SHRP-A-623

Review of State and Industry Reports on Asphalt Properties and Relationship to Pavement Performance

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Contents

List of Figures

List of Tables

Abstract

This report reviews the results of ten studies obtained from State Highway Agencies and industry representatives on the relationships between asphalt binder and mix properties, both physical and chemical, and the field performance of pavements. The reports reviewed were in the form of internal documents that had not been published in the usual sources, e.g. Transportation Research Board, or the Association of Asphalt Paving Technologists. A companion report, Asphalt Properties and Relationships to Pavement Performance: Literature Review (SHRP-A/IR-90-015), is available which summarizes information on the "published" information. Other literature reviews within the project "Performance Related Testing and Measuring of Asphalt Aggregate Interactions and Mixtures" have been prepared of which this report is a part.

A brief description of each study is included, together with their results. Although confounding factors were present in each study reviewed, some qualitative and quantitative relationships were established. Four areas of pavement performance were measured in these studies: stripping, low temperature transverse cracking, rutting and bleeding. Some of the quantitative relationships go against established norms such as Palsat's equation for the relationship between cracks and percent compaction. The report makes recommendations on asphalt selection which would decrease the incidence of transverse cracking but concludes that each study did not track similar elements in order to make adequate comparisons.

Executive Summary

As part of SHRP Contract A-003A entitled "Performance Related Testing and Measuring of Asphalt Aggregate Interaction and Mixtures," several literature reviews have been prepared of which this report forms a part. One of the objectives of this contract is to assimilate information in the technical literature relating chemical and physical properties of asphalts to pavement performance and mixture properties.

In June 1989, a letter requesting information related to the SHRP A-003A project was sent to SHRP representatives in all 50 states as well as the District of Columbia, other U.S. territories, and Canada. A select group of representatives in industry was also contacted for information. From this initial contact, reports were received and reviewed--these reviews form the basis of this report. The request emphasized the importance and need for information relating field performance to asphalt properties.

A companion report, Asphalt Properties and Relationships to Pavement Performance: Literature Review (SHRP A/IR-90-015), summarizes information on the "Published" technical literature. This report is similar in nature but concentrates on information obtained from direct contacts with the states and industry groups. A significant amount of information was collected which had not been published in the usual sources, e.g., Transportation Research Board or Association of Asphalt Paving Technologists. However, many of the reports received had been published in some form in the technical literature and were therefore not included for review in this report.

This survey of state and industry research reports was designed to provide information which could relate insitu 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 a 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 <u>all</u> 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

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. A total of 10 reports were included for review:

1. Michigan--Thermal cracking and rutting
2. Iowa--Transverse cracking
3. S. Dakota--Transverse cracking
4. Univ. of Alberta, Canada--low temperature transverse cracking
5. S. Carolina--stripping
6. Colorado--rutting and cracking
7. Alberta--permanent deformation
8. Alaska--asphalt concrete properties and performance
9. Esso--low temperature service
10. Iowa--aging

It is important to note some of the problems which adversely affected the reviewers' ability to develop any consensus for the review of the enclosed reports:

1. Confounding factors with regards to structural section, asphalts, aggregates, traffic, test methods, and performance measurements. For example, while Palsat (1986) was able to develop regression equations relating transverse cracking to pavement properties, he does not indicate the type of aggregate used in the mix. In the South Dakota study, Crawford & Anderson (1968) indicated that the use of limestone aggregate vs. crushed gravel affected the pavement performance with regards to transverse cracking. Another example is the work performed by Busching et al. (1986) in South Carolina where the causes of stripping were studied. Their work did not include the effects of traffic which Davis (1986) noted was an important factor in stripping failures.

2. The majority of these projects evaluated asphalt properties using traditional tests such as penetration, viscosity and Marshall or Hveem stability. 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>

It is recognized that problems do exist in attempting to interpret information from these reports as discussed earlier; however, a cautious effort was made to extract the following qualitative relationships.

1. Stripping: High air voids contents as a result of poor compaction, use of hot plant mixed seal courses, or open graded friction courses could be possible causes for increased stripping of asphalt concrete pavements. However, the effects of high truck tire pressures could also be a factor, as well as drainage.

2. Low Temperature Transverse Cracking: An increase in low temperature transverse cracking could be associated with an increase in:
   - o Asphalt and/or asphalt-aggregate system stiffness;
   - o Ratio of the recovered penetration;
   - o Air voids in mixtures;
   - o Thinner pavements; and
   - o Use of asphalts with high temperature susceptibilities.

3. Rutting and Bleeding: Over-asphalting, lower than design air voids in the wheel paths increase the chances of bleeding. From this review, it appears that rutting does not correlate well with the commonly measured asphalt rheological properties. The reviewed reports indicate that lack of shear strength in the asphalt concrete is the primary cause of rutting.

In addition, some quantitative relationships were available from reports received. Some of the trends in these performance models go against established norms. A good example is the relationship reported by Palsat (1986):

$$CPK = 153.28 + 2.65*AGE + 0.40*OAS - 2.37*COMP \qquad R^2=0.60$$

where:

CPK = Cracks per kilometer
AGE = Pavement age, years
OAS = Original asphalt stiffness, kg/cm^2
COMP = Compaction, %

The negative sign for percent compaction indicates an inverse relationship between cracking frequency and percent compaction. This conflicts with Van der Poel's (1955) relationship for asphalt stiffness, i.e., for a given asphalt content, an increase in the pavement density results in an increase in the pavement stiffness which would result in an increase in the potential for low temperature cracking>

An attempt also was made to test the applicability of the results reports in one report using data from other reports. For example, the recovered penetration ratios were calculated using the data reported by Palsat (1986) and the Cracking Indices calculated using the relationship reported by Defoe (1988). These indices were then compared to the cracking data collected by Palsat (1986). The predicted cracking performance did not match the actual field data. Hence, after a few other such attempts, no further investigation in this direction were conducted.

Finally, the reviewed reports made some recommendations with regard to performance-based specifications.

1. Low Temperature Transverse Cracking: Palsat (1986) recommends using a critical asphalt stiffness of 2.9 x 10^6 Pa (30 kg/cm) as a upper limit for improved cracking performance. This value was predicted using McLeod's method and based on original asphalt characteristics and site specific temperature (at a pavement depth of 50 mm) conditions. Crawford and Anderson (1976) suggest using a softer asphalt with 100% quarried limestone aggregate to improve resistance to transverse cracking. Marks' (1984) research indicates the beneficial effects of sawed transverse joints spacing to prevent low temperature transverse cracking and is cautious in extrapolating the results of his study. Robertson (1987) developed a design chart for selecting asphalt cements to resist low temperature cracking. This chart uses measured values of asphalt stiffness and a stress calculation method which takes into account the variation in relaxation times during the cooling period. This chart gives estimates of thermal cracking temperatures which can be compared to the low temperatures in the area where the asphalts are to be used. Thus, an appropriate asphalt can be selected to reduce low temperature transverse cracking.

2. Stripping: No new specifications were reported for controlling stripping. However, Busching et al. (1986) suggest the incorporation of moisture susceptibility testing into the Marshall mix design method.

3. Rutting and Bleeding: No new specifications are reported for controlling bleeding and rutting of asphalt pavements.

In conclusion, two points should be mentioned:

1. Construction practices influence the in-service rheological properties of the asphalt-concrete systems. For example, if there is a large deviation in field air voids from the specified air voids, these higher air voids (as some investigators showed) result in increased hardening and hence greater susceptibility to cracking. In another case, the higher air voids content showed an increased susceptibility to stripping. These deviations overshadow the actual asphalt properties being studied, resulting in a large scatter in data: the end result is the lack of available correlation between asphalt-system rheology and field performance.

2. Most of these studies, with the exception of Colorado (O'Connor, 1979), did not attempt any asphalt compositional studies. Fractional separation techniques could provide a method for following changes in an asphalt. An attempt has been made

to present the various tests currently in use for determining asphalt composition (see Appendix B). State-of-the-art work has been done in developing this analytical field; what remains to be studied is the availability of associated performance data.

1

Introduction

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.

Pre-implementation activities focused on the preparation of a final research program which culminated with the publication of the Strategic Highway Research Program - Research Plans - Final Report (1986) for each of the original six research areas. 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 Contract A-003A entitled "Performance Related Testing and Measuring of Asphalt-Aggregate Interactions and Mixtures," of which this report forms a part.

SHRP Contract 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.

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. The second is 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 will include additional information from unpublished information or on-going research including SHRP asphalt-related contracts. The information collected 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, mixtures 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.

Objectives and Scope of Report

In June 1989, a letter requesting information related to this project was sent to SHRP representatives in all 50 states as well as the District of Columbia, other U.S. territories, and Canada. A selected group of representatives in industry was also contacted for information. From this initial contact, reports were received and reviewed–these reviews form the basis of this report. The request emphasized the importance and need for information relating field performance to asphalt properties.

A companion report (Finn et al. 1990) that summarizes information in the "published" literature has also been completed. This report herein is similar in nature but concentrates on information obtained from direct contacts with the states and industry groups. A significant amount of information was collected which had not been published in the usual sources, e.g., AAPT or TRB. However, many of the reports received has been published in some form and were therefore not included for review in this report. The objectives are to collect, summarize and analyze information obtained from states and industry (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 to determine if any consensus exists in the literature regarding the above relationships. Threshold values, regression or graphical relations are presented with as much specificity as possible. 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 asphalt of greatest interest are 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.

Organization of Report

This report contains four chapters. Chapter 2 provides in-depth reviews of the reports received from the states and industry. The majority of these reports are test sections or field trials built by the states that have as one of their objectives the relationship of asphalt properties to pavement performance. Chapter 3 summarizes the results of the information reviewed to determine if any consensus exists in the literature regarding the relationship between asphalt properties and pavement performance. Finally, Chapter 4 contains a bibliography of the reports reviewed.

2

Review of State and Industry Reports

This chapter summarizes information contained in reports provided by state highway agencies and by ESSO, as obtained from field trials. For each test road, a description is provided, together with the asphalt properties measured and pavement performance results. The results, conclusions and relationships derived (if any) are also presented.

Michigan DOT - Thermal Cracking and Rutting

Description

Field measurements were made for nine selected pavement sections in Michigan (Defoe, 1988). These measurements included crack surveys (transverse thermal cracking), rut depths and deflection measurements (using a Benkelman beam). Information was provided regarding the type and grade of asphalt used. No information is provided regarding the source of asphalt. Since the projects were widely scattered throughout the state, it is reasonable to assume that the asphalts came from different sources. A summary of the field performance measurements is presented in Table 2.1. Sections 1, 2, 5-9 consisted of a 4 inch bituminous surfacing (binder, leveling and wearing course), 10 inches of aggregate base, and 15 inches of sand sub-base placed on the subgrade (the asphalt cement used was 60/70 pen. grade). Section 3 had been surfaced with a 1 inch oil-aggregate mixture then resurfaced with two bituminous layers using 85/100 penetration asphalt cement in 1956 and a final bituminous course in 1970 made with 120/150 penetration asphalt cement. The total thickness of all bituminous layers in Section 3 was essentially the same as for the seven sections. Section 4 differed from the other sections in that it consisted of only 2-1/2 inches of oil-aggregate surfacing placed on a 5 inch aggregate base.

Cores were sampled from all nine sections. These cores were tested in the laboratory for tensile and thermal properties and for resilient modulus values over a wide range of temperatures. Asphalts recovered from the core samples were tested for penetration and viscosity at several temperatures in order to determine temperature susceptibility. Cores were also analyzed for asphalt content, air voids, aggregate gradation and the

Section	Cracking* Index	Rut Depth, in. Outer Wheel Path	Inner Wheel Path	Maximum Deflection mils	Age**	Traffic, 18 KEALS millions
1	37.5	0.20	0.45	12.6	20	1.38
2	28.3	0.19	0.36	12.9	20	1.38
3	17.5	0.03	0.02	29.0	25	0.32
4	0	0.01	0.02	14.5	34	0.35
5	20.3	0.19	0.25	12.0	19	0.43
6	0	0.18	0.23	12.5	20	0.34
7	4.5	0.14	0.19	15.4	17	0.78
8	0	0.10	0.18	13.1	17	0.76
9	0.2	0.09	0.17	12.1	17	0.84

* Defoe (1988) does not explain how he calculated the cracking index.
** Years of Service at time of 1981 field measurements.

Table 2.1. Summary of field performance measurement in Michigan (Excerpt, Defoe, 1988).

thermal contraction coefficient. Three different stiffness parameters were obtained using different test procedures:

- Failure Stiffness, E
- Resilient Modulus[1], M_r
- Creep Modulus, S_c

The failure stiffness was obtained from indirect tensile breaking tests using the stress and strain values at specimen failure. The resilient modulus was measured at room temperature (74°F) and at 0.1 second load duration. The load was 75 lbs. applied at every 3 seconds. The creep modulus was measured at room temperature (74°F). The results are shown in Table 2.2.

SECTION	FAILURE STIFFNESS, E psi x 10^3	RESILIENT MODULUS, M_r psi x 10^3	CREEP MODULUS, S_c psi x 10^3
1	34	935	17
2	30	735	13
3	21	291	10
4	11	38	4
5	21	532	9
6	20	332	7
7	22	386	13
8	14	313	7
9	18	461	10

Table 2.2. Characteristics of Bituminous mixtures as determined by laboratory testing of core specimens in Michigan (Excerpt, Defoe, 1988).

[1]Defoe (1988) does not indicate test apparatus used.

Tables 2.3 and 2.4 show some of the asphalt binder properties that were used in performance correlations. Defoe (1988) also provides more comprehensive data on asphalt properties of the leveling and wearing courses.

	RECOVERED ASPHALT CEMENT					PENETRATION INDEX	
	PENETRATION		RECOVERED PENETRATION OF ORIGINAL	VISCOSITY		McLEOD'S	HEUKELOM'S
SECTION	@ 25°C-100g 20 sec. dmm	@ 4°C-200g 60 sec.dmm	%	Absolute 140°F,Poises	Kinematic 275°F, cS	METHOD	METHOD
1	28.4	12.7	47.3	16,520	866.3	-0.51	-2.1
2	23.1	10.0	38.5	18,827	923.4	-0.64	-3.1
3	-	-	-	14.6*	-	-	-
4	-	-	-	20.9*	-	-	-
5	28.8	13.0	48.0	13,312	790.1	-0.62	-2.0
6	43.4	18.0	72.3	5,393	603.8	-0.55	-1.4
7	51.4	20.0	85.7	5,783	561.8	-0.45	-1.3
8	52.0	22.4	86.7	3,800	490.4	-0.65	-1.0
9	43.8	18.7	73.0	5,106	548.7	-0.67	-1.2

* Cone plate viscosity @ 77°F, K poises.

Table 2.3. Properties of asphalts recovered from paving mixtures (binder only) in Michigan (Excerpt, Defoe, 1988).

	RECOVERED ASPHALT CEMENT					PENETRATION INDEX	
	PENETRATION		RECOVERED PENETRATION OF ORIGINAL	VISCOSITY		McLEOD'S	HEUKELOM'S
SECTION	@ 25°C-100g 20 sec. dmm	@ 4°C-200g 60 sec.dmm	%	Absolute 140°F,Poises	Kinematic 275°F, cS	METHOD	METHOD
1	28.2	13.6	47.0	13,334	800.0	-0.61	-1.7
2	29.3	14.6	48.4	10,354	747.8	-0.69	-1.3
3	-	-	-	-	-	-	
4	-	-	-	-	-	-	
5	27.3	10.2	45.5	19,700	1029.1	-0.34	-3.2
6	31.6	15.7	52.7	9,651	764.3	-0.59	-1.3
7	53.7	20.9	89.5	5,037	560.6	-0.40	-1.3
8	45.2	22.3	75.3	5,037	541.6	-0.64	-0.5
9	51.9	21.1	86.5	4,682	544.2	-0.51	-1.3

Table 2.4. Properties of asphalts recovered from paving mixtures (top/wearing course only) in Michigan (Excerpt, Defoe, 1988).

Results

The following relationships were developed relating asphalt cracking performance to asphalt physical properties:

1. Cracking as indicated by the Cracking Index was shown to be related to the stiffness of the bituminous mixture (Figure 2.1). The stiffness parameter used here is the failure stiffness, E. The relationship is presented as follows:

   $CI = (2.04)E - 33.366$ ($R^2 = 0.824$, $S_y = \pm 7.172$)

   Where: CI = Cracking Index

   E = Failure stiffness, psi

2. Hardening of asphalts appeared to affect cracking. Hardening was measured in terms of the ratio of the penetration of the recovered asphalt to the penetration of the original asphalt. The following relationships were developed relating recovered penetration ratio to the cracking index:

   CI = 57.983 - 0.698(RP[2]) ($R^2 = 0.773$, $S_y = \pm 8.150$)

   CI = 48.992 - 0.567(RP[3]) ($R^2 = 0.502$, $S_y = \pm 12.064$)

   Where: RP = recovered penetration ratio, percent

3. Figure 2.2 shows the relationship between the temperature susceptibility of the asphalt cements. Temperature susceptibility was measured in terms of Penetration Index (PI) using two methods: the Modified Heukelom Method and the McLeod Method. The modified Heukelom Method uses penetration at 39.2°F and 77°F together with the viscosity at 140°F to determine the PI. McLeod uses the penetration at 77°F and the viscosity at 275°F. The PI calculated by the modified Heukelom method related better to the cracking performance as compared to the PI's determined by the McLeod Method. Defoe (1988) suggests that the lower temperatures of the modified Heukelom Method are thought to be in the range which generates thermal cracking on the roadway and probably accounts for the better correlation.Defoe's statement holds for the average PI's calculated using the modified Heukelom Method. During the course of the review, ARE Inc. analyzed

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[2] Binder only.

[3] Wearing/Top course only.

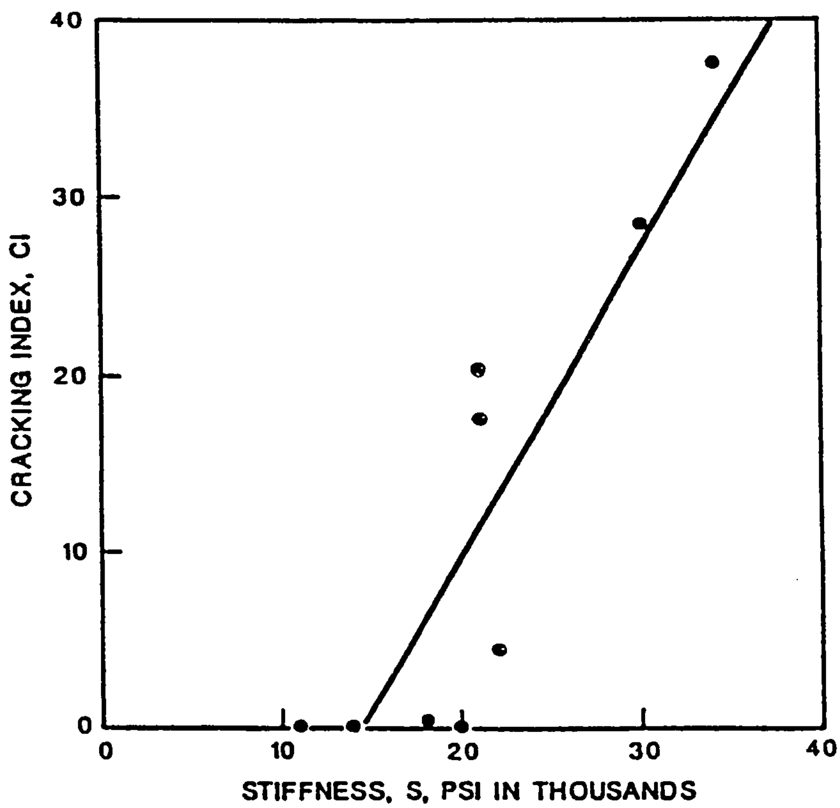

Figure 2.1 Relationship between cracking index and mixture stiffness measured by the indirect tensile test at 74°F in Michigan (Defoe, 1988).

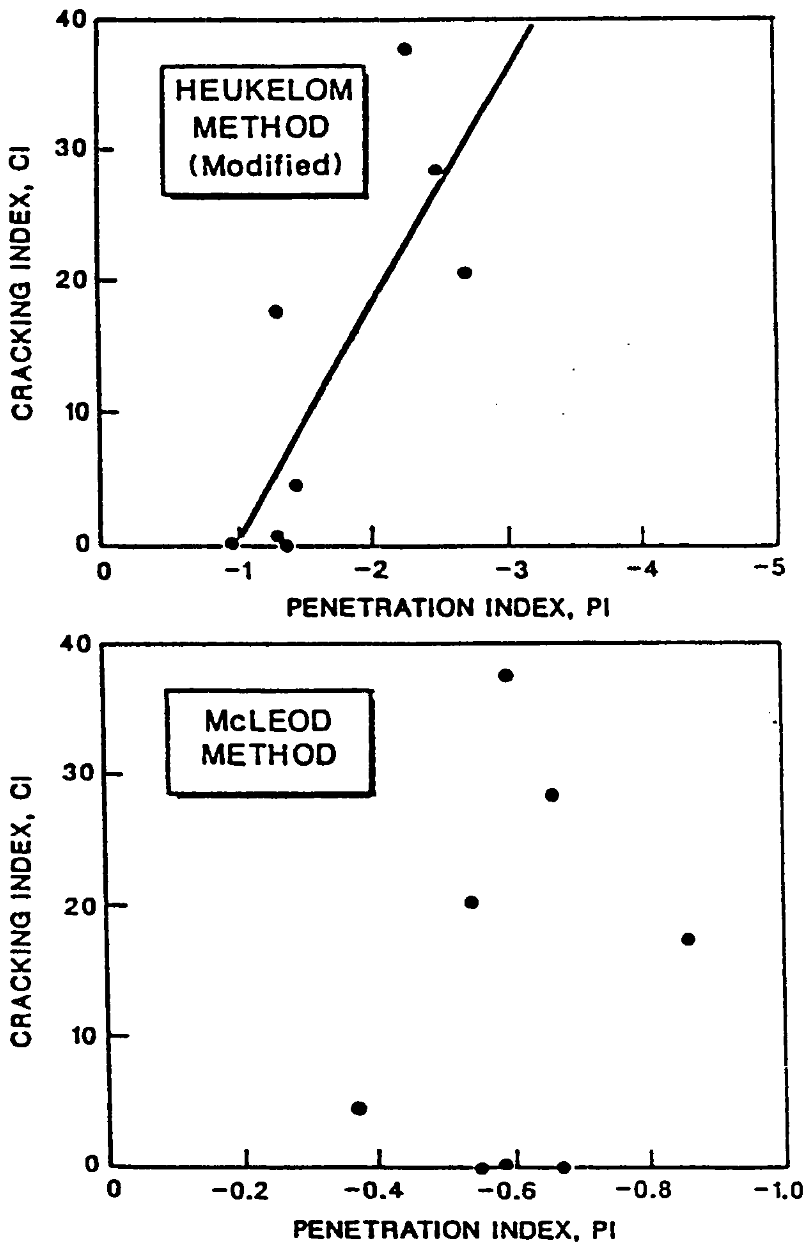

Figure 2.2 Relationship between pavement cracking index and penetration index of recovered asphalt cements in Michigan (Defoe, 1988).

the data further. The PI's for the wearing/top course calculated using themodified Heukelom method show a very poor correlation with the cracking index. These relationships are presented as follows:

a.[4] $CI = 0.337 - 8.343(PI)$ $(R^2 = 0.195, S_y = \pm 15.343)$

b.[5] $CI = -17.449 - 17.599(PI)$ $(R^2 = 0.677, S_y = \pm 9.72)$

c.[6] $CI = -17.061 - 17.649(PI)$ $(R^2 = 0.625, S_y = \pm 5.1)$

Where: CI = Cracking Index

PI = Penetration Index calculated using the Modified Heukelom Method.

The Pen-Vis numbers (PVN) using the penetration @ 77°F and the viscosity @ 275°F for these 9 sections were calculated. Table 2.5 lists the properties used. The correlation between PVN and CI was very poor.

SECTION	PENETRATION @ 77°F, dmm	VISCOSITY @ 275°F, cS	L(1)	M(2)	PVN(3)
1	28.2	800	1300	400	-0.618
2	29.3	747.8	1250	390	-0.662
3	-	-	-	-	-
4	-	-	-	-	-
5	27.3	1029.1	1350	410	-0.342
6	31.6	764.3	1150	380	-0.553
7	53.7	560.6	700	240	-0.311
8	45.2	541.6	880	280	-0.636
9	51.9	544.2	750	270	-0.471

(1) Viscosity @ 275°F, cS, for the penetration @ 77°F for a PVN of 0.0 using Figure A.1, for a particular asphalt cement.

(2) Viscosity @ 275°F, cS, for the penetration @ 77°F for a PVN of -1.5 using Figure A.1, for a particular asphalt cement.

(3) Calculated, after McLeod (1972).

Table 2.5. PVN's (wearing/top course only) calculated for Michigan sections using data presented by Defoe (1988)

Correlations between cracking index and mixture stiffness, penetration index of recovered asphalts (using both the Modified Heukelom and McLeod Methods) and

[4]This relationship uses PI for wearing/top course only.

[5]This relationship uses PI for binder only.

[6]This relationship uses PI averaged for all three AC layers, i.e., wearing course, leveling course and the binder.

recovered penetration are presented by Defoe (1988). Correlations were also developed by ARE: these correlations are considered relatively poor. These correlations (ARE, 1990 and Defoe, 1988) are, at best, only indicators of trends between asphalt properties and performance.

A direct, rather than inverse, relationship between rutting and mix stiffness was obtained (Figure 2.3). Defoe (1988) suggests that this indicates the measured rutting was caused by the deformation in the underlying granular layers or the subgrade rather than the bituminous layers. This is borne out further by Figure 2.4.

The air void content in the pavement mixtures is considered as one factor which influences the hardening of asphalt after it has been placed on the roadway. Figure 2.5 indicates a fair relationship between the air voids and the ratio of the recovered penetration in both the top course and the binder and leveling courses. Grouping sections 1, 2, 5 as cracked sections (average CI = 28.7) and sections 6, 7, 8, 9 (Average CI = 1.2) as uncracked sections, the average percent air voids (wearing/top course only) are 1.64 (σ = 0.44) and 1.01 (σ = 0.24), respectively. The relationship between the air voids and CI is poor:

$$CI = -7.962 + 16.336(\text{Air Voids, \%})(R^2 = 0.246,\ S_y = \pm 14.847)$$

Conclusions

1. Transverse cracking of pavements was found to be directly related to the failure stiffness, the ratio of the recovered penetration, and temperature susceptibility of asphalts.

2. Increased thicknesses of granular bases appeared to have contributed to an increase in rutting, despite the stiffening of the asphalt surface course.

3. Increased air voids in mixtures result in increased stiffness of the mixture after paving. This is due to asphalt hardening.

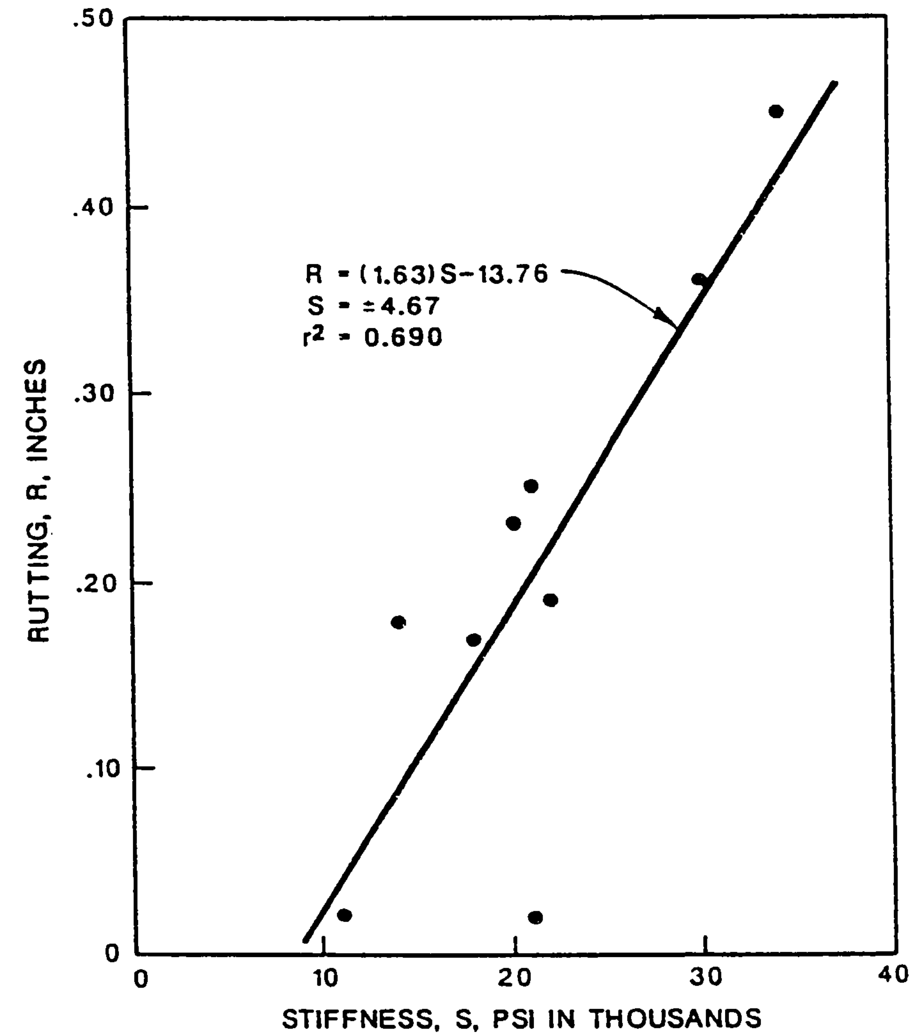

Figure 2.3 Relationship between rutting of the inner wheel path and mix stiffness measured by the indirect tensile test at 74°F in Michigan (Defoe, 1988).

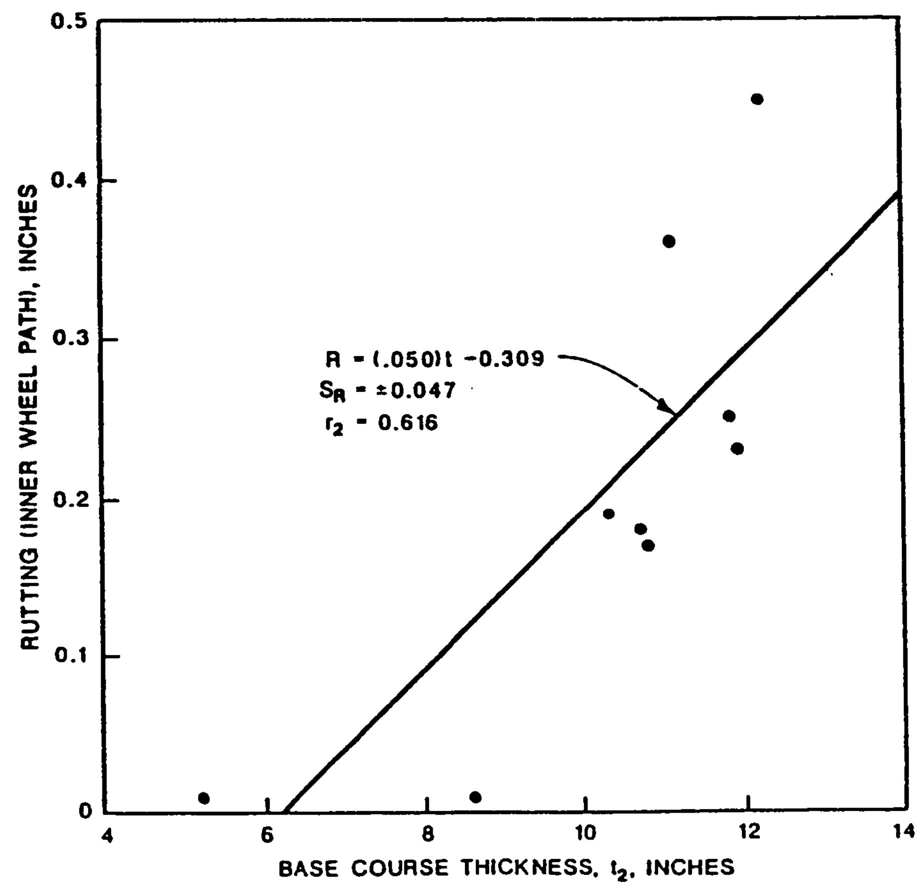

Figure 2.4 **Relationship between rutting and thickness of base course in Michigan.**

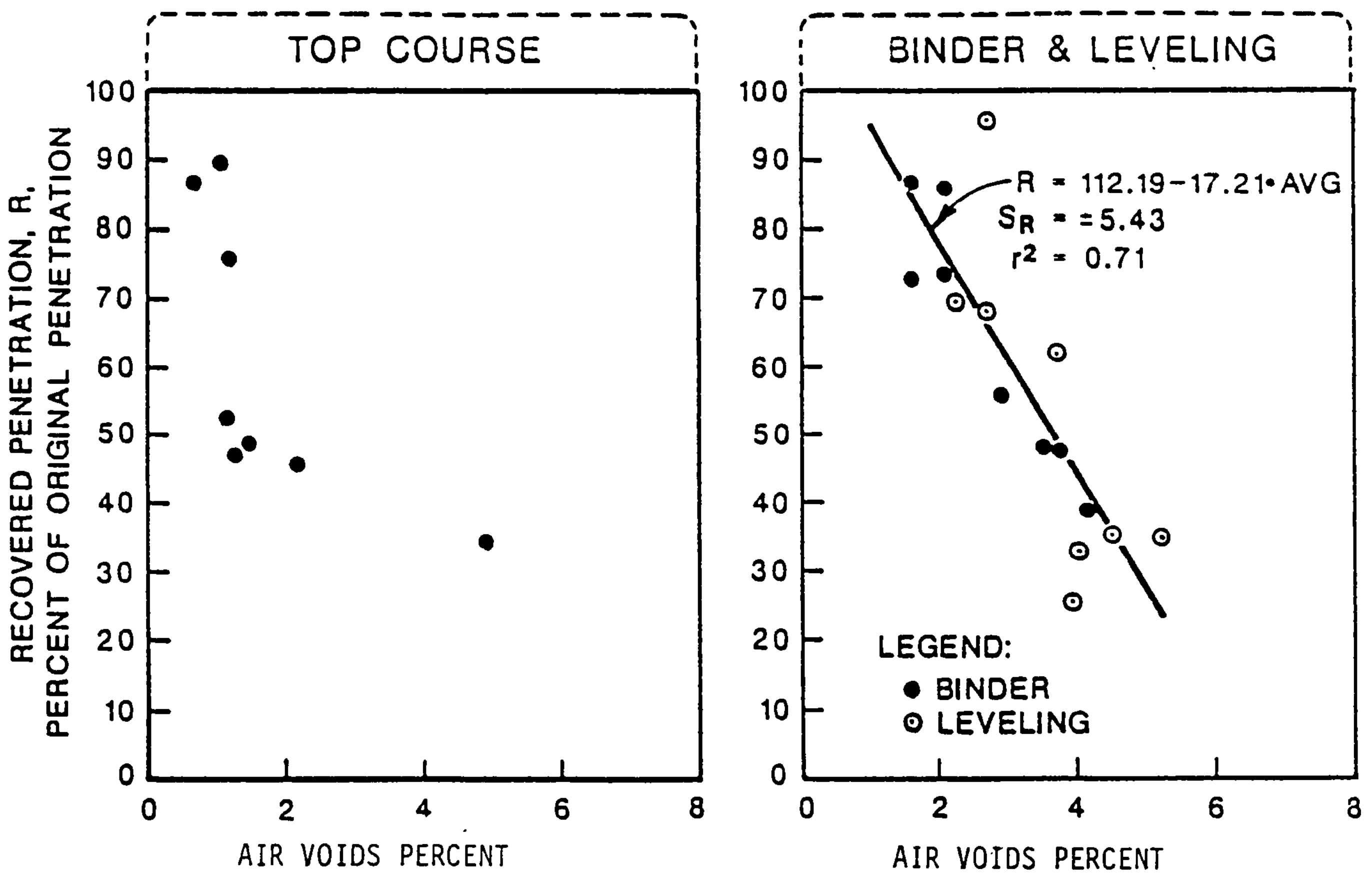

Figure 2.5 **Relationship between asphalt hardening and air voids measured from core samples in Michigan (Defoe, 1988).**

Iowa DOT - Transverse Cracking

Description

The Iowa Department of Transportation (Marks 1984) initiated research to identify methods to reduce the occurrence of transverse cracking and/or to prevent the deterioration of sawed (and sealed) joints to replace the random transverse cracking in Iowa. Eight research sections were incorporated into a paving project on Iowa Highway 64, to study the following three variations in the asphalt concrete pavement:

- To compare low and high temperature susceptible asphalt cement from two different sources
- To vary transverse joint spacing between 40 to 100 feet
- To increase the binder content in the asphalt treated base by one percent

AC-10 grade asphalt from two different sources was used for these test sections:

- Wood River PVN = -0.6 (Pen. @ 77°F = 100, Vis. @ 140°F = 1100 poises)
- Sugar Creek PVN = -1.2 (Pen. @ 77°F = 75, Vis. @ 140°F = 900 poises)

A summary of the eight test sections is presented in Table 2.6. These pavements were constructed in August, 1980. According to Marks (1984) severe[7] cold temperatures prevailed during the winter of 1981-82. Cores were taken at the time of construction and in 1984. Condition surveys, to determine cracking, were conducted annually until 1984.

SECTION	EXPERIMENTAL FEATURE	LENGTH (FT)
1	Sugar Creek AC	1900
2	Sugar Creek AC	2000
3*	Transverse Joints/Wood River AC	2360
4*	Transverse Joints/Wood River AC	2320
5	Increased AC in ATB/Wood River AC	1490
6	Increased AC in ATB/Wood River AC	1980
7	Control (Standard/Wood River AC)	1740
8	Control (Standard/Wood River AC)	2000

* These sections had 15 joints @ 40' spacing, 10 joints @ 60' spacing, 7 joints @ 80' spacing and 6 joints @ 100' spacing.

Table 2.6. Summary of test sections in Iowa (Source: Marks, 1984).

[7] No temperature values nor temperature variations between seasons were reported.

Results

Condition Survey

By 1984, nearly all transverse joints had failed. Failure was defined as de-bonding between the sealant and the face of the saw cut due to thermal contraction stresses. The sealant used was an upgraded rubber asphalt sealant meeting Iowa DOT Standard Specification 4136.02A. Some joints that had been sawed 1/4" wide had widened up to 1" due to thermal movement. Despite these joint failures, there were no transverse cracks between any of the sawed transverse joints until the last survey in 1984. A field review was later conducted in May 1990 and the results are also shown below (Marks, 1990). The average crack intervals for the sections are summarized in Table 2.7.

SECTION	SECTION	AVERAGE CRACK INTERVAL, FT. (1984)	AVERAGE CRACK INTERVAL, FT. (1990)
1,2	Sugar Creek AC	35	30
7,8	Control (Standard Wood River AC)	170	73
3,4,5,6	Wood River AC	528	*106

* Sections 5 & 6 only.

Table 2.7. Transverse crack intervals in 1987 and 1990 in Iowa (Source: Marks, 1934, 1990).

In 1984, the longest distance between transverse joints in the jointed pavement sections was 100 feet, whereas the average crack interval for the standard Wood River AC section was 170 feet. This warrants caution in extrapolation of results from the jointed pavement sections. In the 1990 field review, there were still no cracks between the joints that were sawed at 40, 60, 80 and 100 ft. in sections 3 and 4 (see Table 2.6). However, the joint sealant had failed and there was substantial dipping at the joints which was subsequently filled with a slurry to restore ride quality.

Laboratory Analysis of Asphalts Extracted from Cores

Cores were drilled from all research sections with asphalt mix variables (no cores were taken from the two sections with transverse joints) in 1984. The penetration @ 77°F and viscosity @ 140°F was determined for asphalt cements extracted from various layers. The average values are presented in Table 2.8. Results from the two sections with increased asphalt content in the ATB (Sections 5 and 6) are not discussed, as this report is primarily concerned with binder properties. However, it is worthwhile to note the comparatively large crack spacings for these sections (Table 2.7).

PHYSICAL PROPERTY	SUGAR CREEK AC			WOOD RIVER AC		
	Surface	Binder	AVG	Surface	Binder	AVG
Penetration	38	43	40	56	54	55
Viscosity	3330	2518	2924	2733	3103	2918
PVN	-1.03	-1.05	-1.04	-0.64	-0.58	-0.55

Table 2.8. Summary of tests on asphalt cements extracted from cores taken from Iowa test sections (Source: Marks, 1984).

The only difference between the Wood River and Sugar Creek sections was the asphalt source. In 3.5 years, the average crack interval for the Sugar Creek section was 35' as compared to 170' for the Wood River section. The Sugar Creek asphalt was more temperature susceptible than that used in the Wood River section.

From these results, both the penetration and the PVN of the extracted binder related well to the frequency of transverse cracking. According to Marks (1984), the penetration continued to decrease due to oxidation. In four years, the penetration of Sugar Creek decreased from 75 to 40 and Wood River from 100 to 55. The PVN, on the other hand, remained relatively constant. For this reason, Marks says that it appears that the PVN is a more desirable measure of the potential for transverse cracking.

Conclusions

The author reported the following conclusions:

1. An improved sealant or sealing procedure is needed if transverse joints are to be used in asphalt pavements.

2. The PVN is an effective measure of the temperature susceptibility of asphalt cements.

3. The use of a high temperature susceptible asphalt cement produced more frequent transverse cracking.

South Dakota DOT - Transverse Cracking

Description

Previous research conducted in 1968 by Crawford and Anderson in South Dakota indicated a good correlation between asphalt hardness and tendency of bituminous pavements to develop transverse cracks (see Figures 2.6 and 2.7). The one exception to the above results were pavements in the Black Hills region of South Dakota. All types of asphalts performed well in this region, i.e., pavements were virtually crack free

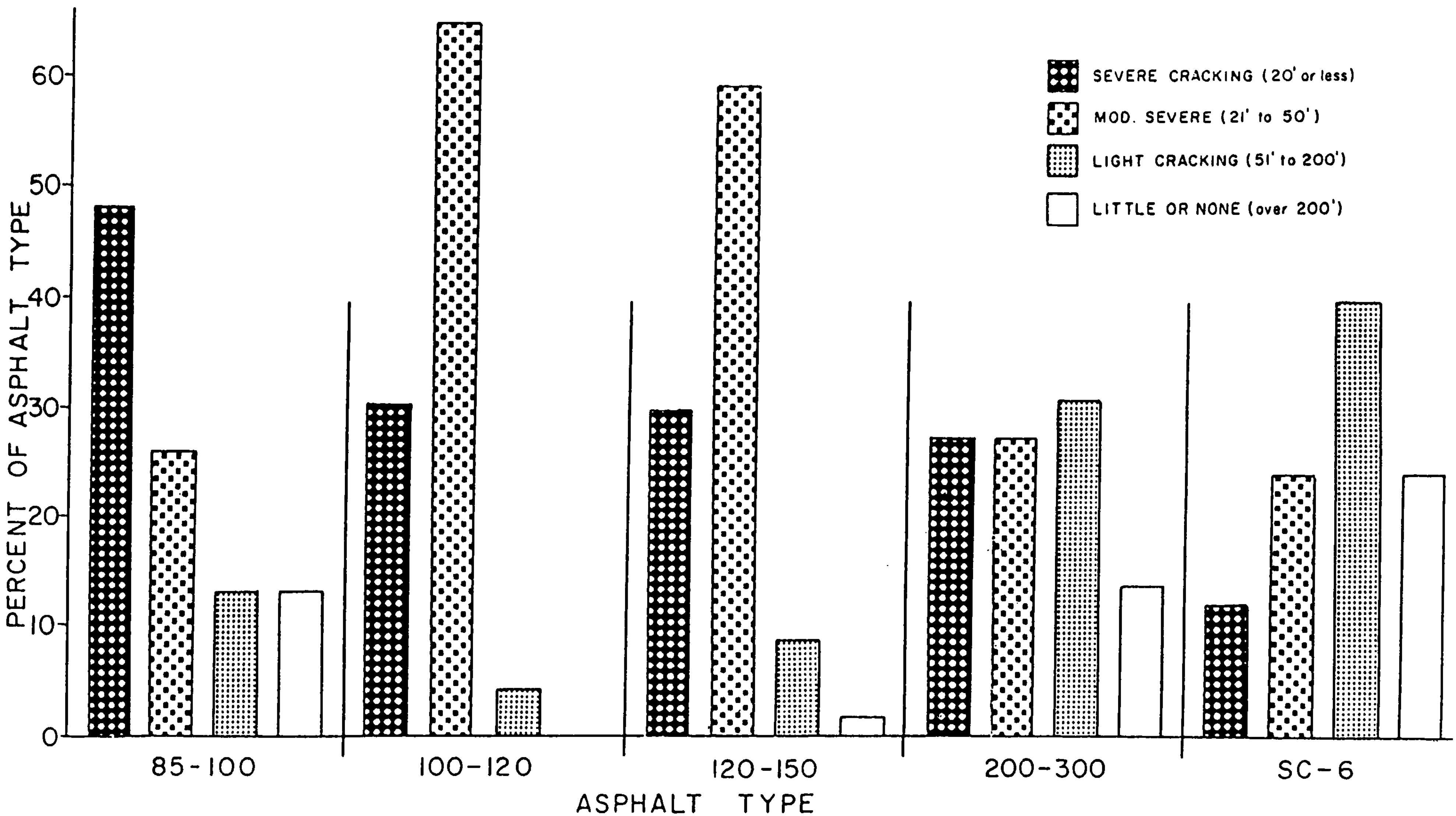

Figure 2.6 Pavement cracking as related to asphalt hardness--Comparison of asphalt types for cracking frequency (Crawford and Anderson, 1968).

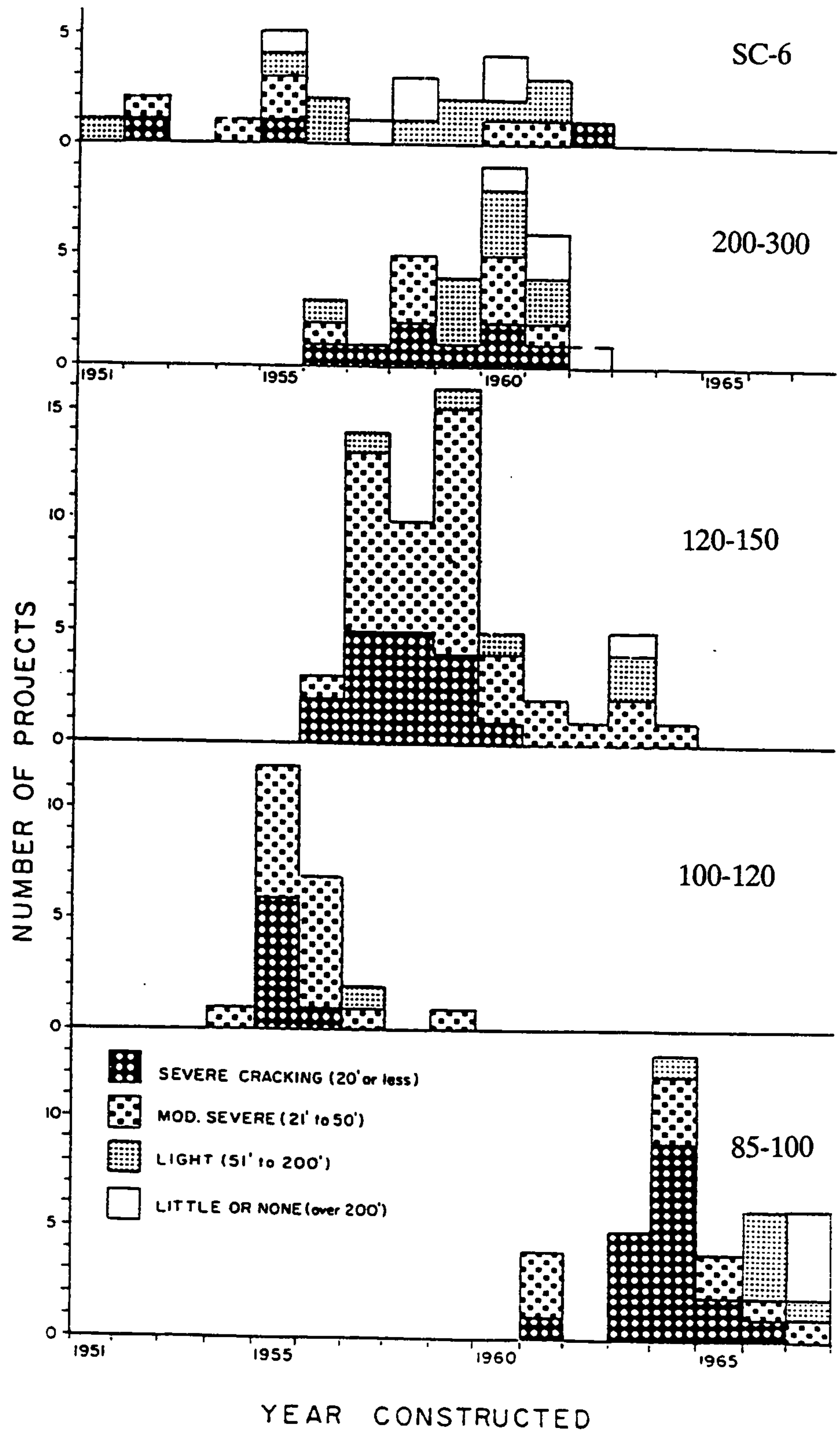

Figure 2.7 **Pavement cracking as related to asphalt hardness (Crawford and Anderson, 1968).**

irrespective of the type of asphalt. The reasons for this exclusive behavior of asphalts in this region included:

- The climate in the Black Hills region is relatively mild as compared to the rest of South Dakota.

- The underlying soils are generally of better quality, i.e., sand or gravelly instead of expansive clays or shale.

- The aggregate generally consists of quarried limestone or crushed limestone gravel.

According to the authors (Crawford & Anderson, 1968), the predominant use of quarried limestone may be the primary reason for the near absence of cracking on these pavements. A subsequent study (Crawford & Anderson, 1976) was carried out to verify the above conclusions.

Four 2800-foot sections were constructed on State Highway 44 near Scenic, South Dakota. The design was constant throughout the test sections, i.e., 6 inches of sub-base, 3 inches of base course and two inches of bituminous surfacing. The four sections were constructed with:

1. Local crushed gravel aggregate with 85-100 pen. asphalt;

2. Black Hills limestone with 85-100 pen. asphalt;

3. Local crushed gravel aggregate with 200-300 pen. asphalt; and

4. Black Hills limestone with 200-300 pen. asphalt.

The temperatures in this area normally reach annual extremes of below -20°F and above 100°F. Rainfall averages 14 inches annually.

Condition surveys were done annually and the cracks mapped to provide information as to the progression of the cracking. Dynaflect deflection measurements were also taken at the test sections. Cores were taken after four years and the following tests were performed:

- Viscosity
- Microviscosity
- Penetration
- Percent Asphalt

Results

Transverse cracks were first noticed after two years. These cracks were mostly in the harder asphalts (85-100 pen) with local aggregate sections. The softer asphalts (200-300 pen) with limestone showed the best resistance to cracking. Deflections were found to be in the range of 2 to 3 mils. These values, though marginal, were acceptable for 200 ADT and 20 commercial vehicles per day.The cores were tested to see if limestone aggregate retarded the aging and hardening of asphalt (Crawford & Anderson 1976). The tests did not indicate any significant difference between the asphalts combined with limestone showing high cracking resistance and those combined with local gravel aggregate and developing relatively severe cracking. The results are tabulated in Table 2.9.

DESCRIPTION	% ASPH.	PEN.	0.05*	0.001**	VIS.**
Local Agg., 85-100	7.0	33.4	8.15	13.4	532
Local Agg., 85-100	7.0	34.0	7.90	16.0	550
L. S. Agg., 85-100	5.4	34.5	7.54	10	551
L. S. Agg., 85-100	5.7	34.5	7.35	11.8	539
Local Agg., 200-300	6.1	70.2	1.65	2.08	344
Local Agg., 200-300	6.5	102.0	.740	1.10	277
L. S. Agg., 200-300	5.9	73.0	1.50	1.85	315
L. S. Agg., 200-300	6.1	84.0	1.11	1.65	305

* It is assumed that these are readings from micro-viscosimeter at 0.05 and 0.001 rate of strain; hence, they are reported at .05 sec^{-1} and 77°F, and at .001 sec^{-1} and 77°F. The report (Crawford & Anderson, 1976) does not provide this information.

** No information regarding test temperature, method, or units is provided.

Table 2.9. Theological properties measured for Highway 44 cores in South Dakota (Excerpt: Crawford and Anderson, 1976).

Conclusions

1. The use of softer asphalts in the 200-300 penetration range retarded the formation of transverse cracking when compared with identical sections utilizing 85-100 penetration asphalt.

2. The use of 100% quarried limestone aggregate produced beneficial results when combined with the 85-100 pen. asphalt, roughly comparable to those obtained with the 200-300 pen. asphalt combined with local crushed gravel aggregate.

3. A combination of 200-300 AC and 100% quarried limestone aggregate produced a pavement with a "good"[8] ability to resist transverse cracking.

4. Tests conducted during this study did not reveal the reason for improved pavement cracking resistance of limestone aggregates.

[8] It is not possible to quantify "good", as the authors (Crawford & Anderson 1976) do not provide any crack interval information for these test sections.

University of Alberta, Canada - Low-Temperature Transverse Cracking

Description

Palsat (1986) investigated the low temperature cracking behavior of 77 pavement sections (55 full depth and 22 with granular base pavement sections) in Alberta. Original asphalt information was collected for these pavements, which included:

- Supplier
- Refinery Location
- Grade and Type
- Penetration @ 25°C, 100g, 5s
- Penetration @ 4°C, 200g, 60s
- Absolute viscosity @ 60°C
- Kinematic viscosity @ 135°C
- PVN (calculated using penetration @ 25°C and absolute viscosity @ 60°C)
- PVN (calculated using penetration @ 25°C and kinematic viscosity @ 135°C)
- Thin Film Oven Test (TFOT) loss
- Penetration @ 25°C, 100g, 5s on residue after TFOT
- Penetration @ 4°C, 200g, 60s on residue after TFOT
- Absolute viscosity @ 60°C on residue after TFOT
- PVN of residue after TFOT

Condition surveys were conducted to estimate the number of transverse cracks per kilometer over the length of the selected pavement sections. Cores were taken at randomly selected locations along the pavement sections. The laboratory testing program included an evaluation of the asphalt pavement cores and of the subgrade soils which included the following:

Asphalt Pavement Cores

- Visual description
- Measurement of lift thicknesses
- Density measurement of each lift
- Asphalt extraction of top lift and Abson recovery
- Sieve analysis of extracted aggregate

Tests on Recovered Asphalt

- Absolute viscosity @ 60°C
- Kinematic viscosity @ 135°C
- Penetration @ 25°C, 100g, 5s
- Penetration @ 4°C, 100g, 5s
- Penetration @ 4°C, 200g, 60s

Subgrade Soils

- Visual description
- Sieve analysis
- Classification (Unified System)

Overall, the author included 55 full depth pavement sections totaling 642 km and 22 sections of pavement constructed over granular base totaling 349 km. A summary of the data included by Palsat in his analysis is presented in Tables A.1 through A.4 in Appendix A.

Results

Full Depth Pavements

Using regression techniques, a mathematical model was developed associating pavement characteristics with the observed low temperature transverse cracking of 55 full-depth pavement sections. This model has a R^2 of 0.64:

$$\begin{aligned}\text{Frequency(cracks/km)} = {} & 49.40 + 3.09 \text{ (pavement age in years)} \\ & + 0.36 \text{ (original asphalt stiffness in kg/cm}^2\text{)} \\ & - 5.60 \text{ (pavement thickness in mm)}^{0.5}\end{aligned}$$

Granular Base Pavements

The best model developed for the granular base pavements has a R^2 of 0.39:

$$\text{Frequency(cracks/km)} = -0.47 + 0.37\text{(original asphalt stiffness in kg/cm}_2\text{)}$$

The granular model does not explain as much of the total variation as the full depth model. However, Palsat (1986) indicates that it is significant to note that the stiffness of the original asphalt cement had the greatest correlation with cracking frequency. The independent variables, asphalt pavement thickness and asphalt pavement age, show very little variation in values due to the way these projects were selected. As a result, both these variables did not enter this particular model.

All Pavements

When all the sections (the full depth pavements and the granular base pavements) were aggregated, a model similar to that for the full depth pavements was developed. This model (R^2 = 0.52) is:

$$\begin{aligned}\text{Frequency(cracks/km)} = {} & 6.59 + 3.64 \text{ (Pavement age in years)} \\ & + 0.37 \text{ (Original asphalt stiffness in kg/cm}^2\text{)} \\ & - 2.91 \text{ (Pavement thickness in mm)}^{0.5}\end{aligned}$$

Full Depth Pavements with Thickness ≥ 150 mm

In order to reduce the effects of sections with relatively thin[9] pavement thickness on the cracking model, 32 full depth sections with pavement thickness greater than or equal to 150 mm were selected. In developing an analytical model for these pavements, nine variables relating to mix and pavement characteristics at the time of construction were included, in addition to the 16 independent variables already included in the analysis of all the full depth pavements. The model developed has a R^2 of 0.60:

$$\begin{aligned} \text{Frequency(cracks/km)} = 153.28 &+ 2.65 \text{ (Pavement age in years)} \\ &+ 0.40 \text{ (Original asphalt stiffness in kg/cm}^2\text{)} \\ &- 2.37 \text{ (percent compaction}^{10}\text{)} \end{aligned}$$

Palsat (1986) points out the fact that the negative sign for the regression coefficient for percent compaction indicates an inverse relationship between cracking frequency and percent compaction. This conflicts with van der Poel's (1954) relationship between asphalt and pavement stiffness, i.e., for a given asphalt content, an increase in pavement density results in an increase in pavement stiffness which should result in an increase in the potential for low temperature cracking.

Based upon his review of the data collected for the 77 sections used to develop the models and on particular case studies, the author suggests that:

- A critical original asphalt stiffness[11] of 2.9 x 10^6 Pa (30 kg/cm^2) be used to separate pavements with "acceptable" from "non-acceptable" transverse cracking;

- An increase in the original pavement thickness is needed to lessen the frequency of transverse cracks.

It should be noted that field sampling and laboratory testing identified that subgrade effects were a large factor in influencing differences in observed cracking. These effects were not within the scope of this review.

During the review it was decided that a further analysis of this data was warranted. The cracking data was categorized; i.e., pavement sections were further divided into cracked and uncracked sections. The resulting matrix is shown in Table 2.10. The asphalt properties used in the analysis were:

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[9] Palsat (1986) does not indicate why he chose a thickness of 150 mm for separating the full depth pavements.

[10] Pavement density/field Marshall density x100.

[11] Using original asphalt characteristics and the temperature at a pavement depth of 50 mm.

- Penetration, 77°F, dmm (P77)
- Viscosity, 140°F, poises (V140)
- Penetration (after TFOT), 77°F, dmm (P77T)
- Viscosity (after TFOT), 140°F, poises (V140T)
- Original calculated asphalt stiffness (calculated using McLeod's nomograph), kg/cm^2

Table 2.10 also shows the mean asphalt properties for the cracked versus uncracked sections. A student's t-test was used to determine if the means for the cracked and uncracked sections belong to separate populations. Looking at Table 2.10, a 'Y' in the significance row for a particular asphalt property indicates that for an $\alpha = 0.1$ (90% confidence level), there is a significant difference in the means of the cracked versus uncracked sections. A Bartlett's test for homogeneity of group variances was also done, but, due to the small sample sizes ($N \leq 30$) and unequal group sizes, the results were not considered reliable. As a result of these elementary tests, the following conclusion can be drawn:

a) For full depth asphalt sections, only one original asphalt property, namely the asphalt stiffness, appears to be useful in predicting asphalt cracking. This result is in agreement with Palsat's (1986) equation for predicting cracking frequency; and

b) For the granular base sections analyzed, penetration at 77°F, viscosity at 140°F after TFOT, and the original asphalt stiffness appear to be good predictors of cracking performance.

These conclusions are based on original asphalt properties and do not lend themselves to a general comparison with the data presented in some of the other state reports. Thus, for this review, these results remain isolated to this particular study.

Conclusions

The following conclusions were drawn by the author (Palsat, 1986) based upon the results of this investigation:

1. The major factors found to influence the low temperature transverse cracking behavior of the pavement sections were:

    - pavement thickness
    - pavement age
    - original asphalt stiffness using McLeod's method and based on site-specific temperature conditions
    - subgrade soil characteristics

2. A critical asphalt stiffness of 2.9×10^6 Pa ($30\ kg/cm^2$) is suggested to control transverse cracking to an acceptable level. This was obtained by using McLeod's method and based on original asphalt characteristics and site-specific temperature

SECTION TYPE	PERFORMANCE	STATISTICS	ASPHALT PROPERTIES				
			P77	V140	P77T	V140T	CAS
GRANULAR BASE SECTION	CRACKED (N=9)	MEAN	237	541	123	1377	54
		STD. DEV	65	218	33	611	18
		VARIANCE	4178	47531	1098	373054	337
	UNCRACKED (N=13)	MEAN	267	421	143	1023	22
		STD. DEV	32	112	18	290	17
		VARIANCE	1052	1247	321	84129	292
SIGNIFICANCE			NO	NO	YES	YES	YES
FULL DEPTH ASPHALT SECTION	CRACKED (N=30)	MEAN	249	428	135	1003	39
		STD. DEV	2508	25732	851	186789	1923
		VARIANCE	50	160	29	432	44
	UNCRACKED (N=25)	MEAN	265	416	139	1005	20
		STD. DEV	44	143	25	345	15
		VARIANCE	1945	20511	642	119661	215
SIGNIFICANCE			NO	NO	YES	NO	YES

P77= Penetration, 77° F, dmm
V140= Viscosity, 140°F, poises
P77T= Penetration after TFOT, 77° F, dmm
V140T= Viscosity after TFOT, 140°F, poises
CAS= Calculated original asphalt stiffness, kg/cm^2
N= Number of observations
STD. DEV= Standard deviation

Table 2.10. Mean asphalt properties for cracked and uncracked section in Alberta Source: ARE analysis of Alberta Study, Palsat, 1986).

(at a pavement depth of 50 mm) conditions, can be used to separate "acceptable" from "non-acceptable" transverse cracking behavior.

South Carolina - Stripping

Description

This study (Busching et al., 1986) focuses on the causes of stripping in asphalt concrete pavements in South Carolina, especially open graded friction courses. Field data collection included extensive coring through a program of random sampling of 500 miles of pavement. A wide variety of materials, soil types, and traffic conditions were covered. Information recorded at coring locations included:

- Cracking
- Flushing
- Rutting
- Status of drainage
- Presence of paved shoulders
- Identification of cut or fill section
- In-place pavement density (using a nuclear backscatter gauge)

The laboratory tests and data collected from these cores included:

- Maximum (Rice) specific gravity (ASTM D 2041)
- Bulk specific gravity
- Indirect tensile strengths (using Tunnicliff and Root saturation procedure) (Chip seals, surface treatments, and other moisture seals were not tested)
- Visual rating of stripping (immediately after tensile strength tests)
- Voids in mineral aggregate
- Air voids
- Asphalt content

The authors (Busching et al., 1986) only included portions of these results in their report. Three experiments were also set up to investigate:

- Test methods used to evaluate effectiveness of anti-strip additives
- Moisture susceptibility of different aggregate sources
- Effects of asphalt cement source on moisture susceptibility

Of the above three experiments, only the results from the last experiment will be presented in this summary. For this particular experiment, asphalt cements of AC-20 grade were acquired from four different sources. Laboratory testing consisted of indirect tensile strength tests of dry and moisture conditioned specimens, to determine the retained strength of the specimen. Refer to Figure A.2 in Appendix A.

Results

The authors (Busching et al., 1986) found that stripping of pavement layers was related to pavement age. Specimen from the extensive coring program were grouped into five-year age intervals. Within each interval the percentage of pavement layers that were severely stripped was computed (Figure 2.8). The highest percentage (13.2%) of severely stripped pavement specimen fall in the six to ten-year interval. The authors believe that moisture related damage became more wide-spread as a result of using the open-graded friction course. The practice of using open-graded friction courses originated approximately 10 years before this study.

There was no correlation between traffic group (truck count) and stripping, but the type of section (level, cut or fill) affected the extent of stripping. Overall, 10.2% of the layers from level sections were stripped, whereas 7.6% and 4.6% of layers from cut and fill sections, respectively, were stripped. Sand asphalt layers and base courses did not exhibit much stripping regardless of the section. HPMSC (hot plant mix seal coat or "pop-corn" mixes) layers from level sections had a stripping frequency of 10%, while the same mixes exhibited 1.1% and 2.9% stripping frequencies in cut and fill sections, respectively.

Retained indirect tensile strength ratios from pavement cores obtained from intensive sampling were higher for mid-lane specimens (87 percent) than for wheelpath specimens (73 percent). The retained tensile strength ratio (TSR) was obtained by dividing the tensile strength following vacuum saturation to 55-80 percent saturation and 24 hour immersion in distilled water at 140°F by the tensile strength of unconditioned specimens. Distress occurs more frequently in the wheel path. Consequently, the ratio of average TSR value from wheel path specimens divided by the average TSR values from mid-lane specimens could be used as one measure of pavement deterioration (Busching et al., 1986). This ratio is 83.5% for this report. Indirect tensile strengths ranged from 84 to 26 psi.

Many of the core specimens observed after strength testing contained cracks with an accumulation of fine granular material in the crack. Those granular materials had, in some instances, accumulated from upward movement of granular base materials into the overlying mixture. This results in a loss of support and is similar to the pumping action that occurs in rigid pavements.

Open-graded friction courses and sand asphalt mixtures typically have high air-void contents. High air-void contents can be a source of stripping through water retention, especially when the mixture is located immediately below open-graded friction courses. The mixtures that had been used regularly under open-graded friction courses (HPMSC) had 7.04% air voids, higher than the 6% average for the rest of the specimens.

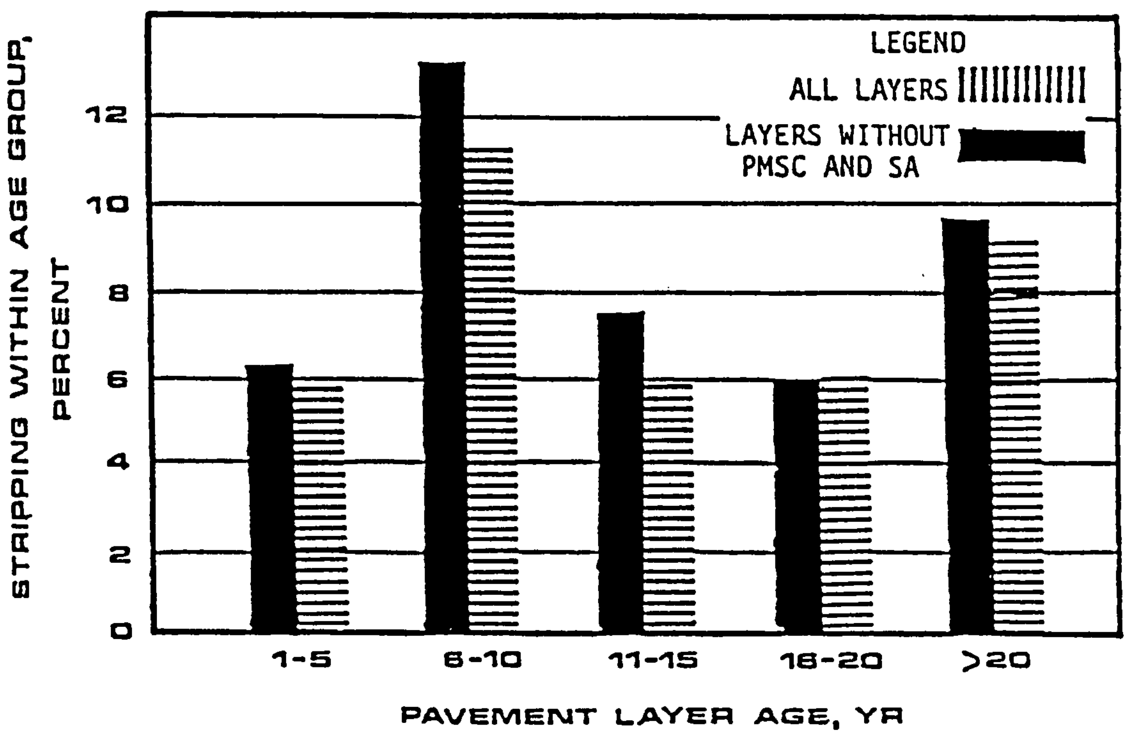

Figure 2.8 Relationship of pavement layer age and stripping (PMSC = Plant-mixed seal course, SA = Sand asphalt) (Busching et al., 1986).

The interasphalt comparisons for moisture susceptibility did not yield any significant results regarding moisture susceptibility. Predicted field lives of some of the mixtures tested in the laboratory were also calculated using the ACMODAS (Asphalt Concrete Moisture Damage Assessment System) computer program. Inputs to ACMODAS include:

- All-dry Design Life
- Percent Allowable Reduction of Field All-dry Design Life
- Field Dry Stage Time
- Test Temperature
- ITS (Dry)-Indirect Tensile Strength
- ITS (Wet)-Indirect Tensile Strength
- Resilient Modulus Data (if any)

A typical output from ACMODAS for various levels of tensile strength and retained tensile strength ratios is shown in Figure 2.9. These figures are used by entering the graph at the dry indirect tensile strength results and reading up to the retained strength.

As an additional note, Davis (1986) indicated that some of the stripping problems observed could have been due to the use of higher truck tire pressures (130-140 psi), whereas the mix design process was originally developed with tire pressures at 70 psi. He believes that bearing capacity failures occur on these pavements, causing the pavement to "open up", thereby increasing air voids and allowing the entry of water into the pavement and shear failure occurring.

Conclusions

The authors (Busching et al., 1986) reported the following conclusions:

1. The frequency of stripping is greater in mixtures located immediately under hot plant-mixed seal courses.

2. Open graded friction courses are highly susceptible to moisture damage and hence stripping.

3. High air-void contents measured for some in-service pavement layers indicated that potential for moisture intrusion in service was greater than that measured by laboratory procedures when these pavements were constructed.

4. The indirect tensile strength test and associated moisture conditioning using methods recommended by Tunnicliff and Root (1984) is useful for assessing moisture susceptibility of mixtures typical of those used in South Carolina. When using the procedure, both the indirect tensile strength (ITS) and the tensile

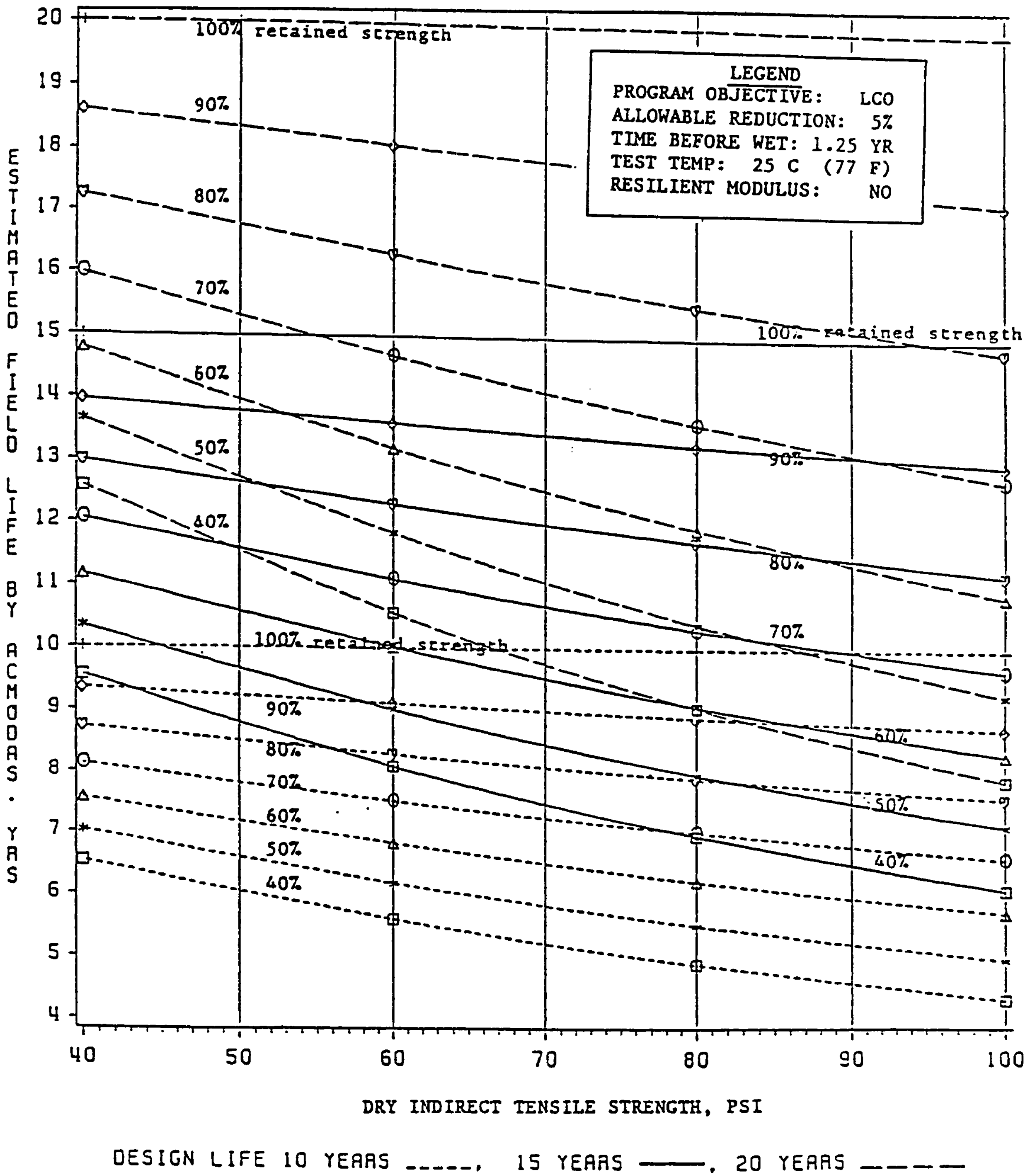

Figure 2.9 Predictions of field dry-wet life using the ACMODAS program for 10, 15, and 20 year design lives for a traffic (regional) factor of 1.10 (very heavy traffic) (Busching et al., 1986).

strength ratio (TSR) should be considered. For example, for mixtures made with gravels and local sands, high TSR values were obtained, but the low magnitude of the ITS value indicates that the mixtures may not perform well in service despite the high TSR values.

5. A procedure is presented to incorporate moisture susceptibility testing into the Marshall mix design method (Figure 2.10).

Colorado - Rutting and Cracking

Description

This study (O'Connor, 1979) was initiated to determine the cause or causes of low quality performance of asphalt pavements in Colorado. Twenty-one projects were evaluated, some of which were good performers while others were poor. Field investigations were conducted to evaluate the surface condition with regard to wheel rutting, shoving, bleeding, ravelling and cracking.

The following information was collected for the pavements surveyed:

- Asphalt cement source, grade, additive, penetration, viscosity at 140°F and at 275°F
- Aggregate filler type and amount
- Average percentage #200 sieve
- Job mix formula asphalt content
- Design and production voids
- Design and production stability
- Plant type
- Design and production resilient modulus
- Date placed (month and year)
- Accumulative 18^k EDLA

The field cores were analyzed for:

- Hveem stability
- Cohesiometer value
- Resilient modulus
- Voids in the specimen
- Asphalt content
- Recovered asphalt cement:
- Penetration
- Viscosity
- Asphalt composition analysis
- Extracted aggregate gradation

A summary of the data collected is presented in Table 2.11.

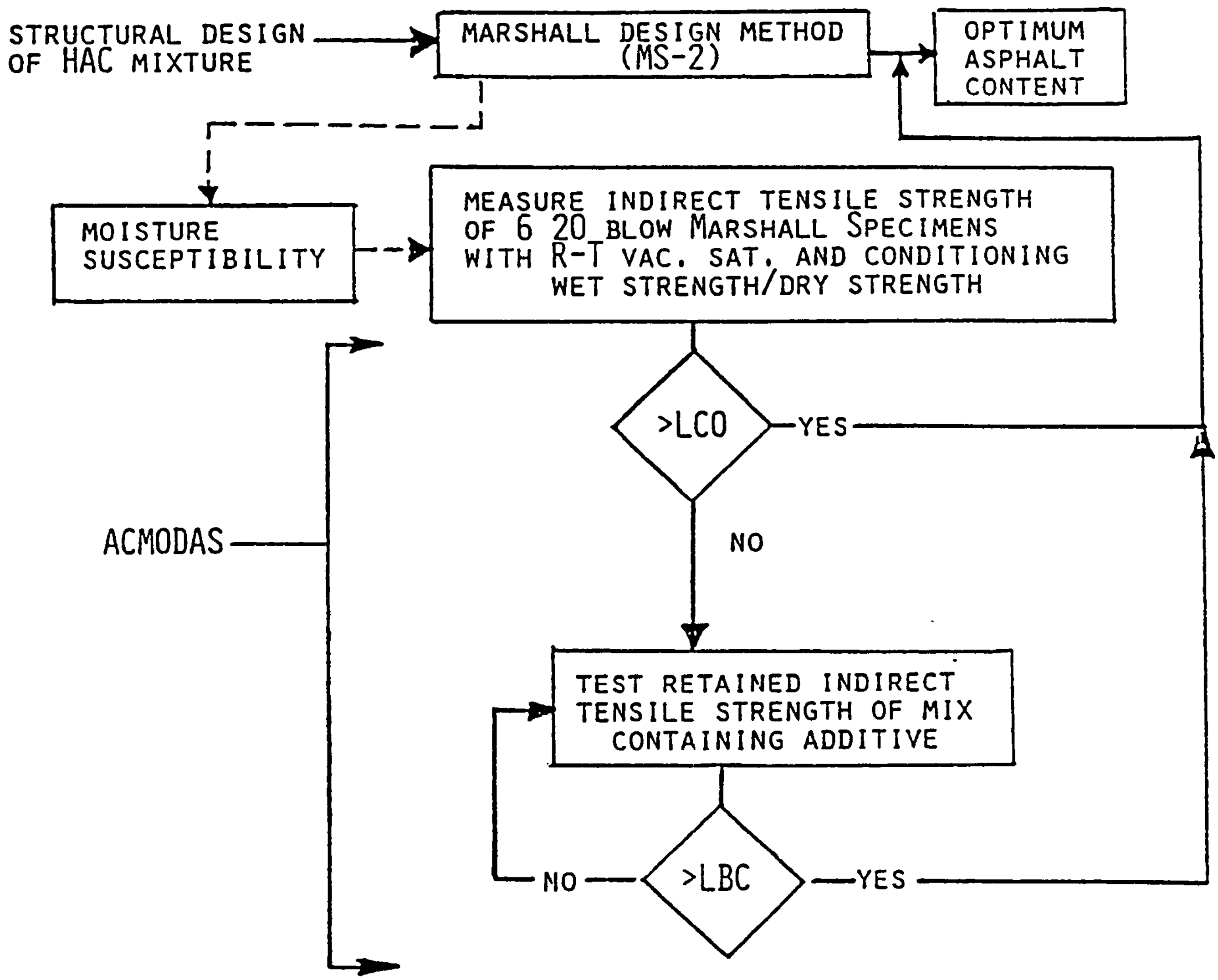

Figure 2.10 Recommended SCDHPT practice for testing moisture susceptibility (Busching et al., 1986).

STUDY NO. 1529

PROJECT	TEAM EVALUATION: Wheel Rutting	Bleeding	Raveling	Cracking	CONSTRUCTION DATA: Asphalt Source	Asphalt Grade	Asphalt Additive	Filler Type & Amount	Average %-200	J.M.F. % A.C.	Ave. Job % A.C.	Design Voids	Production Voids	Design Stability	Production Stability
FC 014-1(8) East of Briggsdale	MOD.	SLT.	SLT.	SLT.	SIN	AC-10		4% F.A.	8.8	6.6	6.4	2.78	2.85	36/97	30/96
RS 0014(6) West of Sterling			SLT.	MOD.	CNX	AC-10	SIL	1% H.L.	7.4	6.9	6.7	6.15	5.56	31/92	30/90
RF 006-9(4) Sterling-East				MOD.	HUS	AC-10	SIL	1% H.L.	7.3	7.0	7.0	5.36	6.18	31/92	28/89
C 13-0138-02 Proctor - S.W.	SLT.	SLT.		MOD.	CNX	AC-10		1% H.L.	8.7	7.7	7.7	8.20	4.42	28/88	28/88
C 13-0138-01 Crook - Proctor				MOD.	HUS	AC-10	SIL	1% H.L.	9.0	7.0	7.1	6.79	3.69	37/93	38/97
FC 006-9(8) Fleming - E & W	SLT.	SLT.			CON	AC-10	SIL	6% F.A.	9.3	7.1	7.0	4.97	3.25	29/89	37/87
RF 006-9(5) Haxtun - Fleming	SEV.	SEV.		MOD.	CON	AC-10	SIL	1% H.L.	9.3	7.0	6.9	6.00	4.27	28/87	27/88
RS 0176(1) Holyoke-N.E.	MOD.				HUS	AC-10	SIL	7% L.D.	7.3	6.5	6.6	3.80	4.81	29/87	30/89
FC 034-3(4) West of Wray	SLT.		SLT.	SLT.	HUS	AC-10	SIL	3% L.D	10.5	5.8	5.8	2.74	4.20	34/100	31/95
FC 034-3(3) West of Wray	MOD.	SLT.	SLT.	SLT.	HUS	AC-10	SIL	2% L.D	9.7	5.8	5.9	3.89	2.53	34/92	29/81
FC 034-3(5) Otis - Yuma				SLT.	CON	AC-10	SIL	7% L.D	10.7	5.8	5.9	4.15	2.52	27/88	30/93
RS 0033(9) FC 036-2(3) Cope - South Cope - Joes		SEV.		SLT.	CON	AC-10	SIL	5% L.D	8.3	7.5	7.5	4.56	2.89	25/85	26/87
TQRS 0003(4) Akron-South					CON	AC-10	SIL	1% H.L	7.1	6.7	6.8	4.68	3.10	28/87	33/93
RS 0003(3) South of Akron					HUS	AC-10	SIL	1% H.L	6.5	6.2	6.5	4.23	2.54	27/89	28/90
I 76-1(44) Wiggins - East Sq. Fat. Rd.	SLT.			SLT.	TEX	AC-20	SIL	7% L.D	5.9	7.2	7.2	5.84	6.48	24/83	23/82
C 27-0036 01 Cope West				SLT.	HUS	120-150		1% H.L.	6.5	6.8	6.7	7.30	8.00	23/83	36/87
C 22-0160-01 Las Animas - East			MOD.		DIS	AC-20		1% H.L	9.5	6.8	6.7	8.5	8.0	39/95	33/92
F 287-1(8) Springfield-North	SEV.	MOD.		SLT.	DIS	AC-10		1% H.L.	10.8	6.8/6.5	6.7	9.2	8.5	31/90	28/90
FC 287-2(3) Prowers-Kiowa Co. Line			SLT.		DIS	AC-20		1% H.L	7.6	6.5	6.5	9.7	7.30	27/88	31/91
RF 287-2(3) Eads-South			SLT.	SLT.	DIS	AC-20		1% H.L	9.4	6.3	6.3	10.1	9.7	40/97	33/35

PROJECT	CONSTRUCTION DATA (cont.): Plant Type	Design M_R	Production M_R	Time of Year	18k E.O.L.A.	Penetration	Viscosity (140° F)	Viscosity (275° F)	CORE DATA: %-200	% A.C.	Voids	M_R	Stab./R_T	Penetration	Viscosity (275° F)	Viscosity (140° F)
FC 014-1(8) East of Briggsdale	DD	594.2		Oct Nov 1977		87	975	281	9.3	7.0	2.77	171.2	21/80	86	318	1436
RS 0014(6) West of Sterling	DD			May 1976		90	900	260	7.8	6.8	3.61	190.4	23/82	81	261	1258
RF 006-9(4) Sterling-East	CONV			Aug 1976		117	900	275	8.8	7.1	3.52	324.6	22/85	74	298	1638
C 13-0138-02 Proctor - S.W.	DD			Nov 1978		36	980	271	13.1	7.0	3.62	377.6	25/90	55	382	2657
C 13-0138-01 Crook - Proctor	CONV			Oct 1975		105	800	242	9.5	8.3	3.23	188.4	17/77	74	321	1678
FC 006-9(8) Fleming - E & W	DD	213.6		Sept 1977		94	1050	292	6.0	7.1	2.54	139.5	30/81	72	398	2167
RF 006-9(5) Haxtun - Fleming	CONV			May 1976		108	1039	280	6.1 6.1	7.5 7.3	1.57 2.56	74.9 138.0	17/77 28/92	84 76	342 378	1652 2007
RS 0176(1) Holyoke-N.E.	CONV	263.3	245.0	Apr 1978		88	1015	272	7.7	6.5	6.16	153.8	18/76	58	373	2293
FC 034-3(4) West of Wray	DD	450.5	453.6	July 1978		92	904	237	10.3	5.7	6.72	315.1	19/81	61	388	2703
FC 034-3(3) West of Wray	CONV	310.3		Dec 1977		87	1119	281	10.0	6.4	1.93	384.5	21/82	65	384	2323
FC 034-3(5) Otis - Yuma	DD	207.4	212.7	May 1978		102	1030	296	14.7	6.0	3.63	199.4	9/59	64	373	1818
RS 0033(9) FC 036-2(3) Cope - South Cope - Joes	CONV			June 1977	...	96	989	291	8.2 7.2	7.4 7.4	3.36 2.73	66.1 237.3	19/41 23/83	82 49	367 440	1814 3019
TQRS 0003(4) Akron-South	DD	217.2		June July 1977		96	998	290	9.2	6.9	4.02	137.5	21/62	90	304	1355
RS 0003(3) South of Akron	CONV			July 1976		50	950	245	6.3	6.2	3.81	310.0	23/65	50	410	4065
I 76-1(44) Wiggins - East Sq. Fat. Rd.	DD			May 1977		83	1983	391	6.6	7.1	4.91	259.4	22/82	39	548	5062
C 27-0036 01 Cope West	CONV			Oct 1971		125			7.4	6.9	3.65	165.8	16/76	66	339	2107
C 22-0160-01 Las Animas - East		373	320	Sept 1976												
F 287-1(8) Springfield-North				May 1974												
FC 287-2(3) Prowers-Kiowa Co. Line		263	445	Oct 1978												
RF 287-2(3) Eads-South				Oct 1975												

DISTRICT TWO PROJECTS

Legend for Performance Data:
SLT = Slight
MOD = Moderate
SEV = Severe
blank = None

Table 2.11. Performance and laboratory data collected for Colorado study (O'Connor, 1979).

Results

From the cores taken inside and outside the wheel paths, the author (O'Connor, 1979) hoped to determine the additional consolidation and changes in voids and stability as a result of traffic. Figure 2.11 graphically depicts the change in voids inside and outside of the wheel paths. Of the six projects with less than 3% voids in the wheel paths, four were performing poorly. On the other hand, of the twelve projects with over 3% voids in the wheel paths, only one was performing poorly. The average values for design, production and core voids are shown in Figure 2.12. Based on these results, the actual void content in the wheel paths can be expected to be approximately one percent lower than the design voids. The points at which bleeding was encountered on the study projects is also shown in Figure 2.12, and occurs below 2.75% voids.The results of compositional analyses performed on the asphalts extracted are shown in Figure 2.13. The section containing asphalt with approximately 19% saturates, 19% asphaltenes, 22% naphtha-aromatics and 40% polar aromatics exhibited the best performance. Sections with the poorest performance tended to have smaller portions of saturates and asphaltenes, and more of naphtha-aromatics.

An extensive multiple regression correlation analysis was performed on about 40 various variables:

Asphalt source
Asphalt additive
Filler type
Plant type
Asphalt type
Time of year paved
Average percent of #200
Job mix formula percent of asphalt cement
Average job percent asphalt cement
Percent asphalt cement (core)
Percent of #200
Design voids
Production voids
Core voids
Design Hveem
Production Hveem
Core Hveem
Core Hveem
Design R_t
Production R_t
Core R_t
Design resilient modulus
Production resilient modulus
Core resilient modulus
18^k EDLA
Production viscosity (140)
Core viscosity (140)
Production viscosity (275)
Core viscosity (275)
Production penetration
Core Penetration
Transverse cracks
Longitudinal cracks
Alligator cracks
Shrinkage cracks
Wheel rutting
Corrugations
Raveling
Shoving
Potholes
Excess asphalt (bleeding)

The two most important performance variables measured for this study, i.e., bleeding and rutting, did not correlate on a one-to-one basis with any of the other variables,

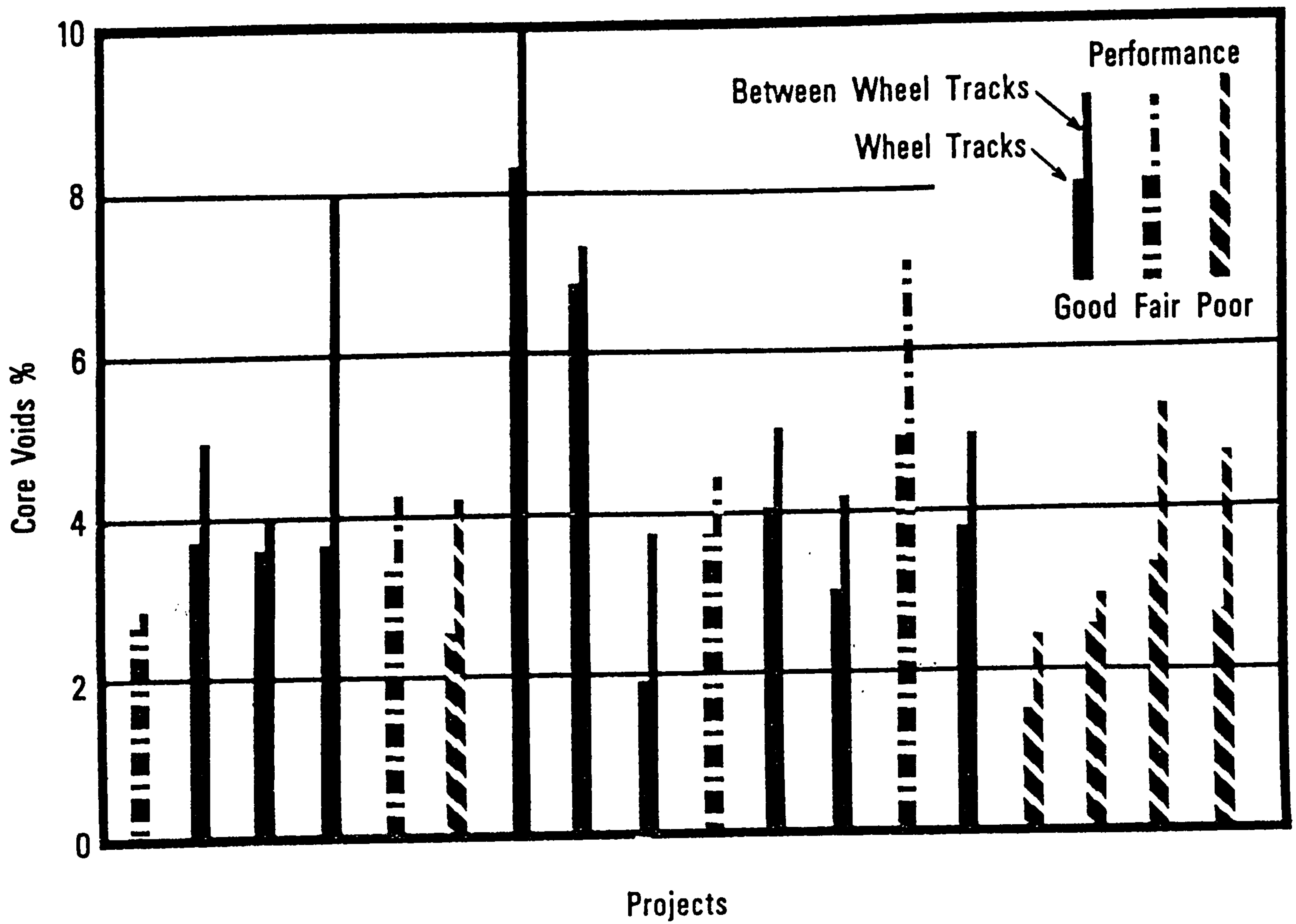

Figure 2.11 Pavement voids vs. performance (O'Connors, 1979).

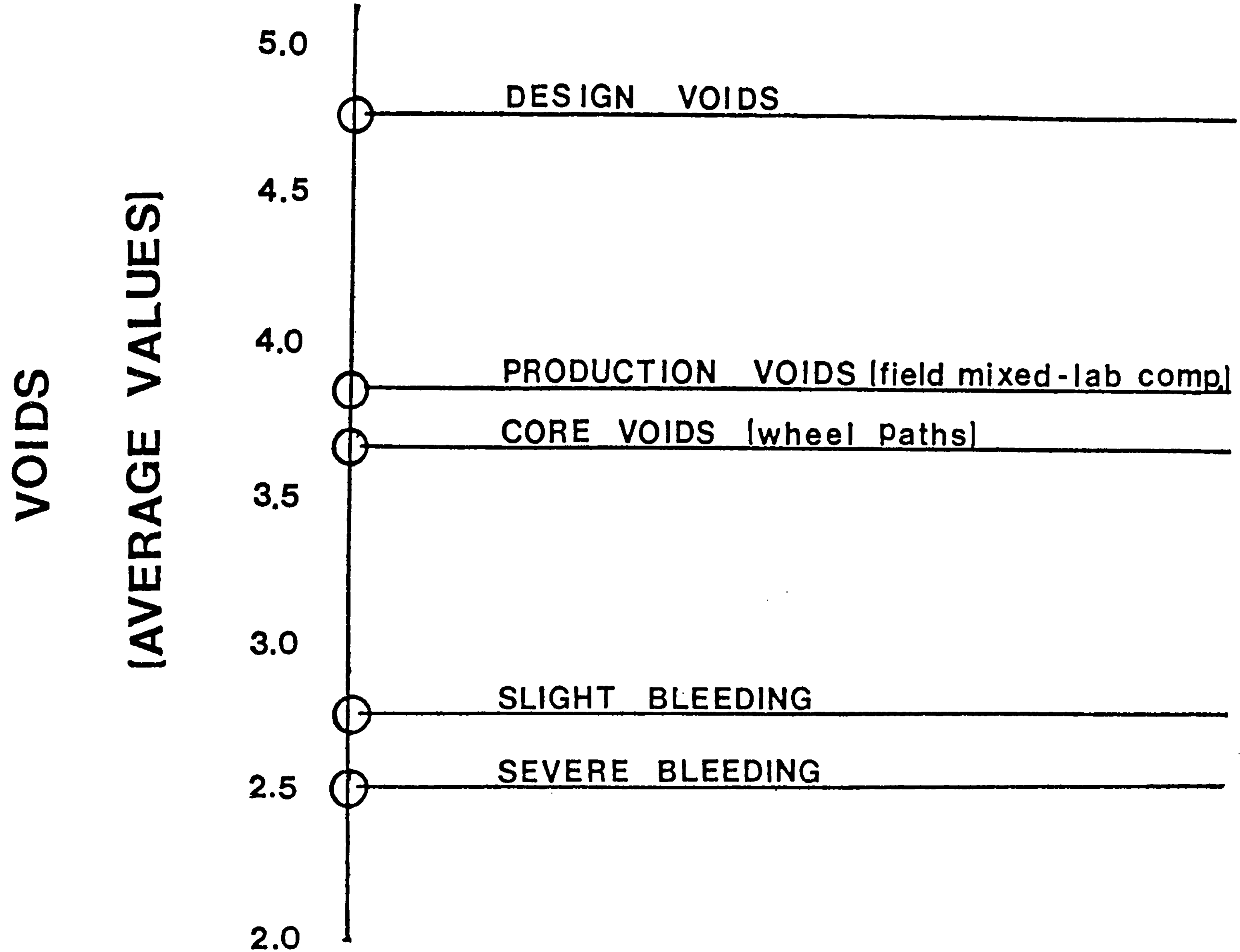

Figure 2.12 Relation of voids to bleeding (O'Connors, 1979).

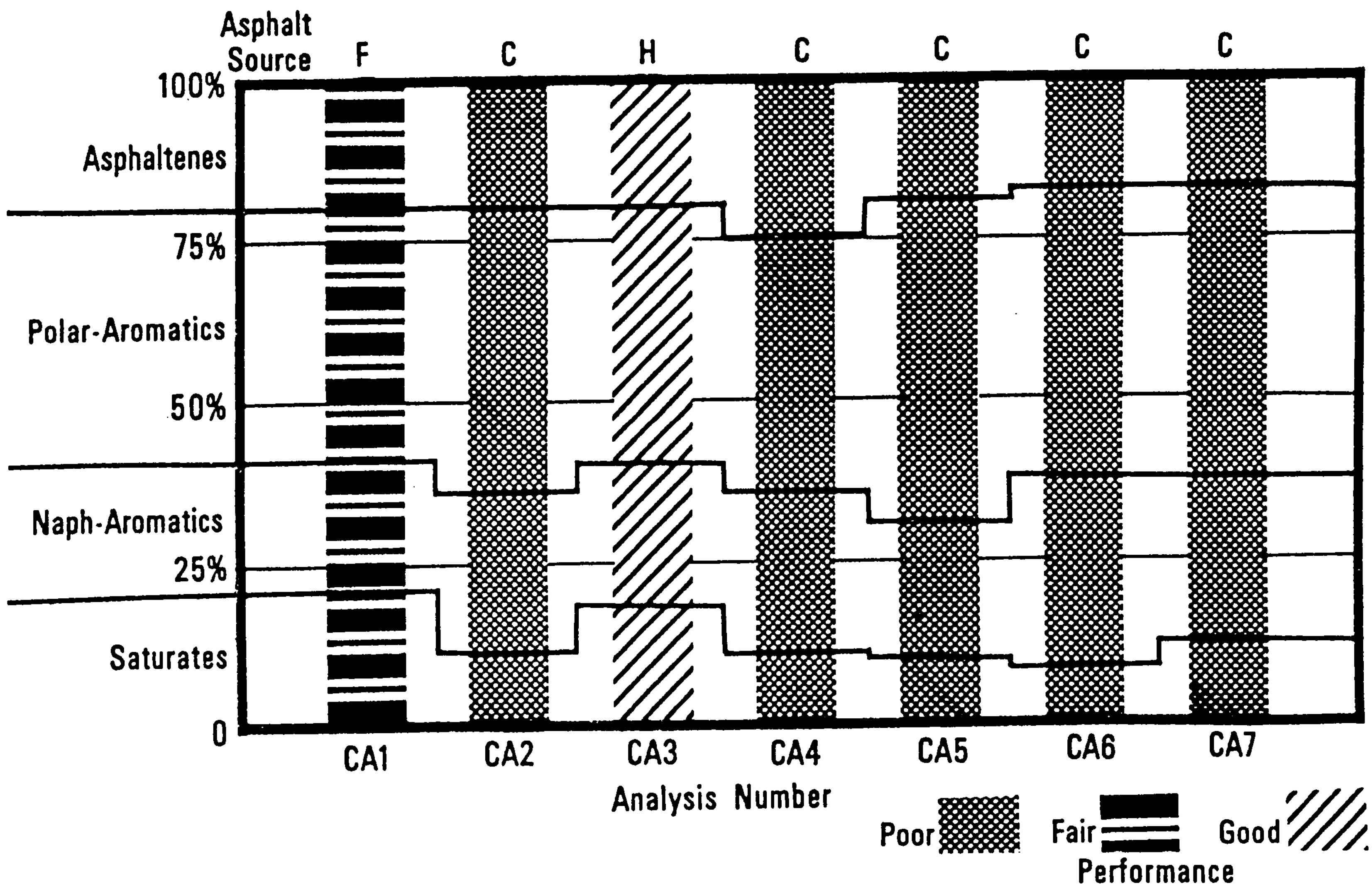

Figure 2.13 Composition and source vs. performance (O'Connors, 1979).

which implies the absence of a direct linear relationship between them and the other variables.

Figure 2.14 shows a plot of recovered asphalt penetration versus pavement performance for the projects studied. The general trend indicates that lower penetrations yield improved performance with regard to rutting and bleeding. From this plot, for a 95 penetration asphalt, the chances of a good performing pavement are about 20 percent; 30 percent for a fair performance and approximately 50 percent for poor performance.

Conclusions

1. Of the four distress factors (wheel rutting, bleeding, raveling, and cracking), only wheel rutting and bleeding were considered significant. The lack of raveling and cracking is believed to be a result of the high percentage of soft asphalt used in these projects.

2. Several of the projects were designed with void contents less than 3%, which had the effect of an over-asphalted mix. Most of these low void content mixes exhibited some bleeding in the wheel path.

3. AC-10 grade asphalt cement had been used almost exclusively in these projects. The hardness or penetration of these AC-10's varied considerably from source to source.

4. Bleeding in the wheel paths, for the pavements surveyed, occurs between 2.5 and 3.0 percent voids. The void content in the wheel paths can also be expected to be about one percent lower than the design voids.

Alberta - Permanent Deformation

This study (McMillan, 1989) examines the role of asphalt binder properties within the overall mechanism of permanent deformation (rutting) of asphalt concrete pavements in Alberta. Other factors contributing to rutting have also been analyzed. These factors include aggregate characteristics, in-place mix characteristics, mix design characteristics and traffic and climate conditions. The binders investigated include:

- Conventional petroleum asphalt
- Polymer modified asphalt (PMA)
- Recycled asphalt binders (RAP)

McMillan (1989) included a literature review, a survey of selected sites within the Alberta Transportation and Utilities roadway network, and an intensive laboratory testing program in his report. For the most part, this study was limited to those pavement structures which had never been overlaid in order to reduce the variables involved. The historic field data (Table A.5) collected included:

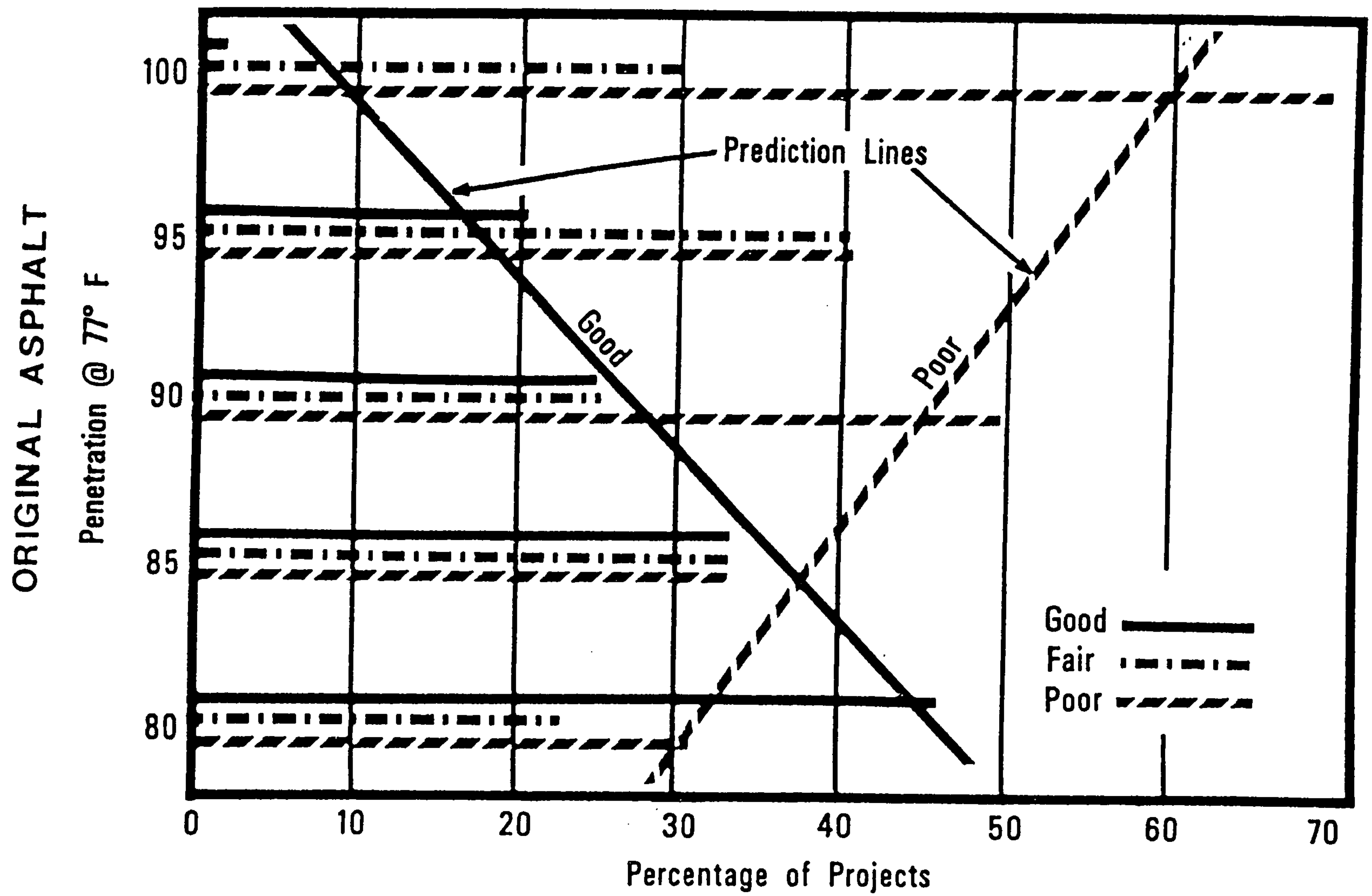

Figure 2.14 Relation of penetration to performance (O'Connors, 1979).

- Pavement structural data, i.e., AC thickness, Asphalt Stabilized Base Course (ASBC) presence and thickness, base thickness
- Traffic (ESALs)
- Original asphaltic properties (Penetration at 25°C and absolute viscosity)
- Climate
- Pavement age

In order to determine the permanent deformation of the laboratory mix samples, the repeated load triaxial test (Hadipour, 1987) was used. A total of 180 laboratory samples were prepared. Sample mix design variables are tabulated as shown in Table 2.12. In addition to these mix design variables, test temperatures of 25°, 35°, and 45°C were used.

VARIABLES	RANGE	LEVELS
Binder Type	Conventional, PMA, RAP	3
Binder	120-150A, 150-200A, 200-300A, 300-400A grade asphalt	4
Aggregate	Blackfolds pit, 12.5 mm/ Goose Lake pit, 16 mm	2

Table 2.12 Design Variables in Alberta Study (Source: McMillan, 1989)

Field test specimens were also prepared. This was done in an effort to better relate the results of the testing program to field conditions. These test specimens were formed in the field using plant-produced mix and a hand-operated kneading compactor. The rheological data from the three binder types are presented in Table 2.13.

ASPHALT	PENETRATION (dmm)	VISCOSITY ABS (Pa.s)	VISCOSITY KIN (mm^2/s)	S.G.	ATFOT LOSS (%)	ATFOT PEN. (dmm)	ATFOT ABS VISC. (Pa.s)
120-150A	142	113	320	1.029	0.249	83	257
150-220A	173	87.5	289	1.029	0.28	94	202
200-300A	273	46.4	209	1.023	0.366	140	107
300-400A	373	28.6	172	1.021	0.705	181	76.4
SC3000	too soft	7.3	79	1.020	5.556	247	40.3
Husky PMA	160	*288	725	1.020	0.527	111	582
Imp. PMA	82	*798	874	1.013	0.645	47	1402
RAP-HW 1	90	160	310	-	0.922	57	350
RAP-HW 2	64	268	395	-	0.950	43	610

* The shear rates for the two PMA absolute viscosities were not equal.

Table 2.13. Rheological data for asphalt cements in Alberta study (Source: McMillan, 1989)

Results

McLeod's Pen-Vis number was calculated as the measure of temperature susceptibility of the asphalt binder. The stiffness values were determined from McLeod's (McLeod 1976) modified nomograph for binder stiffnesses. Rut measurements were made in both inside and outside wheel paths for each lane at each project (Table A.6). Rut profiles measured at each of the field sites showed evidence of mix shoving associated with rutting. McMillan realized that it was difficult to separate the actual contribution of the AC layer to rutting, so he used the overall[12] rut depth in his analysis.

The field data was analyzed using regression analysis. Correlation was sought between the observed rutting (dependent variable) and the mix design data, temperature susceptibility, stiffness, and changes in binder rheology. The model developed based on field measurements has a R^2 of 0.6237. The equation is as follows:

Rut depth = 2.9630 + 0.0076*(Daily Cumulative ESAL's)

- 0.0024*(Daily Cumulative ESAL's)*Log(Abson stiffness)

where: ESALs = equivalent single axle loadings.

Abson stiffness = Calculated stiffness of Abson recovered binder, kPa

Another model was also developed based on design data, which uses original binder stiffness as a predictor variable. The significance of this model is that the inputs required are available at the design stage. This model shown below has a R^2 of 0.4539:

Rut depth = 2.6186 + 0.0060*(Daily Cumulative ESAL's)

- 0.0023*(Daily Cumulative ESAL's)*Log(Original Binder Stiffness, kPa)

These models simply imply that for given loadings, the stiffness of binder will influence the amount of rutting experienced.

The strain data (Table A.10) obtained from the repeated load triaxial testing was also analyzed using regression analysis techniques. Models were developed for each binder type. The best fit model and the respective R^2 is listed for each binder type below.

---

[12]Rutting, due to all structural layers.

a. Conventional asphalt series model (R^2 = 0.8732)

$$\text{Log}(\epsilon) = 0.9521 + 0.5851*\text{Log}(N) - 0.1079*\text{Log}(N)*\text{Log}(\text{Abson Stiffness})$$

b. Recycled asphalt model (R^2 = 0.8313)

$$\text{Log}(\epsilon) = 0.9519 + 0.5634*\text{Log}(N) - 0.8652*\text{Log}(N)*\text{Log}(\text{Abson Stiffness})$$
$$- 0.0295*\text{Log}(N)*\text{Log}(\text{Abson Stiffness})*\text{Log}(\%\ \text{RAP})$$

c. Polymer modified asphalt model (R^2 = 0.8794)

$$\text{Log}(\epsilon) = 0.6323 + 0.3618*\text{Log}(N) - 0.0979*\text{Log}(N)*\text{Log}(\text{Stiffness})$$

where: ϵ = permanent strain (%)

N = number of load applications using repeated load triaxial test

Abson Stiffness = calculated stiffness of Abson recovered binder, kPa

% RAP = percentage of reclaimed asphalt pavement in mix

Stiffness = calculated stiffness of original binder, kPa

Overall, the binder stiffness was shown to greatly influence the observed strain in the asphalt concrete samples.

Combined models were also determined to evaluate the relative influences of the different binder types. The samples from virgin asphalt mixes were treated as the standard mix[13].

a. Virgin/Recycle model (R^2 = 0.8655)

$$\text{Log}(\epsilon) = 0.9494 + 0.5766*\text{Log}(N)$$
$$- 0.1041*\text{Log}(N)*\text{Log}(\text{Abson Stiffness})$$
$$- 0.0135*\text{Log}(N)*\text{Log}(\text{Abson Stiffness})*\text{Log}(\%\ \text{RAP})$$

[13] The mix design data is presented in Table A.7. The densities for the formed samples, the calculated air voids, and the calculated binder stiffness are reported in Table A.8. Following the repeated load tests selected samples were tested to determine the in-situ asphalt rheology of the asphalt using the Abson procedure (Table A.9).

b. Virgin/PMA model $(R^2 = 0.9000)$

$$\mathrm{Log}(\epsilon) = -0.9119 + 0.4885*\mathrm{Log}(N) - 0.1075*\mathrm{Log}(N)*\mathrm{Log}(\mathrm{Stiffness}) - 0.1568*\mathrm{Dummy}$$

where: Dummy = dummy variable (-1 for virgin mixes, +1 for PMA mixes).

According to the data, the polymer modified asphalt (PMA) mixes reduced the strain levels on the order of 50% at the 25°C test temperature, compared to a 120-150A asphalt concrete mix. Hence, the relative effect of binder types to strain can be summarized as: virgin mixes are least resistant, with recycle mixes and PMA mixes exhibiting increasingly more resistance to permanent deformation. Also, the strain of various mix samples measured during the repeated load triaxial testing was significantly effected by the test temperatures. These data are included in Tables A.10a-A.10e.

The main focus of this study was on the influence of binder on rutting of asphalt concrete pavements. On a limited basis, the aggregate grading and mix characteristics were also considered in the analysis. All mix characteristics, for both the laboratory samples and the site samples were at or near targeted design values. This lack of significant range in these characteristics could possibly have resulted in other significant main effects or interactions not being observed within the analysis conducted for this study.

Based on these findings, the author suggests appropriate asphalt grades for various loadings. These suggestions are presented in Table 2.14.

Design Life Traffic (Cumulative ESAL's)	Asphalt Grade
500,000>	200-300A or softer
500,000< >1,000,000	200-300A
>1,000,000	150-200A, 120-150A, or PMA

Table 2.14. Asphalt grades for controlling permanent deformation in Alberta study (Source: McMillan, 1989)

Conclusions

Field Study:

1. Based on the field data analysis, pavements subjected to less than 0.5×10^6 ESALs over their design life are not likely to experience significant rutting.

2. Field observations of rut profiles showed evidence of mix shoving associated with the rutting, indicating an inability of the asphalt concrete to resist the applied shear forces.

3. The models developed to explain the measured rut depths at the field sites were significant. These models have traffic and the binder stiffness (original and Abson) as variables.

Laboratory Study:

1. The binder stiffness and the binder temperature susceptibility both affect the resulting strain experienced by the asphalt concrete.

2. The PMA mixes exhibit the lowest percent permanent strain.

3. On the basis of the results from the repeated load triaxial testing, changing from 150-200A to 200-300A grade asphalt may reduce strains by as much as 30-40 percent.

4. The aging of the binder during mixing is an important issue, as it is the rheology of the binder after the mixing process that influences the permanent deformation characteristics of an asphalt concrete.

5. The use of repeated load triaxial test equipment can be developed for modeling the in-place pavement performance in terms of permanent deformation (rutting).

Alaska-- Asphalt Concrete Properties and Performance

Description

McHattie (1981) collected field data on pavement sections throughout Alaska as part of this study. However, a scarcity of materials and construction records on the asphalt grades originally used in these sections necessitated that performance correlations be based on properties of asphalts extracted from the pavement cores, rather than original properties. The following parameters were evaluated:

• Thickness of pavment	(used standard 6" caliper at 3 locations on each sample)
• Quantitative extraction of asphalt cement	(AASHTO T-164 Alaska Test Method T-16)
• Gradation of aggregate	(AASHTO T-27)
• Abson recovery apshalt content w/ash correction	(AASHTO T170-73)
• Absolute Viscocity at 140°F	(AASHTO T-202)
• Penetration at 77°F	(AASHTO T-49-78)
• Penetration at 39.2°F	(AASHTO T-49-78)
• Indirect tensile strength	(Lottman Tensile Strength Ratio)
• In-place density (SSD)	(Alaska Test Method T-18)
• Maximum density	(procedure was modified by J. A. waddell from ASTM C-70-72 and AASHTO T142-74)

The data was analyzed using both descriptive statistics and correlative studies. The variables used in the analysis were:

Environmental Variables:

Region	Mean precipitation
Climate zone	Mean snowfall
Mean temperature	Degree days freezing
Wet days/year	Degree days thawing
Average diurnal temp. variation	Age
Average season temp. variation	Traffic EAL

Asphalt Concrete Material Variables:

Asphalt cement content	Maximum density
Viscosity	Aggregate gradation
Penetration	Tensile strength
In-place density	

Performance Variables:

Miscellaneous thermal (map) cracking[14]	Full width patching
Major transverse cracks	Longitudinal cracks
Rut depth	Alligator cracking

Miscellaneous:

Pavement thickness

Results

It was determined that the various climate zones in Alaska did not have a significantly different effect on the aging of the asphalt cement. Plots of absolute viscosity and penetration versus pavement age (Figure 2.15) indicate that considerable age hardening of asphalt occurs in the first five years, with the greatest change in the first three years.

[14]Miscellaneous thermal (map) cracks usually form as a randomly oriented interconnected net of fractures of width less than 1/8", affecting only the asphalt concrete surface. The geometric patterns created by map cracking are usually much larger than those exhibited as a result of traffic related fatigue, and intercrack spacing can range up to 10-20 feet. Figure A.3 is a pictorial representation of typical thermal and longitudinal crack types. Individual crack segments are very often oriented either longitudinal or transverse to the centerline, which results in a commonly observed pattern of orthogonal squares and rectangles. Map cracks as mentioned previously are mostly hairline features and can exist unnoticed by the driving public. Ironically, they tend to be made most noticeable as a result of careful maintenance sealing, which outlines and widens the appearance of the cracks and may also induce small but noticeable bumps felt or heard by the motorist. Because it is random in nature, it is difficult to quantify map cracking. In this study, map cracking is measured by counting intersections with one road-width transverse line and one equal length longitudinal line at 11 randomly selected individual locations within each study section (see Figure A.3). Cracks intersecting transverse grid lines are by definition "miscellaneous transverse thermal cracks" while those intersecting longitudinal grid lines are termed "miscellaneous longitudinal thermal cracks." These cracks are characterized by the grid line which is crossed and not by the orientation of the crack segment per se.

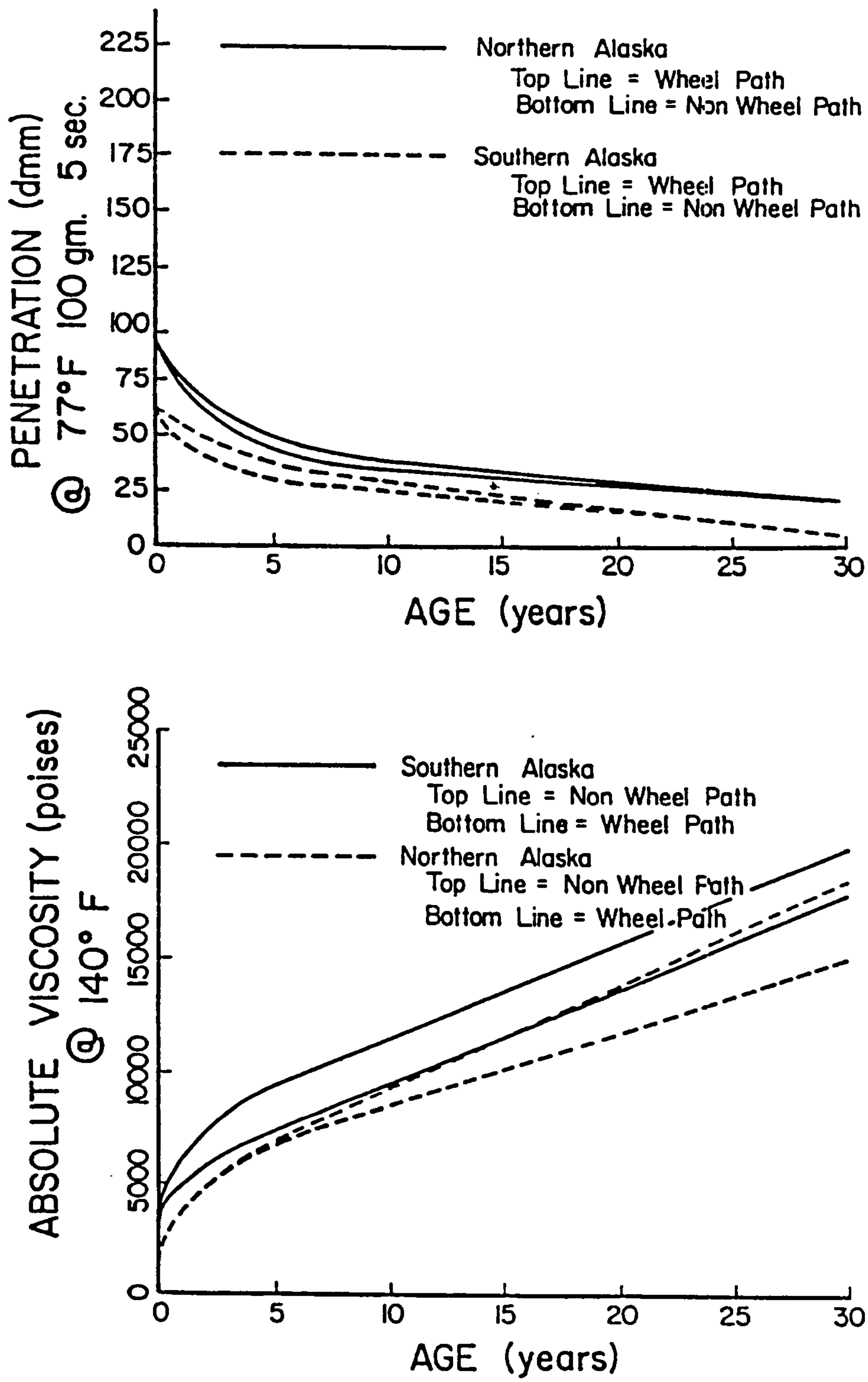

Figure 2.15 The aging process of asphalt cement in Alaska--Apparent best fit lines (McHattie, 1981).

McHattie (1981) states that performance predictions based solely on a knowledge of fresh asphalt properties are not possible because of the non-linearity associated with the asphalt aging process.

Temperature susceptibilities varied widely for in-situ Alaskan asphaltic materials. Aging relationships indicated some tendency for asphalts to become somewhat less temperature susceptible with time when using PVN method. This is inferred from the best fit asphalt cement aging curves which show a leveling-out of the decrease in penetration with time while the absolute viscosity continues a fairly steep climb. McHattie (1981) is of the opinion that due to the nature of the PVN curves, holding penetration constant while increasing viscosity will lead to higher positive PVNs and therefore lower temperature susceptibilities. Although usually considered an extremely important predictor ofpavement performance, voids content of the asphalt mix showed little correlation with extracted asphalt viscosities and an even lower level of correlation with tensile strength of the pavement cores.

Despite extensive statistical analysis, the author (McHattie, 1981) was unable to discover significant correlations aside from general documented trends in AC behavior. Some Bivariant correlation coefficients are calculated in Table 2.15.

As an alternate form of data analysis, material properties corresponding to better performing pavements were also examined. Table 2.16 is a listing of properties derived from the upper 50 percentile pavements on the basis of general fatigue and thermally induced stress.

After all factors were considered, the very best asphalt concrete materials were indicated as being those which remain relatively soft with the passage of time. This holds true in viewing "soft" in terms of lower retained viscosity and "soft" in terms of low retained tensile strength. The project was generally hampered by the lack of available construction records although asphalt cements are assumed to have originally met specifications. A similar but more serious problem was the inability to document the process of performance deterioration with time. According to McHattie (1981) neither pavement performance nor asphalt cement properties change linearly with age. Hence, a great deal of documentation and monitoring is required to measure any of the above over time.

Conclusions

The cataloging (McHattie, 1981) of asphalt concrete materials properties from a large number of Alaskan pavement sections indicated that:

1. A large variation can be expected in common physical properties after long periods of field aging have taken place (see Table 2.16).

2. Results from indirect (diametral) tensile testing were useful because of the information they provided about asphalt concrete strength (range 13-100 psi, mean 48 psi) and their inclusion as materials strength variables in correlation analyses.

BEST CORRELATING VARIABLES	NOT CONTROLLED FOR CLIMATE EFFECTS R-VALUE	CONTROLLED FOR CLIMATE EFFECTS R-VALUE
Rut Depth with:		
Saturated tensile strength*	.50	.41
Dry tensile strength	.38	.28
Absolute viscosity @140°F, WP	.31	.32
Absolute viscosity @140°F, NWP	.28	.32
Penetration @77°F, NWP	-.25	-.28
Penetration @39.2°F, NWP	-.23	-.26
Penetration @39.2°F, WP	-.20	-.19
% - #200 aggregate	.19	.21
% - #40 aggregate	.19	.12
Penetration @77°F, WP	-.18	-.19
Alligator Cracking with:		
Dry tensile strength*	.62	.66
Absolute viscosity @140°F, WP*	.50	.50
Saturated tensile strength*	.49	.54
Absolute viscosity @140°F, NWP	.47	.48
% - #200 aggregate	.36	.37
Penetration @77°F, NWP	-.32	-.35
Penetration @39.2°F, NWP	-.25	-.30
Penetration @39.2°F, WP	-.23	-.24
Penetration @77°F, WP	-.21	-.23
Total pvmt. thickness, WP	-.06	-.09
Edge Longitudinal Cracks with:		
Penetration @39.2°F, NWP	-.33	-.32
Penetration @77°F, NWP	-.30	-.29
Penetration @77°F, WP	-.28	-.27
Penetration @39.2°F, WP	-.26	-.24
Bitumen content, WP	-.22	-.23
Saturated tensile strength	.18	.16
% voids @ shoulder	.14	.15
Maximum density	.14	.18
Top layer pvmt. thickness, WP	-.13	-.18
Absolute viscosity @140°F, NWP	.12	.13
Major Transverse Thermal Cracks with:		
Absolute viscosity @140°F, NWP	.32	.32
Bitumen content, WP-.31	-.22	
Penetration @77°F, NWP	-.30	-.33
Absolute viscosity @140°F, WP	.26	.25
Bitumen content, NWP	-.23	-.20
Total pvmt. thickness, NWP	-.22	-.05
Total pvmt. thickness, WP	-.21	-.05
% - #10 aggregate	-.17	-.07
Saturated tensile strength	.17	.31
Penetration @77°F, WP	-.17	-.16

*Significant Variable
(Note: WP = wheelpath; NWP = non-wheelpath)

Table 2.15. Bivariant Correlation Coefficients for Alaska (Source: McHattie, 1981)

	MEDIAN	MEAN	STD. DEV.
Top layer pavement thickness, wheelpath	1.53 inch	1.72	0.59
Top layer pavement thickness, non-wheelpath	1.59 inch	1.79	0.57
Total pavement thickness, wheelpath	1.65 inch	2.15	1.06
Total pavement thickness, non-wheelpath	1.64 inch	2.24	1.10
Gradation (Cum. % less than) 1"	100%	100	0
3/8"	82%	81	5
#4	57%	58	6
#10	42%	42	4
#40	21%	20	2
#200	7%	7	2
Maximum density of asphalt core	157.6 pcf	157.2	2.2
Average S.S.D. Density	146.9 pcf	146.0	3.4
S.S.D. density in wheelpath	147.6	146.4	3.8
S.S.D. density, non-wheelpath	146.1	145.2	3.5
Average % void content	6.9%	7.2	2.2
% void content, wheelpath	6.2%	6.8	2.3
% void content, non-wheelpath	7.4%	7.6	2.2
Average % bitumen content with ash correction	6.0%	5.9	0.9
Average absolute viscosity	3871 poises	4656	3461
Average viscosity, wheelpath	2728 poises	3500	2563
Absolute viscosity, non-wheelpath	3989 poises	5812	5440
Average penetration at 39.2°F	15 dmm	15	8
Penetration at 39.2°F, wheelpath	16 dmm	16	9
Penetration at 39.2°F, non-wheelpath	14 dmm	14	9
Average penetration at 77°F	46 dmm	51	21
Penetration at 77°F, wheelpath	49 dmm	54	24
Penetration at 77°F, non-wheelpath	45 dmm	48	21
Tensile strength, saturated core	30.0 psi	32.2	14.2
Tensile strength, dry core	29.4 psi	29.0	11.6

Table 2.16. Material properties associated with a high level of performance

Tensile strength appeared to show a good general correlation with pavement performance.

3. Generalized asphalt aging plots indicate that the bulk of age hardening as measured by 77°F penetration tests, occurs within:

   4 - 8 years in South (Coastal Area) Alaska
   7 - 8 years in North (Interior) Alaska

   Laboratory test-method development should consider the above figures as guidelines for developing aged materials specifications and in standardizing test procedures.

4. McHattie (1981) emphasizes that the non-linearity of asphalt cement aging curves would indicate that valid performance assumptions could not necessarily be based on original properties. It was suggested that laboratory procedures be developed to simulate field aging.

5. Generally acceptable properties of asphalts and asphaltic mixtures used in the Alaska environment can be summarized from Table 2.16, as shown in Table 2.17.

ESSO - Asphalt Cements for Low Temperature Service

Description

This paper (Robertson, 1987) examines the relationship between two temperature susceptibility parameters, the Penetration Index (PI) and the Pen-Vis Number, and the low temperature stiffness of asphalts determined by measurements at temperatures down to -40°C. For this purpose, the correlation between the Penetration Index and the Pen-Vis Number, and the tensile relaxation modulus at -30°C was evaluated. The modulus values were determined from measurements of the shear compliance over the temperature range -40 to 40°C with a sliding plate Rheometer. Shear compliance master curves, showing the effects of loading time and temperature for different PI's, were used to construct shear modulus master curves from which the tensile modulus values were calculated using the relationship:

$$E = 3G$$

where: E = Elastic modulus
G = Shear modulus

Two penetration levels (150 ± 5 and 50 ± 5), and a wide range of temperature susceptibilities, as defined by the Penetration Index (-2.59 to +2.18) and the Pen-Vis Number (-1.74 to 0.03), were represented. Table 2.18 is a list of properties of asphalt cements used in this study. Plots of the low temperature modulus versus Penetration Index (Figure 2.16) and versus Pen-Vis Number (Figure 2.17) showed an excellent correlation between the low temperature modulus and the PI, and none between the low temperature modulus and the Pen-Vis Number.

PROPERTY OF ASPHALTS	ACCEPTABLE VALUE $\bar{\mu}$	δ_{n-1}
Absolute Viscosity, poises (140°F)	4656	3461
Penetration, dmm (39.2°F)	15	8
Penetration, dmm (77°F)	51	21
ASPHALTIC MIXTURES	$\bar{\mu}$	δ_{n-1}
Tensile strength, saturated core	32.2	14.2
Tensile strength, dry core	29.0	11.6

Table 2.17 Properties of asphalts and asphaltic mixtures used in the Alaska environment. (Source: ARE analysis)

For the estimation of thermal cracking temperatures, the author looked at both empirical and rational methods currently available. The empirical methods investigated include Hajek and Haas (1972), McLeod (1972), Fromm and Phang (1971), Readshaw (1972), Gaw et al. (1974), and Sugawara et al. (1982). The author maintains that these

CRUDE SOURCE	A	B	B	C	C	A	B	B	C	D
PROCESSING	VACUUM RESIDUE	VACUUM RESIDUE	OXIDIZED	VACUUM RESIDUE	OXIDIZED	VACUUM RESIDUE	VACUUM RESIDUE	OXIDIZED	OXIDIZED	OXIDIZED
Penetration @ 25°C (100/5)	147	152	149	149	147	51	55	52	51	50
10°C (100/5)	29	21	37.5	18	24	12	8.5	19	13.2	16.2
4°C (100/5)	15	9	23	8	12.3	6.5	4.5	12.5	7.7	11.7
Penetration Index (1)	-1.05	-2.28	+0.09	-2.59	-1.67	-0.35	-1.80	+2.18	+0.15	+1.61
Viscosity at 60°C, Pa.s	82.0	36.0	28.4	56.8	60.6	592.3	172.2	5670	442.5	1359
135°C, cSt	273	151	118	217	219	648	296	584	515	822
Pen-Vis Number (2)	-0.36	-1.31	-1.74	-0.73	-0.73	-0.27	-1.27	-0.39	-0.58	+0.03
Wax Content, m% (3)	2.4	4.5	9.7	3.0	5.1	2.2	4.6	11.4	~5	~3.5

(1) Calculated from penetration at 25°, 10° and 4°C.
(2) Calculated from penetration at 25° and viscosity at 135°C.
(3) By Differential Scanning Calorimetry.

Table 2.18. Properties of asphalt cements used in Esso study (Source: Robertson, 1987)

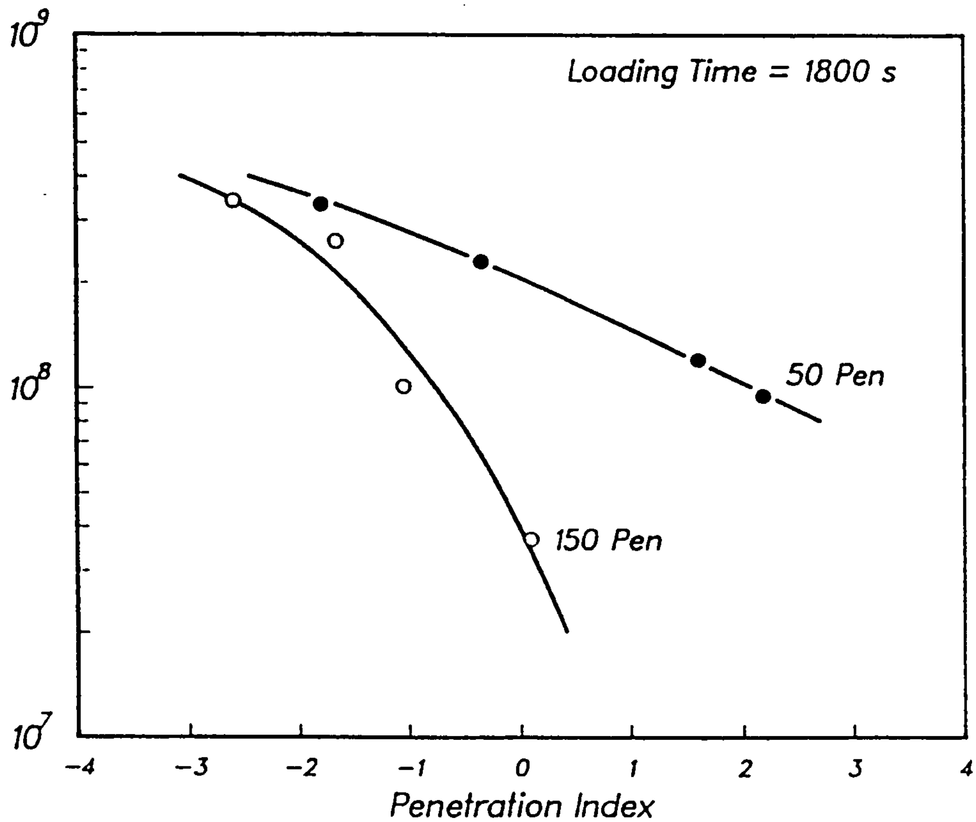

Figure 2.16 Low temperture modulus vs. Penetration Index (Robertson, 1987).

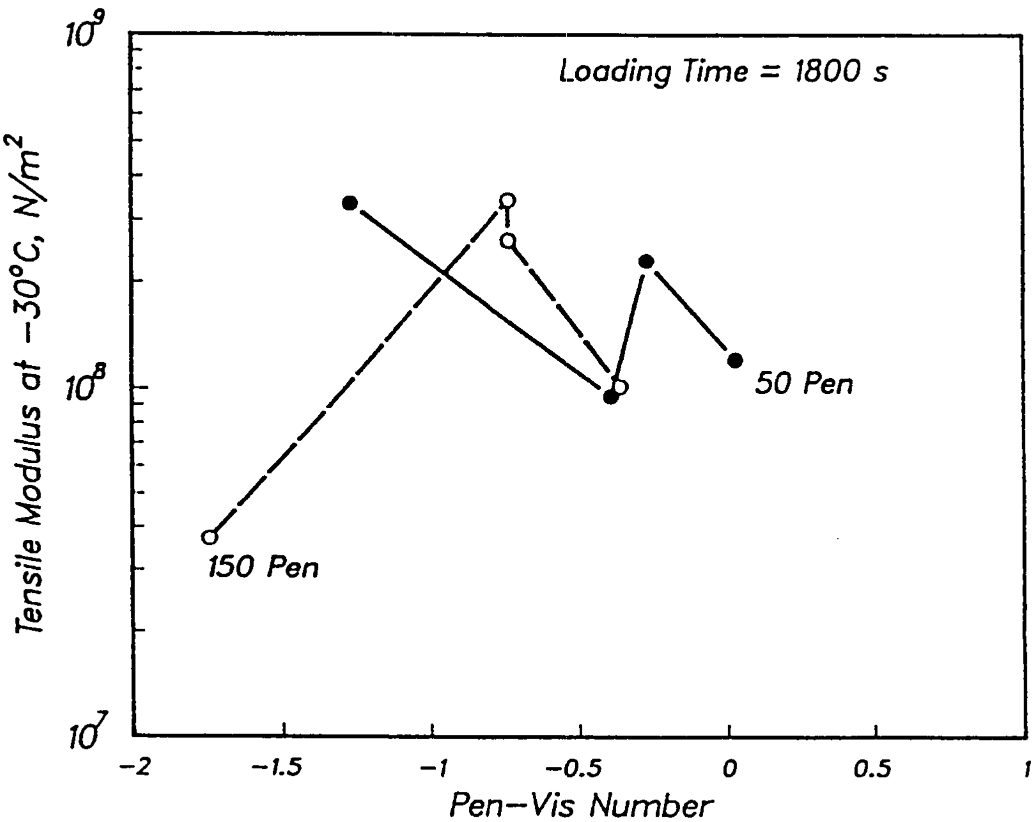

Figure 2.17 Low temperature modulus vs. Pen-Vis number (Robertson, 1987).

empirical methods, while noteworthy, are not useful for the general case in which available asphalts include both oxidized and vacuum reduced materials, or those from crude oil sources not included in the empirical correlation. The author based the development of a design chart to select asphalts for low temperature service on the rational method. The rationale is to limit the tensile stress in the pavement resulting from thermal contractions under restraint, to prevent thermal cracking.

Results

A design chart was developed using the rational approach. The fracture temperature was taken as the temperature to develop a tensile stress of 5×10^5 N/m². This value was taken from Hills (1974), as being the average fracture stress for asphalt. Hills (1974) determined this by calculating the fracture temperatures of a number of asphalts experimentally by cooling thin films applied to a quartz substrate. Hills (1974) used the following equation to arrive at the fracture stress:

$$\sigma_T = -\alpha_T \Sigma S(\Delta t, T) \Delta T$$

where:

σ_T = Thermal stress, N/m^2

α_T = coefficient of linear expansion, /°C

$S(\Delta t, T)$ = Van der Poel stiffness modulus at the average temperature in the interval ΔT and loading time Δt, N/m^2

ΔT = temperature interval for each calculation step, °C

In applying this equation, Δt is usually taken as the time required to cool over the temperature interval ΔT.

Experimentally determined modulus values were used in the stress calculations to avoid errors associated with the generalized correlation of Van der Poel (1954), which is not applicable to asphalts having a high wax content. The asphalts used are described in Table 2.18. They were selected to represent wide ranges of wax content and temperature susceptibility, so that the design chart would be applicable to the many types of asphalt cement currently available. For each asphalt, the thermal stress versus temperature relationship was estimated using the following equation:

$$\sigma_T = - \int_{T_o}^{T} \alpha_T E(t,T)\, dT$$

where:

σ_T = Thermal stress, N/m^2
T_o & T = initial and final temperatures respectively, °C
α_T = coefficient of linear expansion, /°C
$E(t,T)$ = time and temperature dependent tensile relaxation modulus, N/m^2

This equation considers the relaxation of stresses with time, and the variation in modulus with temperature. It is applicable to any time-temperature history.

From plots of thermal stress versus temperature, the temperature at which the stress reached 5×10^2 N/m^2 was determined for each asphalt, and this was taken as the fracture temperature of the bitumen. Two typical thermal stress versus temperature curves for asphalts cooled from 0°C at a rate of 10°C/h are shown in Figure 2.18. For each level of penetration at 25°C (50 ± 5 and 150 ± 5), the Penetration Indices corresponding to fracture temperature between -25°C and -55°C were estimated using the plots of fracture temperature versus bitumen Penetration Index shown in Figure 2.19. These values were used to prepare the design chart in Figure 2.20. This chart gives, for penetrations at 25°C between 30 and 400, the minimum Penetration Index needed to avoid thermal cracking. A safety factor of 10°C has been incorporated into the chart (i.e., a design temperature of -30°C corresponds to a fracture temperature of -40°C).

The author also defines (for this particular design chart) the pavement design temperature as the lowest temperature expected during the design life. The suggested pavement design temperatures for the pavement surface and lower layers for various minimum air temperatures between -15°C and -50°C are shown in Table 2.19. These can also be derived by using the following equation (Robertson 1987):

$$T_{min} = 0.859\ T_{air} + (0.02 - 0.0007\ T_{air})\ D + 1.7°C$$

where: T_{min} = minimum temperature at depth D, °C
D = distance below the surface, mm
T_{air} = minimum air temperature during winter, °C

MINIMUM AIR TEMPERATURE, °C	-15	-20	-25	-30	-35	-40	-45	-50
DESIGN TEMPERATURE, °C								
<100 mm below surface	-11	-15	-20	-24	-28	-33	-37	-41
100-200 mm below surface	-8	-12	-16	-20	-24	-28	-32	-36
4200-300 mm below surface	-5	-9	-12	-16	-19	-23	-27	-30
>300 mm below surface	-2	-5	-9	-12	-15	-18	-22	-25

Table 2.19. Suggested pavement design temperatures for low temperature service in Esso study (Source: Robertson, 1987)

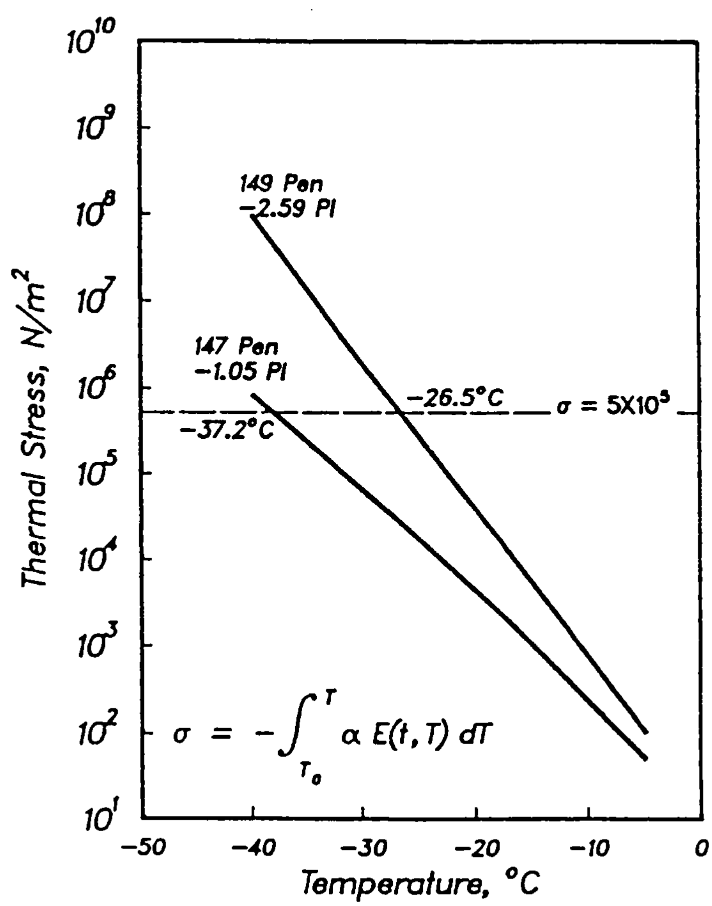

Figure 2.18 Thermal stress from cooling asphalts in ESSo study (Robertson, 1987).

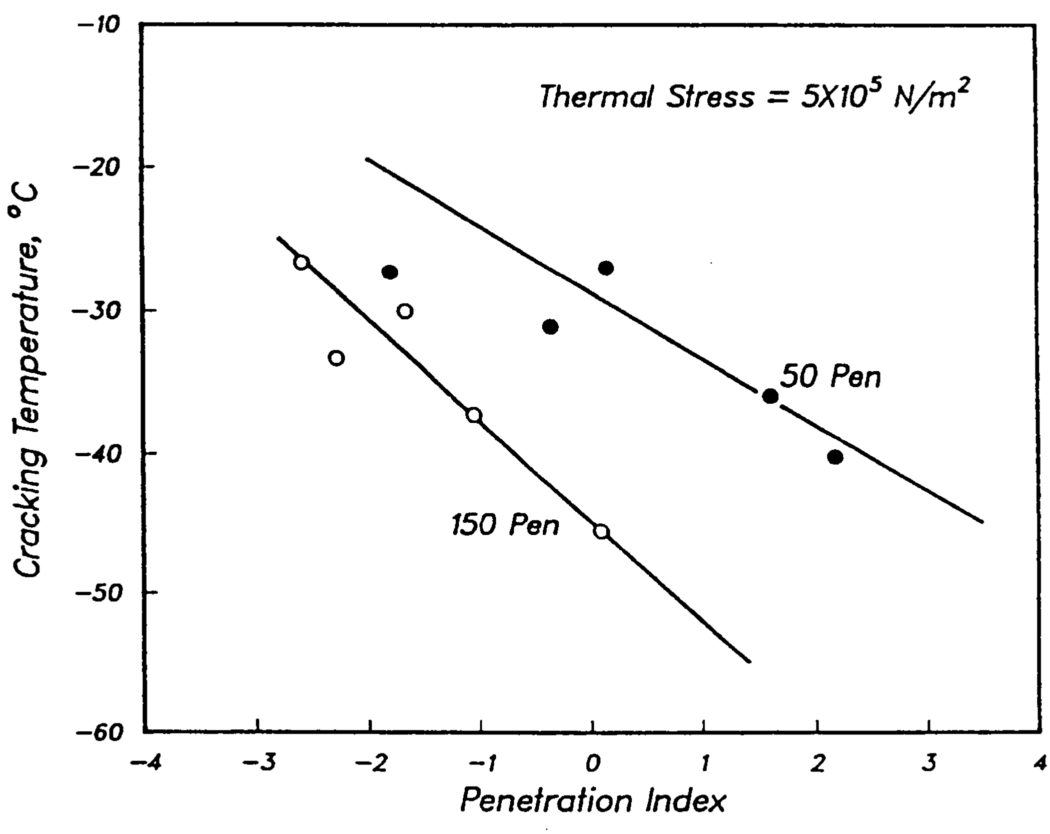

Figure 2.19 Estimated cracking temperature vs. Penetration Index (Robertson, 1987).

The author recommends the following guidelines to select an asphalt cement for use in a pavement located in a cold climate region:

<u>Procedure for Selecting Asphalt Binders:</u> This method is used in conjunction with the Design Temperature Table (Table 2.19) and the Asphalt Selection Chart (Figure 2.20) to select an asphalt cement for use in a pavement located in a cold climate region:

1. Using local weather records for the area where the road is located, determine the minimum winter air temperature expected during the design life of the pavement.

2. Select the pavement design temperature for the expected minimum air temperature and the location of the mix in the pavement structure using the Design Temperature Table. For mixes which will be overlaid, the depth of the mix in the pavement is its location relative to the surface during the first winter following construction.

3. For each grade of asphalt cement available in the area of the project, to arrive at the minimum penetration limit, determine the minimum acceptable Penetration Index for that grade at the pavement design temperature.

4. Evaluate the Penetration Indices of the available asphalt cement supplies, and select one meeting the minimum Penetration Index requirement for its penetration grade.

The following example will illustrate the use of this process:

<u>Example:</u> An asphalt cement is to be selected for use in a surface course pavement in an area where the winter temperature is not expected to fall below -30°C. Asphalts available in the area include the following penetration grades: 80-100, 150-200 and 300-400. The PI for each grade is -1.2.

1. From the Design Temperature Table, the design temperature for a surface course pavement is -24°C.

2. Using the Design Chart and a design temperature of -24°C, select:

   80 minimum penetration asphalt with PI > 0.0, or
   150 minimum penetration asphalt with PI > 1.5, or
   300 minimum penetration asphalt with PI > 3.3.

3. The available 80-100 penetration does not have high enough PI for use in this pavement, but either the 150-200 or the 300-400 grade would be satisfactory. The choice would depend on factors other than resistance to thermal cracking, such as type and volume of traffic, the thickness of the pavement layer, and the quality of the aggregate to be used in the mix.

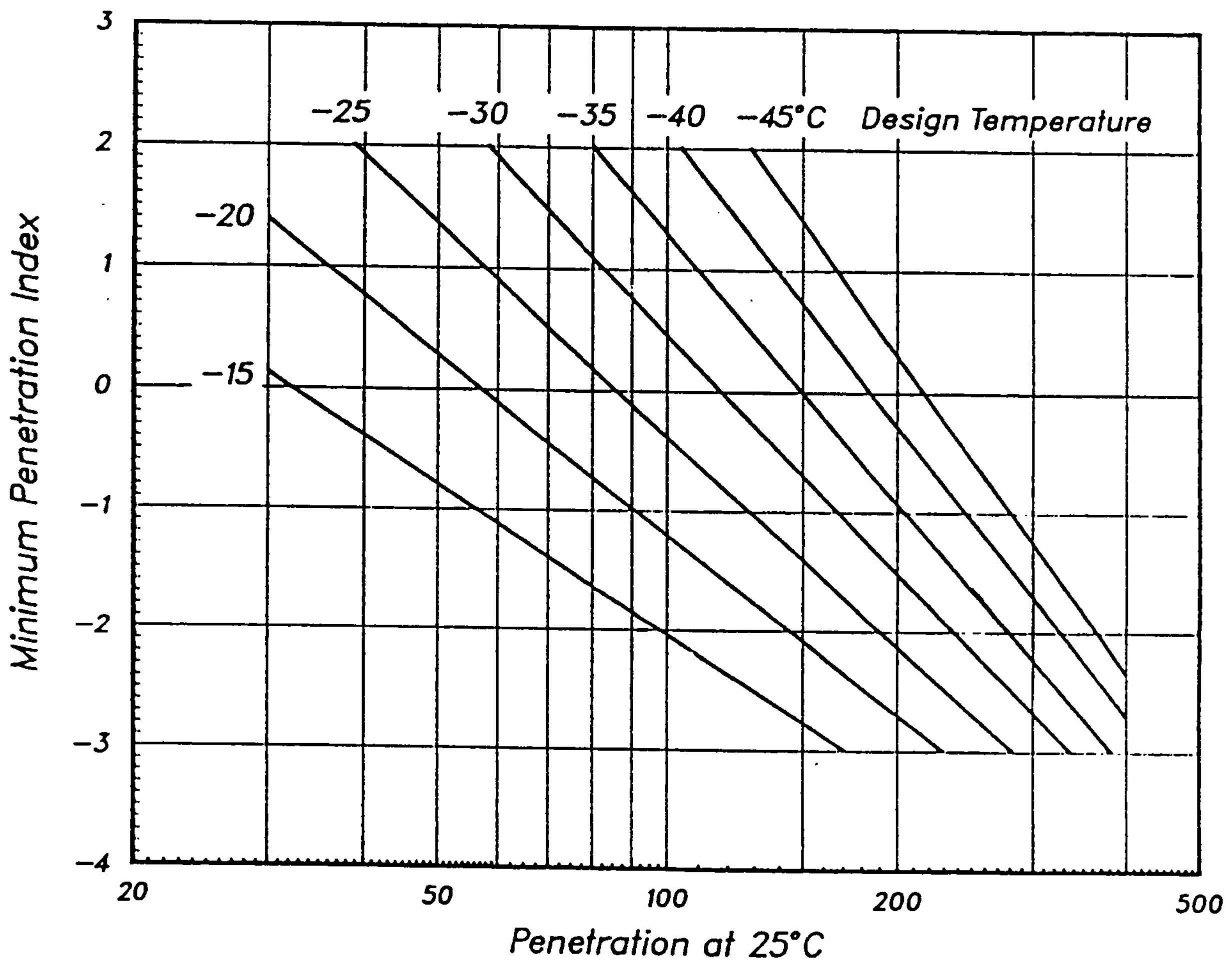

Figure 2.20 Minimum Penetration index to avoid low temperature cracking (Robertson, 1987).

Unfortunately, there were few field measurements of thermal cracking temperatures which could be used to evaluate the accuracy of the design chart. However, data from two laboratory studies which were used to determine the cracking temperatures of asphalt concrete specimen cooled under restraint, and the well documented pavement cracking temperatures of the Ste. Anne test road were used for comparison with values obtained from the design chart. Tables A.11, A.12, and A.13 show the properties of the asphalts in these paving mixes and the observed fracture temperatures. (For further related data see Tables A.14 through A.19.) The predicted cracking temperatures are compared to the actual cracking temperatures of the pavements or paving mix specimen in Figure 2.21. The observed cracking temperatures were, on the average, slightly higher than those obtained from the design chart. Fracture temperatures in laboratory tests were higher than predicted, while those of the Ste. Anne test road were slightly lower. The author (Robertson, 1987) also provides a comparison (Tables A.11 through A.13) between the minimum asphalt cement PI's specified by this design method and those required by Readshaw (1972), Gaw et al. (1974), and Sugawara (1982).

Conclusions

A new design chart for selecting asphalt cements to resist low temperature cracking was presented using measured values of asphalt stiffness and a stress calculation method which takes into account the variation in relaxation times during the cooling period. This chart gives estimates of thermal cracking temperatures which are comparable to measured values in laboratory thermal fracture tests and observations from the Ste. Anne test road.

It can also be concluded that if the PI of the asphalt can be increased by one point on the PI scale, the minimum temperature can be reduced by 3 to 5°C (9°F). Or, if the PI can be increased by one point, the penetration of the selected asphalt could be reduced by one penetration grade.

Iowa - Aging

Description

In 1986, Highway Research Project HR-298 (Enüstün et al., 1990) was initiated to study the relationships between the performance of locally available asphalts and their physicochemical properties under Iowa conditions. The objective was to develop local performance-based asphalt specifications for durable pavements.

Three groups of asphalt samples were tested during this investigation: a) 12 samples from 2 local asphalt suppliers and their TFOT residues; b) 6 cores of known service records; and c) 79 asphalt samples from 10 pavement projects, including original, laboratory aged and recovered asphalts from field mixes and pavement cores as well as laboratory-aged mixes. The 10 field projects selected (four AC-5's, two AC-10's, and four AC-20's) were from a variety of locations (2 interstates, 3 primary and five secondary highways). From each project, the virgin asphalt and aggregate, plant mix as well as core samples were obtained. Table 2.20 lists these projects.

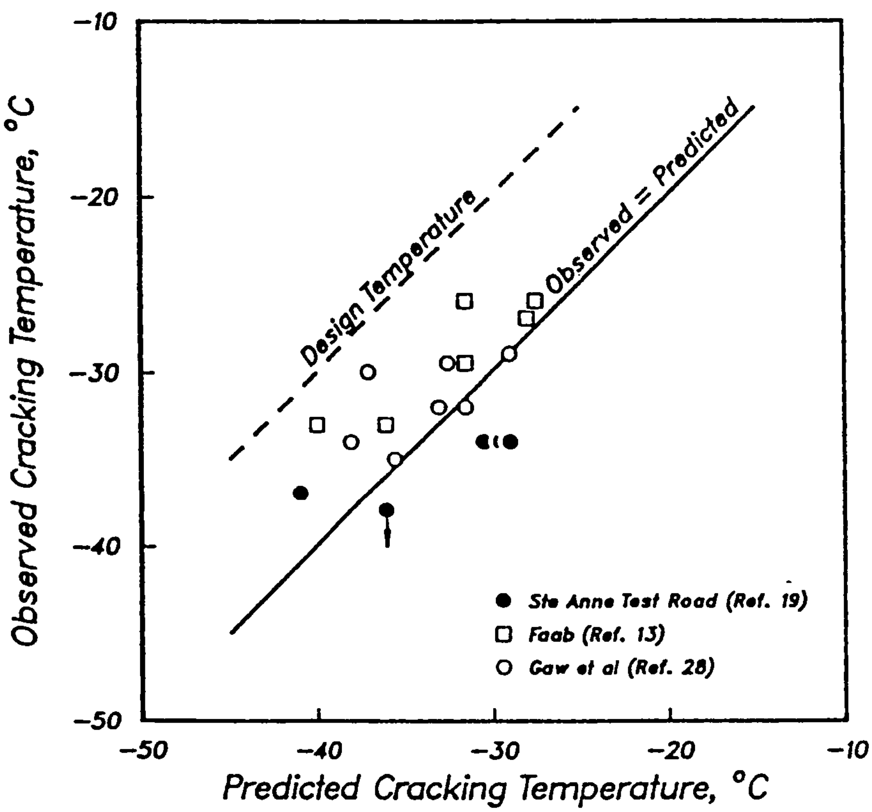

Figure 2.21 Observed vs. predicted cracking temperatures for asphalt paving mixes in Esso Study (Robertson, 1987).

For each set of field samples, the following asphalt cement samples were derived:

1. PAO - Virgin or original asphalt.
2. PAR - Thin-film oven test residue (ASTM D 1754).
3. PO - Laboratory-aged asphalt following pressure-oxidation (20 atm of oxygen @ 150°F for 46 hours).
4. PO5 - Same as above but aged for only five hours.
5. PM - Asphalt cement extracted and recovered from plant mix.
6. PC - Asphalt cement extracted and recovered from core samples taken right after compaction.
7. PC1 - Asphalt cement extracted and recovered from core samples taken after one year of service.
8. LM - Asphalt cement recovered from laboratory prepared hot mix following plant job mix formula using virgin aggregates and asphalt cement from the project.
9. L35 - Asphalt cement recovered from laboratory mix, compacted by 35-blow Marshall procedure and aged in oven at 140°F for 12 days. This procedure was developed to simulate in-service asphalt aging in pavements of high voids.
10. L75 - Asphalt cement recovered from laboratory mix, compacted by 75-blow Marshall procedure and aged in oven at 140°F for 12 days. This procedure was designed to simulate in-service asphalt aging in pavements of low void levels.

In the following discussion, these asphalt sample codes will be preceded by the project number identified in Table 2.20.

The following tests were performed on the above samples: Penetration at 5°C & 25°C (100 g, 5 sec), penetration at 4°C (200 g, 60 sec), viscosity at 25, 60 and 135°C, and ring-and-ball softening point tests were performed on original (PAO), TFOT aged residue (PAR) and pressure-oxidation aged asphalts (PO and PO5), as well as asphalts recovered from plant mixes (PM), core samples (PC and PC1) and laboratory mixed (LM), compacted and aged samples at two void levels (L35 and L75). From these data penetration ratio (PR), penetration index (PI), pen-vis number (PVN), viscosity temperature susceptibility (VTS), cracking temperature (CT), critical stiffness and critical stiffness temperature were calculated. Based on viscosity data at 25°C, shear index (SI, the slope of log shear stress versus log shear rate plot) were also determined.

Newtonian viscosities and elastic shear moduli of these samples were also determined at +5°C after cooling from +25°C, and after warming from a quenching temperature of -30°C. A modified cone and plate viscometer was used to measure rotational

Project	County	AC source	Pavement
1	Monona	AC-10 KOCH, Algona	surface, S[a]
	3/4" AGG.	70% gravel	
		30% crushed gravel	
2	Story	AC-20 KOCH, Tama	binder, P[b]
	3/4" AGG.	65% 3/4" crushed limestone	
		10% 3/8" chips	
		25% sand	
3	Dallas	AC-20 KOCH, Dubuque	surface, I[c]
	3/4" AGG.	50% 3/4" crushed gravel	
		35% 3/4" quartzite	
		15% concrete sand	
4	Grundy	AC-5 KOCH, Dubuque	base, S
	1/2" AGG.	70% 3/4" gravel	
		12% 3/4" crushed gravel	
		18% 1/2" crushed limestone	
5	Hardin	AC-5 KOCH, Dubuque	base, S
	3/4" AGG.	70% 3/4" gravel	
		30% 3/4" crushed limestone	
7	Webster	AC-5 KOCH, Algona	base, S
	3/4" AGG.	60% 3/4" crushed limestone	
		40% 3/4" gravel	
8	Plymouth	AC-5 KOCH, Algona	base, S
	3/4" AGG.	17% 3/4" wash rock	
		83% 3/4" pit run	
10	Harrison	AC-20 JEBRO, Sioux City	surface, P
	3/4" AGG.	35% 3/4" quartzite	
		14% concrete sand	
		51% 3/4" crushed rock	
11	Harrison	AC-10 KOCH, Algona	binder, P
	3/4" AGG.	30% 3/4" limestone	
		30% 3/8" limestone	
		40% crushed gravel	
12	Pottawattamie	AC-20 KOCH, Omaha, NE	binder, I
	3/4" AGG.	50% 3/4" stone	
		35% 3/8" stone	
		15% sand	

[a]Secondary road.
[b]Primary road.
[c]Interstate Highway.

Table 2.20. Summary of field samples in Iowa (Enustun et al., 1990).

displacements to 1/100th of a degree. The Newtonian viscosity is estimated from the slope of the linear asymptotic section of the rotation versus time plot, and the elastic shear modulus is estimated from its intercept.

Both high pressure gel permeation chromatography (HP-GPC) and nuclear magnetic resonance (NMR) tests were also run. HP-GPC chromatograms of 12 virgin asphalt samples, TFOT residues, 6 recovered core samples for 7-year old pavements and 72 asphalt samples from the 10 field projects were analyzed using 3, 4 and 8 slices. NMR studies on asphalts provide information on the average chemical functionality, eg., carbon and hydrogen aromaticity for asphalt characterization. Thermal analysis techniques were also used to determine the glass transition point to predict low-temperature cracking of asphalt pavements. Finally, water sensitivity and aging tests were performed. One-year old core samples were evaluated by measuring resilient modulus (RM) and indirect tensile strength (IRS) before and after an accelerated Lottman conditioning procedure. Three aging procedures, thin film oven test (TFOT), Iowa durability test (IDT) and mix aging, were used to study age hardening characteristics.

Results

The rheological properties (penetration, softening point, viscosity, shear susceptibility, and complex flow) for the asphalts from the 10 pavement projects are shown in Tables A.20 and A.21. The rheological properties of the TFOT residue (PAR) and the recovered asphalts after one year of field aging (PC1) show that:

1. Recovered asphalts from Projects 4, 5 and 10 aged more than TFOT residues in all properties.

2. Recovered asphalts from remaining projects aged less than TFOT residue in all properties except complex flow (CF) and shear index (SI).

3. From Table A.28, Projects 4, 5 and 10 also have the highest air voids (5.26%, 4.91%, and 7.19%) in their respective asphalt grade. This confirms previous research which indicates that air voids is a major factor affecting age hardening.

The temperature susceptibilities are shown in Table A.22. Low temperature cracking properties of the 79 asphalts from the 10 projects are summarized in Table A.23. These properties include cracking temperature (CT), temperature at a thermal cracking stress of 72.5 psi, temperature of equivalent asphalt stiffness at 20,000 psi at a loading time of 10,000 s (TES), estimated stiffness at -23°C and a loading time of 10,000 s (S23), and stiffness at -29°C and a loading time of 20,000 s (S29).

The researchers noted that there was variability in the asphalt grades tested and that laboratory aging increased the differences in some properties but decreased the properties in others. The results of viscoelastic measurements at 5°C (Table A.24) show striking differences between the responses of these samples to low temperature conditioning and the lapse of time. All samples exhibit an increase in viscosity at 5°C after cooling from 25°C to various extents. The trend is more pronounced with viscous asphalts.

From the HP-GPC results (which were too extensive to be included in this review), the following observations were noted:

1. LMS contents of the virgin asphalts ranged from 20.6 to 35.9% which is higher than that reported for Montana (16-17%).[15]

2. A weak correlation was observed between LMS content and temperature susceptibility which implies that higher LMS content increases the possibility of low temperature cracking.

3. HP-GPC may be used to monitor and predict aging as virgin asphalts show an increase of 1.2 to 14.4% in LMS after TFOT.

The glass transition temperatures (Tg) for the 79 samples are summarized in Table A.25. It was observed that:

1. The glass transition temperature of the original asphalts ranged from -34°C to -22.5°C, increasing with viscosity from AC-5 to AC-20.

2. In general, aging at high temperatures reduced Tg, Tsp, ML and MH,[16] while aging at low temperatures increased the thermal responses, i.e., different aging mechanisms resulted in different trends of thermal responses.

The results of the NMR analyses indicated that due to the nature of asphalt (a complex mixture of hundreds of thousands of different molecular structures), finding differences in the NMR spectra that could be used to characterize asphalts is problematic.

Table A.26 summarizes the effects of the Lottman accelerated moisture conditioning procedure. Cores (1 year old) from Project 11 showed the least resistance to moisture-induced damage.

Levels of aging due to the TFOT were compared with those due to actual construction. the rheological properties of TFOT residues (PAR), asphalt samples recovered from plant mixes (PM), cores right after construction (PC), and laboratory mixes (LM) were

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[15]See a companion report for more detailed review on HP-GPC, "HP-GPC and Asphalt Characterization - Literature Review," by M.T.Y. Yapp, A.Z. Durrani & F.N. Finn, TM-ARE-A-003A-90-02, Submitted to SHRP for publication, September 1990.

[16]
Tsp	=	The softening temperature at which the displacement of the probe reaches a maximum.
ML	=	A parameter associated with a thermogram and which measures low temperature thermal coefficient of expansion at the glassy state. It has been proposed as an index to predict the performance quality of an asphalt.
MH	=	Measures the coefficient of expansion after glass transition.

measured. In general, TFOT caused more hardening for soft asphalt (AC-5) than the harder asphalts (AC-10 or AC-20). For the AC-20, the TFOT showed similar hardening as hot mixing when the P5, P25, P4 and V25 properties are measured, but not in other properties. The order of aging for a project, in general, was PAO < PC1 < PO5 < PO in terms of rheological and HP-GPC properties. However, only 3 asphalts showed this trend when the thermomechanical analysis (TMA) parameters (Tg, Tsp, ML) were observed.

Regression analyses were then performed to relate physical parameters, TMA parameters, and HP-GPC parameters. Table A.27 summarizes the results of these analyses. Finally, Table 2.21 is a proposed trial specification for asphalt cements.

Conclusions

The researchers (Enüstün et al., 1990) concluded in this study that:

1. Hardening occurs in two stages: hardening during short periods in the mixer at higher temperatures and rates, and hardening during longer periods of road service in pavement at relatively lower temperatures and rates.

2. There were differences in temperature susceptibility between the samples supplied by different suppliers and between samples from the same supplier over time.

3. Distinctly different GPC chromatograms, TA results and X-ray patterns were obtained among asphalts of the same grade and supplier but supplied at different times.

4. The strikingly different effect of a cold shock (-30°) on the viscoelastic properties of the core sample from the surface course of the Sugar Creek project from the other samples might have an important bearing on its poor field performance.

5. Elastic shear modulus measured at a low temperature may be correlated to low temperature field performance.

6. In contrast to thermal analytic behavior and X-ray diffraction spectra, LMS rating is found to be conclusively and unidirectionally sensitive to aging and when analyzed over the entire spectra of molecular size distribution by the 8-slice method, can be used to predict behavior and performance of asphalts. However, for specification purposes, both original and lab-aged asphalts must be tested.

7. Asphalts used in the 1988 construction season from a limited number of sources in Iowa showed differences not obvious by either physical or physicochemical tests alone. For example, the asphalt used in Project 7 had a large percent increase in LMS due to aging, but this is not reflected by changes in physical properties, eg, viscosity ratio. On the other hand, Project No. 11 had a high viscosity ratio after TFOT aging, but this was not reflected in an increase in LMS.

8. Aging, both in the field and in the lab, is accompanied by hardening, reduction in temperature susceptibility by most measures, an increase in shear susceptibility, decrease in complex flow, increase in temperature for limiting stiffness, increase in stiffness at low temperatures, increase in LMS and a decrease in SMS. For some asphalts, aging characteristics during high temperatures (short-term) and service temperatures (long-term) were very different.

9. The glass transition points determined by TMA are in general agreement with those determined by DSC, and correlate fairly well with low-temperature cracking properties.

10. Both TMA and HP-GPC parameters correlated well with physical properties. Tsp correlates well with both rheological and low-temperature properties, Tg correlates well with low-temperature properties and ML is a strong predictor of rheological properties. Molecular size distribution based on HP-GPC and the 8-slice method can be used to predict many of the physical properties.

11. While TMA parameters and HP-GPC parameters did not correlate well, physical and low-temperature properties can be predicted by combinations of these two sets of parameters, especially using Tsp, ML, X2 and X7.

12. The relative significance of the more than 30 physicochemical parameters in predicting the field performance can only be established through correlation with field performance data. It is possible that the predictive equation must contain both physical and physicochemical parameters.

Test	AC-5	AC-10	AC-20
Original Asphalt:			
*Viscosity @ 60 C, poises**	*500+/-100*	*1000+/-200*	*2000+/-400*
*Viscosity @ 135 C, cSt, min.**	*175*	*250*	*300*
*Penetration, 25 C, min.**	*140*	*80*	*60*
*Flash point, C, min.**	*177*	*219*	*232*
*Solubility in trichlo, % min.**	*99.0*	*99.0*	*99.0*
Residue from TFOT:			
*Viscosity, 60 C, p.,max.**	*2000*	*4000*	*8,000*
Residue from pressure-oxidation, 46 hrs.@ 150 F:			
Viscosity at 60 C, poises, max.	10,000	20,000	40,000
Penetration, 25/100/5, min.	20	20	20
Penetration, 4/200/60, min.	5	5	5
Penetration, 5/100/5, min.	10	8	7
Softening point, F, max.	160	160	160
Stiffness,-23 C, 10,000 sec., psi	20,000	20,000	20,000
Viscosity, 25 C, megapoises, max.	20	20	20
Shear susceptibility, max.	0.55	0.55	0.55
X2(HP-GPC), %, max.	20	20	20
X7(HP-GPC), %, min.	5	5	5
Tg(TMA), C, max.	-10	-10	-10
Tsp(TMA), C, max.	28	28	28
ML(TMA), max.	0.4	0.4	0.4

**AASHTO M226*

Table 2.21 Proposed trial specification for asphalt cement in Iowa study (Enüstün et al., 1990).

3

Conclusions

This survey of state and industry research reports 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 <u>all</u> 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

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. Table 3.1 summarizes the kinds of information collected in these reports and Table 3.2 describes in greater detail the kinds of tests performed as well as the performance parameters investigated.

DATA FROM / STATES REPORTING	PERFORMANCE FOUND RELATED TO					
	CONSTRUCTON VARIABLES		PHYSICAL TESTS		ASPHALT COMPOSITION	
	FIELD	LAB	FIELD	LAB	FIELD	LAB
ALBERTA	YES	—	YES	YES	—	—
ALBERTA	—	—	YES	YES	—	—
COLORADO	YES	—	SOME	—	SOME	—
IOWA	YES	—	YES	—	—	—
SOUTH DAKOTA	SOME	—	—	—	—	—
ALASKA	NO	—	SOME	—	—	—
SOUTH CAROLINA	SOME	—	YES	YES	—	—
MICHIGAN	—	—	YES	—	—	—
ESSO ONTARIO	—	—	—	YES	—	—

Table 3.1 Studies relating asphalt composition to pavement performance.

Table 3.2 Summary of State Reports

STATE	PERFORMANCE PARAMETERS INVESTIGATED	TESTS DONE FOR SIMULATING AND/OR DETECTING CHANGES IN ASPHALT PERFORMANCE	ASPHALT AND/OR AC PROPERTIES MEASURED: PHYSICAL	CHEMICAL	ASPHALT-AGGREGATE	NEW DEVELOPMENT	OVERALL SYNOPSIS
Michigan (Defoe, 1988)	Low Temperature Transverse Cracking and Rutting	None	Penetration, dmm, 25C, 100g, 20s Penetration, dmm, 4C, 200g, 60s Absolute viscosity, 140°F, Poises Kinematic viscosity, 275°F, cS	None	Asphalt content Air voids Aggregate gradation Thermal coefficient	None	Relationships reported between cracking & failure stiffness, cracking & the ratio of recovered penetration, and cracking and temperature susceptibility of asphalts.
Iowa (Marks, 1984)	Low Temperature Transverse Cracking	Eight research sections, about 2000' ea. in length were constructed to monitor low temperature transverse cracking.	Penetration, dmm, 77F Viscosity, poises, 140F	None	None	None	No transverse cracks were found in the transverse jointed pavement. But caution is warranted in extrapolating these results. PVN is an effective measure of temperature susceptibility of asphalts.
South Dakota (Crawford & Anderson, 1976)	Low Temperature Transverse Cracking	None	Penetration Viscosity Microviscosity	None	Asphalt content	None	Use of limestone aggregate showed some improvement in the cracking performance of relatively hard asphalts. However, no reasons were found for this improvement.
Alberta (Palsat, 1986)	Low Temperature Cracking	TFOT	Penetration, 4°C, 200g, 60s, (Recovered) Penetration, 4C, 100g, 5s, (Recovered) Penetration, 25C, 100g, 5s, (Orig. asphalt & recovered, after TFOT) Absolute viscosity, 60°C Kinematic viscosity, 135°C	None	Percent compaction	None	Crack prediction models developed; most of which utilize original asphalt stiffness as a predictor variable. Palsat (1986) suggests a critical original asphalt stiffness of $2^{9} \times 10^{8}$ Pa(30 kg/cm^{2}) as an upper limit for acceptable field cracking performance.
South Carolina (Busching et al., 1986)	Stripping Cracking Bleeding	Indirect tensile tests on saturated specimen	None	None	Maximum (Rice) specific gravity Bulk specific gravity Voids in mineral aggregate Asphalt content	Moisture susceptibility testing based on indirect tensile strength test on moisture conditioned samples.	A procedure is presented to incorporate moisture susceptibility (measured using both the indirect tensile strength and the tensile strength ratio) testing into the Marshall mix design method.
Colorado (O'Connor, 1979)	Rutting & bleeding	None	Penetration, dmm, 77F Viscosity, 140 & 275F (original & recovered) Resilient modulus	Saturates Naphthene Aromatics Polar Aromatics Asphaltenes	Asphalt content Avg. % #200 sieve Design & production voids Design & production stability Plant type	None	Bleeding in wheelpath generally occurs between 2.5 and 3.0% voids. Void content in wheelpaths can be generally expected to be about one percent lower than the design voids.

Table 3.2 Summary of State Reports (continued)

STATE	PERFORMANCE PARAMETERS INVESTIGATED	TESTS DONE FOR SIMULATING AND/OR DETECTING CHANGES IN ASPHALT PERFORMANCE	ASPHALT AND/OR AC PROPERTIES MEASURED			NEW DEVELOPMENT	OVERALL SYNOPSIS
			PHYSICAL	CHEMICAL	ASPHALT-AGGREGATE		
Alberta (McMillan, 1988)	Permanent deformation (Rutting)	Repeated load triaxial test TFOT	Penetration, dmm, 25C, 5s Absolute Viscosity, Pa.s, 0C Kinematic Viscosity, mm^2/s_2, 135C (before & after TFOT) specific gravity stiffness, kPa, • of virgin binder, 25 35, 45C. • of recovered binder (using Abson recovery) @ 25, 35, 45C.	None	Asphalt content(design) Marshall stability Marshall flow Air voids (design and measured) Design VMA	Repeated load triaxial test showed promise in simulating rutting.	A number of models developed for predicting permanent deformation, most of which use the stiffness of recovered binder as a predictor variable.
Alaska (McHattie, 1981)	Aging	None	Absolute Viscosity, poises, 140F Penetration,dmm, 77F Penetration, dmm, 39.2F	None	Percent voids (cores) Asphalt content (cores) % #200 aggregate (cores) % #40 aggregate (cores) In-place density	None	McHattie concluded that due to the non-linearity of the asphalt cement aging curves, valid performance assumptions could not necessarily be based on original properties.
ESSO (Robertson, 1987)	Low Temperature Transverse Cracking	None	None	None	None	Rational design chart to select asphalts for low temperature service.	Based on minimum air temperature, location of mix in the pavement structure, and given penetration limit for a grade of asphalt, a minimum acceptable penetration index can be arrived at. An asphalt for good low temperature service can be selected based on this minimum acceptable penetration index.

Before discussing the overall results from the review of the literature, it is important to note some of the problems which adversely affected the reviewers' ability to develop any consensus for the review of the enclosed reports.

1. Confounding factors with regards to structural section, asphalts, aggregates, traffic, test methods and performance measurements. For example, while Palsat (1986) was able to develop regression equations relating transverse cracking to pavement properties, he does not indicate the type of aggregate used in the mix. In the South Dakota study, Crawford & Anderson (1968) indicated that the use of limestone aggregate vs. crushed gravel affected the pavement performance with regards to transverse cracking. Another example is the work performed by Busching et al. (1986) in South Carolina where the causes of stripping were studied. Their work did not include the effects of traffic which Davis (1986) noted was an important factor in stripping failures.

2. The majority of these projects evaluated asphalt properties using traditional tests such as penetration, viscosity and Marshall or Hveem stability. 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.

Qualitative Relationships

Recognizing that problems do exist in attempting to interpret information from these reports as discussed earlier, a cautious effort has been made to extract the following qualitative relationships.

Stripping

High air voids contents as a result of poor compaction, use of hot plant mixed seal courses, or open graded friction courses could be possible causes for increased stripping of asphalt concrete pavements. However, the effects of high truck tire pressures could also be a factor, as well as drainage.

Low Temperature Transverse Cracking

An increase in low temperature transverse cracking could be associated with an increase in:

- Asphalt and/or asphalt-aggregate system stiffness;
- Ratio of the recovered penetration;
- Air voids in mixtures;
- Thinner pavements; and
- Use of asphalts with high temperature susceptibilities.

Rutting and Bleeding

Over-asphalting, lower than design air voids in the wheel paths increase the chances of bleeding. It appears that rutting does not correlate well with the commonly measured asphalt rheological properties. The reviewed reports indicate that lack of shear strength in the asphalt concrete is the primary cause of rutting.

Quantitative Relationships

Table 3.3 summarizes the most broadly applicable performance models reported in the reviewed reports. Some of the trends in these performance models go against established norms. A good example is the relationship reported by Palsat (1986):

$$CPK = 153.28 + 2.65*AGE + 0.40*OAS - 2.37*COMP \qquad R^2 = 0.60$$

where: CPK = Cracks per kilometer

AGE = Pavement age, years
OAS = Original asphalt stiffness, kg/cm^2
COMP = Compaction, %

The negative sign for percent compaction indicates an inverse relationship between cracking frequency and percent compaction. This conflicts with Van der Poel's (1955) relationship between asphalt and pavement stiffness, i.e., for a given asphalt content, an increase in the pavement density results in an increase in the pavement stiffness which would result in an increase in the potential for low temperature cracking.

This equation is based on laboratory measurements of original asphalt properties. The data available in this study (Palsat, 1986) is unique in that it is separated into full-depth asphalt sections and granular base sections. An explanation about the performance data and laboratory data (based on original asphalt properties) collected by Palsat (1986) is presented in Section 2.4. During the review it was decided that a further analysis of this data was performed.

As a result of this elementary analysis, the following conclusion can be drawn:

a) For full-depth asphalt sections, only one original asphalt property, namely the asphalt stiffness, appears to be useful in predicting asphalt cracking. This result is in agreement with Palsat's (1986) equation for predicting cracking frequency.

b) For the granular base sections analyzed, penetration at 77°F, viscosity at 140°F after TFOT, and the original asphalt stiffness appear to be good predictors of cracking performance.

These conclusions are based on original asphalt properties and do not lend themselves to a general comparison with the data presented in some of the other state reports. Thus, for this review, these results remain isolated to this particular study.

Cracking is a major part of the pavement performance in Alaska (McHattie, 1981). McHattie's (1981) data provides acceptable values for properties of asphalts used in the

PERFORMANCE VARIABLE	RELATIONSHIP REPORTED	R^2	STATE REPORTING
Low Temperature Transverse Cracking	CI = -33.366 + 2.040 * E	0.82	Michigan[1] (Defoe, 1988)
	CI = 48.992 - 0.567 * RP	0.50	Michigan[1] (Defoe, 1988)
	CI = -17.061 - 17.649 * PI	0.63	Michigan[1] (Defoe, 1988)
	CPK = 153.28 + 2.65 * AGE + 0.40 * OAS - 2.37 * COMP	0.60	Alberta (Palsat, 1986)
Rutting	LOG ($\mathcal{E}$) = 0.9521 + 0.581 * LOG* (N) * LOG (ABST)	0.87	Alberta (McMillan, 1989)

CI = Cracking Index
E = Failure stiffness, psi
RP = Recovered Penetration Ratio, %, Wearing/Top Course only
PI = Penetration index calculated using modified Heukelom Method
CPK = Cracks per kilometer
AGE = Pavement age, years
OAS = Original Asphalt Stiffness, kg/cm^2
T = Pavement Thickness, mm
COMP = Compaction, %
$\mathcal{E}$ = Permanent strain, %
N = Number of load applications
ABST = Calculated Stiffness of Abson recovered binder, kPa
1 = Developed by ARE using reported data

Table 3.3. Pavement performance prediction models reported.

Alaska environment (Table 2.16). These values were compared with the Sisko and Brunstrum (1968, 1969) data analyzed by Finn et al. (1990). The relation reported by Finn et al., between Penetration (77°F), dmm and the crack rating (1 = Low cracking, 5 = severely cracked, Scale: 1-5) is:

$$\text{Crack Rating} = 6.3967 - 0.0993 * \text{Penetration, 77°F, dmm} \qquad (R^2 = 0.655)$$

McHattie's acceptable value for Penetration (77°F) is 51 ± 21 dmm, which yield a range of crack rating of 0-3.4 and the mean (Pen = 51 dmm) is 1.3. The acceptable range of crack rating is 2-3 (Finn et al., 1990). Similarly the relationship between Viscosity at 140°F, poises, and the crack rating reported by Finn et al., is:

$$\text{Crack Rating} = 0.0703 * \text{Viscosity (140°F, poise)} \quad 0.3751 \qquad (R^2 = 0.596)$$

Using the values from Table 2.16, (4656 ± 3461 poises) the range of cracking rating is 1.0-2.1, and a mean (Vis140 = 4656 poises) crack rating of 1.7. Thus, McHattie's (1981) reported values compare favorably with the Sisko & Brunstrum data[17] (1968, 1969).

The Sisko and Brunstrum data was again used as a baseline for comparing the asphalt chemical composition data reported by O'Connor (1979). The relationship developed by Finn et al. using the Sisko and Brunstrum data is:

$$\text{Rut Depth rating} = 6.2102 = 0.2104 * \text{Asphaltenes, \%} \qquad (R^2 = 0.712)$$

The section showing the best rutting performance (O'Connor, 1979) had 19 percent asphaltenes. This translates to a rut depth rating[18] of 2.4 using the above relationship. This indicates an agreement with the acceptable limits for rutting performance provided by Finn et al. (1990).

An attempt was made to test the applicability of the results reported in one report using data from other reports. For example, the recovered penetration ratios were calculated using the data reported by Palsat (1986) and the Cracking Indices calculated using the relationship reported by Defoe (1988). These indices were then compared to the cracking data collected by Palsat (1986). The predicted cracking performance did not match the actual field data. Hence, after a few other such attempts, no further investigations in this direction were conducted.

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[17] For further reading on analysis of the Sisko & Brunstrum (1968, 1969) data the reader is referred to Finn et al., 1990. Also refer to Tables A.28 and A.29.

[18] This rating system is analogous to the crack rating system mentioned earlier; therefore a 5 is the highest crack rating and 1 is the lowest. With 2 through 3 being the acceptable rut depth rating range.

Performance-Based Specifications

Stripping

No new specifications were reported for controlling stripping. However, Busching et al. (1986) suggest the incorporation of moisture susceptibility testing into the Marshall mix design method.

Low Temperature Transverse Cracking

Palsat (1986) recommends using a critical asphalt stiffness of 2.9×10^6 Pa (30 kg/cm) as a upper limit for improved cracking performance. This value was predicted using McLeod's method and based on original asphalt characteristics and site specific temperature (at a pavement depth of 50 mm) conditions.

Crawford and Anderson (1976) suggest using a softer asphalt with 100% quarried limestone aggregate to improve resistance to transverse cracking.

Marks' (1984) research indicates the beneficial effects of sawed transverse joints to reduce low temperature transverse cracking. He does not report a critical joint spacing to prevent low temperature transverse cracking and is cautious in extrapolating the results of his study.

Robertson (1987) developed a design chart for selecting asphalt cements to resist low temperature cracking. This chart uses measured values of asphalt stiffness and a stress calculation method which takes into account the variation in relaxation times during the cooling period. This chart gives estimates of thermal cracking temperatures which can be compared to the low temperatures in the area where the asphalts are to be used. Thus, an appropriate asphalt can be selected to reduce low temperature transverse cracking.

Rutting and Bleeding

No new specifications are reported for controlling bleeding and rutting of asphalt pavements.

In conclusion, two points should be mentioned:

1. Construction practices influence the in-service rheological properties of the asphalt-concrete systems. For example, if there is a large deviation in field air voids from the specified air voids, these higher air voids (as some investigators showed) result in increased hardening and hence greater susceptibility to cracking. In another case, the higher air voids content showed an increased susceptibility to stripping. These deviations overshadow the actual asphalt properties being studied, resulting in a large scatter in data: the end result is the lack of available correlation between asphalt-system rheology and field performance.

2. Most of these studies, with the exception of Colorado (O'Connor, 1979), did not attempt any asphalt compositional studies (see Table 3.1). Fractional separation techniques could provide a method for following changes in an asphalt. An attempt has been made to present the various tests currently in use for determining asphalt composition (see Appendix B). State-of-the-art work has been done in developing this analytical field; what remains to be studied is the availability of associated performance data.

4

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Appendix A

This appendix contains tables and figures referred to in Chapters 2.0 and 3.0.

Table A.1. Independent variables for all full depth sections in Alberta study (Palsat, 1986).

SECTION	ACP THICKNESS (MM)	AGE (YEARS)	TESTS ON ORIGINAL ASPHALT: PEN@25C 100G 5S (DMM)	PEN@4C 200G 60S (DMM)	VISC@60C (PA.S)	PVN	PEN RATIO	TESTS ON RESIDUE AFTER TFOT: PEN@25C 100G 5S (DMM)	PEN@4C 200G 60S (DMM)	VISC@60C (PA.S)	PVN	PEN RATIO	ASPHALT STIFFNESS (KG/CM)	FREEZING INDEX C-DAYS	MINIMUM TEMP C
1:10-1	255	11	243	68	291	-.67	27	96	35	1071	-.76	36	30	1111	-37
1:10-2	255	16	277	80	318	-.31	28	149	48	669	-.55	32	9	1111	-37
1:10-3	225	14	256	68	281	-.62	26	127	41	670	-.82	32	25	1111	-37
1:12-1	255	11	190	53	600	-.24	27	112	37	1195	-.39	33	28	1111	-37
1:12-2	255	11	318	100	299	-.11	31	182	61	586	-.35	33	7	1111	-37
1A:06	175	13	267	99	454	.08	37	152	55	1054	.01	36	7	1000	-36
1A:08	200	13	256	62	272	-.66	24	119	35	671	-.93	29	20	1000	-37
12:08	255	9	269	103	434	.03	38	131	51	1193	-.12	38	11	1333	-38
14:06	250	11	270	70	303	-.42	25	156	42	579	-.64	26	25	1500	-39
2:12-1	300	7	182	56	620	-.28	30	104	38	1350	-.38	36	30	1055	-36
2:64	175	14	251	62	277	-.70	24	117	34	654	-.97	29	190	1666	-46
2A:06	225	11	275	106	441	.01	38	142	57	1108	-.06	40	7	1055	-36
20:02-1	125	15	258	58	284	-.59	22	153	37	496	-.85	24	30	1333	-38
20:02-2	125	14	271	101	438	.05	37	145	55	1044	-.09	37	10	1333	-38
21A:10	200	10	264	100	477	.12	37	143	57	1114	-.04	39	13	1444	-39
22:16	175	12	291	90	336	-.14	30	160	53	701	-.37	33	8	1055	-36
3:02	250	7	182	56	620	-.28	30	104	38	1350	-.38	36	30	777	-37
3:12-1	225	8	168	53	839	-.07	31	95	37	1916	-.15	38	50	1111	-38
3:12-2	225	8	168	53	839	-.07	31	95	37	1916	-.15	38	50	1111	-38
35:02-1	175	7	174	53	480	-.67	30	88	33	1380	-.63	37	90	944	-38
36:02-2	175	7	196	70	630	-.13	35	99		1760	-.17	0	25	944	-38
501:06-1	125	12	270	101	450	.09	37	144	56	1081	-.06	38	8	1166	-37
501:06-2	125	10	314	97	309	-.09	30	165	55	677	-.36	33	7	1166	-37
512:02	175	9	295	92	345	-.07	31	170	57	721	-.23	33	8	833	-38
519:02	175	7	173	51	480	-.68	29	83	33	1450	-.67	39	90	833	-38
519:04-1	125	15	285	87	332	-.19	30	165	53	649	-.41	32	13	833	-38
526:02	150	8	162	59	876	-.09	36	93	36	2024	-.13	38	60	888	-38
53:12	200	9	266	101	459	.08	37	139	58	1111	-.09	41	16	1611	-39
543:02	175	7	349	105	176	-.62	30	171	55	514	-.62	32	7	1000	-37
544:02	175	10	272	104	437	.07	38	150	56	1023	-.05	37	15	1166	-39
583:02	165	8	253	92	431	-.02	34	139	53	1046	-.16	38	12	1222	-38
590:02-1	150	12	271	104	429	.04	38	143	57	1089	-.07	39	12	1333	-38
590:02-2	150	11	328	104	275	-.15	31	196	64	541	-.31	32	7	1333	-38
655:02	180	7	178	42	432	-.77	23	109	29	870	-.79	26	150	1388	-41
661:02-1	175	7	285	92	358	-.10	32	148	49	863	-.27	33	25	1444	-42
661:02-2	175	7	285	92	358	-.10	32	148	49	863	-.27	33	25	1444	-42
661:06	230	7	301	96	336	-.07	31	155	55	773	-.32	35	15	1500	-41
684:02-1	175	8	288	96	345	-.12	33	152	53	812	-.29	34	60	1722	-45
72:10-2	125	15	248	55	290	-.64	22	150	37	502	-.87	24	28	1111	-37
743:02	175	7	285	98	340	-.16	34	159	54	790	-.24	33	70	1777	-46
769:02	150	10	302	107	355	.01	35	156	53	922	-.10	33	11	1444	-41
771:06	150	11	270	69	303	-.42	25	152	41	612	-.62	26	19	1388	-38
805:02-1	100	11	324	98	297	-.08	30	180	62	599	-.35	34	10	1277	-38
805:02-2	150	11	324	98	297	-.08	30	180	62	599	-.35	34	10	1277	-38
806:02	125	14	251	71	262	-.74	28	123	42	609	-.98	34	30	1222	-38
813:02	150	10	287	96	369	-.04	33	155	54	855	-.20	34	20	1666	-42
817:04	175	7	175	52	470	-.68	29	87	29	1360	-.65	33	90	1111	-38
842:02	100	8	172	71	787	-.10	41	96	36	2025	-.08	37	45	888	-38
855:08	175	13	269	98	448	.08	36	145	55	1050	-.08	37	15	1611	-39
857:04-1	175	7	258	104	440	-.03	40	141	54	1120	-.06	38	19	1611	-40
857:04-2	175	7	258	104	440	-.03	40	141	54	1120	-.06	38	19	1611	-40
872:04	150	13	273	101	431	.06	36	144	54	1058	-.09	37	15	1555	-39
879:04	100	13	255	97	455	.06	36	141	54	1085	-.10	38	10	1111	-38
881:12	175	8	261	90	450	.02	34	135	60	1135	-.12	44	20	1722	-41
887:04	175	8	262	86	433	-.02	32	125	45	1194	-.20	35	11	1166	-38

Table A.2. Transverse cracking data for all full depth sections in Alberta study (Palsat, 1986).

SECTION	NO. OF KILOMETRES WITH INDICATED FREQUENCY 0	1	2	3	4	5	5+	TOTAL (KM)	FREQUENCY (CR/KM)
1:10-1	2	1	2	1	0	1	0	7	1.8
1:10-2	1	1	2	0	0	0	2	6	4.7
1:10-3	1	2	1	0	0	0	4	8	4.8
1:12-1	0	0	0	0	0	0	7	7	45.0
1:12-2	5	0	1	1	1	0	0	8	1.3
1A:06	6	6	2	4	1	1	0	20	1.6
1A:08	1	3	0	0	2	0	3	9	3.9
12:08	1	1	0	0	0	2	0	4	2.8
14:06	0	0	0	0	0	0	9	9	11.0
2:12-1	7	1	0	0	0	1	0	9	.7
2:64	0	0	0	0	0	0	13	13	92.0
2A:06	0	2	1	0	0	0	0	3	1.3
20:02-1	0	0	0	0	0	0	5	5	53.0
20:02-2	0	0	0	0	0	0	11	11	26.0
21A:10	4	1	0	0	0	0	0	5	.2
22:16	0	0	0	0	0	1	11	12	11.0
3:02	7	0	0	0	0	0	0	7	.0
3:12-1	4	0	2	0	2	1	1	10	2.6
3:12-2	0	0	0	0	0	0	10	10	12.0
36:02-1	0	0	0	0	0	0	10	10	33.0
36:02-2	1	3	0	1	1	3	4	13	5.8
501:06-1	0	0	0	0	0	0	17	17	29.0
501:06-2	0	0	0	0	0	0	15	15	40.0
512:02	10	0	0	0	0	0	0	10	.0
519:02	0	0	0	1	1	0	17	19	29.0
519:04-1	0	0	0	0	0	0	3	3	74.0
526:02	0	0	0	0	0	0	16	16	18.0
53:12	5	0	0	0	0	0	0	5	.0
543:02	12	1	0	0	0	0	0	13	.1
544:02	1	2	0	1	0	2	5	11	4.8
583:02	12	1	0	0	0	0	0	13	.1
590:02-1	0	0	0	0	0	1	16	17	34.0
590:02-2	0	0	0	0	0	0	5	5	27.0
655:02	0	0	0	0	0	0	10	10	50.0
661:02-1	0	0	0	0	0	0	3	3	30.0
661:02-2	7	2	0	1	0	0	0	10	.7
661:06	16	0	0	0	0	0	0	16	.0
684:02-1	0	0	0	0	0	0	14	14	26.0
72:10-2	0	0	0	0	0	0	18	18	34.0
743:02	4	0	0	0	1	1	3	9	4.0
769:02	25	0	2	0	0	0	0	27	.1
771:06	11	0	0	0	0	0	0	11	.0
805:02-1	0	0	0	0	0	0	17	17	25.0
805:02-2	0	0	2	1	0	4	9	16	8.4
806:02	0	0	0	0	0	0	9	9	42.0
813:02	0	0	0	0	0	0	6	6	43.0
817:04	0	0	0	0	0	0	12	12	40.0
842:02	0	0	0	0	0	0	4	4	15.0
855:08	0	0	0	0	2	0	18	20	20.0
857:04-1	6	1	3	0	2	1	2	15	2.8
857:04-2	0	0	0	0	0	0	2	2	13.0
872:04	0	0	0	0	0	0	17	17	32.0
879:04	0	0	0	0	0	0	41	41	63.0
881:12	7	4	1	1	1	1	1	16	1.5
887:04	0	0	0	0	2	0	17	19	17.0

Table A.3. Independent variables for all granular base course sections in Alberta study (Palsat, 1986).

SECTION	ACP THICKNESS (MM)	AGE (YEARS)	TESTS ON ORIGINAL ASPHALT PEN@25C 100G 5S (DMM)	PEN@4C 200G 60S (DMM)	VISC@60C (PA.S)	PVN	PEN RATIO	TESTS ON RESIDUE AFTER TFOT PEN@25C 100G 5S (DMM)	PEN@4C 200G 60S (DMM)	VISC@60C (PA.S)	PVN	PEN RATIO	ASPHALT STIFFNESS (KG/CM)	FREEZING INDEX C-DAYS	MINIMUM TEMP C
11:06	100	8	288	94	349	-.11	32	151	52	825	-.29	34	9	1166	-37
11:06&08	100	7	267	89	390	-.12	33	134	59	960	-.33	44	12	1166	-37
11:08	100	10	269	91	350	-.24	33	149	50	826	-.31	33	10	1222	-37
12:20-1	100	7	265	103	450	.05	38	135	56	1140	-.12	41	15	1611	-39
12:20-2	125	8	182	58	699	-.14	31	96	36	1799	-.20	37	50	1611	-39
16:06	150	8	170	72	781	-.14	42	82	32	2383	-.17	39	70	1277	-41
33:10-1	100	11	265	100	460	.08	37	141	57	1118	-.06	40	27	1444	-43
33:10-2	100	11	265	100	460	.08	37	141	57	1118	-.06	40	27	1444	-43
35:16-1	100	7	301	99	340	-.05	32	156	55	810	-.25	35	65	2777	-47
35:16-3	100	7	301	99	340	-.05	32	156	55	810	-.25	35	65	2777	-47
36:06	100	7	164	53	730	-.28	32	86	32	1820	-.37	37	40	1055	-39
36:10	125	7	300	102	330	-.10	33	157	54	800	-.25	34	13	1333	-39
40:12	100	7	268	85	320	-.36	31	154	56	750	-.36	36	7	944	-35
45:08	65	8	284	109	404	.05	38	150	65	996	-.08	43	14	1666	-41
531:02	100	8	165	64	845	-.10	38	99	42	1878	-.10	42	60	888	-38
663:08	125	8	295	109	340	-.09	36	165	65	754	-.23	39	15	1722	-41
668:02	200	8	255	89	453	-.02	34	120	44	1325	-.16	36	50	1444	-44
67:18	175	7	291	97	350	-.08	33	156	56	770	-.31	35	70	2500	-47
750:02	150	8	275	90	428	.06	32	133	53	1189	-.10	39	55	1666	-45
770:06	150	8	299	94	327	-.12	31	168	61	735	-.23	36	11	1388	-39
776:02	125	8	265	90	441	.02	33	140	52	1073	-.12	37	20	1444	-41
794:02-1	175	7	167	55	750	-.22	32	96	36	1820	-.19	37	60	1444	-40

SECTION	ASPHALT STIFFNESS (KG/CM)	FREEZING INDEX (C-DAYS)	MINIMUM TEMP (C)	GBC THICKNESS (MM)
11:06	9	1167	-37	300
11:06&08	12	1167	-37	300
11:08	10	1222	-37	300
12:20-1	15	1611	-39	125
12:20-2	50	1611	-39	100
16:06	70	1278	-41	450
33:10-1	27	1444	-43	275
33:10-2	27	1444	-43	275
35:16-1	65	2788	-47	350
35:16-3	65	2788	-47	250
36:06	40	1056	-39	250
36:10	13	1333	-39	150
40:12	7	944	-35	150
45:08	14	1677	-41	300
531:02	60	889	-38	100
663:08	15	1722	-41	125
668:02	50	1444	-44	125
67:18	70	2500	-47	100
750:02	55	1667	-45	200
770:06	11	1389	-39	200
776:02	20	1444	-41	150
794:02-1	60	1444	-40	100

Table A.4. Transverse cracking data for all GBC sections in Alberta study (Palsat, 1986).

SECTION	NO. OF KILOMETRES WITH INDICATED FREQUENCY 0	1	2	3	4	5	5+	TOTAL (KM)	FREQUENCY (CR/KM)
11:06	5	3	3	1	2	2	6	22	5.1
11:06&08	6	5	5	2	1	1	1	21	1.9
11:08	4	2	8	3	1	3	5	26	3.0
12:20-1	2	3	1	0	0	1	7	14	5.6
12:20-2	0	0	0	0	0	0	16	16	43.0
16:06	0	0	0	0	0	0	5	5	7.4
33:10-1	2	1	0	0	0	0	9	12	9.7
33:10-2	12	6	1	2	0	0	0	21	.2
35:16-1	0	0	0	0	0	0	6	6	35.0
35:16-3	0	0	0	0	0	0	15	15	28.0
36:06	0	0	0	0	0	0	20	20	25.0
36:10	0	0	0	0	2	2	16	20	12.0
40:12	12	6	1	4	1	0	3	27	1.7
45:08	1	4	4	3	0	0	5	19	3.4
531:02	0	0	1	0	1	0	16	18	17.0
663:08	9	1	1	0	0	0	4	15	3.1
668:02	0	0	0	0	0	0	3	3	24.0
67:18	0	0	0	0	0	0	20	20	39.0
750:02	14	2	0	0	0	0	0	16	.1
770:06	3	1	0	1	0	0	1	6	2.0
776:02	13	0	0	1	1	0	0	15	.5
794:02-1	3	0	2	3	1	2	1	12	2.8

Table A.5. Historical data for field test sites (Sites 1 to 20) in Alberta study (Source: McMillan, 1989).

SITE	PROJECT	STRUCTURE			ESAL's	ORIGINAL ASPHALT		CLIMATIC	AGE
		ASPHALT CONCRETE	ASBC	BASE	(X1000)	PENETRATION (dmm,25C,5s)	ABSOLUTE VISCOSITY (Pa.s)	VALUE (°C)	(YEARS)
1	22:28	100mm (200-300)	yes	250mm GBC	89.4	263	39.8	28	5
2	1A:02	100mm (200-300)	no	150mm GBC	674	168	67.0	27	7
3	2A:06	200mm (200-300)	no	Fulldepth	133*	317	30.1	28	12
4	24:02	50mm (200-300)	yes	175mm GBC	551*	266	32.9	29	17
5	24:02	50mm (200-300)	yes	305mm GBC	551*	266	32.9	29	17
6	21:12	50mm (200-300)	yes	150mm CS	365*	285	30.0	29	17
7	21:12	75mm (200-300)	yes	150mm CS	365*	285	30.0	29	17
8	1:10	250mm (200-300)	no	Fulldepth	1710	243	29.1	29	12
9	41A:02	50mm (200-300)	yes	230mm GBC	412*	278	25.5	33	14
10	1:16	150mm (150-200)	no	280mm GBC	803*	SEAP		31	5
11	1:16	150mm (150-200)	no	280mm GBC	803*	SEAP		31	5
12	887:04	150mm RACP	no	100mm GBC	41.2	168	62.0	29	3
13	529:04	150mm RACP	no	100mm GBC	11.7*	208	67.5	28	4
14	512:02	150mm	no	Fulldepth	82.1*	295	34.5	28	10
15	524:04	125mm (200-300)	no	100mm GBC	86.5	266	43.5	28	5
16	507:02	125mm RACP	no	150mm GBC	23.7*	160	72.0	29	4
17	16:12	130mm (120-150)	yes	460mm GBC	627	166	76.5	28	4
18	22:30	100mm (200-300)	yes	250mm GBC	313	284	36.0	28	5
19	22:30	100mm (200-300)	yes	200mm GBC	576	311	34.9	28	6
20	11:12	100mm (200-300)	yes	50mm GBC	1449	N/A	N/A	29	31

* Estimated values based on 1986 data
(The ESAL values are cumulative over the life of the pavement)

Table A.5 (Continued). Historical data for field test sites (Sites 21 to 40) in Alberta study (Source: McMillan, 1989).

SITE	PROJECT	STRUCTURE			ESAL's	ORIGINAL ASPHALT		CLIMATIC	AGE
		ASPHALT CONCRETE	ASBC	BASE	(X1000)	PENETRATION (dmm,25C,5s)	ABSOLUTE VISCOSITY (Pa.s)	VALUE (°C)	(YEARS)
21	11:10	100mm (150-200)	yes	250mm GBC	943	N/A	N/A	28	30
22	11:08	100mm (200-300)	yes	300mm GBC	182*	268	45.0	28	13
23	11:08	100mm (200-300)	yes	250mm GBC	179*	287	35.0	28	11
24	11:06	100mm (200-300)	yes	250mm GBC	109*	300	33.7	28	9
25	11A:02	75mm (200-300)	yes	N/A	121	274	38.1	28	17
26	2A:18	100mm (150-200)	yes	300mm GBC	1481	N/A	N/A	29	29
27	12:20	125mm (150-200)	no	100mm GBC	74.8*	182	69.7	30	9
28	12:20	50mm (200-300)	yes	200mm GBC	113*	270	44.8	30	15
29	9:14	63mm (200-300)	yes	150mm SC	310*	248	42.3	30	20
30	49:02	50mm (200-300)	yes	180mm SC	139	278	39.7	27	6
31	49:04	200mm (200-300)	no	Fulldepth	456*	238	29.8	27	18
32	35:12	100mm (200-300)	yes	150mm SC	376	286	23.7	27	12
33	35:16	100mm (200-300)	yes	300mm GBC	212*	222	34.6	27	10
34	881:12	180mm	no	Fulldepth	77.4	268	43.4	29	9
35	881:12	130mm (200-300)	no	100mm GBC	77.4	270	42.0	29	7
36	2:38	50mm (200-300)	yes	150mm SC	328*	N/A	N/A	28	21
37	2:40	50mm (200-300)	yes	150mm SC	265	258	29.9	28	19
38	63:01	50mm (200-300)	yes	175mm SC	806	267	43.1	28	16
39	2:32	300mm (200-300)	no	Fulldepth	3686*	230	54.5	28	8
40	520:02	100mm (200-300)	no	Fulldepth	92.3	270	44.1	29	12

* Estimated values based on 1986 data
(The ESAL values are cumulative over the life of the pavement)

Table A.6. Structural and rheological data from field test results (from cores) (Sites 1-20) in Alberta study (Source: McMillan, 1989).

SITE	STRUCTURE		ABSON EXTRACTED ASPHALT			
	ASPHALT CONCRETE	BASE	PENETRATION (dmm, 25°C)	ABSOLUTE VISCOSITY (Pa.s, 60°C)	KINEMATIC VISCOSITY (mm^2/s, 135°C)	MEASURED RUT DEPTHS (mm)
1	200mm	230mm GBC	106 (40.3)	167.9 (4.2)	339	4.0
2	- mm	- mm GBC	-	-	-	0.0
3	208mm	Fulldepth	156 (49.3)	78.8 (2.6)	236	6.0
4	146mm	200mm GBC	127 (47.9)	89.1 (2.7)	226	7.5
5	114mm	308mm GBC	105 (39.7)	112 (3.4)	259	5.5
6	100mm	135mm CS	122 (42.7)	85.8 (2.9)	214	3.8
7	131mm	105mm CS	115 (40.3)	104 (3.5)	236	6.3
8	241mm	Fulldepth	118 (48.5)	79.7 (2.7)	237	6.0
9	- mm	- mm GBC	-	-	-	2.0
10	- mm	- mm GBC	-	-	-	9.5
11	- mm	- mm GBC	-	-	-	15.0
12	162mm	90 mm GBC	87 (51.6)	204 (3.3)	374	2.3
13	160mm	108mm GBC	91 (43.9)	216 (3.2)	405	3.3
14	165mm	Fulldepth	170 (57.3)	76.7 (2.2)	248	7.3
15	- mm	- mm GBC	-	-	-	5.8
16	123mm	150mm GBC	66 (41.1)	325 (4.5)	422	3.8
17	196mm	415mm GBC	82 (49.2)	282 (3.7)	456	3.5
18	160mm	350mm GBC	126 (44.2)	121 (3.4)	298	2.3
19	- mm	- mm GBC	-	-	-	1.0
20	- mm	- mm GBC	-	-	-	4.5

Note: Bracketed values under Penetration and Absolute viscosity are % retained penetration and viscosity ratio respectively.

Table A.6 (Continued). Structural and rheological data from field test results (from cores) (Sites 21-40) in Alberta study (Source: McMillan, 1989).

SITE	STRUCTURE		ABSON EXTRACTED ASPHALT			
	ASPHALT CONCRETE	BASE	PENETRATION (dmm, 25°C)	ABSOLUTE VISCOSITY (Pa.s, 60°C)	KINEMATIC VISCOSITY (mm^2/s, 135°C)	MEASURED RUT DEPTHS (mm)
21	175mm	332mm GBC	84 (43.9)	289 (9.6)	480	7.5
22	- mm	- mm GBC	-	-	-	1.0
23	- mm	- mm GBC	-	-	-	3.5
24	- mm	- mm GBC	-	-	-	1.5
25	140mm	124mm GBC	129 (47.0)	117 (3.1)	276	3.3
26	144mm	407mm GBC	71 (41.9)	180 (9.0)	302	7.8
27	135mm	166mm GBC	77 (42.0)	308 (4.4)	432	3.0
28	102mm	239mm GBC	75 (27.8)	321 (7.2)	452	2.5
29	- mm	- mm GBC	-	-	-	2.8
30	121mm	140mm SC	99 (35.6)	184 (4.6)	345	5.0
31	187mm	Fulldepth	91 (38.3)	143 (4.8)	289	3.8
32	156mm	165mm SC	76 (26.7)	142 (6.0)	273	4.0
33	192mm	- mm GBC	109 (49.2)	91.4 (2.6)	235	2.8
34	156mm	Fulldepth	-	-	-	5.0
35	137mm	120mm GBC	119 (44.2)	162 (3.9)	352	4.0
36	117mm	147mm SC	95 (33.7)	161 (4.0)	337	5.8
37	103mm	115mm SC	130 (50.6)	82.2 (2.7)	271	3.5
38	73 mm	223mm SC	177 (66.3)	80.0 (1.9)	262	7.0
39	310mm	Fulldepth	194 (84.3)	84.6 (1.6)	264	28.5
40	173mm	Fulldepth	61 (22.5)	471 (10.7)	532	3.0

Note: Bracketed values under Penetration and Absolute viscosity are % retained penetration and viscosity ratio respectively.

Table A.7. Mix design data in Alberta study (Source: McMillan, 1989).

SAMPLE SERIES	BINDER	DESIGN DENSITY	DESIGN % A.C.	MARSHALL STABILITY	MARSHALL FLOW	DESIGN AIR VOIDS	DESIGN VMA
1LG	120-150A	2,372.	5.4	9,200.	2.3	4.4	14.6
2LG	150-200A	2,366.	5.4	9,000.	2.2	4.6	14.8
3LG	200-300A	2,370.	5.4	9,700.	2.1	4.4	14.7
4LG	300-400A	2,375.	5.4	7,150.	2.0	4.2	14.5
5LG	Husky PMA	2,371.	5.4	11,000.	2.3	4.2	14.6
6LG	Imp. PMA	2,375.	5.4	10,900.	2.4	4.0	14.5
7LG	eq 120-150	.	.	.	.	.	.
8LG	eq 150-200	.	.	.	.	.	.
9LG	eq 200-300	.	.	.	.	.	.
10LG	eq 300-400	.	.	.	.	.	.
11LG	eq 120-150	.	.	.	.	.	.
12LG	eq 200-300	.	.	.	.	.	.
1LB	120-150A	2,345.	5.9	10,500.	2.5	3.2	13.8
2LB	150-200A	2,344.	5.9	10,150.	2.3	3.2	13.9
3LB	200-300A	2,341.	5.9	8,250.	2.0	3.3	14.0
4LB	300-400A	2,341.	5.9	7,900.	2.1	3.3	14.0
5LB	Husky PMA	2,340.	5.9	12,550.	2.8	3.1	14.0
6LB	Imp. PMA	2,335.	5.9	11,250.	2.9	3.3	14.2
7LB	eq 120-150	2,327.	5.7	11,400.	2.5	3.8	14.3
8LB	eq 150-200	2,334.	5.7	11,500.	2.4	3.5	14.1
9LB	eq 200-300	2,327.	5.7	9,950.	2.3	3.8	14.3
10LB	eq 300-400	2,338.	5.7	9,800.	2.1	3.4	13.9
11LB	eq 120-150	2,350.	5.5	13,600.	2.8	3.3	13.3
12LB	eq 200-300	2,356.	5.5	12,600.	2.5	3.0	13.1

Note: No extra Marshall Stability testing was done on the recycle LG series (Goose Lake Pit) because of a shortage of material.

Table A.8. Mix characteristics and stiffness data in Alberta study (Source: McMillan, 1989).

Series	Density (Kg/m³)	Air Voids (%)	STIFFNESS (KPa) Virgin Binder 25°C	35°C	45°C	Abson Binder 25°C	35°C	45°C
1LG	2392	3.6	392.0	58.9	15.0	1079.0	245.3	54.0
2LG	2395	3.5	294.3	49.1	14.7	981.0	196.2	40.2
3LG	2401	3.3	117.7	34.3	7.8	490.5	73.6	19.6
4LG	2389	3.8	68.7	-	4.7	225.6	-	16.7
5LG	2391	3.5	294.3	-	19.6	686.7	-	46.1
6LG	2377	4.2	882.9	-	49.1	3237.3	-	176.6
7LG	2387	4.0	392.4	58.9	14.8	1962.0	392.4	78.5
8LG	2383	4.0	294.3	49.1	14.7	1863.9	294.3	49.1
9LG	2388	3.0	117.7	29.4	8.8	549.4	107.9	26.5
10LG	2376	4.5	981.0	-	5.9	608.2	-	29.4
11LG	2374	5.3	-	58.9	-	-	1962.	-
12LG	2389	4.4	-	29.4	-	-	490.5	-
1LB	2335	4.0	392.0	58.9	15.0	981.0	166.8	39.2
2LB	2330	4.1	294.3	49.1	14.7	686.7	117.7	27.5
3LB	2347	3.6	117.7	34.3	7.8	206.0	49.1	10.8
4LB	2337	4.0	68.7	-	4.7	166.8	-	10.8
5LB	2335	4.1	294.3	-	19.6	490.5	-	29.4
6LB	2335	4.0	882.9	-	49.1	941.8	-	50.0
7LB	2334	3.7	392.4	58.9	15.0	1177.2	215.8	98.1
8LB	2333	3.7	294.3	49.1	14.7	882.9	147.2	37.3
9LB	2340	3.6	981.0	24.5	6.9	392.4	73.6	17.7
10LB	2330	4.0	49.0	-	4.9	294.3	-	11.8
11LB	2348	3.4	-	63.8	-	-	490.5	-
12LB	2346	3.5	-	24.5	-	-	117.7	-

Table A.9. Abson recovered binder rheology data in Alberta study (Source: McMillan, 1989).

SERIES	PENETRATION (dmm, 25°C, 5s)	ABSOLUTE VISCOSITY (Pa.s, 60°C)	KINEMATIC VISCOSITY (mm^2/s, 135°C)
1LG	78	358	534
2LG	82	302	483
3LG	133	137	330
4LG	188	85.6	261
5LG	111	761	1272
6LG	46	3523	2095
7LG	54	647	640
8LG	64	537	582
9LG	103	188	364
10LG	103	445	495
11LG	25	19343	3407
12LG	50	1613	780
1LB	89	281	462
2LB	100	217	411
3LB	182	95.2	282
4LB	221	67.4	235
5LB	117	643	1247
6LB	84	1352	1205
7LB	76	385	524
8LB	87	386	521
9LB	138	134	342
10LB	164	104	278
11LB	51	848	742
12LB	102	203	369

Table A.10(a). Percent permanent strain--Virgin LG series--in Alberta study (Source: McMillan, 1989).

Temperature	N	1LG	2LG	3LG	4LG
25	10	0.177 (0.02)	0.206 (0.07)	0.287 (0.15)	0.501 (0.36)
	50	0.317 (0.06)	0.394 (0.08)	0.526 (0.18)	0.701 (0.41)
	100	0.394 (0.06)	0.485 (0.08)	0.649 (0.18)	0.797 (0.43)
	1000	0.672 (0.05)	0.844 (0.05)	1.160 (0.16)	1.225 (0.45)
	5000	0.918 (0.06)	1.201 (0.04)	1.694 (0.10)	1.791 (0.39)
	10000	1.042 (0.08)	1.416 (0.11)	2.002 (0.04)	2.206 (0.33)
	20000	1.187 (0.13)	1.710 (0.15)	2.355 (0.04)	2.759 (0.26)
	30000	1.268 (0.17)	1.905 (0.18)	2.487 (0.03)	3.183 (0.22)
	40000	1.329 (0.22)	2.043 (0.21)	2.615 (0.03)	3.523 (0.22)
	50000	1.387 (0.25)	2.147 (0.25)	2.778 (0.06)	3.806 (0.24)
	60000	1.452 (0.26)	2.234 (0.28)	2.891 (0.15)	4.049 (0.28)
	70000	1.510 (0.28)	2.316 (0.32)	2.998 (0.21)	4.268 (0.31)
	80000	1.559 (0.31)	2.386 (0.36)	3.087 (0.26)	4.496 (0.31)
	90000	1.610 (0.33)	2.445 (0.40)	3.180 (0.30)	4.709 (0.32)
35	10	0.341 (0.17)	0.498 (0.16)	0.561 (0.01)	
	50	0.546 (0.19)	0.744 (0.17)	0.902 (0.04)	
	100	0.642 (0.19)	0.857 (0.17)	1.072 (0.07)	
	1000	1.056 (0.14)	1.376 (0.21)	1.861 (0.06)	
	5000	1.507 (0.20)	2.029 (0.35)	2.844 (0.02)	
	10000	1.810 (0.29)	2.460 (0.47)	3.563 (0.08)	
	20000	2.267 (0.47)	3.073 (0.70)	4.608 (0.22)	
	30000	2.655 (0.62)	3.582 (0.96)	5.348 (0.28)	
	40000	3.027 (0.75)	4.036 (1.25)	5.914 (0.35)	
	50000	3.388 (0.86)	4.440 (1.52)	6.380 (0.43)	
	60000	3.760 (0.99)	4.780 (1.71)	6.769 (0.50)	
	70000	4.061 (1.07)	5.020 (1.77)	7.127 (0.58)	
	80000	4.426 (1.19)	5.280 (1.88)	7.456 (0.66)	
	90000	4.860 (1.37)	5.575 (2.00)	7.792 (0.77)	
45	10	0.528 (0.07)	0.655 (0.01)	0.815 (0.05)	1.000 (0.56)
	50	0.851 (0.05)	0.987 (0.03)	1.261 (0.08)	1.344 (0.56)
	100	1.003 (0.05)	1.182 (0.05)	1.509 (0.09)	1.555 (0.60)
	1000	1.773 (0.09)	2.301 (0.15)	2.974 (0.07)	2.906 (0.99)
	5000	2.927 (0.45)	4.225 (0.11)	5.386 (0.27)	5.209 (1.51)
	10000	3.823 (0.78)	6.111 (0.39)	7.051 (0.56)	7.383 (1.18)
	20000	5.337 (1.20)	9.473 (1.82)	9.649 (0.50)	10.71 (0.05)
	30000	6.841 (1.50)	10.37 (-)	11.39 (0.42)	12.56 (-)
	40000	8.321 (2.70)			
	50000	9.316 (3.23)			
	60000	10.07 (3.43)			
	70000	12.80 (-)			

Bracketed values are standard deviation for the average strain values

Table A.10(b). Percent permanent strain--Virgin LB series--in Alberta study (Source: McMillan, 1989).

Temperature	N	1LB	2LB	3LB	4LB
25	10	0.254 (0.11)	0.257 (0.04)	0.307 (0.07)	0.207 (0.27)
	50	0.460 (0.17)	0.525 (0.13)	0.538 (0.09)	0.585 (0.10)
	100	0.549 (0.20)	0.627 (0.14)	0.639 (0.10)	0.693 (0.11)
	1000	0.829 (0.26)	0.985 (0.15)	1.035 (0.08)	1.186 (0.04)
	5000	1.083 (0.21)	1.354 (0.16)	1.525 (0.10)	2.161 (0.52)
	10000	1.245 (0.15)	1.608 (0.16)	1.904 (0.20)	3.029 (1.16)
	20000	1.477 (0.00)	1.951 (0.15)	2.499 (0.42)	4.383 (2.35)
	30000	1.651 (0.13)	2.203 (0.14)	2.997 (0.65)	5.510 (3.46)
	40000	1.799 (0.25)	2.411 (0.13)	3.465 (0.91)	6.408 (4.37)
	50000	1.929 (0.36)	2.592 (0.12)	4.062 (1.30)	7.143 (5.15)
	60000	2.045 (0.47)	2.767 (0.09)	4.617 (1.74)	7.746 (5.78)
	70000	2.162 (1.58)	2.930 (0.03)	4.151 (1.85)	8.259 (6.31)
	80000	2.277 (0.69)	3.097 (0.04)	4.412 (2.13)	8.701 (6.76)
	90000	2.389 (0.81)	3.262 (0.14)	4.671 (2.38)	
35	10	0.385 (0.03)	0.557 (0.14)	0.657 (0.23)	
	50	0.580 (0.05)	0.810 (0.14)	0.917 (0.28)	
	100	0.667 (0.05)	0.928 (0.14)	1.039 (0.30)	
	1000	1.143 (0.10)	1.541 (0.14)	1.662 (0.41)	
	5000	1.927 (0.58)	2.491 (0.19)	2.495 (0.48)	
	10000	2.660 (1.10)	3.259 (0.31)	3.130 (0.57)	
	20000	3.986 (1.88)	4.387 (0.49)	4.061 (0.74)	
	30000	5.304 (2.29)	5.242 (0.62)	4.849 (0.95)	
	40000	6.804 (2.17)	5.927 (0.72)	5.565 (1.19)	
	50000	8.634 (1.38)	6.502 (0.80)	6.260 (1.45)	
	60000	10.79 (0.13)	7.021 (0.86)	6.997 (1.82)	
	70000	11.71 (0.08)	7.503 (0.91)	7.716 (2.20)	
	80000		7.959 (0.96)	8.473 (2.66)	
	90000		8.393 (0.99)		
45	10	0.790 (0.35)	0.395 (0.26)	0.650 (0.33)	0.865 (0.20)
	50	1.041 (0.35)	0.715 (0.37)	1.016 (0.36)	1.266 (0.17)
	100	1.195 (0.37)	0.907 (0.42)	1.236 (0.39)	1.511 (0.20)
	1000	2.289 (0.55)	2.245 (0.60)	2.666 (0.66)	3.092 (0.67)
	5000	5.266 (1.60)	5.185 (0.78)	5.411 (1.47)	6.259 (0.71)
	10000	8.222 (2.29)	7.574 (0.46)	7.497 (1.87)	9.514 (1.12)
	20000	11.67 (0.51)	10.75 (0.51)	10.20 (1.88)	11.83 (0.27)
	30000		12.72 (-)		
	40000				
	50000				
	60000				
	70000				

Bracketed values are standard deviation for the average strain values

Table A.10(c). Percent permanent strain--Recycle LG series--in Alberta study (Source: McMillan, 1989).

Temperature	N	7LG	8LG	9LG	10LG
25	10	0.275 (0.28)	0.154 (0.06)	0.219 (0.08)	0.100 (0.01)
	50	0.380 (0.29)	0.263 (0.08)	0.361 (0.07)	0.189 (0.01)
	100	0.427 (0.30)	0.313 (0.09)	0.426 (0.06)	0.230 (0.01)
	1000	0.606 (0.35)	0.489 (0.11)	0.643 (0.07)	0.376 (0.01)
	5000	0.758 (0.40)	0.637 (0.14)	0.847 (0.09)	0.506 (0.01)
	10000	0.842 (0.44)	0.720 (0.15)	0.971 (0.13)	0.601 (0.02)
	20000	0.938 (0.50)	0.820 (0.18)	1.148 (0.20)	0.717 (0.05)
	30000	1.003 (0.54)	0.885 (0.21)	1.286 (0.26)	0.789 (0.06)
	40000	1.053 (0.58)	0.937 (0.23)	1.403 (0.32)	0.845 (0.06)
	50000	1.093 (0.60)	0.985 (0.26)	1.511 (0.37)	0.890 (0.06)
	60000	1.130 (0.63)	1.025 (0.30)	1.604 (0.41)	0.928 (0.07)
	70000	1.160 (0.65)	1.062 (0.34)	1.685 (0.45)	0.961 (0.07)
	80000	1.191 (0.67)	1.099 (0.38)	1.765 (0.48)	0.992 (0.07)
	90000	1.229 (0.68)	1.142 (0.41)	1.832 (0.51)	1.008 (0.08)
35	10	0.388 (0.12)	0.347 (0.08)	0.320 (0.03)	
	50	0.552 (0.13)	0.505 (0.09)	0.485 (0.05)	
	100	0.628 (0.14)	0.568 (0.09)	0.541 (0.08)	
	1000	0.899 (0.16)	0.820 (0.10)	0.840 (0.11)	
	5000	1.140 (0.22)	1.060 (0.16)	1.315 (0.03)	
	10000	1.304 (0.25)	1.214 (0.23)	1.657 (0.15)	
	20000	1.519 (0.31)	1.409 (0.36)	2.166 (0.32)	
	30000	1.684 (0.35)	1.539 (0.45)	2.594 (0.47)	
	40000	1.814 (0.39)	1.642 (0.54)	2.956 (0.59)	
	50000	1.921 (0.42)	1.736 (0.62)	3.271 (0.68)	
	60000	2.018 (0.46)	1.817 (0.69)	3.576 (0.72)	
	70000	2.106 (0.49)	1.890 (0.76)	3.876 (0.75)	
	80000	2.188 (0.53)	1.961 (0.81)	4.180 (0.77)	
	90000	2.272 (0.58)	2.032 (0.87)	4.510 (0.80)	
45	10	0.433 (0.15)	0.445 (0.15)	0.518 (0.18)	0.602 (0.18)
	50	0.634 (0.12)	0.655 (0.18)	0.731 (0.19)	0.871 (0.15)
	100	0.746 (0.11)	0.765 (0.20)	0.853 (0.20)	1.034 (0.10)
	1000	1.291 (0.04)	1.337 (0.21)	1.600 (0.31)	2.043 (0.35)
	5000	2.204 (0.09)	2.316 (0.07)	3.119 (0.38)	3.877 (0.88)
	10000	2.920 (0.18)	3.218 (0.23)	4.359 (0.57)	5.479 (1.15)
	20000	3.900 (0.34)	4.604 (0.82)	5.951 (0.88)	8.384 (1.25)
	30000	4.485 (0.71)	5.659 (1.26)	7.152 (1.19)	10.63 (1.41)
	40000	4.925 (1.22)	6.581 (1.45)	8.154 (1.47)	11.39 (1.64)
	50000	5.059 (1.29)	7.427 (1.55)	9.026 (1.73)	11.72 (-)
	60000		8.214 (1.58)	9.809 (1.96)	
	70000		8.973 (1.59)	10.55 (2.12)	

Bracketed values are standard deviation for the average strain values

Table A.10(d). Percent permanent strain--Recycle LB series--in Alberta study (Source: McMillan, 1989).

Temperature	N	7LB	8LB	9LB	10LB
25	10	0.271 (0.22)	0.153 (0.02)	0.415 (0.15)	0.330 (0.11)
	50	0.384 (0.23)	0.319 (0.06)	0.598 (0.20)	0.507 (0.06)
	100	0.276 (-)	0.392 (0.06)	0.679 (0.22)	0.586 (0.03)
	1000	0.606 (0.20)	0.625 (0.08)	0.931 (0.18)	0.960 (0.06)
	5000	0.765 (0.15)	1.833 (0.07)	1.315 (0.21)	1.495 (0.11)
	10000	0.873 (0.09)	0.953 (0.07)	1.575 (0.25)	1.855 (0.09)
	20000	1.023 (0.01)	1.089 (0.05)	1.939 (0.30)	2.361 (0.02)
	30000	1.139 (0.11)	1.170 (0.04)	2.202 (0.36)	2.754 (0.05)
	40000	1.243 (0.19)	1.228 (0.03)	2.099 (-)	3.082 (0.08)
	50000	1.343 (0.29)	1.270 (0.02)	2.243 (-)	3.367 (0.09)
	60000	1.440 (0.38)	1.303 (0.01)	2.355 (-)	3.622 (0.08)
	70000	1.536 (0.47)	1.324 (0.01)	2.440 (-)	4.845 (0.05)
	80000	1.622 (0.57)	1.344 (0.02)	2.532 (-)	4.053 (0.01)
	90000	1.710 (0.66)	1.358 (0.03)	2.591 (-)	4.249 (0.07)
35	10	0.365 (0.05)	0.301 (0.08)	0.529 (0.03)	
	50	0.568 (0.09)	0.464 (0.09)	0.769 (0.05)	
	100	0.655 (0.11)	0.536 (0.10)	0.890 (0.05)	
	1000	1.074 (0.27)	0.828 (0.13)	1.585 (0.05)	
	5000	1.627 (0.59)	1.169 (0.17)	2.721 (0.09)	
	10000	2.048 (0.79)	1.417 (0.24)	3.690 (0.11)	
	20000	2.737 (1.12)	1.775 (0.38)	5.219 (0.19)	
	30000	3.396 (1.48)	2.045 (0.50)	6.419 (0.35)	
	40000	4.049 (1.91)	2.279 (0.61)	7.416 (0.50)	
	50000	4.708 (2.37)	2.484 (0.72)	8.284 (0.63)	
	60000	5.346 (2.85)	2.681 (0.82)	9.078 (0.74)	
	70000	5.986 (3.38)	2.884 (0.92)	9.757 (0.84)	
	80000	6.658 (3.97)	3.091 (1.01)	10.35 (0.88)	
	90000	7.401 (4.64)	3.314 (1.09)	10.89 (0.89)	
45	10	0.543 (0.03)	0.490 (0.04)	0.220 (0.07)	0.527 (0.03)
	50	0.793 (0.01)	0.711 (0.08)	0.441 (0.10)	0.852 (0.06)
	100	0.917 (0.01)	0.827 (0.09)	0.571 (0.10)	1.066 (0.09)
	1000	1.705 (0.09)	1.460 (0.02)	1.446 (0.07)	2.803 (0.70)
	5000	3.351 (0.32)	2.482 (0.37)	2.992 (0.51)	7.034 (2.66)
	10000	4.657 (0.47)	3.391 (0.76)	4.295 (0.86)	
	20000	6.498 (0.54)	4.947 (1.38)	5.898 (0.95)	
	30000	7.760 (0.55)	6.897 (1.04)	7.125 (1.00)	
	40000	8.459 (0.28)	9.653 (0.66)	8.154 (1.00)	
	50000	9.027 (-)	10.54 (-)	9.083 (1.02)	
	60000	9.744 (-)	11.79 (-)	9.775 (0.82)	
	70000	10.46 (-)	12.99 (-)	10.61 (1.07)	

Bracketed values are standard deviations for the average strain values

Table A.10(e). Percent permanent strain--PMA series--in Alberta study (Source: McMillan, 1989).

Temperature	N	5LG	6LG	5LB	6LB
25	10	0.145 (0.03)	0.075 (0.00)	0.177 (0.08)	0.111 (0.02)
	50	0.260 (0.05)	0.142 (0.00)	0.307 (0.10)	0.201 (0.03)
	100	0.315 (0.06)	0.175 (0.01)	0.366 (0.10)	0.242 (0.03)
	1000	0.519 (0.07)	0.286 (0.02)	0.562 (0.11)	0.370 (0.04)
	5000	0.695 (0.08)	0.356 (0.02)	0.705 (0.14)	0.456 (0.02)
	10000	0.785 (0.11)	0.385 (0.02)	0.780 (0.15)	0.484 (0.02)
	20000	0.884 (0.15)	0.419 (0.01)	0.848 (0.17)	0.512 (0.02)
	30000	0.964 (0.21)	0.437 (0.02)	0.881 (0.18)	0.525 (0.03)
	40000	1.022 (0.25)	0.459 (0.03)	0.904 (0.19)	0.536 (0.03)
	50000	1.107 (0.33)	0.465 (0.04)	0.921 (0.20)	0.547 (0.04)
	60000	1.215 (0.47)	0.461 (0.02)	0.936 (0.21)	0.558 (0.04)
	70000		0.467 (0.02)	0.946 (0.21)	0.568 (0.05)
	80000		0.469 (0.02)	0.959 (0.22)	0.584 (0.07)
	90000		0.472 (0.02)	0.969 (0.22)	0.596 (0.09)
45	10	0.529 (0.10)	0.356 (0.07)	0.537 (0.13)	0.434 (0.21)
	50	0.769 (0.17)	0.530 (0.03)	0.739 (0.11)	0.617 (0.21)
	100	0.881 (0.20)	0.604 (0.01)	0.828 (0.11)	0.694 (0.21)
	1000	1.384 (0.36)	0.848 (0.09)	1.226 (0.05)	0.963 (0.21)
	5000	2.135 (0.81)	1.017 (0.14)	1.710 (0.10)	1.194 (0.20)
	10000	2.637 (1.17)	1.089 (0.17)	2.017 (0.21)	1.309 (0.19)
	20000	3.152 (1.49)	1.176 (0.18)	2.391 (0.39)	1.424 (0.17)
	30000	3.446 (1.61)	1.222 (0.20)	2.624 (0.51)	1.481 (0.15)
	40000	3.647 (1.64)	1.263 (0.22)	2.805 (0.58)	1.526 (0.13)
	50000	3.842 (1.65)	1.302 (0.24)	2.965 (0.61)	1.564 (0.13)
	60000	4.021 (1.67)	1.335 (0.24)	3.112 (0.65)	1.597 (0.12)
	70000	4.178 (1.70)	1.366 (0.26)	3.321 (0.75)	1.634 (0.12)

Bracketed values are standard deviation for the average strain values

Table A.11. Experimental fracture temperatures for asphalt concrete beams in Esso study (Source: Robertson, 1987).

Bitumen	Blown LV 150/200(1)	Blown LV 100/120(1)	HV 200/300(1)	HV 300/400(1)	Blown LV 150/200(2)	Blown LV 85/100(2)	HV 150/200(2)
Penetration at 25°C	130	56	84	144	127	81	70
Penetration Index	-0.7	+0.6	-0.6	-1.0	-0.5	+0.3	-1.0
Observed Fracture Temperature, °C(3)	-30	-29.5	-32	-32	-34	-35	-29
Predicted	-37	-32.5	-31.5	-33	-38	-35.5	-29

(1) Recovered from Richer, Manitoba road trial after 4 years service.
(2) Recovered from Shebandowan, Ontario road trial after 6 months service.
(3) Mix specimen cooled at 10°C/h in a quartz frame.
(4) Design temperature -10°C.

Table A.12.
Experimental fracture temperatures for asphalt concrete beams in Esso study (Source: Robertson, 1987).

Bitumen	T1 50/70	T2 50/70	T3 50/70	T1 80/100	T2 80/100	T3 80/100
Penetration at 25°C	59	64	57	85	79	91
Penetration Index	−0.6	−0.0	+1.2	−1.2	−0.4	+0.7
Observed Fracture Temperature, °C(1)	−26	−26	−33	−27	−29.5	−33
Predicted (2)	−27.5	−31.5	−36	−28	−31.5	−40

(1) Cooled at 10°C/h under restraint.
(2) Design temperature −10°C.

Table A.13.
Observed fracture temperatures for the Ste. Anne test road in Esso study (Source: Robertson, 1987).

Bitumen	HV 150/ 200(1)	LV 150/ 200(1)	LV 300/ 400(1)	HV 150/ 200(2)	LV 300/ 400(2)
Penetration at 25°C	159	192	313	55	119
Penetration Index	−1.4	−2.7	−2.9	−0.1	−1.5
Observed Fracture Temperature, °C(3)	< −38	−34	−37	−34	−34
Predicted (4)	−36	−29	−41	−29.5	−30.5

(1) Original asphalt (1967).
(2) Field aged asphalt (1972).
(3) Pavement cracking temperature.
(4) Design temperature −10°C.

Table A.14.
Maximum paving mix stiffness to avoid low temperature pavement cracking (Source: McLeod, 1972).

Minimum Temperature at 50 mm Depth, °C	Maximum Allowable Mix Stiffness at 20000 s Loading Time, N/m^2
−40	3.5×10^9
−32	2.4×10^9
−23	1.4×10^9
−12	2.4×10^8

Table A.15.
Minimum asphalt cement PVN to avoid thermally induced pavement cracking (Source: McLeod, 1972)

Minimum Air Temperature, °C(1)	Minimum Pavement Temperature, °C	Minimum PVN 85/100	Minimum PVN 150/200	Minimum PVN 300/400
−23	−18	(+0.7)	−1.0	−3.7
−29	−23	NR	−0.2	−2.2
−36	−29	NR	(+0.2)	−1.2
−41	−34	NR	NR	−0.4

(1) Estimated from minimum pavement temperature using correlation of Deme et al. (1975)

Table A.16.
Minimum asphalt cement PVN to avoid thermally induced pavement cracking
(Source: Fromm and Phang, 1971)

Winter Design Temperature, °C	Minimum PVN		
	85/100	150/200	300/400
−25	−1.1	−1.6	−2.0
−30	−0.6	−1.0	−1.5
−35	(+0.3)	−0.4	−0.9
−40	NR	(+0.2)	−0.5

Table A.17.
Minimum penetrations at 10°C and 4°C to avoid thermally induced pavement cracking (Source: Readshaw, 1972)

Winter Design Temperature = −30°C

Penetration at 25°C (100/5)	40	60	100	200	300
Penetration at 10°C (100/5) min.	9	11	15	22	37
Penetration at 4°C (100/5) min.	5	5.6	7	9	18

Winter Design Temperature = −40°C

Penetration at 25°C (100/5)	40	60	100	200	300	400
Penetration at 10°C (100/5) min.	12	16	23	38	53	65
Penetration at 4°C (100/5) min.	8	9.5	13	19	25	31

Table A.18.
Comparison of calculation procedures for estimating asphalt fracture temperatures (Source: Robertson, 1987)

Penetration at 25°C	Penetration Index	Calculated Fracture Temperature, °C(1) Equation 5(2)	Equation 4
149	+0.09	−46.5	−45.6
52	+2.18	−42.4	−39.7
147	−1.05	−40.7	−37.2
55	−1.80	−31.6	−27.0
51	+0.15	−30.9	−26.9

(1) 10°C/h cooling rate. Fracture Stress = 5×10^5 N/m^2.
(2) Hills' equation. Temperature increment = 10°C
Loading time for stiffness modulus = 7200 s.

Table A.19. Comparison of asphalt cement selection criteria.

Pavement Design Temperature, °C	Minimum Penetration Index Readshaw (Ref. 18)	Gaw (Ref. 19)(1)	Sugawara et al. (Ref. 20)(1)(2)	Design Chart (Figure 12)
	85/100 Penetration Asphalt			
−15	—	−3.4	−1.1	−1.7
−25	−1.9(3)	−1.6	+0.7	0.0
−35	−0.2(4)	−0.1	+2.8	+1.8
	150/200 Penetration Asphalt			
−15	—	+4.0	−2.1	−2.8
−25	−2.6(3)	−2.4	−0.5	−1.4
−35	−0.9(4)	−0.9	+1.1	0.0

(1) Minimum PI for Design Temperature = Fracture Temperature + 10°C.
(2) PI calculated from softening point and penetration.
(3) For −30°C air temperature corresponding to −24°C pavement temperature.
(4) For −40°C air temperature corresponding to −33°C pavement temperature.

Table A.20. Rheological properties in Iowa study - I (Enüstün et al., 1990).

sample ID	P5	P25	P4	S.P. C
(AC-5s)				
4PAO	19	181	64	41.5
4PAR	14	100	52	45.5
4PC1	11	98	35	44.6
4PO5	11	86	31	48.8
4PO	10	52	25	54.0
4PM	18	144	56	43.5
4PC	15	105	45	46.5
4L35	15	105	54	54.0
4L75	12	55	29	55.0
5PAO	18	191	68	41.5
5PAR	13	103	40	48.0
5PC1	11	83	33	49.3
5PO	11	53	25	54.0
5PM	19	156	61	45.5
5L35	12	77	39	51.0
5L75	13	86	37	50.0
7PAO	16	193	60	39.0
7PAR	12	94	38	43.5
7PC1	14	105	41	45.3
7PO5	11	84	32	49.3
7PO	10	46	24	56.0
7L35	17	105	44	45.5
7L75	14	91	39	50.0
8PAO	17	196	58	38.5
8PAR	13	95	39	46.5
8PC1	15	107	43	44.5
8PO5	11	83	30	50.2
8PO	10	46	25	56.0
8L35	15	88	36	49.5
8L75	14	84	42	50.0
(AC-10s)				
1PAO	8	82	29	47.5
1PAR	7	50	21	52.0
1PC1	7	52	21	51.4
1PO5	6	44	16	54.1
1PO	6	27	14	61.5
1PM	10	55	29	56.0
1L35	8	27	15	66.5
1L75	9	32	17	62.5

Table A.20. (Continued)

sample ID	P5	P25	P4	S.P. C
11PAO	15	133	44	44.0
11PAR	10	69	29	51.5
11PC1	11	91	36	49.0
11PO5	8	58	24	53.2
11PO	7	35	21	59.5
11L35	10	60	27	55.0
11L75	11	65	28	54.0
(AC-20s)				
2PAO	7	54	15	49.0
2PAR	6	38	17	55.5
2PC1	7	45	19	55.8
2PO5	5	36	14	52.6
2PO	6	25	14	67.0
2PM	7	35	18	59.0
2PC	6	30	17	61.0
2LM	7	39	19	60.5
2L35	5	31	16	60.5
2L75	7	35	16	58.5
3PAO	9	75	30	47.0
3PAR	8	48	22	54.5
3PC1	11	89	36	47.2
3PO5	5	41	16	55.8
3PO	6	26	14	63.0
3PM	9	41	22	58.0
3PC	9	40	24	58.0
3L35	7	30	16	66.5
3L75	6	33	18	61.5
10PAO	9	82	29	49.0
10PAR	7	47	19	50.5
10PC1	6	36	18	59.0
10PO5	5	40	16	56.5
10PO	5	24	14	62.5
10L35	8	32	19	65.0
10L75	10	81	30	48.5
12PAO	8	82	28	47.0
12PAR	6	47	20	53.5
12PC1	9	67	27	49.6
12PO5	5	40	15	56.2
12PO	4	23	14	63.0
12L35	9	54	21	56.0
12L50	10	65	27	53.0
12L75	9	51	24	54.0

P5: penetration @ 5C, 100g, 5sec; P25: penetration @ 25C, 100g 5sec; P4: penetration @ 4C, 200g, 60sec; S.P.: Ring & Ball softening point

Table A.21. Rheological properties in Iowa study - II (Enüstün et al., 1990).

sample ID	VIS 25 poise	C.Flow	S.Index	VIS 60 poise	VIS 135 cSt
(AC-5)					
4PAO	1.50E+05	0.98	0.030	583	250.3
4PAR	7.20E+05	0.96	0.025	1574	368.6
4PC1	9.00E+05	0.96	0.065	1730	343.0
4PO5	1.06E+06	0.92	0.200	2049	394.0
4PO	4.15E+06	0.94	0.080	4682	553.1
4PM	2.90E+06	0.60	0.330	856	1094.8
4PC	5.60E+05	0.92	0.050	1410	361.5
4L35	1.75E+06	0.79	0.220	4457	573.3
4L75	3.80E+06	0.79	0.210	6804	707.2
5PAO	1.88E+05	0.96	0.045	632	247.5
5PAR	9.00E+05	0.95	0.060	1470	395.5
5PC1	1.60E+06	0.91	0.150	2352	540.0
5PO	4.75E+06	0.84	0.160	4509	500.1
5PM	4.70E+05	0.91	0.066	1341	285.9
5L35	1.21E+05	0.90	0.100	2368	443.9
5L75	1.23E+06	0.87	0.130	2529	447.1
7PAO	2.50E+05	0.99	0.027	734	250.8
7PAR	7.10E+05	0.98	0.023	1742	398.5
7PC1	5.40E+05	0.96	0.064	846	346.0
7PO5	1.45E+06	0.87	0.100	2365	414.0
7PO	5.25E+06	0.78	0.230	6383	618.7
7L35	4.90E+05	0.89	0.070	1431	388.9
7L75	1.44E+06	0.89	0.110	2324	445.4
8PAO	1.86E+05	1.00	0.034	670	253.0
8PAR	7.15E+05	0.98	0.019	1832	404.7
8PC1	7.00E+05	0.98	0.096	1276	380.0
8PO5	1.10E+06	0.93	0.080	2430	429.0
8PO	4.20E+06	0.90	0.100	5080	550.4
8L35	1.40E+06	0.86	0.160	2161	436.2
8L75	1.40E+06	0.89	0.090	3484	500.8
(AC-10)					
1PAO	9.10E+05	0.95	0.045	1576	368.7
1PAR	3.50E+06	0.90	0.100	3722	515.3
1PC1	3.20E+06	0.85	0.108	4015	581.0
1PO5	6.50E+06	0.84	0.160	5603	552.0
1PO	1.55E+07	0.74	0.260	13210	788.4
1PM	2.80E+06	0.82	0.200	6235	664.8
1L35	1.67E+07	0.57	0.420	51768	1431.3
1L75	1.12E+07	0.65	0.390	32534	1185.6

Table A.21. (Continued)

sample ID	VIS 25 poise	C.Flow	S.Index	VIS 60 poise	VIS 135 cSt
11PAO	3.95E+05	0.96	0.037	1110	444.4
11PAR	1.90E+06	0.92	0.070	3558	559.0
11PC1	1.09E+06	0.93	0.077	2024	452.0
11PO5	2.57E+06	0.89	0.140	4602	592.0
11PO	9.50E+06	0.82	0.190	10426	770.3
11L35	3.30E+06	0.83	0.180	4481	638.5
11L75	2.70E+06	0.92	0.090	4220	625.3
(AC-20)					
2PAO	3.40E+06	0.94	0.063	3571	889.2
2PAR	8.20E+06	0.72	0.320	6306	986.7
2PC1	4.30E+06	0.71	0.290	5491	817.0
2PO5	8.00E+06	0.65	0.290	10977	1080.0
2PO	1.95E+07	0.55	0.460	39716	1654.7
2PM	8.30E+06	0.62	0.350	52329	1975.4
2PC	1.20E+07	0.39	0.580	16986	1384.5
2LM	6.50E+06	0.55	0.440	12315	934.1
2L35	1.40E+07	0.58	0.420	55202	1757.9
2L75	1.15E+07	0.66	0.340	17653	1063.9
3PAO	1.17E+06	0.96	0.030	2730	477.3
3PAR	3.60E+06	0.90	0.100	6107	713.3
3PC1	1.45E+06	0.88	0.110	2485	495.0
3PO5	5.70E+06	0.84	0.140	8948	810.0
3PO	1.94E+07	0.72	0.280	21408	1201.6
3PM	4.60E+06	0.81	0.190	15891	1088.2
3PC	6.70E+06	0.72	0.270	13398	964.6
3L35	1.05E+07	0.74	0.260	17218	1039.9
3L75	1.25E+07	0.72	0.290	22750	1183.6
10PAO	1.01E+06	0.97	0.030	2105	459.8
10PAR	3.75E+06	0.93	0.070	6334	732.7
10PC1	7.10E+06	0.78	0.254	14554	1030.0
10PO5	6.00E+06	0.90	0.170	7977	799.0
10PO	1.85E+07	0.73	0.270	18360	1091.0
10L35	1.20E+07	0.69	0.360	37654	1630.2
10L75	1.53E+06	0.90	0.100	2507	489.5
12PAO	1.04E+06	0.98	0.023	2337	470.0
12PAR	4.40E+06	0.91	0.080	6503	774.7
12PC1	2.70E+06	0.91	0.130	2544	440.0
12PO5	6.50E+06	0.90	0.160	9543	828.0
12PO	2.05E+07	0.73	0.260	22624	1139.7
12L35	5.70E+06	0.79	0.200	722	819.7
12L50	4.00E+06	0.89	0.120	4611	715.7
12L75	4.90E+06	0.90	0.100	5489	713.3

VIS 25: viscosity @ 25C; C.Flow: complex flow; S.Index: shear index; VIS 60: viscosity @ 60C; VIS 135: viscosity @ 135C

Table A.22. Temperature susceptibility in Iowa study (Enüstün et al., 1990).

sample ID	PR	PI	CN	VTS	PVN,60	PVN,135
(AC-5)						
4PAO	0.354	0.150	7.963	3.381	-0.367	-0.243
4PAR	0.520	-0.632	2.628	3.474	-0.279	-0.347
4PC1	0.357	-0.980	-0.714	3.573	-0.210	-0.481
4PO5	0.360	-0.114	0.552	3.526	-0.247	-0.418
4PO	0.481	-0.153	-2.208	3.576	-0.203	-0.466
4PM	0.389	-0.042	27.546	2.384	-0.329	1.907
4PC	0.429	-0.166	3.981	3.445	-0.316	-0.320
4L35	0.514	1.903	-15.241	3.530	0.937	0.396
4L75	0.527	0.212	-8.185	3.527	0.252	-0.070
5PAO	0.356	0.400	4.636	3.427	-0.171	-0.190
5PAR	0.388	0.221	5.037	3.387	-0.303	-0.204
5PC1	0.398	-0.095	3.844	3.328	-0.160	0.011
5PO	0.472	-0.106	-3.780	3.641	-0.211	-0.585
5PM	0.391	0.983	-7.158	3.624	0.336	-0.211
5L35	0.506	0.153	1.697	3.487	-0.275	-0.364
5L75	0.430	0.214	-2.340	3.507	-0.025	-0.229
7PAO	0.311	-0.640	0.980	3.481	0.030	-0.153
7PAR	0.404	-1.461	2.912	3.451	-0.273	-0.300
7PC1	0.390	-0.558	16.607	3.265	-0.872	-0.388
7PO5	0.381	-0.046	-1.611	3.543	-0.134	-0.370
7PO	0.522	-0.001	-4.995	3.605	-0.089	-0.438
7L35	0.419	-0.478	4.975	3.390	-0.300	-0.207
7L75	0.429	0.388	-1.538	3.477	-0.019	-0.169
8PAO	0.296	-0.803	3.073	3.433	-0.051	-0.116
8PAR	0.411	-0.482	1.684	3.459	-0.201	-0.264
8PC1	0.402	-0.741	7.054	3.362	-0.393	-0.220
8PO5	0.361	0.162	-1.342	3.525	-0.126	-0.330
8PO	0.543	-0.001	-1.390	3.611	-0.308	-0.594
8L35	0.409	0.148	0.636	3.464	-0.153	-0.239
8L75	0.500	0.144	-7.584	3.542	0.273	-0.087
(AC-10)						
1PAO	0.354	-0.620	7.221	3.474	-0.599	-0.569
1PAR	0.420	-0.711	3.467	3.544	-0.485	-0.602
1PC1	0.404	-0.764	2.889	3.480	-0.353	-0.399
1PO5	0.364	-0.519	-2.827	3.645	-0.280	-0.633
1PO	0.519	-0.053	-5.255	3.684	-0.189	-0.626
1PM	0.527	0.434	-7.211	3.542	0.166	-0.155
1L35	0.556	0.818	-28.138	3.718	1.029	0.108
1L75	0.531	0.476	-24.338	3.697	0.875	0.044

Table A.22. (Continued)

sample ID	PR	PI	CN	VTS	PVN,60	PVN,135
11PAO	0.331	-0.165	8.504	3.175	-0.174	0.307
11PAR	0.420	-0.026	-1.686	3.463	-0.033	-0.151
11PC1	0.396	0.113	2.210	3.409	-0.166	-0.146
11PO5	0.414	-0.080	-3.036	3.517	-0.051	-0.259
11PO	0.600	0.103	-6.383	3.618	-0.034	-0.417
11L35	0.450	0.436	-1.676	3.449	-0.024	-0.117
11L75	0.431	0.419	-2.453	3.442	0.044	-0.057
(AC-20)						
2PAO	0.278	-1.276	13.393	3.116	-0.411	0.226
2PAR	0.447	-0.534	10.235	3.255	-0.382	-0.016
2PC1	0.422	-0.095	5.292	3.341	-0.266	-0.088
2PO5	0.389	-1.278	-1.395	3.392	0.056	0.045
2PO	0.560	0.746	-16.497	3.535	0.676	0.209
2PM	0.514	0.005	-27.443	3.507	1.457	0.790
2PC	0.567	0.066	-2.815	3.372	0.187	0.173
2LM	0.487	0.534	-9.454	3.536	0.283	-0.060
2L35	0.516	0.039	-28.427	3.601	1.306	0.507
2L75	0.457	-0.093	-13.336	3.568	0.453	-0.003
3PAO	0.400	-1.008	0.067	3.485	-0.170	-0.287
3PAR	0.458	-0.230	-1.974	3.481	-0.067	-0.203
3PC1	0.404	-0.484	-0.745	3.419	0.014	-0.036
3PO5	0.390	-0.316	-5.360	3.526	0.059	-0.196
3PO	0.538	0.142	-8.030	3.548	0.187	-0.144
3PM	0.537	0.153	-14.002	3.516	0.601	0.193
3PC	0.600	0.097	-11.564	3.542	0.402	0.008
3L35	0.533	1.038	-9.235	3.575	0.199	-0.185
3L75	0.545	0.357	-16.154	3.579	0.596	0.073
10PAO	0.354	-0.199	3.939	3.411	-0.296	-0.241
10PAR	0.404	-1.211	-1.850	3.474	-0.064	-0.189
10PC1	0.500	0.066	-9.750	3.524	0.317	-0.016
10PO5	0.400	-0.222	-1.967	3.494	-0.086	-0.238
10PO	0.583	-0.100	-3.586	3.563	-0.062	-0.336
10L35	0.594	0.920	-21.449	3.528	1.009	0.446
10L75	0.370	-0.372	0.946	3.431	-0.133	-0.163
12PAO	0.341	-0.764	1.676	3.436	-0.186	-0.209
12PAR	0.426	-0.505	-1.415	3.443	-0.038	-0.114
12PC1	0.403	-0.592	2.979	3.522	-0.421	-0.526
12PO5	0.375	-0.286	-5.961	3.533	0.082	-0.192
12PO	0.609	-0.092	-7.215	3.603	0.062	-0.322
12L35	0.389	0.388	58.905	2.518	-1.981	0.114
12L50	0.415	0.181	-2.116	3.373	0.133	0.134
12L75	0.471	-0.200	-0.724	3.441	-0.077	-0.139

PR: pen. ratio, P4/P25; PI: pen. index; CN: class number; VTS: viscosity-temp susceptibility; PVN,60: pen-viscosity number @ 60C; PVN,135: pen-viscosity number @ 135C

Table A.23. Low-temperature cracking properties in Iowa study (Enüstün et al., 1990).

sample ID	CT C	TES C	S,-23 ksi	S,-29 ksi
(AC-5)				
4PAO	-43.5	-49.0	0.189	0.508
4PAR	-44.0	-38.0	1.740	3.625
4PC1	-38.0	-35.4	2.030	4.930
4PO5	-39.0	-39.2	1.595	3.190
4PO	-43.0	-33.5	4.350	9.425
4PM	-47.0	-44.5	0.399	0.870
4PC	-44.0	-40.5	1.088	2.175
4L35	-48.0	-55.0	0.363	0.580
4L75	-46.0	-36.0	2.900	5.438
5PAO	-42.5	-51.5	0.109	0.363
5PAR	-41.5	-43.0	0.943	1.813
5PC1	-39.5	-38.8	1.740	3.190
5PO	-44.0	-33.5	4.060	7.975
5PM	-45.0	-54.5	0.247	0.363
5L35	-41.0	-39.5	1.450	2.900
5L75	-43.5	-41.0	1.088	2.320
7PAO	-39.0	-44.0	0.218	0.725
7PAR	-39.0	-32.0	3.625	8.700
7PC1	-43.0	-38.8	1.450	2.900
7PO5	-39.5	-39.7	1.305	2.900
7PO	-45.0	-32.5	5.075	10.150
7L35	-45.5	-39.0	1.305	2.900
7L75	-41.5	-43.0	1.088	1.885
8PAO	-40.0	-43.5	0.363	1.160
8PAR	-42.5	-38.0	1.160	2.900
8PC1	-44.0	-37.5	1.450	3.335
8PO5	-39.5	-39.8	1.088	2.465
8PO	-45.0	-32.5	4.785	8.700
8L35	-46.5	-40.5	1.088	2.320
8L75	-46.5	-40.0	1.160	2.610
(AC-10)				
1PAO	-35.0	-36.0	2.610	5.800
1PAR	-36.0	-30.5	6.090	12.325
1PC1	-36.3	-29.1	5.075	11.600
1PO5	-35.0	-28.9	7.250	13.775
1PO	-40.0	-26.0	13.050	21.750
1PM	-42.5	-37.0	2.900	5.800
1L35	-47.5	-49.5	7.250	12.325
1L75	-47.5	-48.5	7.250	11.600

Table A.23. (Continued)

sample ID	CT C	TES C	S,-23 ksi	S,-29 ksi
11PAO	-42.5	-43.0	0.508	1.450
11PAR	-40.0	-37.0	2.175	4.350
11PC1	-38.5	-41.0	1.305	2.755
11PO5	-37.5	-34.9	3.190	6.670
11PO	-40.0	-29.5	6.525	13.050
11L35	-41.5	-38.0	2.320	5.075
11L75	-42.5	-39.0	2.175	3.915
(AC-20)				
2PAO	-36.0	-28.0	10.875	24.650
2PAR	-35.5	-28.5	10.875	21.750
2PC1	-37.5	-32.2	5.800	10.875
2PO5	-33.0	-24.4	18.850	50.750
2PO	-40.0	-30.0	8.700	14.500
2PM	-40.0	-29.5	7.250	14.500
2PC	-37.5	-28.0	9.425	21.750
2LM	-38.5	-34.0	5.075	8.700
2L35	-34.0	-28.5	10.150	18.850
2L75	-40.0	-31.0	8.700	14.500
3PAO	-37.5	-33.0	4.350	9.425
3PAR	-39.0	-32.0	5.075	10.875
3PC1	-39.0	-36.9	1.450	3.915
3PO5	-32.5	-29.3	7.250	14.500
3PO	-40.0	-27.0	11.600	20.300
3PM	-44.0	-32.0	5.075	10.150
3PC	-44.0	-31.5	5.365	10.440
3L35	-42.5	-34.0	5.800	10.875
3L75	-37.0	-31.0	7.250	13.050
10PAO	-37.0	-38.0	1.740	3.770
10PAR	-37.0	-27.5	9.425	21.750
10PC1	-36.3	-30.5	6.815	12.325
10PO5	-32.5	-29.6	7.250	13.775
10PO	-36.5	-25.0	14.500	24.650
10L35	-45.0	-34.0	5.220	8.700
10L75	-38.5	-37.0	2.320	5.365
12PAO	-35.0	-35.0	2.900	5.510
12PAR	-34.0	-31.0	5.510	10.875
12PC1	-38.5	-33.4	3.625	8.700
12PO5	-32.5	-28.9	7.250	13.775
12PO	-32.5	-24.5	15.950	24.650
12L35	-42.5	-37.0	2.900	5.510
12L50	-41.0	-37.5	2.175	4.350
12L75	-41.5	-33.0	4.350	7.975

CT: cracking temp.; TES: temp. of equivalent stiffness @ 20ksi, 10,000sec; S,-23: stiffness @ -23C, 10,000sec; S,-29: stiffness @ -29C, 20,000sec

Table A.24. Viscoelastic properties of thermal cycled samples at +5°C in Iowa study (Enüstün et al., 1990).
n : Viscosity, MP
G : Elastic shear modulus, psi

Sample	3rd day value after cooling from +25°C		% variation in the first 3 days after: cooling from +25 C		warming from −30 C		% change*	
	n	G	n	G	n	G	n	G
J05-01-0	225	12	31	0	29	−38	4.5	-27
J10-01-0	1580	16	44	−18	--	--	--	--
J20-01-0	6990	13	93	−38	--	--	--	--
SC-S	29200	130	71	0	160	650	-140	-118
WR-S	9720	54	43	20	16	−14	13	-25

*: % difference between third day value after warming from -30°C, and that after cooling from +25°C.

Table A.25. Summary of TMA results in Iowa study (Enüstün et al., 1990).

sample ID	Tg C	Tsp C	ML um/C	MH um/C
(AC-5s)				
4PAO	-31.3	7.0	0.176	0.466
4PAR	-30.3	5.0	0.189	0.441
4PC1	-37.7	3.9	0.127	0.559
4PO5	-28.8	7.3	0.132	0.422
4PO	-27.3	13.0	0.213	0.532
4PM	-29.5	6.0	0.158	0.441
4PC	-29.3	8.0	0.249	0.512
4L35	-33.5	7.0	0.162	0.547
4L75	-33.3	12.5	0.162	0.487
5PAO	-30.0	14.5	0.149	0.510
5PAR	-36.3	7.0	0.039	0.407
5PC1	-36.7	6.9	0.047	0.433
5PO	-34.0	13.0	0.162	0.503
5PM	-31.3	5.0	0.106	0.451
5L35	-30.5	10.5	0.160	0.561
5L75	-32.5	10.5	0.167	0.510
7PAO	-34.0	15.0	0.195	0.494
7PAR	-26.8	10.5	0.215	0.571
7PC1	-36.5	5.2	0.051	0.380
7PO5	-29.4	7.0	0.193	0.452
7PO	-28.5	14.5	0.231	0.577
7L35	-31.0	3.0	0.249	0.603
7L75	-37.0	12.5	0.225	0.618
8PAO	-26.8	15.0	0.209	0.577
8PAR	-29.9	12.0	0.264	0.695
8PC1	-32.1	4.2	0.079	0.331
8PO5	-29.2	6.8	0.141	0.464
8PO	-30.9	12.5	0.174	0.392
8L35	-32.0	9.5	0.113	0.441
8L75	-33.3	7.0	0.240	0.630
(AC-10s)				
1PAO	-33.0	-4.0	0.094	0.299
1PAR	-22.5	14.0	0.208	0.682
1PC1	-35.3	10.1	0.182	0.374
1PO5	-30.9	12.2	0.105	0.435
1PO	-27.5	12.5	0.264	0.647
1PM	-31.9	12.0	0.235	0.566
1L35	-28.0	25.0	0.249	0.483
1L75	-28.0	14.5	0.216	0.477

Table A.25. (Continued)

sample ID	Tg C	Tsp C	ML um/C	MH um/C
11PAO	-27.5	3.5	0.126	0.488
11PAR	-28.0	12.0	0.259	0.687
11PC1	-34.9	4.5	0.139	0.454
11PO5	-29.8	9.4	0.182	0.386
11PO	-24.0	13.5	0.180	0.451
11L35	-34.0	4.0	0.251	0.732
11L75	-25.5	25.0	0.141	0.505
(AC-20s)				
2PAO	-25.0	17.5	0.167	0.477
2PAR	-28.5	16.0	0.240	0.508
2PC1	-30.6	14.9	0.213	0.380
2PO5	-29.0	15.5	0.268	0.490
2PO	-28.3	25.0	0.231	0.481
2PM	-27.8	17.5	0.224	0.514
2PC	-32.5	18.0	0.235	0.503
2LM	-29.9	12.0	0.260	0.503
2L35	-33.3	17.5	0.244	0.440
2L75	-33.9	19.5	0.138	0.429
3PAO	-22.5	12.0	0.117	0.499
3PAR	-27.0	7.0	0.154	0.367
3PC1	-34.4	7.0	0.114	0.404
3PO5	-24.5	12.2	0.198	0.431
3PO	-22.0	17.5	0.231	0.545
3PM	-29.4	17.5	0.214	0.510
3PC	-33.0	12.5	0.186	0.465
3L35	-27.5	16.0	0.220	0.521
3L75	-25.3	25.0	0.211	0.507
10PAO	-23.5	13.0	0.204	0.523
10PAR	-22.5	13.5	0.244	0.601
10PC1	-34.9	8.8	0.109	0.375
10PO5	-27.8	9.2	0.109	0.388
10PO	-28.5	14.0	0.212	0.508
10L35	-31.0	15.0	0.245	0.477
10L75	-32.0	19.5	0.160	0.444
12PAO	-24.0	13.0	0.222	0.625
12PAR	-25.0	11.5	0.195	0.521
12PC1	-30.1	11.0	0.123	0.402
12PO5	-23.1	13.4	0.211	0.483
12PO	-21.5	25.0	0.203	0.554
12L35	-28.0	25.0	0.268	0.657
12L50	-27.3	12.5	0.182	0.521
12L75	-28.3	11.5	0.191	0.525

Tg: glass transition temp., Tsp: softening temp., ML & MH: slopes of the expasion curve below and above Tg, repectively

Table A.26. Water sensitivity of mixes in Iowa study (Enüstün et al., 1990).

Project	One Year Old Core Samples		
	% Air	RM ratio	ITS ratio
AC-5			
4	5.26	1.05	0.93
5	4.91	0.95	0.95
7	2.81	1.29	1.26
8	2.77	1.38	1.31
AC-10			
1	3.83	1.10	1.16
11	4.04	0.39	0.52
AC-20			
2	6.43	0.96	1.05
3	5.52	1.10	1.09
10	7.19	1.04	0.75
12	5.46	1.26	1.00

Table A.27. Regression analyses: Physical properties against TMA and HP-GPC parameters (n = 73) in Iowa study (Enüstün et al., 1990).

Dependent Variables	TMA & HP-GPC parameters		Selected variables from stepwise reg.	
	P-value	R**2	TMA parameters	HP-GPC parameters
Rheological properties				
P5	0.0001	0.666	**Tsp**	**X2**,X6,**X7**
P25	0.0001	0.669	Tsp,ML,MH	**X4**,X6,**X7**
P4	0.0001	0.667	Tsp	**X2**,X4,X6,**X7**,X8
VIS25	0.0001	0.766	**Tsp**	X2,**X4**,X6,**X7**,MWT,PIDX
CF	0.0001	0.741	Tg,Tsp,**ML**,MH	X2
SI	0.0001	0.719	Tg,**Tsp**,ML,MH	**X2**
VIS60	0.0001	0.583	**Tsp**	X8
VIS135	0.0001	0.773	**Tsp**,ML,MH	**X7**
SP	0.0001	0.715	**Tsp**,ML,MH	**X1**,**X3**,**X6**,MWT
Temperature susceptibility				
PR	0.0001	0.636	Tsp	X2,**X4**,X5
PI	0.0517	0.309	Tg,Tsp	**X4**,PIDX
CN	0.2076	0.246	Tsp	X5
VTS	0.5121	0.187		X3
PVN60	0.0098	0.368		X2,X8
PVN135	0.0001	0.496		**X2**,**X8**
Low-temperature cracking properties				
CT	0.0008	0.438	Tg	X2,**X5**,**X7**,X8
TES	0.0001	0.564	Tg	X2,**X7**
S23	0.0001	0.655	**Tsp**,ML,MH	**X2**,X4,**X7**,PIDX
S29	0.0001	0.588	Tsp,ML,MH	X2,X5,**X7**

Bold face indicates significantly correlated variable.

Table A.28.
Asphalt properties associated with acceptable and unacceptable performance (Finn et al., 1990)

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 A.29.
Asphalt properties for acceptable performance based on analysis of Sisko & Brunstrum data (Finn et al., 1990).

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

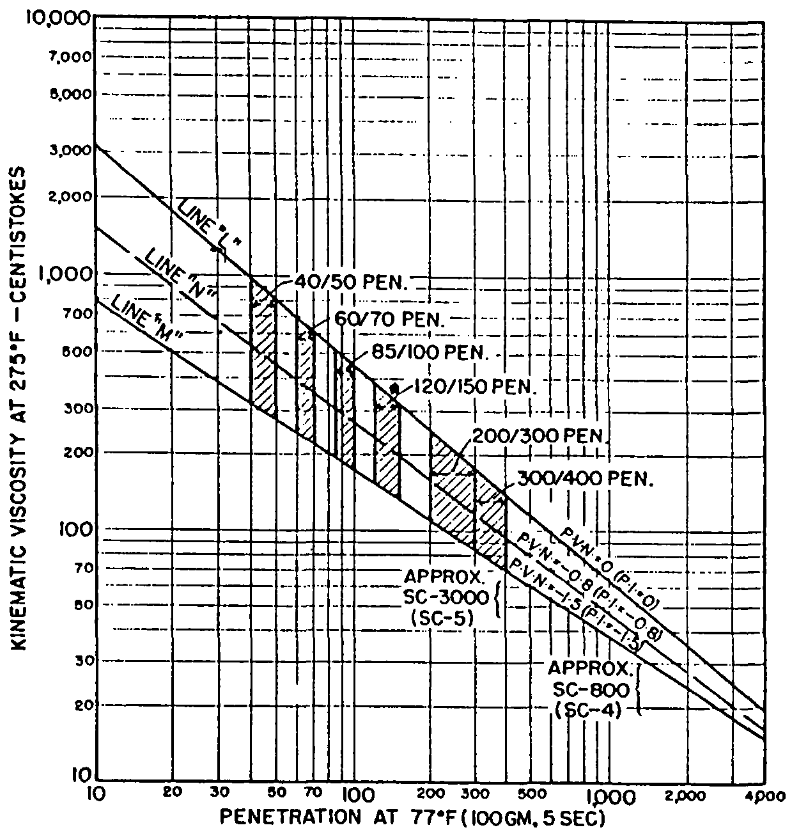

Figure A.1. Correlation between Viscosity in Centistokes at 275°F and Penetration @ 77°F (McLeod, 1972).

This procedure is to measure moisture susceptibility of asphalt concrete specimens tested in indirect tension following special conditioning procedures.

Preparation of Test Specimens.

1. (a) Laboratory or Field-Mixed Specimens: Prepare at least 6 Marshall testing specimens for dry and wet condition testing. Compaction of the test specimens shall be completed using Marshall impact compactor. Compactive effort for specimens to be used for measuring moisture susceptibility shall be less than 100% of design density based on the job-mix formula. (For example, 20 compactive blows per face may be appropriate rather than 50 blows per face).

(b) Field Core Specimens: Select randomly at least 6 cores for dry and wet condition testing. Specimens shall be cut neatly using a masonry saw.

Procedure for Determination of Tensile Strength Ratio

2. Indirect Tension Test for Dry-Conditioned Specimen.

(a) Measure the height and weight of each specimen after specimens have attained room temperature (77°F). Submerge each specimen in water for 3 minutes and record submerged weight to the nearest 0.1gm. Remove each specimen from the water bath and record the saturated-surface-dry (SSD) weight to the nearest 0.1gm. Compute the bulk volume of the specimen as: SSD weight - submerged weight.

(b) Measure indirect tensile strength at 77°F for 3 randomly-selected specimens. Apply load using Marshall testing machine by rate of 2 inches per minute through a 0.5-inch-wide metal strip. The indirect tensile strength of each specimen will be

$$\text{ITS} = \frac{2(\text{Max. Load Applied})}{(\pi)(\text{Height})(\text{Diameter})} = \qquad (\text{psi})$$

3. Indirect Tension Test for Wet-Conditioned Specimen

(a) Measure the height, weight, submerged weight and saturated-surface-dry weight SSD (1) according to procedure 2.(a).

(b) Place each specimen in a distilled water bath (77°F) and vacuum saturate it for 5 min. at 20"Hg vacuum.

(c) Obtain the saturated-surface-dry weight SSD (2) to the nearest 0.1gm and calculate;

Absorbed water = SSD (2) - SSD (1)

% saturation = (Absorbed water/Volume of air) x 100% *

(d) Place the same specimens or core samples for moisture conditioning in a 140°F distilled water bath for 24 hours and then in a 77°F distilled water bath for 1 hour before conducting indirect tensile strength test at 77°F.

(e) Find submerged weight and the saturated-surface-dry weight SSD (3) and calculate;

Volume of specimen = SSD (3) - Submerged weight (unit=cm3)

Absorbed water = SSD (3) - SSD (1)

% saturation = (Absorbed water/volume of air content) x 100% *

The specimen must be saturated in the range of 55 to 80 %.

(f) Measure indirect tensile strength of specimen at 77°F for 3 randomly selected and wet-conditioned specimens.

3. (a) Calculate the tensile Strength Ratio (TSR) of each specimen.

$$\text{TSR} = \frac{\text{ITS wet}}{\text{ITS dry}} \times 100\ \%$$

(b) Average TSR = (Average ITS wet / Average ITS dry) x 100 %

* The volume of air (cm^3) in specimens should be found beforehand using ASTM D 2041, Theoretical Maximum Specific Gravity of Bituminous Paving Mixtures.

Figure A.2. Field and laboratory determination of moisture susceptibility based on retained strength of asphalt concrete mixture in South Carolina study (Busching et al., 1986).

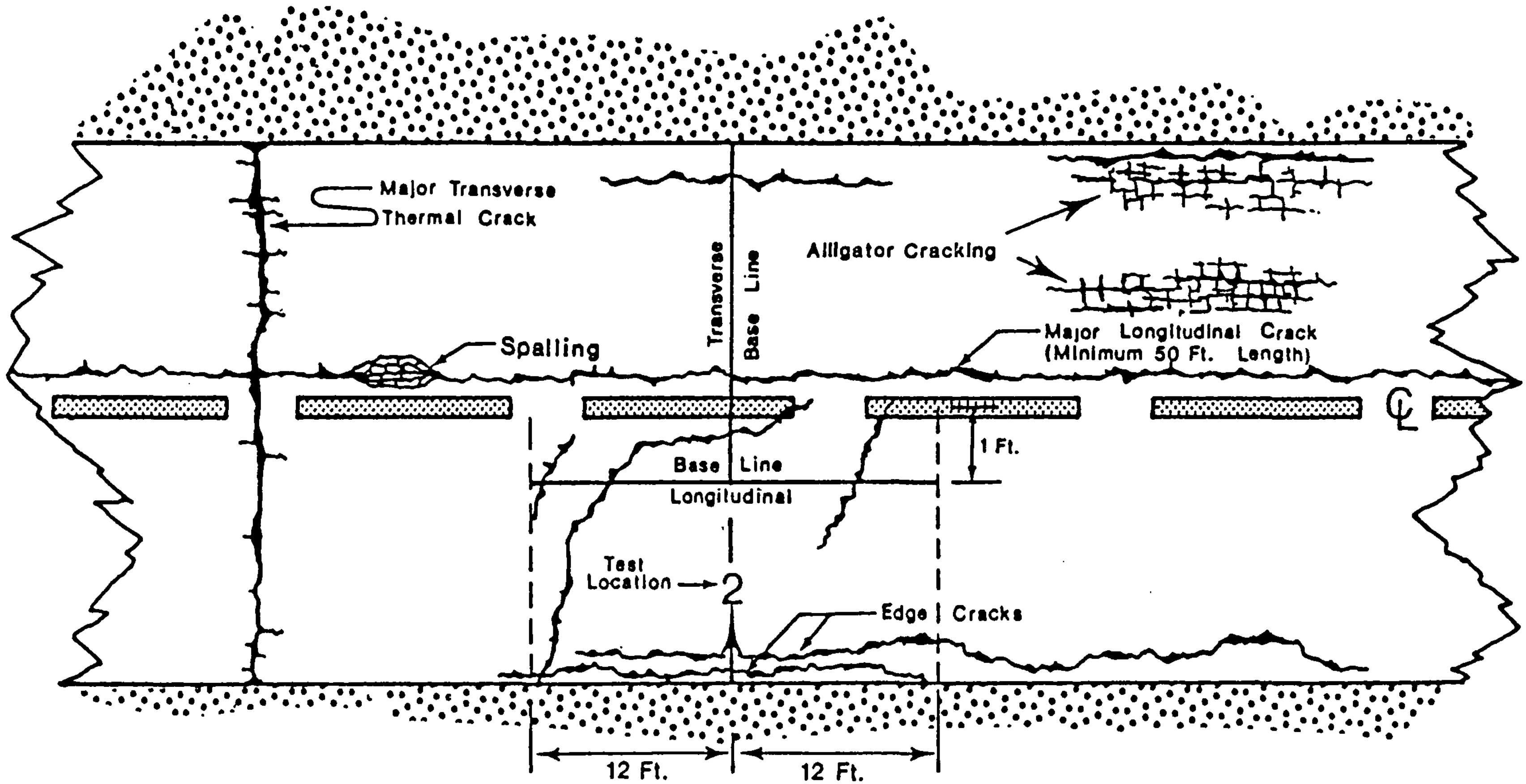

Figure A.3. Illustration of crack type used in Alaska study (Source: McHattie, 1981)

Appendix B

This appendix contains a list of the various test methods available for determining asphalt composition. This list has been adapted from Goodrich et al. (1986).

TESTS USED TO ANALYZE THE COMPOSITION OF ASPHALT	FRACTION NAMES
1. SOLVENT PRECIPITATION a. Solvent precipitation Traxler & Schweyer (1936)	n-butanol asphaltics cyclics paraffinics
2. CHEMICAL PRECIPITATION a. Solvent precipitation + chemical precipitation Rostler & Sternberg (1949) Rostler & White (1959)	asphaltenes (n-pentane insolubles) n-pentane solubles nitrogen bases 1st acidaffins 2nd acidaffins paraffins (saturates)
3. LIQUID CHROMATOGRAPHY--absorption Altgelt (1975) a. Solvent precipitation + absorption chromatography Corbett (1985) [column packing: alumina]	asphaltenes (n-hexane insolubles) n-hexane petrolenes polar aromatics naphthene aromatics saturates
ASTM D 4124 [column packing: alumina]	asphaltenes (n-heptane insolubles) n-heptane petrolenes polar aromatics naphthene aromatics saturates
ASTM D 2007 "Clay-Gel" [ASTM D 2007] [column packing: attapulgus clay, silica gel]	asphaltenes (n-pentane insolubles) polar compounds aromatics saturates
b. HPLC (high pressure LC) Hattingh (1984) [column packing: silica gel] Boduszynski (1985)	Hattingh separated asphalt into ten solvent-defined fractions. Boduszynski used HPLC-FIMS to identify homologous series of specific classes found in asphalt residues.
Fish (1984)	Fish used reversed-phase HPLC (RP-HPLC) with element selective graphite atomic absorption to determine vanadium and nickel distribution profiles on heavy crudes and asphaltenes.
4. LIQUID CHROMATOGRAPHY--ion exchange a. Anion-exchange chromatography Boduszynski (1980, 1977a, 1977b) McKay (1977)	Separated the acidic components (containing carboxylic acids, phenols, pyrroles, 2-quinolones) into four solvent-defined fractions.
b. Cation-exchange chromatography Boduszynski (1980, 1977a, 1977b) McKay (1977)	Separated the basic components (containing pyridines, amides and pyrrolas) into four solvent-defined fractions.
5. LIQUID CHROMATOGRAPHY--coordination a. Ferric chloride coordination chromatography Boduszynski (1980, 1977a, 1977b) McKay (1977)	Separated the neutral Lewis bases from acid and base-free asphalt (see ion-exchange LC). IR analysis showed that amides and pyrroles were the dominant polar groups in this fraction.

TESTS USED TO ANALYZE THE COMPOSITION OF ASPHALT	FRACTION NAMES
6. THIN-LAYER CHROMATOGRAPHY (TLC)	
a. Thin-Layer Chromatography (TLC) Masek (1977) Altgelt (1975)	TLC has been used to detect the presence of benzo-[a]pyrene in distillates. HPLC has substantially replaced the two-dimensional TLC techniques.
Poirier (1983) Ray (1982) Golavko (1984) Yamamoto (1984)	Poirer and Ray report using TLC "chromarods," quartz rods with a sintered layer containing silica gel or alumina as the absorption medium. The "chromarods" are scanned by the flame of a flame ionization detector. The method has been compared to a D 2007 separation. [152]
7. GAS-LIQUID CHROMATOGRAPHY (GLC)	
a. Gas chromatography Altgelt (1975) Botvin'ava (1982) Puzinauskas (1975)	Very useful for analyzing low-to-medium boiling range hydrocarbons. GC was used to analyze the hydrocarbon emissions from asphalt hot mixes. GC can be used to identify the "light ends" from a vacuum distillation of asphalt. GC coupled with mass spectroscopy is a powerful analytical tool for identifying compounds at very low concentrations.
b. Inverse gas-liquid chromatography (IGLC) Robertson (1971) Davis & Peterson (1967) Dorrence & Peterson (1974) Boduszynski (1977a)	Used asphalt and asphalt fractions as the stationary phase (coated on an inert fluorocarbon support material). Measurements were made of the interaction of various model compounds with the stationary phases. IGLC tests on Zaca-Wigmore asphalts correlated well with road performance.
8. SIZE EXCLUSION CHROMATOGRAPHY (SEC)	
a. Gel permeation chromatography (GPC) Altgelt (1970a, 1970b, 1965) Long (1979) Dickie & Yan (1967) Marvillet (1975) Winniford (1960)	Separates compounds based on molecular aggregate size--largest size is aluted from the Styragel column first. Time defined fractions are collected. Used to estimate molecular mass based on the retention time of polystyrene standards.
b. High pressure GPC (HP-GPC) Bynum (1970) Hattingh (1984) Winniford (1960) Monin (1984) Boduszynski (1981, 1984, 1977b)	HP-GPC chromatograms result in an apparent molecular aggregate size profile of the sample. Winniford and Monin cautioned about asphaltene associations (aggregations) in dilute solvent solutions. Molecular masses within an asphalt as measured by GPC range from 300 to 100,000 polystyrene equivalent weight.
Jennings (1981,1977,1980,1984)	Jennings believes that road cracking is related to the "LMS" fraction determined from the GPC chromatogram.
Plummer (1984)	Plummer finds that the cracking is not related to a large/small molecular size imbalance; rather, that cracking is related to the mean molecular size. Both Jennings and Plummer find that
8. SIZE EXCLUSION CHROMATOGRAPHY (SEC)	

TESTS USED TO ANALYZE THE COMPOSITION OF ASPHALT	FRACTION NAMES
(continued)	the cracked roads have harder asphalts (penetration/ductility).
Blanchard (1983)	Blanchard looked for correlations between HP-GPC distributions and Tg.
Chollar (1985)	Chollar concluded that "extreme caution is warranted when considering HP-GPC as a tool for predicting the potential low-temperature behavior of any particular asphalt."
9. VAPOR PRESSURE OSMOMETRY (VPO) a. VPO (i.e., ASTM D 3592) Boduszynski (1981) Ferris (1967) Speight (1979, 1978) Long (1979) Marvillet (1975)	Used to estimate molecular weight by measuring the heat of absorption of solvent vapor into a solution of the asphalt in the same solvent. Average molecular mass of D 4124 asphalt fractions by VPO: saturates = 1000, asphaltenes = 4000.
10. MASS SPECTROMETRY (MS) a. low resolution MS Clerc (1960) b. high resolution MS Dickie & Yan (1967) Gallagos Puzinauskas (1975) c. field ionization MS (FIMS) Boduszynski (1981, 1980, 1984, 1985) d. field desorption MS (FDMS) Boduszynski (1985)	Mass spectrometry is used to determine the mass of individual molecules (parent ions) and ionized fragments resulting from a sample being bombarded with electrons. For low molecular weight compounds, the MW of the fragments can be used to estimate the composition of the sample. For higher molecular weight materials such as asphalt, the MS spectra is an unresolved envelope of countless molecular weight fragments FIMS produces unfragmented molecular weight fragments FIMS produces unfragmented molecular ions and their isotope signals. It is ideally suited to measurement of molecular weight. The FI mass spectrum can provide a molecular weight profile of asphalt or asphalt fractions. By FIMS, asphalts include compounds in the 300 to 2000 MW range (average molecular mass of D 4124 asphalt fractions: saturates = 1000, asphaltenes = 900).
11. ELECTROPHOTOMETRIC SPECTROSCOPY a. Infrared spectroscopy and differential IR spectroscopy Dorrence & Petersen (1974) Petersen (1981, 1975a, 1975b) Barbour (1974) Martin (1981) Boduszynski (1980) Plancher (1976) Speight (1978) Puzinauskas (1975)	Petersen et al. have developed methods for quantitatively measuring the types of polar compounds in asphalt which absorb in the carbonyl region (1850 to 1640 cm^{-1}). Measurement of ketones, 2-quinilones, carboxylic acids and dicarboxylic anhydrides allowed and estimation of the mechanism of asphalt oxidation and viscosity increase. Sulfoxides were measured at 1030 cm^{-1}. Speight used IR to indicate hydrogen bonding in asphalt solutions.

TESTS USED TO ANALYZE THE COMPOSITION OF ASPHALT	FRACTION NAMES
b. Ultraviolet spectroscopy Botvin'ava (1982) Corbett (1958) Puzinauskas (1975)	Botvin'ava studied saturated hydrocarbons (n-, iso- and cyclo-paraffins). Absorption at 270, 286, and 258 m-microns was used to detect 1, 2 and 3 ring aromatics in the saturate and naphthene aromatic fractions.
12. NUCLEAR MAGNETIC RESONANCE (NMR) SPECTROSCOPY a. NMR Spectroscopy Couper (1983) Plummer (1984) Corbett (1960, 1958) Paukku (1981) Ferris (1967) Altgelt (1970c,1970a,1970b) Delpusch (1985)	Both proton and carbon-13 NMR have been used to estimate saturate/aromatic carbon ratios. Boduszynski cautions against proton NMR when analyzing asphalt fractions. Paukku reports determining the relative distributions of methyl, methylene and methynal groups, saturated groups in alpha position with respect to the aromatic ring and aromatic structures by integration of the PNMR bands in the regions of 0.5-1, 1-2, 2-5, and 6-8.5 ppm, respectively.
13. ELECTRON SPIN RESONANCE (ESR) SPECTROSCOPY (ELECTRON PARAMAGNETIC RESONANCE (EPR) SPECTROSCOPY)	Measures the abundance of free radical sites.
a. ESR AND EPR Spectroscopy Ferris (1967) Yan (1966) Corbett (1958) Melhotra (1985)	Ferris found a correlation between nitrogen content and ESR peaks. Malhotra used 35GHz EPR to identify the nature of the vanadium coordination sites in asphaltenes.
Reynolds (1985)	Reynolds, using EPR, found that the vanadium-hetaro-atom coordination spheres in D 2007 resin and asphaltene fractions were dominated by 4-nitrogens (resins) or nitrogen, oxygen and 2-sulfurs (asphaltenes).
14. SPECTROCHEMICAL ANALYSIS	
a. X-ray fluorescence spectroscopy	Used to determine heavy metal content.
b. Neutron activation analysis	Provides elemental analysis.
c. X-ray diffraction Ferris (1967) Yan (1966)	Used to estimate the structure of asphaltenes.
d. Atomic absorption Fish (1984)	Used to determine heavy metal content of asphalts and asphaltenes. There are various methods, such as Inductively Coupled Plasma atomic absorption (ICPAA) and Graphite Furnace atomic absorption (GFAA).

TESTS USED TO ANALYZE THE COMPOSITION OF ASPHALT	FRACTION NAMES
15. ELEMENTAL ANALYSIS a. Elemental analysis Generally mentioned in literature; specifically discussed by: Marvillet (1975) Reynolds (1985)	A variety of techniques are used to determine the elemental analysis of asphalt. Elements most often measured include hydrogen, carbon, nitrogen, sulfur and oxygen. The analysis usually involves pyrolysis (combustion or reduction), purification of the resultant gasses, and detection by IR, coulometric titration, chemilumin-escence, etc.
16. DISTILLATION FRACTIONATION a. Short-path, high vacuum distillation Boduszynski (1985)	Permits fractionation up to approx-imately 1300F. Thus a large portion of asphalt can be fractionated for further analysis.
b. Thermogravimetric analysis Boduszynski (1985)	Obtains a distillation profile on a milligram scale sample (may be under vacuum)...effluent can be in-line detected/analyzed.
17. WAX CONTENT a. Methylene chloride Evans (1971)	Wax has the opposite effect of asphaltene on asphalt rheology: wax decreases the PVN (77°-140°F), making asphalt more temperature susceptible.
18. PHOTOCHEMICAL REACTIONS OF ASPHALT a. Traxler procedure (UV box) Traxler (1963) Predoehl (1978)	Thin films of asphalt (5 microns) showed an increase in viscosity correlated to RTFC viscosity increases. Thick films (20 microns) showed little difference between asphalts of differing composition (D 2007).
19. ACID NUMBER a. Neutralization number (ASTM D 664) Marvillet (1975)	May have an effect on asphalt emulsification.
20. INTERNAL DISPERSION STABILITY a. Heithaus parameter, P Heithaus (1960)	The Heithaus "state of peptization" (P) is an attempt to measure the intercompatibility of the components in asphalt; how good a dispersant are the maltenes for the asphaltenes (Po)? How readily are the asphaltenes dispersed (Pa)? P = po/(1-Pa).
Petersen (1984)	Petersen commented that the "P" of the Zaca-Wigmore asphalts correlated with performance better than did the Rostler ratio.
b. Oliensis Spot Test Oliensis (1957, 1933) Heithaus (1959)	This test was designed to identify the presence of cracked asphalt, resulting in a positive spot. Considered by Heithaus, in 1959, to be no longer a useful test: a 24-hour test did correlate with road performance.

TESTS USED TO ANALYZE THE COMPOSITION OF ASPHALT	FRACTION NAMES
20. INTERNAL DISPERSION STABILITY (Continued) c. Asphaltene settling test Plancher (1979)	In this test a 2 gm sample of asphalt is digested into 50 mls of n-hexane for 24 hours. The settlement of the asphaltenes in an undisturbed cylinder is then observed. Settling time is defined by when the asphaltenes have settled to half the solution height. Test precision is a problem.
d. Solubility profiles Hagen (1984)	Solubility parameters (profiles) were determined on several asphalts.
21. TITRIMETRIC/GRAVIMETRIC ANALYSIS Chevron (1964)	A wealth of analytical techniques is available for the analysis of asphalt. Although many of the "wet" chemistry techniques have been replaced with instrumental analysis, still others survive: i.e., acid number (by titration), salt or mineral filler content (by ash), water content (Karl Fischer titration).

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