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Interstate Route 80 in California links the San Francisco–Oakland Bay and Carquinez bridges

IN THIS ISSUE

U.S. DEPARTMENT OF COMMERCE

LUTHER H. HODGES, Secretary

BUREAU OF PUBLIC ROADS

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Changes in Asphalt Viscosities During Thin-Film Oven and Microfilm Durability Tests

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by WOODROW J. HALSTEAD, Head, Bituminous Materials and Chemical Section, and JOSEPH A. ZENEWITZ, Research Chemist

This article presents the results of a special study to compare the changes in viscosity and other properties of asphalt occurring during two accelerated aging tests. Data presented include results of the thin-film oven test and the microfilm durability test on a selected group of 11 asphalts of 85–100 penetration grade. These asphalts were refined by a variety of methods from crude petroleum obtained from various sources. Also included is some background information on the development of the thin-film and microfilm tests.

Viscosity measurements for the 85–100 asphalts and their test residues made at several temperatures showed significant differences in temperature susceptibility and degree of complex flow. Two types of behavior were noted: (a) *The hardening in the microfilm test was greater than the hardening in the thin-film test for the majority of the asphalts. This difference was not considered significant for values of aging index of approximately 2 or lower but the difference increased rapidly as the hardening increased.* (b) *The hardening occurring during the two tests was essentially equal at all levels of resistance for the other asphalts.*

This study also illustrated the value of the sliding plate microviscometer for obtaining detailed rheological information on asphalt in research investigations but revealed limitations of the instrument that are a deterrent to its use for control purposes. The differences in asphalt behavior and the changes in relative rheological characteristics at different temperatures or values of stress emphasize the need for specification control of asphalt consistency at more than one temperature.

Further research is required to establish firmly the significance of some of the trends observed in this study. The Bureau of Public Roads has initiated further research with the same group of asphalts in an attempt to relate their properties to the behavior of the laboratory mixtures.

Introduction

AN ACCELERATED LABORATORY weathering test that would properly measure the potential durability of the asphaltic binder in a bituminous pavement has long been the goal of asphalt technologists. At present, two tests are being used that are believed to approach this goal: the thin-film oven test developed in 1940 by the Bureau of Public Roads, and the microfilm durability test developed in 1955 by the Shell Development Company.

In the development of the thin-film test, major emphasis was placed on the temperature conditions existing at the mixing plant; in the development of the microfilm test, an attempt was made to approach the conditions existing in the pavement during service. Obviously, neither test duplicates field or construction conditions exactly; consequently, there has been considerable discussion among asphalt technologists as to the relative significance of the results obtained by each test. The Bureau of Public Roads conducted a study in an attempt to pinpoint the significant differences in degree and type of hardening occurring in both types of tests. This article sets forth some of the findings from tests made during this study.

Development of the sliding plate microviscometer provided a means not only for comparing the results of the two tests on the same basis but also made it possible to obtain data on the more complex rheological properties of the original asphalts and residues. However, limitations of the microviscometer encountered during this study indicate that, while useful for research purposes, adoption of this instrument for control tests appears to be undesirable at this time.

Summary of Findings

This study was not conclusive in all its aspects and further research will be required to establish firmly the significance of some of the trends noted. Observations of principal interest in the study are summarized in the following paragraphs.

Information obtained from several experimental projects indicated a general correlation between the results of the thin-film and microfilm tests for most of the asphalts tested. However, detailed studies of a group of asphalts derived from crude petroleum obtained from various sources and of different methods of refining revealed two types of behavior.

For most of the asphalts tested, the hardening in the microfilm test generally was greater than that occurring in the thin-film test; however, the differences shown by these two tests varied according to the level of hardening. For values of aging index of approximately 2 or lower, the difference was not considered significant; but, as the hardening increased, the hardening in the microfilm test became increasingly greater than that obtained in the thin-film test.

On the other hand, some asphalts showed approximately equal hardening in the two tests at all levels of aging index. However, studies of the viscosity-temperature relationships indicated that the rheological properties of the two residues may differ significantly. The significance of these differences with respect to the behavior of the asphalt as a road binder has not been determined and additional studies are being conducted.

Data obtained with the microviscometer indicated large differences in the degree of complex flow for various asphalts and that the flow characteristics of a given asphalt may change rapidly with temperature changes. Although the relation of these changes to the

[1] Presented at the 64th annual meeting of the American Society for Testing Materials, Atlantic City, N.J., June 1961.

behavior of pavement mixtures made with asphalt has not been established as yet, it is believed that some coefficient of complex flow will provide a very useful parameter of behavior. Such a coefficient is represented by the slope of the line obtained by plotting the logarithm of the shearing stress against the logarithm of the rate of shear.

The high degree of complexity of asphalt flow in the range of temperatures most often encountered in the pavement, indicates that the control of consistency at only one temperature is inadequate for specification purposes. Tests at two or more temperatures appear to be desirable.

The change shown in degree of complex flow with temperature changes, as well as differences in viscosity-temperature susceptibility noted, indicated that the substitution of absolute viscosity at a single temperature for control by penetration values may be of more limited advantage than first believed. To obtain more information on this subject, further studies should be conducted over a wider range of temperatures than that employed in this study. For such studies, instruments other than the sliding plate microviscometer will be required.

Description of Tests

The term thin-film test first was applied to the ⅛-inch film test used to denote a comparison with the standard volatilization test. Admittedly, this is not a thin film in the sense of the asphaltic films in a pavement mixture. However, because alternative suggestions for a title are unwieldy, thin-film oven test generally is used to designate the ⅛-inch test. The test developed by the Shell Development Company is referred to either as the microfilm durability test or the aging index test. For convenience and brevity the terms thin-film test and microfilm test are used in this article.

Thin-film test.—The thin-film test is based on the changes that occur in an asphalt when a 50-milliliter sample in a ⅛-inch film is heated in an oven for 5 hours at a temperature of 325° F.

The test has been adopted as an ASTM Tentative Standard (D1764–60T) under the title, *Method of Test for Determining the Effect of Heat and Air on Asphaltic Materials When Exposed in Thin Films.* The American Association of State Highway Officials has also adopted the test and designated it, *Thin-Film Oven Test* (AASHO Method T179). The ASTM and AASHO methods are identical with the exception of the tests to be made on the residues. The ASTM method includes only the penetration of the residue as a designated test, while the AASHO method includes both penetration and ductility of the residue as designated tests. The determination of the weight loss is optional in the ASTM method, but mandatory in the AASHO method.

Microfilm test.—The microfilm durability test, sometimes designated as the aging index test, has not been adopted as an ASTM Standard. Subcommittee B–19 of Committee D–4 has published a proposed method for information, and cooperative studies to determine its precision have been conducted. This test measures the change in absolute viscosity that occurs when a film of asphalt, 5 microns in thickness, is exposed in an oven for 2 hours at 225° F. The result is expressed as the ratio of the final viscosity divided by the original viscosity of the asphalt, both measured at 77° F. The viscosities are determined in each case by the sliding plate microviscometer.

RESUME OF PRIOR RESEARCH

Thin-film tests

Because the thin-film test was first introduced in 1940, some asphalt technologists may not be completely familiar with the original research conducted to establish its usefulness. A brief resume of the major findings of this research are presented here.

The thin-film test was first introduced by Lewis and Welborn of the Bureau of Public Roads (*1*).[2] The first report was based on findings of tests on 50–60 and 85–100 penetration grade asphalts that were representative of asphalt production in the United States at that time. Results were included on tests made for 2, 5, 7, and 10 hours at film thicknesses of ⅛, ¹⁄₁₆, and ¹⁄₃₂ inch. These tests showed that the changes occurring in the ⅛-inch film at 5 hours most nearly reproduced the changes occurring in the Shattuck mixing test, a test that was developed for measuring the hardening of asphalt in a laboratory mixture. Changes other than in penetration also were shown to be generally of the same order of magnitude for the ⅛-inch, 5-hour test and the Shattuck test. Accordingly, this ⅛-inch film thickness and 5-hour exposure time were selected for further study. From this work, it was concluded that the ability of the asphalt to withstand hardening during mixing in commercial operations could be measured by the thin-film test. Further, the report suggested that this property was related to the hardening that occurred during service. The report also included data to show that the standard oven loss test is of little value for predicting the degree of hardening of asphalt that occurs during the mixing process or in service.

A second Public Roads report, made in 1946 by Lewis and Halstead (*2*), presented the results of thin-film tests on asphalts of the 60–70, 100–120, and 120–150 grades obtained from the same sources as the asphalts tested for the first report. The data of the second report generally supported the findings of the earlier report for the 50–60 and 85–100 grades, thereby indicating a general applicability of this test to all grades of paving asphalts.

A third Public Roads report, by Pauls and Welborn, published in 1952 (*3*), compared the hardening of asphalts as measured by the abrasion tests, weathering-strength tests, the Shattuck test, outdoor exposure, and the thin-film oven test. The results of all these tests, in which mixtures were used to evaluate hardening, were shown to be related very closely to the hardening that occurred in the asphalt. Because the thin-film test is a direct measure of hardening and is not complicated by extraction and recovery processes, it was considered to be the most suitable test to use for specification purposes.

The most recent Public Roads studies on asphalts representing production from all parts of the United States were reported in 1959 (*4*) and 1960 (*5*). The data from these reports indicate that the percentage of retained penetration generally tends to be lower for the higher penetration asphalts than for those with low penetration; therefore, for equal severity, different limits for each grade of asphalt should be used in establishing specification requirements for asphalt cement.

It is believed that the combined findings of the foregoing reports, as well as other investigations, support the conclusions that (1) relative resistance to change during heating and mixing is an important property of paving asphalt, (2) a test to measure this resistance is needed in specifications, and (3) the thin-film test fulfills this need.

Microfilm tests

Because the research concerning the microfilm test is more recent than that for the thin-film test, it may be more familiar to those presently engaged in asphalt research. One of the first reports on a microfilm test was presented by Griffin, Miles, and Penther in 1955 (*6*). These authors based their studies on some earlier work by Labout and van Oort (*7*).

Generally, the microfilm test is considered to be a measure of the potential durability of the asphalt in service. The factors of variability in design and construction affect asphalt behavior to a considerable extent. However, results for experimental construction projects, in which variables other than the asphalt have been eliminated as much as possible, indicate a general correlation between behavior of asphalts in microfilm tests and in road behavior.

Relation of Microfilm and Thin-film Test Results

Data are available from several published reports that provide a general comparison of the results of the microfilm and thin-film tests. Results for asphalts used in the Zaca-Wigmore tests in California are contained in two reports. One, the study by Simpson, Griffin, and Miles (*8*), includes the microfilm test results and the other, by Skog (*9*), includes the thin-film test results. Results from both tests on another group of asphalts used in Texas experimental projects are included in a report by Jimenez and Galloway (*10*).

The data taken from these reports are shown in table 1. Several laboratories, including that of the Bureau of Public Roads, cooperated with the Texas Transportation Institute in making tests on duplicate samples of the materials used in the Texas experimental roads reported on by Jimenez and Galloway.

[2] References cited by italic numbers in parentheses are listed on page 217.

Table 1.—Comparison of thin-film and microfilm test results

Asphalt identification	Original penetration, 100 g. 5 sec. at 77° F.	Thin-film test, percent retained penetration	Microfilm test, aging index
ZACA-WIGMORE PROJECT			
A	225	57	8.5
A-2	218	56	9.6
B-1	255	47	17.1
B-2	223	43	65.4
C	247	54	17.8
C-2	246	51	15.8
D	243	52	15.8
D-2	228	53	7.3
E	227	30	428.0
E-2	239	21	282.0
F	215	40	29.6
G	238	41	33.2
G-2	215	39	59.0
H	230	50	14.3
H-2	212	47	27.4
J	237	58	7.8
TEXAS PROJECT			
1	167	63	4.2
2	172	65	3.1
3	128	58	5.0
4	202	58	5.2
5	202	39	12.2
6	156	49	6.2
7	131	63	4.1
8	123	62	4.8
9	233	30	19.4
10	145	65	3.9
11	163	52	7.0

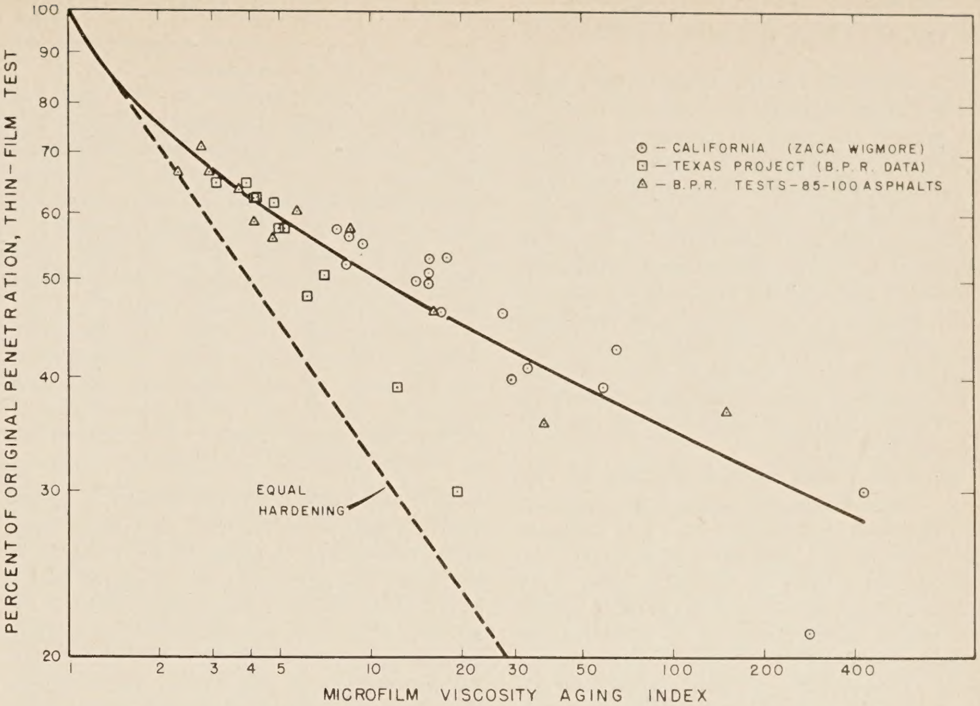

Figure 1.—Relation between results of thin-film and microfilm tests.

The data supplied by Public Roads were selected for presentation in table 1. The thin-film test results selected from the California samples (Zaca-Wigmore project) were the averages of the results obtained by several cooperating laboratories, as reported by Skog.

Figure 1 shows the percentage of retained penetration obtained in the thin-film test plotted against the viscosity aging index obtained in the microfilm test, using a logarithmic scale on both axes. (This figure also presents data for 85–100 penetration grade asphalts selected for the Public Roads special study, which will be discussed later.)

The penetration of the original asphalts from which these data were obtained varied from 85 to 250 yet, despite this relatively wide variation, a general relation exists between the two test results that approaches a straight line. The dashed line in figure 1 shows the curve that would result if the hardening in the asphalt were the same in both tests. This curve was obtained by calculating the equivalent viscosity for a given penetration using an approximate relation established by tests made in the Bureau of Public Roads laboratory. This relation agrees closely with that reported by Saal, Baas, and Heukelom (*11*). With these equivalents, the aging index for any percentage of retained penetration is easily calculated.

The deviation of the experimental curve from the line indicating equal values shows that the hardening of the asphalt occurring in the microfilm test was greater than that occurring in the thin-film test, and that the difference increased with the increase in the susceptibility of the asphalt to hardening. Because of an indication that the data for some asphalts deviated from the general relation more than could be accounted for by experimental error, Public Roads conducted a special study in an attempt to determine the cause of such deviations. This study was conducted on a group of 11 asphalts of 85–100 penetration grade refined by a variety of methods from petroleum obtained from various sources.

Table 2.—Source, method of refining, and test characteristics of 11 selected 85–100 penetration grade asphalts

Sample No.	Source of crude	Method of refining	Test characteristics: Penetration 100 g. 5 sec. at 77° F.	Test characteristics: Ductility, 5 cm./min. at 77° F.	Test characteristics: Softening point	Test characteristics: Furol viscosity at 275° F.	Test characteristics: Specific gravity at 77° F.	Thin-film oven test—⅛-inch film at 325° F., 5 hrs.: Loss [1]	Thin-film oven test, Tests on residue: Softening point	Thin-film oven test, Tests on residue: Ductility, 5 cm./min.	Thin-film oven test, Tests on residue: Percent of original penetration
				Cm.	*° F.*	*Sec.*		*Pct.*	*° F.*	*Cm.*	
2	Venezuela	Steam and vacuum refining to grade	88	225	118	137	1.018	2.04	131	250+	38
3	Mexico, Panuco	Atmospheric steam distillation to grade	87	248	121	318	1.037	.55	134	138	60
13	Venezuela	(Not given)	89	175	120	255	1.033	.18	132	175	62
38	Midcontinent	Vacuum distillation to 225-275 penetration; blown at 480° F.—to grade.	90	170	120	189	1.000	.00	129	120	62
53	Midcontinent	Vacuum distillation to 80° F.-90° F. softening point; blown to grade—10 hours.	86	151	119	195	.995	+.03	126	150	69
[2] 56	Kansas	Vacuum distillation to grade	86	166	121	180	1.004	+.03	139	13	62
[3] 69	West Texas crudes	Steam-vacuum distillation to grade	87	112	116	120	1.029	+.02	125	178	61
71	Midcontinent-mixed base	Propane de-asphalting and vacuum distillation (Propane asphalt blended with 300+ penetration).	87	200	116	190	1.010	+.08	121	190	70
100	California, Santa Maria	Steam distillation to grade	88	245	120	228	1.031	1.03	136	125	49
[4] 109	Colorado	Vacuum distillation to grade	87	195	118	100	1.019	+.10	129	63	57
119	Colorado	Propane de-asphalting to hard base; fluxed to grade with flux oil.	92	238	116	123	1.031	2.18	129	250+	38

[1] + values indicate gain in weight.
[2] Xylene-naphtha equivalent 4–8.
[3] Xylene-naphtha equivalent 12–16.
[4] Positive spot in 100 percent xylene.

SPECIAL STUDY OF 85–100 PENETRATION GRADE ASPHALTS

The materials for the special study were selected from asphalts previously included in the general studies of properties of highway asphalts for which reports have been issued in recent years (*4*, *5*). The same identification numbers are used here to permit convenient reference to the earlier work. The sources of the crude petroleums and methods of refining the asphalts are given in table 2, which also shows some of the more significant test characteristics previously reported (*4*). These asphalts were chosen so as to represent widely different types of crudes and methods of refining, degree of volatility, and asphalt composition.

The special tests on these asphalts for this study included the thin-film oven test and the microfilm durability test. The absolute viscosities of the original asphalts and the residues from each of the film tests were determined by use of the sliding plate microviscometer. Determinations to establish the viscosity-temperature characteristics were made at three different temperatures for all materials and at five temperatures for some asphalts of special interest. The results of these tests are given in table 3 and the first five columns of table 4.

Table 3.—Characteristics of 85–100 penetration grade asphalts before and after laboratory film tests

Sample identification	Original asphalt		Thin-film residue				Microfilm residue	
	Penetration at 77° F.	Viscosity [1] at 77° F. in megapoises v_o	Penetration at 77° F.	Percent original penetration	Viscosity [1] at 77° F. in megapoises v_t	Viscosity aging index v_t/v_o	Viscosity [1] at 77° F. in megapoises v_m	Viscosity aging index v_m/v_o
2	86	0.860	31	36	8.21	9.55	32.5	37.8
3	88	1.04	51	58	4.43	4.26	9.00	8.65
13	88	1.16	54	61	3.77	3.25	6.70	5.78
38	89	1.20	57	64	5.60	4.67	4.44	3.70
53	88	1.20	59	67	3.00	2.48	2.80	2.31
56	78	1.67	51	65	4.70	2.81	4.95	2.96
69	88	1.22	49	56	3.45	2.83	5.90	4.84
71	85	0.879	60	71	2.23	2.53	2.45	2.79
100	90	1.21	42	47	6.32	5.22	19.5	16.1
109	83	1.35	49	59	5.22	3.87	5.50	4.07
119	93	0.78	34	37	5.74	7.36	118	151.0

[1] Calculated at a shear rate of 0.05 sec.$^{-1}$

Test results for penetration of the original asphalts and the thin-film residues shown in table 2 and table 3 are not always in agreement because tests reported in table 3 were made subsequent to those shown in table 2. However, the agreement is considered excellent in that retained penetration differed by no more than 2 percentage points except for asphalt samples 69 and 56. For sample 69, the penetration of the residue was 4 points lower and the original penetration was higher than previously reported so that the result for retained penetration differed by 5 percentage points. For sample 56, the penetration of the thin-film residues agreed to within 2 percentage points but the original penetrations differed by 8, the last results being the lower. Because both these changes were in the same direction, the percentage of retained penetration differed by only 3 percent.

Viscosity Aging Indexes

Table 3 summarizes the viscosity and penetration data obtained from the microfilm and thin-film tests at 77° F. A viscosity aging index, which is the viscosity of the aged residue divided by the original viscosity, is shown for each test. Unless specifically stated

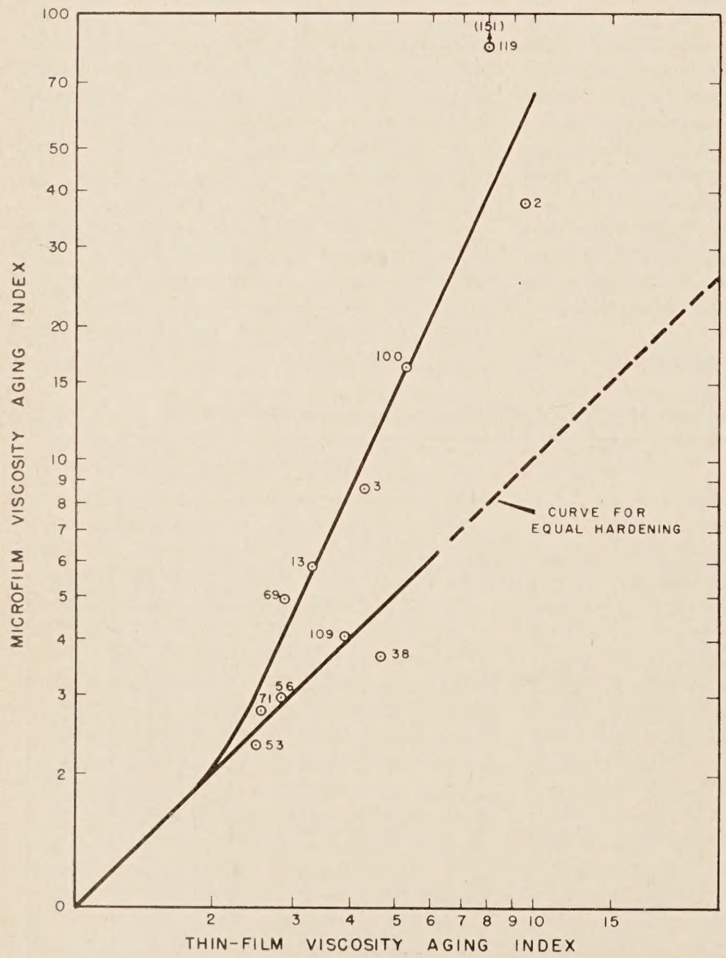

Figure 2.—Relation between viscosity aging indexes for microfilm and thin-film tests.

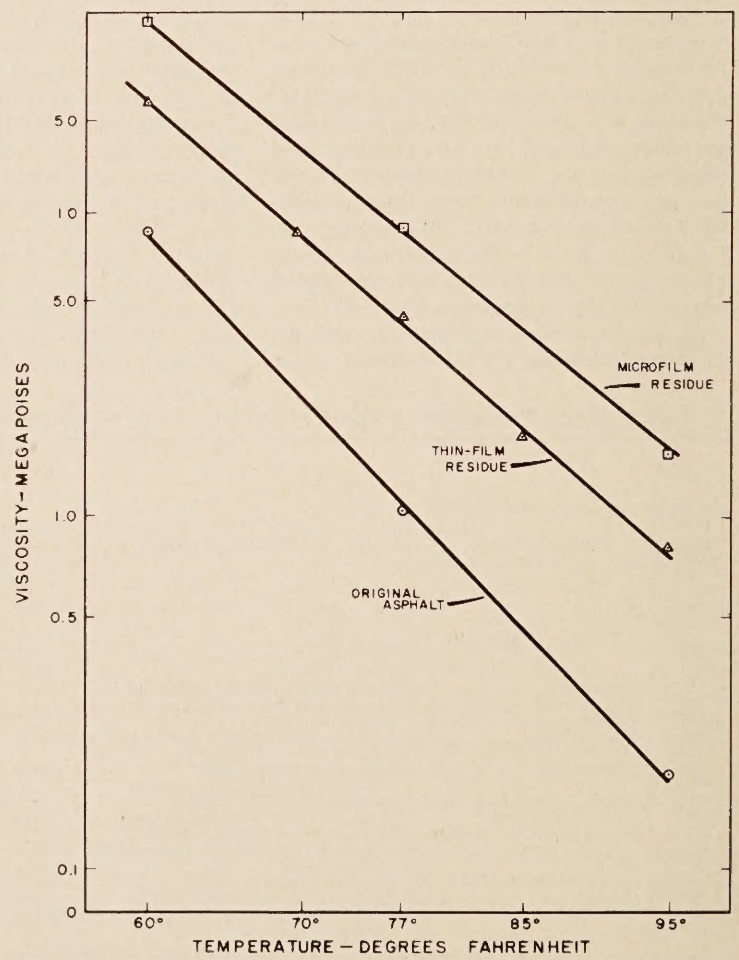

Figure 3.—Viscosity-temperature relations for asphalt sample 3.

otherwise, all viscosities recorded in this table and elsewhere in this article were based on calculations at a shear rate of 0.05 sec.$^{-1}$ Figure 2 shows the viscosity aging index obtained from the microfilm test plotted against that obtained from the thin-film test.

The data presented in figure 2 indicate that for values of aging indexes around 2 there was no significant difference in the results of the two tests. As the resistance to hardening decreased, the increase in viscosity index for the microfilm test increased much more rapidly than that for the thin-film test. However, results for two asphalts very definitely did not follow this trend; essentially the same hardening occurred for asphalt sample 109 in the two tests, but less hardening occurred in the microfilm than in the thin-film test for asphalt sample 38. Check determinations were made for the asphalts showing a different trend; the same results were obtained each time. The results on asphalt samples 53, 56, and 71 showed approximately equal hardening behavior during both tests. However, no large differences would be expected in these three asphalts because of low values of aging index. A consideration of these results, together with data to be presented later, indicates that there were at least two types of behavior of the tested asphaltic materials. In one type of behavior, the difference in hardening of asphalt between the aging indexes obtained by the microfilm and thin-film tests increased exponentially as the index increased; for the other, the hardening indexes were approximately equal.

Viscosity-Temperature Relationship

Viscosities were determined over a range of temperatures to provide information on the temperature susceptibility for the asphalts and the test residues. The data obtained are given in the first five columns of table 4. The range of temperatures used (60° F. to 95° F.) generally represents the limits of the microviscometer for this group of asphalts.

To show the viscosity-temperature relationship, the viscosities were plotted against temperature by using a basis similar to that used for the standard ASTM viscosity-temperature charts. In figures 3–7, the double logarithm (log of the log) of viscosity in poises is plotted against the logarithm of the absolute temperature in degrees Rankine (°F. + 459.7). (For convenience, temperatures are shown in °F on the figures.) The ASTM charts are based on the double logarithm of the viscosity in centistokes. However, for the range of values obtained for these asphalts and because the densities of the asphalts closely approximated 1 gram per cubic centimeter, either scale should give an approximately straight-line relationship for true viscous liquids.

The data points for 7 of the 11 asphalts tested fell in straight lines within small experimental errors. In four cases, which are discussed later, there was some curvature that apparently was not related to experimental error. The viscosity-temperature susceptibility index (m) shown in the seventh column of table 4 is the slope of these plotted lines, except for the four noted. In these cases, the calculations are based on an assumption that the portion of the curve between 70° F. and 85° F. is straight.

Figure 3 shows the viscosity-temperature relations of the original asphalt and the residues from the microfilm and thin-film tests for asphalt sample 3. The curves shown are typical for those asphalts (as mentioned previously) that showed higher aging indexes for the microfilm residues than for the thin-film residues. The temperature susceptibilities of both residues were slightly less than for the original asphalt and, in most cases, the microfilm residue showed slightly less susceptibility than the thin-film residue; but these differences generally were not considered significant. The curves for asphalt 71 in figure 4 indicate that this material generally followed the behavior pattern as shown for sample 3, although the degree of hardening was approximately the same in both the microfilm and the thin-film tests.

Those asphalts showing approximately equal hardening in the thin-film and microfilm tests at all levels are illustrated in figures 5–7 for asphalt samples 38, 56, and 109, respectively. Although the deviation was small for some of these asphalts, the data points do not fall in a straight line. Also, the curves for the microfilm and thin-film residues generally cross. Viscosities determined at 60° F. for the microfilm test residues were higher than those for the thin-film test residues; but the reverse was true at 95° F. At 77° F., the temperature used for the viscosity aging indexes, hardening values were either higher or lower depending on the asphalt.

Because of the curvature of some of the plotted data, it was necessary to base the viscosity-temperature susceptibility index given for these asphalts in table 4 on the portions of the curves between 70° F. and 85° F. Thus, the indexes are not valid over a wide range of temperature, but the values shown are indicative of the trends. The change in slope of the residues compared to that of the original asphalt was generally greater for this group of materials than for asphalts of the other behavior group. This was particularly true for samples 56 and 109.

Complex flow characteristics

It generally is recognized that most asphalts at atmospheric temperature do not have the characteristics of a true liquid (Newtonian flow) but exhibit complex flow; that is, the ratio of the shearing stress to the rate of shear is not a constant. Accordingly, further study of the rheological properties of the asphalts and the residues was made by considering the proposals of Schweyer and Bransford (*12*). These authors, following earlier suggestions by Romberg and Traxler (*13*), recommended that viscosity determinations

Table 4.—Complex flow coefficients and asphalt viscosities at various temperatures and shear rates

Sample identification[1]		Viscosity (0.05 sec.$^{-1}$) in megapoises at—					Viscosity temperature susceptibility index[2]	Viscosity, 1,000 ergs per sec. per cm.^{3} in megapoises at—			Coefficient of complex flow at— c		
		60° F.	70° F.	77° F.	85° F.	95° F.	m	60° F.	77° F.	95° F.	60° F.	77° F.	95° F.
2	o	11.1	--------	0.860	--------	0.148	4.8	10.5	0.881	0.122	1.00	1.00	0.65
	t	113	--------	8.21	--------	.960	4.7	137	10.7	1.07	.95	.84	.85
	m	--------	--------	32.5	--------	2.80	--------	--------	40.4	3.41	--------	.91	.79
3	o	8.70	--------	1.04	--------	.181	4.3	9.80	1.10	.200	.92	.92	1.20
	t	26.0	8.60	4.43	1.78	.802	3.5	41.4	5.07	.853	.80	.89	.87
	m	53.7	--------	9.00	--------	1.55	3.3	127	11.3	1.69	.76	.82	.89
13	o	10.6	2.51	1.16	.468	.226	4.1	13.3	1.26	.200	.90	.94	.95
	t	24.8	--------	3.77	--------	.573	3.8	36.8	4.08	.578	.86	.95	.96
	m	43.7	--------	6.70	--------	1.02	3.6	77.6	7.95	1.03	.88	.91	.95
38	o	10.1	2.62	1.20	.470	.132	4.5	12.1	1.28	.122	.81	.86	.88
	t	20.7	11.0	5.60	2.87	.940	[3] 3.2	38.8	6.62	.978	.70	.90	.95
	m	22.7	9.80	4.44	1.77	.513	[3] 4.1	52.2	5.26	.665	.67	.89	.89
53	o	9.80	3.30	1.20	.482	.155	[3] 4.9	11.3	1.22	.116	.91	1.05	.96
	t	16.6	6.40	3.00	1.24	.389	[3] 3.9	26.3	3.13	.327	.78	.96	.98
	m	16.4	7.20	2.80	1.08	.372	[3] 4.3	35.5	3.00	.295	.70	.91	.92
56	o	8.02	3.27	1.67	.675	.206	[3] 4.2	13.2	3.83	.204	.71	.90	.98
	t	16.2	7.42	4.70	3.04	1.32	[3] 2.5	48.1	17.4	1.46	.54	.78	.84
	m	18.4	7.40	4.95	2.88	1.11	[3] 2.6	60.5	15.4	1.24	.53	.63	.83
69	o	20.0	--------	1.22	--------	.150	5.3	20.9	1.25	.158	.99	1.00	1.02
	t	34.7	--------	3.45	--------	.352	4.7	50.1	3.41	.356	.87	1.03	.99
	m	60.0	--------	5.90	--------	.577	4.7	117	6.56	.541	.77	.91	1.33
71	o	10.5	2.50	.879	.311	.0930	5.4	13.4	.909	.0961	.88	.99	1.00
	t	18.5	5.40	2.23	.700	.222	4.9	24.4	2.34	.219	.88	.98	.96
	m	26.7	--------	2.45	--------	.300	4.7	35.0	2.59	.293	.88	.97	.95
100	o	11.2	--------	1.21	--------	.185	4.5	13.3	1.23	.190	.93	1.00	1.00
	t	51.5	--------	6.32	--------	.802	4.0	89.6	7.27	.904	.78	.94	.92
	m	87.6	--------	19.5	--------	2.53	3.2	443	24.2	3.44	.60	.86	.86
109	o	17.2	3.39	1.35	.520	.200	5.0	35.7	1.42	.200	.68	.93	.93
	t	24.9	10.3	5.22	2.77	1.57	[3] 3.0	81.6	7.12	1.65	.57	.78	.89
	m	25.2	10.4	5.50	2.24	.585	[3] 3.2	71.0	7.63	.605	.62	.68	.94
119	o	10.8	2.17	.780	.230	.072	5.7	10.3	.800	.072	1.03	.97	1.00
	t	86.0	20.5	5.74	1.67	.540	5.0	123	5.73	.530	.97	1.00	.99
	m	--------	--------	118	34.3	9.20	4.5	--------	--------	11.6	--------	--------	.88

[1] o=original asphalt; t=thin-film residues; m=microfilm residues.
[2] Slope of plotted line: log=log viscosity vs. log absolute temperature.
[3] For portion of curve between 70° F. and 85° F.

be reported on the basis of a constant value of the product of shearing stress multiplied by rate of shear, the unit of which is ergs per sec. per cm.[3] or power output per unit of volume. They also recommended that the degree of complex flow be expressed by a coefficient that is equal to the slope of the line obtained when the shearing stress in dynes per cm.[2] is plotted against the shear rate (sec.$^{-1}$) on logarithmic coordinates. This is based on the general rheological equation:

$$v = F/S^c$$

where,

v = viscosity in poises.
F = shearing stress, dynes per cm.[2]
S = rate of shear, sec.$^{-1}$
c = coefficient of complex flow.

Experimentally, the values of viscosity and the coefficient of complex flow were determined by plotting on logarithmic coordinates the shearing stress in dynes per cm.[2] for each loading of the microviscometer against the shear rate (sec.$^{-1}$) obtained at that loading. The actual slope of this line is the coefficient of complex flow. The values of F and S, at the point where this line intersects the line representing $F \times S = 1{,}000$, were used as the basis for calculating the viscosity ($v = F/S$). The results of these computations are given in the right-hand portion of table 4.

The coefficients of complex flow for the asphalts are of particular interest. A value of 1.00 indicates Newtonian flow. It will be noted that in two cases, the computed value significantly exceeded 1.00: the original values for asphalt sample 3 at 95° F. and the microfilm residue for asphalt sample 69 at 95° F. These computations were based on values obtained with only one specimen and such an unusual flow could have been an accidental result or could indicate some instrumental factor.

General Trends of Test Results

Further study is needed to determine the cause of these unusual results and also the repeatability of the values. However, it is believed that the general trends noted are significant. Generally, the degree of complex flow decreased as the temperature increased (the value of the coefficient increased, approaching unity). However, test results for asphalt sample 2 indicated the reverse trend, which might have been an experimental error. It was also noted that the degree of complex flow was usually greater for the test residues than for the original asphalts. The significant exception noted was the unusual behavior of asphalt sample 2 at 95° F., as previously stated.

The viscosity data presented in table 4 show that large differences in reported viscosity can occur because of the basis used for reporting. For example, the viscosity for the microfilm test residue for asphalt sample 100 (at 60° F.) was 87.6 megapoises when calculated on the basis of a shear rate of 0.05 sec.$^{-1}$ but was 443 megapoises when calculated on the basis of 1,000 ergs per sec. per cm.[3]

In these studies the significance of the various differences in asphalts or the causes of such differences were not determined. Public Roads has initiated further research with the same group of asphalts in an attempt to relate some of these properties to the behavior of the asphalt in a mixture. It is hoped that such studies will establish the significance of the differences reported here.

Usefulness of Microviscometer

A secondary, but important, objective of these investigations was to evaluate the usefulness of the sliding plate microviscometer for studying the changes in properties of asphalt. Suggestions have been made that the results of the thin-film test be evaluated on the basis of viscosity changes. Figure 8 (p. 218) shows the relations between the percentage of original penetration and the thin-film viscosity aging index. If a constant relation existed between penetration and viscosity, this plotted curve would be a straight line through the origin. With the possible exception of sample 38, deviations from the plotted curve generally were within experimental error. Therefore, figure 8 provides a basis for converting thin-film test results from percentage of original penetration to a viscosity aging index.

For example, a retained penetration of 47 percent in the thin-film test, the limit used for the 85–100 penetration asphalts in several specifications, is found, by using figure 8,

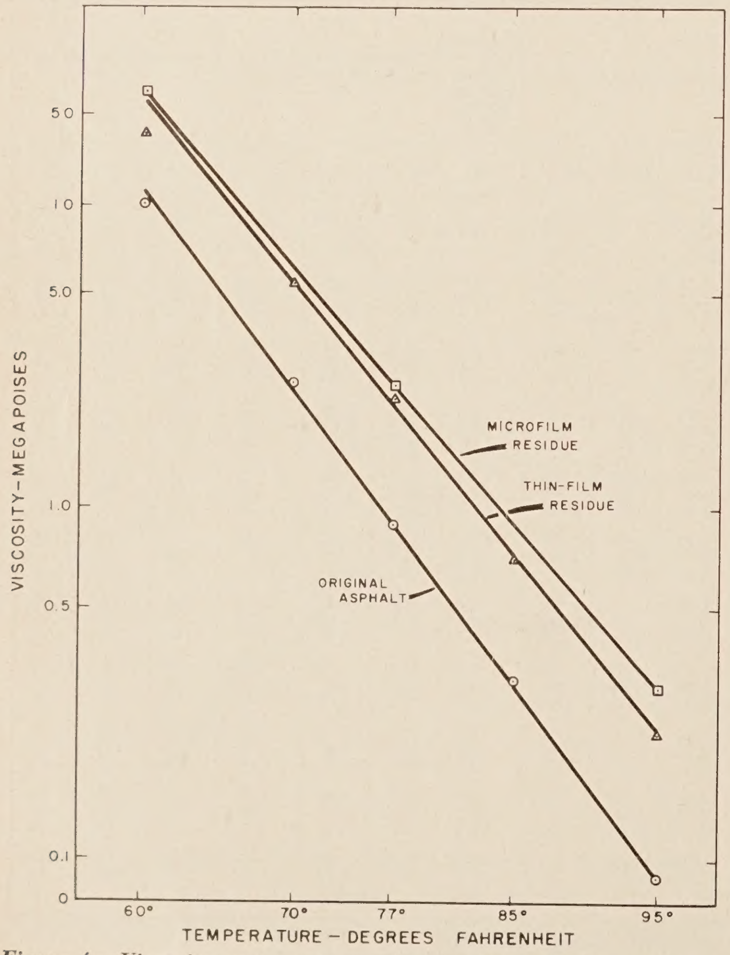

Figure 4.—Viscosity-temperature relations for asphalt sample 71.

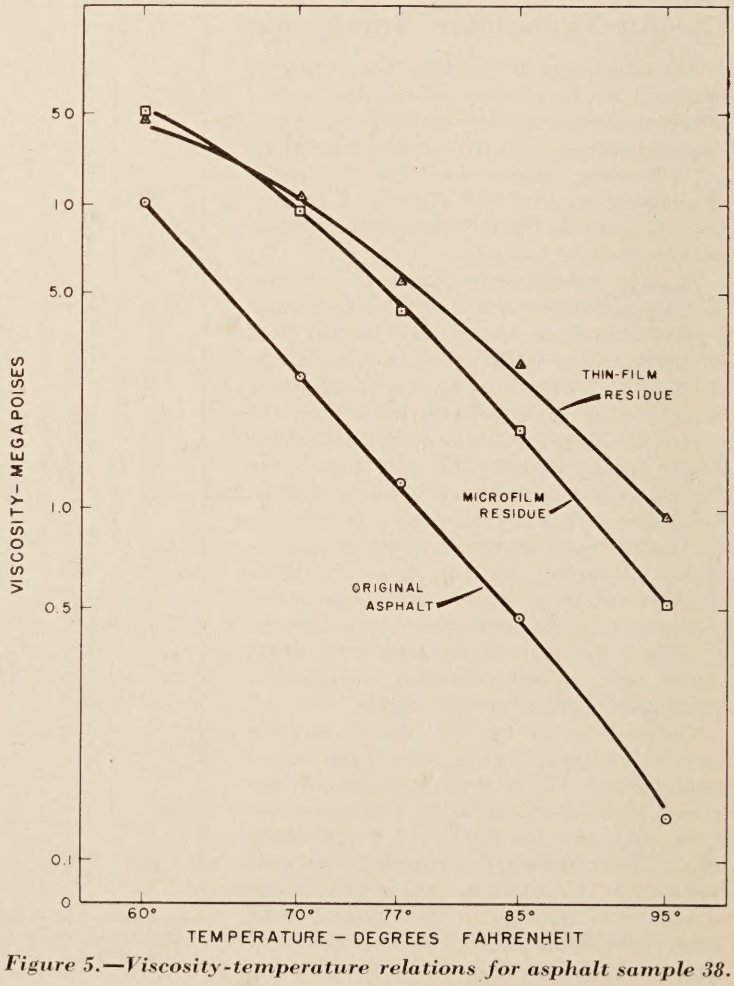

Figure 5.—Viscosity-temperature relations for asphalt sample 38.

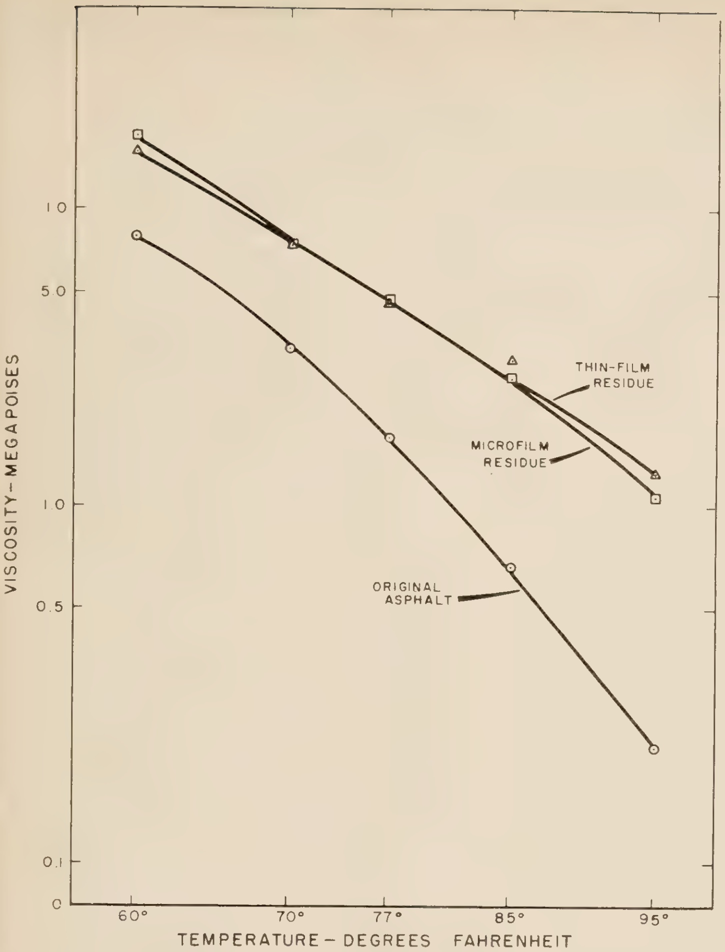

Figure 6.—Viscosity-temperature relations for asphalt sample 56.

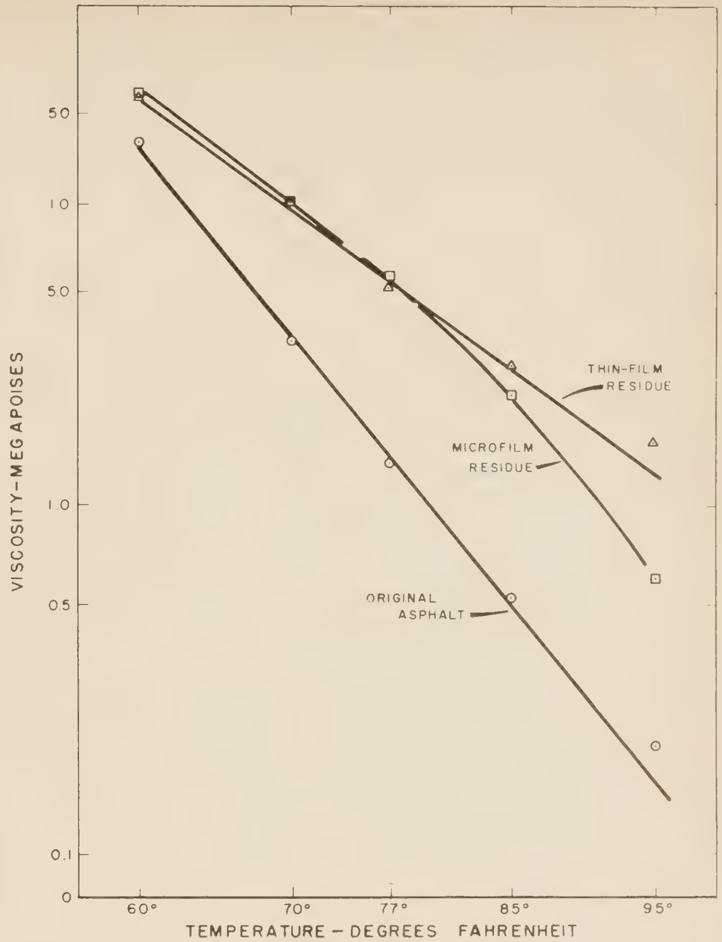

Figure 7.—Viscosity-temperature relations for asphalt sample 109.

to be equivalent to a thin-film viscosity aging index of 4.5. This should not be confused with the equivalent microfilm aging index, which figure 1 shows to be approximately 15.

These tests showed that evaluation of the results of the thin-film test could be made on the basis of absolute viscosities obtained by the sliding plate microviscometer, without materially changing the acceptance or rejection of material. The high degree of technique required for obtaining suitable microviscometry results, plus the fact that only a few laboratories have the equipment, make the recommendation of general adoption of such a procedure undesirable at the present time.

Reports concerning the use of the microviscometer generally tend to emphasize the simplicity of the instrument. Experience with the microviscometer in this investigation indicated that this simplicity has been overemphasized. It was found that an operator needs considerable experience to develop the technique of preparing films so as to obtain repeatable results. The recording instrument requires careful calibration, which must be checked frequently. Determinations for at least four, and preferably five, different loadings must be made at each temperature. Then, computations must be made for each loading; the values must be plotted; and the viscosity must be interpolated for the selected shear rate. For example, each viscosity determination reported in this article required 21 individual computations.

The useful range of the commercial instrument used in these studies was found to be from 0.1 to 100 megapoises for a shear rate of 0.05 $sec.^{-1}$ For values of less than 0.1 megapoise, the instrumental balance became critical and greatly affected the results. For values of about 100 megapoises, the maximum load permitted by the design of the instrument was required. This range of 0.1 to 100 megapoises corresponds approximately to a range of 10 to 300 penetration.

A change in the design of the instrument, such as a horizontal position for the plates or the capacity for greater loading, would permit extension of the range. The upper range might also be extended somewhat if the basis used for reporting viscosity values was changed to viscosities at a constant power output or if lower shear rates were used.

These limitations are pointed out to indicate that adoption of the instrument for control tests appears undesirable at the present time. However, for research purposes it offers many advantages. A considerable amount of information concerning the flow properties of an asphalt can be obtained with this instrument that is not available from empirical tests. It also makes possible more detailed comparisons of changes in asphalt properties induced by laboratory tests and service behavior.

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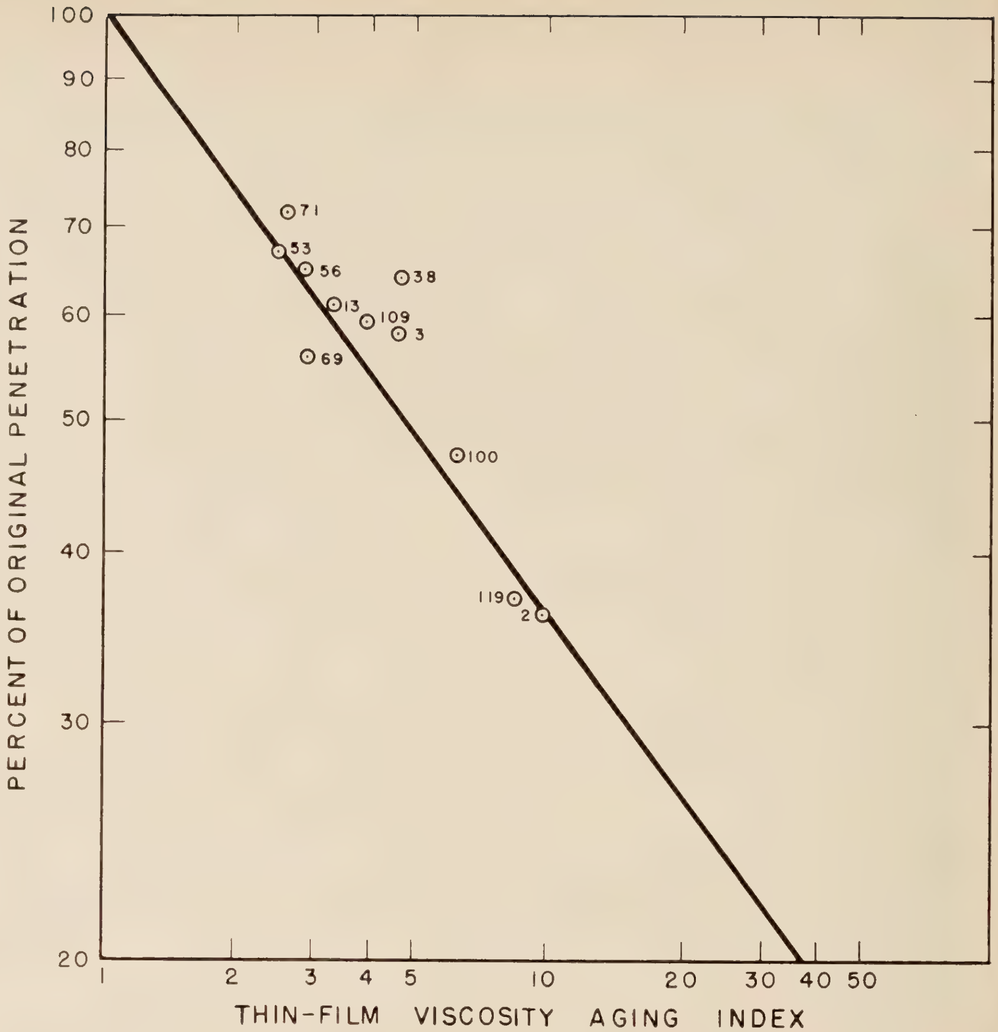

Figure 8.—Relation between percentage of original penetration and viscosity aging index in thin-film test.

Life-Saving Benefits of the Interstate System

by **CHARLES W. PRISK, Special Assistant to the Assistant Commissioner for Research**
Bureau of Public Roads

The Interstate System, with its many advanced design features including control of access, will have a key role in highway safety. A reappraisal of its value in this respect indicates that in 1973, the first year after its scheduled completion, more than 5,000 lives will be saved by virtue of the safety features of the Interstate System.

Introduction

IN EACH YEAR OF the past decade, traffic accidents on all roads and streets in the United States have taken 35,000 to 40,000 lives. The total annual number of fatalities has fluctuated within this range during the decade and while it has been increasing somewhat during the past few years, this increase has by no means matched the concurrent growth in the number of vehicles registered annually and the miles that they travel. A reasonable and widely accepted measure of highway safety is the number of fatalities per 100 million vehicle-miles of travel. With this measure showing a fairly steady decline from 7.5 in 1951 to 5.4 in 1959, the record of progress in highway safety provides some encouragement.[1]

There is much greater promise for the future. The 41,000-mile National System of Interstate and Defense Highways, now being built with a target date of 1972 for completion, represents only 1.2 percent of the Nation's total road and street mileage, but it will carry 21 percent of all motor-vehicle travel. The Interstate System, therefore, has a key role in highway safety. Features being built into the system, such as control of access, divided roadways independently designed, and carefully planned interchanges, are expected to have a favorable and lasting impact on the traffic accident problem.

From examination of available facts, it is conservatively estimated that completion of the Interstate System will each year thereafter spare the lives of at least 5,000 persons who otherwise would have died in traffic accidents. This new estimate is 25 percent higher than an earlier estimate of 4,000 made some few years ago.

The accident experience on highways having Interstate System freeway design has already been carefully compared with that on conventional highways carrying major traffic loads in the same area (*1, 2*).[2] From those data, based on studies of selected facilities in the 1950's, the effect of controlled access and other Interstate System design features on traffic fatality rates is shown in the inset bar chart in figure 1. In urban areas the Interstate type of highway has a fatality rate of 2.0 deaths per 100 million vehicle-miles of travel, as compared with 4.0 for conventional highways; in rural areas the rates were 3.3 and 8.7, respectively.

From analysis of these data, national trends in accidents, and forecasts of traffic, it appears that Interstate freeways are about two-and-one-half times as safe as the highways of earlier design they are replacing. This is an overall relationship. The nature of the benefits varies somewhat between the city and the open countryside. Judged by the number of accidents and deaths per 100 million vehicle-miles of travel, the total accident reduction benefits of freeways are greater in the cities, but their life-saving values are higher in the rural areas.

The Interstate System is scheduled for completion in 1972, and 1973 will be the first full year during which the entire 41,000-mile system will have been in use. In that year, according to estimates from the highway cost allocation study (*3*), travel on the Interstate System will amount to 81.5 billion vehicle-miles on its urban portions and 147.4 billion on its rural segments. In the same year, incidentally, total travel on all roads and streets is estimated at 444.1 billion urban and 651.8 billion rural vehicle-miles.

It is a conservative assumption that fatality rates on the Interstate System will remain essentially constant, although some modest improvement may occur as the motoring public becomes more accustomed to freeway driving and as anticipated refinements in design and operating practices are accomplished. It does seem reasonable to forecast a small reduction of fatality rates in rural areas but the urban rate is already so low that only a slight change could be expected at best. Accordingly, for the purposes of this estimate,

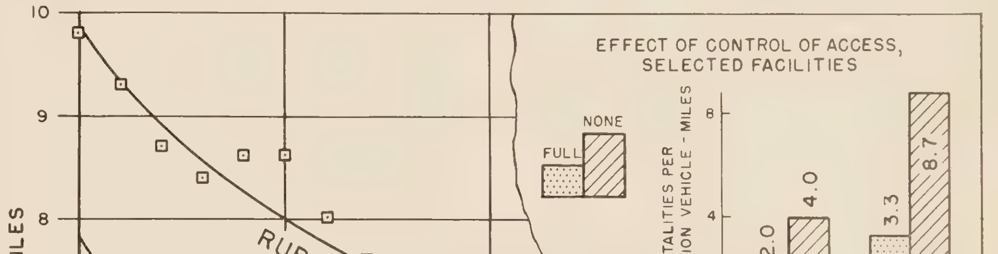

Figure 1.—Highway accident fatality rates in the 1950's, and projection to 1973.

[1] At the time this article was prepared, 1960 data were not yet available.

[2] References cited by italic numbers in parentheses are listed on page 220.

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a value of 2.0 fatalities per 100 million vehicle-miles of travel is assumed for the urban portions of the Interstate System in 1973, and 3.0 for the rural portions.

Measurement for Safety

As a yardstick for measuring the safety benefits of the Interstate System, it also was necessary to estimate the probable fatality rates for all roads and streets in 1973. Figure 1 shows, for urban, rural, and total travel, the fatality rates for all roads and streets for each of the years 1951–59, as reported by the National Safety Council (*4*). During this period the rates declined with reasonable uniformity from 4.8 to 3.2 fatalities per 100 million vehicle-miles of urban travel and from 9.8 to 7.4 for rural travel. (These are akin to the 4.0 and 8.7 indicated for selected facilities without control of access, shown in the bar chart.)

With this background trend, it is anticipated that the accident and fatality rates for all roads and streets will decline further during the next decade. However, it is unlikely that the average rates of decline of the 1951–59 period can continue. As the rates drop, it becomes increasingly difficult to reduce them further. The urban fatality rate is already low, and expansion of urban areas—that is, growth of suburban areas—tends to bring the urban death rate closer to that of the rural areas, which is typically higher.

In figure 1, curves fitted by eye to the plotted fatality rate points for 1951–59 have been projected to 1973, with the assumption that by that year the annual change in rates will be small. The 1973 rates, thus projected, are 2.8 for urban travel and 6.1 for rural.

The application of the various estimates made is shown in table 1. Simply put, the difference between the 1973 fatality rates for all roads and streets (conventional highways) and for the Interstate System is the benefit anticipated from the latter; and the product of this benefit and the travel on the Interstate System in 1973 (both in the same terms, 100 million vehicle-miles) represents the number of lives saved by the completion of the Interstate System. For rural and urban travel combined, the total is over 5,200. It is believed that this estimate is reasonably conceived and conservatively based.

Table 1.—Calculation of lives expected to be saved by the Interstate System in 1973

	Urban	Rural
Death rate (100 million veh.-mi.):		
Conventional highways	2.8	6.1
Interstate highways	2.0	3.0
Benefit (difference)	0.8	3.1
Interstate travel (100 million veh.-mi.)	815	1,474
Total lives saved (benefit×travel)	652	4,569
Total lives saved, urban and rural	5,221	

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A Laboratory-Field Study of Hot Asphaltic Concrete Wearing Course Mixtures

Reported [1] by JOSEPH F. GOODE, Highway Research Engineer,
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Results of a laboratory-field study of properties and performance of asphaltic concrete pavements, ranging in age from 3 to 12 years, from six test sections of Maryland road construction projects are described in this article.

The Los Angeles abrasion wear loss for the coarse aggregates used in the test sections ranged from 17 to 39 percent. Compaction from construction and traffic both appeared to have caused minor degradation of the aggregates. The degree of degradation was insignificant in most instances and in no instance was sufficient to affect service behavior of the pavements.

Air voids in pavement following construction compaction ranged from 5.6 to 14.5 percent. Traffic compaction produced appreciable reductions in percentage of air voids. Pavement performance and rate of asphalt hardening were related to the amount of air voids. A high percentage of air voids resulted in early deterioration of the pavement. The results of the study tend to confirm that the 6-percent air void criterion of the Bureau of Public Roads mix design procedure is satisfactory.

Introduction

INFORMATION PRESENTED in this article was based on a 13-year laboratory-field study (1947–59) conducted by the Bureau of Public Roads and the Maryland State Roads Commission pertaining to degradation of aggregates occurring in hot asphaltic concrete wearing course mixtures.

Objectives established for the study were to determine: (1) The effect of the gradation and toughness of the aggregate used in asphaltic mixtures on the aggregates' resistance to degradation during construction and after various intervals of pavement service, (2) whether aggregate degradation affected the rate of pavement densification under traffic, and (3) the effect of air voids in the pavement on changes in the physical characteristics of the contained asphalt. It was anticipated that findings from this study would make it possible to relate the foregoing factors to pavement performance and to establish the validity of the Los Angeles abrasion wear loss test and Public Roads design criteria.

The study was undertaken because only limited published information was available concerning the extent of degradation of dense-graded aggregate in hot asphaltic concrete occurring during construction or from subsequent traffic. The majority of specifications include a requirement based on the Los Angeles abrasion test (AASHO Method T 96 or ASTM Method C 131) for a coarse aggregate of sufficient toughness to resist degradation. The percentage of degradation permitted within this requirement ranges from 30 to 60 percent for maximum wear loss but most specifications limit the maximum wear to 40 percent. At the time this study was undertaken, however, practically no published information was available to justify use of such varying wear loss values in specifications or to establish that the 40 percent commonly used was the most satisfactory maximum value.

It is believed that the anticipated weight and volume of traffic and the gradation of the aggregate also should be considered during preparation of specifications establishing the maximum allowable percentage of loss. Furthermore, it was noted that few specifications were written to permit use of a softer coarse aggregate when aggregate gradations are high in percentage of material passing the No. 10 sieve, which provides a cushion for the coarse aggregate.

When this study was in the planning stage, it was believed that sufficient information could be obtained by using relatively few road test sections representing a wide range in type and toughness of the coarse aggregate used in the hot asphaltic concrete mixture. The test sections were to be selected from actual Maryland road construction projects and the contractor was not to be required to make any deviations from his normal construction operations. It was known that information concerning aggregates would be limited to that obtained for aggregates having a maximum Los Angeles abrasion test wear loss of 45 percent because this is the maximum value permitted by Maryland specifications for the lowest type of asphaltic concrete. A maximum of 40 percent abrasion loss is specified for the coarse aggregates of the higher type of asphaltic concrete.

Test sections

The six test sections selected for this study were on Maryland roads constructed of different wearing course mixtures that conformed to the Maryland specifications, *Bituminous Concrete, Specification "B"*, and *Bituminous Concrete, Bank-Run Gravel Aggregate*. The Los Angeles abrasion wear loss test of the coarse aggregates in these pavements ranged from 17 to 39 percent. The age of the pavements tested ranged from 3 to 12 years at the time of the last sampling in 1959. The locations of the test sections and their designations in this report were: Section 1, on State Route 216 between Laurel and Fort Meade; section 2, on U.S. Route 40 in Frederick; section 3, on State Route 4 near Upper Marlboro; section 4, on State Route 214 near Davidsonville; section 5, on U.S. Route 1 near Laurel; and section 6, on State Route 24 in Belair. The wearing courses were constructed on old pavement structures of both rigid and flexible types.

Conclusions

Specific conclusions concerning this study were not considered warranted because of the many variables encountered and the limited number of test sections. However, the test data did provide sufficient information to justify the statements made in the following paragraphs.

Degradation of aggregates

The amount of degradation that can be tolerated by the aggregate in a hot asphaltic concrete wearing course mixture could not be definitely determined. Only minor degradation of the aggregates occurred in the tested sections as a result of construction compaction or from subsequent traffic. This minor degradation did not affect the service behavior of these pavements.

The degradation of aggregate caused by steel-tired roller compaction during construction was slightly greater in pavement overlying a rigid pavement structure than for that in a similar pavement overlying a flexible structure. In relation to type of underlying structure, the reverse was true for degradation of aggregate caused by traffic.

[1] Presented at the 64th annual meeting of the American Society for Testing Materials, Atlantic City, N.J., June 1961.

Table 1.—Description of test sections

	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
Date constructed	May 1947	June 1947	Aug. 1948	Sept. 1954	May 1955	July 1956
WEARING COURSE MIXTURE:						
Asphalt:						
Penetration grade	85–100	85–100	85–100	85–100	85–100	85–100
Amount, mix basis, percent	5.5	5.5	6.2	5.1	5.1	5.5
Coarse aggregate	Slag	Crushed stone	Pit-run gravel	Pit-run gravel	Crushed stone	Crushed stone.
Fine aggregate [1]	Stone screenings and sand.	Stone screenings	Sand	Stone screenings and sand.	Stone screenings	Stone screenings and sand.
Aggregate passing sieve [2] —has confusing appearance. Set [2] —						
1-inch, percent			100	99		100
3/4-inch, do	100	100	99	93	100	99
3/8-inch, do	83	94	84	75	75	74
No. 4, do	71	73	74	64	59	52
No. 10, do	59	47	63	51	37	38
No. 40, do	32	17	28	21	19	22
No. 200, do	6	8	3	4	7	5
PAVEMENT STRUCTURE:						
New wearing course:						
Width, feet	24	30–36	24	24	46	24
Thickness, inches	2	2	2	1½	2	1½
New binder course thickness, do						2
Underlying structure:						
Type	Flexible	Rigid	Rigid	Flexible	Semirigid	Flexible.
Description [3]	Surface treated macadam.	Surface treated P.C.C.	P.C.C	Asphaltic concrete.	Asphaltic concrete on P.C.C.	Asphalt macadam.
COMPACTING EQUIPMENT:						
2-axle tandem roller	One 8-ton and one 10-ton.	One 10-ton	Two 10-ton	One 8-ton	Two 10-ton	One 8-ton.
3-axle tandem roller				One 13-ton	One 14-ton	One 14-ton.
CONSTRUCTION TEMPERATURES (°F.):						
Air	71	89	94	88	78	90
Mix at plant	300	300	250		292	290
Mix at paver hopper	275	275	250	300	280	250
Mix at first roller passage	185	210	140	140	230	250

[1] Sand used was natural sand.
[2] Average result from two samples.
[3] P.C.C. signifies portland cement concrete.

Pavement densities

Based on the molding procedure of AASHO Method T 167 and ASTM Method D 1074, construction specifications requiring pavement density equal to 98 percent of the pavement mixture's density established in laboratory tests does not seem to be unreasonable and a higher percentage might be justifiable. This statement is based on findings that the pavement densities after construction compaction (determined from AASHO Method T 167) ranged from 95 to 100 percent or more of the densities recorded from laboratory tests, and in only two cases was this ratio less than 98 percent. Higher percentages were obtained for pavements constructed over rigid bases than for those constructed over flexible bases.

The results of the study indicated that the ultimate density of a pavement will be approximately 104 percent of the density established by laboratory tests based on the molding procedure used.

Air voids

Rapid hardening of the contained asphalt from pavements included in this study was attributed to a high percentage of air voids in the pavement. To prevent such rapid hardening of the asphalt, and perhaps an early deterioration of the pavement, the asphalt content of the dense-graded surfacing mixture should be set high enough to assure a low percentage of initial air voids in the pavement. Test results from this study indicated that pavement after adequate compaction should have a volume of initial air voids of no greater than 7 percent.

To prevent eventual flushing of asphalt to the surface or an excessive loss in stability of the pavement, the asphalt content of the paving mixture should be low enough to permit retention of a 1 or 2 percent volume of air voids when the pavement has reached its ultimate density. From this study, it is noted that, in a properly compacted pavement, initial air voids of at least 5 percent generally will assure an adequate amount of air voids, 1 or 2 percent, when the pavement reaches its ultimate density.

On the basis of pavement performance and results of laboratory compaction tests, the 6-percent air void criterion of the Bureau of Public Roads mix design procedure, in which the immersion-compression test is used, appears to be satisfactory.

Test Procedures

Each of the six test sections used for this study represented an area of the road covered by one truckload of mixture, an area of about 700 square feet, on a conventional paving construction project. In each instance, the truckload of material selected was one that appeared to represent the contractor's normal construction mixture. All observations presented in this article were based on resules of tests of samples obtained from these loads of mixture and the areas of pavement constructed with them. A description of the composition of the test sections is given in table 1.

All mixtures contained an 85–100 penetration grade of asphalt; otherwise, the mixtures were quite different in composition. Coarse aggregates used included slag, crushed stone, and pit-run gravel. Fine aggregates used included stone screenings, natural sand, and combinations of these two materials. The gradation of the aggregates varied in nominal maximum size from 3/8- to 3/4-inch; in percentage passing the No. 10 sieve from

Table 2.—Characteristics of asphalts used in mixtures

	Test section and asphalt identification					
	1	2	3	4	5	6
Producer of asphalt	A	A	B	A	A	B
Original asphalt:						
Specific gravity, 77° F./77° F	1.036	1.017	1.025	1.015	1.019	1.034
Flash point, ° F	490	530	531	572	554	490
Softening point, ° F	120	115	113	118	116	118
Penetration	101	96	97	93	96	91
Ductility, centimeters	250+	205	200	165	228	250+
Bitumen, percent	99.90	99.83	99.85	99.77	99.93	99.71
Organic matter, insoluble, percent	0.06	0.09	0.10	0.17	0.07	0.25
Inorganic matter, insoluble, percent	0.04	0.08	0.05	0.06	0.00	0.04
After standard oven test:						
Loss, percent	0.20	0.20	0.10	0.02	0.00	0.02
Penetration	84	78	87	81	84	77
Retained penetration, percent	83	81	90	87	87	85
After thin-film oven test:						
Loss, percent	--------	--------	--------	0.03	0.00	0.76
Softening point, ° F	--------	--------	--------	126	125	133
Penetration	--------	--------	--------	54	60	47
Retained penetration, percent	--------	--------	--------	58	62	52
Ductility, centimeters	--------	--------	--------	250+	218	173

Table 3.—Traffic volumes on test sections

	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
Traffic lanes	2	[1] 3	2	2	4	2
Date constructed	May 1947	June 1947	Aug. 1948	Sept. 1954	May 1955	July 1956
Average daily traffic for all traffic lanes:						
1947	560	5,760	--------	--------	--------	--------
1948	700	6,430	4,290	--------	--------	--------
1949	790	6,610	4,890	--------	--------	--------
1950	830	7,580	5,110	--------	--------	--------
1951	880	8,760	5,260	--------	--------	--------
1952	[2] 940	8,830	5,180	--------	--------	--------
1953	--------	9,680	6,010	--------	--------	--------
1954	--------	10,870	6,400	4,730	--------	--------
1955	--------	11,500	6,490	2,980	15,560	--------
1956	--------	12,310	6,220	3,100	16,180	3,880
1957	--------	11,420	6,690	2,900	17,550	4,820
1958	--------	11,980	6,810	3,020	18,450	5,200
1959	--------	12,500	7,420	3,120	15,950	7,480
Average per year [3]	800	9,550	5,900	3,120	16,970	5,140
Estimated average for test section lane [4]	400	5,700	3,000	1,600	3,400	2,600

[1] One-way traffic. One lane is used for parking.
[2] Later traffic not included as study on this section was discontinued in 1952.
[3] From date of construction to date of final sampling.
[4] Section 2 is in right-hand travel lane and section 5 is in left-hand travel lane, and allowance was made for differences in volume of traffic between different traffic lanes. All values are rounded to nearest 100.

37 to 63; and in percentage passing the No. 200 sieve from 3 to 8.

Physical characteristics of the asphalts used in the paving mixtures for each of the six test sections are shown in table 2. For convenience each asphalt is identified by the number of its test section. Thin-film oven tests (AASHO Method T 179) were performed on asphalts 4, 5, and 6. From these data, it will be noted that asphalt 6 had a relatively high loss in weight, 0.76 percent, in the thin-film test, and its retained penetration was considerably below that of the other two asphalts tested. Asphalts 1, 2, and 3 were not subjected to the thin-film oven test and their properties cannot be compared in a similar manner.

The wearing course portion of the six test pavements ranged in thickness from 1½ to 2 inches. Underlying structures consisted of old pavements of portland cement concrete, asphaltic concrete, or asphaltic macadam.

Pavement samples were obtained the day following construction and thereafter at various intervals. These samples were taken in the outer wheel path; they consisted of slabs, about 12 inches square, cut from the pavement with a diamond blade saw. After each sample had been tested for pavement density, it was warmed so that it could be broken down and remixed to obtain a representative portion for use in the centrifuge method of extracting bitumen. The material adjacent to the saw-cut edges was discarded.

Tests also were made on asphalts recovered from samples of loose mixtures and from most samples of pavement, except for the pavement samples taken immediately after construction.

During construction, other samples taken from each project included samples of the asphalt, the aggregates from which the mixture was prepared, and a second sample of loose mixture from the truck. This latter sample was kept hot in an insulated container and delivered to the laboratory for use in preparing laboratory specimens.

The type and weight of rolling equipment used, shown in table 1, was typical of that employed in pavement construction by the State of Maryland.

The temperatures of mixtures as they were prepared and delivered were considered normal. For two test sections, the mixture behind the spreader was not rolled until it had reached a temperature of 140° F.

Traffic intensity over the six sections varied considerably. Table 3 reports recorded traffic data by year for the entire road width and shows the estimated traffic intensity for the particular traffic lane of each test section, the average daily traffic ranging from 400 to 5,700. Test sections 2 and 3 carried 20 and 10 percent

Table 4.—Progressive changes in gradation of aggregate

Section No.	Type of base	Traffic in test lane [1]	Coarse aggregate	Los Angeles abrasion test loss	Age of pavement	Properties of extracted aggregate: Gradation, percentage passing sieve—: ¾-inch	⅜-inch	No. 4 *a*	No. 10 *b*	No. 20 *c*	No. 40 *d*	No. 80 *e*	No. 200 *f*	Surface area [2]	Increase in surface area attributed to—: Construction compaction	Traffic
		A.D.T.		*Pct.*	*Years*									*Sq. ft./lb.*	*Sq. ft./lb.*	*Sq. ft./lb.*
1	Flexible	400	Slag	32	(3)	100	83	71	59	49	32	15	6.3	42	------	------
					0	100	86	76	66	54	35	14	7.0	44	2	------
					1.4	100	85	74	63	51	31	14	7.8	44	------	0
					5.4	100	84	75	66	53	33	14	8.2	45	------	1
2	Rigid	5,700	Stone	18	(3)	100	94	73	47	26	17	11	8.0	35	------	------
					0	100	93	74	49	30	22	14	9.8	41	6	------
					1.5	100	94	74	48	30	21	14	10.0	41	------	0
					5.4	100	92	72	48	30	21	14	10.1	42	------	1
					7.9	100	93	74	48	30	20	14	10.2	42	------	1
					9.1	100	92	70	47	28	20	14	9.9	41	------	0
					12.0	100	96	77	49	31	22	15	10.4	43	------	2
3	Rigid	3,000	Pit-run gravel	39	(3)	99	84	74	63	50	28	8	2.9	31	------	------
					0	97	81	73	62	50	29	9	3.5	33	2	------
					4.2	98	86	75	64	52	28	9	4.2	34	------	1
					6.7	97	85	72	62	52	29	9	4.5	34	------	1
					7.9	100	89	76	65	52	30	10	4.9	36	------	3
					10.7	97	87	75	65	52	30	9	4.3	35	------	2
4	Flexible	1,600	Pit-run gravel	39	(3)	93	75	64	51	37	21	9	4.2	29	------	------
					0	90	76	64	51	37	22	9	4.6	30	1	------
					1.8	95	79	68	54	40	24	11	6.5	36	------	6
					4.6	97	80	69	56	42	26	12	5.5	36	------	6
5	Semirigid	3,400	Stone	32	(3)	100	75	59	37	29	19	11	6.8	32	------	------
					0	100	74	58	37	29	20	12	7.3	34	2	------
					1.2	100	83	62	41	31	21	13	7.9	37	------	3
					4.0	100	79	60	40	32	22	14	7.2	36	------	2
6	Flexible	2,600	Stone	17	(3)	99	74	55	38	30	22	11	5.4	31	------	------
					0	100	72	55	40	33	24	12	5.7	33	2	------
					1.4	100	77	58	41	34	26	13	5.5	34	------	1
					2.8	98	78	62	45	36	26	13	6.1	36	------	2

[1] Approximate. See table 3.
[2] Derived from formula: Surface area $=2+0.02a+0.07b+0.11c+0.26d+0.66e+1.66f$. (See symbols underneath sieve sizes.) This formula gives surface areas comparable to those obtained by the California method.
[3] Loose mix at time of construction.

commercial vehicles, respectively; similar data are not available for the other four test sections.

Initial plans called for obtaining information on the degradation of aggregates caused by the mixture's passage through the paver. Samples of loose mixtures both from the truck and from behind the paver were taken for this purpose. It soon became evident that normal experimental error in sampling and testing obscured such minor degradation as might have occurred. To reduce the effect of these sampling inconsistencies, a decision was made to average the aggregate gradations for the two loose mixtures of each test section. These averages are the gradations shown in table 1.

Aggregate Degradation

Table 4 shows, for the six test sections, the gradations of the aggregates in the loose mixture and in the pavement samples as determined by AASHO Method T 30, as well as the type of underlying structure, average daily traffic, and Los Angeles abrasion test wear loss. In addition, the table shows the computed values of surface area per pound of aggregate and the increase in surface area that may be attributed to degradation of aggregates caused by construction compaction and traffic. Aggregate surface area was chosen as a means of judging degradation because surface areas are sensitive to changes in grain size of aggregates. Degradation of aggregate is indicated by increasing values of surface area.

The aggregate surface areas shown in table 4 were computed according to the formula at the bottom of the table. The surface area factors of the formula were derived mathematically and are based on the factors used by California (*1*) [2] for a different set of sieve sizes. Surface areas reported in the table have values equivalent to those computed by the California procedure.

For simplicity, the data on degradation of aggregates occurring during construction and those on degradation occurring under traffic, as measured by changes in area, are tabulated in table 5. The data are grouped according to the type of underlying pavement structure, and within each such group arranged in the order of magnitude of their respective values of the Los Angeles abrasion wear loss.

Degradation from construction compaction

The increase in aggregate surface area under construction compaction with steel-tired rollers, as shown in table 5, indicates that the construction operation apparently caused degradation of the aggregate for all test sections. The increase in surface area was no more than 2 square feet per pound for five of the test sections and this is not considered significant. The increase in surface area of 6 square feet per pound of aggregate noted for test section 2 may be significant, however.

For the test sections constructed over rigid bases, the data show an average increase in aggregate surface area of 4 square feet per pound, but the average increase was only 2 square feet for those test sections constructed over semirigid or flexible bases. This is considered an indication that more degradation of aggregate occurs when pavement is constructed over rigid bases than over semirigid or flexible bases. This indication is strengthened by the fact that the grouped data show about the same averages for the Los Angeles abrasion test loss and for surface area of aggregate prior

[2] References cited by italic numbers in parentheses are listed on page 228.

Table 5.—Effect of construction compaction and traffic on degradation of aggregate

Type of base and section No.	Los Angeles abrasion test loss	Effect of construction compaction:[1] Aggregate surface area—			Effect of traffic				
		Prior to compaction	After compaction	Increase attributed to construction compaction	Traffic in test lane [2]	Age of pavement	Accumulated traffic [3]	Increase in aggregate surface area attributed to traffic [3]: For accumulated traffic	Increase in aggregate surface area attributed to traffic [3]: For first million vehicles
	Pct.	*Sq. ft./lb.*	*Sq. ft./lb.*	*Sq. ft./lb.*	*A.D.T.*	*Years*	*Million vehicles*	*Sq. ft./lb.*	*Sq. ft./lb.*
Flexible:									
Section 6	17	31	33	2	2,600	1.4	1	1	1
						2.8	3	2	
Section 1	32	42	44	2	400	1.4	------	0	1
						5.4	1	1	
Section 4	39	29	30	1	1,600	1.8	1	6	6
						4.6	3	6	
Average	29	34	36	2	------	------	------	------	------
Semirigid:									
Section 5	32	32	34	2	2,400	1.2	2	3	2
						4.0	5	2	
Rigid:									
Section 2	18	35	41	6	5,700	1.5	3	0	0
						5.4	11	1	
						7.9	16	1	
						9.1	19	0	
						12.0	25	2	
Section 3	39	31	33	2	3,000	4.2	5	1	1
						6.7	7	1	
						7.9	9	3	
						10.7	12	2	
Average	28	33	37	4	------	------	------	------	------

[1] Compacted with steel-wheel rollers.
[2] Approximate. See table 3.
[3] Rounded to whole numbers.

Table 6.—Density and change in density of asphalt pavement mixtures and pavement

Type of base and section No.	Age of pavement	Accumulated traffic [1]	Pavement density: Density of sample	Pavement density: Relation of pavement density to laboratory density	Pavement density: Increase in pavement density from traffic
	Years	*Million vehicles*	*Lb./cu. ft.*	*Percent*	*Lb./cu. ft.*
Flexible:					
Section 6	(2)	0	149.4	--------	--------
	0	0	144.3	96.6	--------
	1.4	1	150.1	100.5	5.8
	2.8	3	150.4	100.7	6.1
Section 1	(2)	0	134.7	--------	--------
	0	0	131.7	97.7	--------
	1.4	--------	136.2	101.1	4.5
	5.4	1	138.8	103.1	7.1
Section 4	(2)	0	141.5	--------	--------
	0	0	133.9	94.6	--------
	1.8	1	141.3	99.9	7.4
	4.6	3	142.5	100.7	8.6
Semirigid:					
Section 5	(2)	0	158.7	--------	--------
	0	0	155.4	97.9	--------
	1.2	2	160.5	101.1	5.1
	4.0	5	161.2	101.6	5.8
Rigid:					
Section 2	(2)	0	148.8	--------	--------
	0	0	147.4	99.0	--------
	1.5	3	150.6	101.2	3.2
	5.4	11	153.2	102.9	5.8
	7.9	16	152.7	102.6	5.3
	9.1	19	153.9	103.4	6.5
	12.0	25	154.0	103.5	6.6
Section 3	(2)	0	136.0	--------	--------
	0	0	135.7	99.8	--------
	4.2	5	141.6	104.1	5.9
	6.7	7	141.9	104.3	6.2
	7.9	9	141.4	103.9	5.7
	10.7	12	141.9	104.3	6.2

[1] Rounded to whole numbers.
[2] Laboratory samples of hot mixture from field, molded by compaction procedure of AASHO Method T167.

to compaction. From a study of the data of table 5, it is obvious that the magnitude of the Los Angeles abrasion test loss cannot be related satisfactorily to degradation of aggregates caused by construction compaction as measured by increase in the surface area of the aggregates.

Degradation from traffic

The last group of columns in table 5 contains data on the degradation of aggregate caused by traffic, as indicated by increases in aggregate surface area. Data are provided on the estimated total number of vehicles that had passed over each test section prior to each sampling; the increase in aggregate surface area attributed to the traffic; and the increase in surface area after the passage of the first million vehicles, which was calculated by interpolation or extrapolation.

The increase in surface area of the aggregate attributable to traffic was negligible for all test sections except No. 4, which showed an increase of 6 square feet per pound after passage of 1 million vehicles.

All other conditions being comparable, the data show that traffic compaction caused slightly greater degradation of aggregates in pavements built over flexible bases than in pavement placed over rigid bases. It will be noted that averages of abrasion losses and of surface area of the aggregates prior to traffic for the pavements constructed over flexible bases are comparable to those for the pavements constructed over rigid bases. However, compaction from passage of 1 million vehicles caused increases in surface areas of 1, 1, and 6 square feet per pound, respectively, for the three test sections overlying flexible bases; but for the two sections overlying rigid bases, no increase in surface area was recorded for one and an increase of 1 square foot per pound was recorded for the other. Thus, as might be expected, degradation of aggregates resulting from traffic has a reverse relationship to the type of underlying structure from that of degradation caused by construction compaction.

The data in table 5 indicate no significant relationship between results of the Los Angeles abrasion test and the degradation of aggregate from either construction compaction or traffic. Aggregate degradation measured by changes in surface area was insignificant in most instances and had no apparent effect on pavement performance.

Pavement Densities

A comparison of the densities of the pavement mixture as originally determined for laboratory samples, as subsequently found for samples of the pavement after construction, and after varying periods of traffic service, is presented in table 6. Comparison of these data with those presented in table 5 reveals little or no relation between aggregate degradation and pavement densification.

Densities for laboratory specimens shown in table 6 were obtained from tests made on three or more 4- by 4-inch cylindrical specimens of hot field mixtures that were molded under a static load of 3,000 p.s.i. using the double plunger method (AASHO Method T 167 or ASTM Method D 1074).

The primary purpose of determining density in the laboratory was to provide a basis for checking the adequacy of construction compaction. Because the contractor was not requested to deviate from his regular procedure, the control of compaction was that normally required by the State of Maryland. In all instances, the density after construction compaction was essentially between 95 and 100 percent of the density of laboratory specimens. Section 4, with the lowest relative construction density, 95 percent, showed the highest increase in density from traffic compaction.

Construction compaction densities had a higher percentage of relationship to laboratory densities for the test sections overlying rigid bases than for those overlying flexible bases. This confirms the widely held belief that a firm base is more conducive to high density compaction of surfacing material than a yielding base. In all test sections, traffic compaction caused a considerable increase in density.

Test sections 2 and 3, constructed on rigid bases, appeared to have reached ultimate densities after 12 and 7 years of service and after passage of 25 and 7 million vehicles, respectively. This ultimate pavement density was approximately 104 percent of laboratory density. None of the other test sections appears to have reached ultimate density but, with the exception of section 1, none has been in service more than 5 years or has carried total traffic of more than 5 million vehicles.

Section 1 was constructed in the same year as section 2 but was surface treated soon after the 5.4-year period of sampling. The section was sampled a few times after application of this treatment but the test results were not comparable with those obtained from samples prior to treatment because of enrichment of the top portion of the pavement. This section was, therefore, discontinued as a part of the study after 5.4 years of service.

Air Voids

Pavement performance

The volume of contained air voids, which is a function of asphalt content for a particular aggregate, is an important factor affecting the performance of a dense-graded pavement. A low percentage of air voids in a pavement (high asphalt content) creates conditions conducive to rutting or shoving of the pavement or to surface bleeding. Too high a percentage of air voids (low asphalt content) creates conditions conducive to rapid hardening of the asphalt, which may result in early deterioration of the pavement from surface pitting, raveling, or cracking.

Table 7.—Properties of original and recovered asphalt compared with pavement age and percentage of air voids

Section No.[1] and source of sample	Air voids	Properties of original and recovered asphalt			
		Penetration	Retained penetration	Softening point	Ductility
Section 2:	*Percent*		*Percent*		*Cm.*
Original asphalt	--------	96	--------	115	205
Loose mix [2]	4.8	80	83	123	203
Pavement at age—					
0 years	5.7	--------	--------	--------	--------
1.5 years	3.6	78	81	121	210+
5.4 years	2.0	59	62	127	240
7.9 years	2.3	--------	--------	--------	--------
9.1 years	1.5	--------	--------	--------	--------
12.0 years	1.4	50	56	127	198
Section 5:					
Original asphalt	--------	96	--------	116	228
Loose mix [2]	3.6	77	80	120	210
Pavement at age—					
0 years	5.6	--------	--------	--------	--------
1.2 years	2.5	62	65	125	250+
4.0 years	2.1	54	56	128	250+
Section 6:					
Original asphalt	--------	91	--------	118	250+
Loose mix [2]	4.1	84	92	123	250+
Pavement at age—					
0 years	7.4	--------	--------	--------	--------
1.4 years	3.6	48	53	133	250+
2.8 years	3.5	35	38	139	173
Section 3:					
Original asphalt	--------	97	--------	113	200
Loose mix [2]	9.4	86	88	117	150+
Pavement at age—					
0 years	9.6	--------	--------	--------	--------
4.2 years	5.7	45	46	133	175+
7.9 years	5.8	29	30	142	138
10.7 years	5.5	30	31	139	98
Section 4:					
Original asphalt	--------	93	--------	118	165
Loose mix [2]	7.0	78	84	122	222
Pavement at age—					
0 years	11.9	--------	--------	--------	--------
1.8 years	7.1	41	44	133	198
4.6 years	6.3	30	32	139	64
Section 1:					
Original asphalt	--------	101	--------	120	250+
Loose mix [2]	12.6	--------	--------	--------	--------
Pavement at age—					
0 years	14.5	--------	--------	--------	--------
1.4 years	11.6	40	40	139	92
5.4 years	9.9	30	30	150	33

[1] Arranged in sequence according to air void content of most recent sample.
[2] Air voids shown are for laboratory molded specimens.

Table 7 and figure 1 show the effect of the percentage of air voids and pavement age on changes in ductility and retained penetration of the contained asphalt used in construction of the six test sections. The percentages of air voids were computed from the density data, recorded in table 6, and the maximum specific gravities of the mixtures shown in table 8. Data are also presented in table 8 on specific gravity and absorption for the aggregate portion of mixture; these data may be used, if desired, for computing the percentages of voids by other methods.

The data for the various test sections have been arranged in table 7 and figure 1 in sequence according to the magnitude of volume of air voids, as most recently determined. Thus section 2, with the lowest percentage of voids at the latest testing, is placed first.

The percentages of air voids for the laboratory molded specimens are also shown in table 7. The molding procedure followed for this study was that used by the Bureau of Public Roads in its current method of mix design (*3*) for wearing course mixtures. In this procedure, the asphalt content for a pavement to be subjected to light, medium, or heavy traffic is set at the value that will produce air voids of 6 percent in the laboratory specimen.

On the basis of this 6-percent air void criterion, tests on the molded laboratory specimens showed that the pavement mixtures for test sections 2, 5, and 6, with air voids of 4.8, 3.6, and 4.1 percent respectively, contained excess asphalt. To date, the pavement in these test sections has not shown evidence of excessive rutting or bleeding. However, incipient bleeding did develop in the pavement of test section 2 after about 6 years of service. The tests on the molded laboratory specimens from test sections 3, 4, and 1, which produced air voids of 9.4, 7.0, and 12.6 percent, respectively, showed that the pavement in these sections was deficient in asphalt. The pavement of section 3

Table 8.—Specific gravity and absorption data

	Material from section—					
	1	2	3	4	5	6
Mixture: Maximum specific gravity [1]	2.469	2.504	2.406	2.437	2.638	2.498
Aggregate:						
Bulk specific gravity	2.59	2.73	2.57	2.55	2.87	2.68
Effective specific gravity [2]	2.69	2.74	2.64	2.64	2.88	2.72
Apparent specific gravity	2.76	2.78	2.66	2.67	2.92	2.72
Asphalt absorption, by weight ... percent [2]	1.5	.1	1.0	1.4	.1	.6

[1] By Rice vacuum saturation procedure (*2*) using loose mixture broken down from most recent pavement sample.
[2] Computed.

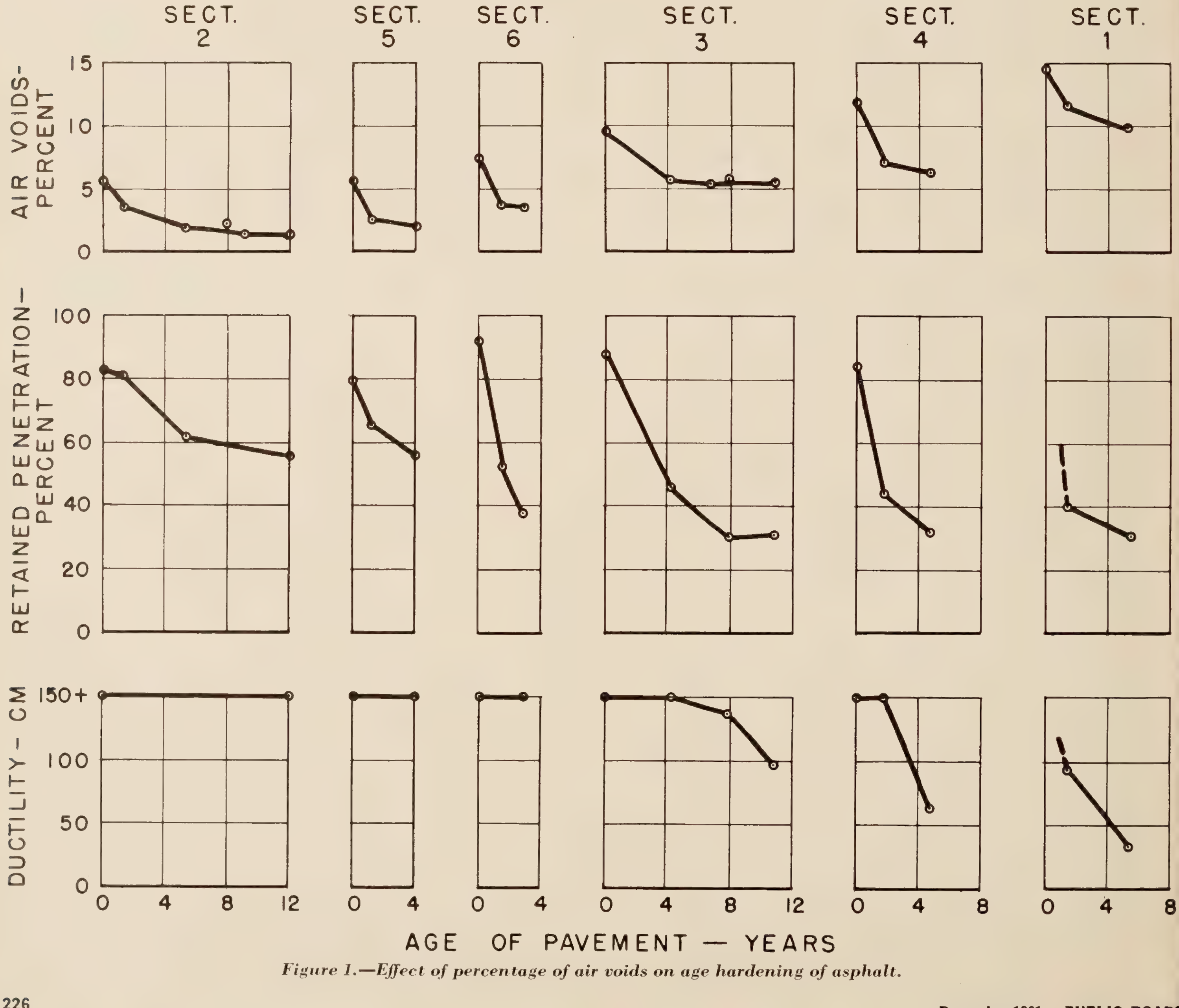

Figure 1.—Effect of percentage of air voids on age hardening of asphalt.

began to show evidence of slight surface pitting after 10 years of service and that of section 4 after approximately 2 years of service.

The pavement of test section 1 began to deteriorate rather severely early in its life. After about 6 years of service this pavement had developed such extensive hair cracking and surface pitting that a surface treatment was required. Based on the immersion-compression method of mix design, which accounts for asphalt absorption, the mixture for this pavement was deficient in asphalt by more than 2 percentage points.

The test data on pavement performance and the percentage of air voids obtained from tests on the laboratory specimens, tend to confirm the validity of the 6-percent air void criterion of the current mix design procedure used by the Bureau of Public Roads. The 6-percent value is greater than the 4.8-percent value of the laboratory specimen of section 2, which developed incipient bleeding; and less than the values developed for specimens of sections 3, 4, and 1, which showed distress in the form of pitting or cracking.

Asphalt hardening

The effect of the volume of air voids in the pavement on the rate of change in the characteristics of the contained asphalt also is shown in table 7 and illustrated in figure 1. The set of curves at the top of the figure are individual plots of air voids versus pavement age for each of the six test sections. The individual plots have been arranged in the same sequence of test sections as in table 7, as already described. The two sets of curves in the center and bottom of figure 1 are corresponding plots of retained penetration and ductility of the asphalt recovered from pavement samples. A combination of low penetration and low ductility is undesirable because it has been related to deterioration or impending failure of pavement in the form of cracking, pitting, or raveling (*4*, *5*). The rate at which such changes occur will be indicative of the length of satisfactory pavement performance.

Because all asphalts of the study were of an 85–100 penetration grade, the plotted values of percentages of retained penetration correspond very closely to actual values of penetration. From a comparison of the plotted results for retained penetration with those for air voids, it is evident that the percentage of air voids in the pavement affects the rate of hardening of the asphalt. Except for test section 6, the higher percentages of air voids in several test sections resulted in increasing rates of loss in retained penetration during the early life of the pavement. The results of the thin-film oven test (see table 2) indicated that the asphalt used for test section 6 was more susceptible to hardening than the other two asphalts subjected to this test; this might account for the faster hardening of asphalt in the pavement of section 6 than occurred in the pavement of test sections 3 and 4, which had higher percentages of air voids.

Comparison of the plotted results for ductility with the plotted percentage of air voids shows a very pronounced effect of the volume of air voids on changes in the ductility of the asphalt. It is noted that the asphalt ductility did not drop below 150 centimeters for the three test sections containing the asphalts with the lowest percentages of air voids. In the asphalts of the other three test sections (3, 4, and 1), the ductility fell below 150 centimeters; and the rate of decrease corresponded to the increasing percentage of air voids. The asphalt in test section 1, with the highest percentage of air voids, had the lowest ductility of any test section in this study, and it was this section that required surface treatment after approximately 6 years of service.

Figure 2 further shows the relation of the percentage of air voids in the pavement to the rate of change in retained penetration or hardening of the contained asphalt. In this figure, the percentage of initial air voids in the pavement is plotted against the retained penetrations of the asphalts after 4 years of pavement service. Retained penetrations were determined from the penetration values of the asphalt recovered from the loose mixture, after mixing; and from the values after 4 years of service, obtained by interpolation or extrapolation of the data from table 7. Figure 2 shows that, except for test section 6, the greater the percentage of initial air voids in the pavement, the faster the rate at which hardening of the contained asphalt occurs: For example, pavements in the test sections with a range in initial air voids of from 6 to 10 percent exhibited a range of from 72 to 49 percent in retained penetration.

From consideration of the data illustrated by figures 1 and 2 and shown in table 7, it may be concluded that a pavement with a volume of air voids appreciably greater than 7 percent at the time of its construction will have a relatively rapid rate of asphalt hardening.

The minimum percentage of air voids desired for pavement as compacted in construction, of course, depends on the probable percentage of air voids that will remain when the pavement has reached its ultimate density. If the percentage of air voids is too low at the time of construction compaction, eventual bleeding or loss of pavement stability will occur. The pavement of test section 2, which developed incipient bleeding, probably could not have tolerated an appreciable increase in asphalt; it was adequately compacted and had 5.7 percent initial air voids. It therefore appears that, for best assurance of satisfactory performance, a dense-graded asphaltic concrete pavement should have an air void volume of 5 to 7 percent immediately after construction, assuming adequate compaction. Because percentage of air voids is controlled primarily by the asphalt content for a particular aggregate, the limits previously described provide a good basis for making proper field adjustment of asphalt content based on the determinations of percentage of air voids in pavement samples after adequate compaction.

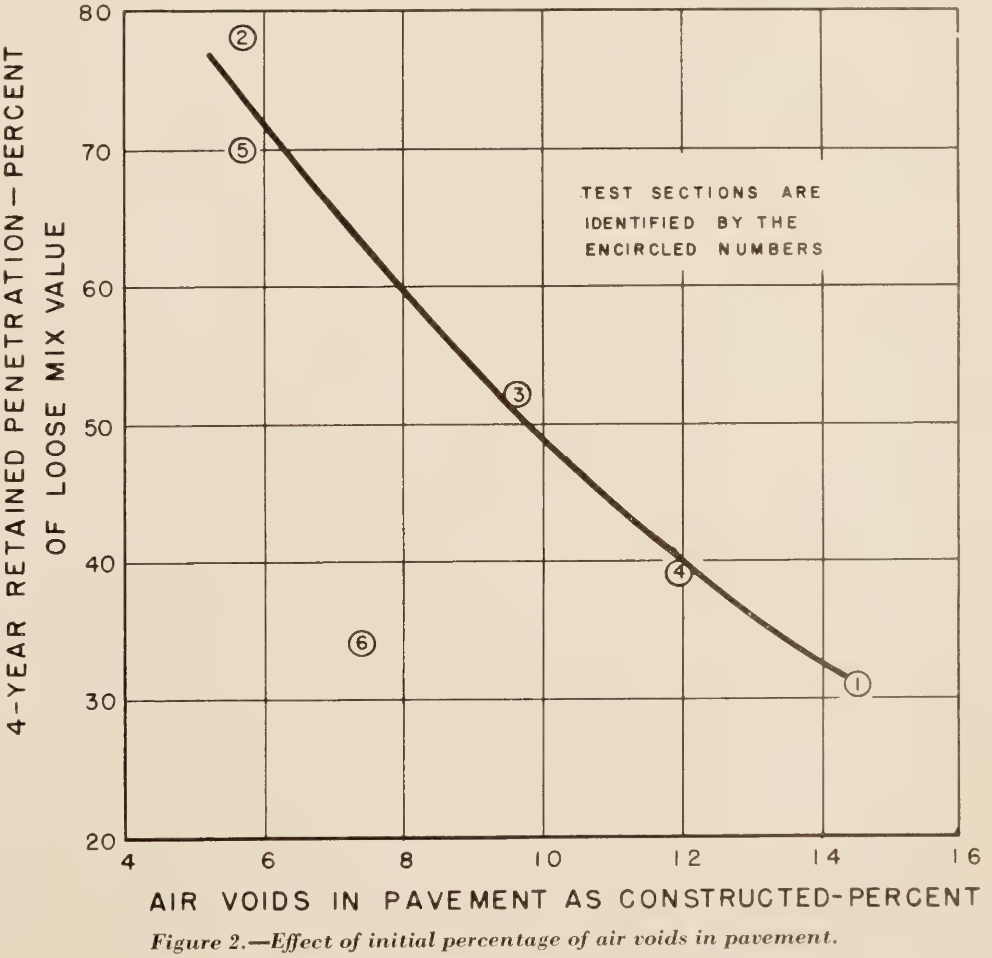

Figure 2.—Effect of initial percentage of air voids in pavement.

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New U.S. Highway Map

The Bureau of Public Roads has recently produced a 40- by 25½-inch map of the United States showing the routes of the National System of Interstate and Defense Highways, the Federal-aid Primary Highway System, and the U.S. Numbered Highway System. The map, printed in red, green, and black, is drawn to a scale of 1 inch equals 80 miles (1:5,000,000). The new map is available from the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., at 25 cents per copy.

For the 41,000-mile National System of Interstate and Defense Highways, commonly called the Interstate System, the locations of all routes are shown on the map, but only about one-fourth of the mileage is open to traffic at present. The System is scheduled for completion by 1972. The Federal-aid Primary System totals about 224,000 miles (exclusive of the Interstate System); the great majority of its routes are parts of the State highway systems. The U.S. Numbered System, 170,000 miles in extent, was devised by the American Association of State Highway Officials as a means for guiding travelers, and does not designate Federal-aid highways. However, most U.S. numbered routes are on the road systems eligible for Federal aid.

It should be noted that while the map will serve a variety of useful purposes, it is not a touring or road-condition map.

U.S. GOVERNMENT PRINTING OFFICE: 1961

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