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CONTENTS

FOREWORD v

ASPHALT PROPERTIES IN RELATION TO
PAVEMENT PERFORMANCE
Prithvi S. Kandhal and Monroe E. Wenger 1

PAVING ASPHALT PROPERTIES AND PAVEMENT DURABILITY
James M. Roberts and William H. Gotolski 14
Discussion
William H. Clark III 25
Authors' Closure 26

ASPHALT BINDER HARDENING IN THE MICHIGAN TEST ROAD
AFTER 18 YEARS OF SERVICE
L. W. Corbett and R. E. Merz 27

USE OF ASTM TESTS TO PREDICT LOW-TEMPERATURE
STIFFNESS OF ASPHALT MIXES
R. J. Schmidt 35
Discussion
Richard L. Davis 45

COMPOSITIONAL CONSIDERATIONS OF ASPHALT
FOR DURABILITY IMPROVEMENT
Rowan J. Peters 46

EVALUATION OF PROPERTIES OF AC-20 ASPHALT CEMENTS
Prithvi S. Kandhal and Monroe E. Wenger 56

SPONSORSHIP OF THIS RECORD 65

FOREWORD

A goal of the asphalt paving technologist is to predict the performance of an asphalt pavement on the basis of well-defined properties that can be measured in the laboratory. Because of the many variables that influence the performance of asphalt pavements, some poorly defined and some uncontrollable, quantitative relationships between asphalt properties and pavement performance have been difficult to establish. However, as indicated by the papers in this RECORD, research on the property-performance relationships of asphalt continues to receive major emphasis because of the potential benefits of increased pavement life.

The papers fall into two categories. The first four papers were presented at a symposium at the 54th Annual Meeting. The remaining two papers report laboratory studies of asphalt properties that, although not correlated with field studies, are important to field performance. All papers should be of value to material engineers and asphalt technologists who are concerned with the development of paving mixtures with improved performance properties and service life.

The paper by Kandhal and Wenger on asphalt property-performance relationships describes changes in the physical properties of six recovered asphalt cements as a function of time in service. These changes were correlated with pavement performance as judged by an evaluation team. The rate of increase in shear susceptibility relative to the increase in viscosity appeared related to pavement performance.

Roberts and Gotolski report results of their extended 12-year study of the properties of recovered asphalts taken from 13 pavements in Pennsylvania. The time of the year when pavements were sampled affected test results. Pavement air void volume is shown to be a major factor influencing pavement durability, and a number of recommendations are made for improving durability.

Asphalt binder hardening in the well-known Michigan Road Test is reported by Corbett and Merz. Changes in the component composition of the binder after 18 years of service were determined and related to the resultant binder hardening during service and to changes in wear, weathering, and raveling qualities of the road test sections.

The paper by Schmidt correlates the low-temperature stiffness of asphalt mixes as measured by creep tests with the low-temperature stiffness estimated from ASTM penetration and viscosity measurements on rolling thin film oven residues and asphalts recovered from mixes. Low-temperature mix stiffness was related to thermally induced pavement cracking. A chart is given for determining low-temperature stiffness of asphalts from normal ASTM penetration tests.

Peters describes laboratory studies of the blending of asphalts to achieve desired aging index and viscosity properties in the resultant blends. The chemical reactivity ratio correlated with the laboratory hardening rate of the base stock asphalts used in the study.

In the final paper by Kandhal and Wenger, the physical properties of 20 AC-20 asphalt cements were cataloged and evaluated to assist in the future development of specifications based on fundamental units of measurement. Correlations between a number of the properties were developed, and the suggestion is made that shear susceptibility be used to replace ductility as a specification requirement.

—J. Claine Petersen

ASPHALT PROPERTIES IN RELATION TO PAVEMENT PERFORMANCE

Prithvi S. Kandhal and Monroe E. Wenger,
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Six asphalt cements from different sources were used during 1964 in the construction of the pavements studied and reported in this paper. These asphalt cements ranged in viscosity at 140 F (60 C) from 966 to 2,649 poises (96.6 to 265 Pa·s) and in penetration from 62 to 149 units. Tests have been conducted to determine the properties of the original asphalts, as well as the asphalts recovered from time to time from the pavements in service. The performance of these test pavements was evaluated in 1974 by a team of eight evaluators. Changes in the asphalt properties have been observed to be a hyperbolic function of time. According to this study, viscosity or shear susceptibility of the aging asphalt alone does not, necessarily, indicate the pavement performance. The rate of gain in shear susceptibility relative to increase in viscosity at 77 F (25 C) seems to be one of the major factors affecting pavement performance. Asphalt ductility values, determined at 39.2 F (4 C) before and after pug mill mixing, seem to be consistent with the pavement performance observed so far. Higher ductility values are associated with better pavement performance.

•THE ability of a bituminous pavement structure to support traffic-induced stresses and strains as well as adverse effects of climatic conditions during its service life depends on the rheological properties of the pavement system. These properties in turn are related to the properties of the constituents of the bituminous mixture. Most durability studies in the past have been confined to analyzing the properties of aging asphaltic binder since it is this main constituent that contributes to the cohesiveness and adhesiveness of the mixture and thus affects pavement performance. In this study, effort has been directed toward evaluating and characterizing those physical properties that are associated with aging and their relationship to pavement performance.

DETAILS OF TEST PAVEMENTS

The six test pavements, totalling 3.67 miles (5.3 km) in length, are located in Clinton County on LR 219 (US-220) between Mill Hall and Beech Creek, Pennsylvania. The original pavement consisted of 18 to 20 ft (5.5 to 6.1 m) in width of 8-in. (203-mm) reinforced concrete, which was constructed in 1929 and 1934. This pavement was resurfaced with 2 in. (51 mm) of ID-2 binder and 1 in. (25 mm) of ID-2 wearing course during October 1964. This study is limited to an evaluation of six experimental sections of dense-graded ID-2 bituminous wearing course surfaces, each containing a different asphalt. An asphalt of 70 to 85 penetration grade was used in the binder course throughout the entire project. Average daily traffic on this road is 4,200 vehicles.

Properties of the six asphalts used in the project are given in Table 1. Excellent control was maintained throughout the project to ensure uniform construction of these six pavements. Limestone aggregate gradation and asphalt content (6.4 percent optimum) were held consistent. Details of design and construction of these pavements are given elsewhere (1). In this closely controlled research project, the only significant

Table 1. Asphalt test properties.

Property	Asphalt 1	2	3	4	5	6
Before Pug Mill Mixing						
Viscosity, poises						
At 140 F	1613	1544	1447	966	2200	2649
At 275 F[a]	339.6	343.0	475.4	318.5	509.4	556.7
At 39.2 F at 0.05 sec^{-1}	1.19×10^9	2.65×10^8	4.22×10^7	9.50×10^7	1.68×10^8	2.57×10^8
At 77 F at 0.05 sec^{-1}	3.05×10^6	1.06×10^6	4.83×10^5	9.15×10^5	1.32×10^6	1.85×10^6
At 115 F at 0.05 sec^{-1}	2.09×10^4	1.54×10^4	1.15×10^4	1.15×10^6	2.19×10^4	2.80×10^4
Shear susceptibility						
At 39.2 F	0.20	0.20	0.35	0.45	0.32	0.32
At 77 F	0.05	0.02	0.11	0.06	0.12	0.09
At 115 F	0.02	0.03	0.04	0.03	0.04	0.02
Penetration						
At 39.2 F, 200 g, 5 sec	9	11	28	19	15	12
At 77 F, 100 g, 5 sec	62	92	149	114	94	80
After Pug Mill Mixing						
Penetration at 77 F, 100 g, 5 sec	36	69	98	66	69	60
Penetration, percentage retained	58	76	67	58	73	76
Viscosity at 140 F, poises	3645	2505	2971	2078	3463	4770
Viscosity ratio at 140 F	2.27	1.62	2.06	2.16	1.57	1.80
Aging index[b] after mixing based on viscosity at 77 F, 0.05 sec^{-1} shear rate	3.3	1.9	2.5	3.3	2.1	1.9
Viscosity at 77 F, 0.05 sec^{-1} determined from aging indexes, poises	1.01×10^7	2.01×10^6	1.21×10^6	3.02×10^6	2.77×10^6	3.52×10^6

Note: 1 poise = 0.1 Pa·s; 1 F = 1.8 C + 32.

[a]In centistokes.

[b]No test data available. These were determined from Fig. 4 of the paper by Halstead and Zenewitz (ASTM, STP. 309, 1962), which gives relation between percent of original penetration and viscosity aging index for thin film test.

Table 2. Pavement condition after 113 months.

Item Observed	Asphalt 1	2	3	4	5	6
Riding quality	Good	Good	Good	Good	Good	Good
Raveling (loss of aggregate larger than ¼ in.)	Moderate	Slight	Slight	Slight to moderate	Slight	Slight to moderate
Spalling	Slight	None	None	None to slight	None to slight	None to slight
Loss of matrix (loss of fines)	Moderate	Slight	Slight	Slight	Slight	Slight
Rutting (in.)	0.20	0.18	0.20	0.19	0.21	0.21
Transverse cracking	Slight	None to slight	None	None to slight	None to slight	None to slight
Longitudinal cracking	Slight to moderate	None	None	None	None	None
Alligator cracking	None	None	None	None	None	None
Surface texture	Average	Closed	Closed	Closed	Closed	Closed to average

Note: 1 in. = 25 mm.

variable is the asphalt type.

Since construction of these six pavements, core samples have been obtained periodically to determine the percentage of air voids in the pavements and rheological properties of the aged asphalts. The last core sampling was done in March 1974, 113 months after construction.

PERFORMANCE OF TEST PAVEMENTS

No differences in texture or color tones of the asphalts were observed when the pavements were visually inspected just after construction and after 1 year of service. Visual evaluation during April 1967 (after 30 months of service) indicated that the road surface was good except for the asphalt 1 section, which showed some raveling.

A rating method suggested by Olsen, Welborn, and Vallerga (2) to evaluate the effect of asphalt aging on pavement condition was used as a guideline in the visual survey of pavement condition. Visual evaluation included riding quality, raveling, spalling, loss of matrix, rutting, cracking (transverse, longitudinal, and alligator), and surface texture. A team of five engineers evaluated these sections during 1971 after 80 months of service (1, 3). For comparison, it should be pointed out that the designations of asphalts 2 and 3, as reported in earlier references, have been interchanged in this report. A performance evaluation was made most recently in 1974 (after 113 months of service) by eight evaluators. Because the differences between some asphalts were relatively small, the rating form was revised to include intermediate levels of observed distress such as none to slight and slight to moderate. The overall ratings obtained on individual pavements were as follows:

Asphalt	Rating
1	51.1
6	59.8
4	60.1
2	60.4
5	61.2
3	61.5

An ideal pavement according to this performance evaluation would rate 72. Brief details of the pavement condition after 113 months in service are given in Table 2. Reflection cracks due to the underlying portland cement concrete pavement were not considered in the evaluation.

ANALYSIS OF RESEARCH DATA

Data on percentage of air voids in the test pavements and rheological properties of the recovered asphalts have been gathered since construction. Changes in the percentage of air voids, viscosity at 77 and 140 F (25 and 60 C), and shear susceptibility at 77 F (25 C) were found to follow the hyperbolic model suggested by Brown, Sparks, and Larsen (4) and confirmed by Lee (5). According to this theory, the changes in these physical properties are a hyperbolic function of time and approach a definite limit with time. Brown and coworkers suggested the following equations to express the hardening of asphalts in the field:

$$\Delta Y = \frac{T}{a + bT} \tag{1}$$

Figure 1. T versus T/ΔY plots for changing asphalt properties.

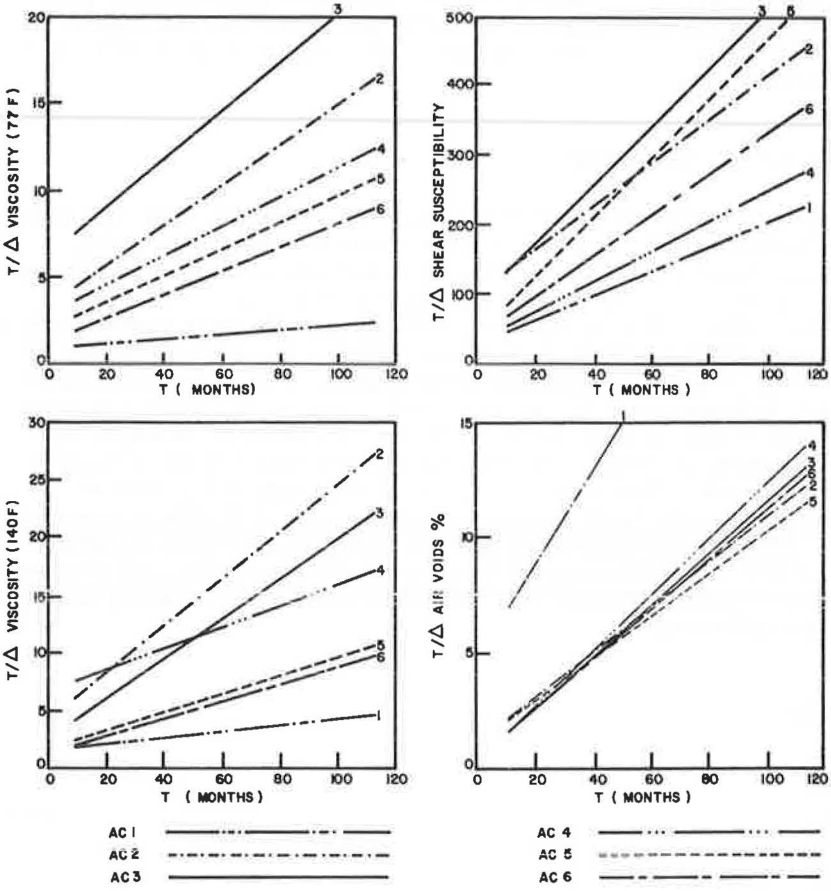


Table 3. Linear regression data (T versus T/ΔY).

	Asphalt					
Test Property Y	1	2	3	4	5	6
Viscosity at 77 F						
r	0.970	0.950	0.834	0.987	0.968	0.950
a	0.495	3.233	5.965	2.740	1.717	0.961
b	0.017	0.116	0.141	0.086	0.081	0.071
Viscosity at 140 F						
r	0.975	0.941	0.974	0.935	0.982	0.961
a	1.531	4.144	2.756	3.447	1.399	1.834
b	0.029	0.207	0.122	0.118	0.081	0.071
Shear susceptibility						
r	0.991	0.929	0.998	0.998	0.993	0.999
a	27.523	104.914	86.351	34.407	45.151	42.243
b	1.740	2.993	4.225	2.197	4.171	2.890
Percentage of air voids						
r	0.982	0.998	0.999	0.999	0.999	0.999
a	5.155	1.178	0.328	0.389	1.080	0.599
b	0.207	0.099	0.117	0.123	0.095	0.111

Note: 1 F = 1.8 C + 32.

or

$$\frac{T}{\Delta Y} = a + bT \tag{2}$$

where

ΔY = change in test property (such as viscosity, penetration, shear susceptibility) with time T or the difference between the zero-life value and the value for any subsequent year,
T = time,
a = intercept of the equation 2 line on the ordinate,
b = slope of the equation 2 line, and
$1/b$ = ultimate change (limiting value of change) of the property at infinite time.

The terms a and b are constants: a is primarily a measure of the rate of change or degree of curving of the hyperbola, and b is the measure of the ultimate magnitude of the variable at infinite time. $T/\Delta Y$ is the reciprocal of the overall average rate of change of the inspection value over the life period T. In this form, equation 2 is recognized as linear in $T/\Delta Y$ versus T.

Experimental data on viscosity, shear susceptibility, and percentage of air voids were fitted to equation 2 by the least squares linear regression method (Figure 1). Almost without exception, the fit, as indicated by correlation coefficient r, was excellent. The values of a, b, and r are given in Table 3. Change in percentage of air voids has been regarded as positive for plotting, even though percentage of air voids decreases with age of the pavement.

Inasmuch as equations 1 and 2 have only two constants, the mathematical solution depends on determination of Y at zero life and at any two known later T-values. Hence, inspection test values both at time zero, or immediately after compacting, and after the first and second years of aging suffice to determine the course of the whole hyperbolic relationship between inspection test values and time, including the limiting value to be approached after extreme aging. Thus, instead of analyzing a few samplings each year for a prolonged period, sampling during the early life of the pavement should be increased. By doing so, the early life values will be statistically more valid, and, from these, the changes to be experienced over later years can possibly be calculated without waiting out the time (4).

However, it has been observed that core samples must be obtained from the test pavements during the same time of the year. Most core samples in this study were obtained during the spring. The data from core samples obtained during fall in a few instances did not fit equation 1 or 2. No explanation can be given for this. It has also been observed by Gotolski and Roberts (6) on several experimental projects.

Figure 2 shows the percentage of air voids in the test pavements versus time. The curves in this and later figures showing change of properties with time were plotted based on data from the straight-line relationship ($T/\Delta Y$ versus T) shown in Figure 1. The following equation was used in conjunction with equation 1 or 2:

$$Y_T = Y_0 + \Delta Y$$

where

Y_T = value of property after time T,
Y_0 = value of property at zero time, and
ΔY = change in test property in time T determined from equation 1 or 2 by using constants a and b.

Figure 2. Air voids versus time in months.

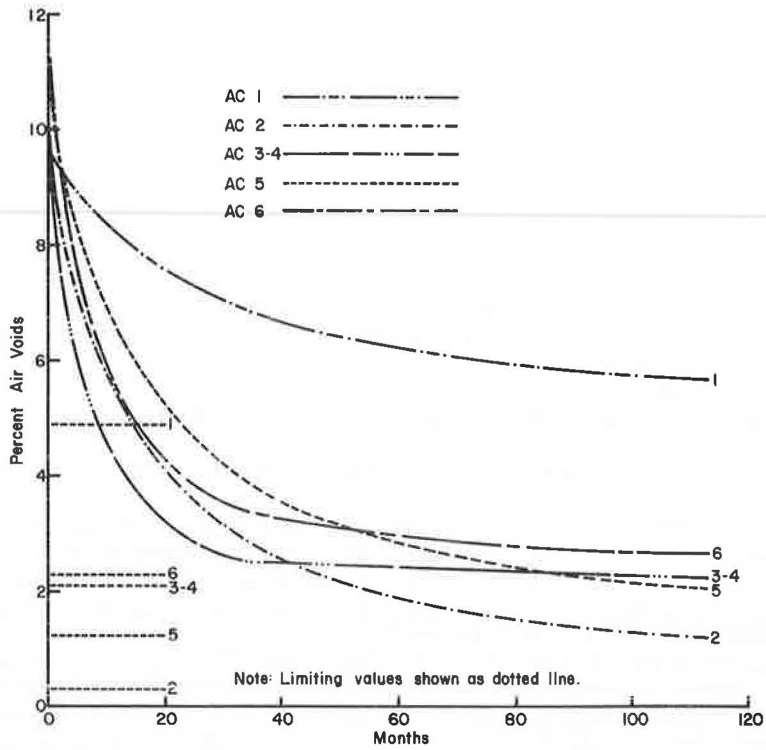
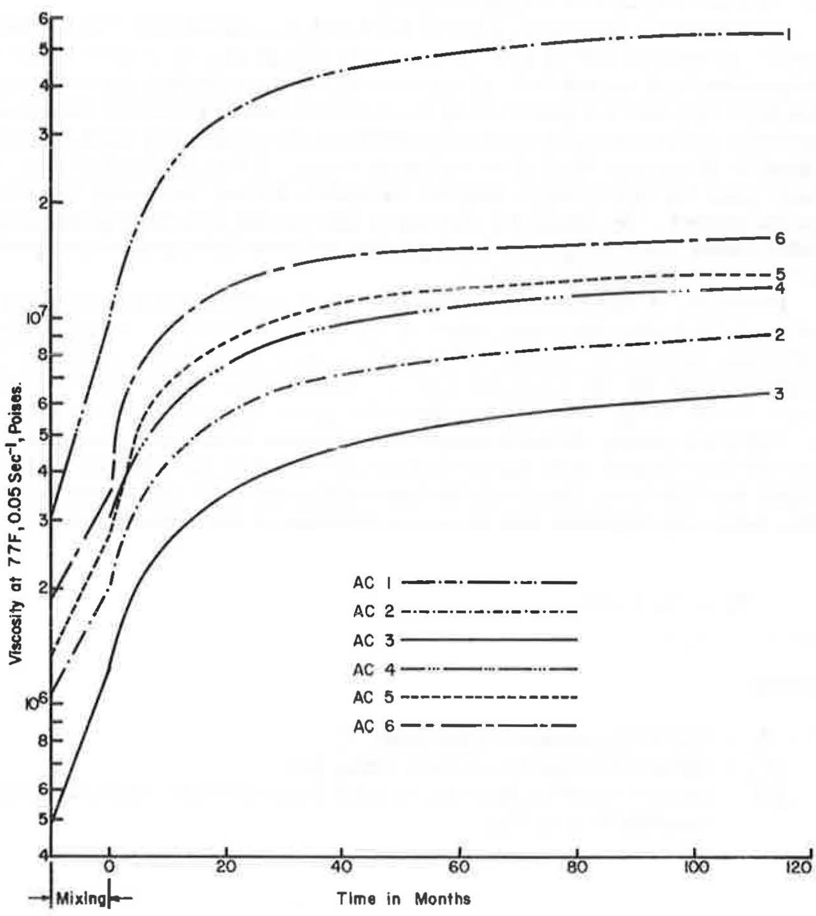


Figure 3. Viscosity at 77 F, 0.05 sec^{-1} versus time.



RHEOLOGICAL PROPERTIES OF ASPHALTS

Viscosity at 77 F (25 C)

Viscosity at 77 F (25 C) was determined for shear rates of 0.05 and 0.001 sec^{-1} by using the sliding plate microviscometer designed by Shell Oil Company. Both glass and stainless steel plates were used. A plot of viscosity at 77 F (shear rate of 0.05 sec^{-1}) versus time is shown in Figure 3.

Viscosity at 140 F (60 C)

Viscosity at 140 F (60 C) was determined by using the Cannon-Manning Vacuum Viscometer. A plot of viscosity at 140 F versus time is given in Figure 4.

Shear Susceptibility at 77 F (25 C)

The shear susceptibility (or shear index) value as used in this study is the tangent of the angle of log shear rate versus log viscosity determined with the microviscometer. Shear susceptibility values were determined for the six asphalts after increasing periods of aging. Results are shown graphically in Figure 5. The relationship between shear susceptibility and viscosity at 77 F is shown in Figure 6.

Aging Index

Considering the changes in viscosity at 77 F (0.05 sec^{-1} shear rate), aging indexes were determined from the following:

$$\text{Aging index} = \frac{\text{viscosity after aging}}{\text{viscosity before aging}}$$

Use of the aging index tends to eliminate the variability caused by differences in the viscosities of the original asphalts and gives a clearer picture of the hardening rate.

Figure 7 shows the relationship between shear susceptibility and aging index of the aged asphalts.

DISCUSSION OF RESULTS

Pavement Performance in Relation to Its Physical Properties

Initial air voids in the six pavements, when constructed, were within the permissible range in terms of Marshall design and control criteria. According to these criteria, the pavements are expected to compact under traffic during the first 1 to 2 years to less than 5 percent air voids. For these six test pavements, the type and gradation of aggregate, asphalt content, asphalt-filler ratio, traffic, and climate were identical. However, the asphalt 1 pavement offered maximum resistance to compaction under traffic. After 9½ years in service it still has more than 5 percent air voids (Figure 2). Asphalt 6 is second to asphalt 1 in offering resistance to densification by traffic. However, pavements using asphalts 2 through 6 were compacted under traffic below 5 percent air voids during the first 2 years.

Examination of the viscosities of the asphalts (after mixing in the pug mill) at 77 and 140 F (25 and 60 C) shows that the viscosity at 77 F affects, to a larger extent, the

Figure 4. Viscosity at 140 F versus time.

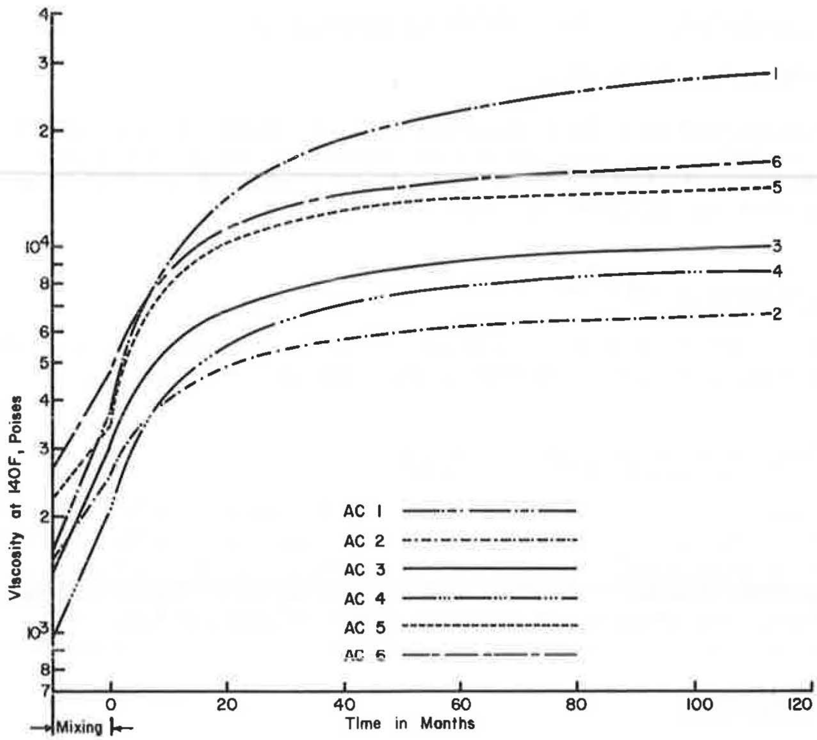


Figure 5. Shear susceptibility at 77 F versus time.

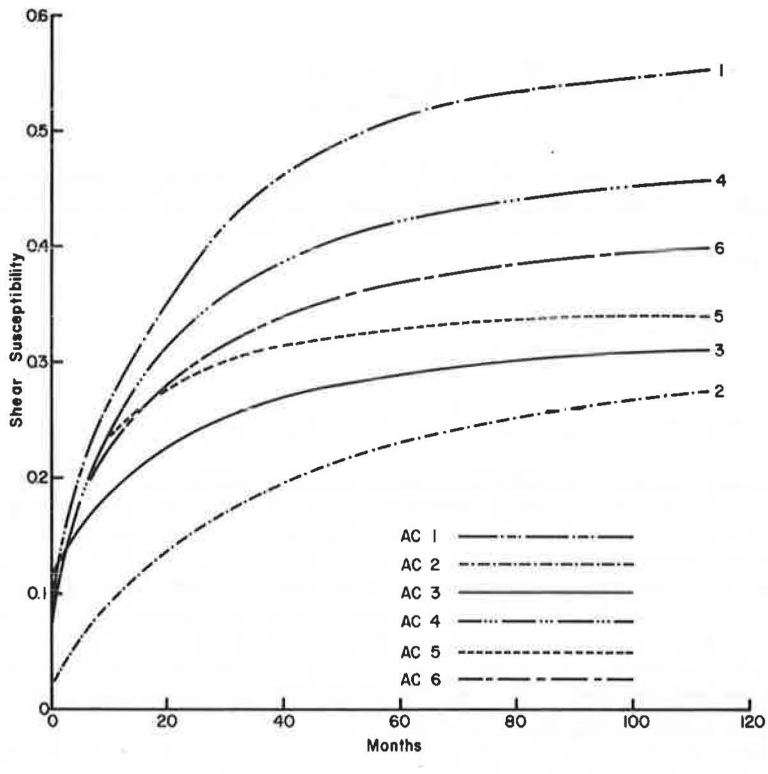


Figure 6. Shear susceptibility versus viscosity at 77 F.

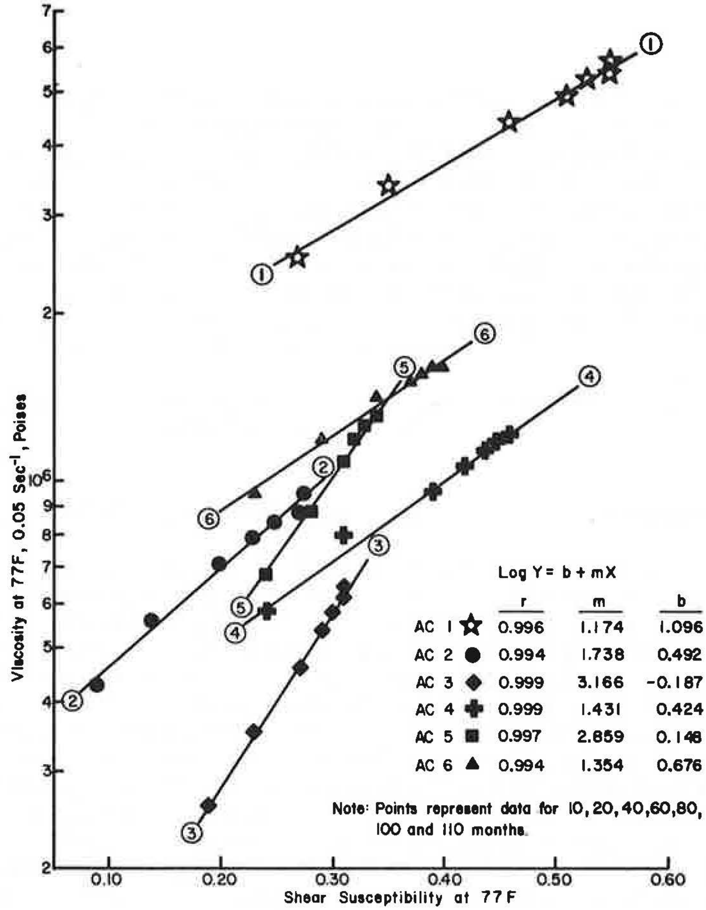
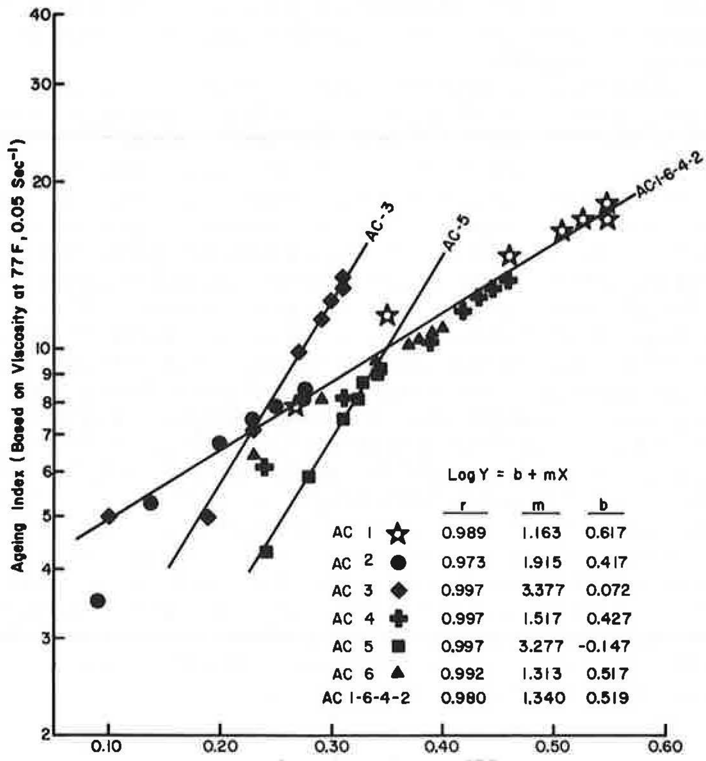


Figure 7. Shear susceptibility versus aging index (77 F).



ability of the pavements to compact under traffic. Asphalt 1, with the highest viscosity of 10.1 megapoises (1.01 MPa·s) after mixing, offered the most resistance to such compaction, followed by asphalt 6, which is second highest in viscosity and in resistance to compaction. Viscosity at 140 F does not indicate this trend.

Comparison of pavement evaluation ratings with the percentage of air voids after 113 months indicates the general trend established by many researchers that durability of the asphalts is affected by the air voids in the pavement. However, as has been pointed out (7), the air voids should be considered a secondary control factor since they are affected by other primary parameters, one of which is asphalt consistency after pug mill mixing. If the viscosity is too high, the design percentage of air voids may not be met.

Pavements containing asphalts 2 through 6 had air voids within a narrow range of 3.0 to 4.7 percent (Figure 2) after 2 years in service.

Pavement Performance in Relation to Rheological Properties

Viscosities at 77 and 140 F (25 and 60 C) and shear susceptibility at 77 F of the aging asphalts in the test pavements versus age of the pavements are plotted in Figures 3, 4, and 5 to examine their relation to pavement performance. The ranking order of these values relative to pavement performance after $9\frac{1}{2}$ years of service is given in Table 4.

It can be seen that the viscosity or shear susceptibility of the aged asphalt alone does not affect pavement performance. In earlier studies (3), the aging index-shear susceptibility relationship seemed to determine the pavement durability and performance. Shear susceptibility versus viscosity at 77 F and shear susceptibility versus aging index were plotted to examine this relationship (Figures 6 and 7). On semi-log plot, the data representing 10, 20, 40, 60, 80, 100, and 110 months of pavement life have a straight linear relationship for individual test pavements. In Figure 6 the slope m of these straight lines indicates the rate of change of shear susceptibility relative to change in viscosity or vice versa. Higher values of slope m would mean relatively lower gain in shear susceptibility with the corresponding increase in viscosity and are associated with better pavement performance. The displacement of the lines, one from another, is due to the differences in asphalt viscosity after construction. The ranking of slope m for the six test pavements (Figure 6) is consistent with the pavement performance (Table 5).

To eliminate the relative displacement of straight lines as seen in Figure 6, semi-log plots were made of shear susceptibility versus aging index (Figure 7). Two general groups of asphalts are clearly evident. Asphalts 1, 6, 4, and 2 (group 1) have essentially the same shear susceptibility-aging index relationship. Asphalts 3 and 5 (group 2) tend to gain shear susceptibility at a lower rate than group 1 on aging. At the present time, differences in the pavement conditions of asphalts 6, 4, and 2 are relatively small. The performance rating numbers, which are averages of observations made by eight evaluators, also indicate negligible differences.

Asphalt 1 exhibits raveling and loss of fines more than the other test pavements. Raveling was first observed in this test pavement after 30 months when the aging index and shear susceptibility values exceeded 12 and 0.40 respectively. At the present time this test pavement can be considered to have failed and needs resurfacing. The failure was probably caused by extremely high viscosity [more than 50 megapoises (5.0 MPa·s) at 77 F (25 C) and 25 kilopoises (2.5 kPa·s) at 140 F (60 C)] in spite of the fact that the aging index-shear susceptibility relationship of asphalt 1 is similar to that of asphalts 6, 4, and 2. It is not possible simply to set an upper limit of viscosity beyond which failure under traffic will occur because the deflection of the road determines the magnitude of tensile stress developed, which in turn leads to failure (8). Because these test pavements were built on an existing concrete road, they might show some distress but no real failures in spite of the high viscosities of asphalt because of the low pavement deflections involved. It is hoped that the remaining five asphalt pavements will weather in different ways and present different forms of pavement failure, resulting in larger differences in the future.

Table 4. Relationship of viscosity and shear susceptibility to pavement performance.

Performance Rating	Viscosity at 77 F, 0.05 sec^{-1}	Viscosity at 140 F	Shear Susceptibility
1 (poorest)	1 (highest)	1 (highest)	1 (highest)
6	6	6	4
4	5	5	6
2	4	3	5
5	2	4	3
3 (best)	3 (lowest)	2 (lowest)	2 (lowest)

Note: 1 F = 1.8 C + 32.

Table 5. Relationship of slope m (from Figure 6) to pavement performance.

Test Pavement	Performance Rating (113 months)	Slope m
1 (poorest)	51.1	1.174
6	59.8	1.354
4	60.1	1.431
2	60.4	1.738
5	61.2	2.859
3 (best)	61.5	3.166

Table 6. Ductility data.

Asphalt	Pavement Performance Rating	Ductility (cm): 39.2 F, 1 cm/min: Original	Ductility (cm): 39.2 F, 1 cm/min: After Mixing	Ductility (cm): 60 F, 5 cm/min After 113 Months[a]
1 (poorest)	51.1	14	4.1	0
6	59.8	21.9	7.3	8
4	60.1	23.5	7.5	7
2	60.4	53.3	11.9	19
5	61.2	68.3	24.3	19
3 (best)	61.5	101.0	42.2	49

Note: 1 F = 1.8 C + 32.

[a]Based on one test only.

Ductility

The significance of ductility requirements in the paving asphalt specifications has been the subject of debate. Some asphalt technologists believe that ductility, under the present standard method, is of little value as an indicator of asphalt quality. Others believe that the ductile properties of asphalt give an asphalt pavement its quality of flexibility. Regardless of the merits of the various arguments, a number of studies (9, 10) have related ductility to pavement performance. Although the ductility of asphalts recovered from these test pavements has not been measured periodically, some initial data are available and might be of some interest to the researchers. The ductility at 60 F (15.5 C), 5 cm/min, of these asphalts, recovered from most recent core samples, has been determined. These data together with performance ratings are given in Table 6.

Surprisingly, in spite of the empirical nature of the test, ductility test values of the asphalts at 39.2 F (4 C), before and after pug mill mixing, are consistent with the pavement performance ratings. Higher ductility values are associated with better pavement performance. Asphalt 1 with the lowest ductility shows the poorest performance. It is most likely, as pointed out by Halstead (10), that the ability of the asphalt to undergo elongation is not the primary characteristic affecting durability; rather, the ductility test result is an indication of an internal phase relationship of the asphaltic constituents, which in turn have an important bearing on the serviceability factors of the asphalt. It is also possible that the ductility test result obtained at 39.2 or 60 F (4 or 15.5 C) reflects indirectly the viscosity-shear susceptibility relationship at these temperatures. When additional ductility data are available in future, they will be analyzed further.

Because these test pavements were constructed on existing cement concrete surface, transverse reflection cracks are visible on all the test sections. However, it is interesting to note that the asphalts with higher ductility values, such as asphalts 5 and 3, tend to have narrower reflection cracks. Reflection cracks in low-ductility asphalts have developed secondary cracks, resulting in spalling at places.

CONCLUSIONS

Based on the rheological properties of the six asphalts studied and the preceding discussion, the following conclusions are drawn.

1. Changes in percentage of air voids and asphalt properties such as viscosity and shear susceptibility are a hyperbolic function of time and approach a definite limit with time. If the changing asphalt properties are determined during the early life of the pavement (2 or 3 years), the changes to be experienced over later years can possibly be calculated.
2. Pavement performance is affected significantly by the extent of air voids in a pavement. The rate of hardening of asphalts is checked considerably if the pavements can compact under traffic during the first 1½ to 2 years, to air voids of less than 5 percent.
3. After optimum compaction during construction, the apparent viscosity at 77 F (25 C) after pug mill mixing seems to control the ability of the pavements to compact further under traffic at ambient temperatures, all other factors affecting compaction being the same. Air voids should, therefore, be considered a secondary control factor since they are affected by other primary parameters including the apparent viscosity at 77 F after mixing.
4. Viscosity or shear susceptibility of the aging asphalt alone does not, necessarily, indicate the pavement performance. The rate of gain in shear susceptibility relative to increases in viscosity at 77 F seems to be one of the major factors affecting pavement performance. Relatively lower gain in shear susceptibility with the corresponding increase in viscosity is associated with better pavement performance in this study.
5. Asphalt ductility values determined at 39.2 F (4 C) before and after pug mill mixing seem to be consistent with the pavement performance observed so far. Higher ductility values after pug mill mixing indicate better pavement performance.

The significance of some of the rheological properties of asphalts indicated by the fundamental viscosity measurements will perhaps be more clear with the continual evaluation of the remaining five test pavements until failure. It is believed that the asphalts weather in different ways and may present different forms of pavement failure.

ACKNOWLEDGMENTS

The opinions, findings, and conclusions expressed are those of the authors and not necessarily those of the Pennsylvania Department of Transportation.

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PAVING ASPHALT PROPERTIES AND PAVEMENT DURABILITY

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Thirteen asphalt concrete pavements built in Pennsylvania were studied from September 1961 to March 1973. As a result of an extensive sampling and analysis program, considerable information has been gained on the durability of asphaltic pavement. Based on physical test data (penetration, viscosity, and ductility) and percentage of asphaltenes, all of the asphalts used in the various pavements hardened with time. The time of year when a pavement is sampled was shown to influence test results among and within test pavements. In experimental pavement studies all test pavements should be sampled at the same time of the year on an annual basis. Inasmuch as conditions appear to be more detrimental in summer than in winter, an additional 5 to 6 months of pavement life may be realized if the pavements are built in the fall rather than in the spring. This allows some pavement compaction to occur before the agents of asphalt hardening become active. The air void volume of an asphalt pavement has been shown to be a major factor in pavement durability and, with deterioration, the safety of those using the pavement. Based on construction results on experimental pavements, air void contents of Pennsylvania highway pavements are on the average too high. Multiple air void specifications should be replaced by a single number—10 percent of maximum theoretical. Field control must become somewhat continuous and totally enforceable. Lower viscosity asphalts, greater field compaction and control, and greater selectivity in design are offered as means of fulfilling lower air void content specifications. The hardening of asphalts in the pug mill should be studied closely. Lower mixing temperatures and shorter mixing times in conjunction with lower viscosity asphalts should provide a paving viscosity similar to that now used but with lower initial hardness.

•EXTENDING the useful life of a bituminous pavement is a problem for both the users and producers of asphaltic materials. During recent years, much research (1, 2, 3, 4) has been performed to determine the factors necessary to evaluate or predict the durability of an asphalt roadway and to gain an understanding of the physical and chemical changes that occur during the age hardening of asphalt. Hardening of an asphaltic pavement is confined to the asphalt cement. Any investigation of the hardening of an asphalt mix should concentrate, therefore, on the changing properties of the asphalt cement. Accordingly, it is important to understand the processes by which an asphalt hardens.

Oxidation is probably the most important factor in the hardening and loss of binding power of asphalt (5). It is a continuous process that occurs at the surface of the pavement and that depends on temperature, time, and rate of oxygen diffusion.

Photooxidation influences the hardening of asphalt at the surface. However, because 99 percent of the light waves can only penetrate to a depth of 10 microns (6), photooxidation does not harden asphalt that is internally situated in the wearing course of an asphaltic pavement. Photooxidation forms a hardened, impermeable film that is soluble in water but insoluble in common asphaltic solvents such as benzene, chloroform, and carbon tetrachloride (7). This exposed film becomes very hard, loses adhesiveness, and erodes away.

Age hardening or steric hardening is a phenomenon that occurs in asphalt at temperatures below the softening point of asphalt. When a sample of asphalt is cooled, a structure forms within the asphalt with the passage of time. As this structure develops, the penetration decreases, thus indicating that the asphalt has hardened. The structure is somewhat thixotropic in nature in that most of the structure is destroyed by the application of heat or mechanical energy (8). Age hardening is not a completely reversible process because some permanent hardening does occur.

There are many other factors that affect the hardening of an asphalt. Traxler (5) summarizes these factors as photochemical, polymerization, syneresis, action of water, absorption by solids, adsorption of components at solid surface, chemical reactions or catalytic effects at interface, and microbiological deterioration.

All of these affect the hardness of the asphalt and in turn the durability of an asphaltic pavement, but there are other properties that contribute as much or more to the durability of an asphaltic pavement. Stability, the ability to withstand loads, is very important to the longevity of an asphaltic pavement, as are also tensile, flexural, and flow properties. An asphaltic pavement must be resilient and able to rebound after an instantaneous load. These properties are affected by the temperature and duration of wet mixing in the pug mill, uniformity in batching, gradation of aggregate, aggregate shape, physical properties of aggregate such as strength and porosity, temperatures during spreading and rolling, segregation during spreading, amount and type of rolling, percentage of asphalt, subsoil and subgrade conditions, shoulder conditions, and so on.

Under the sponsorship and with the cooperation of the Federal Highway Administration and the Pennsylvania Department of Transportation, the Civil Engineering Laboratories of the Pennsylvania State University undertook an investigation of the physical and chemical properties of in-service asphalt concrete material. The principal aim of this investigation was to study the physical and chemical changes of asphalt and asphalt mixtures over time, with a view to gaining an understanding of the factors affecting the durability of asphaltic pavements.

TEST PROJECTS

This study was concerned with the performance of 13 test pavements designed and constructed by the Pennsylvania Department of Transportation. The pavements were originally studied as three groups of pavements. Group 1 pavements consisted of four road projects that were selected when the Pennsylvania Department of Transportation introduced specifications using the Marshall method for design and control of bituminous paving mixtures in 1961. Initial data, construction data, some in-service data, and the history of the pavements are given in the literature (4, 11). A brief description of the type of surface and aggregate used is given in Table I.

Six additional pavements (group 2) were included in the study in fall of 1963, but evaluation of four of these was discontinued in 1970. Sampling was discontinued because these pavements had characteristics—air voids, average daily traffic density, aggregate type, and transverse pavement location—whose order of importance in asphalt hardening was difficult to determine.

Group 3 consisted of three pavements constructed to determine whether penetration specifications for asphalt should be complemented by specifications for absolute viscosity. Data on the material properties of the asphalt and aggregates are reported in the literature (9, 10). A complete history of the group 3 pavements was reported in 1968 (9, 11). In addition to being constructed from different aggregates, each pavement was built with six different asphalts as described in the literature (9, 10).

TESTING PROCEDURES

The sampling and construction procedures were formulated for each project before each roadway was constructed. A complete discussion of these procedures was reported in 1967 (11). For this investigation, the testing procedure adopted for penetration was ASTM Designation D 5-65 and for absolute viscosity was ASTM Designation D 2171-63T.

Specific gravity determinations, based on Pennsylvania Department of Transportation

Table 1. Description of test projects.

Group	County	Legislative Route	Type of Surface		Type of Aggregate		Date Constructed (overlayed)
			Wearing	Binder	Coarse	Fine	
1	Washington	113	ID-2	ID-2	Slag	Slag	June 1963
	Beaver	538	ID-2	ID-2	Slag	Slag	October 1961
	Lycoming	41037	ID-2	ID-2	Limestone	Limestone	September 1961
	Lebanon	141	ID-2	ID-2	Limestone	Limestone	May 1962
2	Allegheny	652	ID-2	ID-2	Gravel	Sand and gravel	September 1963
	Armstrong-Butler	387	FJ-1	ID-2	Limestone	Glacial sand	November 1963
	Butler[a]	75	FJ-1	ID-2	Limestone	Glacial sand and fly ash	October 1963
	Butler[a]	251	FJ-1	ID-2	Limestone	Glacial sand and fly ash	October 1963
	Clarion[a]	214	FJ-1	ID-2	Limestone	Glacial sand	October 1963
	Clarion[a]	66	FJ-1	ID-2	Limestone	Glacial sand and fly ash	October 1963
3	Clinton	219	ID-2	ID-2	Limestone	Limestone	October 1964
	McKean	101	ID-2	ID-2	Gravel	Sand and gravel	August 1965
	Jefferson	338	FJ-1	ID-2	Limestone	Sand	September 1965

[a]Sampling discontinued in 1970.

Figure 1. Percentage of retained penetration versus time for group 1 pavements.

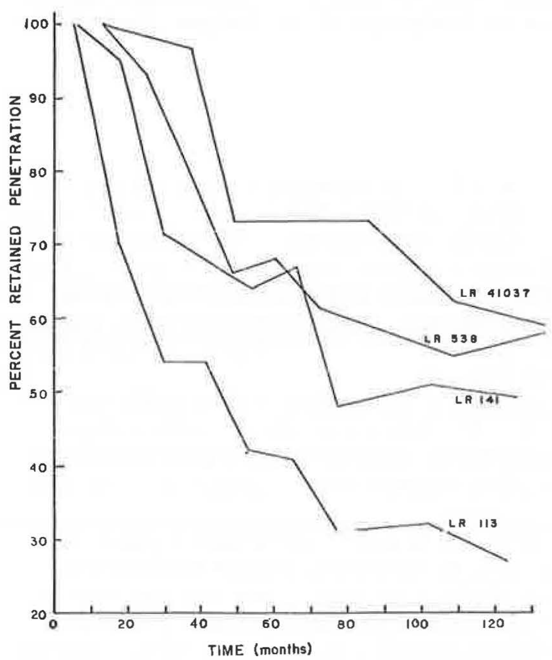
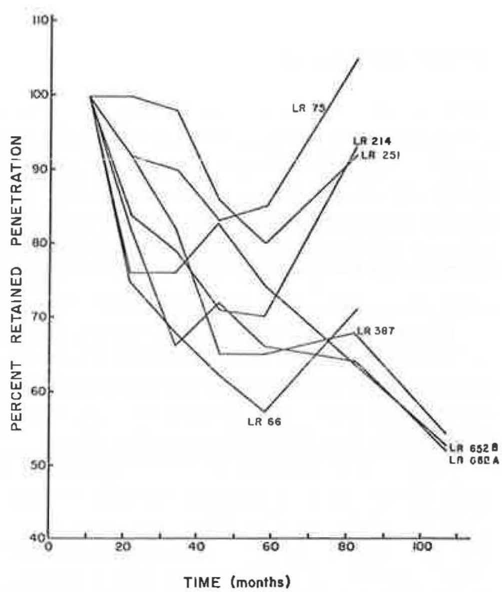


Figure 2. Percentage of retained penetration versus time for group 2 pavements.



specifications, were made on the mix samples to ascertain degrees of compaction at construction and during service life.

Asphalt cement samples were obtained from mix and field core specimens by the immersion-reflux method of extraction and the Abson method of recovery (ASTM Designation D1856-65). Benzene was used as the solvent to minimize any chemical reaction between solvent and asphalt during the contact time of the recovery process.

RESULTS AND DISCUSSION

The specific test values of penetration and absolute viscosity for each asphalt used on each test pavement have been presented elsewhere (9-15). To look at asphalt hardening relative to all other asphalts, graphs were used in place of tabulated data wherever possible. All figures involving changes in penetration with time, viscosity with time, and so on were drawn by using the sample that initiated an annual pattern of sampling on the particular test pavement. In the use of such samples, only the asphalt hardening that occurred after construction was reflected in the figures. The penetration and viscosity of these samples were used as the base (the original values) for determining the percentage of retained penetration and percentage of original viscosity. A research program involving the study of hardening of asphalt pavements should sample all the test pavements at the same time of year. As some of the research pavements begin to reach a limiting value of hardness, these pavements can be switched to biennial or even triennial sampling, but still at the same time of year.

As a result of the above procedure, hardening data collected for a particular asphalt pavement may or may not represent the total hardening that the pavement experienced since the time of construction. The data not useful in pavement comparisons include the hardness incurred from the time of construction to the sampling month. In some cases, this hardening may be quite substantial. As a result, future researchers may wish to secure a sample at the time of pavement construction in order that total pavement hardening may be determined. Of course, the ideal method is to construct all research pavements at the same time of year. All data would then reflect total pavement hardening.

Penetration

Figures 1 through 5 show the relationship of percentage of retained penetration to time for the 13 pavements under study. Group 1 pavements (Figure 1) reveal that, after 10 years of aging, the retained penetration values ranged from 65 to 28 percent. The change in LR 113 should be noted. Group 2 pavements (Figure 2) reveal the effect of the authors gaining experience. LRs 66, 75, 214, and 251 were discontinued from the study because each pavement differed from the group in some significant fashion (sampling location, aggregate type, traffic density, excessive oil deposition). Group 3 pavements (Figures 3 to 5) permit examination of differences in performance of both the pavements and the asphalts. Comparison of LRs 219, 101, and 338 shows the better performance of LR 101 (as a group) and the very poor performance of asphalt 1 on LR 219. Air void data presented later will explain these differences.

Absolute Viscosity

Figures 6 through 10 show the relationship of percentage of original absolute viscosity to time for the 13 pavements. Examination of these figures shows differences in the vertical scale. For example, although the six asphalts of LR 101 (Figure 9) first seem to have a typical performance, the highest original viscosity value is only 375 percent. Comparison of all pavements shows that LR 113 and LR 101 (asphalt 1) increased more than 700 percent since first sampling. Again, air void data will explain this performance.

Figure 3. Average percentage of retained penetration versus time for LR 219, group 3.

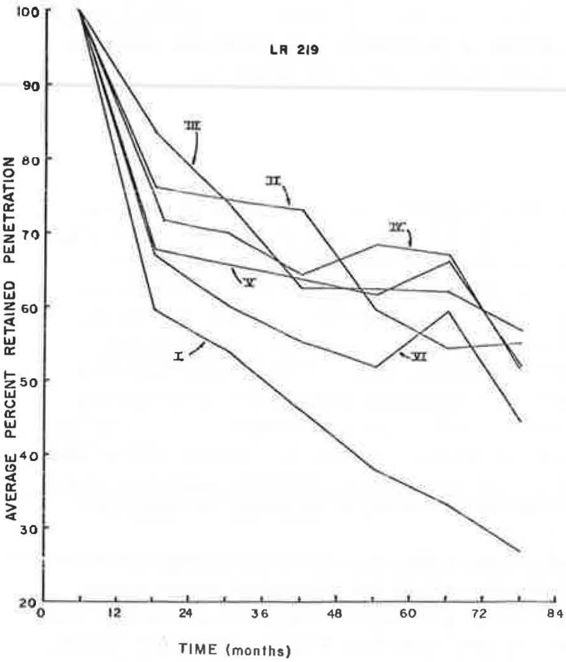


Figure 4. Average percentage of retained penetration versus time for LR 101, group 3.

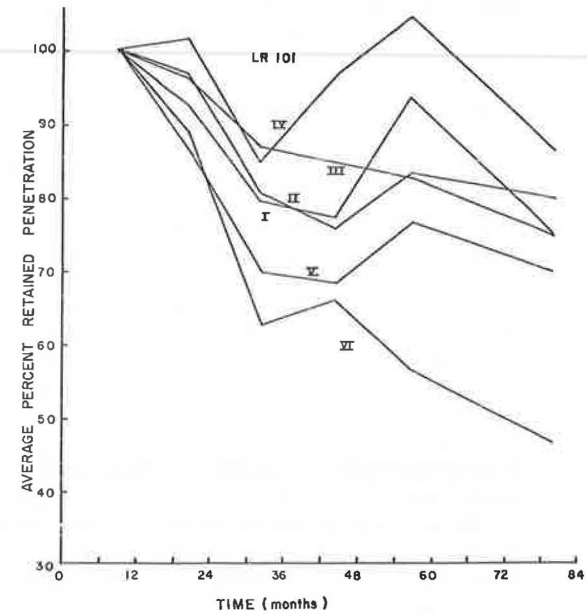


Figure 5. Average percentage of retained penetration versus time for LR 338, group 3.

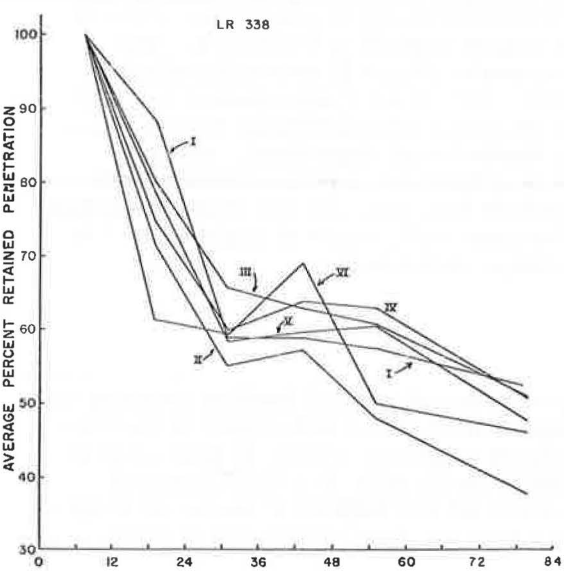


Figure 6. Percentage of original absolute viscosity versus time for group 1 pavements.

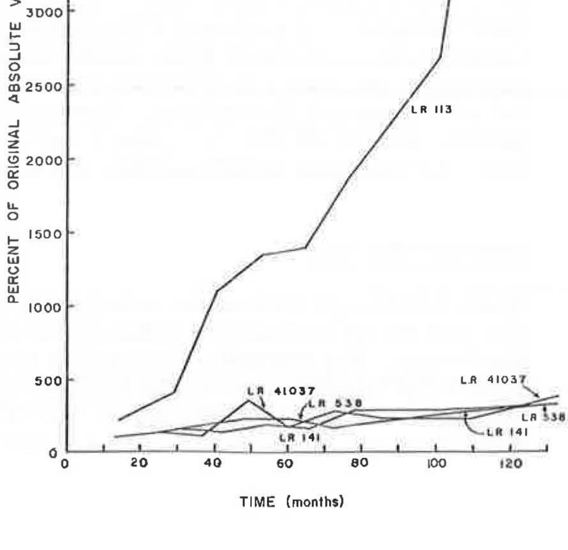


Figure 7. Percentage of original absolute viscosity versus time for group 2 pavements.

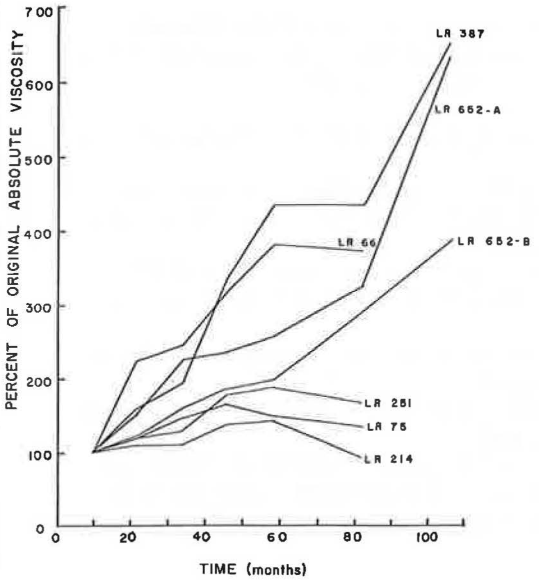


Figure 8. Average percentage of original viscosity versus time for LR 219, group 3.

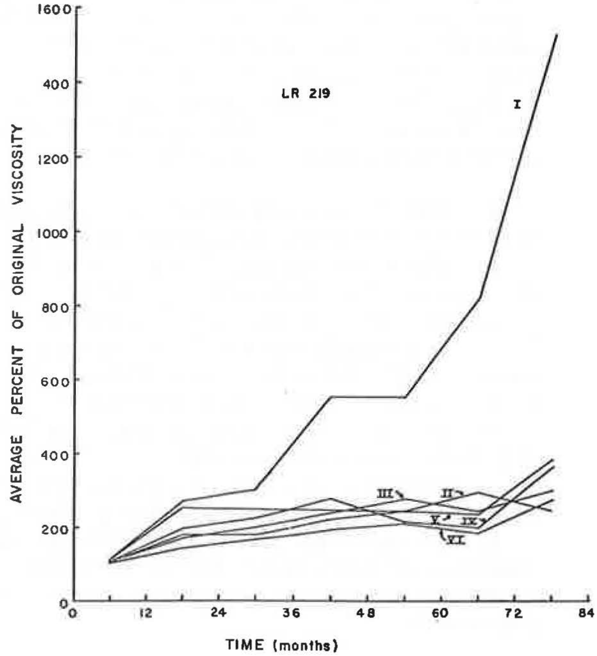


Figure 9. Average percentage of original viscosity versus time for LR 101, group 3.

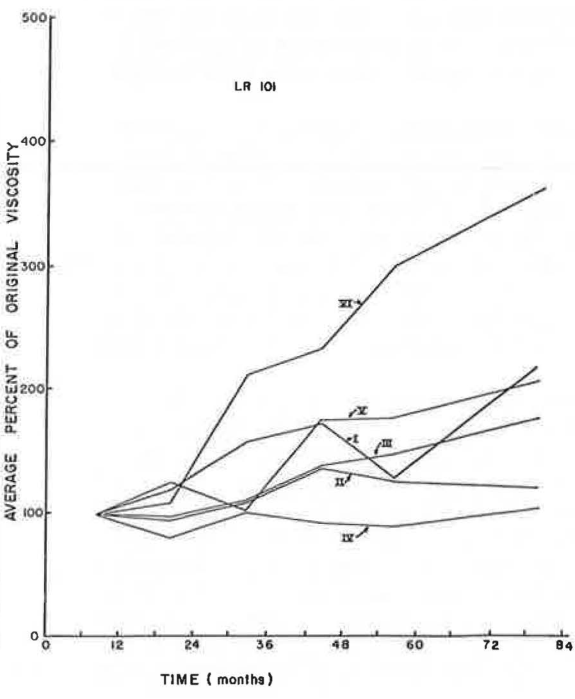
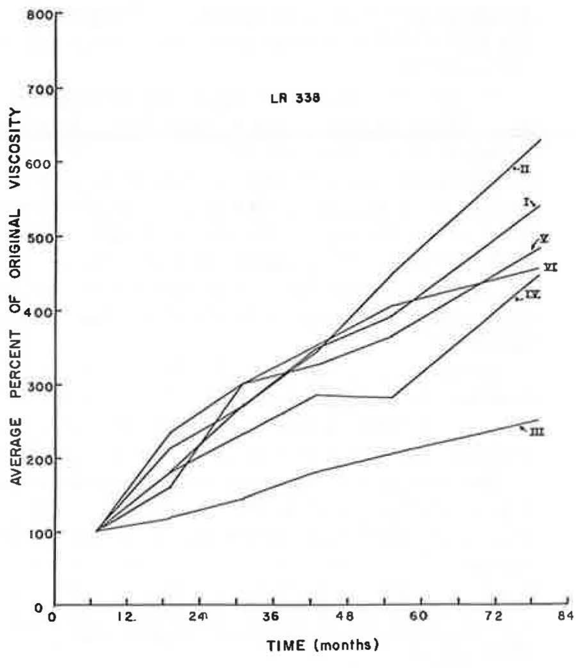


Figure 10. Average percentage of original viscosity versus time for LR 338, group 3.



Air Voids

Air voids are one, if not the greatest, factor affecting the rate of hardening of an asphalt pavement. The influence of this variable appears to be so pronounced that it completely overshadows the performance of asphalt type, aggregate type, traffic density, and microclimate differences.

Correlating asphalt cement performance with air void content is a very difficult task because of the variability of air voids in an asphalt pavement. The following types of air void variability have been recognized in this research (10, 15):

1. The inherent variability from point to point in a pavement due to varying degrees of aggregate interlock and asphalt content;
2. The gently sloping air void trends in the longitudinal direction of the pavement due to variability in gradation, asphalt content, mixing temperature, compaction temperature, and thickness of lift during the construction day;
3. The steeply sloped air void variability across the transverse direction of the pavement due to the decreasing lateral support of the mixture from the center of the traffic lane to its edges during compaction;
4. The air void variability among asphalt cement types on any one pavement due to differences in asphalt cement viscosity during compaction;
5. The air void variability among pavements due to gradation, aggregate type, and differences in degree of hardening in the pug mill; and
6. The decrease in air voids with time due to traffic, particularly in the wheel or load zone of a pavement, and the variability in decreases in air voids among asphalt pavements due to differing traffic densities and the degree of initial compaction among pavements.

Figures 11 through 15 show the changes in air voids with time for each asphalt on each pavement. As before, any durability comparisons of asphalt cement type and air voids should be based on air void values obtained at the same sampling times each year. Figure 11 clearly shows the atypical performance of LR 113. Of the 13 pavements under study, this pavement has the highest initial and highest residual air void content. Figure 12 shows that LR 387 had considerably higher air void contents than did the remaining pavements of group 2. Figures 13 to 15 show the performance of asphalt 1 on LR 219 and the poorer performance of LR 338, on the whole, when compared to LRs 219 and 101.

Figure 16 shows the relationship between retained penetration and air void percentage. This figure is based on values obtained after approximately 78 months of field aging of group 3 pavements. The differences between the pavements are attributed to the average daily traffic, initial extent of hardness after mixing, air temperatures during compaction, and shape characteristics of the aggregates, which in turn affect the rate of change of the air void percentages. ADT plays a conflicting role in pavement performance. The higher the density is, the greater are the compaction under traffic and the oil deposition, both favorable. On the other hand, the greater the traffic density (and percentage of trucks) is, the quicker will be the failure after the asphalt has hardened.

Hardening in the pug mill is a factor that quickens the hardening process. The greater the initial hardening is, the more rapid will be the rate of hardening due to the exponential nature of the hardening process (1). A close examination of the pug mill mixing process might reveal means for reducing asphalt hardening in the pug mill.

Because the sensitivity of the penetration test decreases as the asphalt approaches a penetration value of 10, the authors believe that a better understanding of the continuing hardening process can be obtained by using absolute viscosity data. Accordingly, Figure 17 is presented to demonstrate that asphalt hardening does not reach a limiting value; rather, it increases exponentially with time. It appears that, when the asphalt has an absolute viscosity of roughly seven times its original viscosity, the rate of asphalt hardening increases rapidly. The problem, of course, is to prevent the asphalt from hardening sevenfold.

Figure 11. Percentage of total air voids versus time for group 1 pavements.

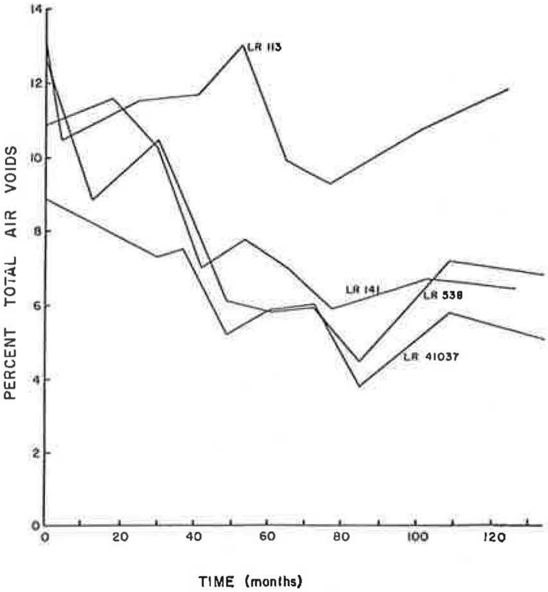


Figure 12. Percentage of total air voids versus time for group 2 pavements.

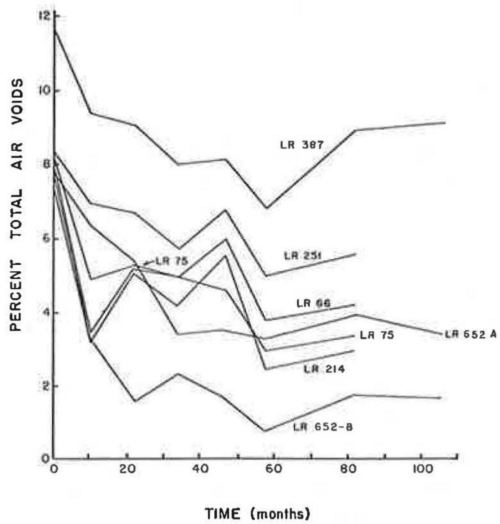


Figure 13. Average percentage of air voids versus time for LR 219, group 3.

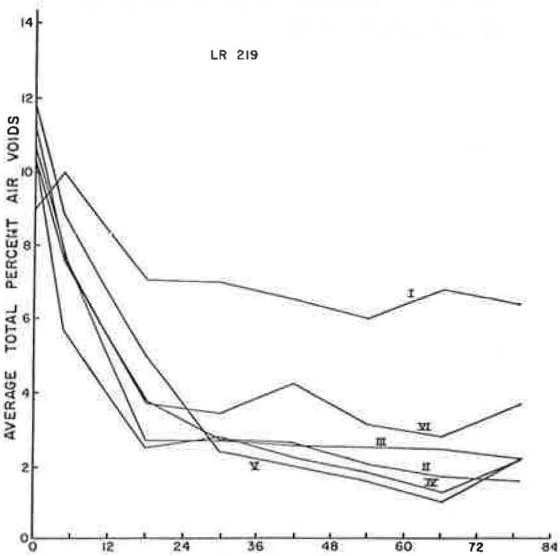


Figure 14. Average percentage of air voids versus time for LR 101, group 3.

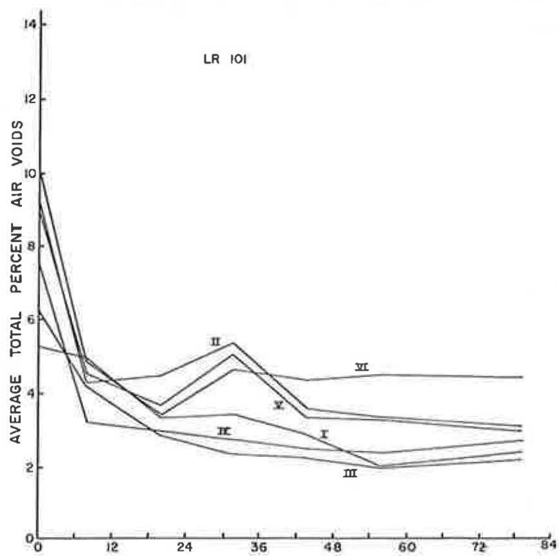


Figure 15. Average percentage of air voids versus time for LR 338, group 3.

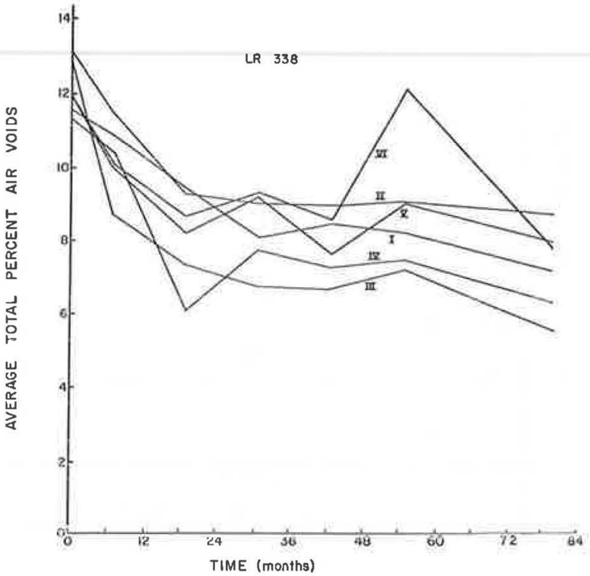


Figure 16. Percentage of retained penetration versus percentage of air voids (78-month values) for group 3 pavements.

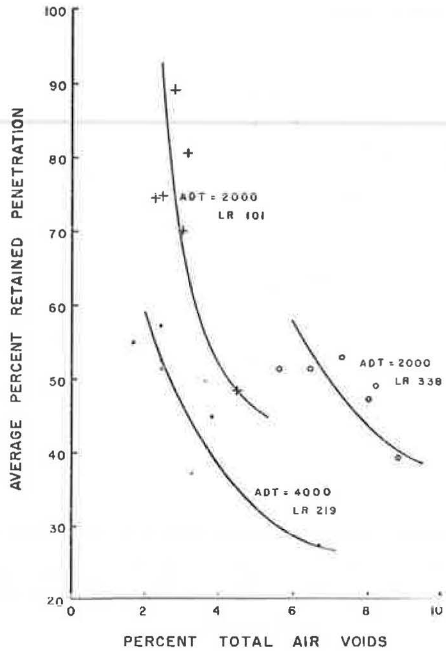


Figure 17. Percentage of original viscosity versus time for all groups.

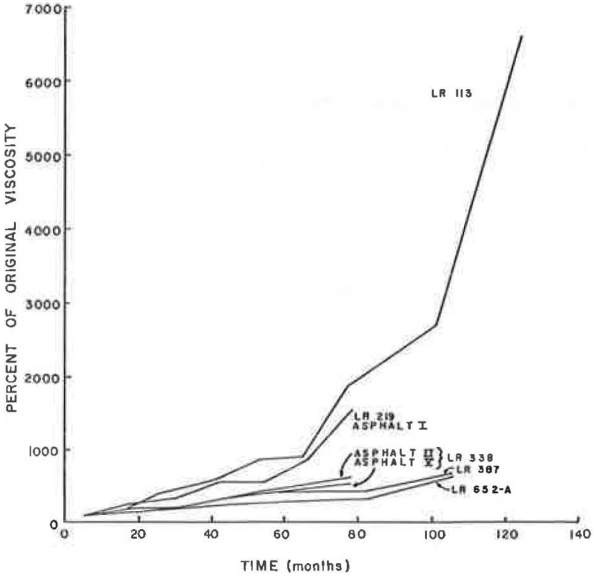
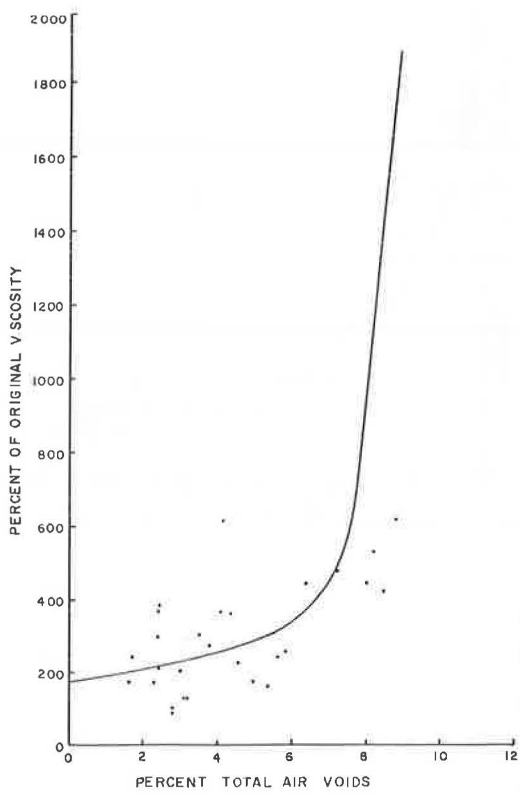


Figure 18. Percentage of original viscosity versus total air voids for all groups.



From Figure 17, one might expect the asphalt 1 pavement section on LR 219 to fail next. LR 387 is not so hard, yet it is more severely cracked than LR 219, asphalt 1. This cracking is most noticeable over the portion of the LR 387 pavement that was constructed on an aggregate base course. The remainder of the LR 387 pavement was constructed over a portland cement concrete base course and is not severely cracked. The LR 219 pavement was constructed entirely on a portland cement concrete base course.

It should be clearly understood that pavement hardness merely sets the stage for pavement cracking and other types of failure. A pavement such as the LR 338 project has a number of test sections that could eventually reach a hardness similar to that of LR 113. The LR 338 pavement, however, consists of 12 in. (305 mm) of asphalt concrete on a portland cement concrete base course. This very thick pavement will most likely never show such severe signs of distress as LR 113, which is just 3 in. (76 mm) of asphalt concrete on a portland cement concrete base course. Base thickness, therefore, is yet another variable confounding any analysis of pavement service.

Figure 18 shows the relationship between percentage of original viscosity and total air voids for all pavement sections after 80 months of service (data points obtained from figures presented earlier at 80 months of service). This figure contains all the variability discussed earlier, due to not only air voids, but also sampling time and location, initial hardness, pavement thickness, asphalt film thickness, asphalt grade, shear susceptibility, etc. Considering all of these factors, Figure 9 presents a significant relationship. There is also an abrupt increase in hardening at the 6 to 7 percent air void content (after 80 months of service). It is essential that this percentage figure be based on the maximum theoretical density and not on the daily plant Marshall density. Under current specifications (23), the daily Marshall density may vary from 94 to 98 percent of the maximum theoretical density (6 to 2 percent voids) and the field density may be as low as 95 percent of daily Marshall density. Combined, these specifications allow a range in total air voids of 2.0 to 10.3 percent. This is excessively high.

The pavements studied received better than average design and field control; yet these pavements were constructed with void contents as high as 13 percent. This underscores the need for more restrictive specifications and closer field control.

As early as 1968 the authors concluded that low void contents that enhanced durability were incompatible with regard to developing high skid resistance. Research work on the Blair County pavement (16) has shown that permeability, increased by high void contents, is lost quickly; thus skid resistance should not be used as an excuse for high air void content pavements.

SUMMARY AND RECOMMENDATIONS

From the research conducted to date, the following summary statements and recommendations are presented.

1. In general, all the asphalts from the 13 pavements are hardening with time based on physical test data.
2. The time of year when a pavement is sampled influences test results among and within test pavements. All test pavements should be sampled at the same time each year on an annual basis.
3. An additional 5 to 6 months of pavement life may be realized if the pavement is constructed in the fall rather than in the spring. This allows some pavement compaction to occur prior to the time the agents of asphalt hardening become active.
4. The air void content of an asphalt pavement is a major factor in pavement durability and, with deterioration, the safety of those using the pavement.
5. The hardening of asphalts in the pug mill should be studied closely. Lower mixing temperatures and shorter mixing times in conjunction with lower viscosity asphalts should provide an asphalt with similar viscosity but lower initial hardness.
6. Air void contents of Pennsylvania highway pavements are on average too high. Multiple air void specifications should be replaced by a single number—10 percent of

maximum theoretical. Field control must become continuous and totally enforceable. Lower viscosity asphalts, greater field compaction and control, and greater selectivity in design are means of fulfilling lower air void content specifications.

ACKNOWLEDGMENTS

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The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Pennsylvania Department of Transportation or the Federal Highway Administration.

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DISCUSSION

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The authors have produced a relevant paper of immediate practical significance, and they are to be congratulated for both the paper and their perserverance in following the study for 13 years. However, I feel that they have overextended their data in recommending implementation of fall or cold-weather paving and an end product specification for a maximum of 10 percent voids in the finished pavement.

In 1962, the New York State Department of Transportation undertook a research project to study the physical properties of asphalt concrete. That project, which paralleled much of the research performed by the authors in Pennsylvania, concentrated on the New York State DOT type 1A top course, a mixture quite similar to Pennsylvania DOT ID2 top course. Inasmuch as these two states have similar specifications, materials, and climate, I feel qualified to comment based on my involvement in the New York research (17, 18, 19, 20).

Briefly, the New York State DOT research in asphalt concrete density showed that controlling the density of pavement top course required a concerted effort by the designer in providing a pavement structure of proper stiffness on which to compact the top course mixture, the materials engineer in supplying a uniform mixture of proper gradation and asphalt content to the project, and the project engineer in seeing that the mixture is properly compacted while the mixture is hot. The thruway authority's research (21) confirmed the value of a properly prepared surface in overlaying existing pavement, especially ruts in the existing pavement caused by studded tire wear. Density of the test pavements increased with time with or without traffic but seemed to level off at 100 ± 2 percent of the laboratory (Marshall) density.

Research into asphalt hardening and pavement condition or durability showed that, for the study pavements, condition and percentage of retained penetration at 77 F were synonymous. It also showed that initial properties of the study pavements and their AADTs could be used as predictors of later condition and that they outweigh minor age differences and various environmental factors.

The conclusion was that maximum durability could be achieved by ensuring that the design criteria and specifications be directed to maximizing the level of compaction and thus to minimizing total air voids. The authors' recommendations are not fully consistent with those goals. Specifically, fall paving is very critical considering that low ambient temperatures are not conducive to compacting asphalt concrete. Anything gained by reduced oxidation rates in cold weather could easily be lost by a poor initial level of compaction (higher void content and high permeability). At its worse, cold-weather paving can cause what we call the late season paving syndrome—rapid hardening of the asphalt cement leading to cracking, raveling, and general distress at an early age.

Recognized mix design criteria call for air void contents from 2 to 5 percent, and according to the Asphalt Institute (22) a minimum of 97 percent of the laboratory density should be expected on a properly compacted pavement; therefore, air voids in place should not exceed 8 percent in the worst case, if both standards are applied consecutively. Mix quality assurance should control the potential voids in the mixture; project quality assurance should control compaction. With a single number specification, a maximum of only 5 percent voids would be allowable since compaction level can often equal or exceed 100 percent of laboratory values and the mix itself should compact in the laboratory to 5 percent air void content or less. The multiple specification can take into account that 100 percent densification cannot always be achieved on the project and that some slightly lesser conformance will not necessarily be detrimental considering the positive effects of traffic; a single specification cannot. Therefore an end product specification of 10 percent would give a very poor level of control.

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AUTHORS' CLOSURE

We are pleased that Clark has shown an interest in our work. We believe it necessary, however, to clarify two points of potential disagreement. We believe the net result will be agreement between us and Clark.

The detrimental effect of cold-weather paving or the late season paving syndrome is mentioned. In citing a desire for fall paving, we did not mean to imply cold-weather paving. We are in agreement with Clark that cold-weather paving is undesirable in terms of the high void contents and high permeabilities that develop. We recommend that fall paving under desirable field temperatures is preferable to similar temperatures during the spring months.

We agree with Clark that air voids in the compacted material should be less than those in the experimental pavements we studied. The debate of the desirability of an end product specification, in our minds, is controlled by the need for a specification that can be easily enforced by field personnel. In our opinion the number of pavements constructed with air voids in excess of a specified value would be fewer in number with an end product specification than with the existing set of specifications. In retrospect, whether the maximum end product air void specification is set at 10 or 8 percent or less remains open for discussion.

In 1973 the Pennsylvania DOT introduced a restricted performance specification for bituminous concrete. The specific gravities of all the materials and the maximum specific gravity of the mixture are determined in accordance with Pennsylvania DOT procedure. The percentage of unfilled voids and the percentage of aggregate voids filled with bitumen are based on the maximum specific gravity and the asphalt content determined for each group of specimens prepared from the same sample.

This approach is being used so that contractors will provide and maintain a quality control system that will ensure that all materials and products submitted for acceptance conform to contract requirements. A new concept of compaction called control strips is also introduced in these specifications. The contractor is afforded the opportunity to proof roll a test area to determine the optimum roller pattern and procedures under existing job conditions. Measurement and control of this strip are achieved by means of a nuclear gauge. The strip is placed and rolled until a maximum density is obtained. Then, the average density of the strip as determined by 10 density measurements should not be less than 92 percent of the theoretical maximum density. Thereafter, each layer or course of the compacted mixture should achieve a target density of at least 96 percent of the control strip density (23).

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ASPHALT BINDER HARDENING IN THE MICHIGAN TEST ROAD AFTER 18 YEARS OF SERVICE

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Analysis of six binders used in a Michigan test road has given an indication of the mechanism of compositional change and resultant hardening occurring during service. All the binders show the same directional changes in composition, namely a decrease in naphthene aromatics, which convert to polar aromatics and in turn to asphaltenes. This conversion is more pronounced in the top ⅛-in. (3.2-mm) layer than in the underlying ¼-in. (6.4-mm) layer. The mechanism proposed here clearly explains the increase in softening point, increase in hardness, and loss of ductility. Change in composition and physical properties also accounts for the slight but observable differences in the wear, weathering, and raveling qualities of the different sections. The Michigan test road and its overlays are still in service, although they show considerable reflection and joint cracking. Superficial judgment indicates that two of the test sections show more wear and weathering than the others. Although this is of technical interest, these differences are not large enough to permit quality judgment or selectivity between sources.

•IN 1954 the Michigan highway department constructed a 6-mile (9-km) test section on US-10, a four-lane highway between Pontiac and Flint. This was identified as M63-30, C8-R and consisted of 3 in. (76 mm) of hot mix (binder and surface) placed over existing portland cement concrete in six 2,400-ft-long (732-m) sections. The purpose of this test project was to correlate the comparative behavior of six typical asphalt binders available in Michigan by observing actual construction handling qualities and in-service response. Meticulous care was exercised in controlling aggregate gradation, binder content, temperatures, placing and compacting techniques, etc., so that only the source of the binder was a major variable. Details of planning, construction, and earlier observations are documented in three Michigan reports (2, 3, 5). Based on this work, the Michigan highway department approved all six sources of asphalt cement for use in bituminous construction on state trunk lines. Figure 1 shows scenes of this test road taken in March 1974, almost 20 years after the placement of the overlays.

After this test project had served its purpose, the original binders and binders extracted from each section were sampled and analyzed to

1. Determine the extent of change in chemical composition of the asphalt binders over long periods of service,
2. Relate those changes to the mechanism of binder hardening, and
3. Relate, if possible, the compositional changes with respect to wear and weathering qualities of the pavement.

Because the experiment was so well controlled, it was felt that such additional analyses offered an excellent opportunity for relating binder source, or its composition, with service observations. After binders extracted from cores taken in 1967 were examined, a final set of cores was taken in 1972, 18 years after the initial placement of the overlays. Although these test sections had been examined many times by the Michigan highway department and others interested in the project, a final rating was made by five engineers for the purpose of this report. This paper summarizes the

Figure 1. (a) Section 6 north and (b) section 1 south.

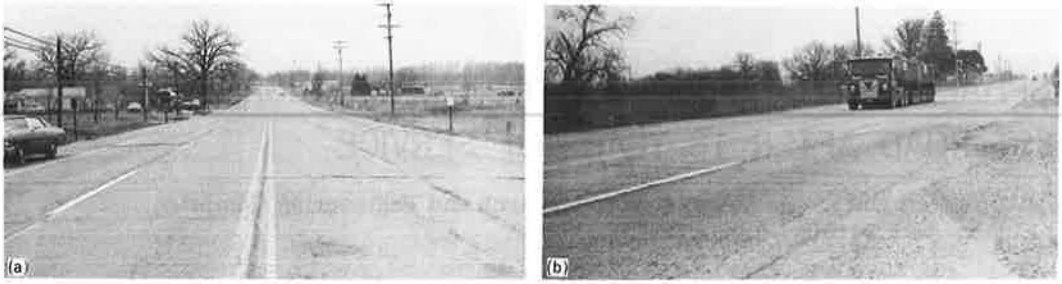


Table 1. Test data on asphalt cements (1954).

Property	Section 1	2	3	4	5	6
Softening point, F	120	125	119	123	120	124
Penetration at 77 F	63	60	67	60	61	65
Ductility at 60 F, cm	150+	150+	150+	150+	150+	77
Ductility at 77 F, cm	150+	150+	150+	150+	150+	150+
Kinematic viscosity at 140 F[a], stokes	2,457	4,336	2,361	3,159	3,869	4,418
Absolute viscosity at 140 F[a], poises	2,467	4,392	2,370	3,149	3,830	4,458
Saybolt furol viscosity at 275 F, sec	192	313	197	217	276	337
Thin film oven test at 325 F for 5 hours						
Percentage of loss	0.02	0.13	0.08	0.09	+0.09	0.02
Res. penetration at 77 F	36	38	40	42	39	45
Percentage of original penetration at 77 F	56	63	60	70	64	69
Res. ductility at 77 F, cm	150+	150+	150+	150+	150+	77

Note: 1 F = 1.8 C + 32; 1 stoke = 0.0001 m^2/s; 1 poise = 0.1 Pa·s.
[a]1965 data.

Table 2. Compositional analysis of original and recovered asphalts.

Sample	Fraction	Section 1	2	3	4	5	6
Original	Saturates	9.8	6.0	8.6	13.9	7.9	8.6
	Naphthene aromatics	32.5	28.8	32.6	31.3	42.0	38.7
	Polar aromatics	41.7	45.1	46.7	40.9	36.5	32.4
	Asphaltenes	16.0	19.2	12.0	12.8	13.3	19.7
Recovered, top ⅛-in.	Saturates	9.8	7.1	9.7	15.7	9.6	9.9
	Naphthene aromatics	25.9	20.7	25.9	22.7	28.4	24.2
	Polar aromatics	43.9	43.8	41.2	40.5	40.5	35.2
	Asphaltenes	19.3	27.7	19.4	20.3	20.7	28.8
Recovered, ¼-in. minus	Saturates	8.9	5.7	8.0	13.7	7.8	8.7
	Naphthene aromatics	34.6	28.9	35.5	30.4	32.7	35.0
	Polar aromatics	39.5	40.4	40.1	40.3	43.4	31.5
	Asphaltenes	16.9	22.8	15.6	15.0	15.9	24.7

Note: 1 in. = 25 mm.

compositional changes that occurred and relates these changes to the final ratings.

TESTING PROCEDURES

The original bulk retained asphalt binders are numbered from 1 to 6 as identified with the same number used for the six test sections in which they were used. Each binder was graded in the 60 to 70 penetration range to meet the Michigan specifications as well as ASTM D946. Complete physical properties and source identification are given elsewhere (2).

A summary of the physical properties, as measured on the original binders (2), is given in Table 1. Included are data on kinematic viscosity at 140 F (60 C) from the 1967 report (5) and an estimated value for absolute viscosity at 140 F. It is of interest to note that, relative to the current viscosity grading system, two of the binders would have been close to the AC-20 grade and four binders would have fallen in, or close to, the AC-40 grade.

Table 2 gives the results of compositional analyses that were performed on the original bulk samples in 1972 and the recovered samples. In accordance with Corbett's method (7), each binder was separated into four generic fractions: saturates, naphthene aromatics, polar aromatics, and asphaltenes. Although some aging may have occurred during the long-term storage of the bulk samples before this analysis, we believe that such changes were minimal. In any event, these differences are minor, especially since the hardening changes occurring in binders from the road cores were substantially larger.

Asphalt binders from pavement cores taken in 1972 were extracted with benzene (10) and recovered by using the Abson method (6). The cores were sawed into two layers, as shown in Figure 2. The surface mix in each section was 1¼ in. (32 mm), and a binder course comprised the remainder of the 3-in. (76-mm) overlay. All cores were taken in the passing lane in the wheel track, where vehicle drippage is minimal. Thus, each core provided a binder sample from the top ⅛-in. (3.2-mm) layer and another sample from the ¼-in. (6.4-mm) minus layer. All asphalt binders recovered from the core layers were then analyzed.

DISCUSSION OF RESULTS

Binder Inspections

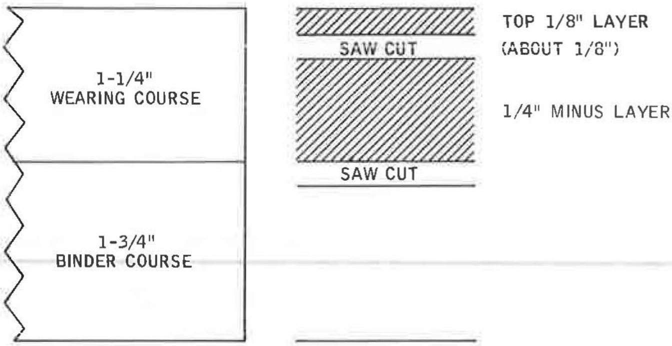
Compositional analyses, together with the physical properties of both the original (Table 2) and recovered binders (Table 3), led to the following observations.

In all cases, softening points and viscosities increased, and penetrations and ductilities decreased. These changes were greater in the binder from the ⅛-in. (3.2-mm) top layer than in the binder from the ¼-in. (6.4-mm) minus layer. This seems to indicate that hardening changes are more pronounced at or near the surface of a pavement because of greater exposure to air, sunlight, and other atmospheric effects. The binders from sections 2 and 6 in general had the highest consistencies and lowest ductilities, in both the top and minus layers.

The saturate content from recovered binders was virtually unchanged from that of the original binders. The saturate content in the top ⅛ in. (3.2 mm) showed some change, generally an increase, which is attributable to drippage. This drippage effect has been observed in other analyses of pavement cores (8). The amount of naphthene aromatics decreases in all cases in a manner similar to that reported when asphalt is air blown (4). There also seems to be greater change in composition in the ⅛-in. (3.2-mm) top layer than in the ¼-in. (6.4-mm) minus layer, and this appears consistent in all examples. Asphaltenes also consistently increase, especially in the top layer; the polar aromatics show no distinct pattern.

Figure 3 shows the average change in hardening for all sections. Basically, the

Figure 2. Method of preparing core for binder recovery.



1-1/4" = 3.18 cm
1-3/4" = 4.45 cm
1/8" = 0.32 cm
1/4" = 0.64 cm

Table 3. Physical properties of original and recovered samples.

		Section					
Sample	Property	1	2	3	4	5	6
Original	Softening point, deg F	126	124	124	125	125	126
	Penetration at 77 F	46	52	48	49	51	49
	Ductility at 60 F, cm	150+	150+	150+	67	16	27
	Viscosity at 140 F[a], poises	3,880	5,310	3,290	4,480	6,410	7,740
Recovered, top ⅛-in.	Softening point, deg F	130	150	140	147	147	165
	Penetration at 77 F	26	19	23	23	23	19
	Ductility at 77 F, cm	150+	7	150+	6	7	5
	Ductility at 60 F, cm	5	0.5	4.5	—	3	2.5
Recovered, ¼-in. minus	Softening point, deg F	129	136	133	135	138	147
	Penetration at 77 F	37	34	36	35	36	32
	Ductility at 77 F, cm	150+	150+	150+	150+	150+	40
	Ductility at 60 F, cm	8	8.5	8.5	6.5	6	4.5
	Absolute viscosity at 140 F, poises	7,320	17,041	7,752	9,705	11,787	34,414

Note: 1 in. = 25 mm; 1 F = 1.8 C + 32; 1 poise = 0.1 Pa·s.

[a]1965 data.

Figure 3. Mechanism of change for (a) top ⅛-in. layer and (b) ¼-in. minus layer.

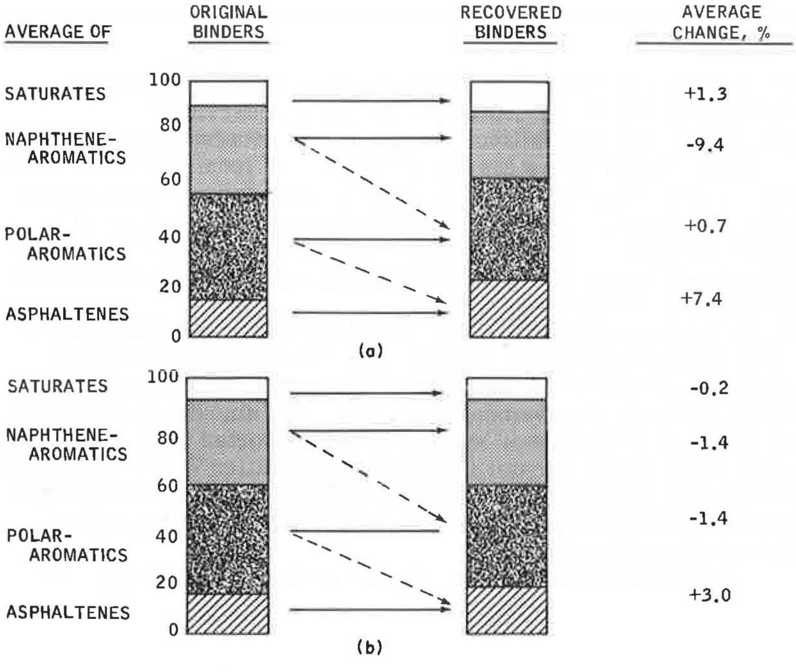


figure reveals a significant reduction of naphthene aromatics and an increase in asphaltenes. This figure confirms that the chemical change is consistently greater in the top layer than in the minus layer.

The compositional changes due to aging decrease in the liquid component (naphthene aromatics) and increase in the solid components (polar aromatics and asphaltenes), which is in line with the expected and observed physical data showing hardening. The average change in physical properties of the binders emphasizes the very substantial difference between the top and minus layers.

Property	Top Layer	Minus Layer
Softening point, F	26	15
Penetration at 77 F	-41	-28
Percentage of original penetration	35	56
Ductility at 77 F, cm	7	150+

This comparison and the detailed data in Table 3 tend to confirm that below the top $\frac{1}{8}$ in. (3.2 mm) the binders are less affected, and thus the major bulk of the binder continues to be capable of performing its functional role for long periods. Furthermore, there is little distinction among binder sources in aging resistance in the $\frac{1}{4}$-in. (6.4-mm) minus layer.

It is known that volatilization of light components from the binder during pavement construction and aging is a factor that should be considered in evaluation of the hardening tendency. However, the volatility factor did not appear to be significant with these six binders. They all showed negligible losses by thin film oven testing (Table 1), and the slight differences reported in the original binder testing did not appear to correlate with the physical changes of the aged binder or with pavement performance.

Pavement Inspections

Over the past year or so, each of the test sections was inspected and rated by five engineers. Although pavement cracking was prevalent in all sections, this was not considered in the ratings because much of the cracking was obviously reflection cracks from the underlying portland cement concrete. Cracks from transverse expansion joints as well as joints made by the paving machine were very much in evidence. Other cracks, which probably result from cracking within the concrete slabs, were also in evidence but would be expected in pavements of this age. The ratings given in Table 4 are thus the average of the five appraisals in which judgment was based largely on wear and weathering of the surface, with some consideration of edge raveling. The method used by the engineers was to visually observe the surface, and to note the extent of aggregate exposure and loss of mortar. Because the differences in these qualities are difficult to judge, ratings were based on selecting the two best and the two poorest and leaving the other two as intermediate. Averaging all of these factors gave sections 1, 3, and 5 as best, 2 and 6 as poorest, and 4 as intermediate. There was complete unanimity in rating sections 2 and 6 as the poorest in wear and weathering qualities. Ratings on the other sections were not so decisive. Again, these do not represent great differences, as is shown by photographs in Figure 4 (taken in March 1974), but there were enough differences to permit superficial ratings.

Although the nature of this study did not permit complete identification of the chemical mechanisms involved in aging, the study does shed light on the natural overall hardening process. There is definite conversion of the naphthene aromatics fraction and a conversion of some of the polar aromatics to higher molecular weight asphaltene fractions, postulated by various condensation mechanisms. The apparent greater reactivity of naphthene aromatics seems to be in line with the laboratory photooxidation studies carried out by Thurston and Knowles (13), who showed that greater oxygen

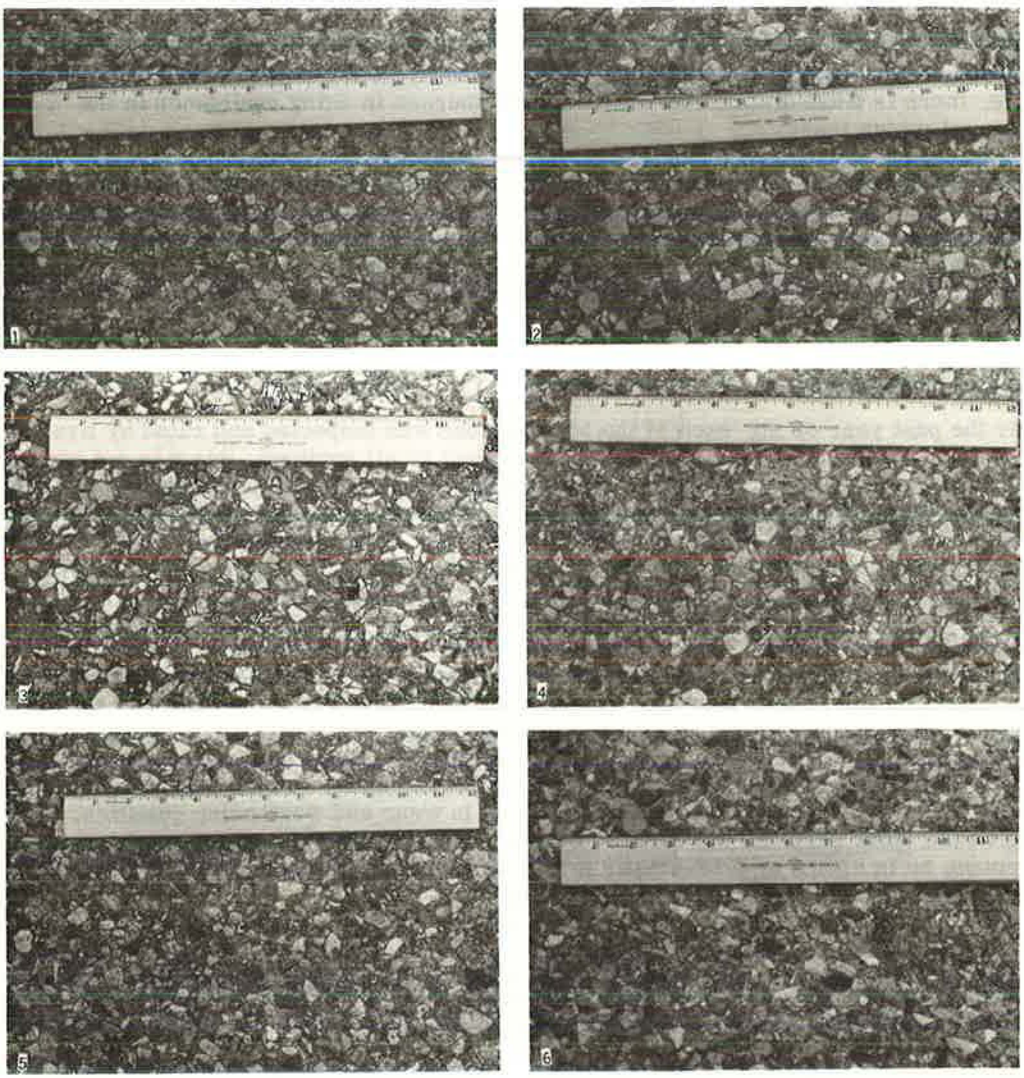
Table 4. Relation of visual inspection of pavement to binder changes.

Section	1	2	3	4	5	6
Average pavement rating[a]	1.4	3.0	1.4	1.8	1.4	3.0
Changes in top ⅛-in. layer						
Δ Softening point, F	+16	+25	+21	+24	+27	+41
Δ Penetration at 77 F	-37	-41	-44	-37	-38	-46
Percentage of original penetration at 77 F	41	32	34	38	38	20
Δ Ductility at 77 F, cm	0	-143	0	-144	-143	-145
Changes in ¼-in. minus layer						
Δ Softening point, F	+9	+11	+14	+12	+18	+23
Δ Penetration at 77 F	-26	-26	-31	-25	-25	-33
Percentage of original penetration at 77 F	59	57	54	58	59	49
Δ Ductility at 77 F, cm	0	0	0	0	0	-110

Note: 1 in. = 25 mm; 1 F = 1.8 C + 32.

[a]1 = best and 3 = poorest.

Figure 4. Sections 1 through 6.



consumption occurs with an isolated naphthenic fraction of asphalt than with other fractions.

It is of interest to note that binders 2 and 6, which had the highest asphaltene contents both before and after aging, showed greater change in physical properties and somewhat more wear and weathering. It is believed that these higher consistency properties can be related to the amount of plasticizing components present in the binder. As shown in other fractionation work (4), the saturates and naphthene aromatics fractions are low-viscosity components, and it is believed that they, therefore, function as plasticizers for the high-viscosity components, i.e., the polar aromatics and asphaltenes. Binders 2 and 6 appear to show relatively less of the plasticizing components and thus are associated with binders of higher consistencies. It is noted, of course, that binders 2 and 6 had the highest viscosity among these penetration-graded cements. This is generally expected in view of their higher asphaltene contents. Binder 5 also had a relatively high original viscosity, not far from binders 2 and 6, and it showed fairly aggressive changes with aging in increasing its asphaltene content and its softening point. The favorable road evaluation of section 5 may be attributable to a higher proportion of the plasticizing components present in its original form. These being carried through during its service life resulted in the relatively better rating as compared to sections 2 and 6.

CONCLUSIONS

1. Although this test project on US-10 has shown considerable reflection and joint cracking for some time, the road is still serviceable for secondary traffic and does carry heavy-duty vehicles. Wear, weathering, and raveling are evident in all sections, although more pronounced in sections 2 and 6.
2. Increases in binder consistency and loss in ductility are prevalent in binders from all sections, but the same two sections noted above showed greatest changes in these qualities.
3. Changes in composition occurring in binder hardening are directionally the same in all cases; naphthene aromatics converted to polar aromatics and then in turn to asphaltenes. Again, sections 2 and 6 consistently show larger compositional changes.
4. The top ⅛-in. (3.2-mm) layer consistently exhibits greater changes in consistency and larger changes in composition than the ¼-in. (6.4-mm) minus layer.
5. This study alerts the paving technologist to how core depth can affect results. That is, to obtain a more meaningful characterization in core analysis may require a workup of the various layers involved rather than an overall composite of the entire core.
6. Based on the present level of hardness of these binders and the age of the road overlay, there is no gross distinction among binders involved and no practical need to distinguish one binder from another.

ACKNOWLEDGMENT

The authors wish to express their appreciation to the Michigan highway department for cooperating in supplying asphalt binder samples and permission to report the findings. This analysis was only made possible because the original test was well planned and controlled from the start. We are especially grateful to Paul Serafin and Ward Parr who played important parts during the early planning and construction of this experiment road and who rendered valuable assistance in arranging for core samples, recovery of binders, and inspection of the pavement during recent years.

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USE OF ASTM TESTS TO PREDICT LOW-TEMPERATURE STIFFNESS OF ASPHALT MIXES

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This investigation tested the reliability of ASTM tests to predict low-temperature stiffness of mixes made with a wide variety of asphalts. Mix stiffnesses obtained from creep tests performed with a new low-cost non-destructive method on ordinary Marshall or Hveem specimens were compared with ASTM stiffnesses. The relationship reported between thermally induced pavement cracking and T_L or the temperature at which the 10,000-sec asphalt stiffness is 20,000 psi (138 000 kPa) is qualitatively supported by field tests. T_L estimated from ASTM penetration at 39.2 F (4 C), along with the ASTM penetration at 77 F (25 C) or the viscosity at 140 F (60 C) on rolling thin film (RTF-C) oven residua, correlates well with the measured T_L. The use of penetration at 39.2 F (4 C) is supported as a specification requirement. Good correlation was also found on asphalts recovered from the mixes. T_L correlated poorly with penetration at 77 F along with softening point or viscosity at 140 F on both RTF-C and recovered residua. T_L estimated from viscosities at 140 F, along with viscosities at 275 F on RTF-C or recovered residua, show no correlation with measured T_L. Resistance to low-temperature thermally induced cracking should not be implied on diverse types of asphalts from high-temperature viscosity measurements. Ductilities at 45 F show no correlation with T_L. A modified asphalt test data chart that permits the low-temperature stiffness of asphalts to be determined from normal ASTM penetration tests is given.

•THERMALLY INDUCED cracking or transverse shrinkage cracking, caused by rapid drops in temperature of asphalt concrete pavements, occurs in Canada and in the northern parts of the United States and Europe.

Canadian researchers in particular have made extensive investigations (1-10) to determine the importance of a number of factors on the occurrence of thermally induced pavement cracking. Based on their findings, several factors are now considered in advanced pavement design systems in cold climates.

Stiffness of the asphalt-treated mix is the principal independent variable of those that can be controlled by the highway engineer. Although mix stiffness depends on the voids and asphalt content, it is primarily dependent on the stiffness of the asphalt. Therefore, most highway departments concerned with thermal cracking use the softest grade of asphalt that is consistent with the other mix property requirements.

Although thermally induced cracking can be markedly reduced by using a soft grade of asphalt, considerable reduction can also be obtained by choice of asphalt type. Maximum resistance to thermally induced cracking can be obtained by using the softest practical grade of asphalt having the lowest available temperature susceptibility.

A number of investigators (2, 3, 4, 5, 6, 7) have shown good agreement between measured stiffness and observed field cracking of asphalt-treated mixes. For the asphalt estimated by means of penetration at 77 F (25 C) together with the softening point and van der Poel's nomograph (11), only fair to poor agreement was obtained (8, 9, 10) between field cracking and stiffness. When low-temperature penetrations were used with an improved bitumen test chart by Heukelom (12), agreement was greatly improved (2, 7). Although Heukelom showed that his chart could be used with a wide variety of asphalts, its use to predict thermally induced cracking or to estimate the low-temperature stiffness of mixes was shown by others to be limited to a small variety of asphalts. Consequently, the extent to which the Heukelom chart and van der Poel's nomograph could be relied on was not clear.

The purpose of this investigation was to test the reliability of ASTM tests to predict low-temperature stiffness of mixes made with a wide variety of asphalts. Mix stiffness obtained from creep tests made with a new low-cost nondestructive method on ordinary Marshall or Hveem specimens is compared with ASTM stiffness. This provides a basis for selecting the most appropriate asphalt for use at the lowest pavement temperature expected.

RELATIONSHIP OF STIFFNESS TO PAVEMENT CRACKING TEMPERATURES

There are several methods for estimating the temperature at which thermally induced pavement cracking can be expected. Some methods assume, in the analysis, a linearly viscoelastic plate (13, 14) or beam (1, 7, 15). Necessary input includes the stiffness and tensile strength of the mix over the temperature range considered, a rate of temperature drop, and the coefficient of expansion of the mix (24).

An alternate approach is the limiting stiffness method, which relies on the concept that, on the average, a mix will not crack if its stiffness (for some appropriate loading time) does not exceed a certain value at the lowest expected field temperature. It is assumed (18) that pavement temperature drops at 10 deg/hour.

Although the maximum level of stiffness tolerable without cracking depends on the mix design, i.e., gradation, voids, and asphalt content, the effect of varying asphalt characteristics can be compared directly if the same mix design and aggregate are used in all specimens. Limiting stiffness can be expressed in terms of either mix stiffness (S_{LM}) or binder or asphalt stiffness (S_{LA}). Readshaw (21), who related S_{LA} to the thermally induced cracking observed in Canadian pavements, concluded that pavements will not crack if S_{LA} does not exceed 29,000 psi (200 000 kPa) at 7,200-sec loading at their lowest service temperature. Fromm and Phang (18) suggest a limiting (S_{LA}) of 20,000 psi (138 000 kPa) at a 10,000-sec loading time. McLeod (16, 17) suggests that S_{LM} equals 1,000 ksi (6900 MPa) at 20,000-sec loading. He uses indirect methods to calculate S_{LM} on a dense, well-graded mix from the asphalt properties. Fromm and Phang also calculate S_{LM} indirectly from asphalt properties; however, unlike McLeod, they use the properties of asphalt residua after a thin film oven (TFO) test and thus simulate the asphalt as it exists in the mix after hot-mix hardening.

These limiting stiffness values are about the same after they are adjusted for different loading times and by whether they reflect S_{LA} or S_{LM} values. Thus, any one of these limiting stiffness values can be used for comparison of the effect of asphalt properties on the relative temperatures at which thermally induced cracking can be expected. We have chosen to use Fromm and Phang's S_{LA} of 20,000 psi (138 000 kPa) at a 10,000-sec loading time. This is equivalent to S_{LM} of about 1,500 ksi (10 300 MPa) on the mix described later in this study. This equivalency was calculated by using Heukelom and Klomp's method (19) together with van Draat and Sommer's (20) correction for air voids greater than 3 percent.

Approach

As previously mentioned, the purpose of this investigation was to clarify the extent to which ASTM tests can be used to predict thermally induced pavement cracking. Tests were made on residua from the rolling thin film (RTF-C) oven exposures (California test method 346E) instead of residua from the TFO as used by Fromm and Phang and by Readshaw. Use of RTF-C exposure instead of the TFO exposure is recommended in the Pacific Coast uniform specification for paving asphalts. Although both oven exposures are about equally severe, the RTF-C test is more efficient and has better precision than the TFO test. For confirmation, tests were also made on residua recovered from specimens tested in creep.

From tests made on both the RTF-C residua and the recovered asphalts, the temperatures at which the 10,000-sec S_{LA} attains a 20,000-psi (138 000-kPa) value were estimated indirectly. This temperature is the limiting stiffness temperature T_L. T_L is estimated by using a Heukelom bitumen test data chart (12) along with a new chart derived from Heukelom's chart that permits ASTM penetration at 39.2 F (4 C) to be used. This new chart is shown in Figure 1. Values obtained from these charts are then used on van der Poel's nomograph (11) to obtain the temperature at which the asphalt stiffness at 10,000 sec is 20,000 psi (138 000 kPa). Thus, from several routine asphalt tests we estimate T_L at which thermally induced cracking might be expected to begin.

Indirectly determined T_L is compared with T_L obtained directly from creep measurements of mixes made with the same asphalts. The relationship of indirectly determined T_L to directly determined T_L is used as a test of the dependability of the indirect method.

Mix Used in Comparison

Extensive studies have been made on low void mixes. In our comparison a similar low void mix is used so that the S_{LM} and S_{LA} levels can be assumed to be the same as those established by other investigators. The characteristics of the mix are as follows:

ASTM Sieve Size	Percentage Passing	ASTM Sieve Size	Percentage Passing
$^5/_8$ in.	100	No. 10	40
$^1/_2$ in.	95	No. 40	18
$^3/_8$ in.	82	No. 80	11
$^1/_4$ in.	65	No. 200	5

(The aggregate density was 2.82. The asphalt content was 6.2 g/100 g of aggregate. Mixes were prepared and compacted according to California test method 304E.)

The mix is representative of the design mix used in the 1967 construction of I-90 in eastern Washington between Renslow and Rye Grass. A section of I-90 made with this aggregate developed excessive thermally induced cracks during the first and second winters after construction (22).

Asphalts Used in Comparison

Ten types of asphalt made from a wide variety of crude oils and processing methods were used. All of the asphalts have been used to build pavements in the United States or Canada. Two grades of most of these asphalts were included. Test properties of the asphalts used are given in Tables 1 and 2.

Retained samples of the asphalt used on I-90 were not available. However, asphalt 1 was similar to the asphalt used on I-90.

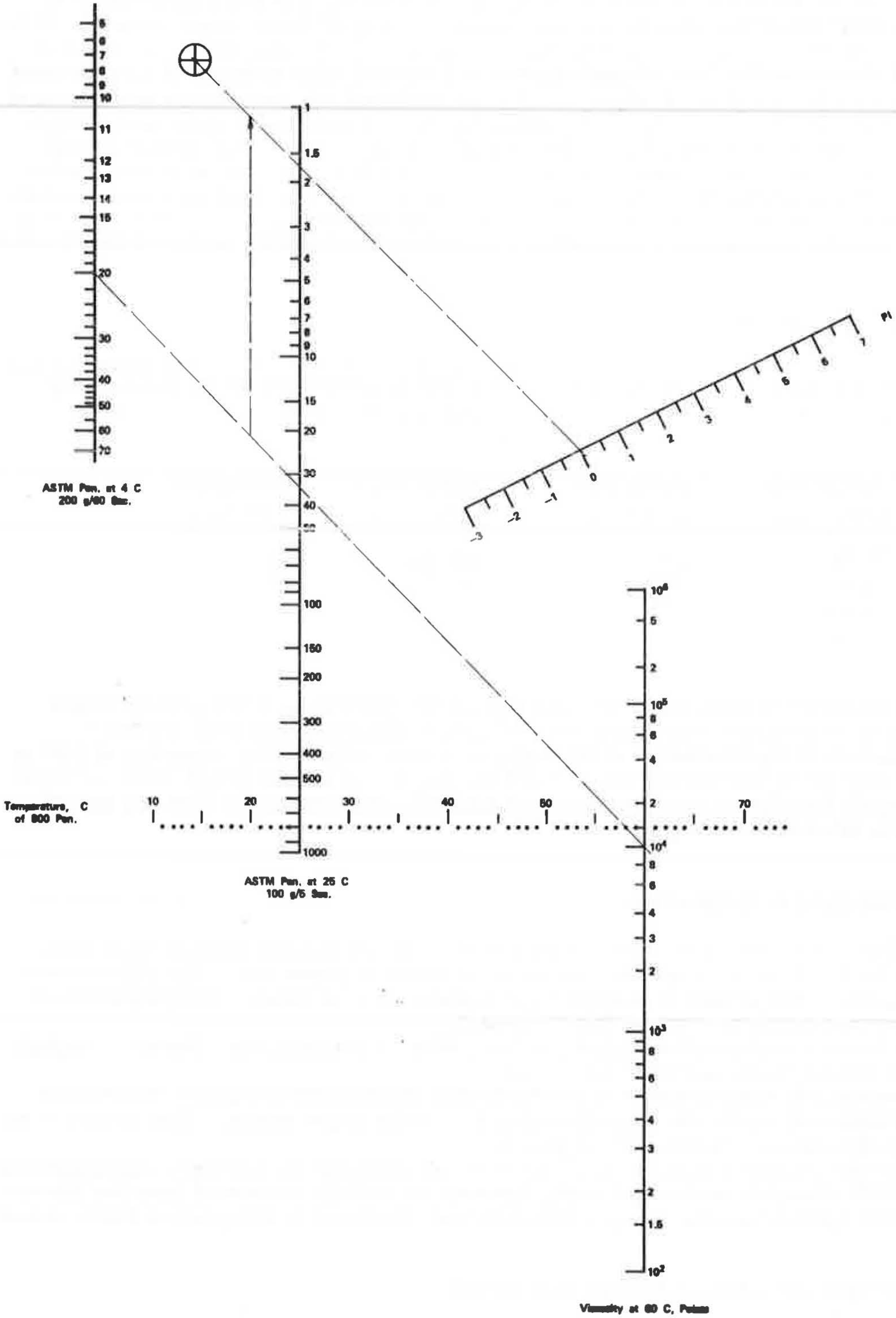
Asphalt 3 is representative of an asphalt that the Washington highway department associates with moderate thermally induced cracking in pavements. They associate no thermally induced cracking with asphalt 6.

Complete tests on asphalts 1, 3, and 6 are not available because there was insufficient material. Complete tests were made, however, on asphalts recovered from the laboratory mix specimens after creep modulus testing. Test results are given in Tables 1 and 2.

STIFFNESS OF ASPHALT-TREATED MIXES

Direct tensile creep measurements to obtain stiffness of asphalt-treated mixes are normally difficult and costly to do, particularly at the low creep rates found in the range of T_L. Tensile measurements are generally much easier and less costly when made on

Figure 1. Nomograph for obtaining PI and T_{800} penetration from ASTM tests.



cylindrical specimens tested in the split-tension or diametral configuration. In the diametral configuration a stress is applied across a vertical diameter of a cylinder, and the resultant deformation across the horizontal diameter is measured.

Very small deformations obtained at low temperatures require extra sensitive and stable instrumentation. An instrument having these requirements is routinely used to measure the dynamic stiffness or resilient modulus of Hveem or Marshall specimens (23). This instrument can also be used for creep measurements. Instead of the dynamic loads normally used with the resilient modulus device, a static load is applied to the specimen by an air-driven piston.

Deformations are determined at several temperatures and are recorded on a variable-speed strip chart. The data are translated into curves of stiffness versus time of loading. Subsequently, these curves are combined by the superposition shift method into a master curve. These master curves for each specimen permit the stiffness at any temperature or time of loading to be determined. As previously discussed, the loading time of interest is 10,000 sec. Superposition shifts can be omitted if the creep measurements are made in 10,000-sec time periods. Because this was not practical, generation of the master curve allows the 10,000-sec stiffness to be projected from shorter time data.

Curves of stiffness at 10,000 sec versus temperature were then constructed. An example of mixes used to make the previously described pavements in eastern Washington is shown in Figure 2. Each of these pavements had a different resistance to thermally induced cracking. The difference in field behavior (no cracking to severe cracking) is reflected by the 18-deg spread shown in T_L at which 10,000-sec S_{LM} equals 1,500 ksi (10 300 MPa). The laboratory specimens shown in Figure 2 were made from the same asphalts and aggregate used in the pavements. The 18-deg spread in T_L and reported field performance qualitatively support the idea that measured mix stiffness is related to thermally induced cracking.

Laboratory-prepared asphalt concrete specimens were made from each of the asphalts given in Tables 1 and 2 by using a Hveem kneading compactor. Densities and diametrally measured T_L values for each of these specimens are given in Tables 1 and 2.

ESTIMATING T_L INDIRECTLY FROM ASPHALT BINDER PROPERTIES

van der Poel's (11) nomograph was used to estimate the T_L of the asphalt binders for an S_{LA} of 20,000 psi (138 000 kPa) when stressed for 10,000 sec. The penetration indexes (PI) and temperature at which the penetration is 800 (T_{800}) used in van der Poel's nomograph were determined from Heukelom's bitumen test data chart (12) for showing the effect of temperature on the physical behavior of asphaltic bitumen. With Heukelom's chart, a number of different routine test results can be used to estimate the stiffness of the asphalt. This recent chart no longer requires that the penetration at 77 F (25 C) and softening point be the sole criteria of asphalt characteristics. Penetrations and viscosities at any temperature can be used. This allows the low-temperature stiffness to be estimated by a number of different combinations of test results.

Heukelom's test data chart does not provide for the use of the conventional ASTM low-temperature penetration, i.e., penetrations determined at a 200-g load for 60 sec. All penetration values on Heukelom's chart are for 5-sec, 100-g loading. Figure 1 is derived from the Heukelom test data chart and permits the conventional ASTM low-temperature penetrations to be used. The scale relating ASTM penetrations at 39.2 F (4 C) to the scale used by Heukelom was developed empirically by correlating a large number of penetration tests made at 39.2 F at both test conditions. This nomograph is used in the same way as the original Heukelom nomograph. That is, a line is drawn connecting a point representing the penetration at 39.2 F (4 C) with a point representing the penetration at 77 F (25 C) or with the viscosity at 140 F (60 C). The temperature of 800 penetration is taken from the intercept of this line with the horizontal scale. Another line is then drawn through the reference point parallel to the first line. The intercept with the PI scale gives the PI value.

Table 1. Properties of original asphalts and RTF-C oven residua.

Symbol on Figures	Original Asphalt: Penetration (dmm) 77 F, 100 g, 5 Sec	Original Asphalt: Penetration (dmm) 39.2 F, 200 g, 60 Sec	Original Asphalt: Softening Point (F)	Original Asphalt: Viscosity 140 F (poises)	Original Asphalt: Viscosity 275 F (stokes)	Residua From RTF-C Oven Exposure: Penetration (dmm) 77 F, 100 g, 5 Sec	Residua: Penetration (dmm) 39.2 F, 200 g, 60 Sec	Residua: Softening Point (F)	Residua: Viscosity 140 F (poises)	Residua: Viscosity 275 F (stokes)	Residua: Ductility (cm) 45 F, 1 cm/Min	Residua: Ductility (cm) 77 F, 5 cm/Min
•	111	38	118.5	1,273	2.52	63	25	130.5	4,080	4.17	11	135
•	157	52	109.5	690	1.92	86	32	120.5	1,992	2.98	27	150+
×	121	38	117.5	777	2.10	70	20	124.5	1,955	3.25	20	150+
×	91	27	123.0	1,352	2.83	53	16	132.0	3,795	4.42	13	142
Δ	150	52	117.5	771	3.00	89	37	125.5	2,191	4.53	13	112
Δ	129	47	121.5	1,124	3.44	76	31	131.0	3,567	5.34	9	68
□	197	61	110.5	679	2.68	96	32	125.5	2,011	4.47	118	150+
+	36	7	129.5	2,903	3.48	25	7	136.0	6,687	4.83	4	106
○	278	84	105.5	556	2.52	105	35	122.5	2,241	4.93	36	150+
○	117	41	117.0	1,563	4.07	60	23	129.5	5,647	7.47	23	150+
∇	60	15	122.5	1,742	2.54	38	14	136.5	3,493	3.57	13	150+
∇	85	26	114.0	1,125	2.11	53	14	127.0	2,597	3.00	106	150+
⌓	106	29	119.5	795	2.53	68	18	121.5	1,386	3.11	30	150+
⌓	217	67	106.5	565	2.17	104	38	120.5	1,940	3.62	78	150+
⌓	150	51	112.5	913	2.80	75	27	127.5	3,683	5.22	24	150+
◇	72	19	124.5	1,550	3.33	40	10	134.0	3,881	4.70	7	150+
◇	105	25	118.0	916	2.58	60	13	128.0	2,009	3.58	8	150+
①	92	—	—	900	2.20	48	—	—	2,300	3.30	35	—
③	92	—	—	1,650	3.05	52	—	—	4,800	5.00	14	—
⑥	92	—	—	1,350	2.50	50	—	—	3,100	2.52	100+	—

Note: 1 F = 1.8 C + 32; 1 poise = 0.1 Pa·s; 1 stoke = 0.0001 m^2/s.

Table 2. Properties of residua recovered from creep specimens and mix specimens tested in creep.

Symbol on Figures	Residua Recovered From Specimens Tested in Creep: Penetration (dmm) 77 F, 100 g, 5 Sec	Penetration (dmm) 39.2 F, 200 g, 60 Sec	Softening Point (F)	Viscosity 140 F (poises)	Viscosity 275 F (stokes)	Ductility at 45 F, 1 cm/Min (cm)	Mix Specimens Tested in Creep: Density (lb/ft^3)	Volatile Fraction of Aggregate	T_L (F)
•	55	27	133.5	5,123	4.43	8	150.9	0.82	-13
•	92	37	123.0	1,750	2.89	44	150.7	0.82	-20
×	61	23	124.5	2,429	3.51	33	151.5	0.82	-6
×	60	17	135.5	7,103	5.79	8	151.7	0.82	0
Δ	115	46	121.5	1,468	3.76	24	149.7	0.81	-17
Δ	84	39	127.0	1,691	3.59	14	151.6	0.82	-15
□	87	34	125.0	2,912	5.32	38	153.5	0.83	-7
+	36	12	132.0	3,595	4.05	6	152.2	0.82	+10
○	104	39	122.0	2,091	4.64	101	151.4	0.82	-16
○	74	33	129.0	3,877	6.43	52	151.2	0.82	-10
∇	47	14	132.5	3,524	3.63	24	152.1	0.82	+4
∇	40	14	128.5	3,686	3.72	20	152.3	0.82	+2
⌓	62	20	128.0	1,993	3.39	18	151.4	0.82	+4
⌓	125	44	116.5	1,360	3.25	139	153.4	0.83	-15
⌓	90	35	118.5	2,349	4.19	127	153.5	0.83	-11
◇	38	15	132.0	4,453	4.89	7	153.4	0.83	+4
◇	64	22	120.5	2,044	3.63	23	154.6	0.84	0
①	32	12	133.5	4,336	4.16	17	154.3	0.83	+12
③	39	15	133.5	5,201	4.65	12	154.5	0.84	+1
⑥	50	19	130.0	3,841	4.42	14	154.3	0.83	-6

Note: 1 F = 1.8 C + 32; 1 poise = 0.1 Pa·s; 1 stoke = 0.0001 m^2/s; 1 lb/ft^3 = 16 kg/m^3.

Figure 2. Determination of T_L.

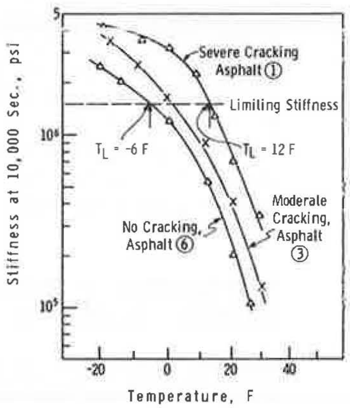


Figure 3. T_L predicted from penetrations at 77 F and 39.2 F of RTF-C residua.

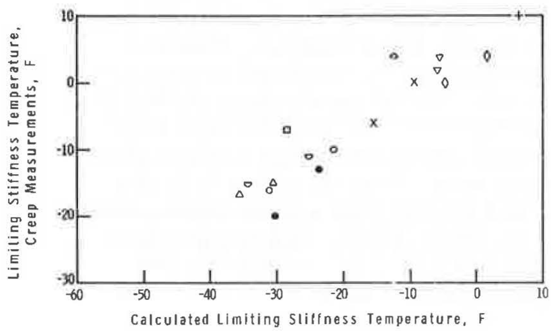
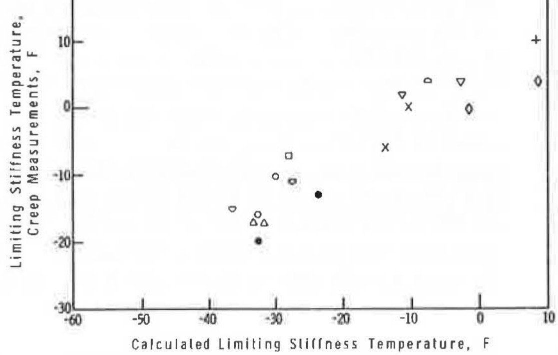


Figure 4. T_L predicted from penetrations at 39.2 F and viscosities at 140 F of RTF-C residua.



COMPARISON OF MEASURED T_L WITH T_L ESTIMATED FROM RTF-C RESIDUA

As previously discussed, the purpose of this investigation is to compare measured T_L with T_L estimated from ordinary ASTM asphalt tests.

Penetrations at 77 and 39.2 F (25 and 4 C)

A comparison of T_L values obtained by diametral creep measurements on the mixes with T_L values estimated from the ASTM penetrations at 77 and 39.2 F (25 and 4 C) of the RTF-C residua is shown in Figure 3. The correlation is good, but the relationship is shifted about 10 deg. There are no large outliers. A difference of 10 to 15 deg between the measured and calculated T_L values is an inherent limitation of estimating the stiffness of a mix from the stiffness of an asphalt. It is only expected to be accurate to within 2 or 3 deg of the actual stiffness of a mix. For this comparison, the same aggregate was used in all cases so that a consistent bias (or mix factor) exists in all samples.

Viscosity at 140 F (60 C) and Penetration at 39.2 F (4 C)

As shown in Figure 4, T_L values estimated from viscosity at 140 F (60 C) and penetration at 39.2 F (4 C) also correlate well with measured T_L values. Again there are no outliers.

Penetration at 77 F (25 C) and Softening Point

Figure 5 shows that T_L values estimated from penetration at 77 F (25 C) and softening point correlate poorly. Outliers are not limited to waxy asphalts.

Penetration at 77 F (25 C) and Viscosity at 140 F (60 C)

Figure 6 shows that T_L values estimated from penetration at 77 F (25 C) and viscosity at 140 F (60 C) also correlate poorly.

Viscosity at 140 F (60 C) and Viscosity at 275 F (135 C)

As shown in Figure 7, there is no correlation between the measured T_L and the T_L estimated from the viscosities at 140 and 275 F (60 and 135 C). These tests appear to be useless for estimating low-temperature stiffness.

Comparison of T_L and Ductility at 45 F (7.2 C)

Ductility at low temperature is often assumed to relate to pavement performance. No method is available to translate this property to a T_L; therefore, the significance of the ductility value could not be tested in the same way as were the other tests. Instead, the ductility values were plotted against the viscosity at 140 F (RTF-C residua). Ductility values were interpolated or extrapolated to 2,000 and 4,000 poises (200 and 400 Pa·s). (These are the center points of the viscosities of the AR-2000 and AR-4000 grades of asphalt as defined in the Pacific Coast uniform paving asphalt specification.) These adjusted ductility values are shown in Figures 8 and 9 versus measured T_L values (also interpolated to the same asphalt grades). No correlation is shown between ductilities at 45 F (7.2 C) and measured T_L for either the 2000 or 4000 grade asphalts.

Figure 5. T_L predicted from penetrations at 77 F and softening points of RTF-C residua.

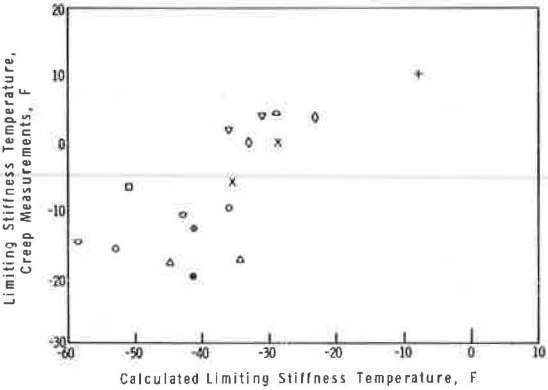


Figure 6. T_L predicted from penetrations at 77 F and viscosities at 140 F of RTF-C residua.

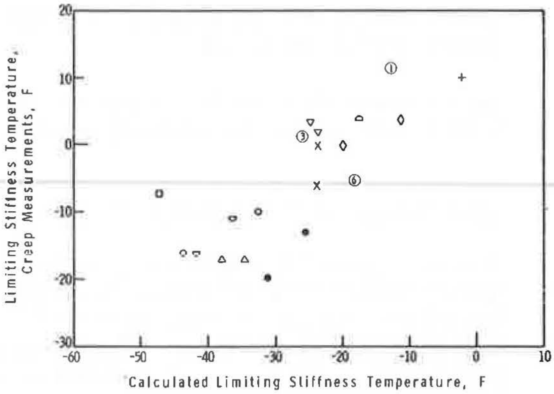


Figure 7. T_L predicted from viscosities at 140 F and 275 F of RTF-C residua.

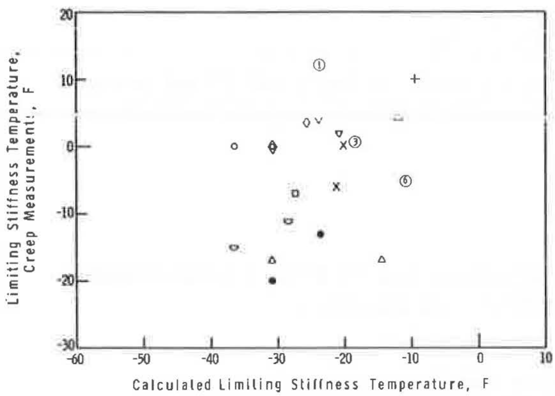


Figure 8. Relationship of T_L of pavements and ductilities at 45 F of RTF-C residua, AR-2000 graded asphalt.

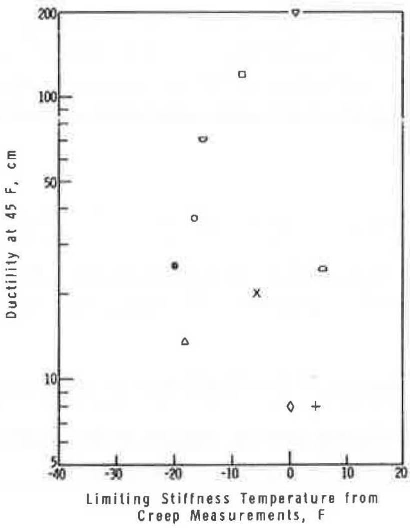


Figure 9. Relationship of T_L of pavements and ductilities at 45 F of RTF-C residua, AR-4000 graded asphalt.

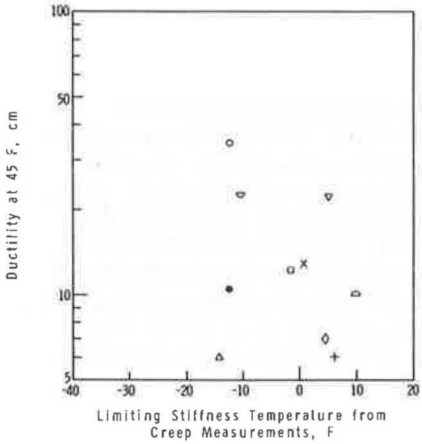
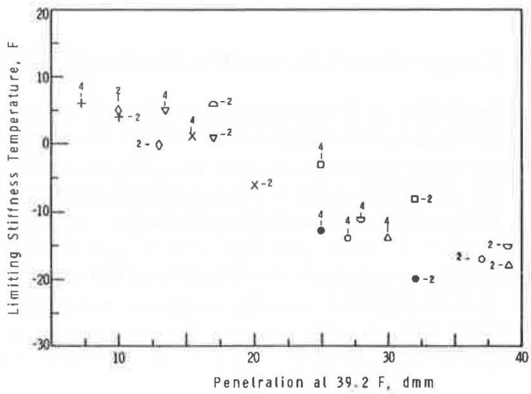


Figure 10. Relationship of T_L and penetrations at 39.2 F of RTF-C residua (2 = AR-2000 grade and 4 = AR-4000 grade).



A similar lack of correlation is shown when T_L and 45 F ductilities are adjusted to penetration grades. Fromm and Phang's extensive studies (10) also showed no correlation between low-temperature ductilities and the extent of thermally induced pavement cracking.

This lack of correlation is not surprising because the ductility test involves both the viscosity and the shear sensitivity of a material. A ductility value makes no distinction between these two properties. Ductility may be significant when it is determined at the same temperature as another test, such as the penetration test, which can be used to estimate stiffness. That is, for a given stiffness, the shear sensitivity as measured by ductility may be of significance. However, when comparisons are made at different stiffnesses, the meaning of ductility appears to be obscured.

COMPARISON OF MEASURED T_L AND PROPERTIES OF RECOVERED ASPHALTS

Measured T_L values were also shown to correlate well with the T_L values estimated from penetration at 39.2 F (4 C) with penetration at 77 F (25 C) or viscosity at 140 F (60 C) of residua recovered from the specimens.

Poor correlations were obtained for T_L estimated from penetration at 77 F and softening point or viscosity at 140 F. No correlation was found with T_L estimated from viscosities at 140 and 275 F. Also, no correlation was found between ductilities and T_L.

PRACTICAL USE OF ASTM 39.2 F (4 C) PENETRATION TO CONTROL T_L

The excellent correlation shown in Figure 4 between T_L obtained from penetration at 39.2 F (4 C) of the RTF-C residua and the viscosity at 140 F suggests that T_L can be obtained from the penetration at 39.2 F of an AR-graded asphalt (RTF-C residue viscosity at 140 F). Figure 10 shows this for the same series of asphalts interpolated for AR-2000 and AR-4000 grades. Both sets of data superpose to form a single band. The importance of using both a softer asphalt and one having a lower temperature susceptibility is evident.

This single curve supports the concept that specifying asphalts with higher penetrations at 39.2 F will limit thermally induced pavement cracking. The limits should be set at levels consistent with available asphalt supplies and the minimum pavement temperature expected.

CONCLUSIONS

1. The relationship reported between thermally induced pavement cracking and the T_L value at which the 10,000-sec asphalt stiffness is 20,000 psi (138 000 kPa) is qualitatively supported by field tests.
2. T_L estimated from ASTM penetration at 39.2 F (4 C), along with the ASTM penetration at 77 F (25 C) or with the viscosity at 140 F (60 C) on RTF-C oven residua, correlates well with measured T_L. This good correlation has no large outliers even though an extreme diversity of asphalt types was included. The use of penetration at 39.2 F (4 C) on an RTF-C residue is supported as a specification requirement for AR-graded asphalts. With penetration or AC-graded asphalts, penetration at 77 F or a viscosity at 140 F on the oven residue is also needed.
3. Good correlation is also found on asphalts recovered from the mixes.
4. Poor correlations are shown between T_L and penetration at 77 F along with softening point or with viscosity at 140 F on both RTF-C and recovered residua.
5. T_L estimated from viscosities at 140 F, along with viscosities at 275 F on RTF-C or recovered residua, shows no correlation with measured T_L. Resistance to low-

temperature thermally induced cracking should not be implied on diverse types of asphalts based on high-temperature viscosity measurements.

6. Ductilities at 45 F show no correlation with T_L, possibly because the ductility test cannot distinguish between the shear sensitivity and the viscosity or stiffness of asphalts. Despite the lack of correlation of the 45 F ductility with T_L, the conventional ductility test at 77 F together with a penetration test at 77 F may have significance in limiting types of pavement failure other than thermally induced cracking.

7. Tensile stiffness and, in turn, T_L of asphalt-treated aggregate mixes are readily measured by compression-stressing cylindrical specimens across a vertical diameter while the resultant tensile deformation across a horizontal diameter (i.e., diametral creep measurement) is measured.

8. The modified asphalt test data chart permits the low-temperature stiffness of asphalts to be determined from normal ASTM penetration tests.

ACKNOWLEDGMENT

The author greatly appreciates the help and suggestions given during the course of this project by R. V. Le Clerc and T. Marshall of the Washington State Highway Department.

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DISCUSSION

Richard L. Davis, Koppers Company, Inc.

This is a very fine paper that should be studied by all who have an interest in the low-temperature performance of asphaltic mixtures and in particular by those who have an interest in asphalt specifications. This paper shows the importance of measuring the stiffness or consistency of asphalt binders at lower temperatures to control the low-temperature properties of asphalt mixtures. Data shown in Figure 8 illustrate the point made at the Symposium on Viscosity Grading of Asphalts at the 1971 Highway Research Board Annual Meeting (25) that viscosities at 140 and 275 F (60 and 135 C) do not give satisfactory control of binder or mix properties at lower temperatures.

The nomograph shown in Figure 2 should be very helpful in making use of ASTM penetrations at 39.2 F (4 C). That the correlation between the calculated limiting stiffness and the measured limiting stiffness improves when lower temperature penetration test results are used in the calculations suggests the importance of investigating still lower temperature penetration methods.

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COMPOSITIONAL CONSIDERATIONS OF ASPHALT FOR DURABILITY IMPROVEMENT

Rowan J. Peters, Arizona Department of Transportation

An investigation was performed to determine whether a laboratory procedure could be established that could provide a means to distinguish the durability (aging) qualities of asphalts when both their chemical and physical characteristics are considered. The investigation was also aimed at developing the means for predicting the effects on viscosity or an aging index of the interblending of various sources of asphalt. The investigation compared the effects on viscosity and aging indexes among asphalts, asphalt blends, and asphalt-additive blends. The research indicated that (a) asphalts can be blended to achieve practically any desired viscosity or aging index or combination of both, (b) Gilsonite in small quantities can be added to asphalt to achieve a desired viscosity without a subsequent loss of durability, (c) the chemical reactivity ratio can predict which base stock asphalts will age the fastest, and (d) the second acidaffin fraction appears to be the most important fraction for the improvement of durability (aging) and is available in commercial products or in selected asphalts for blending.

•IN ARIZONA, two problem areas, asphalt-aggregate adhesion and asphalt durability, have needed investigation. The Arizona Department of Transportation has expended considerable construction and maintenance funds on additive treatments to overcome potential adhesion problems and on asphalt rejuvenating agents to restore asphalt durability to roadway surfaces.

The problems are influenced by many factors such as asphalt characteristics, aggregate properties, mix design, environment, construction practices, and traffic loadings.

An investigation was initiated to consider the possibility of optimizing the composition of asphalt by blending known asphalt supplies.

The methods of evaluation are discussed, and the control method under consideration, i.e., that of possibly improving the durability of asphalt by compositional blending, is explored.

This study of asphalt durability was limited to the laboratory. Field evaluations of durability that do not rely on visual comparison or measurement systems independent of pavement sample recovery methods are subject to inherent weaknesses. To study asphalt aging in the field through use of recovered pavement sections or cores requires that the asphalt be totally recovered because selective absorption of asphalts with various aggregates has a surprisingly strong effect. The lighter volatile fractions that remain in the aggregate by selective absorption greatly influence subsequent testing. A very small quantity of these lighter fractions will considerably affect viscosity, penetration, and resulting chemical fractions.

Work conducted at Pennsylvania State University (1) indicated that the study of pavement durability is difficult because its effects are overshadowed by design considerations and actual air voids, aggregate types, and average daily traffic volumes. Also, in evaluation of asphalt performance, the pavements being compared must be sampled at the same time of year.

Work by Vallerga and Halstead (2) indicated similar problems in correlating pavement distress with original or present asphalt properties, and they recommend that experiments to establish causes of pavement distress be better controlled and have as few variables as possible.

The differences between laboratory and field evaluations must be considered in the development of any field experiment. The differences exist because of the intrinsic nature of research, i.e., the environment is beyond control. Construction variables exist within and between test and control areas. The ability to establish two identical areas for study of one variable is indeed questionable. The problems in sample recovery have already been discussed.

The state of the art of predicting or measuring the durability of asphalt cements has not developed to the point where there are realistic and meaningful tests and specifications that relate to any form of field performance. The California Division of Highways developed a procedure in 1959 that applies heat and air aging to a rolling thin film of asphalt. The rolling thin film oven (RTFO) test was intended to predict the effects of hot-mix aging of asphalts. The original intent was to correlate the aging effects with actual asphalt plant aging. Subsequent work showed little correlation between the RTFO results and field aging; the RTFO test has been modified and is currently used only as a means to grade asphalt cements by viscosity.

The rolling microfilm circulating (RMFC) laboratory procedure reported by Schmidt (3) was designed around the modified RTFO. It was designed to relate laboratory and field predictions for durability, but a considerable amount of correlation work is needed before the procedure can be considered as a testing and specification tool.

Therefore, based on the problems of establishing uniform field experiments and because there are no laboratory-field correlated testing procedures, this durability study was limited to a laboratory evaluation based on ASTM procedure D 2872. Although this study was intended to be a relative comparison of the differences among asphalts, asphalt blends, and asphalt-additive blends and not an absolute quantity subject to field correlations, we feel a noteworthy observation and review can be made.

DEVELOPMENT OF A QUICKER ROSTLER PROCEDURE

Before asphalts and asphalt blends were tested for durability in the rolling thin film oven (ASTM D 2872), a chemical method of asphalt fraction analysis devised by Rostler and Sternberg (4) and modified for operational simplicity was established. Previous asphalt analyses resulted in problems with the standard Rostler procedure because the normal analysis required 50 hours and because less than a complete chemical analysis was found to be adequate.

The 50-hour procedure is necessary for complete chemical separation. The asphalt is separated into six fractions, five of which are based on chemical reactivity and one on solubility (5, 6, 7). If the asphaltene (A) fraction and the chemical reactivity ratio (CRR) are of primary interest, then the A fraction is precipitated with normal pentane and the remainder is treated with 98 percent sulfuric acid. Since this leaves the second acidaffin (A_2) and paraffin (P) fractions, one can then determine the CRR by the difference.

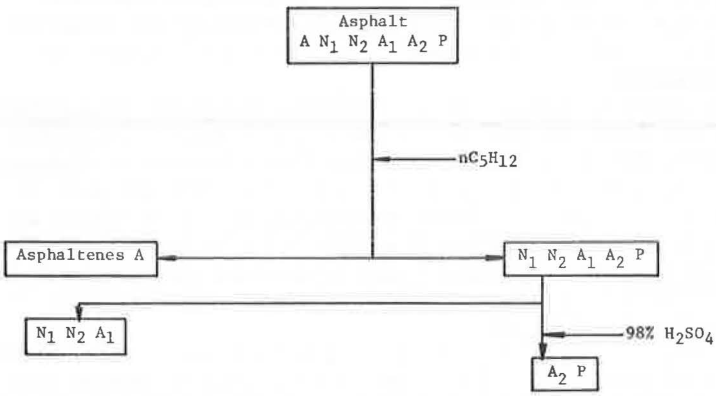
By using the modified procedure (Figure 1), a complete analysis can be performed in 8 hours or less. Because fuming sulfuric acid is not needed in the modified procedure, the analysis is safer and can be performed by laboratory personnel with a minimum of training. As with the standard analysis, the normal practice of six simultaneous analyses was continued. A comparison of the standard and modified procedures is given in Table 1.

ASPHALT BLENDING PROGRAM

Blends were produced by mixing a base stock asphalt with either base stock asphalt from a different source or an asphalt additive or both. Regional location descriptions of the base stock asphalts used are given in Table 2.

The table also indicates the Rostler separation for each base stock and additive. It was assumed that Rostler separations did not change significantly from sample to sample taken from the base stock (sealed can of asphalt). It was also assumed that, when one base stock was blended with another base stock or additive or both, no reaction occurred.

Figure 1. Modified Rostler-Sternberg analysis.



A -	Asphaltenes	Insoluble in normal Pentane-Tech.
N_1 -	1st Nitrogen Bases	Precipitated by 98% Sulfuric Acid
N_2 -	2nd Nitrogen Bases	Precipitated by 98% Sulfuric Acid
A_1 -	1st Acidaffins	Precipitated by 98% Sulfuric Acid
A_2 -	2nd Acidaffins	Unreacted by 98% Sulfuric Acid
P -	Paraffins	Unreacted by 98% Sulfuric Acid

Table 1. Comparison of results of standard Rostler and modified procedures.

	Asphaltenes (percent)		Chemical Reactivity Ratio	
Asphalt	Standard	Modified	Standard	Modified
85/100	19.7	19.6	1.18	1.27
60/70	19.9	19.6	1.80	1.81
85/100, unaged	21.9	21.9	2.07	2.00
85/100, 75 min[a]	23.7	23.6	2.22	2.19
85/100, 300 min[a]	29.0	29.0	1.90	1.86
60/70, unaged	25.9	25.4	1.75	1.74
60/70, 75 min[a]	27.2	27.0	1.78	1.68
60/70, 300 min[a]	35.1	35.1	1.56	1.49

[a]Time in rolling thin film oven at 325 F (163 C).

Table 2. Rostler fractions for asphalt base stocks and additives.

Base Stock	A	N	A_1	A_2	P	CRR	Description
Los Angeles basin 40/50	19.0	37.1	8.0	24.4	11.5	1.26	From crudes that have gravities of approximately 23.5 deg American Petroleum Institute (API) and yield about 40 percent residuum.
Los Angeles basin 85/100	18.5	33.7	13.7	22.3	11.8	1.39	From crudes that have gravities of approximately 23.5 deg API and yield about 40 percent residuum.
Four Corners 85/100	3.7	24.9	23.4	36.9	11.2	1.00	From crudes that can have gravities of more than 30 deg API and usually are low in residuum yield (below 30 percent).
Gilsonite	75.2	20.6	1.0	2.3	0.9	6.75	Using Sparkling Black Grade. Asphaltenes probably have a molecular weight of 10,000 to 14,000 when extracted with normal pentane.
Emulsified petroleum resin	0.6	32.3	16.2	34.8	12.9	1.01	Derived from extraction refining of lube oils. Very high in the A_2 fraction.
Santa Maria	29.2	32.6	9.4	20.8	8.0	1.46	From crudes that have gravities of approximately 15.0 deg API and yield about 60 percent residuum.

That is, each base stock contributed to each Rostler fraction as a function of weight.

After a blend was made, approximately 50 g of material was separated for an unaged microviscosity and 35 g of material for aging in the RTFO, and 50 g was saved for future reference.

Microviscosities of unaged and aged blend material were measured on a Hallikainen sliding plate microviscometer at a constant temperature of 77 F (25 C), which is the proposed ASTM procedure. Glass plates were typically used with unaged blends, except for those blends with viscosities of about 50 megapoises (5 MPa·s). For the very viscous blends, steel plates were used. For blend material aged 300 min in the RTFO, steel plates again were used to measure the microviscosity. For all cases, a sample of blended material was tested with successively lighter weights, which imposed smaller shear rates. Usually at least four shear rates were imposed on a sample. Based on the shear rates and shear stresses, the microviscosity was determined for a constant shear rate of 0.05 sec^{-1}. For each blend, calculated Rostler values and unaged and aged microviscosities were determined.

Discussion of Results

The following rule was adopted to describe the way various blends were formed: Whatever constituent increased the viscosity of the base stock or base blend was considered the constituent being added to the base. For example, when 6-megapoise (0.6-MPa·s) Los Angeles basin 40/50 was blended with 1-megapoise (0.1-MPa·s) Four Corners 85/100 equally by weight, the resulting viscosity was 4 megapoises (0.4 MPa·s). This blend would be described as 50 percent Los Angeles basin 40/50 added to Four Corners 85/100, since this addition raised the final viscosity above that of the Four Corners 85/100.

All blend data included the concentration in percent of one constituent added to the base, microviscosity of the unaged blends in megapoises, the blend aged 300 min in RTFO microviscosity in megapoises, and the aging index. A listing of all blend data and subsequent tables and figures are available in the project final report (8).

After examination of the blend data, it appeared that the relationship of viscosity to concentration of "added to" constituent was a straight line on semilogarithmic paper; i.e., $\log Y = A + B (X)$, where X is the added to concentration and Y is viscosity (Figure 2). Also the theoretical aging index line, arrived at by dividing a value from the 300-min aged fitted straight line by the unaged straight line value at a constant added to concentration, became a measure of the convergence or divergence of the fitted straight lines found. The greater the divergence of the two lines was, the faster the material aged and vice versa. Figure 3 shows an example of a theoretical aging index line and the actual aging index values.

Review of the aging effects of the Four Corners crude (Figures 4 through 8) indicates that there must be a reason for the lower rates of aging for this crude source. From the listing of the Rostler fractions given in Table 2, the chemical nature of the Four Corners crude appears to be significantly different from that of the other crudes listed. The most noticeable differences are the low asphaltene content and comparatively high second acidaffin fraction. The Four Corners crude compares closest to that of the emulsified petroleum resin except for the nitrogen base (N) and A_1 fractions. However, the N and A_1 fraction contents of the emulsified petroleum resin are also similarly prevalent in the other crude sources. The factor common to both the emulsified petroleum resin and the Four Corners crude and the factor that is significantly different from all other crudes listed is the acidaffins and primarily the A_2. Since experience has indicated the rejuvenating effects of the emulsified petroleum resins and the apparent aging benefits of the Four Corners crude are borne out in the RTFO evaluation, it would appear that the one common factor affecting durability (aging) must then be the second acidaffins.

Figure 2. Aging index versus percentage of L. A. basin by weight added to Four Corners.

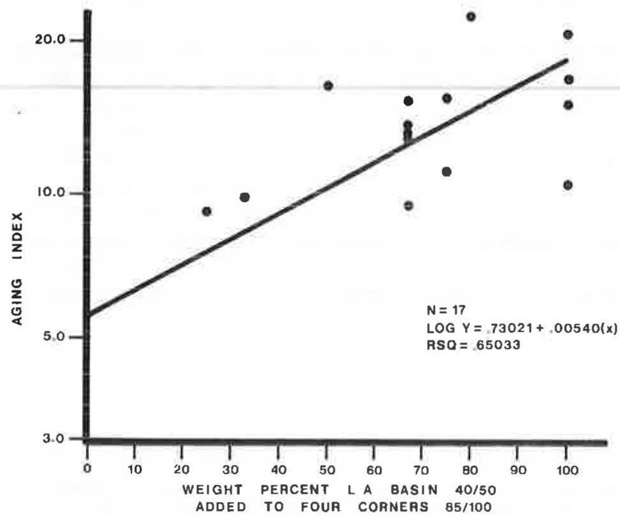
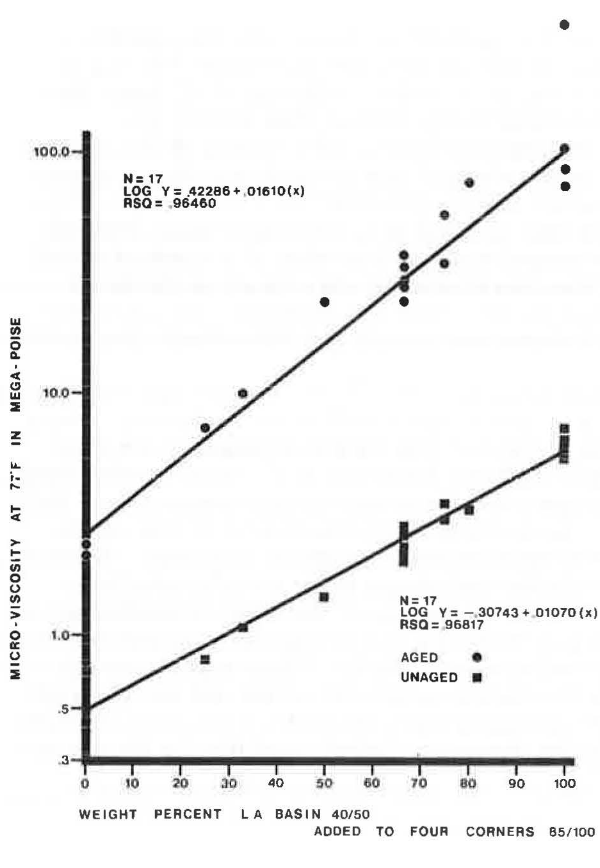


Figure 3. Microviscosity versus percentage of L. A. basin by weight added to Four Corners.



Extended Time Study

Additional information on aging was determined by subjecting selected asphalts to increasing times in the RTFO. The asphalts were removed from the RTFO at 75-, 180-, 240-, and 300-min intervals. Microviscosities were measured at 77 F (25 C), and modified Rostler tests were then performed (Table 3).

A plot of time in the RTFO versus the resulting microviscosity is shown in Figure 4. Also a plot of time in the RTFO versus aging index is shown in Figure 5. The slope of the lines formed from both Figures 4 and 5 is given in Table 4.

The viscosity and aging index increased with time in the RTFO (up to 300 min) as a function of $\log Y = A + B (X)$, where X is time in the oven and Y is viscosity. The function was a straight line on semilogarithmic paper. The CRR gave a good prediction of which asphalt would age the fastest.

The Los Angeles basin 85/100 and the Four Corners 85/100 were then aged in the RTFO at 1.25-, 5-, 16-, and 24-hour intervals. Figure 6 shows the curves for the two asphalts and the resultant aging index relationship of $\log Y = A + B (X)^{0.50}$. It appears that, in the 0 to 300-min range of aging, a straight line approximation is satisfactory and further confirms previous data analysis. An attempt to study the effects of extended aging times on a blend of two asphalts with similar unaged viscosities resulted in the findings shown in Figure 7. Apparently as greater quantities of the faster aging asphalt (pure A) are added to the slower aging asphalt (pure B), the aging index increases as does the viscosity of initial blends. A design viscosity and aging index apparently could be determined for any given series of asphalt blends if desired.

Effects of Gilsonite on Aging

Figure 8 shows the results of blending a slow aging asphalt (pure A) with Gilsonite. For this theoretical treatment, the unaged viscosity of Gilsonite was hypothesized to be 9,000 megapoises (900 MPa·s) inasmuch as a means for directly determining its unaged viscosity at 77 F (25 C) was not available. Subsequent extrapolation of data, however, indicated this value to be conservatively low. The aging index was apparently lowered, and the unaged viscosity was greatly increased. Because the viscosity of Gilsonite is so high, it seems reasonable to expect small additions of Gilsonite (less than 10 percent by weight) to have little detectable influence on aging index although the increase in viscosity will be quite large.

Closing Comments

The physical or chemical properties that influence the rate of aging are probably different from asphalt to asphalt and also between environments. To understand these physical and chemical properties requires that the crude oil source and what processing the crude oil was exposed to (refining process) be known and that the various compounds in the asphalt (composition, size, molecular weight) be investigated thoroughly.

The relationship between durability and its importance to asphalt mixture design needs to be established. Asphalt blends that improve durability may be needed for specific uses, such as surface treatments, whereas the larger volume usage such as in deep lift construction may not need this total refinement. The development of specifications for these controls will require knowledge and use of chemical control in the future. Because of the growing safety interests for more open-graded mixtures for surface courses, asphalts need to be chemically designed for totally different environments than currently considered, i.e., dense graded, low void designs.

In an attempt to define the properties eventually needed, the Arizona Department of Transportation is conducting studies to determine the effects of natural environmental aging on various blends produced under this program. An experimental feature was also added to a construction project in 1972 wherein a blend was made and used as the binder for a seal coat. Discussions of these projects should be forthcoming.

Table 3. Properties of asphalts aged in RTFO.

Type	Time in RTFO (min)	Viscosity at 77 F (megapoises)	A	N + A_1	A_2 + P	CRR	Aging Index[a]
Los Angeles basin 85/100	0	2.08	13.68	47.09	39.24	1.20	1.0
	75	4.43	18.75	44.16	37.08		2.1
	180	8.54	23.31	47.29	26.76		4.1
	240	9.59	25.96	49.00	27.69		4.6
	300	23.50	26.73	44.05	29.22		11.3
Los Angeles basin 40/50	0	5.83	25.13	47.95	26.91	1.78	1.0
	75	12.80	27.90	45.30	26.80		2.2
	180	31.50	32.47	42.20	25.33		5.4
	240	50.80	32.09	39.19	28.72		8.7
	300	52.60	37.10	38.02	24.87		9.0
Los Angeles basin 60/70	0	1.90	18.43	50.55	31.01	1.63	1.0
	75	4.18	21.72	50.14	28.14		2.2
	180	6.55	27.20	43.53	29.28		3.4
	240	14.70	28.89	41.08	30.03		7.7
	300	18.70	29.16	46.27	24.67		9.8
Four Corners 85/100	0	0.61	8.58	47.32	44.10	1.07	1.0
	75	1.10	10.76	45.96	43.28		1.3
	180	1.69	11.59	46.42	41.99		2.1
	240	2.20	12.53	46.17	41.29		3.6
	300	2.10	13.48	44.87	41.65		3.5
Santa Maria 85/100 (No. 1)	0	0.94	29.42	47.17	23.41	2.01	1.0
	75	3.78	31.55	44.03	24.42		4.0
	180	7.72	36.95	41.42	21.62		8.1
	240	13.10	39.95	39.23	20.83		13.9
	300	18.90	42.14	37.34	20.52		20.0
Santa Maria 85/100 (No. 2)	0	1.03	26.39	47.43	26.19	1.81	1.0
	75	2.90	29.14	45.17	25.69		2.8
	180	5.14	30.46	44.70	24.83		5.0
	240	11.90	33.01	41.74	25.25		11.6
	300	17.20	37.00	38.29	24.72		16.7

Note: 1 F = 1.8 C + 32; 1 megapoise = 0.1 MPa·s.

[a] $\frac{\text{Viscosity of aged asphalt}}{\text{Viscosity of unaged asphalt}}$.

Figure 4. Microviscosity versus time in RTFO for various crude sources.

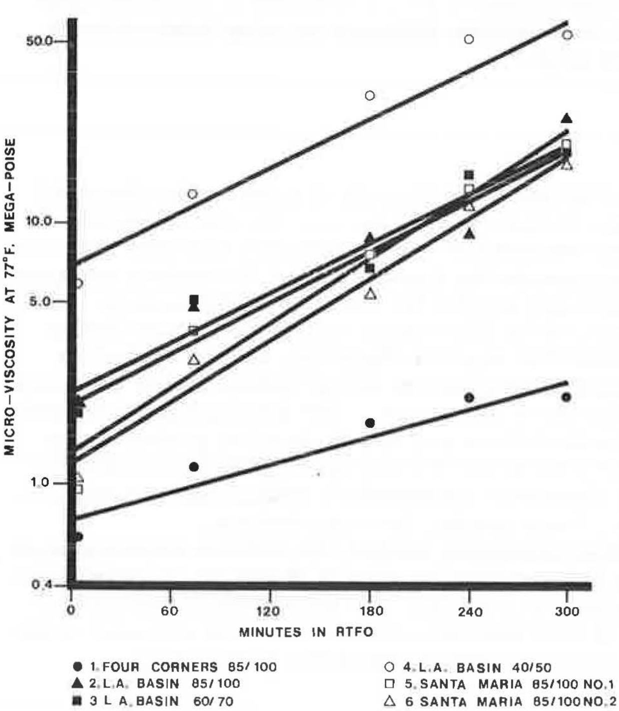


Figure 5. Aging index versus time in RTFO for various crude sources.

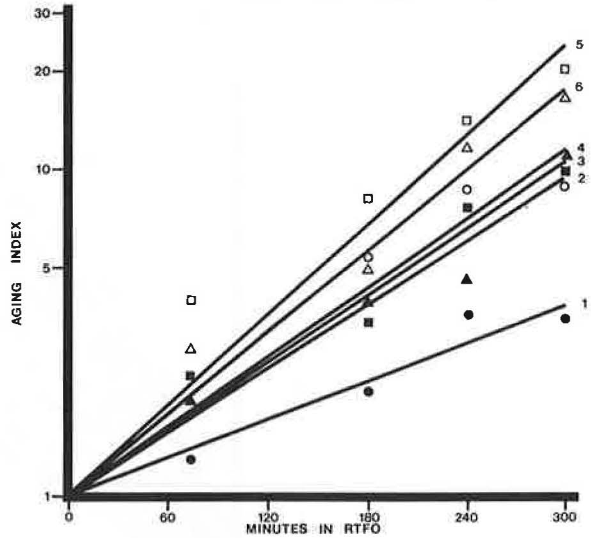


Table 4. Rate of aging of various asphalts.

Type	Viscosity/ Time[a]	Aging Index/Time[b]	CRR
Four Corners 85/100	0.00423	0.00464	1.07
Los Angeles basin 85/100	0.00731	0.00733	1.20
Los Angeles basin 60/70	0.00758	0.00756	1.63
Los Angeles basin 40/50	0.00723	0.00769	1.78
Santa Maria 85/100 (No. 1)	0.00917	0.00919	1.81
Santa Maria 85/100 (No. 2)	0.00954	0.00953	2.01

[a]From Figure 3. [b]From Figure 4.

Figure 6. Microviscosity versus time in RTFO for slow and fast aging asphalts.

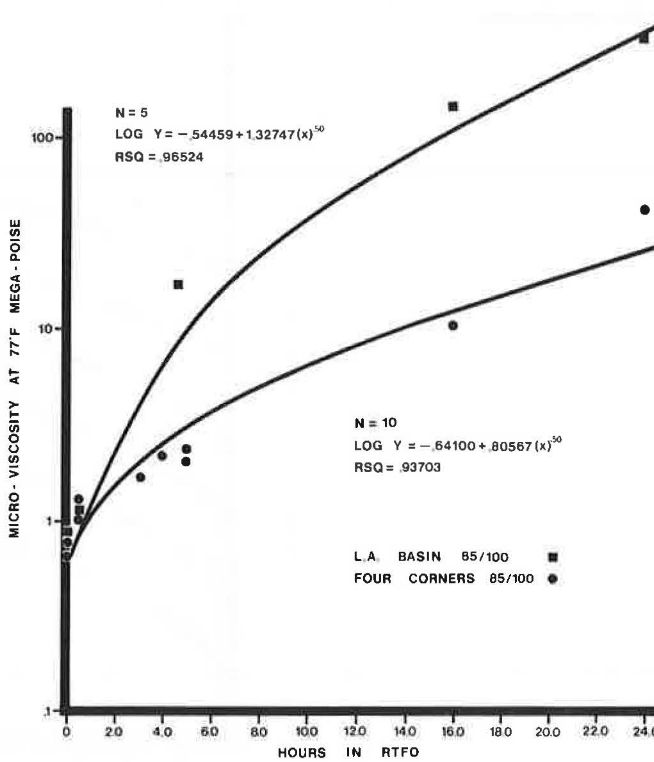


Figure 7. Effect of aging on blend of two asphalts.

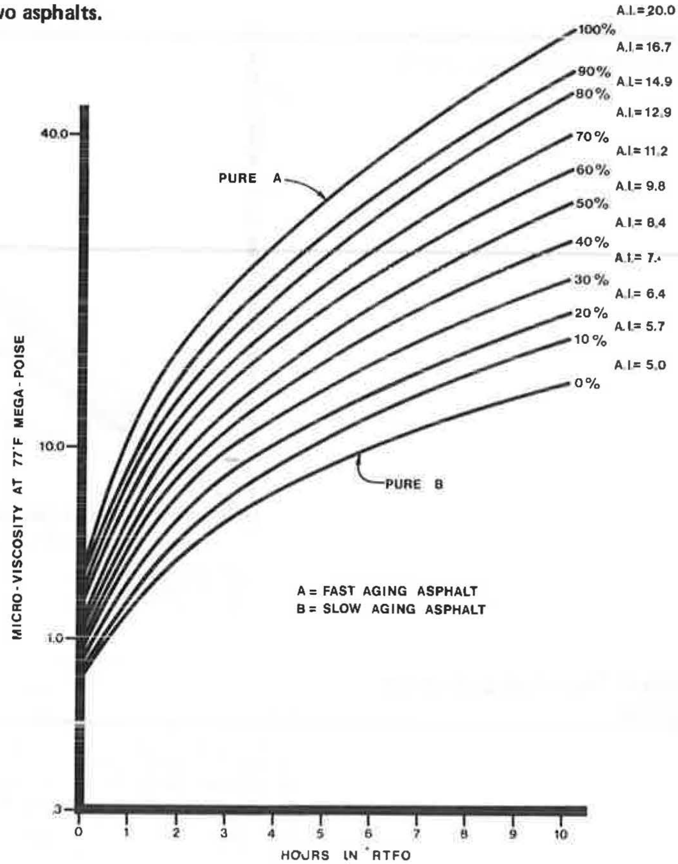
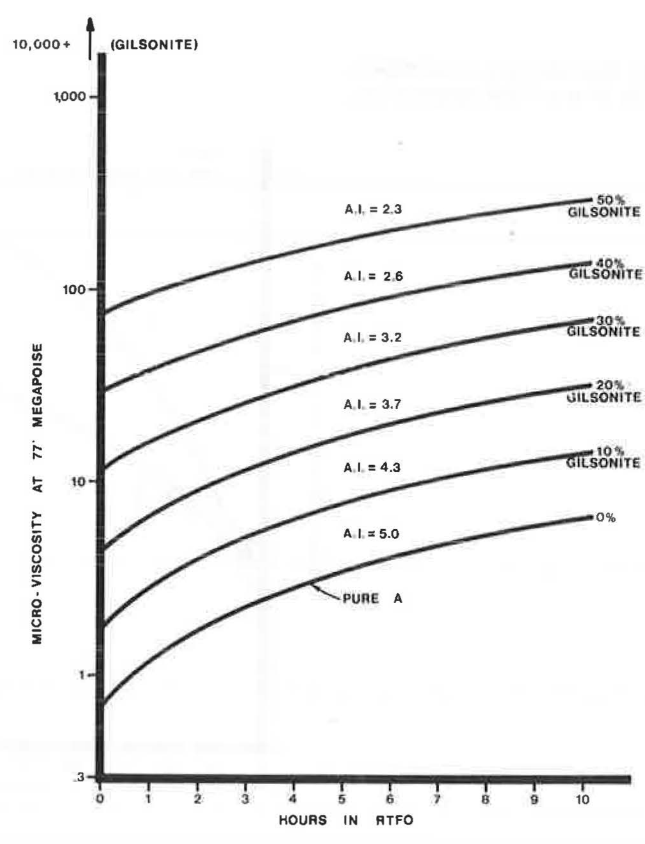


Figure 8. Effect of Gilsonite added to a slow aging asphalt.



CONCLUSIONS

1. Asphalt ages in the RTFO for times longer than 5 hours by the function, log Y = A + B $(X)^{0.50}$, where Y is viscosity and X is time in hours.
2. Asphalt ages in the RTFO for times less than 5 hours by the function, log Y = A + B (X), where Y is viscosity and X is time in hours.
3. The chemical reactivity ratio can predict which base stock asphalts will age the fastest.
4. Asphalts can be blended together to achieve practically any desirable viscosity or aging index or combination of both. The function describing the blending process is log Y = A + B (X), where Y is either viscosity or aging index and X is the concentration of the added to constituent.
5. If the difference between the unaged viscosity of two asphalts is smaller than the difference between the aged viscosities (5 hours in the RTFO or more) of the same asphalts, the aging index will increase with increasing viscosity of any blends of the two asphalts. Likewise, if the original difference in viscosity is greater than the aged difference in viscosity, the aging index will decrease with increasing viscosity of any blends of the two asphalts.
6. Gilsonite can be added to asphalt to achieve desired viscosity level without a subsequent loss in aging properties.
7. The second acidaffin fraction is the most important fraction for the improvement of durability (aging) and is available in commercial products and in selected asphalts for blending.

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EVALUATION OF PROPERTIES OF AC-20 ASPHALT CEMENTS

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In 1971, the Pennsylvania Department of Transportation adopted viscosity graded asphalt cement specifications similar to AASHTO M 226-70. AC-20 asphalt cement, most commonly used for paving in Pennsylvania, was supplied by the various refineries who used different blends of crudes and processing methods to meet the Pennsylvania DOT specifications. This study was undertaken to catalog and evaluate the physical properties of AC-20 asphalt cements as supplied in 1973 by the various refineries. The fundamental and empirical consistency data have been evaluated and compared so that a specification based entirely on fundamental units of measurement can be developed in future. The test data have been made available to other researchers and agencies that are in the process of adopting viscosity graded asphalt cements. No attempt has been made to relate the physical properties of the asphalts to mix characteristics or pavement performance. Very good correlations were obtained between (a) log viscosity and log penetration at 77 F (25 C), (b) log viscosity and log penetration at 60 F (15.6 C), (c) viscosity at 77 and 60 F (25 and 15.6 C), (d) temperature susceptibility in 77 to 140 F (25 to 60 C) and 140 to 275 F (60 to 135 C) ranges, and (e) viscosity ratio at 77 and 60 F. Regardless of crude source, the satisfactory correlation obtained between shear susceptibility at 77 F and ductility at 60 F on thin film oven residues indicates that, in the specifications, shear susceptibility can be used in place of the ductility requirement.

• VISCOSITY REQUIREMENTS were incorporated in the Pennsylvania Department of Transportation specifications for paving grade asphalt cements in 1966. Subsequently, AASHTO adopted Standard Specification M 226-70 for viscosity graded asphalt cements in 1970. In 1971, the Pennsylvania DOT adopted that specification except for its requirements on ductility. AC-20 asphalt cement was supplied by the various refineries who employed different blends of crudes and processing methods to meet the Pennsylvania DOT specifications.

The objectives of this study were (a) to catalog and evaluate the physical properties of AC-20 asphalt cements produced to meet the 1971 specification using viscosity grading at 140 F (60 C), (b) to evaluate and compare the fundamental and empirical consistency data so that a specification based entirely on fundamental units of measurement can be developed, and (c) to make the test data on AC-20 asphalt cement available to other researchers and agencies that are in the process of adopting viscosity graded asphalt cements.

Similar studies (1, 2) were conducted by the Bureau of Public Roads and the Asphalt Institute based on study specifications developed by the Asphalt Institute. This study was primarily concerned with the asphalts produced to meet AASHTO M 226-70 and represents most sources catering to the mid-Atlantic states. No attempt was made to relate the physical properties of the asphalts to mix characteristics or pavement performance.

ASPHALT CEMENTS AND TEST METHODS

Asphalt Cements

The 20 AC-20 asphalt cements included in this study were supplied by seven manufacturers during 1973 and represent their regular production during that year. Details of crude sources and methods of refining (Table 1) were obtained from the producers. Thus a wide range of properties (for example, 40 to 112 penetration) was involved in evaluating AC-20 asphalt cements. The asphalt cements were supplied to meet the 1971 Pennsylvania DOT specifications given in Table 2.

Test Methods

Standard AASHTO test procedures were used to obtain the data except for the absolute viscosities at 60 and 77 F (15.6 and 25 C) over a range of shear rates, which were determined by using the sliding plate microviscometer (both glass and stainless steel plates were used for these tests).

DESCRIPTION AND DISCUSSION OF TEST DATA

Data on physical properties of the original AC-20 asphalt cements are given in Tables 3 and 4. Properties of thin film oven (TFO) residue are given in Tables 4 and 5. Tables 6 and 7 give the correlations between various properties of these asphalts. Some of the correlations are shown graphically in Figures 1 through 4.

Flash Point and Softening Point

The flash point ranged from 510 to 640 F (265 to 338 C), average being 597 F (314 C). The softening point ranged from 125 to 138 F (52 to 59 C), averaging 132 F (56 C).

Penetration at 60 and 77 F

Penetration at 60 F (15.6 C) ranged from 11 to 43, and penetration at 77 F (25 C) ranged from 40 to 112 for all AC-20 asphalt cements (Table 3). Thus viscosity grading at 140 F (60 C) results in a wider range in consistency at temperatures below 77 F than was found for asphalts controlled by penetration at 77 F.

Viscosity at 60 and 77 F

Viscosity at 60 F (15.6 C) ranged from 7.29 to 80.72 megapoises (730 to 810 kPa·s) and viscosity at 77 F (25 C) ranged from 0.92 to 5.11 megapoises (92 to 510 kPa·s) at shear rates of 0.05 sec^{-1} (Table 4). This is understandable because of the differences in temperature susceptibility of AC-20 asphalt cements.

The sliding plate microviscometer, which used controlled rates of shear, provided a satisfactory method for determining viscosities at low temperatures. Viscosities at 60 and 77 F (15.6 and 25 C) have been determined for shear rates of 0.05 and 0.001 sec^{-1} and also at a constant shear stress of 16,300 $dynes/cm^2$ (1630 Pa). This shear stress value has no particular significance except that so far it has conveniently avoided any extrapolation of the test data at these temperatures. This constant shear stress system was reported by Chipperfield and Welch (3). Test data on original asphalts and their TFO residues are given in Table 4.

Viscosity at 77 F (25 C) showed very good correlation with viscosity at 60 F (15.6 C)

Table 1. Crude sources and methods of refining.

Sample	Crude Source	Method of Refining
R-1 through R-5	80 percent Canadian, 20 percent domestic	Propane deasphalting
R-6, 7, 9, 10, 16, 17, and 19	Blend of Argentinian and Middle East	Atmospheric and vacuum distillation
R-8	90 percent Arabian light, 10 percent Nepco	Straight vacuum steam distillation
R-11 and 14	80 percent Wyoming, 20 percent Arabian light	Straight vacuum steam distillation
R-12	Blend of Venezuelan and Arabian	Atmospheric and vacuum steam distillation
R-13	Venezuelan	Vacuum distillation
R-15	Mississippi crude	Atmospheric steam distillation
R-18	45 percent Wyoming, 40 percent Bow Canadian, 15 percent Arabian light	Straight vacuum steam distillation
R-20	Venezuelan	Atmospheric and vacuum steam distillation

Table 2. Pennsylvania DOT specifications for asphalt cements.

Property	Minimum	Maximum
Flash point, deg F	450	—
Absolute viscosity at 140 F, poises	1,600	2,400
Penetration at 77 F, 100 g, 5 sec	40	120
Kinematic viscosity at 275 F, stokes	21,000	—
Ductility at 60 F, 5 cm per min, cm	75	—
Solubility in trichloroethylene, percent	99.0	—
Thin film oven test at 325 F, 50 ml, 5 hours		
Ductility of residue at 60 F, 5 cm per min, cm	10	—
Absolute viscosity at 140 F, poises		8,000

Note: 1 F = 1.8 C + 32; 1 poise = 0.1 Pa·s; 1 stoke = 0.0001 m^2/s.

Table 3. Properties of original AC-20 asphalt cements.

Sample	Specific Gravity at 77 F	Flash Point (F)	Softening Point (F)	Penetration 77 F	Penetration 60 F	Absolute Viscosity at 140 F (poises)	Kinematic Viscosity at 275 F (stokes)	Ductility[a] at 60 F (cm)	Shear Susceptibility 77 F	Shear Susceptibility 60 F
R-1	1.024	570	136	40	11	1,726	2.80	150+	0.08	0.19
R-2	1.029	570	136	40	11	1,641	2.67	100+	0.10	0.24
R-3	1.025	575	130	46	12	1,673	2.82	100+	0.12	0.18
R-4	1.017	580	128	50	17	2,042	3.29	26	0.16	0.42
R-5	1.028	565	135	51	20	1,718	2.87	15	0.21	0.56
R-6	1.024	635	130	60	19	2,145	4.03	150+	0.03	0.24
R-7	1.024	640	132	64	19	2,214	4.26	150+	0.06	0.16
R-8	1.021	610	130	67	23	2,149	4.50	150+	0.04	0.23
R-9	1.024	635	135	68	22	1,978	4.13	150+	0.03	0.12
R-10	1.024	620	133	69	23	1,961	4.10	150+	0.04	0.13
R-11	1.018	605	125	71	24	2,128	4.24	150+	0.08	0.31
R-12	1.031	595	125	71	21	1,805	3.94	150+	0.08	0.22
R-13	1.030	600	126	71	20	1,907	4.54	150+	0.03	0.10
R-14	1.022	605	133	71	23	1,654	3.75	150+	0.02	0.12
R-15	1.025	595	133	75	24	1,890	4.30	150+	0.08	0.09
R-16	1.027	600	138	75	24	1,863	3.87	150+	0.03	0.11
R-17	1.024	615	136	79	25	1,731	3.68	150+	0.03	0.13
R-18	1.018	595	136	79	26	1,754	3.83	150+	0.03	0.18
R-19	1.024	625	136	88	27	1,698	3.97	150+	0.02	0.18
R-20	1.036	510	130	112	43	2,030	4.96	100+	0.14	0.25

Note: 1 F = 1.8 C + 32; 1 poise = 0.1 Pa·s; 1 stoke = 0.0001 m^2/s.

[a]Ductility at 77 F was 150+ cm for all samples.

Table 4. Viscosities at 77 and 60 F (in megapoises) of AC-20 asphalt cements.

	Original Viscosity at 77 F			Original Viscosity at 60 F			Viscosity at 77 F, TFO			Viscosity at 60 F, TFO		
Sample	0.05 Sec^{-1}	0.001 Sec^{-1}	Constant Stress	0.05 Sec^{-1}	0.001 Sec^{-1}	Constant Stress	0.05 Sec^{-1}	0.001 Sec^{-1}	Constant Stress	0.05 Sec^{-1}	0.001 Sec^{-1}	Constant Stress
R-1	5.11	6.98	5.30	72.48	153.31	150.01	10.59	17.82	13.0	119.0	630.0	190.0
R-2	4.93	7.28	5.10	80.72	206.28	219.47	11.57	14.85	12.8	110.0	394.0	870.0
R-3	4.07	6.45	4.20	64.42	131.23	124.26	11.54	15.97	12.8	129.0	413.0	600.0
R-4	4.78	9.07	5.10	35.67	184.57	196.00	11.82	35.54	19.2	59.6	821.6	1,200.0
R-5	4.32	9.98	4.60	28.90	256.15	440.49	11.21	49.94	24.0	48.0	573.0	3,600.0
R-6	2.48	2.80	2.50	29.48	75.56	59.00	6.59	8.86	6.8	58.0	283.3	260.0
R-7	2.30	2.90	2.20	28.22	52.52	42.00	5.93	8.16	6.3	52.0	243.0	300.0
R-8	2.00	2.90	1.95	19.95	49.54	34.00	6.34	10.63	7.0	42.0	200.0	230.0
R-9	2.10	2.40	2.10	27.97	44.50	37.00	5.03	7.19	5.3	40.4	177.1	190.0
R-10	2.05	2.85	2.00	22.20	36.90	29.42	6.06	8.88	6.5	48.9	222.2	263.4
R-11	2.20	2.95	2.10	20.19	68.05	53.00	6.25	12.39	7.0	40.2	250.6	356.1
R-12	1.80	2.97	1.69	19.52	46.57	32.50	4.64	6.42	4.8	47.7	142.5	132.0
R-13	1.50	1.70	1.48	18.46	27.48	22.60	3.60	3.97	3.6	37.8	98.4	83.6
R-14	1.50	1.65	1.50	19.39	30.82	24.80	3.38	3.75	3.4	31.4	68.2	55.0
R-15	1.20	1.60	1.13	16.54	23.94	19.90	4.90	6.49	5.0	38.0	111.7	98.0
R-16	1.55	1.78	1.50	17.06	26.18	21.00	4.51	5.83	4.9	37.6	136.4	1,220.0
R-17	1.42	1.65	1.40	15.88	26.82	20.20	4.83	6.16	5.0	37.9	169.6	174.6
R-18	1.43	1.61	1.39	14.52	29.88	20.50	4.66	7.00	4.8	34.3	150.9	140.0
R-19	1.50	1.65	1.40	15.16	31.13	21.50	4.87	7.25	5.2	35.1	126.7	110.2
R-20	0.92	1.60	1.10	7.29	19.48	9.42	3.96	9.35	4.2	23.6	105.5	80.1

Note: 1 F = 1.8 C + 32; 1 megapoise = 0.1 MPa·s.

Table 5. Properties of thin film oven residue of AC-20 asphalt cements.

Sample	Percentage of Loss	Absolute Viscosity at 140 F (poises)	Viscosity Ratio 140 F	77 F	60 F	Temperature Susceptibility at 77 to 140 F	Ductility at 60 F (cm)	Shear Susceptibility 77 F	60 F
R-1	0.040	3,015	1.75	2.07	1.64	-6.33	20	0.13	0.45
R-2	0.038	3,593	2.19	2.35	1.36	-6.19	11	0.06	0.33
R-3	0.043	3,312	1.98	2.84	2.00	-6.28	12	0.08	0.30
R-4	0.016	5,508	2.70	2.47	1.67	-6.29	7	0.28	0.67
R-5	0.006	4,359	2.54	2.59	1.66	-5.96	7	0.38	0.63
R-6	0.184[a]	3,741	1.74	2.66	1.97	-5.83	23	0.08	0.34
R-7	0.077	4,862	2.27	2.58	1.84	-5.48	41	0.08	0.40
R-8	0.156	4,462	2.08	3.17	2.11	-5.61	29	0.13	0.40
R-9	0.028	3,485	1.76	2.40	1.44	-5.75	54	0.09	0.38
R-10	0.056	3,864	1.97	2.96	2.20	-5.74	29	0.10	0.39
R-11	0.046[a]	4,157	1.95	2.84	1.99	-5.68	26	0.15	0.47
R-12	0.016	4,145	2.30	2.58	2.44	-5.51	43	0.08	0.28
R-13	0.039	3,770	1.98	2.40	2.05	-5.46	150+	0.03	0.24
R-14	0.141	2,924	1.77	2.25	1.62	-5.71	79	0.03	0.20
R-15	0.053	3,016	1.60	4.08	2.30	-5.89	150+	0.07	0.28
R-16	0.106	3,362	1.80	2.91	2.20	-5.72	117	0.07	0.33
R-17	0.015	2,760	1.59	3.40	2.39	-5.98	150+	0.06	0.38
R-18	0.019[a]	2,935	1.67	3.26	2.36	-5.89	108	0.10	0.38
R-19	0.066	4,068	2.40	3.25	2.32	-5.56	132	0.10	0.33
R-20	0.115	7,456	3.67	4.30	3.24	-4.80	22	0.22	0.38

Note: 1 F = 1.8 C + 32; 1 poise = 0.1 Pa·s.

[a]Gain in weight.

Table 6. Correlation coefficients of original properties.

X	Y	Correlation Coefficient
Log viscosity at 77 F (0.05 sec^{-1})	Log penetration at 77 F	-0.948
Log viscosity at 77 F (0.001 sec^{-1})	Log penetration at 77 F	-0.875
Log viscosity at 77 F (constant stress)	Log penetration at 77 F	-0.937
Log viscosity at 60 F (0.05 sec^{-1})	Log penetration at 60 F	-0.977
Log viscosity at 60 F (0.001 sec^{-1})	Log penetration at 60 F	-0.769
Log viscosity at 60 F (constant stress)	Log penetration at 60 F	-0.727
Viscosity at 77 F (0.05 sec^{-1})	Viscosity at 60 F (0.05 sec^{-1})	+0.868
Viscosity at 140 F	Kinematic viscosity at 275 F	+0.640
Viscosity at 77 F (0.05 sec^{-1})	Shear susceptibility at 77 F	+0.597
Shear susceptibility at 77 F	Shear susceptibility at 60 F	+0.809
Temperature susceptibility (77 to 140 F)	Temperature susceptibility (140 to 275 F)	+0.944

Note: 1 F = 1.8 C + 32.

Table 7. Correlation coefficients of thin film oven residue properties.

X	Y	Correlation Coefficient
Viscosity ratio at 77 F (0.05 sec^{-1})	Viscosity ratio at 140 F	+0.294
Viscosity ratio at 77 F (0.001 sec^{-1})	Viscosity ratio at 140 F	+0.170
Viscosity ratio at 77 F (constant stress)	Viscosity ratio at 140 F	+0.300
Viscosity ratio at 140 F	Percentage of loss in weight	-0.280
Viscosity ratio at 77 F	Percentage of loss in weight	-0.172
Viscosity ratio at 77 F (0.05 sec^{-1})	Viscosity ratio at 60 F (0.05 sec^{-1})	+0.829
Viscosity at 77 F (0.05 sec^{-1})	Shear susceptibility at 77 F (TFO)	+0.498
Shear susceptibility at 77 F	Shear susceptibility at 60 F	+0.872
Shear susceptibility at 60 F	Log ductility at 60 F	-0.635
Shear susceptibility at 77 F	Log ductility at 60 F	-0.653
Temperature susceptibility (77 to 140 F, original)	Temperature susceptibility (77 to 140 F, TFO residue)	+0.798

Note: 1 F = 1.8 C + 32.

Figure 1. Viscosity at 77 F versus penetration at 77 F for original asphalts.

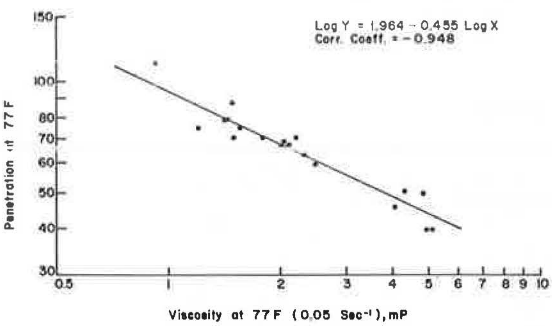


Figure 2. Viscosity at 60 F versus penetration at 60 F for original asphalts.

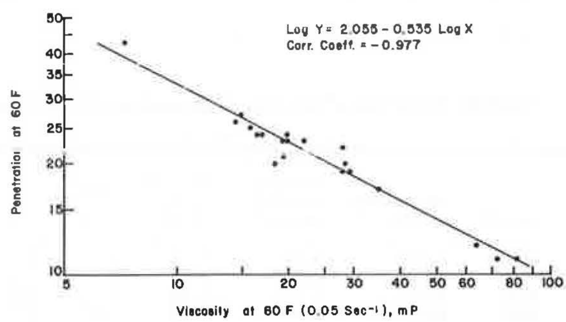


Figure 3. Shear susceptibility at 77 F versus ductility at 60 F for TFO residue.

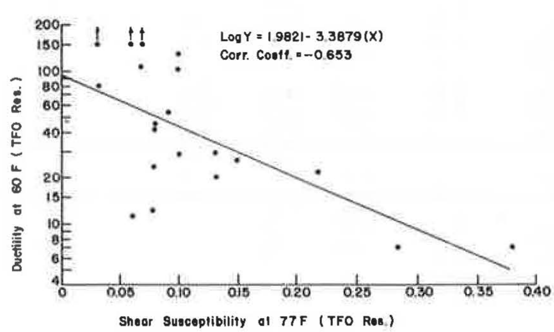
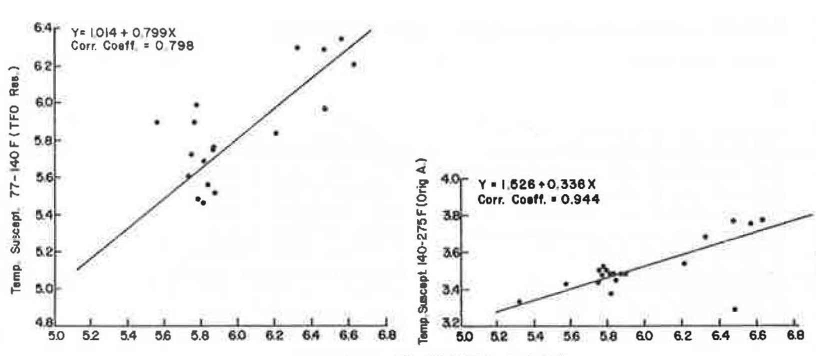


Figure 4. Temperature susceptibility of original asphalts and TFO residue.



(correlation coefficient of +0.868). Its correlation with viscosity at 140 F (60 C) is extremely poor, which indicates the necessity of consistency control at 77 F or lower temperatures for the asphalt cements graded by viscosity at 140 F. The correlation between viscosity and shear susceptibility at 77 F is fair (correlation coefficient of +0.597). This indicates that higher shear susceptibility is likely to be associated with higher viscosity at 77 F.

Viscosity at 140 and 275 F

Absolute viscosity at 140 F (60 C) ranged from 1,641 to 2,214 poises (164 to 221 Pa·s) averaging 1,885 poises (188 Pa·s); kinematic viscosity at 275 F (135 C) ranged from 26,700 to 49,600 stokes (2.7 to 4.9 m^2/s), averaging 38,300 stokes (3.8 m^2/s). Because of different temperature susceptibilities in the range of 140 to 275 F, the correlation between viscosities at 140 and 275 F was not very good (correlation coefficient of +0.540), which indicates that consistency control at 275 F is necessary.

Effects of Heating

Properties of the TFO residue of all these asphalts were determined (Tables 4 and 5), and correlations are given in Table 7. The viscosity ratio at 140 F (60 C), which appears in the specification, has extremely poor relation with the viscosity ratio at 77 F (25 C) (determined at two shear rates and at constant stress) after the thin film oven test. In some experimental test pavements, the aging indexes (viscosity ratio) based on viscosity at 77 F (25 C) were found to provide a more meaningful indication of pavement performance than aging indexes based on viscosity at 140 F (60 C) (4). In this study, the viscosity ratio ranged from 1.36 to 3.24 (average 2.04) at 60 F (15.6 C), from 2.07 to 4.30 (average 2.87) at 77 F, and from 1.59 to 3.67 (average 2.09) at 140 F. There was good correlation between viscosity ratios at 77 and 60 F.

Percentage of loss on heating in the thin film oven test has no relation with viscosity ratios at 77 and 140 F (Table 7). Thus, a higher percentage of loss on heating is not necessarily associated with a higher viscosity ratio.

Temperature Susceptibility

A double logarithm of viscosity in poises versus the logarithm of the absolute temperature expressed in degrees (empirical Walther's equation) was used to define the temperature susceptibility of the asphalt cements. The numerical values for slopes of these lines were indicated as the temperature susceptibility of the asphalt cements within the specified temperature ranges. At temperatures below 140 F (60 C), slopes of the lines tend to deviate from slopes as established at 140 F (60 C) and higher temperatures, because shear-dependent viscosities are encountered at lower temperatures. Therefore, temperature susceptibility data on original asphalts and TFO residues were determined separately for temperature ranges of 77 to 140 F (25 to 60 C) and 140 to 275 F (60 to 135 C). For original AC-20 asphalt cements, temperature susceptibility between 77 and 140 F ranged from -5.31 to -6.63 (-5.97 average). For the range of 140 to 275 F, the temperature susceptibility ranged from -3.29 to -3.77; -3.51 was average. Generally, temperature susceptibility in the range 77 to 140 F decreased after exposure to heat, averaging only -5.78. Temperature susceptibility before and after the TFO test correlated very well (Figure 4).

Shear Susceptibility

The shear susceptibility (or shear index) value used in this study is the tangent of the angle of log shear rate versus log viscosity as determined during performance of the viscosity test from the microviscometer. Shear susceptibility of asphalts has been related to pavement performance and thus is considered a specification requirement (5).

The observed range of shear susceptibility values was 0.09 to 0.56 at 60 F (15.6 C) and 0.02 to 0.21 at 77 F (25 C) for original AC-20 asphalt cements, averaging 0.21 and 0.11 respectively. For original asphalts, shear susceptibility at 77 F has good correlation with the shear susceptibility at 60 F (correlation coefficient of +0.809). However, in the case of TFO residues, there is very good correlation between the values at 77 and 60 F (25 and 15.6 C) (correlation coefficient of +0.872). Thus the specifications can have a requirement of shear susceptibility on TFO residue at 77 F rather than 60 F.

Normally for asphalts of high shear susceptibility, apparent viscosities at 77 F tend to be higher. This general trend has been confirmed in this study also (Table 6).

Relation Between Viscosity and Present Empirical Tests

The fundamental properties of AC-20 asphalt cements were compared to the characteristics measured by conventional empirical methods by examining the relationships of viscosity to penetration and ductility.

Penetration

The relation of penetration values to viscosity at 77 and 60 F (25 and 15.6 C) is shown in Figures 1 and 2. The figures show excellent correlations at both test temperatures. The equations derived by least squares for the relation between log viscosity and log penetration at each temperature are (in megapoises)

1. At 60 F, log penetration = 2.055 - 0.535 log viscosity and
2. At 77 F, log penetration = 1.964 - 0.455 log viscosity.

Thus the viscosity at 77 F (25 C) can be substituted for penetration at 77 F in the AC-20 asphalt cement specifications, and control checks on project samples can be exercised by conducting relatively simpler penetration tests.

Ductility

The value of ductility requirements in specifications has been the subject of debate. Some asphalt technologists believe that ductility, under the present standard method, is of little value as an indicator of asphalt quality. Others believe that asphalt ductility gives an asphalt pavement its quality of flexibility. Regardless of the merits of the various arguments, a number of studies (6, 7) have related ductility to pavement performance.

Ductility values at 60 and 77 F (15.6 and 25 C) for original asphalts and TFO residues of these asphalts are given in Tables 3 and 5. No correlation was found between ductility and viscosity at 60 F for asphalts from different sources. However, a relatively good correlation between ductility and shear susceptibility at 60 F was obtained on TFO residues.

Present specifications for AC-20 asphalt cement in Pennsylvania use ductility requirements on TFO residue at 60 F (15.6 C) in place of 77 F (25 C) as provided in the AASHTO Specification M 226-70. This has been done because, at temperatures as low as 60 F, ductility values are lower, better defined, and more reproducible than the values are at higher temperatures, which are determined on long, thin threads of asphalt. If the viscosity is determined at 77 F (25 C) as a substitution for penetration at 77 F, the value of shear susceptibility at 77 F is simultaneously obtained. The correlation between ductility at 60 F (15.6 C) and shear susceptibility at 77 F on TFO residue was therefore attempted and found to be satisfactory, considering poor repeatability of the ductility test especially for the values higher than 30 cm. The relationship is shown by the least square line,

$$\text{Log ductility (60 F)} = 2.527 - 2.481 \text{ (shear susceptibility at 77 F)}$$

SUMMARY AND CONCLUSIONS

The paper presented and evaluated the physical properties of AC-20 asphalt cements manufactured from various crude sources and supplied to the Pennsylvania Department of Transportation in 1973 in accordance with the viscosity graded asphalt cement specifications, which are similar to AASHO M 226-70 except for the ductility requirements. The principal findings for the 20 asphalts included in this study are summarized as follows:

1. Viscosity grading at 140 F (60 C) results in a wider range in consistency (penetration or viscosity) at temperatures below 77 F (25 C) than was found for asphalt controlled by penetration at 77 F (25 C). A specification requirement to control the low-temperature properties of AC-20 asphalt cements is apparently needed.
2. Viscosity at 77 F (25 C), determined by the sliding plate microviscometer, correlates well with viscosity at 60 F (15.6 C), both determined at a shear rate of 0.05 sec^{-1}.
3. A good correlation was obtained between log viscosity at a shear rate of 0.05 sec^{-1} and penetration at 60 and 77 F. Thus the viscosity at 77 F (25 C) can be substituted for penetration at 77 F (25 C) in the AC-20 asphalt cement specification, and project samples can be controlled by conducting relatively simpler penetration tests.
4. No correlation was obtained between viscosity ratio at 140 F (60 C) and that at 77 F (25 C) on TFO residues. A higher percentage of loss on heating is not necessarily associated with a higher viscosity ratio.
5. In general, AC-20 asphalt cements had lower temperature susceptibilities after they were exposed to heat.
6. Regardless of the crude source, satisfactory correlation was obtained between shear susceptibility at 77 F (25 C) and ductility at 60 F (15.6 C) on the TFO residues, which indicates that shear susceptibility can be used in place of a ductility test. If the viscosity at 77 F is determined in lieu of penetration, the shear susceptibility value is simultaneously obtained.

The data presented in this report on viscosity graded AC-20 asphalt cements can be applied to the development of optimum specifications based entirely on fundamental properties. More knowledge is needed to adequately correlate pavement performance with asphalt properties measured by the present empirical tests and those related to the fundamental characteristics. Attempts should be made to do so through closely designed experimental projects by using asphalts described on the basis of fundamental properties.

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