply	217	58	35	102	184	544	2.3	0.8	-0.8	28.1
	σ	(15)	(4)	(3)	(2)	(10)	(36)				
	TFOT	112	38	24	110	–	1,127	24.3	2.0	-0.9	34.0
	σ	(7)	(1)	(1)	(1)		(212)				
	Mix	102	38	26	111	271	1,217	14.5	4.5	-1.3	36.3
	σ	(13)	(5)	(2)	(2)	(31)	(403)				

[a] ASTM D 36 26 [b] ASTM D 2171-66. [c] Sliding plate microviscometer at shear rate of 10^{-3} sec^{-1}.

Table A.52 Changes in asphalt properties with time at Alberta. (Anderson & Shields 1971)

Supplier	Age	Asphalt Content (percent)	Unit Weight (pcf)	Physical Properties of Asphalt Cement: Penetration (1/10 mm) 77 F	Penetration (1/10 mm) 39.3 F	Softening Point (deg F)	Absolute Viscosity at 140 F (poise)	Viscosity at 77 F (poise x 10^5) 10^{-3} sec^{-1}	Viscosity at 77 F (poise x 10^5) 10^{-1} sec^{-1}
1	As supplied			265	68	103	237	4.1	2.2
	After TFOT			117	43	108	358	17.9	4.1
	After plant mixing and spreading	5.2	138.2	138	34	109	446	15.9	4.0
	12 mo.	5.9	146.8	191	41	105	258	7.3	2.3
	24 mo.	5.75	146.6	196	44	102	258	2.8	2.0
	34 mo.	5.5	147.9	144	29	107	343		
2	As supplied			215	62	101	610	3.0	2.6
	After TFOT			125	41	110	1,111	9.2	6.7
	After plant mixing and spreading	5.3	136.7	113	41	111	1,460	22.5	7.8
	12 mo.	5.7	142.2	136	41	107	860	8.8	5.0
	24 mo.	5.4	143.1	127	38	107	954	6.2	4.9
	34 mo.	5.4	144.1	105	30	111	1,281		
3	As supplied			217	58	102	544	2.6	2.6
	After TFOT			112	38	110	1,127	12.1	10.2
	After plant mixing and spreading	4.4	134.2	102	38	111	1,217	15.2	8.6
	12 mo.	5.5	139.4	116	36	110	844	12.9	6.1
	24 mo.	5.3	141.6	90	28	113	1,215	11.7	10.6
	34 mo.	4.3	142.3	70	26	117	1,801		

Table A.53 Asphalt test properties in Pennsylvania. (Kandhal & Wenger 1975)

Property	Asphalt 1	2	3	4	5	6
Before Pug Mill Mixing						
Viscosity, poises						
At 140 F	1613	1544	1447	966	2200	2649
At 275 F[a]	339.6	343.0	475.4	318.5	509.4	556.7
At 39.2 F at 0.05 sec^{-1}	1.19×10^{8}	2.65×10^{8}	4.22×10^{7}	9.50×10^{7}	1.68×10^{8}	2.57×10^{8}
At 77 F at 0.05 sec^{-1}	3.05×10^{6}	1.06×10^{6}	4.83×10^{5}	9.15×10^{5}	1.32×10^{6}	1.85×10^{6}
At 115 F at 0.05 sec^{-1}	2.09×10^{4}	1.54×10^{4}	1.15×10^{4}	1.15×10^{4}	2.19×10^{4}	2.80×10^{4}
Shear susceptibility						
At 39.2 F	0.20	0.20	0.35	0.45	0.32	0.32
At 77 F	0.05	0.02	0.11	0.06	0.12	0.09
At 115 F	0.02	0.03	0.04	0.03	0.04	0.02
Penetration						
At 39.2 F, 200 g, 5 sec	9	11	28	19	15	12
At 77 F, 100 g, 5 sec	62	92	149	114	94	80
After Pug Mill Mixing						
Penetration at 77 F, 100 g, 5 sec	36	69	98	66	69	60
Penetration, percentage retained	58	76	67	58	73	76
Viscosity at 140 F, poises	3645	2505	2971	2078	3463	4770
Viscosity ratio at 140 F	2.27	1.62	2.06	2.16	1.57	1.80
Aging index[b] after mixing based on viscosity at 77 F, 0.05 sec^{-1} shear rate	3.3	1.9	2.5	3.3	2.1	1.9
Viscosity at 77 F, 0.05 sec^{-1} determined from aging indexes, poises	1.01×10^{7}	2.01×10^{6}	1.21×10^{6}	3.02×10^{6}	2.77×10^{6}	3.52×10^{6}

Note 1 poise = 0.1 Pa s, 1 F = 1.8 C + 32.

[a] In centistokes.

[b] No test data available. These were determined from Fig. 4 of the paper by Halstead and Zenewitz (ASTM, STP, 309, 1962), which gives relation between percent of original penetration and viscosity aging index for thin film test.

Table A.54 Pavement condition after 113 months in Pennsylvania (Kandhall & Wenger 1975)

ITEM OBSERVED	ASPHALT					
	1	2	3	4	5	6
Riding Quality	Good	Good	Good	Good	Good	Good
Raveling (loss of aggregate larger than ¼ in.)	Moderate	Slight	Slight	Slight to moderate	Slight	Slight to moderate
Spalling	Slight	None	None	None to slight	None to slight	None to slight
Loss of matrix (loss of fines)	Moderate	Slight	Slight	Slight	Slight	Slight
Rutting (in.)	0.20	0.18	0.20	0.19	0.21	0.21
Transverse cracking	Slight	None to slight	None	None to slight	None to slight	None to slight
Longitudinal cracking	Slight to moderate	None	None	None	None	None
Alligator cracking	None	None	None	None	None	None
Surface texture	Average	Closed	Closed	Closed	Closed	Closed to average

Note: 1 in. = 25 mm.

Table A.55 Physical properties of the experimental AC-20 grade asphalt cements in Texas. (Adams & Holmgreen 1986)

	A	B	C	D	E
Viscosity					
77°F, poises X10^6	3.55	1.55	1.70	1.11	2.25
140°F, poises	2242	3012	2183	1812	1911
275°F, poises	6.42	5.33	3.22	4.09	3.10
Penetration					
39.2°F, dmm, 200 gm, 60 sec	13	26	18	26	13
77°F, dmm, 100 gm, 5 sec	61	64	58	81	45
Softening Point, °F	123	125	121	122	119
Thin Film Oven Residue					
Percent Loss	0	0.04	0.05	0.08	0.15
Viscosity, 140°F, poises	4681	5008	4017	3236	4285
Penetration, 77°F, dmm	41	53	32	56	32
Rolling Thin Film Oven Residue					
Percent Loss	0	0	0	0.11	0.18
Viscosity, 140°F, poises	5180	7350	5356	6369	5345
Penetration, 77°F, dmm	32	50	24	42	29

Table A.56 Physical properties of the experimental AC-10 cements. (Adams & Holmgreen 1986)

	A	B	C	D	E
Viscosity					
77°F, poises X10^6	0.66	0.22	0.66	0.50	0.88
140°F, poises	973	773	1268	931	955
275°F, poises	2.76	2.76	2.85	3.18	2.34
Penetration					
39.2 F, dmm, 200gm, 60 sec	20	35	16	35	17
77 F, dmm, 100 gm, 5 sec	106	166	80	113	80
Softening Point, °F	111	106	114	116	115
Thin Film Oven Residue					
Percent Loss	0	0.04	0.03	0.13	0.37
Viscosity, 140°F, poises	1280	1210	2539	2381	2436
Penetration, 77°F, dmm	69	121	50	68	42
Rolling Thin Film Oven Residue					
Percent Loss	0	0	0	0.01	0.51
Viscosity, 140°F, poises	1663	1608	2911	2865	4886
Penetration, 77°F, dmm	62	113	37	59	28

Table A.57 Physical properties of the virgin asphalt cements delivered to the Dickens, Texas site. (Adams & Holmgreen 1986)

Asphalt Producer	Asphalt Grade	Viscosity, poises			Penetration, dmm	
		77 °F poises X10^6	140 °F	275 °F	39.2 °F 200g, 60s	77 °F 100g, 5s
A	AC-20	4.00	2175	7.15	8	65
A	AC-10	1.35	1220	4.51	15	95
B	AC-20	1.20	2523	4.64	27	77
C	AC-20	2.75	2576	3.55	7	43
D	AC-20	2.50	2151	4.53	22	69
E	AC-20	1.90	1515	2.87	9	53
E	AC-10	1.15	1264	2.55	15	72

Table A.58 Physical properties of the thin film oven residue of the virgin asphalt cements delivered to the Dickens, Texas site. (Adams & Holmgreen 1986)

Asphalt Producer	Asphalt Grade	Percent Loss	Viscosity, poises 140°F	Penetration, dmm 77°F
A	AC-20	0	3536	45
A	AC-10	0.02	2477	55
B	AC-20	0.14	5285	53
C	AC-20	0	5223	28
D	AC-20	0.01	5169	45
E	AC-20	0.04	5006	27
E	AC-10	0.22	2673	41

Table A.59 Physical properties of the asphalt cement recovered from field mixed, laboratory compacted from Dickens, Texas site. (Adams & Holmgreen 1986)

Asphalt Producer	Asphalt Grade	Viscosity, poises			Penetration, dmm	
		77°F poises X10^6	140°F	275°F	39.2°F 200 gm, 60 sec	77°F 100g, 5s
A	AC-20	18.4	9564	11.03	14	20
A	AC-10	12.0	2000	5.93	21	62
B	AC-20	7.0	11250	8.30	20	38
C	AC-20	16.0	8668	6.06	8	21
D	AC-20	1.40	11355	8.74	21	32
E	AC-20	14.0	4750	4.49	9	29
E	AC-10	1.3	4322	4.16	8	28

Table A.60 Physical properties of the virgin asphalt cements delivered to the Dumas, Texas site. (Adams & Holmgreen 1986)

Asphalt Producer	Asphalt Grade	Viscosity, poises			Penetration, dmm	
		77°F poises X10^6	140°F	275°F	39.2°F 200g, 60s	77°F 100g, 5s
A	AC-20	1.90	2155	6.39	16	61
A	AC-10	0.56	958	4.65	16	104
B	AC-10	0.36	961	3.63	39	133
C	AC-10	0.83	1388	3.06	15	74
D	AC-10	0.53	1030	3.21	30	105
E	AC-20	1.60	2354	3.17	10	54
E	AC-10	0.97	1038	2.48	16	71

Table A.61 Physical properties of the thin film oven residue of the virgin asphalt cements delivered to the Dumas, Texas site. (Adams & Holmgreen 1986)

Asphalt Producer	Asphalt Grade	Percent Loss	Viscosity, poises 140°F	Penetration, dmm 77°F
A	AC-20	0.02	4344	44
A	AC-10	0.07	2053	67
B	AC-10	0.05	2057	86
C	AC-10	0.14	2716	47
D	AC-10	0.05	2592	63
E	AC-20	0.12	4016	32
E	AC-10	0.02	2502	41

Table A.62 Physical properties of the asphalt cement recovered from field mixed, laboratory compacted from Dumas, Texas site. (Adams & Holmgreen 1986)

Asphalt Producer	Asphalt Grade	Viscosity, poises			Penetration, dmm	
		77 F poises $X10^6$	140 F	275 F	39.2 F 200gm, 60 sec	77 F 100g, 5s
A	AC-20	3.4	2984	7.13	12	51
A	AC-10	1.6	1723	5.24	20	75
B	AC-10	0.56	1360	3.49	57	107
C	AC-10	2.9	2995	3.86	7	45
D	AC-10	1.5	1989	3.98	26	66
E	AC-20	5.0	2374	3.42	12	41
E	AC-10	2.0	1943	3.08	12	47

Table A.63 Physical properties of the virgin asphalt cements delivered to the Lufkin, Texas site. (Adams & Holmgreen 1986)

Asphalt Producer	Asphalt Grade	Viscosity, poises			Penetration, dmm	
		77°F poises X10^6	140°F	275°F	39.2°F 200gm, 60s	77°F 100g, 5s
A	AC-20	1.80	1728	5.05	16	70
B	AC-10	0.76	932	3.63	25	95
C	AC-20	1.55	1811	3.19	6	64
D	AC-20	0.96	1913	3.96	23	79
D	AC-10	0.42	1040	2.88	33	111
E	AC-20	1.90	1858	2.82	10	58

Table A.64 Physical properties of the thin film oven residue of the virgin asphalt cements delivered to the Lufkin, Texas site. (Adams & Holmgreen 1986)

Asphalt Producer	Asphalt Grade	Percent Loss	Viscosity, poises 140° F	Penetration, dmm 77° F
A	AC-20	0.28	4534	46
B	AC-20	1.08	4003	45
C	AC-20	0.08	3867	34
D	AC-20	0.07	4210	47
D	AC-10	1.23	4600	30
E	AC-20	0.47	3360	52

Table A.65 Physical properties of the asphalt cement recovered from field mixed, laboratory compacted from Lufkin, Texas site. (Adams & Holmgreen 1986)

Asphalt Producer	Asphalt Grade	Viscosity, poises			Penetration, dmm	
		77 °F poises X10^6	140°F	275° F	39.2°F 200 gm, 60s	77°F 100g,5s
A	AC-20	4.5	3781	7.45	15	52
B	AC-20	3.45	3600	5.46	22	57
C	AC-20	3.3	2939	4.40	9	48
D	AC-20	4.1	5940	5.77	5	45
D	AC-10	1.28	1866	3.90	19	73
E	AC-20	9.0	4307	4.25	4	28

Table A.66 Physical properties of the asphalt cement recovered from cores after one week service from Lufkin, Texas site. (Adams & Holmgreen 1986)

Asphalt Producer	Asphalt Grade	Viscosity, poises			Penetration, dmm	
		77°F poises $X10^6$	140°F	275°F	39.2°F 200g, 60s	77°F 100g, 5s
A	AC-20	3.55	3735	6.92	10	48
B	AC-10	3.70	3891	6.26	7	56
C	AC-20	3.80	2754	3.87	10	46
D	AC-20	3.80	2975	4.98	12	52
D	AC-10	1.20	2418	4.44	16	63
E	AC-20	14.50	8790	5.06	4	23

Table A.67 Physical properties of the asphalt cement recovered from field mixed, field compacted samples from Lufkin, Texas after one year service. (Adams & Holmgreen 1986)

Asphalt Producer	Asphalt Grade	Viscosity, poises			Penetration, dmm	
		77°F poises x 10^6	140°F	275°F	39.2°F 200g, 60s	77°F 100g, 5s
A	AC-20	4.8	5553	1.21	0	20
B	AC-10	23.0	8666	6.69	3	20
C	AC-20	26.0	8970	6.16	0	15
D	AC-10	2.3	1866	3.99	17	68
D	AC-20	24.0	8400	6.40	0	19
E	AC-20	7.1	5190	6.34	9	38

Table A.68 Physical properties of the asphalt cement recovered from cores after one year service from Dickens, Texas site. (Adams & Holmgreen 1986)

Asphalt Producer	Asphalt Grade	Viscosity, poises			Penetration, dmm	
		77°F poises X10^6	140°F	275°F	39.2°F 200gm, 60sec	77°F 100g, 5s
A	AC-20	12.0	12300	11.76	5	25
A	AC-10	17.0	12439	7.29	15	37
B	AC-20	21.0	9787	5.67	3	17
C	AC-20	8.0	5523	8.08	8	32
D	AC-20	21.0	8670	5.79	2	20
E	AC-20	30.0	15466	7.45	0	18
E	AC-10	18.5	23115	9.70	10	21

Table A.69 Physical properties of the asphalt cement recovered from field compacted samples from Dickens, Texas. (Adams & Holmgreen 1986)

Asphalt Producer	Asphalt Grade	Viscosity, poises			Penetration, dmm	
		$77^{o}F$ poises x 10^{6}	$140^{o}F$	$275^{o}F$	$39.2^{o}F$ 200g, 60s	$77^{o}F$ 100g, 5s
A	AC-10	13.0	8418	6.65	12	31
A	AC-20	18.5	11.2	10.26	16	27
B	AC-20	19.0	18.7	9.90	13	28
C	AC-20	20.5	13816	9.00	12	26
D	AC-20	29.0	15.0	7.79	3	17
E	AC-10	33.0	10682	5.94	5	16
E	AC-20	40.0	17.1	7.54	2	16

Table A.70 Physical properties of the asphalt cement recovered from cores after one year service from Dumas, Texas. (Adams & Holmgreen 1986)

Asphalt Producer	Asphalt Grade	Viscosity, poises			Penetration, dmm	
		77°F poises X10^6	140°F	275°F	39.2°F 200gm, 60sec	77°F 100g, 5s
A	AC-20	5.8	4468	8.41	10	41
A	AC-10	2.0	2263	4.81	15	57
B	AC-10	0.7	1453	3.59	30	90
C	AC-10	2.06	2477	4.41	15	62
D	AC-10	1.28	1930	4.04	26	71

Table A.71 Physical properties of the asphalt cement recovered from field compacted samples from Dumas, Texas after two years service. (Adams & Holmgreen 1986)

Asphalt Producer	Asphalt Grade	Viscosity, poises			Penetration, dmm	
		$77^{o}F$ poises x 10^{6}	$140^{o}F$	$275^{o}F$	$39.2^{o}F$ 200g, 60s	$77^{o}F$ 100g, 5s
A	AC-10	1.2	2573	6.89	16	62
A	AC-20	8.8	7511	10.20	12	43
B	AC-10	2.0	2106	4.58	83	25
D	AC-10	3.2	2625	4.94	18	64
E	AC-10	3.9	4244	4.50	5	30
E	AC-20	50.0	36230	7.13	5	20

Table A.72 Chemical composition of original asphalts (Rostler-Sternberg Analysis). (Adams & Holmgreen 1986)

Asphalt Producer	Grade	Rostler-Sternberg Fraction, percent: Pentane Asphaltenes (A)	Nitrogen Bases (N)	1st Acidaffins (A_1)	2nd Acidaffins (A_2)	Paraffins (P)	Rostler Parameter, $\frac{N + A_1}{P + A_2}$	$\frac{N}{P}$
E	AC-10	17	21	21	22	19	1.02	1.11
	AC-20	16	15	24	27	18	0.86	0.83
A	AC-10	5	--*	--	--	--	--	--
	AC-20	4	--*	--	--	--	--	--
C	AC-10	14	20	20	26	20	0.87	1.00
	AC-20	15	20	21	29	15	0.93	1.33
B	AC-10	19	18	16	27	20	0.72	0.90
	AC-20	21	13	16	29	21	0.58	0.62
D	AC-10	20	13	21	27	19	0.74	0.68
	AC-20	24	16	12	28	20	0.58	0.80

*Asphalts from Refinery A typically do not give satisfactory results from the Rostler-Sternberg analysis.

Table A.73 Chemical composition of original asphalts (Corbett). (Adams & Holmgreen 1986)

Asphalt Producer	Grade	Corbett Fraction			
		Asphaltenes*	Saturates	Naphthene Aromatics	Polar Aromatics
E	AC-10	11	14	45	30
	AC-20	12	10	43	35
A	AC-10	2	6	49	43
	AC-20	2	2	52	44
C	AC-10	11	9	45	35
	AC-20	11	9	47	33
B	AC-10	10	16	56	18
	AC-20	17	11	47	25
D	AC-10	15	12	47	26
	AC-20	18	9	42	31

*Insoluble in heptane.

Table A.74 Temperature susceptibility of asphalts in Texas. (Adams & Holmgreen 1986)

Asphalt Producer	Asphalt Grade	Penetration Ratio	Penetration Index	Pen/Vis No.			
				77°F to 275°F	77°F to 140°F		
		Original	Original	Original	Original	TFOT	RTFOT
E	AC-10	21	-1.09	-1.31	-1.16	-1.14	-1.03
	AC-20	29	-1.84	-1.44	-1.28	-0.97	-0.90
A	AC-10	19	-0.97	-0.71	-0.70	-1.07	-0.97
	AC-20	21	-0.59	-0.09	-0.69	-0.55	-0.80
C	AC-10	20	-1.26	-1.02	-0.86	-0.86	-1.14
	AC-20	31	-0.99	-1.15	-0.79	-1.03	-1.14
B	AC-10	21	-0.38	-0.22	-0.19	-0.24	-0.05
	AC-20	41	-0.19	-0.34	-0.32	-0.11	+0.18
D	AC-10	31	+0.13	-0.47	-0.65	-0.46	-0.50
	AC-20	32	+0.04	-0.47	-0.47	-0.45	-0.23

Table A.74 (continued).

Asphalt Producer	Asphalt Grade	Viscosity-Temperature Susceptibility 77 to 140°F			Temperature of Equal stiffness °F(°C)
		77 to 140°F	140 to 275°F	77 to 275°F	
		Original	Original	Original	Original
E	AC-10	4.21	3.66	3.85	-57 (-50)
	AC-20	4.13	3.70	3.85	-43 (-41)
A	AC-10	4.05	3.52	3.71	-65 (-54)
	AC-20	4.22	3.18	3.55	-58 (-50)
C	AC-10	3.84	3.60	3.69	-57 (-49)
	AC-20	3.90	3.72	3.79	-55 (-48)
B	AC-10	3.67	3.42	3.51	-81 (-63)
	AC-20	3.62	3.44	3.50	-65 (-54)
D	AC-10	3.94	3.38	3.58	-80 (-62)
	AC-20	3.83	3.45	3.58	-72 (-58)

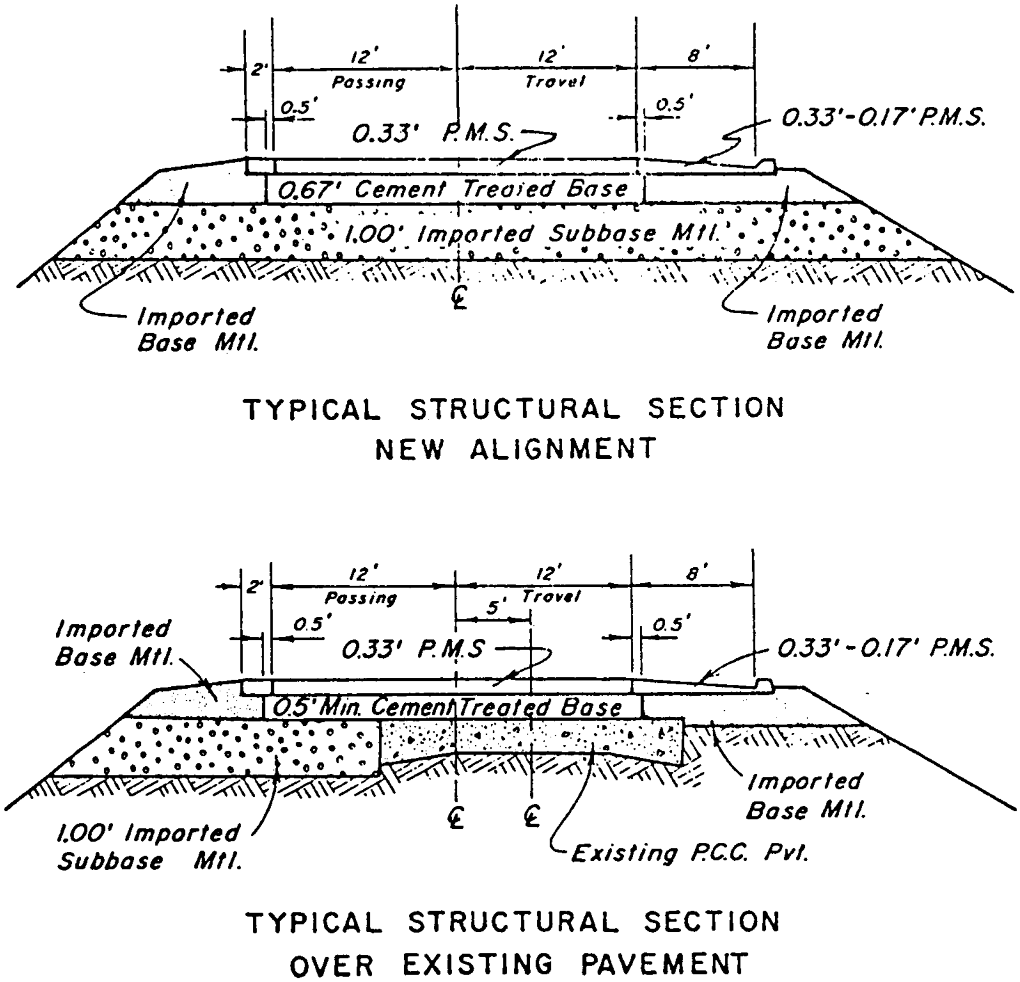


Figure A.1 Typical structural section, Zaca-Wigmore.
(Hveem et al. 1959)

Figure A.2 Bitumen Test Data Chart for Pennsylvania asphalt T-1.
(Kandhal et al. 1984)

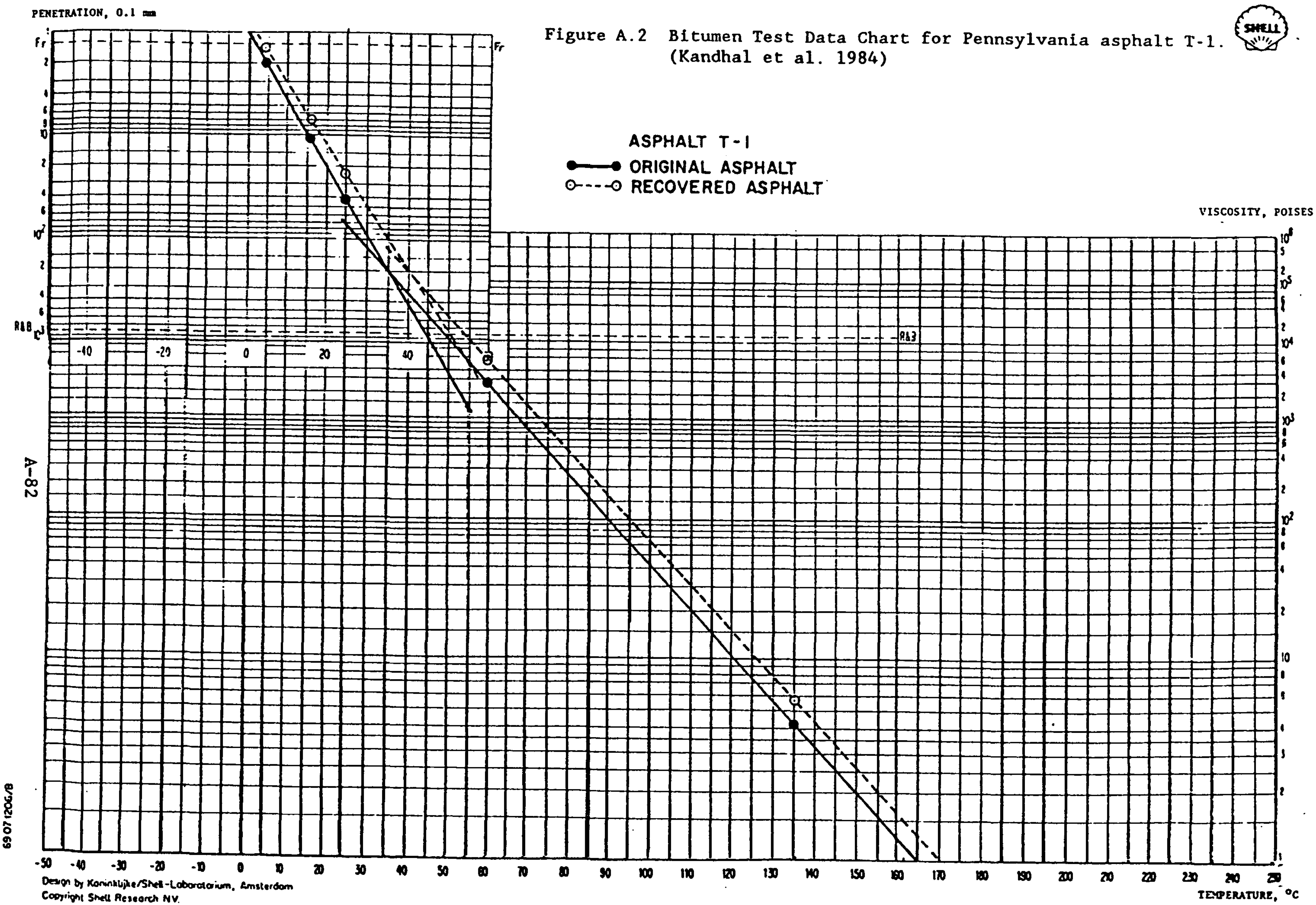


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Figure A.3 Bitumen Test Data Chart for Pennsylvania asphalt T-2. (Kandhal et al. 1984)

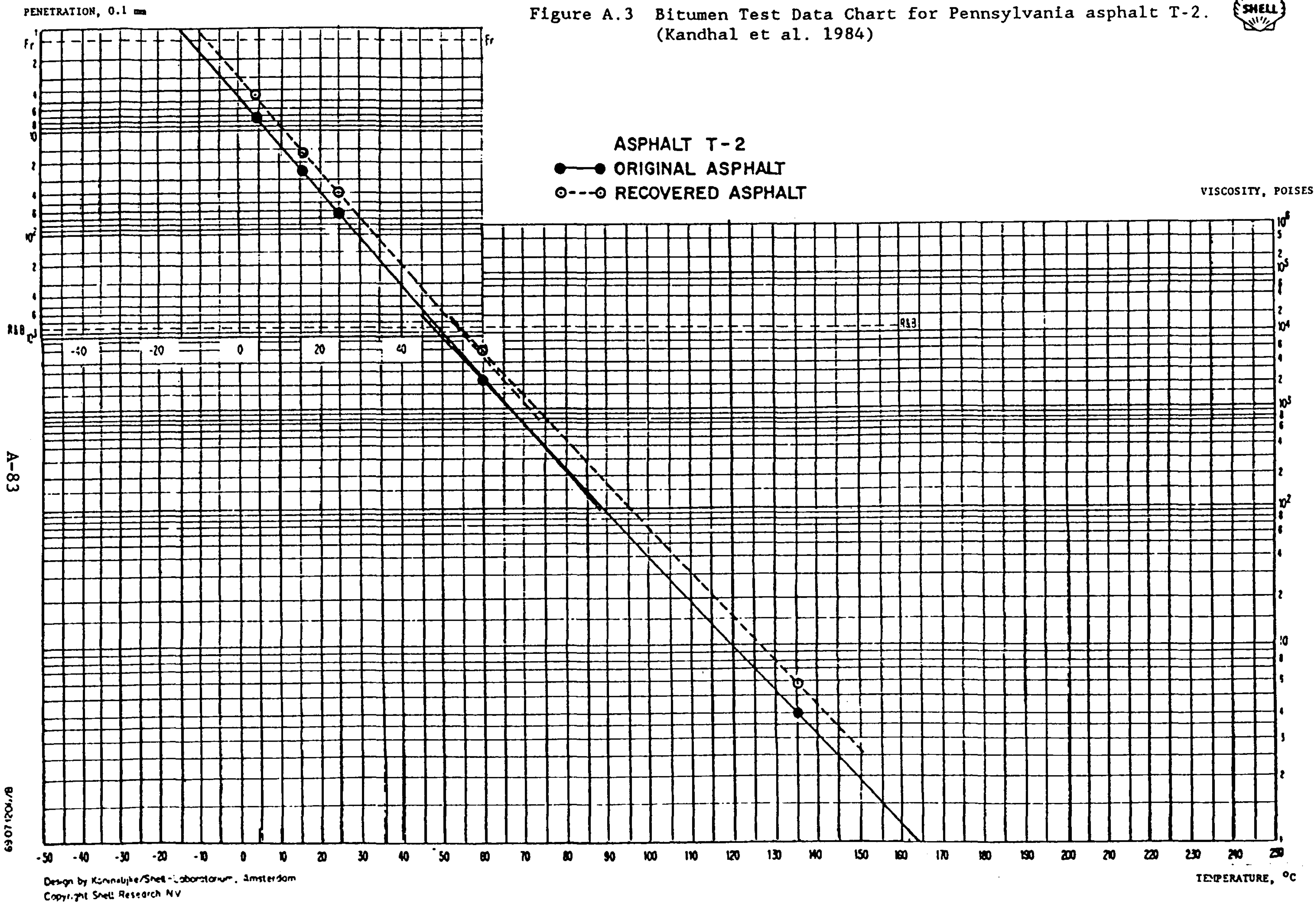

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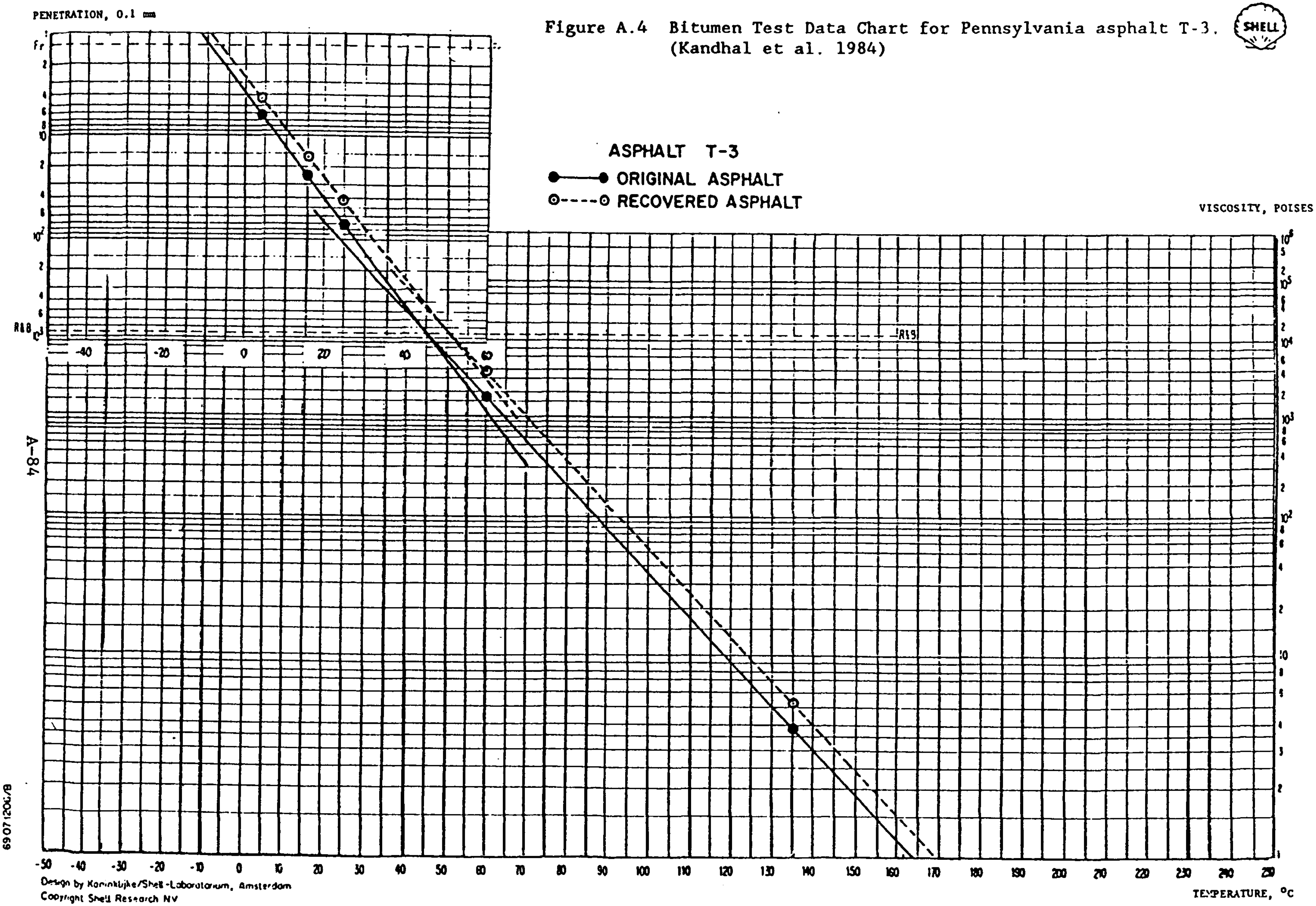
Figure A.4 Bitumen Test Data Chart for Pennsylvania asphalt T-3. (Kandhal et al. 1984)



6907 1206/B

Figure A.5 Bitumen Test Data Chart for Pennsylvania asphalt T-4. (Kandhal et al. 1984)

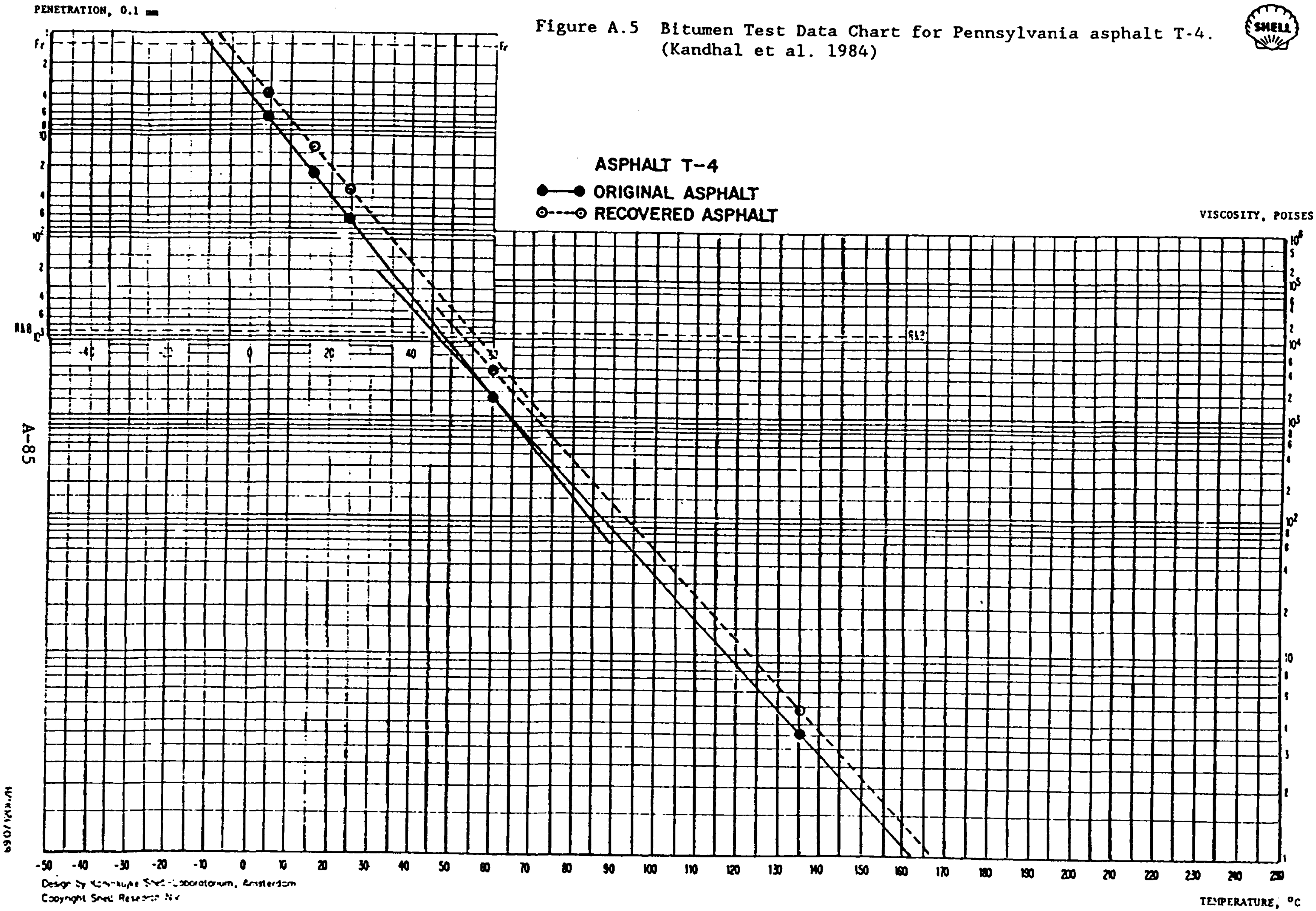


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Figure A.6 Bitumen Test Data Chart for Pennsylvania asphalt T-5. (Kandhal et al. 1984)

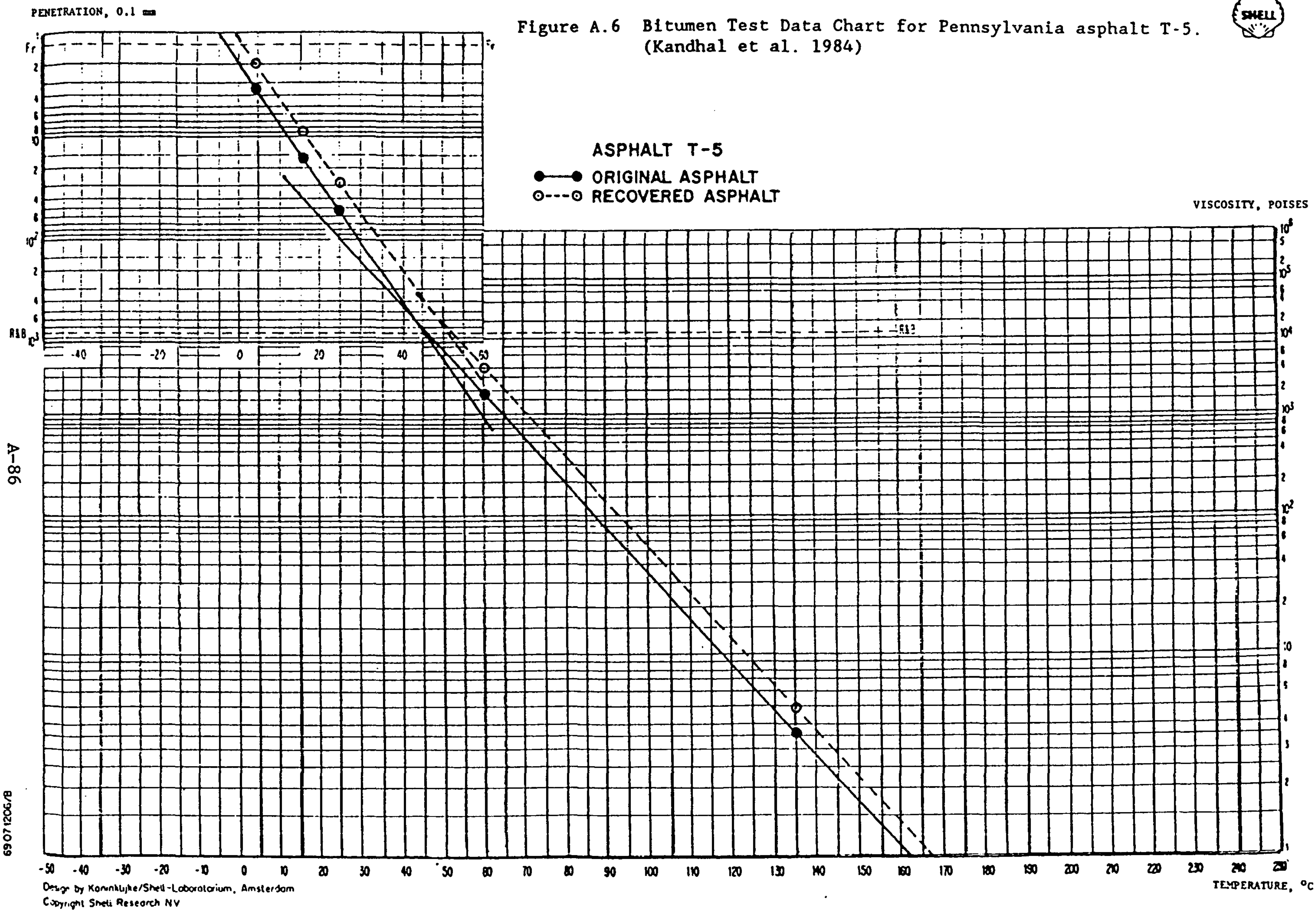

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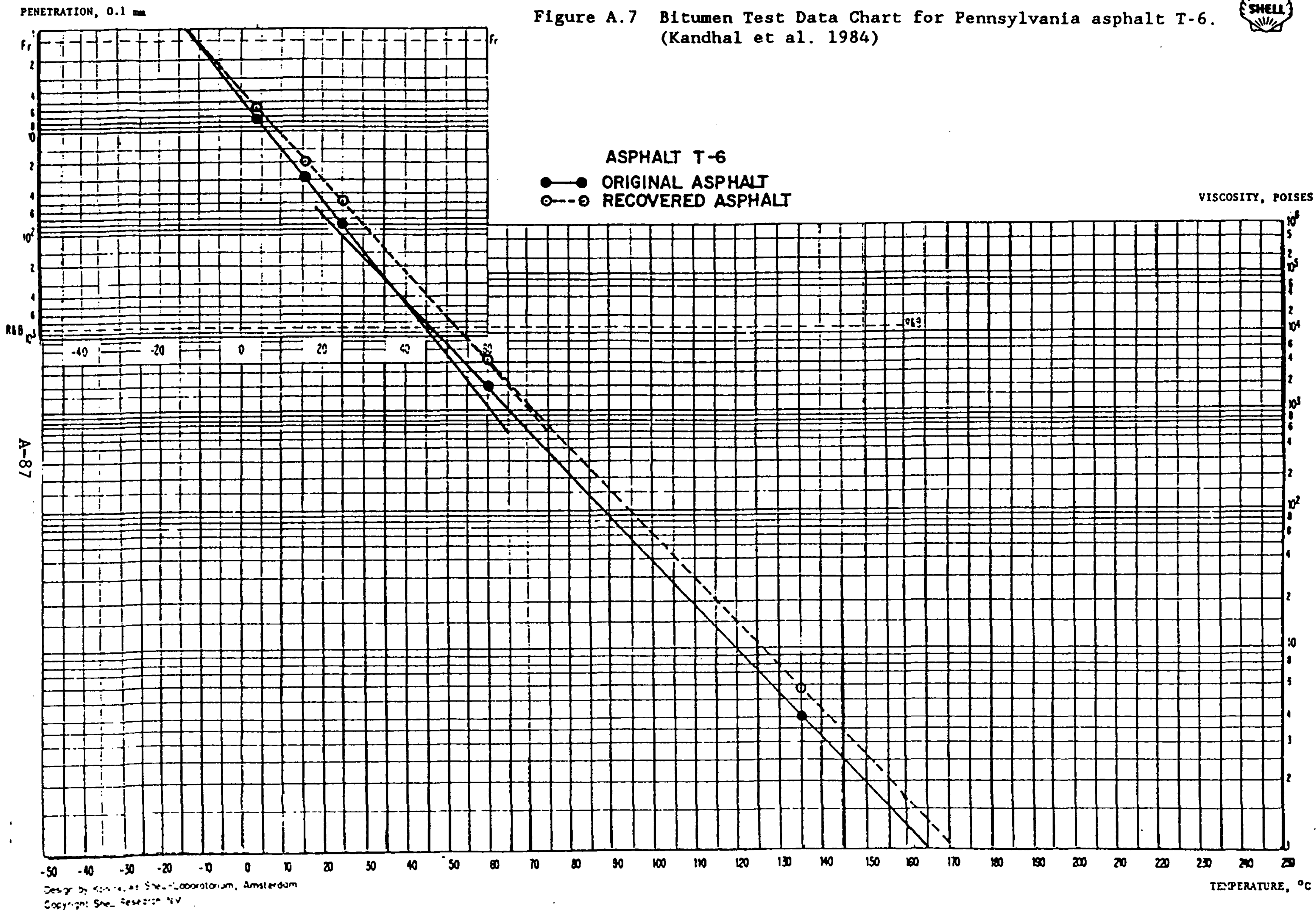
Figure A.7 Bitumen Test Data Chart for Pennsylvania asphalt T-6. (Kandhal et al. 1984)



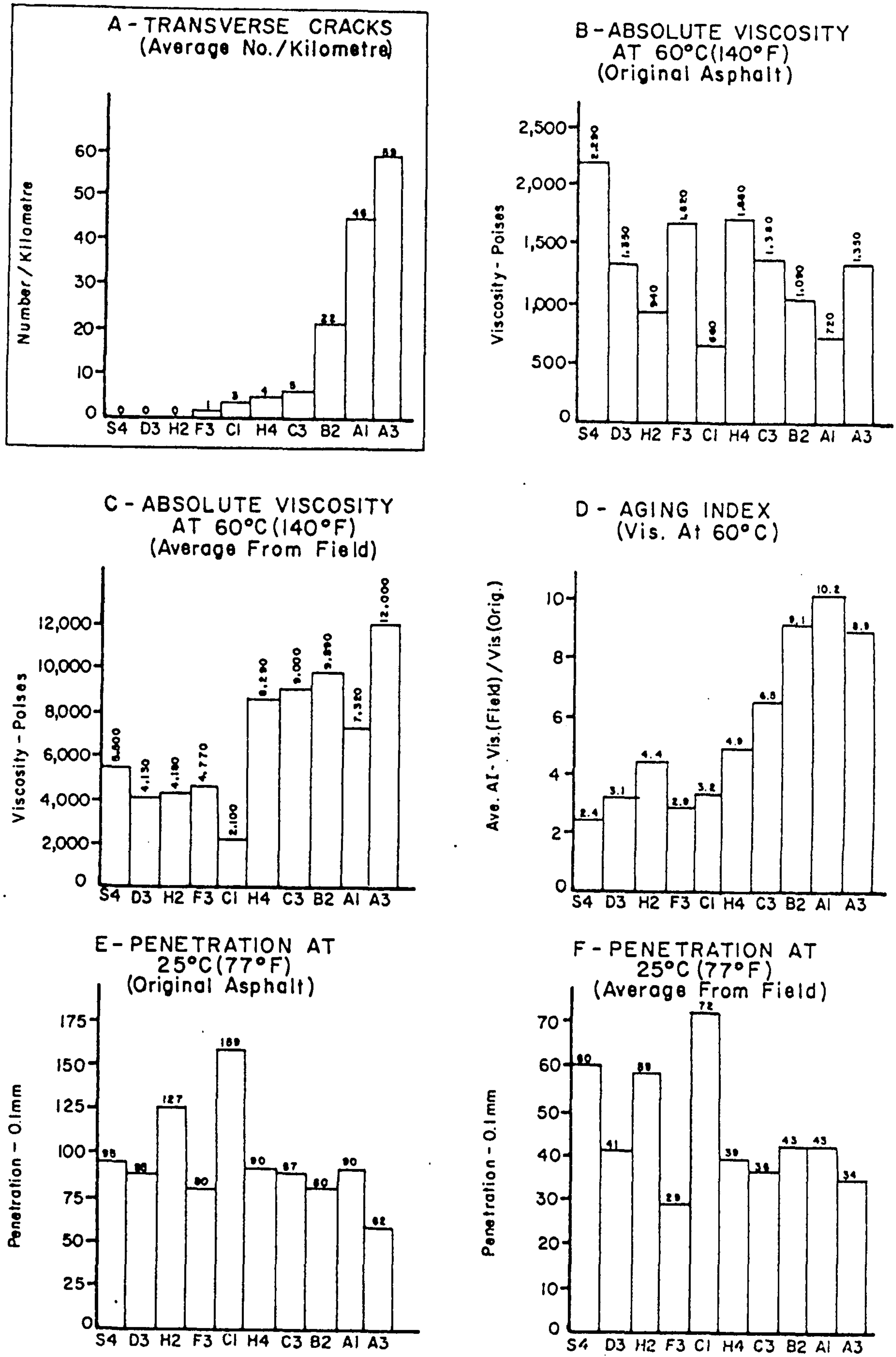


Figure A.8 Summary of physical properties and transverse cracks measured for Utah test loads; objective evaluation. (Anderson et al. 1976)

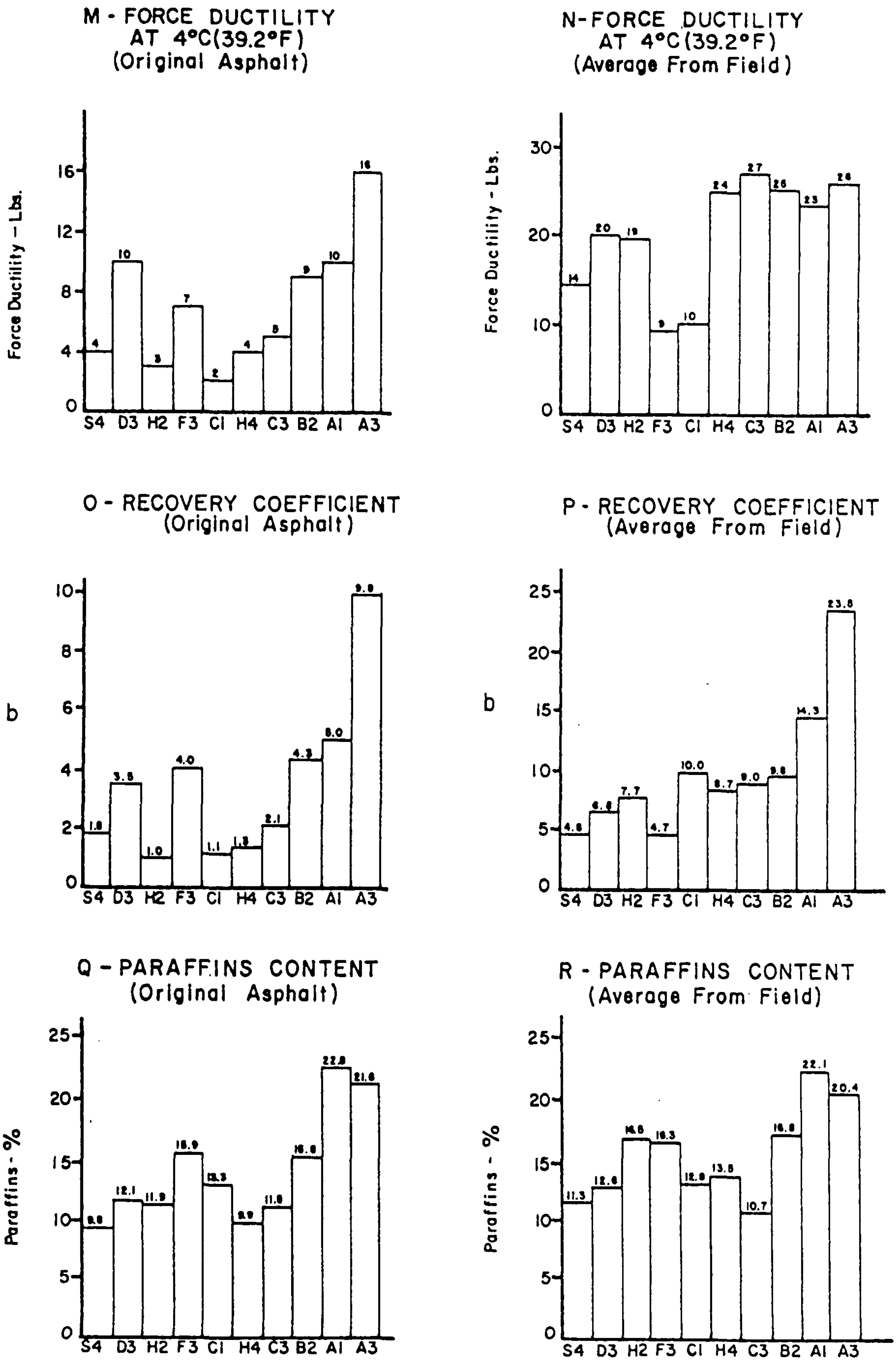
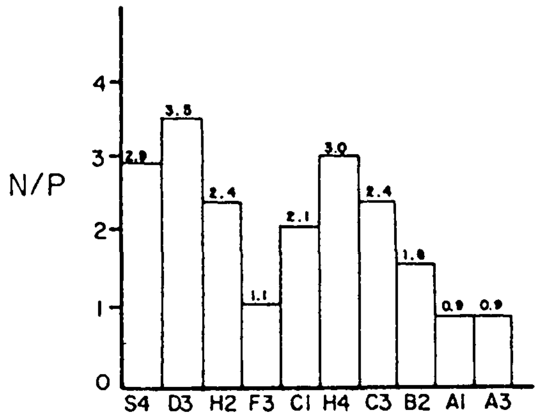
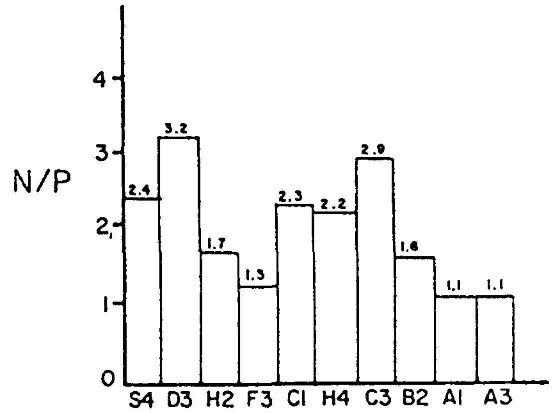


Figure A.8 (continued).

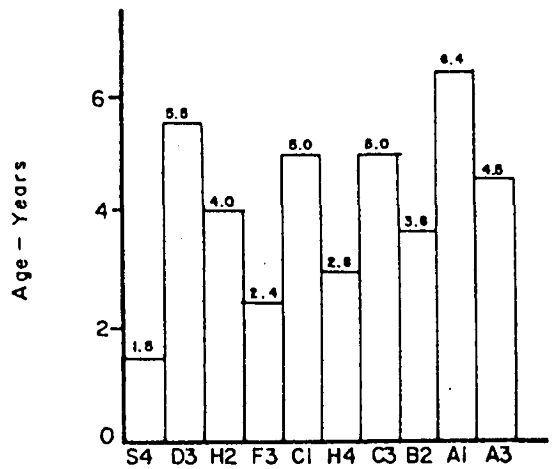
S - NITROGEN BASES/PARAFFINS
(Original Asphalt)



T - NITROGEN BASES/PARAFFINS
(Average From Field)



U - AVERAGE PAVEMENT
AGE



A - American (Salt Lake)
B - Phillips
C - American (Casper)
D - Douglas (Para)
F - Arizona
G - Golden Bear
H - Husky
S - Douglas (Santa Maria)

1 - AC 5
2 - AC 10
3 - AC 15
4 - AC 20
5 - AC 20+
6 - AC 40

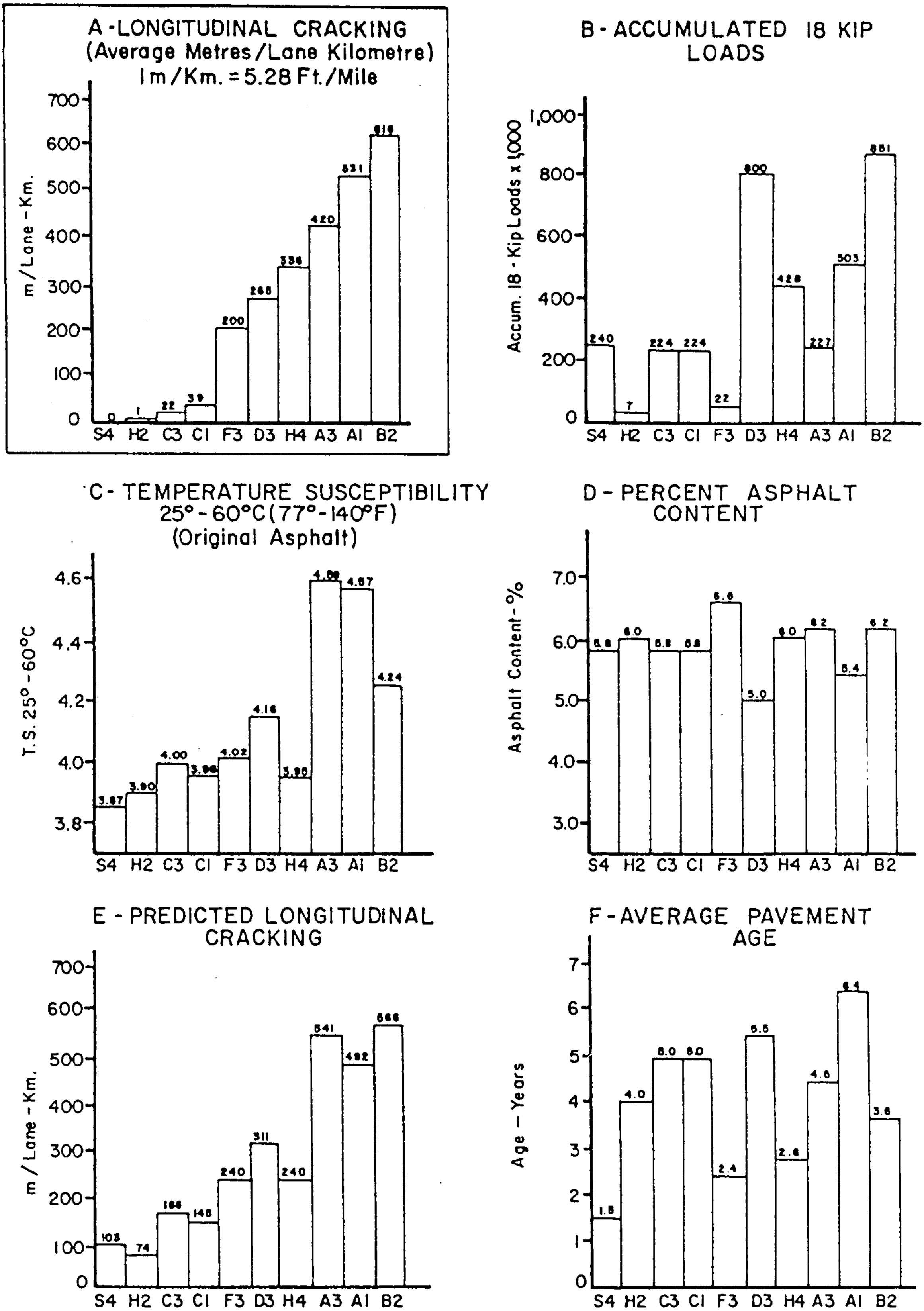
$$\text{Temperature Susceptibility} = \frac{\log\log\eta_1 - \log\log\eta_2}{\log T_2 - \log T_1}$$

where η_1 , and η_2 are viscosity readings at temperatures T_1 and T_2

Recovery Coeficient (b) is defined by the Force Ductility curves using the points after the peak in the equation:

$$\text{Force (lbs)} = a - b \ \ln \text{(elong.)}$$

Figure A.8 (continued).



Predicted Long. = 3.33×10^{-4} (Accum. 18^{k}) + 524(Temp. Susept.) + 163(% AC) − 2950

Figure A.9 Longitudinal cracking prediction; objective evaluation. (Anderson et al. 1976)

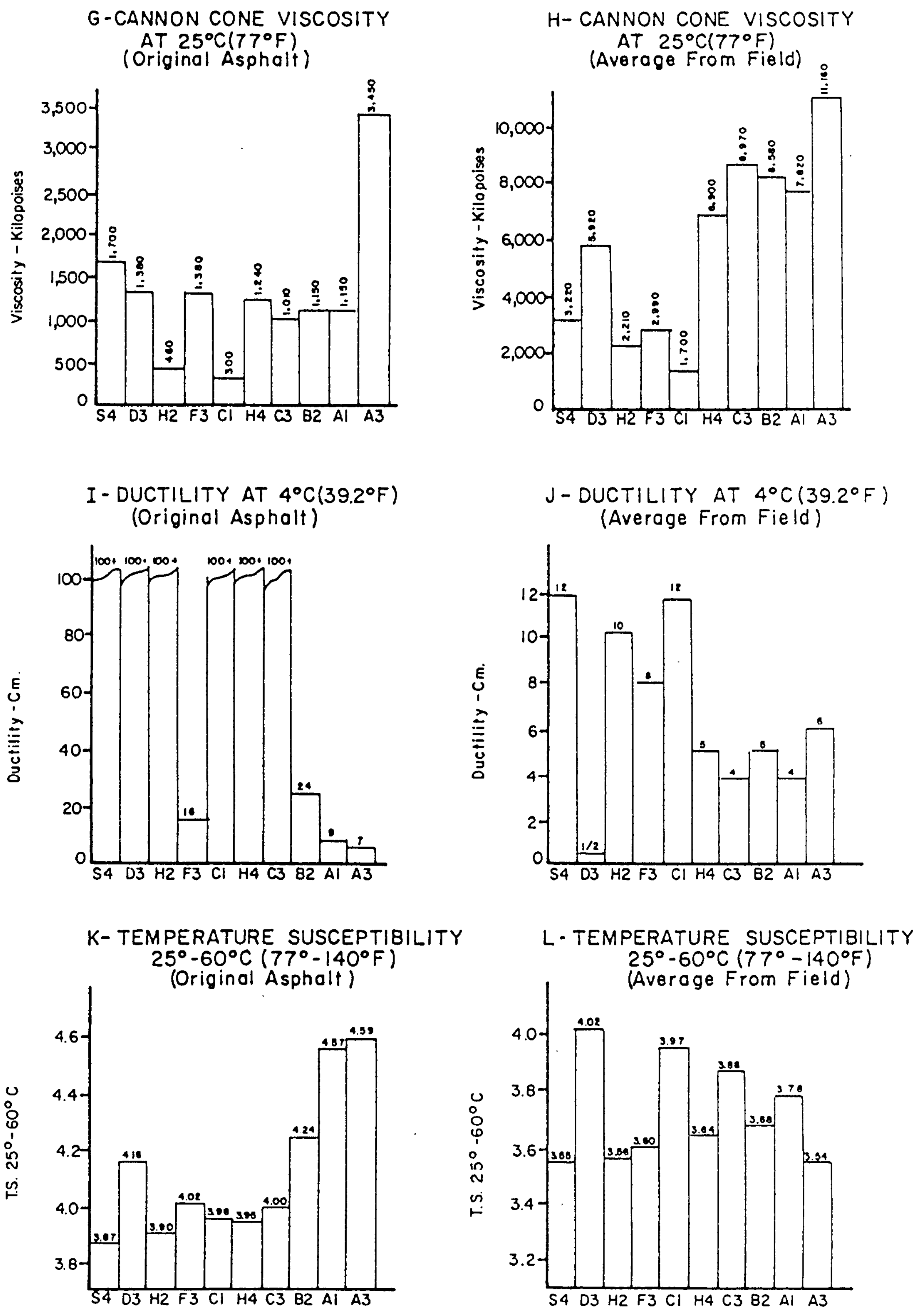


Figure A.9 (continued).

APPENDIX B

UNCONTROLLED TEST ROADS

This appendix contains supplementary tables and figures for the test roads discussed in Chapter 4. They are organized in the same order as the test roads in Chapter 4.

LIST OF TABLES

LIST OF FIGURES

Table B.1 Cores and mix properties for Oregon. (Thenoux et al. 1988)

Proj. (#)	Thickness (in)	Max. Sp. Grav.	Air Voids	A/C %	Asphalt Suplier	Mix Type	Mr (ksi)	Nf (1)
1	1.72	2.476	11.1	5.0	Chevron AR4000w	B-mix	862.00	80350
2	2.44	2.580	8.5	5.7	Chevron AR4000w	B-mix	1103.19	10005
3	1.91	2.459	11.8	5.9	Chevron	B-mix	771.87	276292
3a	-	-	-	-	AR4000	-	-	-
4	1.44	2.421	5.0	7.0	Douglas AR4000	B-mix	281.94	-
5	1.41	2.497	8.3	5.8	Chevron	B-mix	568.97	42480
5s	1.83	2.484	9.0	5.6	AR4000		703.78	129064
6	2.49	2.535	6.1	5.2	Witco AR2000	B-mix	1031.63	4112
7	1.55	2.444	4.3	6.7	Douglas	B-mix	243.30	295241
7s	1.92	2.434	4.3	6.9	120/150p		186.30	1876282
8	1.44	2.158	8.7	7.6	Shell AR2000	C-mix	621.94	87662

(1) Nf, calculated for 100 microstrains

Table B.2a Chemical composition of original samples and after Rolling Thin Film Oven Test for Oregon. (Thenoux et al. 1988)

Sample	Asph. %	Sat. %	N-Arom. %	P-Arom. %	Total %
Orig.-1	16.5	9.5	26.2	47.0	99.2
Orig.-2	15.7	9.1	26.9	46.6	98.3
Orig.-3	22.7	8.4	24.5	43.3	98.9
Orig.-4	20.2	9.1	24.9	43.8	98.0
Orig.-5	16.6	9.3	26.5	46.9	99.3
Orig.-6	6.0	11.6	33.1	48.1	98.8
Orig.-7	17.0	9.6	27.9	45.3	99.8
Orig.-8	6.9	10.9	32.4	48.3	98.5
RTFO-1	21.4	7.9	25.5	43.7	98.5
RTFO-2	20.5	7.8	25.2	45.1	98.6
RTFO-3	27.8	7.1	22.3	41.8	99.0
RTFO-4	24.5	8.1	22.8	43.9	99.3
RTFO-5	21.8	8.2	25.0	43.8	98.8
RTFO-6	11.2	10.9	29.7	46.9	98.7
RTFO-7	21.3	9.2	24.6	44.5	99.6
RTFO-8	14.2	10.3	27.1	47.0	98.6

Table B.2b Fractional composition of recovered asphalt using Method-A, projects 1 to 8 for Oregon. (Thenoux et al. 1988)

Sample	Asph. %	Sat. %	N-Arom. %	P-Arom. %	Total %
1-Top	32.2	7.1	20.9	39.3	99.5
2-Top	20.0	9.7	22.5	45.6	97.8
3-Top	36.8	5.6	20.0	36.4	98.8
3a-Top	39.1	6.1	19.1	35.5	99.8
4-Top	23.4	9.6	21.8	43.6	98.4
5-Top	25.4	8.0	22.2	42.2	97.8
5s-Top	28.4	7.6	22.4	40.4	98.8
6-Top	19.7	10.6	23.3	44.4	98.0
7-Top	28.5	8.4	20.7	41.8	99.4
7s-Top	28.8	8.3	23.2	39.7	100.0
8-Top	24.0	9.6	21.9	44.0	99.5
1-Base	32.4	6.1	20.8	39.8	99.1
2-Base	20.8	10.0	22.5	45.4	98.7
3-Base	36.1	5.8	20.1	37.2	99.2
3a-Base	34.3	6.9	20.2	38.3	99.7
4-Base	24.0	9.6	21.9	43.4	98.9
5-Base	-	-	-	-	-
5s-Base	-	-	-	-	-
6-Base	13.8	11.3	24.5	49.1	98.7
7-Base	25.2	8.6	22.2	43.1	99.1
7s-Base	24.2	7.9	24.0	43.9	100.0
8-Base	16.5	10.2	24.8	46.6	98.1

Table B.3 Physical properties of original samples and after Rolling Thin Film Oven Test for Oregon. (Thenoux et al. 1988)

Sample	Data Available from Date of Construction				Data Measured During 1985			
	Pen-4	Pen-25	Vis-60 (poises)	KVis-135 (cStokes)	Pen-4	Pen-25	Vis-60 (poises)	KVis-135 (cStokes)
1-Orig.	18	73	1552	352	23	72	1783	364
2-Orig.	18	73	1552	352	26	77	1613	352
3-Orig.	50	139	-	-	47	128	1169	353
4-Orig.	49	134	1110	340	48	128	1124	335
5-Orig.	20	80	1504	368	22	74	1577	345
6-Orig.	17	85	1052	201	17	88	1059	190
7-Orig.	46	140	762	236	31	128	768	244
8-Orig.	25	100	-	-	15	84	992	190
1-RTFO	-	39	4191	572	16	43	4216	545
2-RTFO	-	39	4191	572	16	44	3960	526
3-RTFO	-	66	4306	608	30	60	4592	665
4-RTFO	-	65	4344	633	32	65	4193	619
5-RTFO	-	46	3858	494	20	52	3858	513
6-RTFO	-	66	1876	255	15	66	1678	247
7-RTFO	-	-	2164	-	30	66	2524	393
8-RTFO	-	60	2051	260	14	54	2068	267

Table B.4a Physical test results of recovered asphalt from Oregon using Method-A, projects 1 to 8. (Thenoux et al. 1988)

Sample #	Pen-4	Pen-25	Vis-60 (poises)	KVis-135 (cStokes)
1-Top	2	11	59284	1933
2-Top	4	15	13299	572
3-Top	9	10	225129	3952
3a-Top	6	12	100000	3802
4-Top	26	51	3403	399
5-Top	14	22	13584	885
5s-Top	12	26	11755	853
6-Top	11	27	4440	330
7-Top	32	63	5524	745
7s-Top	31	80	3611	616
8-Top	6	11	25104	665
1-Base	5	12	74152	1953
2-Base	6	14	12571	569
3-Base	6	16	106328	2798
3a-Base	9	22	45564	1916
4-Base	-	51	3194	401
5-Base	*	*	*	*
5s-Base	15	47	4360	531
6-Base	-	42	2565	268
7-Base	-	80	1769	466
7s-Base	-	114	1752	465
8-Base	13	45	2845	318

* Sample Contaminated

Table B.4b Physical test results of asphalts from projects 3, 5 and 7 in Oregon obtained using all four methods of extraction and recovery. (Thenoux et al. 1988)

Sample #	Method	Pen-4	Pen-25	Vis-60 (poises)	KVis-135 (cStokes)
3-Top	A	2	11	59284	1933
	B	4	11	160000	3472
	C	134	140	3213	493
	D	14	21	46179	1969
3a-Top	A	6	12	100000	3802
	B	5	10	185999	6509
	C	16	23	56593	2242
	D	7	16	44869	2872
5-Top	A	14	22	13584	885
	B	3	21	17253	896
	C	8	20	15284	985
	D	32	80	19444	451
7-Top	A	32	63	5524	745
	B	-	-	-	-
	C	22	54	6392	764
	D	22	48	7507	819
3-Base	A	6	16	106328	2798
	B	6	16	85619	2413
	C	16	123	1228	407
	D	6	15	102308	2187
3a-Base	A	9	22	45564	1916
	B	6	20	42937	1926
	C	18	43	18040	1222
	D	10	22	10799	1797
5-Base	A	*	*	*	*
	B	9	36	7630	717
	C	12	36	6681	657
	D	11	32	8036	718
7-Base	A	-	80	1769	466
	B	38	89	3060	545
	C	27	80	3002	560
	D	-	-	-	-

* Sample Contaminated

Table B.5a Properties of original asphalt in surviving pavements for FHWA Study. (Vallerga and Halstead 1971)

Project No.	Asphalt No.	Penetration at 77°F [a]	Viscosity at 77°F [a] .05 Sec.$^{-1}$ (megapoise) [d]	Viscosity at 140°F [b] (poise) [e]	Viscosity at 275°F [c] (centistoke) [f]
2	29	80	5.05	2537	520
3	30	88	1.70	3044	494
4	32	84	2.06	1973	403
6	73	87	1.51	1784	450
7	67	86	1.40	1875	484
9	74	88	1.49	1176	495
10	72	92	1.89	2139	384
11	25	88	1.78	2782	581
13	24	88	1.38	2173	494
15	70	86	1.72	1721	424
16	71	85	1.34	1550	369
17	7	92	1.10	1655	344
18	7	92	1.10	1655	344
19	1	82	2.49	2253	474
20	1	82	2.49	2253	474
21	11	85	1.36	1668	412
22	19	89	1.35	1878	416
24	57	94	1.96	1777	437
25	61	86	4.70	729	210
26	7	92	1.10	1655	344
27	7	92	1.10	1655	344
29	92	92	1.33	1311	278
31	95	83	1.49	2095	421
38	19	89	1.35	1878	416

Table B.5a Continued

Project No.	Asphalt No.	Penetration at 77^{o}F	Viscosity at 77^{o}F[a] .05 Sec.$^{-1}$ (megapoise)[d]	Viscosity at 140^{o}F[b] (poise)[e]	Viscosity at 275^{o}F[c] (centistoke)[f]
39	19	89	1.35	1878	416
40	7	92	1.10	1655	344
50	11	85	1.36	1668	412
52	57	94	1.96	1777	437
53	30	88	1.70	3044	494
54	28	86	1.20	1685	337
55	61	86	4.70	729	210
56	71	85	1.34	1550	369
57	166	55	3.90	4729	643
58	158	57	4.20	3595	582

(a) 77^{o}F = 25^{o}C

(b) 140^{o}F = 60^{o}C

(c) 275^{o}F = 135^{o}C

(d) A megapoise = 100k Pa·s

(e) A poise = dPa·s

(f) A centistoke = mm^2/s

Table B.5b Chemical composition of original asphalt binders in surviving pavements for FHWA Study. (Vallerga and Halstead 1971)

Project Identif.	Asphalt Number	Composition, percent by weight of binder				
		Asphaltenes	Nitrogen Bases	Acidaffins		Paraffins
		(A)	(N)	First (A_1)	Second (A_2)	(P)
2	29	30.8	9.0	21.6	28.3	10.3
3	30	30.9	7.6	17.9	29.3	14.3
4	32	20.1	11.7	22.2	28.9	12.1
6	73	22.9	13.5	24.7	28.3	10.6
7	67	26.5	11.5	24.1	27.5	10.4
9	74	6.2	-	-	-	-
10	72	24.5	10.7	18.3	28.1	18.4
11	25	30.6	16.8	23.0	21.6	8.0
13	24	28.3	20.3	24.7	19.4	7.3
15	70	18.2	26.9	21.2	23.6	10.1
16	71	11.9	20.9	28.3	29.9	9.0
17	7	21.4	19.2	21.9	25.7	11.8
18	7	21.4	19.2	21.9	25.7	11.8
19	1	27.6	22.0	20.8	21.7	7.9
20	1	27.6	22.0	20.8	21.7	7.9
21	11	23.3	15.7	19.3	27.0	14.7
22	19	25.9	14.2	20.5	27.5	11.9
24	57	20.8	13.6	21.7	28.1	15.8
25	61	15.8	24.8	16.4	26.1	16.9
26	7	21.4	19.2	21.9	25.7	11.8
27	7	21.4	19.2	21.9	25.7	11.8
29	92	21.3	33.3	17.7	16.6	11.1
31	95	32.0	22.4	22.1	14.5	9.0

Table B.5b Continued

Project Identif.	Asphalt Number	Composition, percent by weight of binder: Asphaltenes (A)	Nitrogen Bases (N)	Acidaffins: First (A_1)	Acidaffins: Second (A_2)	Paraffins (P)
38	19	25.9	14.2	20.5	27.5	11.9
39	19	25.9	14.2	20.5	27.5	11.9
40	7	21.4	19.2	21.9	25.7	11.8
50	11	23.3	15.7	19.3	27.0	14.7
52	57	20.8	13.6	21.7	28.1	15.8
53	30	30.9	7.6	17.9	29.3	14.3
54	28	21.5	19.2	21.9	25.0	12.4
55	61	15.8	24.8	16.4	26.1	16.9
56	71	11.9	20.9	28.3	29.9	9.0
57	116	22.6	13.3	28.4	26.6	9.1
58	158	21.8	11.4	27.6	28.6	10.6

Table B.6 Penetration and viscosity data of laboratory-aged binders and recovered field-aged binders for FHWA Study. (Vallerga and Halstead 1971)

Proj. No.	Asphalt No.	Penetration at 77°F (a)		Viscosity at 77°F (a) and .05 sec^{-1} rate of shear (megapoise) (d)			Viscosity at 140°F (b) (kilopoise) (e)			Viscosity at 275°F (c) (centistoke) (f)		
		TFOT Res.	Rec. Binder	TFOT 1/ Res.	7 hr. MTFOT 2/ Res.	Rec. Binder	TFOT Res.	7 hr. MTFOT Res.	Rec. Binder	TFOT Res.	7 hr. MTFOT Res.	Rec. Binder
2	29	52	21.0	8.70	20.4	44.8	8.91	38.4	124.	805	1605	2125
3	30	50	57.3	7.50	23.0	5.0	15.4	158.	9.75	966	2267	763
4	32	55	27.7	4.35	13.4	21.8	4.60	14.1	20.8	570	927	1037
6	73	50	23.3	5.42	12.2	39.5	5.82	11.0	150.	733	946	2565
7	67	52	18.2	5.05	13.8	56.8	5.90	28.7	290.	722	1370	3205
9	74	65	31.2	2.53	7.8	26.4	2.05	5.5	-	632	969	1755
10	72	56	32.2	7.75	14.8	19.2	10.1	85.7	101.	657	1428	1396
11	25	51	40.0	6.30	27.0	11.8	9.82	71.2	17.1	1093	2669	1182
13	24	52	17.7	4.67	18.0	54.6	6.81	39.6	132.	829	1967	2767
15	70	45	41.3	5.85	17.5	8.9	4.88	17.2	8.12	595	1006	712
16	71	52	40.7	3.59	7.2	6.7	2.70	4.8	5.14	469	608	582
17	7	48	31.5	5.50	27.7	18.5	4.24	11.3	19.0	582	938	940
18	7	48	47.6	5.50	27.7	8.4	4.24	11.3	9.06	582	938	681
19	1	48	25.0	5.55	22.1	22.6	7.02	32.4	23.3	795	1672	1278
20	1	48	35.3	5.55	22.1	15.0	7.02	32.4	13.7	795	1672	999
21	11	53	29.6	5.60	17.9	19.8	5.97	28.3	27.5	992	1203	1058
22	19	55	14.9	4.95	19.0	84.4	4.80	20.7	114.	605	1274	2033

1/ Thin film oven test
2/ Modified thin film oven test

Table B.6 Continued

Proj. No.	Asphalt No.	Penetration at 77°F(a)		Viscosity at 77°F(a) and .05 sec^{-1} rate of shear (megapoise)(d)			Viscosity at 140°F(b) (kilopoise)(e)			Viscosity at 275°F(c) (centistoke)(f)		
		TFOT Res.	Rec. Binder	TFOT Res.	7 hr. MTFOT Res.	Rec. Binder	TFOT Res.	7 hr. MTOFOT Res.	Rec. Binder	TFOT Res.	7 hr. MTFOT Res.	Rec. Binder
24	57	58	67.7	5.20	15.8	3.2	6.40	51.8	3.50	615	1223	508
25	61	53	60.6	4.20	17.5	2.6	2.00	5.85	1.52	288	428	280
26	7	48	21.3	5.50	27.7	36.9	4.24	11.3	46.8	582	938	1335
27	7	48	46.7	5.50	27.7	11.0	4.24	11.3	11.6	582	938	684
29	92	45	5.0	4.45	21.3	277.3	3.41	12.7	468.	591	723	2946
31	95	44	5.8	7.80	30.5	241.5	8.36	48.5	772.	783	2231	6252
38	19	55	43.4	4.95	19.0	20.7	4.80	20.7	40.3	605	1274	1149
39	19	55	22.8	4.95	19.0	37.2	4.80	20.7	88.9	605	1274	2089
40	7	48	35.0	5.50	27.7	11.2	4.24	11.3	9.38	582	938	686
50	11	53	26.0	5.60	17.9	26.5	5.97	28.3	39.4	992	1203	1267
52	57	58	63.0	5.20	15.8	3.44	6.40	51.8	4.35	615	1223	596
53	30	50	21.2	7.50	23.0	43.7	15.4	158.	539.	966	2267	4375
54	28	54	19.2	4.60	16.3	59.0	3.88	11.9	56.8	497	829	1370
55	61	53	51.5	4.20	17.5	3.65	2.00	5.85	1.97	288	428	304
56	71	52	38.8	3.59	7.2	9.43	2.70	4.84	11.4	469	608	741
57	166	42	53.7	8.90	24.3	4.35	9.24	24.5	5.04	835	1218	660
58	158	41	34.5	9.00	25.0	16.3	7.63	24.1	17.6	772	1141	959

(a) 77°F = 25°C
(b) 140°F = 60°C
(c) 275°F = 135°C

(d) A megapoise = 100k Pa·s
(e) A kilopoise = 100 Pa·s
(f) A centistoke = mm^2/s

Table B.7 Rheological properties and stiffness moduli of recovered asphalt cements for Oklahoma. (Noureldin and Manke 1978)

Site No.	Cracking Index (C.I.)	Ring & Ball Softening Point, F (C)	Penetration at 77 F (25 C)	Absolute Viscosity at 140 F (60 C), Poise	Kinematic Viscosity at 275 F (135 C), cSt	Stiffness Modulus	
						kg/m^2	Pa
1	6.5	168 (76)	26	651,700	5,408	282	27.7
2	10.5	160 (71)	32	65,327	3,063	250	24.5
3	15.5	136 (58)	44	22,311	1,214	343	33.6
4	24.5	173 (78)	18	218,360	2,353	1000	98.1
5	2.0	124 (51)	57	3,606	546	190	18.6
6	9.0	140 (60)	41	20,630	1,146	258	25.3
7	20.0	129 (54)	58	3,853	536	205	20.1
8	0.0	126 (52)	59	4,058	639	170	16.7
9	0.9	123 (51)	62	2,610	502	189	18.5

Table B.8 Composition and stiffness moduli of field specimens for Oklahoma. (Noureldin and Manke 1978)

Site No.	Cracking Index (C.I.)	Asphalt Content, %	Bulk Specific Gravity	Per Cent Density	Volume Conc. of Agg. Mix, (C_V)	Stiffness Modulus at -23.3 C	
						kg/cm^2	Pa
1	6.5	5.6	2.35	94.4	0.839	30,463	2,987.4
2	10.5	5.9	2.33	93.5	0.824	22,361	2,192.9
3	15.5	5.6	2.40	97.1	0.860	49,226	4,827.4
4	24.5	5.9	2.18	91.4	0.814	49,292	4,833.9
5	2.0	5.5	2.44	97.5	0.874	34,091	3,343.2
6	9.0	4.9	2.40	96.5	0.872	53,268	5,223.8
7	20.0	5.8	2.39	97.4	0.854	32,236	3,161.3
8	0.0	5.5	2.44	97.2	0.859	31,267	3,066.2
9	0.5	5.6	2.41	97.4	0.859	33,940	3,328.4

Table B.9 Asphalt cement properties for Ontario. (Fromm and Phang 1972)

SAMPLE NO.	CONTRACT NO.	ORIGINAL ASPHALT: PEN. 77°F. dmm.	ORIGINAL ASPHALT: VISC. 275°F. cs.	RECOVERED ASPHALT: PEN. 77°F. dmm.	PEN. 39.2°F. dmm.	SOFT POINT °F.	VISC. 275°F.	MICRO VISC. 60°F. X10⁴	COMPOSITION: % ASPH.	% SAT.	% L.AROM.	% H. AROM.
3	60-200	95	205	48	18	132	352	7900	28	18	22	32
5		95	205	48	18	132	352	7900	28	18	22	32
8		95	205	35	12	132	369	9350	28	18	22	32
12	60-200	95	336	55	25	134	553	2910	36	13	24	27
15		95	336	55	25	134	553	2910	36	13	24	27
18		95	336	32	13	138	734	4060	36	13	24	27
21	60-200	95	427	41	13	134	784	8100	30	9	24	37
23		95	427	41	13	134	784	7020	30	9	24	37
26		95	427	41	13	134	784	7020	30	9	24	37
27	63-088	87	338	32	8	134	630	9600	27	13	23	36
28		87	338	35	15	134	646	7840	27	13	23	36
30	61-078	84	425	38	9	131	760	9250	33	9	23	35
31		84	425	38	9	131	760	9250	33	9	23	35
32		84	425	38	9	131	760	9250	33	9	23	35
33		84	425	38	9	131	760	7600	33	9	23	35
34		84	425	37	15	133	762	7600	33	9	23	35
35		84	425	37	15	133	762	6250	33	9	23	35
36		84	425	34	14	134	830	6250	33	9	23	35
37		84	425	34	14	134	830	6250	33	9	23	35
38	61-025	87	208	32	15	133	395	11000	29	13	24	34
39		87	208	32	15	133	395	11000	29	13	24	34
40		87	208	26	3	138	464	14000	29	13	24	34
41		87	208	26	3	138	464	14000	29	13	24	34
42		87	208	34	11	133	393	10000	29	13	24	34
43		87	208	34	11	133	393	10000	29	13	24	34
44	60-102	171	141	33	21	142	801	9250	30	15	24	31
45		171	141	50	24	130	435	4000	30	15	24	31
50	60-193	89	331	36	19	141	875	10900	35	14	21	30
51		89	331	36	19	141	875	10900	35	14	21	30
52		89	331	38	20	137	828	7100	35	14	21	30
53		89	331	38	20	137	828	7100	35	14	21	30
54		89	331	36	20	139	725	8900	35	14	21	30
55		89	331	36	20	139	775	8900	35	14	21	30
56	60-123	96	211	40	18	133	378	8250	27	17	21	35
57		96	211	40	18	133	378	8250	27	17	21	35
58		96	211	38	17	133	398	7600	27	17	21	35
59		96	211	38	17	133	398	7600	27	17	21	35
60		96	211	38	16	127	410	7250	27	17	21	35
61		96	211	38	16	127	410	5170	27	17	21	35
62		96	211	48	18	128	330	5170	27	17	21	35
63		96	211	48	18	128	330	5170	27	17	21	35
70	59-062	161	256	68	28	122	414	2080	26	16	21	37
71		161	256	57	26	125	447	2080	26	16	21	37
72		161	256	63	26	123	377	4950	27	16	21	37
73	60-041	145	222	52	17	127	395	4040	32	16	21	31
74		145	222	52	25	128	437	4420	32	16	21	31
75		145	222	53	25	129	449	4210	32	16	21	31
76	60-187	183	171	87	36	121	204	1680	31	14	24	31
77		183	171	61	27	126	234	2510	31	14	24	31
78		183	171	65	29	126	234	3000	31	14	24	31
80	59-177	170	139	83	26	120	213	3110	22	16	27	35
81		170	139	87	24	119	192	1930	22	16	27	35
82	62-138	144	277	81	32	117	386	2370	29	12	20	39
83		144	277	81	32	117	386	2370	29	12	20	39
84		144	277	60	22	128	471	2030	29	12	20	39
85		144	277	60	22	128	471	2030	29	12	20	39
86	62-030	180	313	63	28	125	627	2040	31	9	25	35
87		180	313	63	28	125	627	2040	31	9	25	35
88		180	313	63	27	126	663	2140	31	9	25	35
93		180	313	44	18	129	704	5570	31	9	25	35
102	61-188	84	205	50	12	121	284	5390	23	14	28	35
103		84	205	43	12	[illegible]	301	7950	23	14	28	35
105		157	142	62	20	120	273	2780	18	19	25	38
106		157	142	99	28	116	230	875	18	19	25	38
107		84	205	93	20	118	237	1150	22	17	29	33
108		84	205	50	20	129	358	3150	22	17	29	33
111	53-023	90	350	20	9	138	999	12750	37	9	21	34
112		90	350	27	15	141	999	11200	37	9	21	34
113	49-051	–	–	33	21	130	600	5540	21	12	23	44
114		–	–	35	17	132	580	5850	21	12	23	44
115		–	–	38	18	132	592	5900	21	12	23	44
116	57-121	180	306	49	26	129	633	3650	32	9	23	36
117		180	306	81	32	127	693	2110	32	9	23	36
118		180	306	41	19	132	752	4000	32	9	23	36
119		180	306	56	23	122	702	2090	32	9	23	36
120		180	306	44	19	129	604	3550	32	9	23	36
121		180	306	56	20	127	670	4480	32	9	23	36

Table B.9 Continued.

SAMPLE NO.	CONTRACT NO.	ORIGINAL ASPHALT PEN. 77°F. dmm.	ORIGINAL ASPHALT VISC. 275°F. cs.	RECOVERED ASPHALT PEN. 77°F. dmm.	PEN. 39.2°F. dmm.	SOFT POINT °F.	VISC. 275°F.	MICRO VISC. 60°F. X10⁴	COMPOSITION % ASPH.	% SAT.	% L.AROM.	% H. AROM.
122	58-277	175	270	61	22	124	512	2630	33	11	24	32
123		175	270	43	19	132	670	4540	33	11	24	32
124		175	270	56	24	122	544	2380	33	11	24	32
125	64-140	153	226	53	22	129	444	3550	30	13	26	31
126		153	226	53	22	128	403	1720	30	13	26	31
127		153	226	53	21	125	430	3050	30	13	26	31
128		153	226	47	24	126	408	3810	30	13	26	31
129		153	226	53	24	125	401	2610	30	13	26	31
130	64-012	153	223	76	44	116	314	1230	33	12	24	31
131		153	223	47	17	127	438	2260	33	12	24	31
132		153	223	62	25	124	358	2140	33	12	24	31
133		153	223	41	22	128	458	2430	33	12	24	31
134		153	223	48	19	128	346	4200	33	12	24	31
135		153	223	44	19	126	436	5320	33	12	24	31
136		153	223	47	19	124	427	3120	33	12	24	31
137	62-065	85	420	43	10	129	586	4970	29	11	29	31
138		85	420	46	19	127	540	4100	29	11	29	31
139		85	420	62	24	124	604	3620	29	11	29	31
140	56-384	176	252	40	15	125	660	2520	33	10	27	30
141		176	252	34	15	132	725	6360	33	10	27	30
142		176	252	53	20	132	701	3030	33	10	27	30
144	62-185	151	250	97	38	115	354	809	24	15	31	30
145		151	250	63	29	128	483	2800	24	15	31	30
146	52-080	90	350	27	12	134	717	9700	24	11	22	43
147		90	350	26	15	133	846	10050	24	11	22	43
148		90	350	24	16	136	820	10900	24	11	22	43
149		90	350	31	18	136	842	17600	24	11	22	43
150		90	350	20	10	140	932	17700	24	11	22	43
151		90	350	22	11	134	899	12850	24	11	22	43
152		90	350	26	8	141	758	14800	24	11	22	43
153		90	350	36	18	139	885	18500	24	11	22	43
154		90	350	30	13	133	758	10800	24	11	22	43
155	53-048	90	350	31	13	136	827	7300	30	15	21	34
156		90	350	31	13	136	629	13600	30	15	21	34
157		90	350	23	10	134	768	12800	30	15	21	34
158	61-124	87	382	41	17	132	669	4190	30	12	27	31
159		87	382	33	16	136	593	7200	30	12	27	31
160		87	382	27	9	132	404	8100	30	12	27	31
161		87	382	24	17	130	424	11100	27	15	26	32
162		87	382	33	14	130	344	7330	27	15	26	32
163		87	382	41	12	128	315	5690	27	15	26	32
164	61-113	85	450	44	17	129	631	6150	35	11	27	27
165		85	450	43	18	134	757	5620	35	11	27	27
166		85	450	28	14	136	805	10550	35	11	27	27
167		85	450	33	17	139	993	11300	35	11	27	27
168	62-609	83	216	33	16	137	650	6400	26	15	25	34
169		83	216	38	10	132	619	6850	26	15	25	34
170		83	216	39	18	130	612	7950	26	15	25	34
171		83	216	43	20	134	677	6530	26	15	25	34
172		83	216	37	20	134	661	6220	26	15	25	34
173		83	216	35	18	136	675	5960	26	15	25	34
174	59-038	164	259	44	21	137	572	3170	25	17	24	34
175		164	259	55	33	132	474	2100	25	17	24	34
176		164	259	41	22	130	541	4160	25	17	24	34
177		164	259	51	27	126	463	2970	25	17	24	34
178		164	259	59	29	123	458	2660	25	17	24	34
179		164	259	54	26	126	476	3060	25	17	24	34
180	60-122	159	139	56	20	127	273	6180	24	16	25	35
181		159	139	42	14	127	259	6080	24	16	25	35
182		159	139	53	15	122	242	5460	24	16	25	35
192	60-102	171	141	39	16	133	802	9600	34	14	22	30
193		171	141	33	18	134	753	8710	34	14	22	30
194		171	141	25	14	141	898	8050	34	14	22	30
195	60-183	168	226	46	20	125	280	3070	29	19	25	27
196		168	226	43	17	127	424	4650	29	19	25	27
197		168	226	48	17	122	262	6230	29	19	25	27
198		168	226	38	14	131	615	6360	29	19	25	27
199		168	226	51	23	131	502	4110	29	19	25	27
284		168	226	45	17	127	614	4000	29	19	25	27
285	61-052	168	243	70	24	122	418	2970	22	17	28	33
286		168	243	78	32	117	364	1950	22	17	28	33
287		168	243	80	29	124	382	2400	22	17	28	33
288		168	243	110	46	111	306	917	22	17	28	33
290	60-131	154	203	51	22	129	478	3360	33	15	23	29
291		154	203	40	22	138	599	6350	33	15	23	29
243		154	203	45	20	134	505	5150	33	15	23	29

Table B.10 As constructed asphaltic concrete for Arizona. (Way 1978)

Site	Date Built	Aggregate Type	Grading Percent Passing					Percent Asphalt	Density	Air Voids
			3/4	3/8	4	40	200			
Sybil Top Lift	10/1975	*S & G	100	93	68	17	4.3	5.3	138.0	8.1
Sybil Bottom Lift	6/1960	S & G	97	81	64	23	8.3	5.3	136.3	10.6
Wilmot	12/1957	S & G	97	70	56	16	4.3	3.5	132.1	13.0
Marana	7/1963	S & G	99	78	60	19	3.4	4.9	140.6	9.3
Casa Grande	3/1968	S & G	96	71	55	16	4.1	4.7	135.3	9.1
Williams Field	12/1966	S & G	96	69	52	16	5.1	4.3	138.9	8.2
Tonopah	2/1975	S & G	96	72	55	14	3.6	4.9	140.6	6.5
Upper Deer Valley	7/1964	S & G	98	67	47	17	6.9	4.8	136.0	12.0
Agua Fria	8/1964	S & G	97	70	49	14	6.0	4.5	138.5	12.3
Sunset Point	8/1964	S & G	99	69	51	19	5.7	4.4	140.6	11.1
Cherry	10/1969	S & G	98	75	60	19	7.8	4.7	149.0	6.0
Sedona	8/1960	S & G	98	72	54	21	7.0	4.4	143.6	10.3
Kachina	7/1974	Basalt	99	77	54	9	3.6	6.2	131.1	8.0
Bellemont	10/1965	Cinders	100	78	54	18	6.0	7.7	115.6	22.4
Woody Mountain	10/1968	Cinders	96	74	59	25	6.9	6.4	129.5	9.1
Winona	7/1969	Limestone	98	70	50	22	7.5	5.9	129.1	7.9
Dead River	6/1960	Basalt	99	47	17	4	2.6	4.0	140.4	12.9
Crazy Creek	9/1961	Basalt	98	62	32	9	5.7	4.7	142.2	11.8
Alpine	9/1973	Basalt	97	63	41	15	6.6	6.4	139.7	8.0
Ash Fork	11/1964	Cinders	100	73	49	18	9.1	7.9	126.0	19.1
Avondale	12/1956	S & G	90	67	58	27	3.0	3.7	140.0	9.0
Benson	5/1965	Granite	98	70	53	22	5.6	4.9	135.6	10.6
Cosnino	6/1969	Limestone	98	70	50	22	7.5	5.9	133.3	5.6
Cutter	6/1940	S & G	90	68	52	18	8.1	5 3	——	——
Flagstaff NB	9/1966	Cinders	98	80	61	20	7.6	11.0	120.0	10.0
Flagstaff SB	7/1974	Basalt	99	77	54	9	3.6	6.2	131.1	8.0
Gila Bend	1/1970	S & G	97	69	52	21	3.4	5.2	139.1	8.5
Minnetonka										
Top Lift	5/1972	Cinders	100	92	73	17	4.6	10.1	118.2	12.8
Middle Lift	7/1958	S & G	100	80	61	31	8.0	4.3	134.9	10.1
Bottom Lift	5/1958	Sand	100	97	95	51	6.0	3.3	120.1	14.2
Show Low	7/1946	Cinders	100	98	66	13	3.1	11.5	——	——
Tempe	1/1961	S & G	99	72	53	21	4.5	4.0	138.3	8.3
Topock	6/1966	S & G	97	84	70	33	5.5	4.0	133.1	11.4

*S & G is sand gravel from mixed rock sources.

Table B.11 As constructed asphalt properties for Arizona. (Way 1978)

Site	Unaged			Aged		
	Pen. @ 77 F	Abs. Visc. @ 140 F	Kin. Visc. @ 275 F	Pen. @ 77 F	Abs. Visc. @ 140 F	Kin. Visc. @ 275 F
Sybil Top Lift	170	774	2.33	62	1,482	3.29
Sybil Bottom Lift	145	—	—	—	—	—
Wilmot	129	—	—	—	—	—
Marana	93	—	—	51	—	—
Casa Grande	89	1,165	2.75	47	2,421	3.43
Williams Field	96	1,301	2.00	53	2,515	2.69
Tonopah	58	1,819	2.54	33	3,584	3.45
Upper Deer Valley	93	1,015	2.13	50	—	—
Agua Fria	128	520	1.67	75	—	—
Sunset Point	89	1,062	2.05	48	—	—
Cherry	94	887	1.97	46	—	—
Sedona	132	—	—	64	—	—
Kachina	104	883	2.00	64	1,657	2.44
Bellemont	224	—	—	95	—	—
Woody Mountain	87	1,121	1.90	56	2,261	2.56
Winona	96	1,040	2.10	60	2,340	2.52
Dead River	89	—	—	50	—	—
Crazy Creek	93	—	—	46	—	—
Alpine	99	772	2.75	58	1,608	3.28
Ash Fork	254	—	—	145	—	—
Avondale	159	—	—	—	—	—
Benson	135	402	1.71	72	813	2.10
Cosnino	98	802	2.15	52	1,754	2.62
Cutter	Cut Back	—	—	—	—	—
Flagstaff NB	140	861	1.50	66	1,786	2.01
Flagstaff SB	104	883	2.00	64	1,657	2.44
Gila Bend	87	1,078	2.63	46	2,333	3.02
Minnetonka						
Top Lift	99	912	2.60	60	2,475	—
Middle Lift	219	—	—	—	—	—
Bottom Lift	RC-3 Cut Back	8.1	—	—	—	—
Show Low	Cut Back	—	—	—	—	—
Tempe	90	—	—	49	—	—
Topock	92	819	1.60	51	1,700	1.98

Table B.12 Post construction asphalt properties for Arizona sampled March 1976. (Way 1978)

Site	Age in Months	Pen. @ 77 F (25 C)	Micro Visc. @ 77 F (25 C)	Abs Visc. @ 140 F (60 C)	Percent Asphaltenes
1. Sybil Top Lift	5	75	1.4	1,250	25.1
2. Sybil Bottom Lift	189	12	79.0	27,294	46.5
3. Wilmot	219	6	280.0	Too Viscous	46.0
4. Marana	154	8	173.0	Too Viscous	39.2
5. Casa Grande	96	10	150.0	123,000	36.2
6. Williams Field	111	5	350.0	Too Viscous	43.4
7. Tonopah	13	23	17.5	8,200	29.3
8. Upper Deer Valley	140	8	210.0	Too Viscous	39.5
9. Agua Fria	139	15	46.5	41,200	35.4
10. Sunset Point	139	16	41.2	19,600	32.2
11. Cherry	77	23	19.0	7,120	28.2
12. Sedona	187	14	51.3	43,510	36.9
13. Kachina	20	34	7.9	4,420	22.4
14. Bellemont	125	25	14.1	5,030	30.1
15. Woody Mountain	89	15	42.2	112,700	31.6
16. Winona	80	19	29.6	8,940	28.0
17. Dead River	189	7	201.0	Too Viscous	41.2
18. Crazy Creek	174	6	226.0	Too Viscous	40.3
19. Alpine	30	57	2.4	2,920	21.5
20. Ash Fork	136	22	21.1	10,200	28.7
21. Avondale	231	8	250.0	Too Viscous	40.2
22. Benson	130	32	8.6	5,130	24.6
23. Cosnino	81	27	12.1	18,350	36.2
24. Cutter	429	16	36.4	22,500	41.2
25. Flagstaff NB	114	19	25.4	7,640	29.3
26. Flagstaff SB	20	59	2.1	2,240	22.9
27. Gila Bend	62	18	30.1	14,740	31.0
28. Minnetonka Top	46	19	26.1	18,270	33.3
29. Minnetonka Middle	212	17	33.6	20,230	38.7
30. Minnetonka Bottom	214	5	539.0	Too Viscous	43.6
31. Show Low	356	44	4.7	4,640	27.7
32. Tempe	182	6	275.0	Too Viscous	41.1
33. Topock	117	6	368.0	Too Viscous	42.1

Table B.13 Asphalt aging determined from Minnetonka-East project cores in Arizona. (Way 1978)

Asphalt Grade	Unaged May 1972	March 1972	March 1973	March 1974	March 1975	March 1976	March 1977
			Penetration @ 77 F (25 C)				
40/50	48	34	14	11	12	14	11
85/100	103	36	30	23	21	19	15
120/150	144	115	—	43	39	23	26
200/300	242	89	70	70	54	31	33
			Micro-Viscosity @ 77 F (25 C) in Thousands of Poises				
40/50	3.49	7.78	53.90	87.50	69.20	51.9	91.0
85/100	.88	6.69	10.20	18.20	21.40	28.3	45.5
120/150	.43	.57	—	4.64	5.60	26.3	16.3
200/300	.13	.97	1.61	1.59	2.90	9.8	8.5
			Absolute Viscosity @ 140 F (60 C) in Poises				
40/50	2,492	6,622	37,732	44,782	38,269	34,500	36,310
85/100	1,018	5,443	8,322	14,272	35,510	10,200	20,520
120/150	542	822	—	3,627	3,962	7,120	7,520
200/300	258	1,006	1,410	1,423	2,329	4,4660	6,070
			Percent Asphaltenes by Weight				
40/50	30.1	33.6	30.2	32.7	29.6	35.6	33.00
85/100	26.2	36.4	29.4	—	35.5	36.3	33.4
120/150	28.1	32.5	—	26.3	28.4	31.4	30.6
200/300	20.0	26.4	26.4	26.9	27.3	30.2	29.8

Table B.14 Average rut depth in inches for Arizona. (Way 1978)

Site	1973	1974	1975	1976	1977
Sybil	.21	.23	.25	*olay	—
Wilmot	.45	.50	.59	.62	.65
Marana	.19	.20	.22	.22	.24
Casa Grande	.20	.23	.30	.31	.32
Williams Field	.16	.20	.21	.23	.26
Tonopah	—	—	.10	.12	.13
Upper Deer Valley	.14	.18	.27	.30	.33
Agua Fria	.16	.18	.20	.21	.25
Sunset Point	.15	.17	.20	.21	.22
Cherry	.17	.18	.20	.20	.21
Sedona	.18	.18	.20	.20	.22
Kachina	**conc	—	—	—	—
Bellemont	.28	.30	.33	.36	.42
Woody Mountain	conc	—	—	—	—
Winona	.15	.19	.23	.27	.30
Dead River	.14	.18	.24	olay	—
Alpine	—	.10	.12	.13	.15
Ash Fork	.08	.11	.15	.21	.26
Avondale	.02	.09	olay	—	—
Benson	.18	.22	.26	olay	—
Cosnino	conc	—	—	—	—
Cutter	.09	.10	.14	.16	.20
Flagstaff NB	.08	.16	.25	.26	.27
Flagstaff SB	conc	—	—	—	—
Gila Bend	.14	.14	.15	.15	.16
Minnetonka	olay-1972	—	—	—	—
Show Low	.24	.30	.37	.40	.41
Tempe	.05	olay	—	—	—
Topock	.20	.20	.22	.24	.24

*overlay

**concrete

Table B.15 Average percent cracking for Arizona. (Way 1978)

Site	1973	1974	1975	1976	1977
Sybil	2	2	3	*olay	—
Wilmot	80	80	80	80	80
Marana	3	5	7	9	12
Casa Grande	0	0	0	1	1
Williams Field	1	1	2	3	5
Tonopah	—	—	0	0	0
Upper Deer Valley	2	4	6	8	10
Agua Fria	1	1	1	1	2
Sunset Point	1	1	1	1	2
Cherry	1	1	1	1	1
Sedona	1	1	2	3	3
Kachina	**conc	—	—	—	—
Bellemont	70	70	70	70	70
Woody Mountain	conc	—	—	—	—
Winona	12	18	30	40	olay
Dead River	28	35	olay	olay	—
Crazy Creek	30	39	50	olay	—
Alpine	—	0	0	1	1
Ash Fork	15	20	24	29	35
Avondale	39	43	44	47	51
Benson	10	12	14	olay	—
Cosnino	conc	—	—	—	—
Cutter	32	32	35	40	50
Flagstaff NB	8	10	12	13	15
Flagstaff SB	conc	—	—	—	—
Gila Bend	0	0	0	0	0
Minnetonka	olay-1972	—	—	—	—
Show Low	75	75	75	75	75
Tempe	10	olay	—	—	—
Topock	0	0	1	1	1

*overlay
**concrete

Table B.16 Test results of recovered asphalt for Wyoming: Progressive studies (Gietz and Lamb 1968)

Sample Types: O = Original; S-R = Service, Recovered

Sample Location	Sample Type	Sample Age Yr.- Mo.	Crack Spacing	Pene-tration	Softening Point °F	Pene-tration Index	Calculated Stiffness Modulus Kg/Cm² 12 hr. 32°F	Pavement F/A Ratio	Air Voids Percent
Bosler No. 1	S-R	1-2	None	62	111	-2.1	0.2	0.54	3.2
	S-R	1-7	None	40	124	-1.7	1.5		
Bosler No. 2	S-R	1-4	None	45	117	-2.2	0.7	0.47	4.9
		1-10	None	27	129	-2.0	2.0		
Rock River-McFadden	O	0	--	77	110	-1.9	0.1		
	S-R	1-2	None	24	129	-1.7	3.0	0.28	5.0
	S-R	1-7	None	25	127	-1.6	3.0		
Rock River-North	O	0	--	77	111	-1.8	0.1		
	S-R	1-3	None	23	128	-1.8	4.0	0.40	5.6
	S-R	1-8	None	31	123	-1.9	4.0		
Sodergreen	O	0	--	71	112	-2.0	0.1		
	S-R	1-3	None	41	121	-1.8	1.5	0.30	6.3
		1-8	None	45	120	-1.5	0.6		
Summit No. 1	O	0	--	84	108	-2.2	0.2		
	S-R	0-4	None	12	126	-3.3	--	0.57	11.6
	S-R	0-8	None	40	120	-1.8	1.0		
Summit No. 2	O	0	--	79	109	-2.0	0.1		
	S-R	0-4	None	16	141	-1.1	3.0	0.52	7.2
Rock Springs Marginal									
Core No. 1 (104)	S-R	1-9	None	57	120	-0.9	0.2	0.21	6.6
Core No. 2	S-R	1-9	None	64	117	-1.3	0.2	0.29	4.8

Table B.17a Test results of recovered asphalt for Wyoming: Performance samples. (Gietz and Lamb 1968)

Sample Types: O = Original; S-R = Service, Recovered

Sample Location	Sample Type	Sample Age Yr.- Mon.	Crack Spacing	Pene-tration	Softening Point °F	Pene-tration Index	Calculated Stiffness Modulus Kg/Cm^2 12 hr. 32°F	Pavement F/A Ratio	Air Voids Percent
Woods Landing-West	S-R	8-6	None	46	134	+0.1	0.7	0.37	5.8
Mountain Home	S-R	8-6	69 ft.	23	128	-1.8	2.0	0.36	11.6
Sodergreen-West	S-R	2-6	None	41	124	-1.6	0.5	0.28	6.1
Herrick Lane	S-R	8-6	25 ft.	22	134	-1.4	5.0	0.27	9.0
Pahlow Lane	S-R	5-6	54 ft.	19	139	-1.1	6.0	0.28	9.7
Harmony Lane	S-R	5-6	103	22	141	-0.7	4.0	0.16	7.4
Flaming Gorge	S-R	3-0	None	59	113	-2.1	0.3	0.26	9.8
Walcott	S-R	2-0	None	45	114	-2.7	2.0	0.43	9.6
Table Rock Base	S-R	7	50 ft.	27	127	-1.8	4.5	0.61	11.6
Table Rock Surface	S-R	7	50 ft.	18	134	-1.7	10.0	0.46	8.1
Rock Springs Airport									
Core No. 1 (108)	S-R	2-9	None	30	127	-1.5	1.5	0.24	12.0
Core No. 2	S-R	2-9	None	55	125	-1.6	1.5	0.23	12.5
Rock Springs Superior Jct.									
Core No. 1 (113)	S-R	2-9	None	23	133	-1.6	5.0	0.49	8.4
Core No. 2	S-R	2-9	None	30	124	-2.0	3.0	0.46	9.0
Wamsutter									
Core No. 1	S-R	7	80 ft.	24	128	-1.8	5.0	0.24	4.8
Core No. 2	S-R	7	80 ft.	23	129	-1.8	5.0	0.21	5.5

Table B.17b Test results of recovered asphalt samples for Wyoming: Beacon Hill Site. (Gietz and Lamb 1968)

Sample Types: O = Original; S-R Service, Recovered

Sample Location	Sample Type	Sample Age Yr.-Mo.	Crack Spacing	Pene-tration	Softening Point °F	Pene-tration Index	Calculated Stiffness Modulus Kg/Cm² 12 hr, 32°F	Pavement F/A Ratio	Air Voids Percent
Beacon Hill									
North Site									
Northbound Lane									
Base	S-R	6-6	50 ft.	9	156	-0.6	50.0	0.76	15.4
Surface	S-R	6-6	50 ft.	15	149	-0.5	10.0	0.31	9.9
Southbound Lane									
Base	S-R	6-6	50 ft.	12	154	-0.3	21.0	0.52	14.2
Surface	S-R	6-6	50 ft.	10	160	-0.3	40.0	0.31	8.9
South Site									
Northbound Lane									
Base	S-R	6-6	Extreme	10	156	-0.5	25.0	0.47	12.7
Surface	S-R	6-6	(Spacing	10	161	-0.2	30.0	0.25	9.6
			exceeds						
Southbound Lane			1000 ft)						
Base	S-R	6-6		10	145	-0.5	6.0	0.52	15.9
Surface	S-R	6-6		18	141	-0.8	6.0	0.24	11.2

Table B.18a Results of IGLC tests for Wyoming: Progressive samples. (Gietz and Lamb 1968)

Sample Type: A-O = Original Asphalt; P.M. = Plant Mix (uncompacted); Core = Sample from Completed Road

Sample Location	Sample Type	Sample Age Yr- Mo	Penetration	Softening Point °F	Penetration Index	Calculated Stiffness Modulus Kg/Cm² 12 hr, 32°F	Phenol Initial Ip	Phenol ΔIp after Oxidation	Triethylamine Ip	Piperidine Ip	Air Voids (Pct.)	Crack Spacing
Bosler No. 1	A-O	0	--	--	--	--	128	34	-6	--		
	P.M.	0	--	--	--	--	127	25	-1	62		
	Core	0-9	--	--	--	--	130	24	2	62	3.0	None
	Core	1-2	62	111	-2.1	0.2	118	32	4	65		
	Core	1-7	40	124	-1.7	1.5	125	30	4	73		
Bosler No. 2	Core	1-4	45	117	-2.2	0.7	114	19	1	65	4.9	None
	Core	1-10	27	129	-2.0	2.0	120	13	5	52		
Rock River-McFadden	A-O	0	77	110	-1.9	0.1	125	11	-2	60		
	P.M.	0	--	--	--	--	122	11	0	51		
	Core	0	--	--	--	--	120	10	0	51		
	Core	0-8	--	--	--	--	118	10	4	51	4.6	None
	Core	1-2	24	129	-1.7	3.0	119	9	2	67		
	Core	1-7	25	127	-1.6	3.0	126	8	1	65		
Rock River-North	A-O	0	77	111	-1.8	0.1	123	11	-1	60		
	P.M.	0	--	--	--	--	125	6	2	53		
	Core	0	--	--	--	--	125	6	0	54	5.3	None
	Core	0-8	--	--	--	--	120	9	-1	56		
	Core	1-3	23	128	-1.8	4.0	123	10	6	∞ *		
	Core	1-8	31	123	-1.9	4.0	123	10	1	54		
Sodergreen	A-O	0	71	112	-2.0	0.1	131	29	-1	61		
	P.M.	0	--	--	--	--	134	23	-1	79		
	Core	0	--	--	--	--	127	26	1	53	6.2	None
	Core	0-10	--	--	--	--	131	21	6	65		
	Core	1-3	41	121	-1.8	1.5	126	28	1	67		
	Core	1-8	45	120	-1.5	0.6	135	18	3	68		
Summit No. 1	A-O	0	84	108	-2.2	0.2	118	40	-2	59		
	P.M.	0	--	--	--	--	126	31	0	57		
	Core	0-4	12	126	-3.3	N/A	126	28	6	69	11.5	None
	Core	0-10	40	120	-1.8	1.0	130	23	0	66		
Summit	A-O	0	79	109	-2.0	0.1	122	34	-1	61		
	P.M.	0	--	--	--	--	118	36	5	68	7.2	None
	Core	0-4	16	141	-1.1	3.0	120	31	1	63		

* Ip values in excess of 150 are reported as infinite.

Table B.18b Results of IGLC tests for Wyoming: Performance samples. (Gietz and Lamb 1968)

Sample Type: A-O = Original Asphalt; P.M. = Plant Mix (uncompacted); Core = Sample from Completed Road

Sample Location	Sample Type	Sample Age Yr- Mo	Penetration	Softening Point °F	Penetration Index	Calculated Stiffness Modulus Kg/Cm² 12 hr, 32°F	Phenol Initial Ip	Phenol ΔIp after Oxidation	Triethylamine Ip	Piperidine Ip	Air Voids (Pct.)	Crack Spacing
Herrick Lane	Core	8-6	22	134	-1.4	5.0	138	14	16	73	9.0	25 ft
Pahlow Lane	Core	5-6	19	139	-1.1	6.0	143	4	19	73	9.7	54 ft
Harmony Lane	Core	5-6	22	141	-0.7	4.0	127		7	98	7.4	103 ft
Woods Landing-West	Core	8-0	--	--	--	--	132	16	2	34	5.8	None
	Core	8-6	46	134	0.1	0.7	132	19	6	68		
Mountain Home	Core	8-6	23	128	-1.8	2.0	134	14	20		11.6	69 ft
Sodergreen West	Core	2-0	--	--	--	--	131	20	4	67	6.0	None
	Core	2-6	41	121	-1.8	0.5	127	27	7	98		

Table B.19 Asphalt properties and Spearman's rank correlation coefficient between transverse crack frequency and various physical parameters for West Texas. (Benson 1976)

			Reological Properties					Strength Properties @ 32°				
Site No.	Age (Years)	Trans. Crack Freq.	Visc. @ 77° (MP)	Hard. Index @ 77°	Visc. @ 275° (Stokes)	Pen. @ 32° (MM)	Pen. @ 77° (MM)	Asphalt Stiffness (KG/CM^2)	Schmidt M_R ($PSI \times 10^6$)	Direct Tension ($PSI \times 10^6$)	Indirect Tension ($PSI \times 10^6$)	% Air Voids
28	7	0	5.9	6.8	7.7	22	45	5	4.3		0.46	1.8
38	10	0	12.4	12.7	14.4	13	33	10	2.0		0.45	3.0
41	9	0	18.0	15.7	10.7	14	26	25	2.9		0.41	1.5
47	9	1	16.4	13.9	16.5	15	25	25	2.8		0.42	2.3
36	6 1/2	1	35.6	16.2	14.0	10	12	120	3.2		0.40	3.4
40	9	2	54.0	62.8	8.3	7	13	100	3.4		0.43	5.4
37	6	3	54.0	20.0	22.5	9	15	80	2.5		0.63	5.1
26	6 1/2	3	70.0	26.9	18.5	7	16	70	___†	___†	___†	6.4
39	10	3	76.0	79.2	14.1	5	10	230	4.1		0.39	5.8
		r	0.91*	0.85*	0.58	-0.75*	-0.68	0.74*	0.12		-0.10	0.89*

* Significant at the 95% Confidence Level.

† No cores available.

Temperatures are given in °F.

Table B.20 Asphalt properties and Spearman's rank correlation coefficient between percent voids and various parameters for Group B pizzas from West Texas. (Benson 1976)

Location	Pizza No.	Percent Voids	Penetration @ 77°F (mm)	Viscosity @ 77°F (MP)	S_{BIT} @ 32°F 1 Hr. Loading (KG/CM^2)	M_R @ 32°F (0.1 Sec. Loading) ($PSI \times 10^6$)
El Paso	17B	6.2	21	30	40	3.0
	26B	7.0	24	12	30	4.2
	29B	8.1	31	18	17	3.0
	11B	11.5	68	3.8	2	3.3
	9B	14.4	123	0.5	0.5	2.8
	10B	15.7	47	3.8	4	2.5
		ρ	0.83*	-0.81	-0.83*	-0.64
Bryan	5B	4.9	28	18	20	5.6
	6B	5.8	12	58	130	5.5
	4B	7.2	28	28	20	4.3
	12B	11.2	43	8.4	6	2.9
	2B	11.9	24	19	25	2.9
	14B	12.4	62	1.6	2	2.8
		ρ	0.50	-0.54	-0.47	-0.96*
Control	8B	5.9	14	28	100	4.1
	3B	6.4	33	11	11	3.4
	20B	6.5	22	40	40	2.1
	19B	9.2	32	14	12	3.9
	15B	15.3	110	6.2	0.5	1.5
	16B	16.4	36	11	8	1.5
		ρ	0.77	-0.53	-0.77	-0.79

* Significant at 95% Confidence Level.

Table B.21 Spearman's rank correlation between average recovered penetration and various hardening susceptibility indicators for West Texas. (Benson 1976)

Producer	Average Penetration	Original Penetration	After Actinic Light		After TFOT		Relative Viscosity	Vanadium Content (PPM)
			Viscosity @ 77°F (MP)	Hardening Index	Viscosity @ 140°F	Hardening Index		
8	90.3	132	39	81	1674	1.55	1.69	19
11	49.3	67	90	38	4815	2.49	2.83	31
1	32.7	55	56	25	2853	1.64	4.94	74
6	32.3	55	88	37	3726	1.81	4.02	55
3	15.0	68	84	50	4356	2.00	4.31	51
		-0.33	0.30	-0.30	0.40	0.40	0.70	0.60

Table B.22 Summary of physical properties of recovered asphalt cement for West Texas. (Anderson and Epps 1983)

	Highway Section					
Property	I-20	I-10	FM 1053	SH 18	US 285 (186)	US 285 (195)
Penetration at 77°F (25°C), dmm	18 22 15	15 15 14	21 21	31 55	25 5	32 20
Penetration at 39.2°F (4°C), dmm	15 7 10	9 5 9	10 12	19 22	10 0	12 11
Penetration Ratio*	83 32 67	60 33 64	48 57	61 40	40 -	38 55
Viscosity at 77°F (25°C), x 10^6 poises	29.0 21.0 38.0	23.0 32.0 47.6	14.8 21.8	8.4 6.4	15.6	18.0 22.0
Viscosity at 140°F (60°C) poises	**	**	**	**	**	**
Viscosity at 275°F (135°C), poises	10.5 7.9 11.2	10.8 12.1 11.7	11.5 12.0	7.7 3.6	5.9	7.5 10.9
Softening Point °F (°C) R & B	157(69) 155(68) 161(72)	161(72) 164(73) 167(75)	160(71) 153(67)	130(54) 126(52)	140(60) 185(85)	151(66) 156(69)

*$\frac{\text{Penetration}}{\text{Penetration}}$ at $\frac{39.2\ °F}{77°F}$ X 100, Loading $\frac{200g,\ 60\ sec,\ 39°F}{100g,\ 5\ sec,\ 77°F}$

**Too hard to test

Table B.23a Field condition evaluation data for West Texas, Summer 1974 (Anderson and Epps 1983).

	Highway Section					
	I-20	I-10	FM - 1053	SH 18	US 285 (186)	US 285 (195)
<u>Pavement Rating</u> PRS	34	75	82	90	92	87
Serviceability Index	2.4	4.0	4.4	4.2	4.5	4.4
Length of Cracking ft/100 ft (1974)	75	60	63*	0	0	91
Length of Cracking (1976)	180	120	0	0	0	141
Dynaflect $\bar{x}$	0.293	0.085	0.329	0.176	0.209	0.157
σ	0.071	-	0.051	0.084	0.075	-

*Later inspection in 1976 showed this cracking to be away from the test slab.

Table B.23b Predicted cracking temperature for West Texas--Limiting Stiffness of 1×10^9 N/m^2 (1.45×10^5 psi) (Anderson and Epps 1983)

Pavement Section	Softening Point R&B °F	Softening Point R&B °C	Predicted Cracking Temperature** °F	Predicted Cracking Temperature** °C
I-20	158	70	-4	-20
I-10	164	73	+1.4	-17
FM 1053	156	69	-5.8	-21
SH 18	128	53	-34.6	-37
US 285 (166)	140*	60*	-22	-30
US 285 (195)	154	68	-7.6	-22

*One test only.

**Temperature difference from R&B softening point for P. I. = -1.0 = 90°C from Van der Poel nomograph at 1/2 hour loading time.

Table B.24 Original and recovered asphalts from Texas field test sites 1 through 14. Viscosity at 77°F for sample from each site possessing highest microductility value. (Traxler 1967)

Site	Producer	Age of Sample Having Highest Ductility at 77 F.	Ductility @ 77 F. @ 0.5 cm./min., cms.	Viscosity @ 77 F. Megapoises 5 × 10^{-3} sec.$^{-1}$
3	8	Original	5.1	1.00
4	11	Original	4.3	1.17
9	3	Original	5.0	1.18
10	11	Plant	5.8	1.28
11	6	Plant	10.4	1.45
1	3	Plant	6.1	1.90
2	18	Paver	8.1	1.90
12	2	One Day	6.8	2.05
13	5	Two Weeks	6.9	2.26
8	7	Paver	7.3	2.40
6	7	Paver	6.9	2.60
7	15	One Day	4.2	2.85
14	6	Four Months	10.2	2.92

Table B.25 Asphaltene contents for Texas test sites, percent. (Traxler 1967)

Site	Refinery	Original	Extracted Asphalt After	
			One Year	Two Years
1	3	22.6	24.0	25.0
9	3	25.8	30.6	26.6
11	6	4.9	8.2	11.6
14	6	2.0	6.8	6.9
6	7	14.4	22.1	23.7
8	7	20.0	25.4	21.0
4	11	19.5	23.4	22.4
10	11	20.5	24.1	25.2
7	15	13.3	19.9	18.3
12	2	13.7	22.0	24.0
13	5	9.4	11.5	13.9
Average		15.0	19.8	19.9

Table B.26 Viscosities (poises) of petrolenes at 77°F for Texas. (Traxler 1967)

Site	Refinery	Viscosities of Petrolenes Poises, at 77 F., from Original Asphalt (1)	Viscosities of Petrolenes Poises, at 77 F., from Asphalt Extracted After 2 Years of Service in Pavement (2)	Ratio Vis (2)/Vis (1)
1	3	5,220	8,600	1.65
9	3	7,600	9,600	1.25
11	6	47,600	278,000	5.85
14	6	75,000	1,600,000	20.10
6	7	17,000	21,000	1.25
8	7	10,800	25,000	2.30
4	11	4,780	----	---
10	11	4,040	4,700	1.15
7	15	16,700	26,000	1.55
12	2	29,800	31,200	1.05
13	5	70,000	206,000	2.95
Average		26,230	221,010	3.9

Table B.27 Flow properties of original 90 penetrations asphalts and those recovered from mixtures with various aggregates for Texas. (Traxler 1967)

Site	Pro-ducer	Age of Sample	Sliding Plate Viscometer. at 5 × 10^{-2} sec.[1] Rate of Shear—Megapoises 77 F.	R.V.	95 F.	R.V.	Vacuum Capillary. Stokes 140 F.	R.V.	275 F.	R.V.	Pen. at 77 F. 100 gm./5 sec.
1	3	Original	1.15	--	0.15	--	1830	--	4.35	--	84
		Plant	1.90	1.65	0.20	1.35	2040	1.1	4.80	1.1	66
		Paver	--	--	--	--	--	--	--	--	--
		1 day	3.15	2.7	0.36	2.4	3610	2.0	5.35	1.2	52
		2 weeks	--	--	--	--	--	--	--	--	--
		4 months	10.3	8.95	0.55	3.7	6120	3.3	6.20	1.5	39
		1 year	10.4	9.10	0.67	4.5	7070	3.9	6.25	1.5	36
		2 years	10.6	9.25	0.93	6.2	8820	4.8	6.80	1.55	35.5
9	3	Original	1.18	--	0.17	--	1870	--	4.00	--	81.5
		Plant	2.85	2.4	0.34	2.0	3860	2.1	5.30	1.3	53.5
		Paver	2.82	2.4	0.43	2.5	4090	2.2	5.55	1.4	54.0
		1 day	2.80	2.3	0.41	2.4	3910	2.1	5.60	1.4	55.0
		2 weeks	5.56	4.7	0.48	2.8	4970	2.7	5.95	1.5	54.5
		4 months	8.76	7.4	0.84	7.1	6710	3.6	6.40	1.6	44.5
		1 year	12.00	10.2	1.44	8.5	10400	5.5	8.83	2.2	36.5
		2 years	13.30	11.3	1.45	8.5	11190	6.0	8.90	2.2	32.5
11	6	Original	0.57	--	0.06	--	1100	--	2.70	--	99.5
		Plant	1.45	2.6	0.14	2.6	2020	1.8	3.45	1.25	62.5
		Paver	--	--	--	--	--	--	--	--	--
		1 day	--	--	--	--	--	--	--	--	--
		2 weeks	2.05	3.6	0.20	3.8	2440	2.2	3.80	1.40	54.0
		4 months	8.60	15.2	0.34	6.4	3570	3.2	4.20	1.55	41.0
		1 year	12.40	22.0	0.63	11.9	5610	5.1	4.75	1.75	29.5
		2 years	15.40	27.0	0.70	13.4	6540	5.9	5.00	1.85	27.5
14	6	Original	0.58	--	0.075	--	1280	--	3.55	--	98.5
		Plant	1.03	1.8	0.096	1.3	1780	1.4	4.05	1.1	76.5
		Paver	--	--	--	--	--	--	--	--	--
		1 day	1.10	1.9	0.118	1.55	1910	1.5	4.25	1.2	70.5
		2 weeks	--	--	--	--	--	--	--	--	--
		4 months	2.92	5.0	0.168	2.25	2740	2.1	4.55	1.3	52.5
		1 year	6.20	10.6	0.304	4.05	3905	3.0	5.55	1.6	37.5
		2 years	7.24	12.7	0.530	7.10	6815	5.3	6.60	1.95	33.8
6	7	Original	0.96	--	0.11	--	1150	--	2.45	--	79.0
		Plant	2.50	2.6	0.22	2.0	1950	1.70	3.10	1.25	54.5
		Paver	2.60	2.7	0.23	2.1	2010	1.75	3.20	1.30	51.0
		1 day	3.00	3.1	0.25	2.3	2215	1.90	3.25	1.35	49.0
		2 weeks	5.35	5.6	0.46	4.2	2500	2.20	3.35	1.40	47.0
		4 months	20.80	21.5	1.06	9.6	5820	5.10	4.40	1.80	27.0
		1 year	23.60	25.0	1.52	10.4	7650	6.60	4.70	2.00	24.5
		2 years	38.20	40.0	2.34	21.2	11680	10.10	5.00	2.05	21.5
8	7	Original	0.86	--	0.11	--	1160	--	2.40	--	80.0
		Plant	2.35	2.7	0.34	3.1	1990	1.7	3.00	1.25	50.5
		Paver	2.40	2.8	0.22	2.0	2040	1.8	3.00	1.25	48.0
		1 day	2.80	3.25	0.38	3.5	2200	1.9	3.10	1.30	47.0
		2 weeks	6.56	7.6	0.47	4.3	3320	3.0	3.45	1.40	38.5
		4 months	12.40	14.3	0.73	6.6	4560	3.9	3.90	1.60	30.5
		1 year	15.00	17.5	1.04	9.5	7390	6.4	4.46	1.85	26.0
		2 years	28.60	33.0	2.26	20.5	12380	10.7	4.80	2.00	20.5
4	11	Original	1.17	--	0.13	--	1620	--	2.60	--	85
		Plant	1.96	1.70	0.23	1.75	2270	1.40	3.20	1.2	66
		Paver	2.06	1.75	0.26	2.00	2400	1.50	3.35	1.3	64
		1 day	2.36	2.00	0.28	2.15	2520	1.55	3.85	1.5	59
		2 weeks	2.56	2.20	0.38	2.90	2790	1.70	2.60	1.4	59
		4 months	11.40	9.70	0.98	7.50	6130	3.80	4.25	1.6	40.5
		1 year	12.40	10.65	1.06	8.20	7570	4.70	4.40	1.7	39.5
		2 years	15.00	12.80	1.85	14.20	15100	9.30	5.50	2.1	33

Table B.27 Continued.

Site	Producer	Age of Sample	Sliding Plate Viscometer, at 5 × 10⁻² sec.[1] Rate of Shear—Megapoises 77 F.	R.V.	95 F.	R.V.	Vacuum Capillary, Stokes 140 F.	R.V.	275 F.	R.V.	Pen. at 77 F. 100 gm./5 sec.
10	11	Original	1.12	--	0.13	--	1660	--	2.85	--	88.5
		Plant	1.28	1.15	0.16	1.20	2060	1.25	3.15	1.10	82
		Paver	1.76	1.55	0.20	1.55	2370	1.40	3.45	1.20	71
		1 day	2.10	1.90	0.22	1.70	2630	1.60	3.75	1.30	68.5
		2 weeks	3.20	2.85	0.38	2.90	3560	2.15	4.20	1.50	56
		4 months	9.30	8.30	0.76	5.80	6255	3.70	4.45	1.55	41
		1 year	15.40	13.80	1.84	14.10	16690	10.00	6.10	2.10	33
		2 years	23.00	20.50	2.46	18.90	20220	12.60	6.10	2.10	28
7	15	Original	1.00	--	0.11	--	1165	--	2.75	--	92.5
		Plant	1.75	0.75	0.17	1.55	1660	1.4	3.35	1.2	75
		Paver	1.85	1.85	0.24	2.2	2060	1.75	3.55	1.3	66
		1 day	2.85	2.85	0.31	2.8	2430	2.1	3.70	1.35	58
		2 weeks	4.35	4.35	0.33	3.0	2960	2.5	3.75	1.35	54
		4 months	13.20	13.20	0.86	7.8	5370	4.6	4.45	1.60	36.5
		1 year	14.50	14.50	1.28	11.6	8100	6.9	5.00	1.80	33.5
		2 years	21.40	21.40	2.20	20.0	15100	13.0	5.90	2.15	29
12	2	Original	0.86	--	0.088	--	1190	--	2.25	--	77
		Plant	1.20	1.4	0.116	1.3	1430	1.2	2.80	1.1	68
		Paver	1.40	1.6	0.130	1.5	1780	1.5	2.95	1.15	62
		1 day	2.05	2.4	0.196	2.2	1995	1.7	3.15	1.25	55
		2 weeks	2.20	2.6	0.210	2.4	2175	1.8	3.17	1.25	53.5
		4 months	4.95	5.7	0.400	4.5	3340	2.8	3.70	1.45	39.5
		1 year	13.20	15.3	0.690	7.8	8775	7.4	5.16	2.00	25.5
		2 years	25.00	29.0	2.300	26.0	15600	13.0	6.97	2.75	19
2	18	Original	0.90	--	0.09	--	1410	--	3.35	--	86
		Plant	--	--	--	--	--	--	--	--	--
		Paver	1.90	2.1	0.24	2.7	2450	1.75	4.15	1.25	52
		1 day	2.80	3.1	0.31	3.4	3200	2.30	4.35	1.30	44.5
		2 weeks	3.20	3.6	0.39	4.3	3340	2.40	4.65	1.40	40.5
		4 months	8.35	9.3	0.45	5.0	4510	3.20	4.90	1.45	35
		1 year	12.00	13.3	1.06	11.8	8210	5.80	6.00	1.80	32
		2 years	--	--	--	--	--	--	--	--	--
3	8	Original	0.98	--	0.13	--	2650	--	9.95	--	77
		Plant	2.30	2.3	0.30	2.3	4340	1.6	12.85	1.30	66
		Paver	2.65	2.7	0.35	2.7	4930	1.8	12.70	1.30	60
		1 day	2.78	2.8	0.34	2.6	4980	1.9	12.30	1.20	58.5
		2 weeks	3.50	3.6	0.54	4.1	6690	2.5	13.55	1.35	53
		4 months	5.94	6.0	0.59	4.5	7865	3.0	14.40	1.45	47
		1 year	--	--	--	--	--	--	--	--	--
		2 years	7.30	7.45	0.62	4.8	9225	3.5	14.50	1.47	44.5
13	5	Original	0.81	--	0.074	--	940	--	2.60	--	90
		Plant	1.78	2.2	0.200	2.7	1590	1.7	3.30	1.25	59
		Paver	1.98	2.5	0.200	2.7	1920	2.0	3.40	1.30	57.5
		1 day	--	--	--	--	--	--	--	--	--
		2 weeks	2.26	2.8	0.244	3.3	1990	2.1	3.5	1.35	52
		4 months	4.66	5.75	0.250	3.4	2200	2.3	3.65	1.40	48
		1 year	7.96	9.85	0.660	8.9	5690	6.0	4.35	1.65	30
		2 years	17.80	22.00	1.160	15.7	7790	8.3	5.20	2.00	26

Table B.28 Relative viscosities of the 13 asphalts hardened under various conditions and times, temperature 77°F for Texas. (Traxler 1967)

Site	1	2	3	4	6	7	8	9	10	11	12	13	14	Range	Average
Laboratory Test*	4.2	2.7	2.7	2.7	6.35	3.2	5.1	4.7	2.7	3.2	3.2	2.55	2.8	2.55 to 6.35	3.55
Plant	1.65	--	2.3	1.7	2.6	1.75	2.7	2.4	1.15	2.6	1.4	2.2	1.8	1.15 to 2.7	2.00
Paver	--	2.1	2.7	1.75	2.7	1.85	2.8	2.3	1.55	--	1.6	2.5	--	1.55 to 2.8	2.20
1 Day	2.7	3.1	2.8	2.00	3.1	2.85	3.25	2.3	1.9	--	2.4	--	1.9	1.9 to 3.25	2.60
2 Weeks	--	3.6	3.6	2.20	5.6	4.35	7.6	4.7	2.85	3.6	2.6	2.8	--	2.2 to 7.6	3.95
4 Months	8.95	9.3	6.0	9.70	21.5	13.2	14.3	7.4	8.3	15.2	5.7	5.75	5.0	3.3 to 21.5	9.90
1 Year	9.1	13.3	--	10.65	25.0	14.5	17.5	10.2	13.8	22.0	15.3	9.85	10.6	9.1 to 25.0	14.30
2 Years	9.25	--	7.5	12.8	40.0	21.4	33.0	11.3	20.5	27.0	29.0	22.0	12.7	7.5 to 40.0	20.5

* After Laboratory Hardening Test (see text).

Table B.29 Details of the bitumens used with the various surfacing mixtures from South Africa. (Jamieson and Hattingh 1970)

Mixture Type	Experimental Site	Details of Bitumen: Sample No.	Source of Crude Oil	Grade (nominal Pen. 25°C	Refining Process
Continuously-graded Premix (Grading No. 5)	Urban	734	Middle East	150/200	Straight-run. Refined in South Africa.
		736	Middle East	80/100	Straight-run. Refined in South Africa.
		744	Middle East	60/70	Straight-run. Refined in South Africa.
		752	Middle East	80/100	Air-rectified. Refined in South Africa (experimental).
		754	Middle East	60/70	Air-rectified. Refined in South Africa (experimental).
		758	Middle East	80/100	Air-rectified in presence of a copper sulphate catalyst. Refined in South Africa (experimental).
		762	Trinidad	80/100	Refined prior to importation.
		765	Trinidad	60/70	Refined prior to importation.
Continuously-graded Premix (Grading No. 6)	Urban	778	Blends of Trinidad Lake Asphalt (T.L.A.) and Middle East petroleum bitumen.	80/100	50% T.L.A. + 50% straight-run bitumen blend.
		777		80/100	25% T.L.A. + 75% straight-run bitumen blend.
		779	Middle East	80/100	Thermally cracked + straight-run. Refined in S.A.
* Continuously-graded Premix (Grading A)	Rural	883	Middle East	80/100	Straight-run. Refined in South Africa (Refinery "M").
		885	Middle East	80/100	Air-rectified. Refined on South Africa (Refinery "M").
		884	Middle East	80/100	Air-rectified. Refined in South Africa (Refinery "S").
Gap-graded Premix (BS 594, 55% stone content)	Urban	749	Middle East	40/50	Straight-run. Refined in South Africa.
		756	Middle East	40/50	Air-rectified. Refined in South Africa (experimental).
		760	Middle East	40/50	Air-rectified in presence of a copper sulphate catalyst. Refined in South Africa (experimental).
		767	Trinidad	40/50	Refined prior to importation.

* These bitumens were not normally replicates of those used in the Urban experiment. In particular the air-rectified bitumens were prepared differently.

Table B.30 Standard test results of the 150/200 pen asphalts. (Hattingh 1984)

Description of Sample	Penetration at 25 C/100 g/5 secs	Softening Point (Ring and Ball) C	Ductility at 25 C (mm)	Solubility in Trichloro-Ethylene Percent (m/m)	Thin-Film Oven Test Loss on Heating (%)	Penetration of Residue Percent of Original	Ductility of Residue at 25 C (mm)	n-Heptane-Xylene Equivalent (Spot Test)
Asphalt A	204	40.9	1400+	99.90	0.10	60.8	1400+	35
Asphalt B	167	40.0	1400+	99.90	0.13	62.3	1400+	30
Asphalt C	154	41.2	1400+	99.95	0.37	54.2	1400+	35
Asphalt D	195	39.9	1400+	99.95	0.38	57.4	1400+	35
SABS*Specification	150-200	36-43	1000	99.0	1.4	45	1000	35

*South African Bureau of Standards

Table B.31 Michigan roads evaluated with high performance liquid chromatography (HPLC). (Plummer & Zimmerman 1984)

Road No.	Road(1) Location	Year Constructed	Course Type(2)	Asphalt Content(2)	Crude Source(3)
Roads Exhibiting No Significant Cracking					
1	US-31, Muskegon Co,	1964	0.9 in. wear		
	2 mi. south	1964	1.0 in. leveling	4.9	Wyoming
	Lakewood Road, NBL	1964	2.1 in. binder		
2	I-96, Ottawa Co,	1961	1.1 in. wear		
	8.5 mi. Post, EBL	1961	1.1 in. leveling	4.7	Wyoming
		1961	2.1 in. binder		
8	M-52, Ingham Co,	1972	1.5 in. wear	5.8	
	0.1 mi. south Red	1964	0.2 in. seal		Bow River
	Cedar Bridge, NBL	1952	2.3 in. binder		
9	M-52, Ingham Co,	1972	1.9 in. wear	5.8	Wyoming
	Across from Roadside	1964	0.3 in. seal		
	Park Entrance, NBL	1952	2.8 in. binder		
10	M-52, Ingham Co,	1977	1.3 in. wear		
	Station 373 + 25,	1972	1.3 in. wear	5.6	Bow River
	NBL	1964	0.2 in. seal		& Wyoming
		1952	1.9 in. binder		
Roads Exhibiting Cracking					
3	US-27, Clare Co,	1961	0.8 in. wear		
	1 mi. north M-61	1961	1.0 in. leveling	5.0	Wyoming
	Bridge, NBL	1961	1.9 in. binder		
4	US-27, Clare Co,	1961	0.9 in. wear		
	1/2 mi. south Bailey	1961	0.8 in. leveling	4.9	Wyoming
	Drive Bridge, NBL	1961	2.1 in. binder		
5	US-27, Roscomon Co,	1961	0.7 in. wear		
	1/4 mi. south Canoe	1961	1.1 in. leveling	5.0	Wyoming
	Camp Road, NBL	1961	1.9 in. binder		
6A	US-27, Roscomon Co,	1976	1.1 in. wear		
	0.9 mi. north Canoe	1976	0.7 in. leveling	4.8	
	Camp Road, NBL				
6	US-27, Roscomon Co,	1961	0.9 in. wear		
	0.9 mi. north Canoe	1961	1.4 in. leveling	4.8	Wyoming
	Camp Road, NBL	1961	1.9 in. binder		
7	US-27, Roscomon Co,	1961	0.8 in. wear		
	1/4 - 1/2 mi. north	1961	1.4 in. leveling	4.8	Wyoming
	Canoe Camp Road, SBL	1961	2.1 in. binder		

Comments

(1) All samples taken in right wheel path.
NBL = north bound lane, EBL = east bound lane, SBL = south bound lane
Road 6A is an overlay on road 6

(2) Extractions performed by Michigan Highway Dept. during road construction.

(3) Bow River is a Canadian crude.

Table B.32 Indiana roads evaluated with high performance liquid chromatography (HPLC). (Plummer & Zimmerman 1984)

Road No.	Road(1) Location	Year Constructed	Course Type	Asphalt Grade
Roads Exhibiting No Significant Cracking				
1	I-64, east of US-37	1976	3/4 in sand surface	AE-60
	WBL	1976	full depth	AC-20
2	I-64, east of US-37	1976	1/2 in. sand surface	AE-60
	WBL	1976	full depth	AC-20
3	State 37, south of	1979	1 in. sand surface	AE-60
	I-64, SBL	1979	13 in. full depth	AC-20
4	State 37, south of	1979	5/8 in. sand surface	AE-60
	I-64, SBL	1979	16 in. full depth	AC-20
Roads Exhibiting Cracking				
5	I-64, 90 mi.	1976	1/2 in. sand surface	AE-60
	marker, WBL	1976	full depth	AC-20
6	State 37, 6 mi.	1980	1/2 in. sand surface	AE-60
	north of I-69, NBL	1980	3 in. binder	AC-20
		1980	7 in. No. 2 open grade	-----
		1980	5 in. subbase	-----
7	State 37, south of	1979	5/8 in. sand surface	AE-60
	I-64, SBL	1979	12 in. full depth	AC-20
8	State 37, south of	1980	1/2 in. sand surface	AE-60
	I-64, SBL	1980	14 in. full depth	AC-20

Comments

1) All roads were sampled in the right wheel path.
SBL = south bound lane NBL = north bound lane
WBL = west bound lane EBL = east bound lane

Table B.33 Asphalt properties for Pennsylvania (1961-1962 pavements). (Kandhal 1977)

Property	Beaver 1	Lycoming 2	Washington 3	Lebanon 4
Penetration at 77°F, 100 g, 5 s	96	69	76	76
Viscosity at 140°F, poises	2570	4024	3163	3000
Ductility at 60°F, 5 cm/min, cm	150+	150+	150+	150+
Ductility at 39.2°F, 5 cm/min, cm	12.5	8.0	5.9	7.8
Softening point, ring and ball, °F	118.0	122.0	123.2	126.4
Flash point, °F	510	505	580	585
Thin film oven test				
loss by weight, %	0.135	0.368	0.040	0.060
retained penetration, %	58.4	57.8	59.1	61.2
Rostler Analysis				
asphaltenes	26.3	26.4	19.3	22.2
nitrogen bases	29.9	35.4	23.1	26.4
first acidiffins	14.0	9.5	5.9	8.4
second acidiffins	22.7	20.2	38.4	31.2
paraffins	7.1	8.5	13.3	11.8
Properties of asphalt after mixing in pug-mill				
penetration at 77°F, 100 g, 5 s	60	47	67	56
viscosity at 140°F, poises	7273	15 158	3800	5100
ductility at 60°F, 5 cm/min, cm	39	19	24	45
ductility at 39.2°F, 5 cm/min, cm	5.3	4.2	4.7	5.4

Table B.34 Properties of asphalts studied. (Sisko and Brunstrum 1969)

	UNAGED			TFOT		
BPR NO.	PENETRATION [a] (DM)	VISCOSITY AT 140 F (POISES)	VISCOSITY AT 275 F (CS)	PENETRATION [a] (DM)	VISCOSITY AT 140 F (POISES)	VISCOSITY AT 275 F (CS)
			(a) 1954-55 SURVEY			
92	89	1410	283	47	3700	436
39	88	1660	385	51	4740	580
[b]	78	1460	357	46	4800	553
19	91	1920	426	53	5540	659
9	91	1820	411	51	5080	655
62	85	1610	430	55	4580	632
71	82	1620	388	54	2720	481
97	94	2150	267	36	6270	523
25	85	3000	620	51	11100	1120
74	93	1160	495	64	2120	662
185	78	1470	365	48	3290	480
114	94	1270	306	54	3370	455

[a] All penetrations in this and in other tables are in decimillimeters (dm) at 100 gm, 5 sec, and 77 F.
[b] Not part of the 1954-55 survey.

Table B.35 Properties of asphalts extracted from roads. (Sisko & Brunstrum 1969)

SOURCE	ASPHALT (%)	PENETRATION (DM)	VISCOSITY AT 140 F (KILOPOISES)	VISCOSITY AT 275 F (CS)
Calif.	4.55	19	23.6	895
	5.03	39	26.4	942
Ky.	6.60	16	145	1610
	6.75	21	84.2	1370
Ill.	3.84	38	9.29	691
	3.61	38	9.56	682
Md.	3.68	21	19.8	1100
	3.80	13	171	2520
Mass.	5.18	14	—	—
	5.13	30	16.2	1060
Neb.	4.65	64	3.66	590
	4.19	50	6.40	709
Okla.	4.57	48	3.42	549
	5.06	43	4.32	578
Ore.	5.88	38	5.25	471
	5.36	20	16.0	721
Tenn.	6.17	25	20.6	3140
	5.25	23	750	3710
Tex.	3.80	28	33.0	1980
	3.27	28	25.7	1830
Wis.	5.23	27	46.7	1170
	4.64	30	39.2	1080
Wyo.	5.68	50	2.34	379
	5.68	45	2.51	384

Table B.36 Asphalt compositions. (Sisko & Brunstrum 1969)

ASPHALT	SPECIMEN	COMPONENT[a] (%) A	HR	SR	O
Calif.	Unaged	15	28	35	22
	TFOT	19	25	34	22
	Road	24	30	22	24
Ky.	Unaged	12	20	40	28
	TFOT	15	19	39	27
	Road	24	28	24	24
Ill.	Unaged	16	20	37	27
	TFOT	20	20	34	26
	Road	22	24	29	25
Md.	Unaged	19	19	36	26
	TFOT	22	18	35	25
	Road	24	27	29	20
Mass.	Unaged	15	22	38	25
	TFOT	19	19	39	23
	Road	28	26	27	19
Nebr.	Unaged	11	24	39	26
	TFOT	14	22	38	26
	Road	15	24	35	26
Okla.	Unaged	8	26	44	22
	TFOT	10	27	42	21
	Road	12	32	36	20
Ore.	Unaged	15	28	30	27
	TFOT	20	26	28	26
	Road	21	29	26	24
Tenn.	Unaged	23	18	39	20
	TFOT	26	19	37	18
	Road	37	18	26	19
Tex.	Unaged	1	38	41	20
	TFOT	1	39	40	20
	Road	3	52	26	19
Wis.	Unaged	14	23	39	24
	TFOT	16	25	38	21
	Road	26	25	26	23
Wyo.	Unaged	15	20	43	22
	TFOT	18	21	40	21
	Road	18	23	35	24

[a] A = asphaltenes; HR = hard resins; SR = soft resins; O = oils.

Table B.37 Summary of road evaluations. (Sisko and Brunstrum 1969)

BPR NO.	ROAD LOCATION	TYPE	RIDE	CRACKING	PLASTIC DEFOR-MATION
92	California	Residential	3.5	Moderate	None
39	Kentucky	Gen. purpose	3.5[a]	Severe	Slight
—	Illinois	Heavy duty	4.0	None	None
19	Maryland	Heavy duty	3.5[a]	Severe	Slight
9	Massachusetts	Heavy duty	3.5	Severe	None
62	Nebraska	Heavy duty	3.5	Slight	Moderate
71	Oklahoma	Heavy duty	3.7	None	Moderate
97	Oregon	Heavy duty	4.0	Slight	None
25	Tennessee	Heavy duty	2.5	Slight	Severe
74	Texas	Heavy duty	3.5	None	None
185	Wisconsin	Heavy duty	2.2[a]	Moderate	None
114	Wyoming	Heavy duty	3.0	Slight	Slight

[a] Judged to be in need of service.

Table B.38 Saskatchewan sites assessed for rutting behavior. (Huber & Heiman 1987)

SITE NUMBER	LOCATION	STRUCTURE TYPE	TRAFFIC SPEED	ASPHALT CEMENT	YEAR OF CONSTRUCTION	AGE % LIFE	RUTTING PERFORMANCE
1	C.S. 1-8 EB km 4.24	Overlay	Fast	AC 1.5	1977	55	Poor
2	C.S. 1-8 EB km 1.44	Overlay	Fast	AC 1.5	1977	55	Fair
3	C.S. 1-19 WB km 4.6	Conv.	Fast	AC 1.5	1983	40	Poor
4	C.S. 1-19 WB km 7.0	Conv.	Fast	AC 1.5	1983	40	Poor
5	C.S. 1-19 WB km 9.0	Conv.	Fast	AC 6	1983	40	Fair
6	C.S. 1-19 EB km 19.5	Overlay	Fast	AC 6	1984	5	Good
7	C.S. 9-5 SB km 17.1	Overlay/ Recycle	Slow	AC 1.5	1982	15	Poor
8	C.S. 9-5 SB km 19.5	Overlay	Fast	AC 6	1981	25	Fair
9	C.S. 9-6 SB km 16.1	FDAC	Fast	AC 5	1972	90	Good
10	C.S. 22-2 WB km 24.75	Overlay Recycle	Slow	AC 1.5	1982	20	Good
11	C.S. 80-1 WB km 10.13	FDAC	Slow	AC 6	1975	200	Good

Table B.39 Testing program for Saskatchewan study sites. (Huber & Heiman 1987)

PROPERTY	DESIGN	CONSTRUCTION	PRESENT
Percent Asphalt	X	X	X
Aggregate Gradation	X	X	X
Crushed Aggregate Faces	X	X	X
Percent Air Voids	X	X	X
Percent VMA	X	X	X
Marshall Stability	X	X	X
Hveem Stability		X	X
Asphalt Penetration (25°C)			X
Asphalt Viscosity (60°C)			X
Aggregate BSG	X	X[1]	X[1]
Rice TSG	X	X[1]	X[1]
Rut Depth		X[2]	X
Layer Thickness	X	X	X

[1] Assumed equal to design
[2] Assumed equal zero

Table B.40 Summary of present asphalt concrete parameters for Saskatchewan sites. (Huber & Heiman 1987)

		PRESENT CONDITION (ROAD)									
		ASPHALT CONTENT,	AIR VOIDS,	VMA,	MARSHALL STABILITY,	CRUSHED FACES	DENSITY,	HVEEM STABILITY,	FLOW,	PENETRATION	VISCOSITY,
SITE	LOCATION	%	%	%	N	%	kg/m^3	%	0.25 mm	0.1 mm	pa-sec
1	C.S. 1-8 km 4.24	5.2	0.5	10.6	2265	61.9	2486	40.5	11	94	34.7
2	C.S. 1-8 km 1.44	4.8	3.1	12.0	5989	64.0	2432	43.7	16	95	100.9
3	C.S. 1-19 km 4.6	6.3	1.4	13.6	1965	67.7	2415	26.8	11	225	36.6
4	C.S. 1-19 km 7.0	6.0	1.8	13.4	2199	66.6	2417	24.4	11	236	33.0
5	C.S. 1-19 km 9.0	5.0	3.3	12.5	2743	53.9	2417	46.2	10	166	43.9
6	C.S. 1-19 km 19.5	5.3	8.0	17.3	2196	94.6	2291	46.2	12	95	192.8
7	C.S. 9-5 km 17.1	5.9	0.9	13.5	4396	-	2419	23.8	19	119	126.4
8	C.S. 9-5 km 14.2	5.8	1.3	13.4	4196	49.0	2418	29.9	14	119	126.4
9	C.S. 9-6 km 16.1	5.4	3.7	14.1	5669	56.6	2408	28.6	17	42	384.3
10	C.S. 22-2 km 24.75	5.4	1.4	12.6	5513	65.1	2433	46.1	15	108	118.9
11	C.S. 80-1 km 10.13	5.9	2.0	13.1	5760	56.4	2422	33.1	16	122	124.7

• All values shown are the average of individual test results.

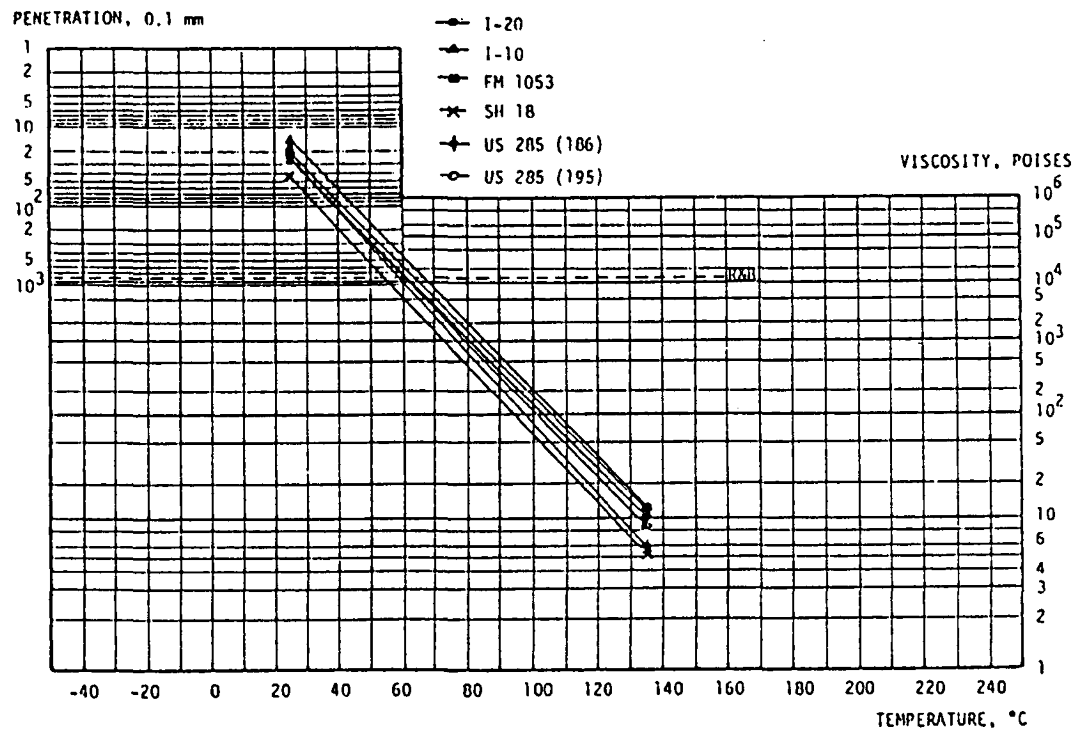


Figure B.1 Recovered asphalt cement properties from all sections plotted on the bitumen test data chart for West Texas. (Anderson and Epps 1983)