VOL. 30, NO. 6 FEBRUARY 1959

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

PUBLISHED
BIMONTHLY BY THE
BUREAU OF
PUBLIC ROADS
U. S. DEPARTMENT
OF COMMERCE
WASHINGTON

A Federal-aid secondary highway improvement in Marion County, Oreg. A sharply curved dog-leg (seen to the right) was eliminated by the new roadway.

IN THIS ISSUE

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BUREAU OF PUBLIC ROADS
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Public Road

A JOURNAL OF HIGHWAY RESEARC

Published Bimonthly

Vol. 30, No. 6 *February 19.*

C. M. Billingsley, *Editor*

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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 28, 1958.

Progressive Alterations in a Sheet Asphalt Pavement Over a Long Period of Service

BY THE DIVISION OF PHYSICAL RESEARCH BUREAU OF PUBLIC ROADS

Reported [1] by JARL T. PAULS, Chief, Bituminous Branch, and WOODROW J. HALSTEAD, Head, Bituminous Materials Section

Progressive changes in the characteristics of the asphalt and the measurable physical properties of a sheet asphalt pavement constructed on upper Connecticut Avenue in Washington, D. C., were studied over a 19-year period. The tests conducted and reported in this article show that a large proportion of the hardening of the asphalt generally occurs during the mixing and rolling of the pavement. Although there was some indication that the extent and rate of change of properties differed with certain variations in the asphalts, the overall age-hardening in the pavement did not appear to be directly related to the type of asphalt used.

Both density of the pavement and Hubbard-Field stability of sample cores increased with time under the action of traffic, the rate of increase being greater in the early life of the pavement. The ultimate value obtained for the density varied with the lateral location of samples taken from the pavement. In the later years of the study, greatly increased traffic combined with the hardening of the asphalt resulted in the loss of a significant amount of the surface material.

THE tendency of asphalts to undergo alteration with accompanying deterioration of the pavement with age has been well established by laboratory studies and observations in the field. There have been few opportunities, however, for determining the rate, as well as the extent, of the alterations that occur under long-time service conditions and the relation of the changes in the asphalt to the measurable physical properties of the pavement. In order to obtain such information, a cooperative laboratory and field study was begun by the Bureau of Public Roads and the District of Columbia Department of Highways in 1935.[2]

During the ensuing period, periodic tests were made on samples taken from specific test sections of the pavement. It is the purpose of this article to present selected portions of the accumulated data that will show the progressive changes that have occurred in some of the more important properties of the asphalt and the pavement over a long period of time.

The experimental sections were planned to determine, if possible, (a) the effect of variations in certain characteristics of the asphalt on the service behavior of the pavement, (b) the effect of differences in volumes of traffic, (c) the changes in structural properties of the sheet asphalt surfacing with age, and (d) the changes that occur with age in the measured properties of the recovered asphalt.

General Description of Test Sections

Portions of two adjoining paving contracts on upper Connecticut Avenue in Washington, D. C., were selected as locations for the experimental sections. These contracts, which are designated in this article as contract 1 and contract 2, were awarded in 1935 to two local paving contractors. The location of the test sections and the general configuration of the pavement are illustrated in figure 1. The areas containing the A, B, and C test sections were part of contract 1, and the D sections were part of contract 2.

The contracts conformed in every respect to established standards for such work in the District of Columbia. During the laying of the test sections no attempt was made to change the contractors' methods of construction or their routine procedures. On both contracts the surfacing was laid in three lanes: two outer lanes each 20 feet wide and a center lane 19 feet wide. These lanes were separated by portland cement concrete headers, designed to serve as traffic guides. When completed, the elevation of the sheet asphalt pavement on each side of the headers was about ¼-inch higher than the headers.

The base for the center third of the street, previously occupied by streetcar tracks, was new portland cement concrete. In the two outer lanes the base was partly old and partly new concrete. In sections where the new surfacing was laid over old concrete, the sheet asphalt was laid directly against the face of the old concrete curb, so that the new surfacing along the curb served as a gutter for surface water. In sections where the old concrete in the outer lanes was replaced with a new concrete base, the sheet asphalt extended only to the edge of a new brick gutter, 12 inches wide. The surface course throughout was standard sheet asphalt. In the center third of the street the surface course was laid on a standard binder course having aggregate particles of ¾-inch maximum size. In the outer portions the binder course was of standard composition when laid on new cement concrete, and of the modified type when laid on old cement concrete. The aggregate in the leveling course or modified binder was ½-inch maximum size. The binder course was laid to about 1½ inches in compacted thickness as was also the sheet asphalt surfacing. Some variations from these thicknesses were found later in taking cores and other field samples. A summary of the information of a general nature pertaining to each contract follows.

Contract 1

Plant.—Stationary type using two driers, each 22 feet long and fired by 42-inch oil burners, and a 36-blade, 2,500-pound capacity, steam-jacketed, twin-pug mixer.

Rollers.—Two 10-ton steam tandems and one gasoline-driven tandem with front wheel diameters of 52 inches and rear wheel diameters of 40 inches.

Materials.—50–60 penetration asphalt was obtained from a local supplier; Potomac River sheet asphalt sand and limestone dust came from a commercial source.

Construction details.—Each load of mix was dumped on a metal plate, distributed with hand shovels, hand raked to grade, and finished with a 4-foot wooden lute. The steam rollers were used for initial compaction and the gasoline roller for transverse rolling and finishing.

Contract 2

Plant.—Stationary type using one 50-ton capacity oilburner drier and a 48-blade, 1,500-pound capacity, steam-jacketed, twin-pug mixer.

Rollers.—Two 10-ton gasoline tandems with front wheel diameters of 60 inches and rear wheel diameters of 49 inches.

Materials.—50–60 penetration asphalt was obtained from a local supplier (not the same as for contract 1). The Potomac River sheet asphalt sand and limestone dust came from the same commercial source as for contract 1.

Construction details.—The mixture was dumped on two metal plates, one on each

[1] This article was presented at the annual meeting of the Association of Asphalt Paving Technologists, Montreal, Canada, in February 1958. Mr. J. T. Pauls retired from the Federal service effective June 30, 1957.

[2] The helpful assistance of Messrs. H. F. Clemmer and N. G. Smith of the District of Columbia Department of Highways in planning and arranging for the various periodic inspections is gratefully acknowledged.

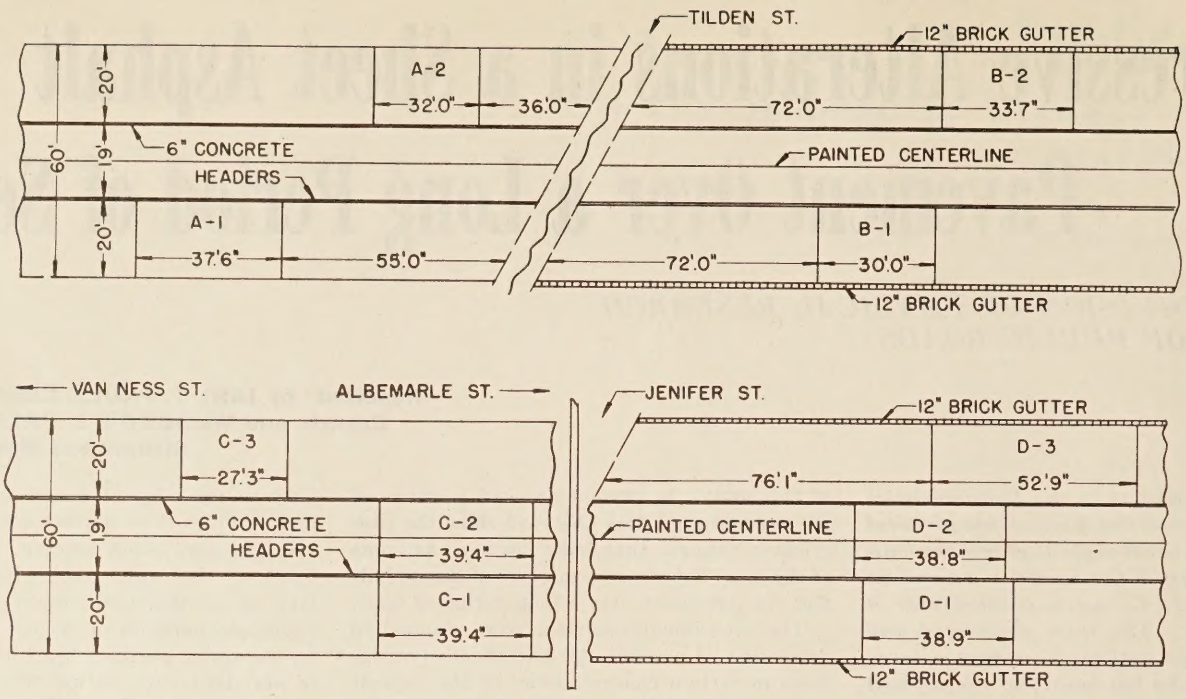

Figure 1.—Location of test sections on Connecticut Avenue in Washington, D. C.

side of a 2- by 4-inch longitudinal gage board placed in the center of each lane. The mixture was spread with shovels, raked to grade, and finished with a 6-foot wooden lute.

As illustrated in figure 1, the test sections designated as A–1, A–2, B–1, B–2, C–1, C–2, and C–3 were in the contract 1 area, and those designated as D–1, D–2, and D–3 were in the contract 2 area. Extensive data were obtained during the preparation of the sheet asphalt mixtures at the paving plant and during the spreading and rolling operations on the street. These data, in abridged form, are shown in table 1. The batch proportions shown in this table are the mix design. The variations indicated in the proportions of asphalt and filler are relatively small and were not considered of sufficient magnitude to significantly affect the properties of the mix.

The temperatures at the plant were recorded for each batch that went into the truckload which constituted the material for each test section. The individual temperatures generally varied about ±20° from the averages shown in the table.

Because of the difficulty in obtaining accurate readings, the temperature of the mixture was recorded in several locations as it was dumped and immediately before rolling. Where two values are shown, these are the minimum and maximum values. When one value is recorded, it is the average of four or five readings at different locations. The date on which each section was laid is indicated in the table and this accounts for the extent of the variations in the air temperature. All samples obtained following construction were taken in June or the early part of July, except for the final samples which were taken in April. Since sections A–1 and A–2 were laid in December 1935 and all the other sections were laid during the period from April 21 to July 7, 1936, the ages of the samples taken from sections A–1 and A–2 were recorded as ½ year greater than those for the other sections. Test sections A–1, A–2, C–1, and C–3 were laid on a leveling binder over old cement concrete, and were located in the outer lanes abutting the curb where they formed the gutters for the pavement. Sections D–1, D–3, B–1, and B–2 were laid on a standard binder over new cement concrete. They were in the outer lanes also, but they abutted a 12-inch-wide brick gutter. The other sections, C–2 and D–2, were in the center lane.

All asphalts used in contracts 1 and 2 came from two local suppliers and met the specification requirements. The asphalts used in contract 1 varied considerably in certain test characteristics, indicating that they may have been obtained from more than one source.

Summary of Findings

This investigation was planned to observe and study the effect of a number of important variables. However, all test sections gave good service and therefore the variations measured are within the range of good performance. In addition, the complexity of the interrelations between various factors makes it likely that any specific changes that occurred were the result of more than one cause. For these reasons specific conclusions cannot be drawn but a general summary is presented.

Table 1.—Mixture composition, temperatures, and other details of construction

Contract	Test section	Date of paving	Batch proportions, by weight			Average temperatures at plant			Average mixing time			Air temperature		Temperature of mix at paving location		Time rolled	Number of rollers
			Sand [1]	Filler [2]	Asphalt	Sand	Asphalt	Mix	Dry	Wet	Total	Minimum	Maximum	As dumped	Before rolling		
			Pct.	*Pct.*	*Pct.*	°*F.*	°*F.*	°*F.*	*Sec.*	*Sec.*	*Sec.*	°*F.*	°*F.*	°*F.*	°*F.*	*Min.*	
	A-1	Dec. 10, 1935	78	11	11	-----	-----	-----	----	----	----	53	56	320-350	160	65	2
	A-2	Dec. 20, 1935	78	11	11	400	290	340	18	67	85	----	30	240-310	190	60	2
	B-1	May 8, 1936	78	11	11	420	285	350	17	64	81	72	78	335	225	100	2
1	B-2	May 27, 1936	79.5	10	10.5	420	270	355	16	67	83	76	88	340-350	260	140	3
	C-1	May 8, 1936	78	11	11	395	290	335	16	61	77	88	92	290	180	105	2
	C-2	Apr. 21, 1936	78	11	11	410	285	345	15	65	80	88	92	270-340	240	75	2
	C-3	May 26, 1936	79.5	10	10.5	400	280	355	16	64	80	88	90	340-350	220	80	3
	D-1	June 17, 1936	79.8	9.7	10.5	480	290	380	0	64	64	----	89	395	240	90	2
2	D-2	May 28, 1936	79	10	11	390	290	335	0	66	66	68	74	300-330	240-280	145	2
	D-3 [3]	July 7, 1936	79.8	9.7	10.5	390	290	335	0	61	61	82	86	270-320	190-200	175	2

[1] Sand contained an average of 3.4 percent passing the No. 200 sieve.
[2] Commercial limestone filler was added.
[3] Two truckloads were used on this section; other sections required only one load each.

Table 2.—Mechanical analysis of sand and filler sampled before incorporation in the mixture

Sieve sizes	Contract 1 test sections							Contract 2 test sections		
	A–1	A–2	B–1	B–2	C–1	C–2	C–3	D–1	D–2	D–3
Percentage (by weight) of sand passing—										
No. 10 sieve	100	100	100	100	100	100	100	100	100	100
No. 20 sieve	98	99	99	98	98	99	98	99	96	98
No. 30 sieve	93	94	95	93	93	95	91	95	90	92
No. 40 sieve	82	82	85	80	82	85	77	82	78	80
No. 50 sieve	63	60	66	59	64	67	56	57	59	62
No. 80 sieve	35	27	38	33	38	37	32	28	36	34
No. 100 sieve	25	18	27	23	27	27	23	19	28	25
No. 200 sieve	4	3	3	3	3	4	3	2	5	4
Percentage (by weight) of commercial filler passing No. 200 sieve	95	96	95	95	96	97	95	95	95	94

Changes in the asphalt.—The asphalts used were from two suppliers but the long period of time over which the construction was carried out creates an uncertainty as to whether all the asphalts from the same supplier were from the same source and method of refining. The characteristics of the materials used in contract 1 indicate differences that are greater than would normally be expected for materials from the same source and method of manufacture. However, these asphalts met the specification requirements, and there is no evidence of failure that can be attributed solely to the characteristics of the materials.

All asphalts showed considerable change in properties during the mixing and laying of the pavement and during the 19 years of service. A large percentage of this change took place prior to compaction of the mix on the street. The changes in penetration and softening points during the mixing cycle were of the same order for all asphalts. However, changes in ductility and the amount of organic insoluble in 86° B. naptha were generally larger for the asphalts used in contract 1 than for those used in contract 2. The changes that occurred in the asphalt while in the pavement did not appear to be directly related to any of the characteristics of the asphalt and the degree of change was evidently affected to a considerable extent by traffic conditions, location in the pavement, and other localized conditions.

Limited studies after 19 years' service showed considerable variation in the hardness of the recovered asphalts within the same section. It was found that the asphalt in the portion of the pavement near the surface hardened to a greater extent and lost a larger proportion of its ductility than did the material in the center or bottom portions of the pavement.

The hardness of the asphalts in some of these sections approached values that have been reported elsewhere as being critical and sufficient to cause disintegration of the surface. Except in locations where the portland cement concrete base had failed, these pavements continued to give good service. This may have resulted from the inflexibility of the base which limited movement in the surface. It is quite possible that similar hardening in the surface placed over a more flexible base would have resulted in failure.

However, it should be pointed out that similarly constructed sheet asphalt pavements in the District of Columbia in locations with smaller traffic volumes have shown severe cracking and deterioration at ages for which the test sections were still in good condition. Therefore it is probable that factors not isolated by these tests have a marked effect on the tendency of the pavement to crack.

Changes in mineral aggregate.—Degradation of the sand aggregate did not occur to any appreciable extent.

Changes in pavement characteristics.—The density of the pavement after the initial rolling was generally lower than was considered the best construction practice, but there was no evidence that this condition affected the performance of the pavement. Compaction of the sheet asphalt surfacing continued under traffic, and a large percentage of the total compaction took place during the first year of service. The density of the pavement varied with the lateral location; the portions of the pavement near the curbs and the concrete headers generally had lower densities than the portions near the center of the various sections.

Stability of the sheet asphalt surfacing, as measured by Hubbard-Field tests on cores taken from the pavement, increased progressively with age and densification of the pavement for about 4 years. Results on cores taken after 10 years were inconclusive because of the inherent difficulties in obtaining satisfactory tests.

Because of the variations in densities obtained at different locations in each section at various times, the specific relation between the void content of the pavement and that obtained by laboratory design procedures was not definitely established, but a comparison of the minimum voids obtained for any group of samples showed that the values calculated from laboratory compaction approached the minimum values obtained in the road after one year under traffic.

Thickness measurements indicated that a considerable percentage of the surfacing was worn away after 19 years. There was some evidence that the amount of material lost from the surface of the pavement by the action of traffic varied with the hardness of the asphalt, but since the center lanes which normally carry the smaller amounts of traffic generally showed the least asphalt hardening, no specific relation was established.

Miscellaneous factor.—Although not a direct factor in this investigation, the performance of brick gutters was greatly superior to gutters constructed with sheet asphalt. Failure of the latter occurred in less than 10 years, whereas the brick gutters were still in good condition after more than 19 years' service.

Table 3.—Bitumen content and grading of the aggregate by extraction tests

Test section	Source of sample	Year	Bitumen extracted, percent	Grading of extracted aggregate, percentage passing sieve sizes—							
				No. 10	No. 20	No. 30	No. 40	No. 50	No. 80	No. 100	No. 200
A–1	Loose mix	1935	10.6	100	97	91	79	59	35	28	15
	Cores	1937	10.7	100	97	91	79	60	38	31	17
	do	1940	11.7	100	97	91	79	61	39	32	17
	do	1946	10.3	100	98	92	79	60	38	31	17
	Cut samples	1955	11.1	-----	----	----	----	----	----	----	----
A–2	Loose mix	1935	10.7	100	97	91	78	58	33	27	15
	Cores	1937	10.7	100	98	92	80	60	37	31	19
	do	1940	11.2	100	97	91	78	58	34	28	15
	do	1946	10.8	100	98	92	79	60	37	31	19
B–1	Loose mix	1936	11.0	100	98	93	82	63	40	32	14
	Cores	1940	11.0	100	98	93	82	64	43	34	15
	do	1946	11.3	100	98	93	81	63	43	35	17
	Cut samples	1955	11.0	-----	----	----	----	----	----	----	----
B–2	Loose mix	1936	10.7	100	98	92	81	63	42	34	14
	Cores	1940	10.6	100	98	93	82	65	45	36	17
	do	1946	10.5	100	98	92	80	63	43	35	16
	Cut samples	1955	11.0	-----	----	----	----	----	----	----	----
C–1	Loose mix	1936	10.7	100	97	92	81	64	42	34	14
	Cores	1940	10.0	100	98	92	80	63	43	34	14
	do	1946	10.7	100	98	91	79	62	43	35	15
C–2	Loose mix	1936	10.9	100	98	92	81	62	36	29	14
	Cores	1940	10.7	100	97	91	78	60	37	29	14
	do	1946	10.7	100	98	92	80	61	39	32	15
C–3	Loose mix	1936	10.3	100	97	90	78	60	40	33	15
	Cores	1946	10.2	100	97	91	78	60	42	35	15
D–1	Loose mix	1936	11.1	100	99	95	84	61	34	26	11
	Cores	1940	10.5	100	98	94	81	59	35	27	12
	do	1946	10.9	100	98	94	79	57	34	26	11
	Cut samples	1955	11.3	-----	----	----	----	----	----	----	----
D–2	Loose mix	1936	10.9	100	97	92	81	65	44	36	15
	Cores	1940	11.1	100	97	92	81	65	45	36	15
	do	1946	10.9	100	98	92	81	65	46	38	16
D–3	Loose mix	1936	10.3	100	97	91	80	64	42	34	16
	Cores	1940	9.1	100	97	91	79	63	42	33	15
	do	1946	10.5	100	98	92	80	64	43	35	16

This may have been caused by a poor seal between the concrete of the curb and the sheet asphalt, which permitted water to get in and under the surface with subsequent acceleration of deterioration by freezing.

Sampling and Testing Procedure

In order to accomplish the stated objectives of the project, individual samples of all the various materials used in each contract and samples of freshly prepared uncompacted sheet asphalt mixtures were obtained at the paving plant and sent to the laboratory for testing. After the surfacing had been compacted and before opening the street to traffic, 2-inch diameter cores were cut from each experimental section except B–1, where it was impractical to close the street. These cores were taken from the pavement in pairs about 6 inches apart longitudinally and at 2-foot intervals across the width of the section. Cores were similarly cut at various times throughout a 10-year period. At the end of 19 years, final samples were cut as slabs by air hammers.

The program of laboratory tests was based on obtaining as complete information as possible on the original materials, expecially the asphalts, and then determining the changes that occurred in the various measurable properties of both the asphalt and the pavement mixture.

Tests made on the original asphalt included penetration tests at various temperatures, ductility tests at various temperatures and speeds, softening points, solubilities in both carbon tetrachloride and 86° B. naphtha, naphtha-xylene equivalents, and special thin-film oven tests on two representative samples. The studies of the mixes and cored samples were based on the determinations of the density (apparent specific gravity) of the top inch of each core as it was taken from the road and Hubbard-Field stability of both the mixes compacted in the laboratory and the cored samples. The asphalt was recovered by the Abson method for the determination of the various properties for comparison with those of the original asphalt. Bitumen content was determined by extraction of the field samples, and a portion of the aggregate extracted from each sample was graded for comparison with the original.

Typical analyses of sand and filler sampled before incorporation in the mixture are given in table 2. The gradation of the sand sampled from the stock piles at the plants was reasonably uniform. Such variations as are shown in the table are not believed to be significant. The filler was limestone dust and was quite uniform with respect to the percentage passing the 200-mesh sieve. The apparent specific gravity of the sand was 2.67 and that of the filler was 2.70.

Results of Extraction and Recovery Tests

Results of sieve analysis of the mineral aggregate, extracted from samples of the original mixtures taken during construction of the sections and from samples taken from the pavement at various times, are shown in table 3. Also given in this table are the percentages of bitumen extracted from the samples. The results reported for the co[...] are those obtained on composite samples c[...]sisting of the top inch of all cores taken from [...] particular test section at any one time, and a[...] not those for single cores. For the cut sampl[...] taken in 1955, the results given in table [...] for sections A–1 and B–2 were obtained fro[...] single samples, whereas those for sectio[...] B–1 and D–1 were from a composite samp[...]

In all sections the average amount of bit[...]men extracted from the cut samples agre[...] closely with the amount previously shown [...] table 1. However, in two sections, A–1 a[...] D–3, the range between maximum and mi[...]imum content of extracted bitumen is 1[...] percent. This spread greatly exceeds t[...] range of asphalt content found in the oth[...] sections which have an average spread of on[...] 0.4 percent, a maximum of 0.8 and a minimu[...] of 0.1 percent. Variations, such as shown [...] sections A–1 and D–3, could possibly affe[...] the service behavior of the pavement b[...] generally for most sections the variatio[...] are within the normal range to be expected [...] routine plant operations.

Grading of the aggregate, also shown [...] table 3, indicates that no appreciable amou[...] of degradation occurred during the 10-ye[...] period of traffic. The specific gravity [...] composite aggregate extracted from the co[...] samples was 2.67.

Table 4 gives the results of tests on ([...] samples of the asphalt taken before bei[...] incorporated into the mixture; (b) sampl[...] of the asphalt taken before being incorp[...]rated into the mixture, dissolved in benzen[...] and then recovered; and (c) bitumen extract[...]

Table 4.—Results of tests on the original asphalt, the original asphalt fluxed with solvent and recovered, and the asphalt extracte[...] from mixes sampled prior to compaction

Test section	Source of asphalt	Specific gravity at 77°/77° F.	Penetration: 100 grams, 5 seconds, At 95° F.	Penetration: 100 grams, 5 seconds, At 77° F.	Penetration: 100 grams, 5 seconds, At 59° F.	Penetration: 100 grams, 5 seconds, At 32° F.	Penetration: 200 grams, 60 seconds, at 32° F.	Softening point	Ductility: At 77° F., 5 cm. per min.	Ductility: At 39.2° F., ¼ cm. per min.	Insoluble in CCl_4: Organic	Insoluble in CCl_4: Inorganic	Organic insoluble in 86° B. naphtha	Naphtha-xylene equivalent
								°F.	Cm.	Cm.	Percent	Percent	Percent	Percent
A–1	Original asphalt from storage tank	1.030	148	58	24	5	14	125	155	8.0	0.04	0.15	24.4	16–20
	Original, fluxed and recovered		142	58	24	5	14	128	118	7.5	.09	.08	24.5	16–20
	Extracted from mixture		68	31	13	3	12	143	29	4.3	.06	.0	29.2	20–24
A–2	Original asphalt from storage tank	1.032	145	58	25	5	14	126	203	7.3	.02	.08	24.9	16–20
	Original, fluxed and recovered		148	60	24	5	14	128	138	7.3	.07	.12	25.2	16–20
	Extracted from mixture		70	34	14	3	11	143	40	4.5	.03	.0	28.6	20–24
B–1	Original asphalt from storage tank	1.026	144	56	24	3	17	127	226	14.3	.13	.0	23.4	0–4
	Original, fluxed and recovered		139	55	23	3	18	128	229	15.5	.14	.0	24.5	4–8
	Extracted from mixture		73	33	14	2	11	141	80	5.3	.15	.0	27.6	4–8
B–2	Original asphalt from storage tank	1.032	134	53	23	3	17	129	233	13.0	.08	.04	25.2	4–8
	Original, fluxed and recovered		124	53	23	5	15	130	237	11.5	.12	.0	25.4	8–12
	Extracted from mixture		68	33	15	4	12	145	36	4.8	.09	.04	28.9	12–16
C–1	Original asphalt from storage tank	1.028	138	56	22	4	15	126	205	13.3	.09	.01	23.1	0–4
	Original, fluxed and recovered		139	57	22	4	15	127	216	13.5	.10	.0	23.7	0–4
	Extracted from mixture		70	33	14	3	11	144	35	4.8	.05	.04	28.9	0–4
C–2	Original asphalt from storage tank	1.027	140	55	22	4	19	126	250+	14.0	.11	.06	23.2	0–4
	Original, fluxed and recovered		123	51	22	4	15	130	250+	10.5	.15	.0	24.3	4–8
	Extracted from mixture		75	36	14	3	12	142	39	5.0	.07	.04	27.9	8–12
C–3	Original asphalt from storage tank	1.030	136	55	23	4	19	127	250+	12.5	.24	.0	24.3	4–8
	Original, fluxed and recovered		129	54	23	5	17	130	195	9.5	.14	.01	25.5	8–12
	Extracted from mixture		67	32	14	2	10	145	33	4.8	.06	.0	28.7	12–16
D–1	Original asphalt from storage tank	1.024	148	57	24	4	15	126	244	14.8	.08	.02	23.0	0
	Original, fluxed and recovered		152	60	24	3	20	127	239	13.3	.14	.0	23.3	0
	Extracted from mixture		72	32	14	2	11	143	105	5.5	.06	.0	26.9	0
D–2	Original asphalt from storage tank	1.022	144	56	24	3	12	126	250	15.5	.10	.0	21.8	0
	Original, fluxed and recovered		130	52	22	3	14	128	248	11.0	.16	.01	23.1	0
	Extracted from mixture		77	34	15	2	11	139	141	5.3	.14	.0	25.2	0
D–3	Original asphalt from storage tank	1.022	150	58	22	4	15	126	250+	17.8	.08	.03	22.2	0
	Original, fluxed and recovered		139	55	22	4	18	128	250+	14.0	.14	.0	23.1	0
	Extracted from mixture		86	36	15	3	10	137	141	6.0	.08	.0	25.1	0

Table 5.—Results of thin-film oven tests on representative asphalts

Film thickness	Time of heating	Tests on residue: Penetration at 77° F.	Softening point	Ductility at 77° F.	Retained penetration
A.—MATERIAL USED IN SECTION A–2, CONTRACT 1					
In.	*Hr.*		*° F.*	*Cm.*	*Pct.*
----	[1] 0	58	126	203	------
1/8	2	42	136	118	72
1/8	5	32	145	29	55
1/8	7	28	151	14	48
1/16	2	37	141	63	58
1/16	5	26	155	8	45
1/16	7	21	163	6	36
1/32	2	30	150	15	52
1/32	5	18	176	4	31
1/32	7	16	185	3	27
B.—MATERIAL USED IN SECTION D–2, CONTRACT 2					
----	[1] 0	56	126	250	------
1/8	5	33	139	155	59
1/16	5	26	147	35	46

[1] Original asphalt.

from loose mixture samples and recovered. Tests on the original asphalt, dissolved and recovered, show the effect of the recovery process on the several test characteristics of the asphalts, whereas the tests on the extracted and recovered bitumen from the mix show the effect of the mixing process on these characteristics.

Considering, first, the characteristics of the original asphalts sampled from the paving plant storage tanks, the results of the penetration test at 77° F. and the softening point determination for the asphalts from all sections in both contracts were in close agreement.

For the asphalts used in contract 1, there were variations in some of the properties that were greater than would normally be expected for asphalts manufactured from the same source by the same method of refining. As shown in table 1, these asphalts were supplied over a considerable period of time, therefore it is quite possible that some changes in the crude material or method of refining occurred. Specific information relative to this possibility is not available. However, the specific gravity at 77° F., the ductility at 39.2° F. (1/4 cm. per minute), the naphtha insoluble, and the naphtha-xylene equivalent values given in table 4 indicate that the asphalts used for sections A–1 and A–2 in December 1935 were similar.

The asphalts used during the period from April 21 to May 8, 1936, for sections C–2, B–1, and C–1 were similar, and likewise the material used May 26 and 27 for sections C–3 and B–2. The test results for the asphalts used in all three sections of contract 2 showed a high degree of uniformity in all test characteristics.

The results given in table 4 show that marked alteration occurred in the asphalts during the mixing and spreading processes. Comparison of the test results of the asphalts extracted and recovered from the samples of loose mixtures with corresponding results of tests on the original asphalts shows pronounced losses in penetration and ductility, and increases in softening point temperatures. The percentage of retained penetration at 77° F. ranged from 71 percent of the initial value in section C–2 to 53 percent in sections A–1 and D–1, and averaged 60 for all sections. Ductilities at 77° F. were reduced to values ranging from 29 cm. in section A–1 to 141 cm. in sections D–2 and D–3. Increases in softening point temperature ranged from a minimum of 9° F. in section D–3 to a maximum of 17 degrees in section C–1.

The changes in penetration and softening points were not significantly different for the asphalts used in contract 1, when compared with the corresponding changes in asphalts used in contract 2. However, the decreases in ductility at 77° F., 5 cm./min., for the asphalts used in contract 1 are generally greater than the corresponding decreases for materials used in contract 2. There were also greater increases in the average amounts of organic insoluble in 86° B. naphtha for contract 1 asphalts than for contract 2 asphalts.

Table 5 shows the results of special thin-film oven tests conducted in order to obtain further information on the possible differences in asphalts used in contracts 1 and 2 and to evaluate their susceptibility to change under controlled conditions. The material designated as *A* was that used in section A–2 and that designated as *B* was used in section D–2. These tests were made by heating 50-ml. (volume at 77° F.) samples of each asphalt in an oven at 325° F. in different size containers so that various film thicknesses were obtained. Various heating periods were used for each film thickness. The residues obtained for each condition of test were examined for penetration, ductility, and softening point.

Studies of the relation of the amount of hardening that occurs in the thin-film oven test have shown that the losses in penetration at 77° F. and the ductility at 77° F., 5 cm./min., for a film thickness of 1/8 inch and a heating time of 5 hours, generally approximate the losses occurring in the plant mixing and laying operation (*1, 2*).[3] Sample *B* which was used in section D–2 showed higher resistance to change than did sample *A* which was used in section A–2, contract 1. The 1/8-inch, 5-hour residue for *B* retained 59 percent of its original penetration and had a ductility of 155 cm. at 77° F. The same residue for *A* retained 55 percent of its original

[3] Italic numbers in parentheses refer to the list of references on p. 144.

Table 6.—Characteristics of asphalts recovered from the pavements at various ages

Test section	Age of pavement	Penetration at 77° F., 100 g., 5 sec.	Softening point	Ductility: At 77° F., 5 cm./min.	Ductility: At 39.2° F., 1/4 cm./min.	Organic insoluble in 86° B. naphtha	Naphtha-xylene equivalent	PTS[1]
	Years		*° F.*	*Cm.*	*Cm.*			
A–1	[2] 0	31	143	29	4.3	29.2	20–24	0.0214
	2½	32	144	26	4.7	29.7	20–24	.0209
	4½	28	146	20	4.0	29.1	24–28	.0211
	10½	24	154	9	2.3	30.6	28–32	.0198
	19½	22	156	7	---------	31.3	16–20	.0198
A–2	[2] 0	34	143	40	4.5	28.6	20–24	.0208
	2½	38	139	100	5.3	29.0	20–24	.0213
	4½	31	142	52	4.8	28.0	24–28	.0217
	10½	27	148	15	3.8	30.7	28–32	.0207
B–1	[2] 0	33	141	80	5.3	27.6	4–8	.0216
	2	29	145	30	4.3	28.8	4–8	.0212
	4	32	143	66	4.5	29.2	4–8	.0212
	10	25	151	14	3.3	30.1	8–12	.0203
	19	14	166	5	---------	--------	---------	.0197
B–2	[2] 0	33	145	36	4.8	28.9	12–16	.0204
	2	26	151	12	2.0	31.8	8–12	.0201
	4	18	158	6	.5	32.1	16–20	.0203
	10	17	164	5	.3	34.6	20–24	.0192
	19	13	171	4	---------	34.0	12–16	.0190
C–1	[2] 0	33	144	35	4.8	28.9	0–4	.0221
	2	29	145	28	4.4	29.0	8–12	.0212
	4	25	146	24	4.5	28.0	8–12	.0218
	10	24	152	12	3.5	30.3	12–16	.0203
C–2	[2] 0	36	142	39	5.0	27.9	8–12	.0207
	2	30	146	36	4.5	28.4	4–8	.0207
	4	28	145	25	4.3	28.2	12–16	.0214
	10	31	145	41	4.5	29.1	12–16	.0208
C–3	[2] 0	32	145	33	4.8	28.7	12–16	.0206
	2	26	150	14	2.8	30.9	12–16	.0204
	4	23	154	8	2.0	31.8	20–24	.0200
	10	18	162	5	0	33.9	20–24	.0194
D–1	[2] 0	32	143	105	5.5	26.9	0	.0212
	2	29	146	23	4.5	28.9	0	.0209
	4	33	144	38	4.5	27.7	0	.0207
	10	26	150	18	1.8	29.1	0–4	.0204
	19	21	157	8	---------	--------	---------	.0198
D–2	[2] 0	34	139	141	5.3	25.2	0	.0221
	2	33	141	62	5.0	26.8	0	.0216
	4	34	141	68	5.0	27.1	0	.0214
	10	32	141	167	5.3	26.3	0–4	.0218
	19	31	144	69	---------	26.9	---------	.0211
D–3	[2] 0	36	137	141	6.0	25.1	0	.0224
	2	31	141	106	5.2	27.0	0	.0221
	4	32	142	114	5.0	26.7	0–4	.0215
	10	23	148	23	3.5	28.4	8–12	.0217
	19	21	153	10	---------	27.6	0	.0208

[1] Penetration-temperature-susceptibility factor.
[2] Samples of loose mixtures at time of construction.

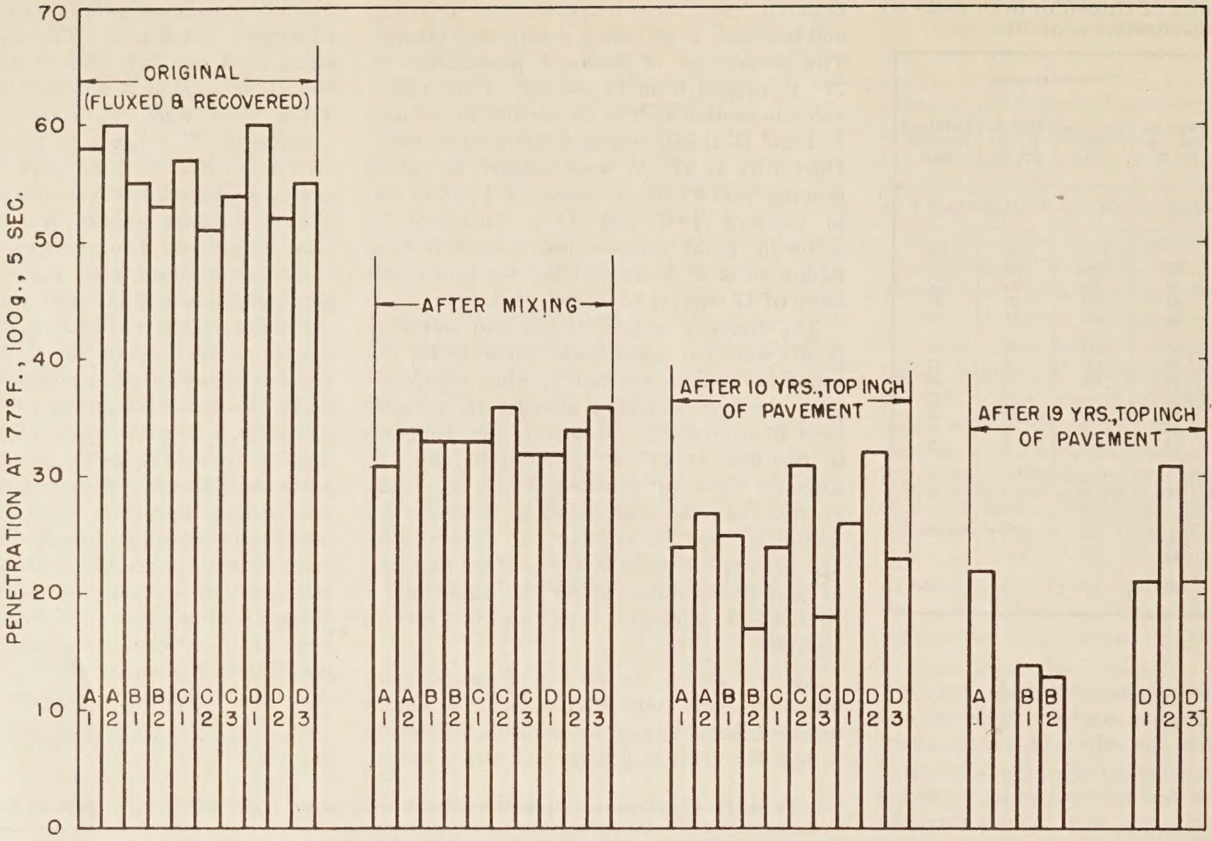

Figure 2.—Penetration of recovered asphalts at various time periods.

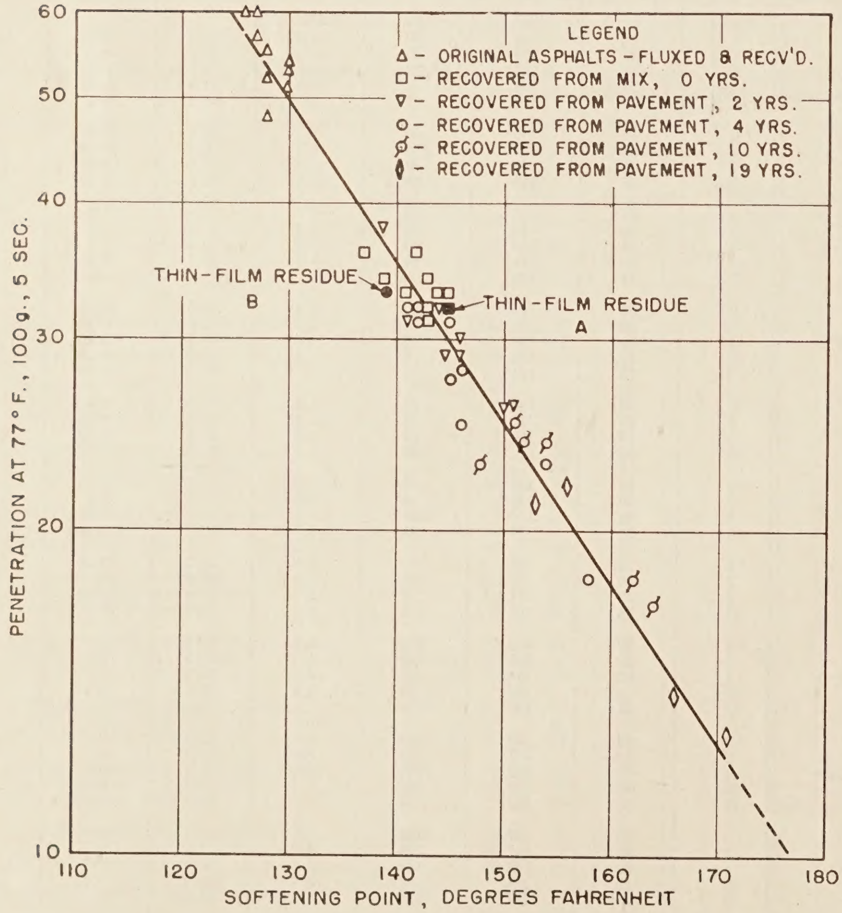

Figure 3.—Penetration-softening point relation for asphalts recovered from pavement test sections.

penetration but had a ductility of only 29 cm. Both of the original asphalts had ductilities greater than 200 cm. It should be borne in mind that these results may not necessarily be typical of the materials actually used in all sections of contract 1 because of the variations in the asphalts.

Further Alterations in Asphalt Characteristics With Age

The effect of age on certain test characteristics of the asphalts extracted from cores and cut pavement samples taken at time intervals over a period of 19 years is shown in table 6. The test results of the bitumen extracted from samples of loose mixtures are shown as the zero age of pavement for each section. These results are taken as the basis for comparison of the progressive alteration of the asphalts in the various sections.

Table 6 shows the progressive hardening of the asphalts that occurred with age. This is evidenced by lower penetration and ductility and by higher softening point temperatures. In addition to these physical changes, some chemical changes also occurred during aging as evidenced by the general trend toward increased percentages of organic material insoluble in 86° B. naphtha, increased naphtha-xylene equivalents, and a decrease in the penetration-temperature-susceptibility factor. The penetration-temperature-susceptibility factor was calculated using the hypothesis of J. Ph. Pfeiffer and P. M. Van Doormal (*3*) that the penetration of an asphalt at the softening point is equal to 800.

$$PTS = \frac{\log 800 - \log p}{t - 77}$$

Where:

PTS = Penetration-temperature-susceptibility factor.

p = Penetration at 77° F., 100 g., 5 sec.

t = Softening point, °F.

Figure 2 provides a comparison of the penetration of the recovered asphalts after mixing and after the several periods of aging of the original asphalt (fluxed and recovered). This graph shows that a large proportion of the change in penetration occurs during the mixing process, being 50 percent or more of the total drop in penetration over the 19-year period for which recoveries were made. Figure 2 also reflects a considerable range in penetration values for the various sections. The recovered asphalt from section D–2 after 19 years had a penetration of 31 at 77° F., the highest for any of the sections for which data were available, and the asphalt from section B–2 after 19 years had a penetration of 13 at 77° F., the lowest obtained.

In order to obtain a more complete picture of the alterations in the properties of the asphalt, in addition to the penetration, the softening points, ductilities, and the relations between these characteristics and the penetrations must also be considered. The penetration-softening point relations are shown in figure 3 in which the penetration of each original and recovered asphalt is plotted on a logarithmic scale against its softening point. Although there is some scatter of the points, a good straight line relation exists. This relation indicates changes that occurred in the penetration-temperature-susceptibility factor were directly related to the change in penetration, and there was no general trend toward an increased degree of oxidization as hardening continued.

Figure 4 shows the logarithm of the ductility values plotted against the penetration. Although two general curves have been drawn to show the differences between asphalts used in contract 1 and those used in contract 2, it is recognized that for penetration values of 30 or below the differences between asphalts from contracts 1 and 2 are no greater than the differences between various asphalts recovered from the same project. For penetration values of 31 or greater, the ductility-penetration relation for the materials from the two contracts appears to be significantly different. The softer materials recovered from contract 2 have considerably more ductility for the same penetration than do the materials recovered from contract 1.

Pavement Changes Related to Increases in Softening Point

Investigations reported by Lewis and Welborn (*4*) show that most of the physical properties of bituminous mixtures vary directly with the logarithm of the penetration of the asphalt rather than with the penetration itself. This means that the effects of equal increments of change in penetration progressively increase as the material becomes harder.

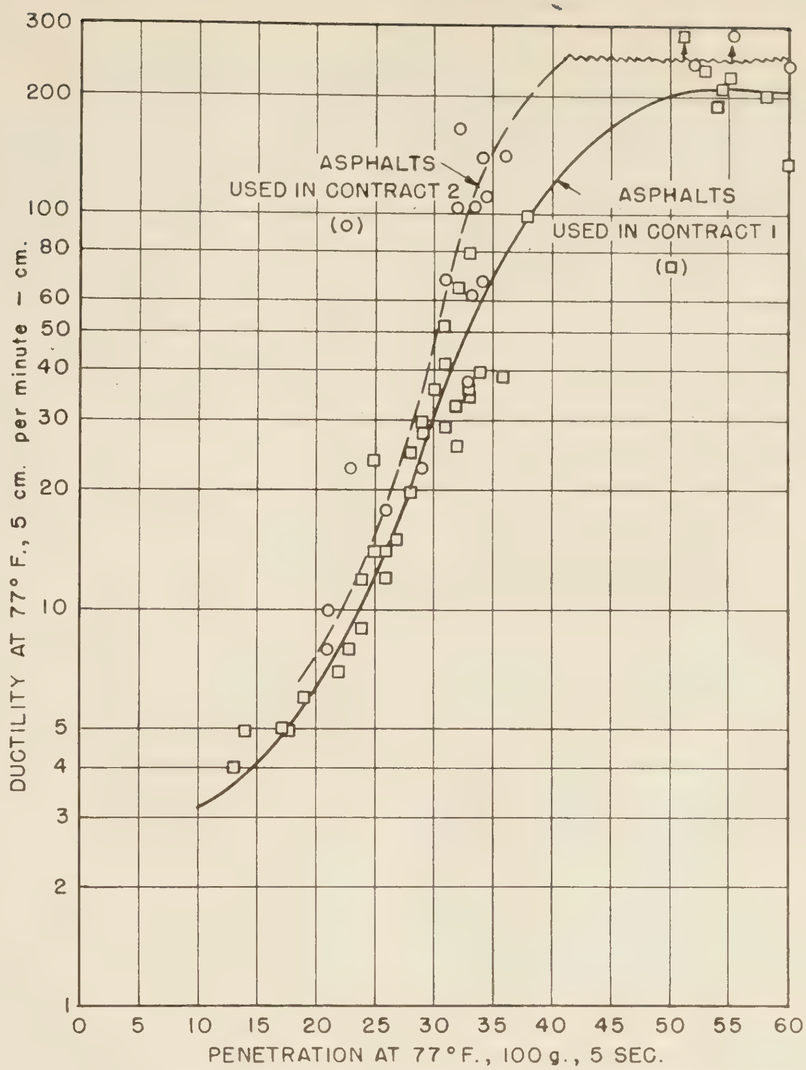

Figure 4.—Ductility-penetration relation for asphalts recovered from pavement test sections.

Figure 3 shows that the softening point also varies as the logarithm of the penetration; thus the softening point increase provides a measure of the hardening of the asphalt that is more directly related to the change in the properties of the pavement than is a consideration of penetration changes only.

A convenient relative index of change that is related to the percentage of total hardening or alteration that can be expected during the lifetime of the pavement can be calculated from the softening point data. Such an index is based on the arbitrary selection of a softening point that is representative of zero hardening and one that is representative of 100 percent hardening. The logical point of zero hardening for any single material is the softening point of the original asphalt, but in order to place all the materials in this series on the same basis a value of 127° F. was chosen. This value is shown by the curve in figure 3 for 55 penetration, the midpoint of the 50–60 specification grade. It also closely approximates the average softening point of all original asphalts which was 126.4° F.

To select a point representative of 100 percent hardening, earlier research studies were reviewed. One frequent observation was that many pavements deteriorate rapidly when the penetration of the asphalt reaches a critical point. This critical value has been variously estimated from 10 to 20 penetration at 77° F., depending somewhat on climatic conditions (*4*). Table 6 shows that the lowest penetration, at 77° F., of the recovered asphalts of the series is 13. Therefore, a penetration value of 10 appears to be a suitable reference for the selection of the point of 100 percent hardening. When the softening-point, log-penetration curve of figure 3 is extrapolated, the softening point for asphalts with a penetration of 10 is 177° F.

Thus, the total change in softening point that is expected for these asphalts during the lifetime of the pavement is 50° F. (177°–127°), and the hardness index of any particular recovered asphalt is the proportion of this total change that has already occurred. This can be expressed as a general algebraic relation as follows:

$$\text{Hardness index} = \frac{SP_x - SP_1}{SP_2 - SP_1}$$

Where:

SP_x = Softening point of the recovered asphalt being considered.

SP_1 = Softening point of the chosen zero hardening point (127° F. for these asphalts).

SP_2 = Softening point of the chosen 100 percent hardening point (177° F. for these asphalts).

Table 7.—Relative hardness of asphalts at various times based on increases in softening point

Comparison	Hardness index[1] for materials from test sections—									
	A–1	A–2	B–1	B–2	C–1	C–2	C–3	D–1	D–2	D–3
Original asphalt	−4	−2	0	4	−2	−2	0	−2	−2	−2
Original asphalt recovered[2]	2	2	2	6	0	6	6	0	2	2
Thin-film residue[3]	----	----	----	----	36	----	----	----	24	----
Asphalt from mix[4]	32	32	28	36	34	30	36	32	24	20
Asphalt from pavement[4] at—										
2 years	----	----	36	48	36	38	46	38	28	28
2½ years	34	24	----	----	----	----	----	----	----	----
4 years	----	----	32	62	38	36	54	34	28	30
4½ years	38	30	----	----	----	----	----	----	----	----
10 years	----	----	48	72	50	36	70	46	28	42
10½ years	54	42	----	----	----	----	----	----	----	----
19 years	----	----	78	88	----	----	----	60	34	52
19½ years	58	----	----	----	----	----	----	----	----	----
Hardening during mixing	30	30	26	30	34	24	30	32	22	18
Hardening in pavement at—										
10 years	[5] 22	[5] 10	20	36	16	6	34	14	4	22
19 years	[6] 26	----	50	52	----	----	----	28	10	32
Total hardening (mix plus service):										
10 years	[5] 52	[5] 40	46	66	50	30	64	46	26	40
19 years	[6] 56	----	76	82	----	----	----	60	32	50

[1] Softening point of 127° F. (equivalent to penetration of 55) taken as zero hardness index; softening point of 177° F. (equivalent to a penetration of 10) taken as 100 hardening index.
[2] Original asphalt dissolved in benzol and recovered by Abson method.
[3] Residue from ⅛-inch film heated 5 hours at 325° F.
[4] Extracted with benzol and recovered by Abson method. [5] Value for 10½ years. [6] Value for 19½ years.

Although this method of representing a hardness index is considered a very convenient way of analyzing any series of data involving recovered asphalts from the same pavement, it should be emphasized that the softening point values of zero and 100 percent hardening will differ for each series depending on the asphalts used and their hardening characteristics.

The hardness indices were calculated for all the original and recovered asphalts in this series, as shown in table 7. Also included are the differences in the indices for the asphalts recovered immediately after mixing and after various periods of service in the pavement. The negative values shown for some of the original asphalts indicate only that the softening point was lower than 127° F., the arbitrarily chosen point of zero hardening.

The indices shown in table 7 generally reflect the same trends previously discussed for the penetration data but to a different degree. The hardening during the mixing (the hardness index of the asphalt recovered from the mix, less the hardness index of the original asphalt after refluxing and recovery) of the asphalts used in contract 1 ranged from 24 to 34 with an average of 29. For contract 2, section D–1 showed a hardening value of 32, while D–2 and D–3 were 22 and 18, respectively. There is no consistent pattern in the amount of hardening that occurred with age in the pavement. The values shown in the table for 10 and 19 years were determined by subtracting the hardness index after mixing from the hardness index of the material recovered from the pavement at the indicated age. For the 10-year period the indices ranged from 6 to 36 for contract 1 asphalts and from 4 to 22 for contract 2 asphalts. For the 19-year period, the indices for the three contract 1 sections for which data were available ranged from 26 to 52, and for the contract 2 sections, the range was 10 to 32. Since by definition these numerical values are related to the percentage of hardening at any particular stage based on the total expected hardening during the lifetime of the pavement, a wide variation in hardening characteristics is evident.

The data upon which this discussion is based represent the properties of the asphalts recovered from composite samples taken from the full width and depth of the pavement. However, it should be borne in mind that considerable variation can occur within each section. To illustrate this point, recoveries were made for samples taken from section B–1 at specific locations across the pavement after 19 years' service. The results of the tests are shown in table 8. Also included in the table are results for special tests made on material from section D–2 after 19 years. In this case, a slab of pavement was sawed in a plane parallel to the surface so as to obtain the material from the top ¼ inch, the center ¼ inch, and the bottom ¼ inch. The asphalt at each of these depths was then extracted and recovered.

Section D–2 showed the least amount of hardening based on the results of the total sample, and the difference in the penetration of the top ¼ inch and the middle ¼ inch is not significant. Both top and center samples showed more hardening than did the bottom. The hardness index value for the bottom ¼ inch is the same as that obtained for the asphalt recovered from the loose material after mixing, which indicates very little change in the asphalt at this level. The ductility values are also of interest since the material recovered from the top ¼ inch had a ductility

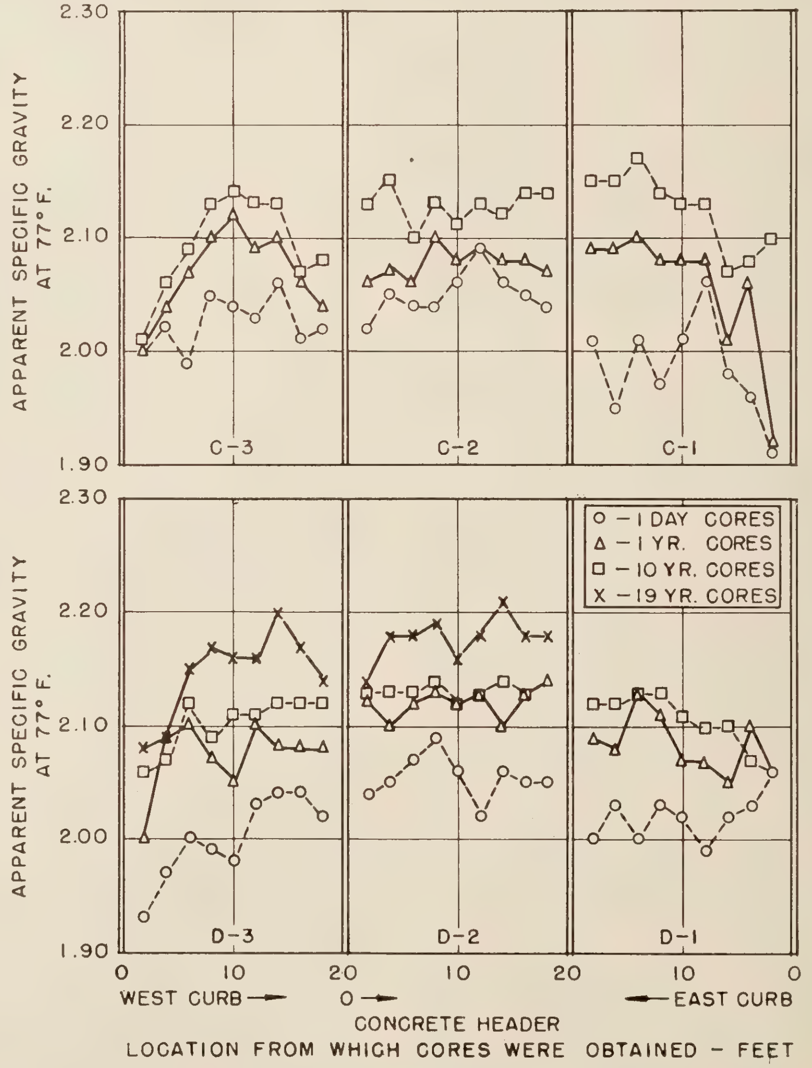

Figure 5.—Relation of density to location of samples taken from pavement test sections.

of only 25 cm. as compared with 247 cm. for the middle ¼ inch. This comparison indicates that changes other than simple hardening occur at the surface of the pavement, but unfortunately the available data are not sufficient to support definite conclusions.

Table 8.—Variations in characteristics of recovered asphalts from different locations and depths in the pavement after 19 years' service

Test section	Location: Distance from curb	Location: Portion of pavement	Bitumen (aggregate basis)	Penetration at 77° F.	Softening point	Ductility at 77° F., 5 cm./min.	Hardness index [1]
	Feet		*Percent*		*°F.*	*Cm.*	
B-1	2	----------	12.1	12	167	3	80
	4	----------	11.6	14	168	4	82
	6	----------	12.2	14	165	4	78
	8	----------	12.0	11	168	3	82
	10	----------	11.6	9	174	3	94
	12	----------	11.8	12	175	4	96
	14	----------	12.2	14	168	4	82
	16	----------	11.5	19	159	7	64
	18	----------	12.2	23	151	15	48
D-2	--------	Top ¼ inch [2]	13.1	31	146	25	38
		Center ¼ inch [2]	12.2	32	140	247	26
		Bottom ¼ inch [2]	12.4	38	139	225	24
		Full thickness [3]	12.5	31	144	69	34

[1] See footnote 1, table 7.
[2] Samples obtained by longitudinally sawing slab; ¼ inch of material removed by saw blade for each cut.
[3] Extraction and recovery for full thickness of slab.

Table 9.—Apparent specific gravity of top inch of cores cut from pavement

Test section	Pavement age when cored, years	Apparent specific gravity of top inch of cores taken from pavement at distances from curb of—: 2 feet	4 feet	6 feet	8 feet	10 feet	12 feet	14 feet	16 feet	18 feet
A-1	[1] 0	1.98	2.02	2.05	2.11	2.11	------	2.08	2.09	2.03
	½	2.03	2.05	2.08	2.13	2.13	2.14	2.15	2.08	2.06
	1½	2.06	1.96	2.08	2.14	2.12	2.15	2.15	2.09	2.08
	2½	2.05	2.10	2.09	2.12	2.12	2.15	2.13	2.13	2.12
	4½	------	2.04	2.09	2.12	2.15	2.18	2.17	2.13	2.15
	10½	2.04	2.07	2.08	2.12	2.14	2.14	2.15	2.18	2.14
	19½	------	------	------	------	------	2.16	2.18	2.19	2.15
A-2	[1] 0	1.90	2.01	1.98	2.00	2.08	2.07	2.04	2.04	2.07
	½	1.94	2.02	2.01	2.03	2.13	2.12	2.10	2.11	2.11
	1½	1.94	2.07	2.07	2.13	2.15	2.15	2.14	2.08	2.15
	2½	1.94	2.02	2.08	2.10	2.15	2.16	2.16	2.16	2.18
	4½	------	2.13	2.17	2.15	2.17	2.16	2.16	2.20	2.16
	10½	2.02	2.07	2.12	2.15	2.20	2.17	2.20	2.19	2.15
B-1	½	1.99	2.04	2.06	2.07	2.12	2.11	2.10	2.10	2.07
	1	1.99	2.03	2.06	2.08	2.10	2.11	2.10	2.05	2.08
	2	1.95	2.02	2.07	2.06	2.11	2.07	2.09	2.08	2.10
	4	------	2.02	2.04	2.06	2.13	2.10	2.13	2.12	2.13
	10	2.00	2.04	2.06	2.08	2.12	2.12	2.13	2.14	2.16
	19	2.01	2.06	2.09	2.10	2.08	2.10	2.10	2.13	2.14
B-2	[1] 0	1.98	1.97	2.03	2.06	2.04	2.00	1.99	2.05	2.03
	1	2.02	2.01	2.05	2.07	2.07	2.06	2.07	2.05	2.07
	2	2.02	2.01	2.04	2.05	2.05	2.06	2.07	2.08	2.08
	4	------	2.02	2.01	2.06	2.05	2.06	2.07	2.08	2.08
	10	2.04	2.06	2.06	2.06	2.08	2.08	2.09	2.11	2.08
	19	------	2.08	2.07	2.10	2.11	2.10	2.13	2.15	2.16
C-1	[1] 0	1.91	1.96	1.98	2.06	2.01	1.97	2.01	1.95	2.01
	½	1.96	2.02	2.12	2.08	2.09	2.11	2.09	2.09	2.08
	1	1.92	2.06	2.01	2.08	2.08	2.08	2.10	2.08	2.09
	2	------	2.04	2.08	2.07	2.09	2.08	2.11	2.08	2.11
	4	------	2.05	2.09	2.09	2.08	2.13	2.11	2.12	2.12
	10	2.10	2.08	2.07	2.13	2.13	2.14	2.17	2.15	2.15
C-2	[1] 0	2.02	2.05	2.04	2.04	2.06	2.09	2.06	2.05	2.04
	½	2.08	2.11	2.10	2.10	2.10	2.09	2.11	2.10	2.11
	1	2.06	2.07	2.06	2.10	2.08	2.09	2.08	2.08	2.07
	2	2.07	2.09	2.07	2.11	2.11	2.12	2.10	2.11	2.11
	4	2.11	2.14	2.12	2.12	2.10	2.14	2.12	2.13	2.11
	10	2.13	2.15	2.10	2.13	2.11	2.13	2.12	2.14	2.14
C-3	[1] 0	------	2.02	1.99	2.05	2.04	2.03	2.06	2.01	2.02
	1	2.00	2.04	2.07	2.10	2.12	2.09	2.10	2.06	2.04
	2	1.98	2.06	2.08	2.11	2.10	2.09	2.11	2.08	2.08
	4	------	2.06	2.07	2.11	2.08	2.12	2.11	2.09	2.09
	10	2.01	2.06	2.08	2.13	2.14	2.13	2.13	2.07	2.08
D-1	[1] 0	2.06	2.03	2.02	1.99	2.02	2.03	2.00	2.03	2.00
	1	2.06	2.10	2.05	2.07	2.07	2.11	2.13	2.08	2.09
	2	2.04	2.07	2.06	2.08	2.08	2.08	2.10	2.08	2.08
	4	------	2.07	2.08	2.09	2.10	2.11	2.12	2.11	2.11
	10	2.06	2.07	2.10	2.10	2.11	2.13	2.13	2.12	2.12
D-2	[1] 0	2.04	2.05	2.07	2.09	2.06	2.02	2.06	2.05	2.05
	1	2.09	2.08	2.13	2.13	2.10	2.08	2.12	2.08	2.12
	2	2.08	2.08	2.12	2.14	2.11	2.10	2.13	2.10	2.12
	4	2.12	2.10	2.12	2.13	2.12	2.13	2.10	2.13	2.14
	10	2.13	2.13	2.13	2.14	2.12	2.13	2.14	2.13	2.14
	19	2.14	2.18	2.18	2.19	2.16	2.18	2.21	2.18	2.18
D-3	[1] 0	1.93	1.97	2.00	1.99	1.98	2.03	2.04	2.04	2.02
	1	2.00	2.09	2.10	2.07	2.05	2.10	2.08	2.08	2.08
	2	2.05	2.06	2.08	2.08	2.08	2.09	2.11	2.08	2.09
	4	------	2.08	2.10	2.10	2.08	2.10	2.10	2.09	2.10
	10	2.06	2.07	2.12	2.09	2.11	2.11	2.12	2.12	2.12
	19	2.08	2.09	2.15	2.17	2.16	2.16	2.20	2.17	2.14

[1] After final rolling.

Changes in Physical Characteristics of Pavement

The densities (expressed in terms of apparent specific gravity) of the top inch of specimens cut from the sheet asphalt surfacing throughout the 19-year period of service are given in table 9. The results for all periods up to and including 10 years are for cores cut from the pavement, whereas those shown for the 19-year period are for rectangular specimens cut from the pavement. The results show the effect of the transverse location of the sample in each traffic lane on the degree of compaction and stability. Although the specific gravities reported in table 9 are for the top inch of the specimen, values for the full thickness of the pavement showed no significant differences.

The general trends are illustrated in figure 5 in which specific gravity values for sections C–1, C–2, and C–3 from contract 1, and sections D–1, D–2, and D–3 from contract 2 are plotted with respect to their relative position in the pavement. The density of the pavement nearest either curb is generally the lowest with a general trend toward higher density in the middle of each segment of the pavement. There is a corresponding decrease in density in the proximity of the concrete headers, but the difference is not as pronounced as that caused by the curbs. The greatest density gains in all sections occurred in the first 12 or 18 months of service. The density patterns after rolling and after consolidation by the traffic follow normal expectations. Generally, the effect of traffic was to decrease the irregularities in the specific gravity results, but the general pattern of higher density in the center of each section was retained. For sections C–3 and A–2 the differences in the densities between the center and edges of the pavement showed a tendency to increase, but for all other sections the densities became more uniform. The initial specific gravities for section D–1 did not follow the general trend since those nearest the east curb were generally high, but after traffic compaction this trend was essentially eliminated.

The Hubbard-Field stability test results reported in table 10 generally show a consistent increase in values up to the end of about the 4-year period, while the results at the end of 10 years are very inconsistent. Much of this variation is believed to be caused by the inherent difficulties in obtaining satisfactory cores from pavements and the test limitations when the asphalt has hardened to the extent shown after 10 years. In such cases it is extremely difficult to eliminate crushed edges or the effects of minute cracks which may form during the sampling. For this reason, cores for the stability tests were not taken at the end of the 19-year period.

Comparison of Laboratory Compaction With Field Compaction

From the standpoint of design practices, the relation between the minimum voids obtained in the actual pavement and the voids obtained by laboratory design procedures is

Table 10.—Hubbard-Field stability of top inch of cores cut from pavement

Test section	Pavement age when cored, years	Hubbard-Field stability (in pounds) at 140° F. of top inch of cores cut from pavement at distances from curb of—								
		2 feet	4 feet	6 feet	8 feet	10 feet	12 feet	14 feet	16 feet	18 feet
A-1	[1] 0	------	1,110	1,190	1,500	1,570	------	1,330	1,470	1,080
	½	1,410	1,580	2,150	2,150	2,200	2,370	2,300	2,300	2,080
	1½	1,890	1,450	2,380	2,700	2,450	2,700	2,100	2,450	2,450
	2½	------	2,180	2,280	2,550	2,680	2,400	2,630	2,630	2,720
	4½	------	2,120	2,960	3,780	4,620	3,880	3,860	3,880	3,950
	10½	3,460	3,970	------	4,140	3,270	------	2,800	4,020	------
A-2	[1] 0	------	1,130	810	920	1,540	1,300	1,130	1,140	1,080
	½	500	1,410	1,550	1,710	2,620	2,140	1,970	2,280	2,060
	1½	------	1,800	2,000	2,600	2,950	2,200	2,450	2,750	2,100
	2½	------	1,800	2,300	2,380	2,600	2,680	2,380	2,430	2,500
	4½	------	1,700	4,720	4,790	4,470	4,300	4,360	3,250	4,070
	10½	2,560	------	3,040	2,860	------	2,840	2,640	------	3,130
B-1	½	975	1,175	1,350	1,650	2,050	1,750	2,050	1,950	1,650
	1	1,550	1,550	2,050	2,050	2,500	1,950	2,100	1,850	2,050
	2	1,160	1,500	2,160	2,000	2,100	2,250	1,920	2,360	2,000
	4	------	2,150	2,450	3,130	3,600	3,420	3,550	2,450	2,050
	10	2,580	2,860	------	3,240	3,160	------	2,860	2,780	------
B-2	[1] 0	1,110	1,200	1,520	1,860	1,420	1,240	1,180	1,400	1,350
	1	1,870	1,700	2,050	2,100	2,250	2,000	2,100	2,100	2,000
	2	1,500	1,620	1,680	1,950	2,330	2,150	2,280	2,420	2,450
	4	------	2,500	2,280	2,900	2,500	3,000	3,560	3,280	3,360
	10	2,850	3,260	------	2,960	3,400	------	2,940	3,060	------
C-1	[1] 0	680	620	940	1,400	1,120	860	1,020	720	940
	½	1,050	1,300	2,350	2,200	2,250	1,850	2,150	2,400	2,050
	1	1,150	2,200	1,900	2,100	2,200	2,450	2,150	2,350	2,250
	2	------	2,300	2,370	2,570	2,430	1,950	2,450	2,000	2,150
	4	------	2,500	4,050	3,600	3,100	4,250	1,800	4,300	4,200
	10	2,800	2,840	------	3,080	3,050	------	2,640	2,450	------
C-2	[1] 0	950	1,060	1,250	1,160	1,400	1,710	1,400	1,220	1,040
	½	1,850	2,300	2,150	1,800	2,100	1,800	2,000	2,550	1,900
	1	2,400	2,250	2,200	2,150	2,200	2,150	2,450	2,100	2,350
	2	1,900	2,600	2,200	2,550	2,550	2,300	2,350	2,450	2,400
	4	2,220	3,650	3,620	3,500	3,050	3,660	2,980	3,780	3,750
	10	2,730	2,150	------	2,500	2,760	------	2,740	2,540	------
C-3	[1] 0	------	1,290	1,200	1,740	1,370	1,600	1,510	1,130	1,100
	1	1,550	2,100	2,300	2,700	2,600	2,800	2,400	2,650	2,550
	2	1,600	2,250	2,900	2,700	2,650	2,500	2,600	2,750	2,450
	4	------	3,250	3,200	4,300	3,200	3,850	3,900	4,100	3,400
	10	3,300	3,760	------	6,050	4,000	------	3,380	4,100	------
D-1	[1] 0	1,340	1,030	1,200	1,010	1,100	1,280	990	1,220	1,160
	1	2,050	2,250	2,050	1,900	2,050	2,450	2,250	2,050	2,050
	2	1,700	1,900	1,950	2,150	2,100	2,250	2,150	2,000	2,100
	4	------	2,600	3,420	3,630	3,770	2,750	3,370	4,080	------
	10	2,740	2,800	------	2,860	2,970	------	2,680	2,840	------
D-2	[1] 0	1,450	1,560	1,720	1,960	1,680	1,390	1,520	1,250	1,110
	1	2,150	------	2,200	2,250	2,250	2,400	1,900	2,250	1,900
	2	2,400	2,250	2,600	2,350	2,500	2,350	2,600	2,200	2,250
	4	4,400	4,510	------	------	3,950	4,750	4,500	3,900	3,760
	10	3,820	3,320	------	2,640	3,070	------	4,160	2,840	------
D-3	[1] 0	650	900	1,090	1,180	900	1,210	1,550	1,470	1,260
	1	1,300	1,650	2,150	1,450	1,800	1,750	1,650	1,650	1,700
	2	1,600	1,750	2,100	2,070	2,250	2,220	2,450	2,020	1,800
	4	------	3,300	3,600	3,400	3,250	3,200	2,560	3,700	3,450
	10	2,740	2,940	------	4,100	3,480	------	3,780	3,130	------

[1] After final rolling.

of interest. The actual voids are affected by the amount of compaction resulting from both the roller and traffic. The amount of compaction is also affected by the position of a particular sample in the pavement. It is believed that the maximum density obtained at any particular time should most nearly reflect the influence of the mix design. The relation of the voids calculated at this maximum density over a period of years to those calculated from laboratory specimens is shown in table 11. The table also includes the mixture voids calculated by determining the void content of the aggregate by the vibratory method as described by Pauls and Goode (*5*). The voids in the mixture were then calculated by reducing these aggregate voids by an amount equivalent to the volume of asphalt used. The percentage of voids obtained by laboratory compaction of the plant mixture from each section at 3,000 pounds per square inch, using the double plunger method, is also shown. The latter method is essentially the same as the ASTM method (*6*).

In general, both laboratory methods gave lower voids than the minimum obtained immediately after rolling. The 3,000-pound molding by the double plunger gave low voids in most instances than those calculate from the vibratory compaction of the aggr gate. The average for all sections after rollin was 8.9 percent. This compares with th averages of 7.9 percent for the vibrator method and 7.4 percent for the double plunge method and a load of 3,000 pounds per squar inch. After 1 year of traffic, the minimu voids in the pavement approached those ob tained by the two laboratory methods of com paction, the latter ranging from 6.1 to 9. percent and averaging 7.5 percent for al sections.

Because of the many variables involved the calculated voids shown in table 11 do no show a definite trend. In some sections, fo example A–1, there was very little chang in the minimum voids (maximum specifi gravity) after the 4-year period; in others a general trend toward lower voids was note as in the case of sample C–2; while in stil others a more or less stable pattern wa evident between 4 and 10 years, but there was a considerable drop in the minimum voids after 10 years, for example sectio D–2. This section had minimum void values of 5.7 for the 2-, 4-, and 10-year periods but dropped to 2.6 at 19 years.

Surface Abrasion Loss

One factor that affects the results of both the physical tests on the pavement and the characteristics of the recovered asphalt is the amount of abrasion loss caused by the traffic on the pavement. It is difficult to obtain an accurate measure of this wear because of the changes in the density of the pavement with time. However, as indicated by the data in table 9, density changes for later years were small for most sections. Accordingly, thickness measurements were not made until 1940, and it is believed that the greater portion of any measured change since that date can be attributed to the wear of the pavement rather than consolidation. The thickness measurements obtained are shown in table 12.

Although complete data were not obtained after the 19-year period, the trends noted are

Figure 6.—General view of test area (sections D–1, D–2, and D–3) in April 1955.

significant. Generally it is indicated that the wear for most sections was accelerated in the period between 10 and 19 years. This can be attributed to two causes: (a) increased hardening of the asphalt, and (b) a considerable increase in the amount of traffic for the later years. This increase in traffic is illustrated in table 13. A second trend is the greater loss of thickness for portions of sections which probably carried the most traffic. The center lanes that would normally carry less traffic showed less wear. A third possible significant item is the relation of the amount of wear to the hardness of the recovered asphalt. Sections that showed the larger amounts of hardening also showed the greatest wear. For example, sections B-1 and B-2 in which the asphalts were reduced to penetrations of 14 and 13, respectively, decreased in average thickness from 1.47 inches to 0.96 inch (a 35-percent loss) for B-1 and from 1.53 to 1.09 inches (29-percent loss) for B-2. Section D-2 which had the highest penetration of the recovered asphalt after 19 years showed a 10-percent loss (0.18 inch) in thickness, from 1.62 to 1.46 inches.

The fact that the abrasion loss in heavy traffic can be such a large proportion of the surface course introduces an additional difficulty in evaluating the amount of hardening that occurs in the asphalt. It has been established that the asphalt in the top portion of the pavement hardens more rapidly than that below the surface; therefore it is the harder material that is always being lost. The recovered asphalt must necessarily be obtained from the pavement that remains, which obviously will be softer than would be the case if all the abraided material could be retained.

Table 11.—Minimum void content of pavement samples compared with calculated voids of laboratory compacted specimens

Test section	Maximum theoretical density [1]	Percentage of voids in pavement samples						Percentage of voids in laboratory compacted specimens	
		After rolling	After 1 year	After 2 years	After 4 years	After 10 years	After 19 years	By vibratory test	By 3,000 lbs./sq. in. double plunger
A-1	2.28	7.5	-------	-------	4.5	4.3	4.3	7.7	7.0
A-2	2.28	8.7	-------	-------	3.5	3.5	-------	7.4	5.6
B-1	2.27	[2] 6.6	7.0	7.0	6.1	4.8	5.7	6.8	7.3
B-2	2.29	10.0	9.6	9.1	9.1	7.8	5.6	11.0	9.1
C-1	2.27	9.2	7.4	7.0	6.1	4.4	-------	7.0	6.7
C-2	2.27	7.9	7.4	6.6	5.7	5.2	-------	6.8	6.4
C-3	2.29	10.1	7.4	7.8	7.4	6.5	-------	8.3	8.1
D-1	2.29	10.0	6.9	8.2	7.3	6.9	-------	8.2	8.6
D-2	2.27	7.9	6.1	5.7	5.7	5.7	2.6	5.9	7.3
D-3	2.29	10.9	8.2	7.9	8.2	7.3	3.9	9.5	7.8
Average	--------	8.9	7.5	7.4	6.4	5.6	4.4	7.9	7.4

[1] Based on specific gravities of asphalt and mineral aggregates.
[2] Results of cores sampled at age of 6 months.

Table 12.—Measured thicknesses of pavement surfaces at various times

Test section	Age of pavement, years	Thickness (inches) of sheet asphalt at distances from curb of—								Average thickness, inches
		4 feet	6 feet	8 feet	10 feet	12 feet	14 feet	16 feet	18 feet	
A-1	4½	1.20	1.45	1.60	1.80	1.65	1.60	1.57	1.47	[1] 1.57
	10½	1.19	1.25	1.50	1.75	1.62	1.62	1.57	1.37	[1] 1.54
	19½	------	------	------	------	1.25	1.00	1.13	1.00	[1] 1.10
A-2	4½	1.50	1.45	1.55	1.50	1.40	1.47	1.30	1.05	1.40
	10½	1.34	1.31	1.28	1.31	1.31	1.33	1.16	.98	1.24
B-1	4	1.55	1.60	1.60	1.60	1.32	1.35	1.32	1.40	1.47
	10	1.55	1.62	1.50	1.25	1.19	1.25	1.25	1.06	1.33
	19	1.38	1.25	1.06	.81	.62	.81	1.00	.75	.96
B-2	4	1.80	1.65	1.50	1.35	1.45	1.45	1.55	1.47	1.53
	10	1.75	1.40	1.50	1.50	1.19	1.19	1.37	1.31	1.40
	19	1.38	1.25	1.25	.81	.88	1.00	1.13	1.00	1.09
C-1	4	1.30	1.40	1.50	1.30	1.25	1.15	1.10	1.10	1.26
	10	1.28	1.44	1.38	1.34	1.00	1.00	1.00	.91	1.17
C-2 [2]	4	1.50	1.40	1.35	1.30	1.20	1.30	1.50	1.55	1.39
	10	1.38	1.31	1.13	1.25	1.28	1.25	1.50	1.50	1.33
D-1	4	1.55	1.60	1.70	1.50	1.65	1.55	1.57	1.45	1.57
	10	1.56	1.62	1.68	1.75	1.75	1.68	1.50	1.38	1.62
	19	[3] 1.98	1.75	1.56	1.44	1.06	1.13	1.25	1.31	1.36
D-2 [2]	4	1.62	1.70	1.70	1.65	1.75	1.65	1.60	1.30	1.62
	10	1.68	1.75	1.50	1.75	1.75	1.69	1.50	1.18	1.60
	19	1.50	1.50	1.50	1.75	1.56	1.50	1.44	1.00	1.46
D-3	4	1.25	1.40	1.40	1.35	1.50	1.45	1.32	1.30	1.37
	10	1.22	1.44	1.44	1.31	1.22	1.22	1.18	1.22	1.28
	19	1.06	1.13	1.19	.94	1.00	.94	1.00	.88	1.02

[1] Average for 12- to 18-foot distances from curb only.
[2] Center section distances are measured from west header.
[3] Not included in average.

Table 13.—Traffic volumes in the vicinity of test sections

Location of control station	Direction of traffic	Total vehicles, 24-hour period		Test sections affected
		1946	1956	
South of Tilden Street	Northbound	10,878	33,200	A-1, B-1
Do	Southbound	10,470	32,100	A-2, B-2
South of Albemarle Street	Northbound	11,381	28,200	C-1, C-2
Do	Southbound	10,974	33,300	C-2, C-3
South of Military Road	Northbound	6,634	23,400	D-1, D-2
Do	Southbound	6,080	26,400	D-2, D-3

Visual Inspection of Pavement

In general, the behavior of the pavement as a whole would be considered satisfactory although there was some indication of progressive deterioration. Visual inspection was made at the time of each sampling but no definite trends were noted. Figure 6 illustrates the general appearance of the test area in April 1955 when final samples were taken. Spalling and cracking of the sheet asphalt surfacing at some of the larger reflection cracks occurred relatively early and became more pronounced in later years as the asphalt hardened and the portland cement concrete base adjoining the joint progressively deteriorated. Figure 7 is an example of one of the more advanced cases of this deterioration. There was no tendency of the surfacing to displace or shove under traffic except at bus stops. Six bus stop areas were rebuilt in recent years with cement concrete pavement.

Sheet asphalt was unsatisfactory in the gutter areas. Figure 8 shows a typical condition after 10 years. This is in contrast to the good condition of the paving brick gutters used on other portions of the two contracts.

Moisture was observed on the surface of the cement concrete base and, to a lesser degree, in the binder course and lower levels of the sheet asphalt surfacing course when samples were taken. There was some indication of slight stripping in the binder but none in the sheet asphalt surfacing. It is believed that the stripping was insufficient to affect the behavior of the pavement except in the gutter area.

The occurrence of moisture between the cement concrete base and the bituminous surfacing after resurfacing old concrete is well known and is a matter of some concern. There is some uncertainty as to the manner by which this water accumulates, but it is believed to

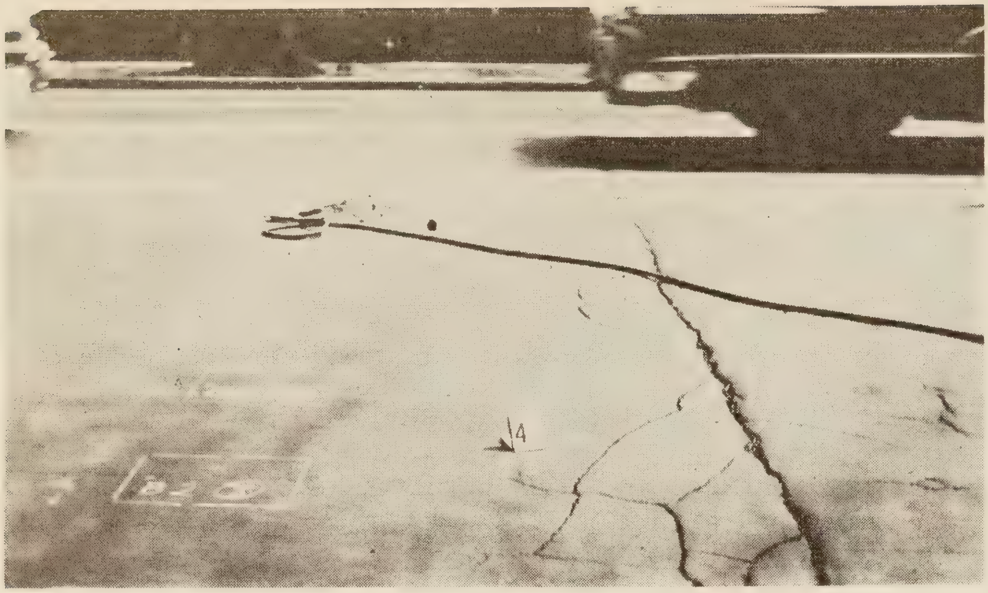

Figure 7.—View of section B–2 in April 1955, showing cracks at joint and over areas of base failure.

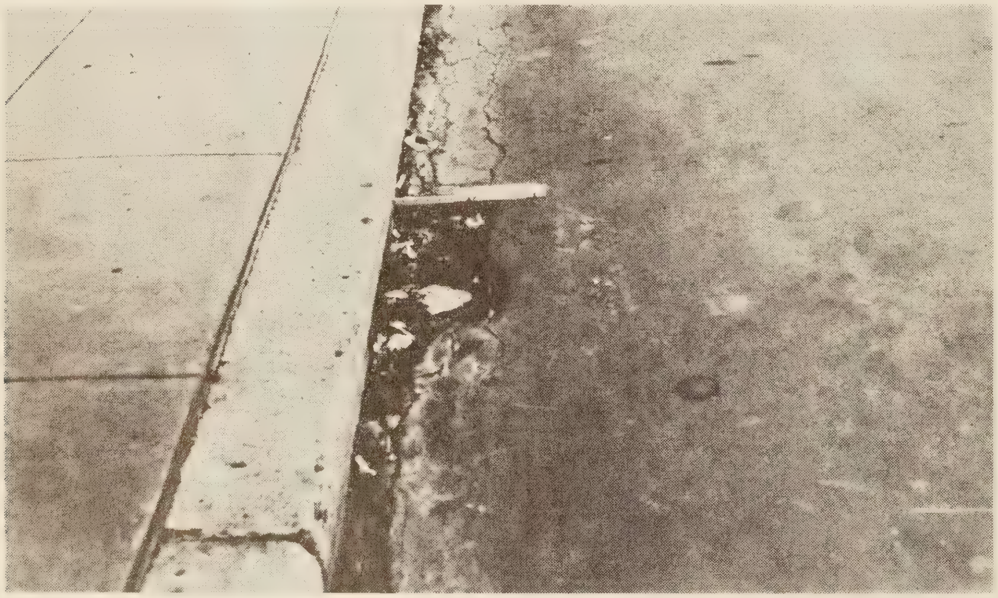

Figure 8.—View of sheet asphalt surface failure in June 1946 (south of section C–1) caused by surface water at the curb line.

be similar to the phenomenon that ofte occurs with impervious paint films on house when moisture diffuses through the walls fro the inside and collects under the film, resultir in blistering and peeling. In the case of tl pavement, the water vapor moves upwai through the sub-base and through the concre base, either directly or by capillary flo through the pores of the concrete durir periods when the surface is warmer than tl base. Because this vapor cannot readil escape through the surface, it remains trappe and will condense as moisture in the binde course or at the interface of the concrete an the asphaltic material.

REFERENCES

(1) Lewis, R. H., and Welborn, J. Y.

The properties of the residues of 50–60 an 85–100 penetration asphalts from oven tests an exposure. Public Roads, vol. 22, No. 2 Apr. 1941, pp. 27–46.

(2) Lewis, R. H., and Halstead, W. J.

Behavior of asphalts in thin-film oven test Public Roads, vol. 24, No 8, Apr.-May-Jun 1946, pp. 220–226.

(3) Pfeiffer, J. Ph., and Van Doormal P. M.

Classifying asphalts by means of penetratio index. National Petroleum News, Refinin Technology, Feb. 23, 1938.

(4) Lewis, R. H., and Welborn, J. Y.

The effect of characteristics of asphalts o physical properties of bituminous mixtures. Public Roads, vol. 25, No. 5, Sept. 1948 pp. 85–94.

(5) Pauls, J. T., and Goode, J. F.

A new vibratory machine for determining the compactability of aggregates. Public Roads, vol. 20, No. 3, May 1939, pp. 55–61.

(6) American Society for Testing Materials

Resistance to plastic flow of fine aggregate bituminous mixtures, D–1138–52. 1955 Book of ASTM Standards, Part 3, p. 1646.

Airphoto Analysis of Terrain for Highway Location Studies in Maine

By ERNEST G. STOECKELER, Airphoto Interpretation Specialist, University of Maine, and WILLIAM R. GORRILL, Soils Engineer, Maine State Highway Commission [1]

Airphoto interpretation techniques are especially useful for highway engineering terrain studies in wilderness areas where little or no detailed information on geology or soils is available. In Maine, time-consuming and expensive field reconnaissance surveys were reduced to a minimum by the intelligent use of aerial photography. Detailed field investigation and laboratory testing are still required to obtain information for final design purposes, especially in critical areas.

The Maine State Highway Commission has successfully employed airphoto interpretation techniques for obtaining a variety of information valuable in various phases of highway engineering. The four types of strip studies described in this article are only a few of many possible applications of this field. It is highly probable that more intensive and specialized photo interpretation studies will be made in the near future in Maine as well as throughout the nation.

THE Airphoto Interpretation Project at the University of Maine is financed jointly by the Maine State Highway Commission and the Bureau of Public Roads, U. S. Department of Commerce. The function of this project is to provide information on soils, drainage, and other natural and cultural features which might be useful in all phases of highway engineering from preliminary planning to construction operations.

Engineering soils, drainage, and materials maps have been prepared for an area of approximately 3,500 square miles. These maps were made by quadrangles, an area of approximately 200 square miles, at a scale of 2 inches equal 1 mile. Precedence was given to the areas where most construction was anticipated. These maps are especially useful in the preliminary planning phases of location studies and in the preparation of cost estimates.

In addition to these maps, a number of strip studies, using airphoto interpretation techniques, were accomplished. The main subject of this article is a discussion of various types of photo-strip studies and their applications in highway engineering.

Types of Strip Studies

Approximately 300 linear miles of new location and relocation strip studies were completed during 1957. Areas covered by these projects are shown in figure 1. Individual projects varied in length from 3 miles to over 100 miles. In practically all cases, aerial photography was performed specifically for the State Highway Commission. Photographic scales ranged from 1:5,400 to 1:20,000, depending on the type of study desired. Most of the photography was undertaken in the spring after snow had disappeared and before new foliage was developed. Photographs taken in the fall after deciduous foliage has been shed are also acceptable. The seasonal aspect is of special significance in Maine, because at least 90 percent of the areas studied to date were in densely wooded wilderness sites where terrain interpretation from summer photography is difficult because surficial features are often masked by the forest canopy.

These studies covered proposed routes of the Interstate highway system as well as a number of highway relocations. The type of data and methods of presentation varied somewhat for individual projects. The four major types of strip studies included (1) engineering soils, (2) drainage, (3) gravel haul, and (4) highway relocation and reconstruction.

The four types of strip studies were made in the reconnaissance phase as well as in the preliminary survey. Based on information furnished by these strip studies alternate routes were compared, cost estimates were prepared, and in some cases preliminary designs were made.

Engineering Soils Strip

The Maine reconnaissance engineering soils classification system was devised by the authors specifically for engineers. While this system is based on a geological approach, the nomenclature is given in engineering terminology. This system reduces the number of soil units to a minimum, yet provides the information required by local highway engineers. The following eight map units are tailored for Maine terrain and climatic conditions, and are adaptable to airphoto interpretation techniques which require a minimum amount of field checking.

- *Rock* (*R*).—Ledge within 5 feet of the surface.
- *Boulder Granular* (*BG*).—Granular drift.
- *Boulder* (*B*).—Boulder clay drift.
- *Granular* (*G*).—Sand or gravel.
- *Fines* (*F*).—Silt or clay.
- *Swamp* (*S*).—Surficial peat less than 3 feet thick.
- *Peat* (*P*).—Surficial peat more than 3 feet thick.
- *Water* (*W*).—Generally more than 2 feet deep.

Figures 2–4 are stereograms that illustrate these soil units.

For preliminary planning soils studies, 1:20,000 scale photography was used. In most cases a single 3-mile-wide strip centered on a preliminary line was provided for delineating soils areas. Occasionally two or three parallel strips were flown where more widely separated alternate routes were to be studied. The minimum size area delineated was approximately 5 acres, except for such elongated forms as esker ridges and narrow valley swamps.

In addition to soils delineations, annotations were made on the photographs emphasizing significant terrain features such as deep bogs, extensive rock outcrops, steep slopes, and other obstacles which should be avoided. Annotations depicting favorable terrain conditions were also indicated.

Figure 5 (p. 148) is an uncontrolled mosaic showing a 5-mile segment of the proposed 110-mile section of the Interstate highway system between Orono and Houlton, Maine. The soils information was transferred directly

[1] This article was presented at the 37th Annual Meeting of the Highway Research Board, Washington, D. C., January 1958.

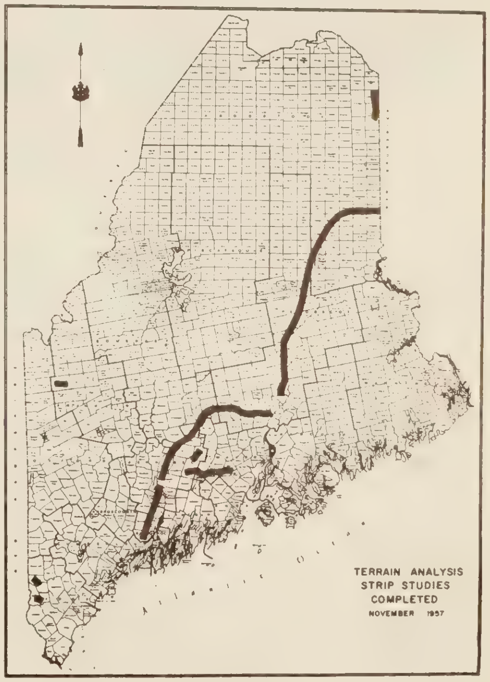

Figure 1.—Airphoto strip studies completed in 1957.

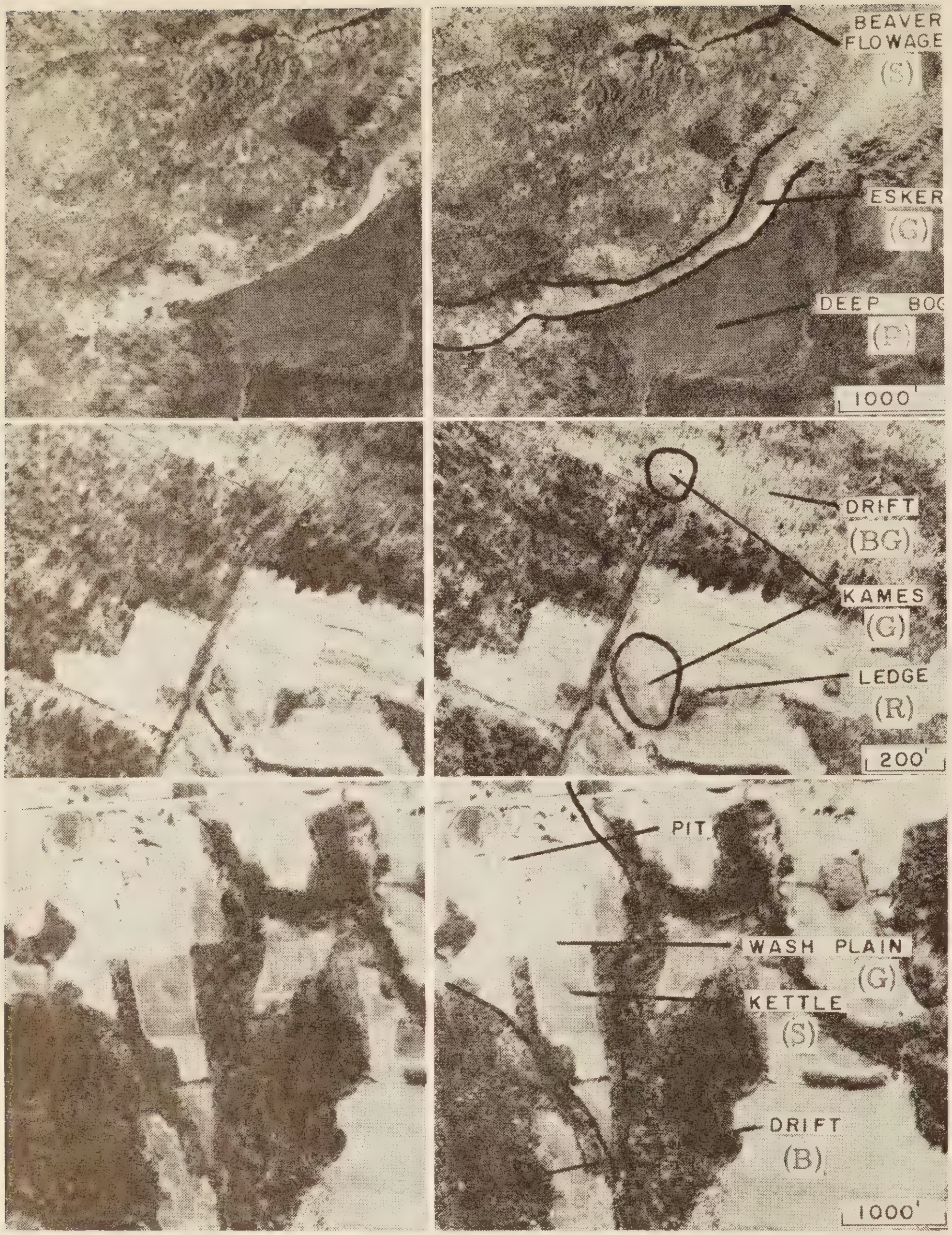

Figure 2.—Stereograms of several glacial landforms. (Courtesy of the Maine State Highway Commission.)

from the airphotos to a profile which had been prepared by photogrammetric methods. The P-line and stationing were placed on the photographs used for soils interpretation to facilitate the procedure of transferring soils data to the profile sheet as shown in figure 6.

With the engineer and the interpreter working together, the grade line was often adjusted to take advantage of terrain characteristics. For example, other conditions permitting, cuts were frequently increased at locations where ledge probably would not be encountered, especially where good granular excavation was anticipated. In areas where rock outcrops were numerous, the grade line was adjusted to minimize the amount of rock excavation. In addition to adjusting the grade line, the preliminary horizontal alinement was modified to avoid difficult terrain and also to take advantage of favorable construction areas. This information was used for the preparation of a preliminary cost estimate of the Interstate highway system.

Another more detailed engineering soils strip study was made of a 30-mile portion of the Interstate highway system in a more advanced planning stage. Airphotos for this study were taken in the spring at a scale of 1:5,400. A private photogrammetric concern was engaged to prepare topographic maps of a 3,200-foot-wide strip at a scale of 1 inch equals 200 feet and a contour interval of 5 feet. For this project the soils information was transferred from the photographs to the topographic sheet, thus enabling the location engineer to evaluate the engineering and economic aspects of a number of possible road lines within the 3,200-foot-wide strip. After detailed analyses of a number of possible lines were made, a centerline was selected and photogrammetric methods were used to determine cross sections.

With large scale photography of this type the interpreter can delineate small problem areas such as rock outcrops and swamps of less than one-tenth acre. In boulder-strewn glacial deposits, and even in wooded areas, it is often possible to measure individual boulders for the purpose of estimating the quantity of ledge-size boulders (larger than 1 cubic yard) contained in the deposit. Areas where seepage, settlement, and frost action might occur were also annotated on the photographs.

The detailed engineering soils map and annotated photographs were then provided to field geologists for detailed soils investigation. Suggested probe and drill-hole locations were pinpointed on the photographs to assist the geologist in determining actual soil type boundaries. Additional explorations were made at the discretion of the geologist in order to obtain an adequate soils profile. After an evaluation was made of the field investigation and laboratory analysis of the samples, the soils map was modified and extended to include detailed information on critical areas. Frequently the field work was expedited by trails and access roads indicated on the photographs. By following this procedure, a maximum amount of soils and terrain data was obtained with the minimum expenditure of manpower, time, and money.

Drainage Strip Studies

In Maine, as elsewhere, the design of drainage structures in relocation projects is often based on observations of the past performance of existing structures not far distant from the proposed new location. If the existing structure proved adequate over a long period of years a comparable design for the nearby location is used. However, for most of the proposed Interstate highway system routes and many major highway relocations, this method of design cannot be employed because often no existing nearby structures are available for evaluation. The U. S. Geological Survey topographic sheets having a scale of 1 inch equals 1 mile and a contour interval of 20 feet are often inadequate for determining watershed areas, especially small watersheds of less than 1,000 acres. In addition, many quadrangle sheets in Maine were compiled over 20 years ago and some before World War I. Because of these inadequacies, highway engineers have recently exhibited considerable interest in photohydrological studies.

The suitability of photography for drainage interpretation studies is dependent principally on scale and season of photography. In densely wooded areas the seasonal aspect is probably of prime importance. In summer photography, many drainageways and minor relief breaks are hidden by the forest canopy. In these cases the interpretation of vegetation provides the only clue to the existence of a drainageway. Due to climatic conditions in Maine, photographs taken during the spring break-up period just after the snow has disappeared are excellent for drainage interpretation purposes. During this period every little drainageway and depression is filled with water, and the photographic patterns are very easy to interpret. Even in dense forests con-

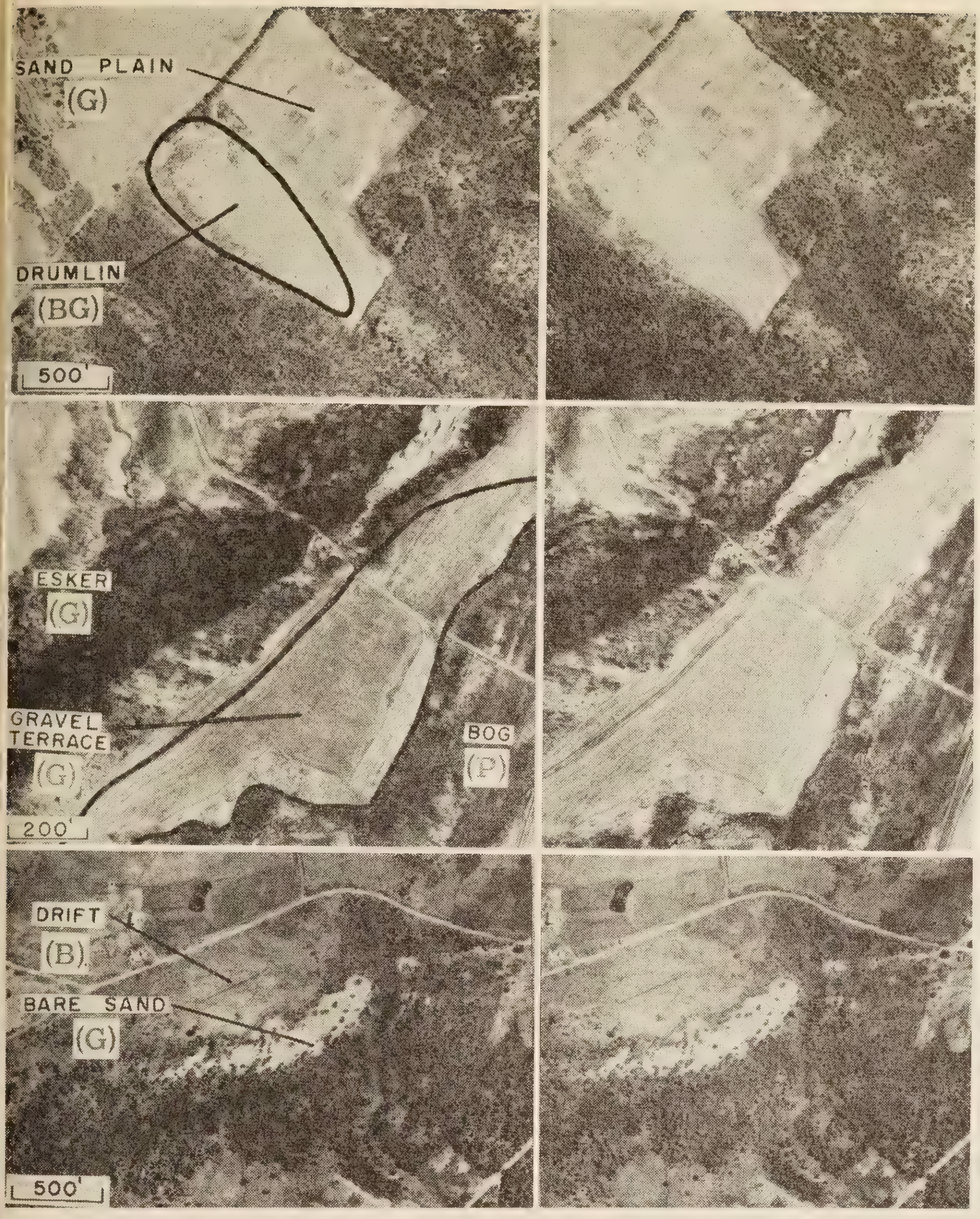

Figure 3.—Stereograms of glacial, water-lain and wind-blown deposits. (Upper and lower panels, courtesy of the Sewall Company, Old Town, Maine; middle panels, courtesy of the Maine State Highway Commission.)

taining 75 percent coniferous trees, there are enough canopy breaks formed by the leafless deciduous trees to permit the interpreter to see a sufficient amount of the earth's surface so that even small drainageways can be traced.

For preliminary reconnaissance drainage surveys, photography having a scale of 1:20,000 is adequate. Figure 7 is an uncontrolled mosaic of a drainage study made along a portion of a proposed Interstate highway system route. Many smaller drainageways and minor ridge lines delineated on the original 1:20,000 photographs were omitted in this illustration for reproduction purposes. Drainage strip studies of this type were made on approximately 240 linear miles of the Interstate highway system in Maine.

Area, slope, and cover-type information for each individual watershed was provided to the highway engineer who calculated the preliminary drainage structure sizes. This information was then transferred to the profile using the same procedure described previously (see fig. 6). In addition, information on nearby drainage structures, the existence of water storage areas in individual watersheds, possible seepage areas, and similar hydrological data were noted on the profile sheet. This information was used for the preparation of preliminary cost estimates.

For more detailed drainage studies, photographs having a scale of 1:5,000 to 1:10,000 are preferred. In densely wooded areas, especially on relatively flat terrain, small drainageways are difficult to identify on 1:20,000 photography. Ridge lines can also be plotted more accurately on large scale photography.

Gravel Haul Strip

Another type of airphoto strip study made in connection with the location of the Interstate highway system was an analysis of the availability and accessibility of granular materials along a 110-mile section. Much of the proposed route was located in a wilderness area several miles from existing roads. The purpose of the study was to provide a basis for estimating the amount of overhaul which could be anticipated at various locations on this large project. In Maine, the maximum free-haul distance is 2 miles.

An engineering soils strip study of this area was completed previously. Granular deposits delineated on these photographs were rapidly restudied to obtain the overall view of the entire length of the 110-mile section. The section was then subdivided into 8 projects to facilitate study of shorter sections for analysis purposes. Arbitrary separation points between projects were based on an evaluation of a number of factors including (1) major stream crossings, (2) number, spacing, and location of potential gravel deposits in the particular area, (3) location of towns and major highway crossings where entrance lanes would probably be required, and (4) general characteristics of the terrain.

Figure 8 (p. 150) is an uncontrolled mosaic of a 5-mile section showing the preliminary line stationing, and possible access routes from the delineated deposit to the project. Figure 9, a stereogram detail of a portion of figure 8, illustrates the type of information which is presented in the gravel haul report.

As seen in table 1, the haul distance was indicated by the following road types:

Improved 2-lane road.—Paved or all-weather gravel roads permitting two-way traffic; no improvements required.

Improved 1-lane road.—Narrow, gravel-surfaced roads, such as country lanes and main haul logging roads which may have turnouts at irregular intervals, likely found in a very soft and inoperable condition during the spring break-up period; improvements to varying degrees should be expected.

Unimproved road.—Single lane roads, either dirt or with a thin gravel surfacing; considerable improvements even for one-way traffic are definitely required.

Road required (*new road*).—Access road from deposit to project is nonexistent or the need for a more direct route is indicated. For construction through wooded areas the letter *W* is appended to the distance, and the letter *C* is added if the area is cleared or brush covered. Where streams must be crossed, the width of the stream in feet is tabulated.

Volume estimates shown in table 1 are for areas designated on the original photography. Where long continuous eskers were found near the preliminary line, select 2,000- to 3,000-foot portions of the deposit were outlined and the volume was appended with a plus sign, which means that additional materials could be obtained by extending the indicated boundaries. In cases where the distance between deposits was rather long, a number of access roads to the project were plotted, especially if a large amount of material was available. In plotting proposed access routes, downhill hauls were chosen and unfavorable terrain was avoided where

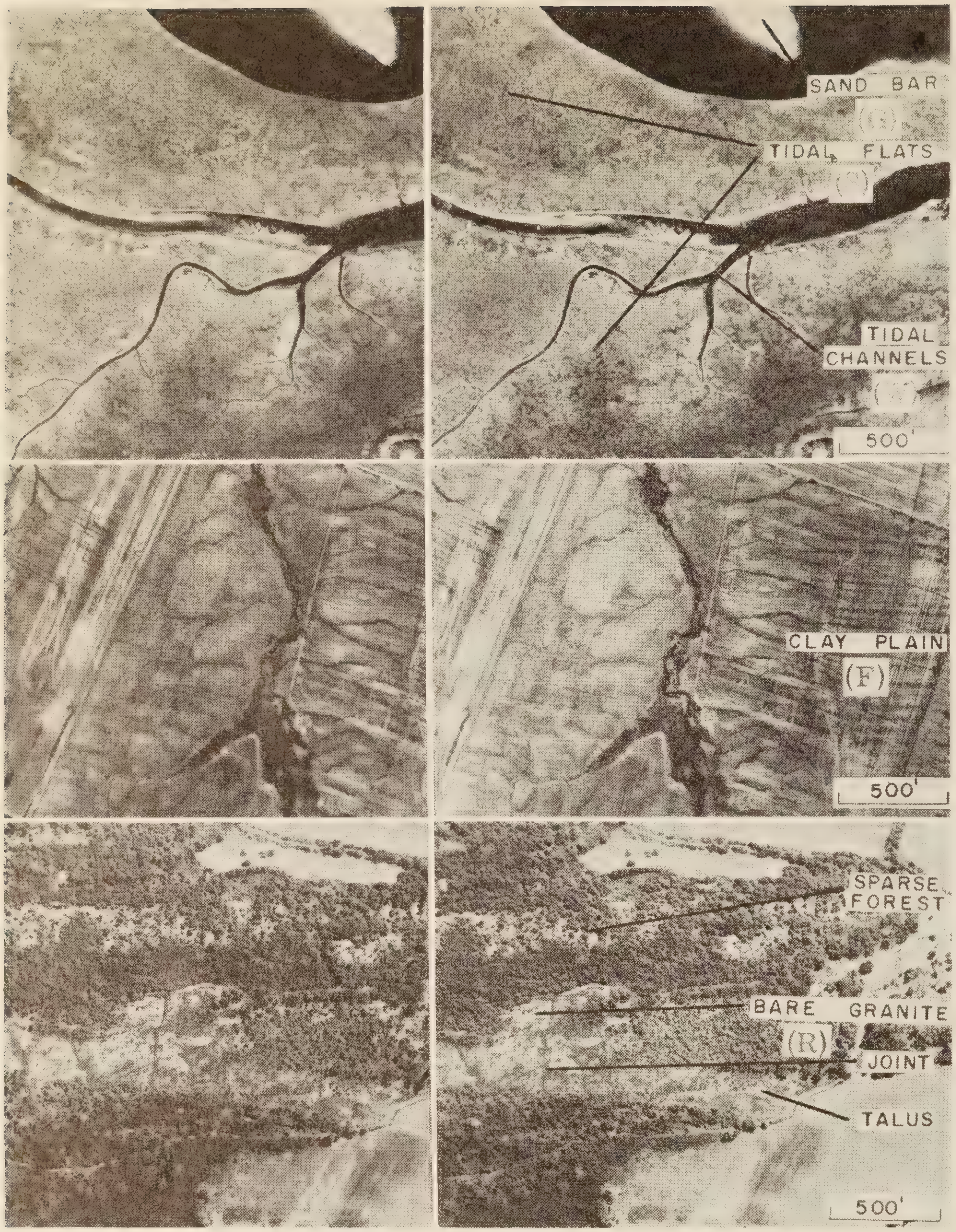

Figure 4.—Stereograms of fine-grained marine and lacustrine deposits and ledge terrain. (Upper and lower panels, courtesy of Maine Cooperative Wildlife Research Unit; middle panels, courtesy of the Maine State Highway Commission.)

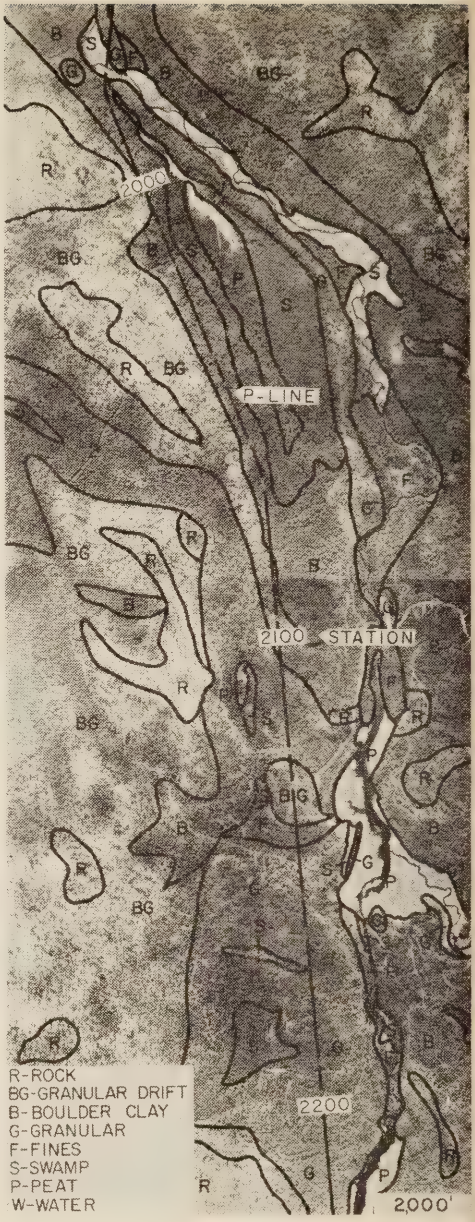

Figure 5.—Uncontrolled mosaic showing engineering soils areas (courtesy of the Maine State Highway Commission); stations were placed at 1,000-foot intervals on the originals.

possible. Existing roads were utilized where feasible. In figures 8 and 9, note that suggested routes cross the narrowest portions of the extensive swampy area found between the deposits and the preliminary line.

A brief description of each project was also included in the gravel haul report. The type of information presented is illustrated by the following quotation from the actual report pertaining to the area shown in figure 8.

"Project 4.—Canadian Pacific R. R.–Medway (Station 2047 to 2813). There are no public roads of any type in this 14.5-mile section except in the immediate vicinity of Medway. A 1-lane main haul logging road which is operable about 11 months per year crosses the preliminary line at several points. In the area immediately north of the railroad there are numerous secondary logging roads and sled trails which are generally too winding to be of much value as access roads. A logging camp, located at Station 2103, 2,800 feet left of centerline, is the only building within a mile of the preliminary line in the 25-mile section between Howland and Medway. The most promising development possibility appears to be Deposit 26 located at the midpoint of the project. Because of the probable dearth of granular materials between Stations 2400 and 2800, a number of access roads are indicated from Deposits 26 and 28 to various points on the project. In several instances it would be necessary to span the East Branch of Medunkeunk Stream which is about 30 feet wide. Most feasible crossings are at Stations 2516, 2618, and 2635. At other points it would be necessary to traverse swampy terrain varying in width from 250 to 1,000 feet. Deposit 29 is . . . ."

Approximately 90 deposits located in the vicinity of the centerline of this 110-mile section of the Interstate highway system were described in the gravel haul report. An analysis of this information indicated that less than 20 percent of the total mileage would probably involve overhaul charges. This information was incorporated in preliminary cost estimates.

In addition to the gravel haul analysis, a number of gravel prospect spot studies were completed during 1957. These studies, not shown in figure 1, usually covered an area of approximately 100 square miles centered on a construction project located in a quadrangle where materials maps have not been prepared to date. Any available photography having scales ranging from 1:3,600 to 1:70,000 was used for interpretation. Considerable photographic coverage of Maine is already in the possession of the State Highway Commission and the University of Maine Photo Interpretation Project. Figures 10 and 11 are reductions of photographs having original scales of 1:70,000 and 1:40,000, respectively, showing extensive gravel deposits on single photographs covering vast areas. It is estimated that over 30-million cubic yards of granular material are in the area shown in figure 10, which covers approximately 80 square miles. Very small scale photographs are useful for reconnaissance studies of extensive areas. Multiple coverage at scales of 1:20,000 and 1:5,000 is ideal for interpretation purposes.

Potential granular deposits were delineated on the photographs, and suggested spots for field checks were pinpointed. Access to potential deposits was plotted on the photographs. In areas devoid of easily recognizable cultural features, conspicuous natural features such as rock outcrops, local forest clearings, and creeks were annotated on the photographs. Distances and compass bearings were also indicated for the convenience of the field checkers. Identification numbers were assigned to each potential deposit on both the photograph and a U. S. Geological Survey topographic sheet.

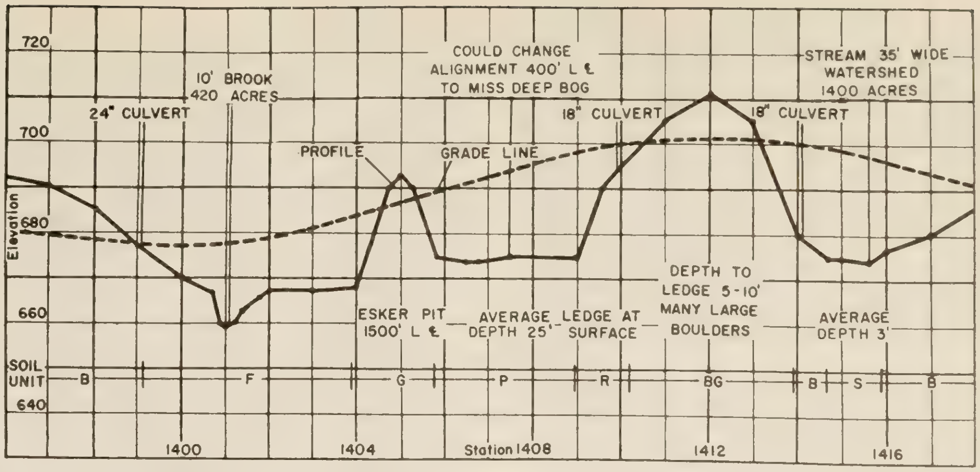

Figure 6.—Profile with engineering soils and drainage annotations.

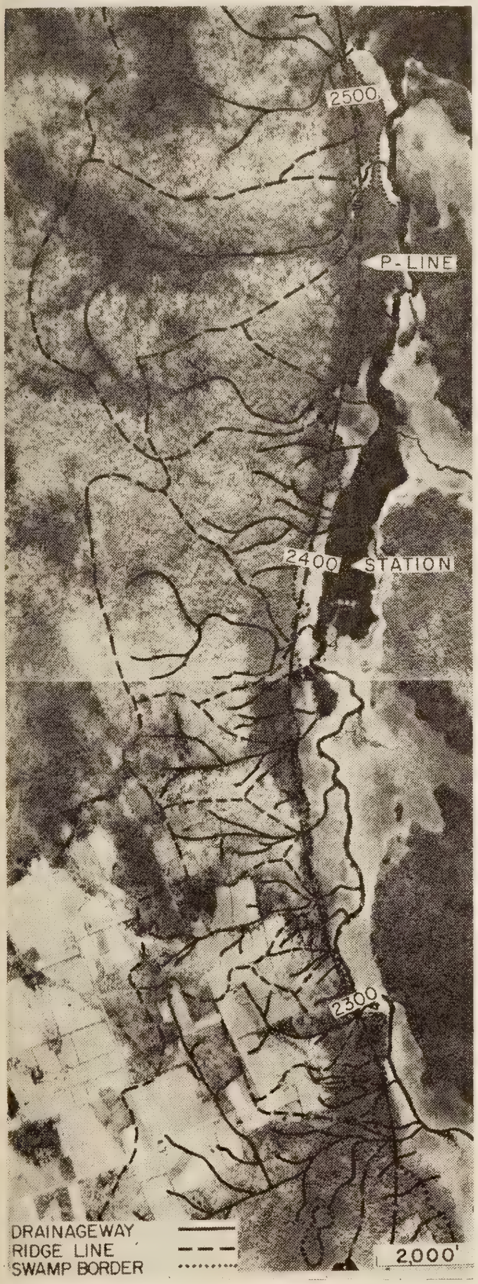

Figure 7.—Uncontrolled mosaic of drainage delineations (courtesy of the Maine State Highway Commission); watershed areas were obtained directly from the photographs.

The annotated photographs and maps were furnished to the field geologists who determined actual field conditions. Volume estimates were prepared and samples were taken for laboratory tests. This information was made available to bidders.

Relocation and Reconstruction Strips

Terrain analyses were made of six proposed relocation projects scheduled by the State Highway Secondary Division. These strip studies varied in length from 3 to 25 miles. Several of the longer strips were flown for the Highway Commission at a scale of 1:12,000. For the shorter studies the best available photography was used.

These studies required close cooperation between the secondary highway engineer and the interpreter. Because of many years' experience in observing the behavior of a particular stretch of road, the engineer was intimately acquainted with practically every bad curve, dip, frost boil, and excessive grade for the entire length of the proposed construction project. He was also well informed on future planning which might influence relocation, traffic counts, local needs and similar important information which cannot be gleaned from the photographs by the interpreter. Consequently, the first stage of relocation studies consisted of the engineer's review of the entire length of the project with the interpreter. The engineer indicated general segments of the road where relocation or improvements of one type or another were anticipated. These locations were noted on the photographs by the interpreter. Pertinent information directly or indirectly affecting the location of the new road was noted directly on the photographs; for example, "feeder road will be relocated in 5 years so the PC cannot be north of this point," or "centerline should not be more than 1,000 feet east of this intersection."

After the interpreter was thoroughly briefed by the engineer, soils areas were delineated in a band of varying width which would amply cover any possible relocation line. Conspicuous natural features such as swamps and rock outcrops were annotated. Significant cultural features such as houses, stores, cemeteries, schools, churches, gas stations, feeder roads, and trails were traced directly from the photograph, using the central portion of the photograph to minimize distortion. Strip tracings were made in 4- to 6-mile sections, or 20 to 30 inches per drawing at a scale of 1:12,000. After the tracing of features for the entire strip was completed, the project was subdivided into shorter sections to facilitate analysis. Depending on terrain and road characteristics, the subsections varied in length from 2,000 to 15,000 feet.

Table 1.—Gravel haul data sheet

Deposit number	Volume, cu. yd.	Access at station number	Haul distance (miles)				
			Improved 2-lane road	Improved 1-lane road	Unimproved road	New road [1]	Total
26	300,000+	2435	-------	0.5	-------	-------	0.5
26A	300,000+	2466	-------	1.2	0.6	-------	1.8
26B	300,000+	2495	-------	1.9	.8	-------	2.7
26C	300,000+	2539	-------	1.9	.9	0.1 W	2.9
27	200,000+	2516	-------	-------	-------	[2].7 W	.7
28	200,000+	2618	-------	-------	1.9	[2].2 W	2.1
28A	200,000+	2635	-------	-------	2.2	[2].1 W	2.3
28B	200,000+	2682	-------	-------	3.3	.1 W	3.4
28C	200,000+	2702	-------	-------	3.7	.1 W	3.8

[1] W indicates wooded land. [2] 30-foot bridges required.

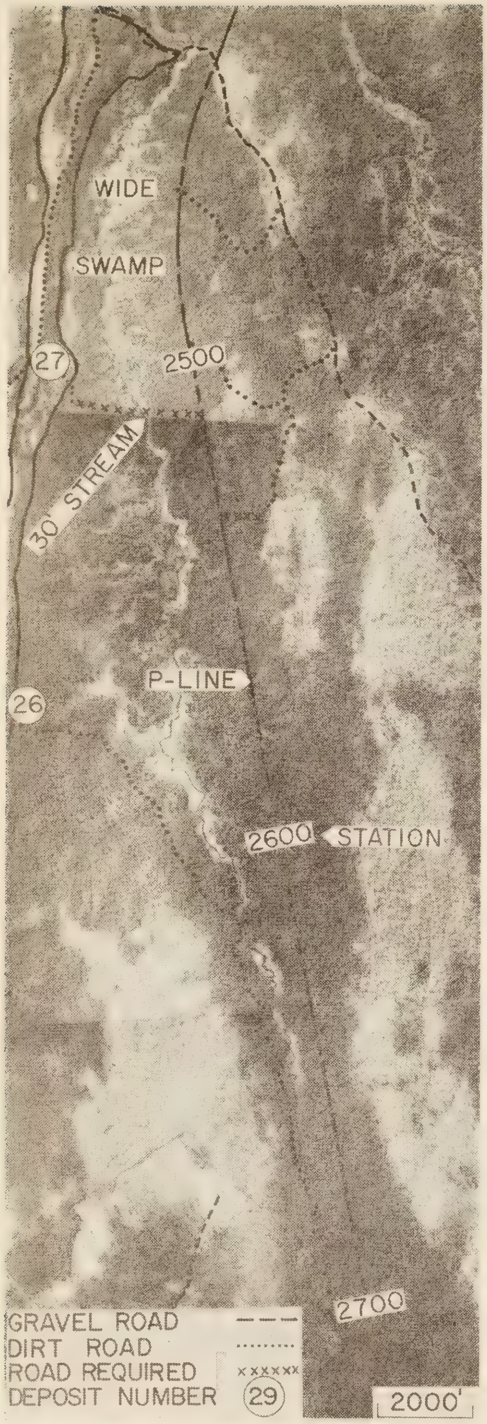

Figure 8.—Uncontrolled mosaic of a gravel haul study. (Courtesy of the Maine State Highway Commission.)

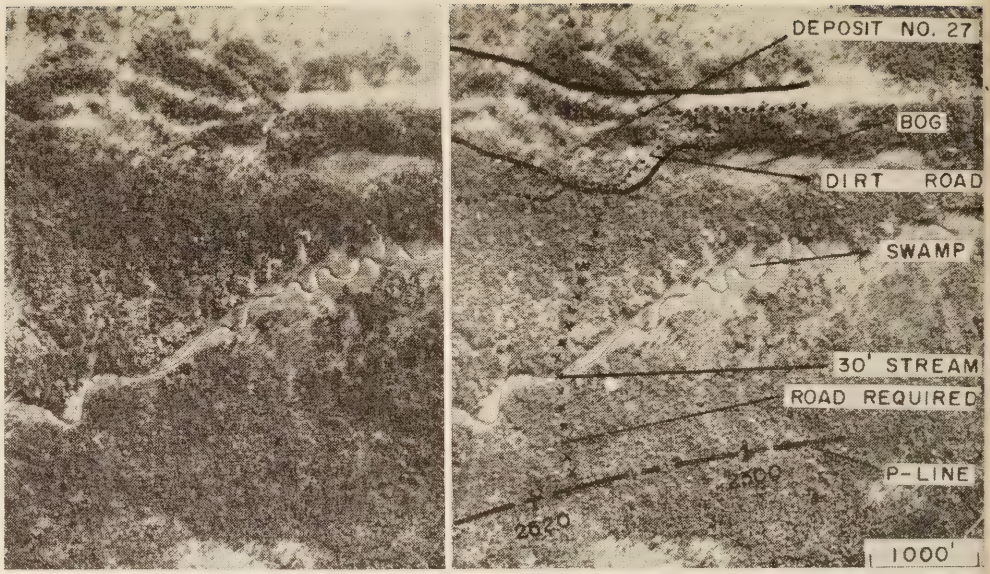

Figure 9.—Stereogram of a portion of the area shown in figure 8. (Courtesy of the Maine State Highway Commission.)

Figure 10.—Esker system in the Penobscot Valley, Maine. (Courtesy of the U. S. Army Map Service.)

The photographs were then restudied in detail to analyze natural and cultural features, and an evaluation of each subsection was made. Instead of plotting a suggested preliminary line and perhaps one or two alternates, a recommended band for detailed study was outlined on the tracing. The intent of this approach was to establish boundaries for a strip which offered the least construction difficulties by avoiding unfavorable terrain where possible. The width of the recommended band ranged from 50 to over 500 feet, depending principally on the length of the cutoff, terrain conditions, and the location of cultural features. This allowed the location engineer considerable leeway in plotting various grade-alinement design combinations to meet the road requirements, still keeping within the recommended band. Alternate bands were given if feasible. In instances where the improvement amounted to a simple road widening, a single line was indicated in

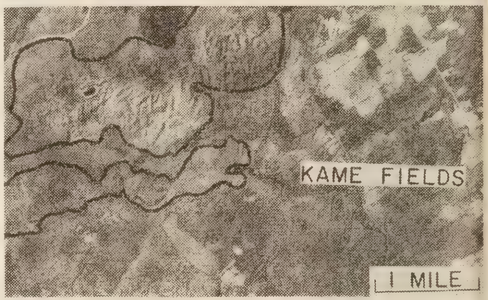

Figure 11.—Vertical photograph of extensive kame fields in western Maine. (Courtesy of U. S. Geological Survey.)

(Continued on next page)

lieu of a band. Spot elevations at major topographic breaks within the recommended band were pinpointed on the tracing. Locations of existing gravel pits and also potential deposits were annotated. The engineer was provided with ozalid prints, transparent overlays, and overlapping airphotos.

A brief verbal description of each subsection was included in relocation study reports. To illustrate, the following is quoted from a report describing the subsection shown in figure 12. "Two recommended bands for detail study are indicated at the Palermo cutoff. The first, labeled B–1, consists of a series of three widenings and one 2,500-foot cutoff located 300 feet south of the main village. This band traverses two granular soils areas which may yield suitable borrow. Band B–2, a wide sweeping curve located 1,000 feet south of the village, would require 8,000 feet of new construction, of which 400 feet is through a shallow swamp and about 3,000 feet is over ledge terrain. The southern edge of the band skirts a very deep bog and the northern boundary is limited by a cemetery. Based on terrain considerations only, the northern portion of Band B–2 is preferred. A wash plain containing an operable pit is located less than 2 miles from the west end of the project . . ."

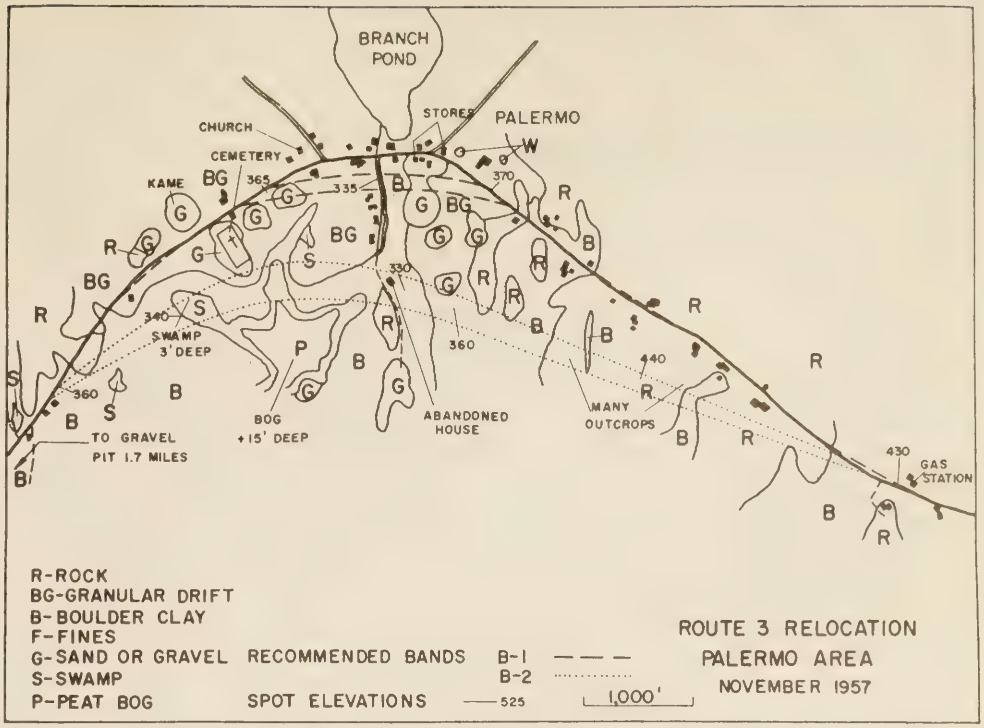

Figure 12.—Secondary highway relocation study showing recommended bands for detailed study.

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1947 (out of print).	1951, 60 cents.	1955, $1.00.
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