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CONSTRUCTING CIRCULAR TRACK FOR BITUMINOUS STABILITY TESTS

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions

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STABILITY EXPERIMENTS ON ASPHALTIC PAVING MIXTURES

Reported by W. J. EMMONS, Associate Professor of Highway Engineering, University of Michigan[1]

IN ORDER to provide a background of service data for the development of laboratory strength tests for bituminous pavement surface mixtures, the Bureau of Public Roads built experimental sections of a wide range of compositions and observed their behavior under controlled traffic. These sections were constructed as a circular roadway 180 feet in mean diameter. The foundation was reinforced concrete. The test surfaces were 13 feet wide between an integral curb, 2 inches in height, at the inner edge and a concentric circle of experimental concrete sections which served as the outer curb. The roadway sloped uniformly from the outer to the inner curb with a drop of eight inches. The concrete foundation was finished to a high degree of smoothness, as it was desired to accelerate the tests, and foundation roughness is quite generally believed to assist in the resistance to displacement of the bituminous surface.

TWO SERIES OF TESTS CONDUCTED

Test sections were constructed and tested in two series. The first series was composed of 27 sections of asphaltic concrete and the second of 28 sections of sheet asphalt and 5 sections of asphaltic concrete. The surface course in all cases approximated 2 inches in depth. The mixtures were prepared at local paving plants and laid by contractors' forces. Surfaces were compressed with an 8-ton tandem roller. Due to the narrow width of the pavement, cross rolling was not possible. Delivery and placing of the mixtures was not always continuous, and for this reason the rolling was probably not uniform over the various sections. This may have resulted in lower densities in some instances, than would have been obtained in normal pavement construction. As far as possible, however, the procedure in construction was in accordance with current practice.

As the tests progressed, measurements were made on the surface mixtures as follows:

(1) The longitudinal displacement under traffic.
(2) The internal displacement at various depths.
(3) The temperature attained under prevailing climatic conditions.

ALL SECTIONS REMAINED STABLE UNDER WINTER TEMPERATURES

The sections were marked into thirds, with radial lines painted on the surface for use in measuring longitudinal displacement. These lines are shown in figure 2, and can be distinguished from the broader lines which bound the sections by the width of line and also by the two short lines normal to them near the inner curb. At each end of these radial lines, permanent metal reference plugs were set in the concrete. Wood screws, ⅝ inch in length were driven at 6-inch intervals along these lines. Before the tests were started, the position of each screw was recorded with reference to a wire stretched between the permanent plugs at the ends. Subsequently, as the tests progressed, the locations of the screws were similarly determined.

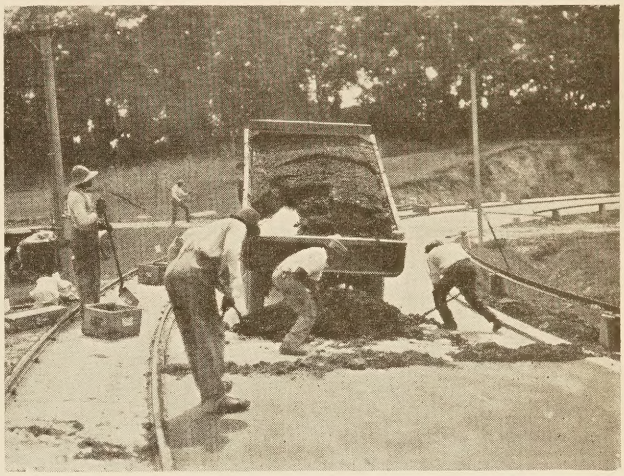

FIGURE 1.—LAYING SURFACE MIXTURE FOR STABILITY EXPERIMENTS.

The internal movement of the surface mixture was studied by observing the movement of short sections of ¼-inch brass rod set one above the other and with a short brass rivet on top. Four such installations were made on each section in the center of the strip over which traffic was to be concentrated. They were located 2 feet from the radial lines and were referenced to the curb and to the wire which was stretched along the radial lines to measure longitudinal displacement.

Internal temperature of the pavement, was measured with thermo couples installed in several of the sections of the first series of tests. The scope of this phase of the investigation was considerably amplified in the second series. In general, temperatures were observed at 10 a.m. and 2 p.m., but some special work was done to investigate temperatures during a 24-hour period. It was found that asphaltic mixtures are more liable to displacement under traffic during periods when high temperatures prevail. Many mixtures proved to be stable throughout the tests. Those which shoved badly under summer conditions were highly resistant to displacement during the cooler months. Under traffic of the type imposed, no mixture, however poorly constituted, was appreciably displaced during periods when the average air temperature was lower than 70° F. The results of the study of pavement temperatures have already been reported in detail.[2]

HEAVY TRAFFIC IMPOSED ON TEST SECTIONS

Loaded trucks equipped with solid rubber tires constituted the traffic. They traveled always in the same direction, and the inner wheels were kept in a path 2½ feet wide, as indicated by short longitudinal lines painted on the pavement. The speed was held as closely as possible to 12 miles per hour. In the first series of tests a 3-ton truck having a total weight of 15,600 pounds, and a rear wheel load of approximately

[1] Highway research specialist, Bureau of Public Roads.

[2] Temperature as a Factor in the Stability of Asphaltic Pavements, Public Roads, vol. 7, no. 2, April 1926.

5,700 pounds was used at the beginning, but later another truck was added to accelerate the test. This second unit was a 5-ton truck, loaded to capacity, with a rear wheel load of about 7,900 pounds. Approximately 50,000 passages over the pavements were made between the start of the tests during the month of October and its discontinuance about the middle of the following August.

Only one truck was used in the second series of tests. The total weight was 17,800 pounds with a rear wheel load of 6,800 pounds. This truck made 64,000 trips over the pavements during two periods from August 28 to November 11, and from May 5 to October 15 of the following year. As the first series had shown that no displacement was to be expected during the cold months, traffic was suspended over the winter in this test. The west side of the roadway was located in a cut, the top of which, some 25 feet back, was fringed with trees and shrubs. These cast a shadow over some of the sections during the latter part of the day. In late September, this began about 1 p.m. on parts of sections 32 and 33. By 4:15, when traffic was discontinued for the day, sections 1, 2, 3, 4, 5, 7, 28, 29, 30, 31, 32, and 33 were either entirely shaded or merely spotted with sunlight which filtered through the foliage. The stability of asphalt mixtures is reduced as their temperature is increased, and it was determined in this investigation that at a given time the temperature of areas exposed to direct sunshine was higher than that of shaded areas. Therefore, it is true that certain of these mixtures were tested under slightly different conditions than the remainder for a part of each clear day. It is impossible to evaluate the effects of these temperature variations, but it is believed that they were not great enough to influence the behavior of the mixtures materially.

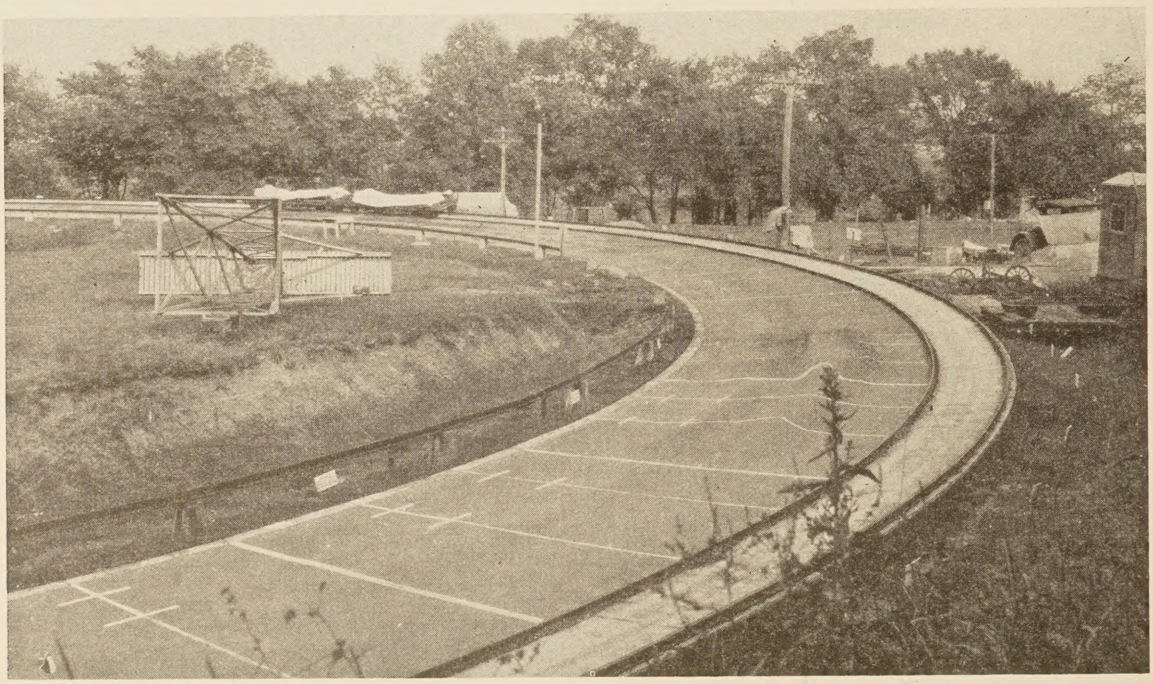

Figure 2.—Second Series of Stability Tests, Showing Section Lines and Effect of Traffic on Test Sections.

FIRST SERIES OF STABILITY EXPERIMENTS DESCRIBED

In the first series of tests, the circular roadway was surfaced with 27 different compositions of asphaltic concrete. The intended variables were the relative percentages of fine and coarse aggregates, consistency of the asphalt cement, gradation of coarse aggregate and gradation of fine aggregate. Trap rock, graded from 1¼ inches to ¼ inch, Potomac River sand, and limestone dust were used respectively, as the coarse aggregate, the fine aggregate, and the filler in these mixtures.

At the time this series of sections was constructed, no paving plant equipped with weighing apparatus was available. All of the materials, even the asphalt cement, were proportioned by volume. This practice precluded strict adherence to the schedule of mixture compositions and probably also resulted in a lack of uniformity between the several batches composing each section. A number of mixtures, containing high percentages of coarse aggregate, gave compressed pavement surfaces of very open texture and a seal coat of asphalt cement and stone chips was applied to all sections. Table 1 shows the analyses of samples taken at the paving plant and the movement of the reference screw which displaced to the greatest extent under traffic. In table 2 the mixtures are grouped according to the consistency of the asphalt and the amount of coarse aggregate and the approximate gradation of the fine and coarse aggregates are shown.

RESULTS OF FIRST TEST NOT CONCLUSIVE

Most of the mixtures were satisfactory under traffic, the maximum displacement of any reference screw in one third of the sections being less than 1 inch. This is a negligible deformation which may merely reflect internal readjustment of the mixtures due to added

TABLE 1.—*Analyses of plant samples (percentages by weight) and maximum displacements of asphaltic concrete mixtures of first series of tests*

Mix no.	Bitumen	Retained on 1¼ inch	Passing and retained on next smaller size: 1¼ inch	1 inch	¾ inch	½ inch	¼ inch	No. 10	No. 40	No. 80	No. 200	Maximum screw movement
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Inches*
1 [1]	4.03	3.6	9.2	15.0	10.6	21.6	11.0	8.4	6.1	5.4	5.1	92.0
1A [1]	4.67	------	3.4	8.5	19.5	29.5	9.7	10.2	6.3	3.9	4.3	(92.0)
2	5.15	------	1.6	4.8	30.9	24.0	8.5	8.5	6.0	5.6	5.0	62.5
3	4.83	------	5.2	12.9	21.5	23.8	8.4	7.6	6.7	4.7	4.4	1.4
4	3.95	2.0	11.2	17.7	16.4	17.7	8.5	7.4	6.1	5.0	4.1	56.5
5	4.44	------	11.1	15.8	11.1	18.9	8.1	12.1	8.7	6.0	3.7	24.2
6	4.59	2.1	10.5	12.1	14.6	17.7	8.2	10.4	9.3	6.6	3.8	42.5
7	5.06	4.0	7.0	12.6	10.9	19.1	10.5	12.5	9.3	5.7	3.3	23.8
8	5.41	------	10.2	3.6	13.8	22.3	10.8	14.4	9.8	6.1	3.6	1.8
9	5.69	5.0	7.1	10.0	12.3	16.2	5.4	14.8	12.2	7.6	3.7	6.4
10	5.97	------	4.1	13.5	11.5	19.5	6.8	14.7	12.2	7.7	4.0	4.6
11 [1]	6.28	2.2	6.2	6.8	13.4	15.5	7.5	16.4	12.2	7.4	6.1	9.1
11A [1]	6.69	------	6.4	6.9	12.0	20.0	7.5	15.2	12.7	7.8	4.8	(9.1)
12	6.03	------	6.1	9.2	12.7	15.3	6.4	15.6	14.7	9.6	4.4	.5
13	5.60	------	7.2	7.2	11.0	8.6	8.8	24.3	14.4	7.6	5.3	1.1
14	6.02	------	2.2	7.3	11.9	13.2	8.1	19.2	17.0	10.3	4.8	.4
15	7.43	------	------	1.8	9.2	14.1	8.4	20.6	21.8	11.7	5.0	2.4
16	6.32	------	4.6	5.4	9.7	10.4	9.5	22.8	16.8	9.4	5.1	.3
17	5.04	7.0	5.7	12.4	9.3	13.6	11.0	15.1	9.6	6.6	4.7	.3
18	5.36	5.5	11.3	3.6	13.0	13.5	11.4	14.7	10.5	7.0	4.1	1.1
19	5.05	6.9	10.0	10.8	9.7	13.4	10.7	13.9	10.1	6.3	3.2	.7
20	6.31	------	6.6	8.7	11.6	16.6	11.4	17.7	11.6	6.3	3.2	2.5
21	6.17	------	6.2	12.2	9.4	15.1	6.8	15.6	14.6	9.5	4.4	1.1
22	6.20	------	8.8	5.3	11.5	15.0	8.2	18.1	14.4	8.8	3.7	.9
23	5.78	------	8.7	9.8	11.8	14.3	6.5	16.2	13.6	8.5	4.8	.5
24	4.92	------	4.2	15.2	13.4	10.0	9.3	20.5	15.1	4.4	3.0	.4
25	5.03	4.4	7.4	11.4	9.7	12.5	11.7	18.1	12.8	4.1	2.9	.2
26	7.54	4.0	2.9	4.0	11.8	14.5	8.7	10.9	13.8	12.5	9.4	2.9
27	7.28	------	1.6	7.2	17.1	11.8	4.9	6.9	17.2	17.0	9.0	7.9

[1] Analyses of batches mixed at different times.

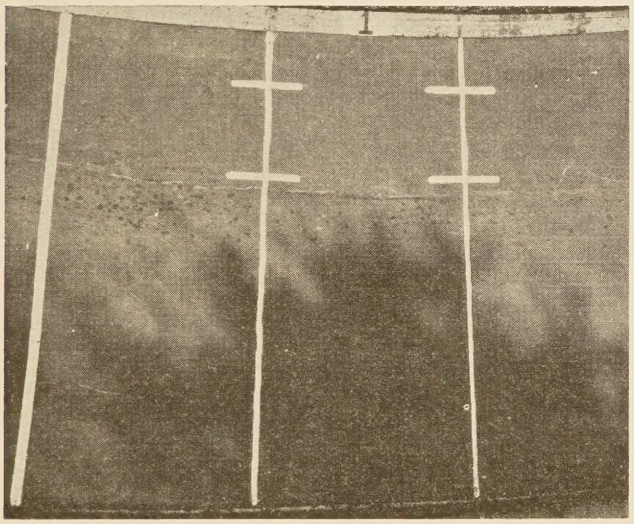

FIGURE 3.—CONDITION OF SECTION 3 ON COMPLETION OF TEST SHOWING FULL AREA OF SECTION.

compaction under traffic. A number of sections were displaced to extreme degrees, failure generally taking the form of rutting and progressive loosening of the bond between the coarse aggregate particles. The mixtures containing the higher percentages of coarse aggregate behaved in this manner. Subsequent examination of specimens taken from the less distorted areas of the sections which failed, revealed that the asphalt cement applied as the seal coat had penetrated to a considerable depth into these comparatively porous mixtures. It is probable that this excess of bitumen contributed largely to their instability. It was further noted that a large part of the coarse aggregate particles of these disintegrated areas were almost or entirely free from a coating of bitumen. This may have resulted from the abrasive action during the progressive loosening of the bond between the stone particles, but there is also the possibility of poor adhesion of the asphalt to the coarse aggregate.

TABLE 2.—*Composition of plant samples (percentages by weight unless otherwise shown) and displacement of asphaltic concrete mixtures of first series of stability tests*

Mix no.	Maximum screw movement	Bitumen	Coarse aggregate retained on no. 10 sieve: By weight	By volume	Passing and retained on next smaller size: 1½ inches	1¼ inches	1 inch	¾ inch	½ inch	¼ inch	No. 10	No. 40	No. 80	No. 200
MIXTURES CONTAINING 45 PENETRATION ASPHALT														
	In.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
16	0.3	6.32	40	33	----	12	14	24	26	24	42	31	17	10
23	0.5	5.78	51	44	----	17	19	23	28	13	38	31	20	11
12	0.5	6.03	50	42	----	12	19	25	31	13	35	33	22	10
19	0.7	5.05	61	54	11	16	18	16	22	17	41	30	19	10
8	1.8	5.41	60	53	----	17	6	22	37	18	42	29	18	11
MIXTURES CONTAINING 55 PENETRATION ASPHALT														
25	0.2	5.03	57	49	8	13	20	17	22	20	47	34	11	8
24	0.4	4.42	52	43	----	8	29	26	19	18	48	35	10	7
22	0.9	6.20	49	41	----	18	11	24	31	17	40	32	20	8
18	1.1	5.36	58	51	9	19	6	22	24	20	41	29	19	11
15	2.4	7.43	34	28	----	----	5	28	42	25	34	37	20	9
20	2.5	6.31	55	47	----	12	16	21	30	21	46	30	16	8
26	2.9	7.54	46	38	9	6	9	26	31	19	23	30	27	20
27	7.9	7.28	43	35	----	4	17	40	27	12	14	34	34	18
11 [1]	9.4	6.48	52	44	2	12	13	24	35	14	38	30	19	13
7	23.8	5.26	64	57	6	11	20	17	30	16	40	30	19	11
4	56.5	3.95	73	66	3	15	24	22	24	12	33	27	22	18
MIXTURES CONTAINING 65 PENETRATION ASPHALT														
14	0.4	6.02	43	36	----	5	17	28	31	19	37	33	21	9
3	1.4	4.83	71	64	----	7	18	30	33	12	33	29	20	19
10	4.6	5.47	55	48	----	7	24	21	36	12	38	32	20	10
6	42.5	4.59	65	58	3	16	19	22	27	13	34	31	22	13
MIXTURES CONTAINING 75 PENETRATION ASPHALT														
17	0.3	5.24	60	52	12	10	21	15	23	19	42	27	18	13
21	1.1	6.17	50	42	----	13	24	19	30	14	35	33	22	10
13	1.1	5.60	43	36	----	17	17	25	21	20	47	28	15	10
9	6.4	5.69	56	50	9	13	18	22	28	10	38	32	20	10
5	24.2	4.44	65	58	----	17	24	17	29	13	40	28	20	12
2	62.5	5.15	70	62	----	2	7	44	35	12	34	24	22	20
MIXTURE CONTAINING 86 PENETRATION ASPHALT														
1 [1]	92.0	4.35	71	64	3	9	16	21	36	15	37	25	19	19

[1] Average of analyses of batches mixed at different times.

The number of factors which affected, in varying degrees, the behavior of these 27 mixtures under traffic, makes it impossible to build up a general theory which will satisfactorily explain the observed results. There are, however, a few interesting observations which can be made:

(1) Mixtures which contained as much as 64 percent by weight (approximately 57 percent by volume) of coarse aggregate were, with one exception, badly displaced by the traffic. This behavior probably was due, as previously suggested, to the increased bitumen introduced by application of the seal coat, or to insufficient mortar to properly reinforce the interlocking of the coarse aggregate. This latter possible cause may have been supplemented by a rather poor degree of adhesion or to a loss of adhesion of the asphalt to the coarse aggregate particles.

(2) Mixtures which were highly and equally stable were prepared with asphalts of 45, 55, 65 and 75 pene-

tration. The amount of bitumen in the mixture and other details of proportioning are undoubtedly far more important factors in their effect upon stability than is the consistency of the asphalt cement.

(3) Most of the mixtures contained fine aggregates which were considerably coarser than usual aggregate specified for sheet asphalt construction. The fine aggregates of those mixtures which were most resistant to traffic displacement did not carry high percentages of filler. The average fine aggregate gradation of the five mixtures prepared with 45 penetration asphalt, all of which were highly stable, was substantially as follows:

Passing no. 10, retained no. 40 sieve	40
Passing no. 40, retained no. 80 sieve	31
Passing no. 80, retained no. 200 sieve	19
Passing no. 200, retained	10
Total	100

(4) There is no indication that the gradation of the coarse aggregate affected the behavior of the several mixtures. If gradation is of significance within the range of sizes used, its effect is obscured by the greater importance of other factors.

SECOND SERIES OF STABILITY EXPERIMENTS INITIATED

The difficulties encountered in correlating the results of the first series indicated the necessity of tests with a smaller number of variables involved. A second group of sections was planned, the major portion of which were sheet asphalt mixtures. Each was to be of different composition and to be laid 2 inches thick directly on the smooth concrete foundation without the customary intermediate or binder course. A few sections of asphaltic concrete were included to obtain direct comparisons with sheet asphalt. Dense asphaltic concrete mixtures were designed to avoid the necessity for a seal coat. Table 3 shows the schedule of mixtures. Potomac River sands typical of those used locally were used with the exception of a very fine sand imported for mixtures 1 to 5. This fine sand was blended with the medium sand used in mixtures 11 to 28. Trap rock was not available for the asphaltic concrete mixtures and a hard limestone was used. The asphalt cements were refined from Mexican crude oil and, with the exception of those which conformed to standard grades, were specially prepared, presumably from the same stock. Their test characteristics are given in table 4. The 42-penetration asphalt was drawn from the storage tanks at the paving plant. The other asphalts, of which only small amounts were needed, were heated in small auxiliary kettles.

FAILURES EXPLAINED BY FAULTS IN PROPORTIONING OR IN MATERIALS

One truck load was sufficient to surface each section of approximately 25 square yards. The frequent changes in formula, with the necessary substitution of aggregates and asphalts, resulted in considerable delay and difficulty in proportioning and in the control of temperatures. The 33 mixtures were produced in two working days. Large samples of each mixture were taken and their analyses are shown in table 5. Some discrepancies exist between the mixtures as planned and the analyses. Those which involve the percentage of filler are doubtless due to variations in the content of the Potomac River asphalt sand passing the no. 200 sieve. This sand was used from the contractor's stock. Sheet asphalt section no. 16 is an outstanding example of such discrepancy. No filler was added to this mixture, but repeated tests on samples yielded between 9.0 percent and 10.0 percent passing the no. 200 sieve.

Although the mixtures do not conform in every detail to those originally outlined, they do cover a wide range of composition and it is quite certain that each section is uniform. It is regretted that mixtures typical of the high filler combinations which have been popular in some quarters were not included in this series. Mixture 19 carried a large quantity of dust, the effect of which, however, was obscured by other factors which are discussed later.

Traffic was started on the pavements in late August as soon as measuring devices had been installed. Dis-

TABLE 3.—*Schedule of mixtures for second series of stability experiments*

Group	Mixtures or section numbers	Percentage of bitumen	Percentage of filler	Type of sand	Coarse aggregate	Type of asphalt	Penetration of asphalt
A	1 to 5	Variable	Constant	Fine	None	Steam-refined	Constant.
B	6 to 10	do	do	Coarse	do	do	Do.
C	11 to 15	do	do	Medium	do	do	Do.
D	16 to 19 and 13	Constant	Variable	do	do	do	Do.
E	20 to 23 and 13	do	Constant	do	do	do	Variable.
F	24 to 28	do	do	do	do	Blown	Do.
G	29 to 33	Variable	do	do	Limestone	Steam refined	Constant.

TABLE 4.—*Analyses of asphalt cements used in second series of tests*

Type	Mexican, steam refined					Mexican, blown				
Sections in which used	20	(1)	21	22	23	24	25	26	27	28
Specific gravity, 25/25° C	1.052	1.052	1.043	1.035	1.034	1.037	1.030	1.024	1.020	1.022
Flash point, ° C	324	299	296	299	296	258	227	235	245	243
Penetration, 0° C., 200 g, 1 min	11	15	16	19	21	17	22	27	28	34
Penetration, 25° C., 100 g, 5 sec	31	42	55	63	72	44	50	59	67	81
Penetration, 46.1° C., 50 g, 5 sec	119	175	234	(2)	(2)	137	126	195	250	(2)
Softening point, ° C	61.9	55.9	53.4	51.4	49.0	60.6	64.4	55.3	53.2	50.3
Ductility, 25° C., cm	77	95	87	100+	100+	27	13.5	55	65	53
Volatilization, percentage change in weight, 50 g, 5 hr., 163° C	+0.002	−0.004	−0.021	+0.004	−0.016	−0.062	−0.052	−0.110	−0.109	−0.146
Penetration of residue from volatilization, 25° C., 100 g, 5 sec	27	39	50	56	62	41	46	55	61	73
Total bitumen, soluble in CS_2 percent	99.77	99.65	99.62	99.56	99.59	99.72	99.63	99.31	99.53	99.50
Organic material, insoluble, percent	0.15	0.20	0.24	0.26	0.20	0.20	0.25	0.31	0.31	0.28
Inorganic material, insoluble, percent	0.08	0.15	0.14	0.18	0.21	0.08	0.12	0.38	0.16	0.22
Bitumen insoluble in 86° B. naphtha, percent	33.07	32.21	30.03	25.10	24.69	35.09	36.24	28.13	30.86	29.35
Fixed carbon, percent	17.18	17.75	15.40	13.65	14.09	15.35	15.00	13.30	13.42	12.60

[1] 1 to 19, 29 to 33, inclusive.
[2] Too soft for test.

placement began immediately in a number of the sections, but as the temperature declined during the autumn months the effect of traffic became increasingly less. The test was discontinued during the winter and recommenced in May and completed in October. Before traffic was resumed in May, a core 4 inches in diameter was punched from a location in each section which had received full compression in construction but which had not been traveled over by the truck. The densities of these plugs were regarded as representative of the degree of compression of the respective sections and are referred to later in this report.

After completion of the traffic tests, 10 slabs or more approximately 24 inches square were cut from each section for use in laboratory studies. These specimens were taken within the area of traffic as well as near the curbs and between the wheel tracks. At the same time the internal displacement rods were excavated, their positions recorded, and movements computed.

The final longitudinal displacements of the several sections are shown in the last column of table 5. The displacement of each section was taken to be the total movement of 25 screws in a radial line derived by taking half of the total movement of 50 screws in two lines. Figures 3 to 6 show the condition of the several sections at the end of the test. These pictures were taken from an elevated platform after the section and guide lines had been repainted. Displacements occurred in the direction of traffic, and they developed to a greater degree under the outer truck wheels. Instability was evidenced more by rutting with lateral displacement than by the formation of transverse waves. Some movement occurred in every section, but in many it was very slight. All failures can be readily explained by faults in the proportioning or quality of asphalt.

The degree of displacement appeared to be fairly uniform through the depth of the surface course. The sections of brass rod closely maintained their original relative positions with the exception of those in the highly plastic mixtures. In a few instances there were somewhat greater movements in the upper inch, but the actual distance moved was but slightly greater than that immediately at the foundation. Sometimes the movement at lower levels appeared a little greater than at the top, but this difference also was negligible. Typical diagrams of the migration of the reference rods are shown in figure 7. There was no bond between the surface course and the smooth foundation. It appears that the use of the customary stable binder course, which provides a positive bond with the surface course, might have improved the behavior of some of these mixtures. It is doubtful if the highly plastic mixtures would have been benefited.

The primary object of the experiment was to supply specimens of known relative degrees of stability for the subsequent development and correlation of laboratory stability tests, but the results provide an opportunity to study the practical effects of some variations in materials and compositions.

SHEET ASPHALT SECTIONS DISCUSSED

Three sand combinations of widely different gradations were used as shown in table 3. Five mixtures were laid with each sand combination, varying only the bitumen content. It was the intention to commence with a very lean mixture, and gradually increase the bitumen content until a condition of excessive richness was reached. Mixtures 1 to 5 with fine sand, 6 to 10 with coarse sand, and 11 to 15 with medium sand constituted these groups. The fine-sand and coarse-sand groups are especially uniform in aggregate composition and illustrate well the influence of changes in bitumen content. In each group resistance to displacement increased with decrease in bitumen content.

No material decrease in the stability of mixtures 1 to 10 was evident until the bitumen was increased to the

TABLE 5.—*Composition, density, voids, laying temperature and displacement under traffic of sheet asphalt sections*

Section or mixture	Bitumen	Retained on no. 10	Passing and retained on next smaller size: No. 10	No. 20	No. 30	No. 40	No. 50	No. 80	No. 100	No. 200	Specific gravity of aggregates	Specific gravity of core from pavement	Theoretical maximum specific gravity	Pavement voids	Laying temperature (°F.)	Displacement: Greatest screw movement	Total movement of 25 screws [1]
	Percent by weight	*Percent by weight*	*Percent by weight*	*Percent by weight*	*Percent by weight*	*Percent by weight*	*Percent by weight*	*Percent by weight*	*Percent by weight*	*Percent by weight*				*Percent*		*Inches*	*Inches*
1	10.2	0.2	1.8	4.3	3.8	9.3	16.3	16.0	25.6	12.5	2.675	2.055	2.312	11.0	400	0.1	1.4
2	10.9	.2	2.1	4.3	4.2	9.2	15.7	18.9	22.0	12.5	2.674	2.048	2.290	10.7	275	.6	4.0
3	12.0	.3	2.0	3.8	3.5	8.3	14.9	19.8	23.3	12.1	2.660	2.086	2.249	7.2	450	.5	2.7
4	12.6	.7	3.7	5.6	4.9	9.9	14.8	16.3	19.1	12.4	2.681	2.104	2.243	6.2	325	32.1	199.8
5	13.9	.3	2.9	5.0	3.8	8.4	14.4	14.2	24.2	12.9	2.666	2.107	2.199	4.0	335	86.0	311.2
6	7.9	6.3	16.7	12.5	7.6	10.8	10.9	7.3	7.5	12.5	2.688	2.150	2.395	10.2	350	.3	1.6
7	8.7	5.5	13.4	12.9	7.8	11.8	12.7	7.3	8.6	11.3	2.674	2.100	2.359	11.0	375	1.7	8.3
8	9.0	6.5	16.6	11.6	7.0	10.4	11.2	8.0	7.9	11.8	2.674	2.175	2.349	7.4	325	5.3	23.4
9	11.1	4.5	14.7	12.7	8.2	11.3	10.8	7.3	7.2	12.2	2.674	2.202	2.283	3.5	300	70.0	379.0
10	12.2	3.7	12.8	12.9	8.0	11.4	12.3	6.4	8.0	12.3	2.666	2.198	2.246	2.1	300	115.9	448.9
11	8.2	.2	3.3	6.0	5.9	13.1	19.5	12.8	18.3	12.7	2.674	2.018	2.374	15.1	280	.7	6.7
12	10.0	.2	4.1	7.4	6.2	13.0	18.7	10.4	16.4	13.6	2.674	2.102	2.317	9.3	325	5.7	56.1
13	10.4	.3	2.6	5.8	5.4	12.2	19.0	11.0	16.8	16.5	2.666	2.150	2.299	6.5	355	4.3	32.8
14	11.5	.2	2.6	5.7	5.5	13.2	17.9	13.7	18.2	11.5	2.676	2.075	2.273	8.7	325	13.2	137.6
15	12.3	.3	3.0	6.4	5.8	13.4	19.6	10.6	16.3	12.3	2.666	2.187	2.243	2.4	340	37.7	267.4
16	9.7	.4	4.3	8.7	7.0	14.8	19.8	10.7	14.7	9.9	2.656	1.999	2.314	13.6	375	.6	7.1
17	9.8	.2	3.8	7.9	7.0	14.5	19.2	11.3	16.1	10.2	2.674	1.990	2.323	14.4	325	1.5	15.6
18	10.1	.2	2.2	5.2	5.1	12.8	18.4	12.9	18.1	15.0	2.688	2.170	2.323	6.5	350	4.5	46.9
19	10.9	.1	2.3	4.5	4.4	10.9	15.2	11.3	16.7	23.7	2.696	2.230	2.304	3.1	280	17.0	140.8
20	9.9	.3	2.3	4.6	4.8	12.7	18.4	14.0	17.0	16.0	2.666	2.010	2.315	13.2	325	.6	2.8
21	10.0	.1	2.2	5.2	5.1	12.3	19.3	11.8	18.2	15.8	2.666	2.091	2.307	9.4	340	.5	5.1
22	9.7	.2	4.1	8.1	6.8	13.7	18.8	10.8	15.0	12.8	2.666	2.091	2.313	8.6	320	18.7	109.0
23	10.4	.2	3.6	6.7	6.3	13.7	18.1	13.0	15.9	12.1	2.668	2.073	2.291	9.5	355	9.1	71.1
24	10.0	.1	3.1	5.6	5.5	12.9	18.3	13.3	15.4	15.8	2.682	2.070	2.315	10.6	340	2.7	28.1
25	9.9	.1	3.6	6.7	6.0	13.9	18.0	13.7	16.3	11.8	2.669	2.009	2.306	12.8	310	4.9	[2] 48.4
26	10.0	.2	2.8	6.3	6.1	13.9	18.3	12.8	17.3	12.9	2.664	1.938	2.296	15.6	350	7.7	82.9
27	10.5	.2	2.8	6.7	6.4	13.6	19.1	10.6	16.5	13.6	2.666	1.940	2.280	14.9	290	36.3	244.2
28	9.7	.2	3.1	7.1	6.2	13.8	18.8	10.7	15.2	15.2	2.681	1.950	2.316	15.9	340	10.5	87.0

[1] Averaged from movement of 50 screws in two lines.

[2] Approximate as section partly disintegrated under traffic.

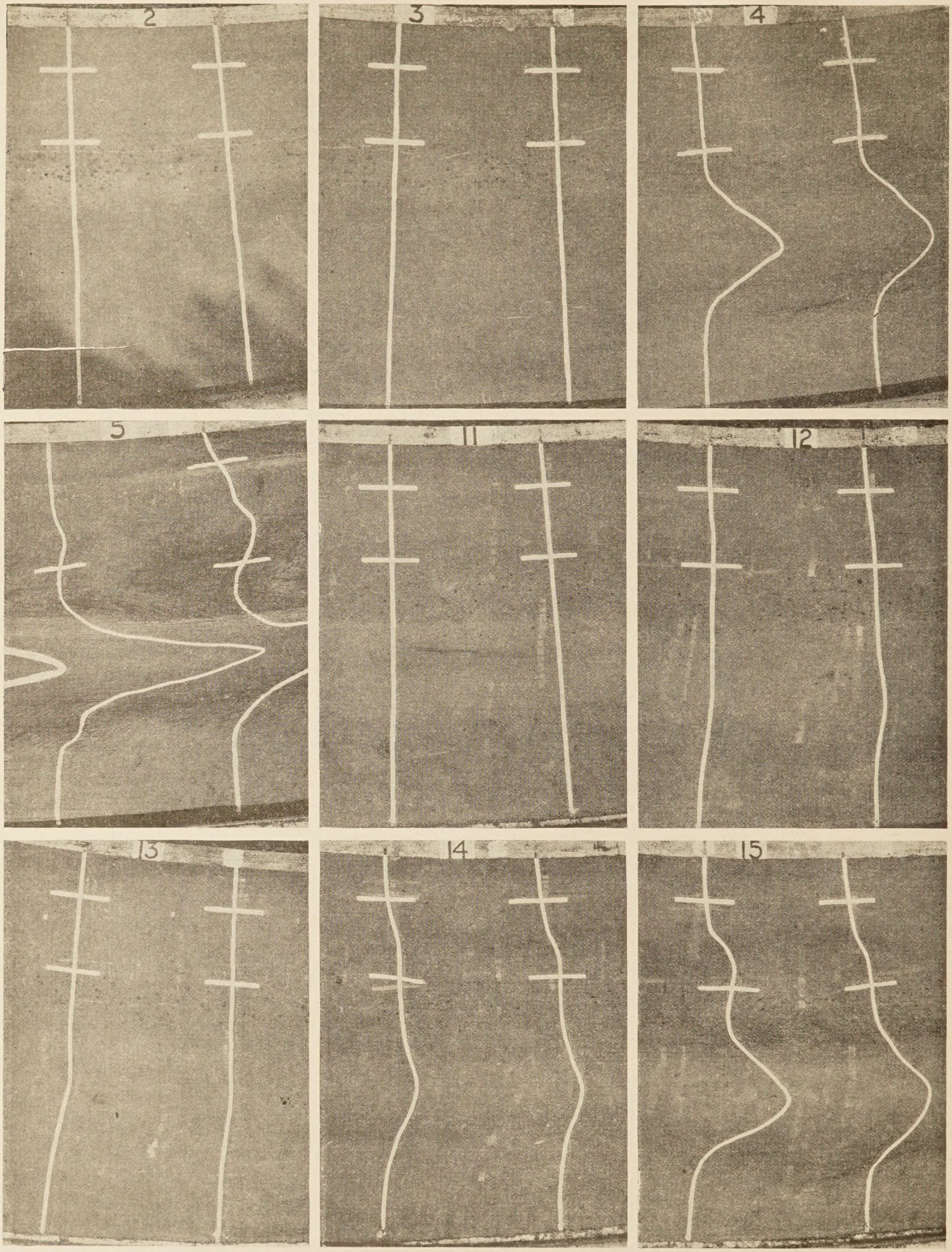

FIGURE 4.—CONDITION OF TEST SECTIONS OF SECOND SERIES ON COMPLETION OF TESTS.

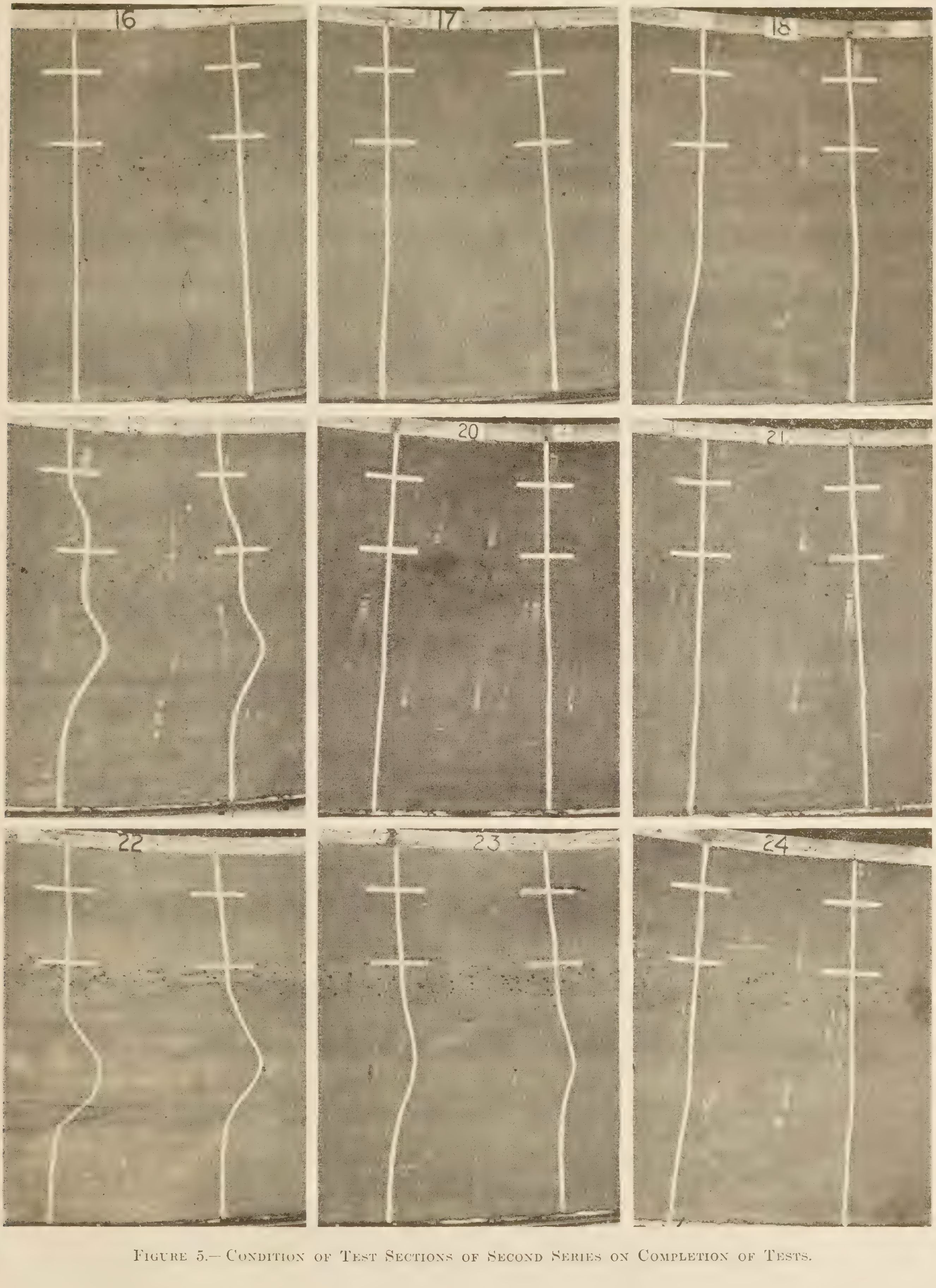

FIGURE 5.—CONDITION OF TEST SECTIONS OF SECOND SERIES ON COMPLETION OF TESTS.

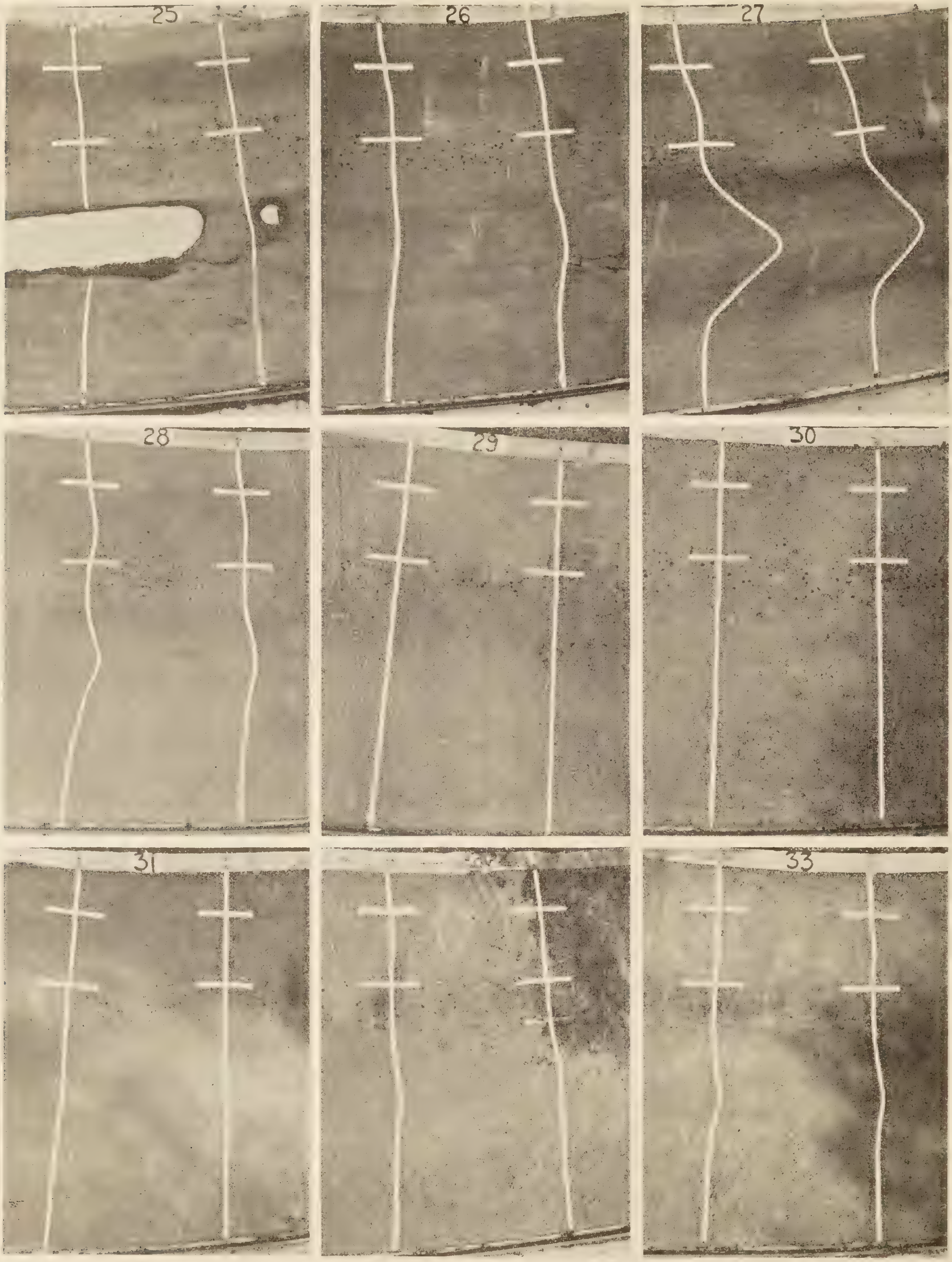

FIGURE 6.—CONDITION OF TEST SECTIONS OF SECOND SERIES ON COMPLETION OF TESTS.

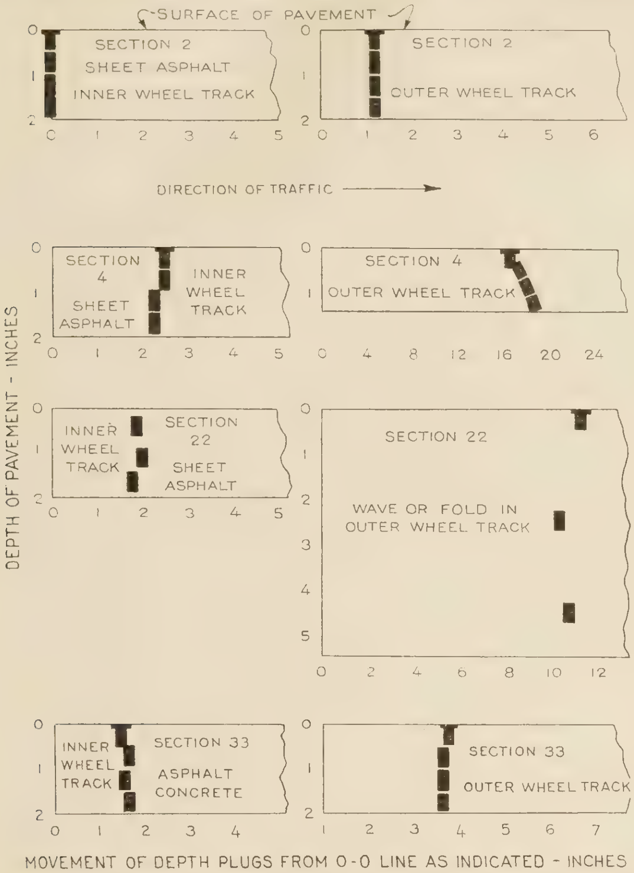

FIGURE 7.—DISPLACEMENT OF MIXTURES AT VARIOUS DEPTHS BELOW SURFACE OF PAVEMENT.

percentages used in mixtures 4 and 9. These two mixtures were so unstable as to indicate a critical percentage somewhere between 12.0 percent and 12.6 percent with the fine sand, and between 9.0 percent and 11.1 percent with the coarse sand. The medium-sand mixtures, numbers 11 to 15, exhibit the same tendencies, but for some reason the aggregate of mixture 13 proved to contain more than the intended percentage of fine material passing the no. 200 sieve and its behavior is somewhat out of line. It is more than probable that the high percentage of material in this mixture passing the no. 200 sieve results from a relatively high silt content of the sand stock, as those mixtures prepared immediately preceding it (numbers 21 and 24) exhibit similar compositions. The slightly greater stability of mixture no. 13 as compared with no. 12 must therefore be attributed to more favorable characteristics of its aggregate. Results obtained in these three groups indicate that, if properly proportioned, satisfactory sheet asphalt mixtures can be prepared with either fine, medium or coarse graded sand.

Mixtures 16 to 19 and including mixture 13 were intended to develop the effects of variations in the amount of filler, the percentage of bitumen being held constant. Analyses of the mixtures do not check with the plant formulas for the respective mixtures, not only in bitumen content but in the percentage of aggregate passing the no. 200 sieve. This is particularly noticeable in the case of mixture 16 to which no dust was added. The material passing the no. 200 sieve in this mixture represents silt in the sand supply. Mixture 13 also contained 3 percent to 4 percent more material passing the no. 200 sieve than should have been derived from the amount of limestone dust which was added and from the normal sand supply. Although these mixtures proved progressively less stable as the filler was increased, the variations in bitumen content and in the nature of the filler material make it impossible to attribute the difference in service behavior to increased filler alone.

Illustrating this point, comparison of mixtures numbers 18 and 21 show them to be almost identical in bitumen content and in aggregate gradation, although it is known that considerably more of the filler material of mixture 21 is silt than is the case in mixture 18. Although containing bitumen of somewhat softer consistency, mixture 21 seems to have been the less compressible and was certainly the more stable of the two mixtures. In this series it is true that increased percentages of the limestone dust filler accompanied by increased bitumen resulted in decreased resistance to traffic displacement. However, it is apparent that the nature and characteristics of the filler material have an effect on the behavior of the mixture. It will be shown later that voids in the mixture and in the aggregates are significant functions which are dependent upon interrelationships between the sand, filler, and bitumen. Therefore a general conclusion to the effect that increased filler is likely to cause instability is not justified, for the most desirable percentage can be determined only by studies of the several constituent materials in various combinations.

Asphalts similar in all characteristics except degree of hardness were used in mixtures 13 and 20 to 23. Here again the effects of the controlled variable are obscured by lack of uniformity in other characteristics. The aggregate composition of mixtures 20, 13, and 21 are closely similar. These mixtures contain asphalts of 31, 42, and 55 penetration respectively and the mixtures may be properly compared with regard to this variable. Between sections 20 and 21, containing asphalt varying in penetration by 24 points, no marked difference in performance was evident. Both sections behaved excellently. Mixture 13 displaced slightly although its bitumen was harder than that in section 21. Its slightly greater richness as well as the presence of an extremely plastic mixture adjacent to it probably explains the small difference in behavior. Mixtures 22 and 23 displaced somewhat more than those just discussed. Comparing mixture 22 with mixture 20 it is found that they differ only in percentages of particles passing the no. 200 sieve and in penetration of asphalt. The same is true of mixtures 23 and 13.

In spite of the slight variations in the bitumen and filler contents of the mixtures in this series and the lack of a consistent relationship between consistency of asphalt and displacement of the mixture, the much greater displacements in sections 22 and 23 are undoubtedly due to the higher penetrations of the asphalts used in these sections. As compared with the material used in section 21, the asphalts in sections 22 and 23 had penetrations at 25° C. (77° F.) higher by only 8 and 17 points, respectively. However, at 46.1° C. (115° F.) the asphalt in section 21 had a penetration of 234 while the asphalts in sections 22 and 23 were too soft to test under standard conditions at this temperature. None of these sections displaced under traffic until the air temperatures became quite high and the pavement temperatures much higher. The greater movements in sections 22 and 23, therefore, may be attributed directly to the softening of the asphalt to a greater extent under the summer heat, resulting in mixtures less resistant to the action of traffic, than was the case with asphalts which were but slightly harder as measured by the penetration at 25° C. (77° F.).

DISINTEGRATION OBSERVED ON BLOWN ASPHALT SECTIONS

The sections containing the blown asphalts (mixtures 24 to 28) behaved, as far as displacement is concerned, very much as did those sections containing steam refined asphalts. The influence of the lower percentage of bitumen and possibly that of the higher content of material passing the no. 200 sieve in promoting stability are apparent. The softer asphalts may be a little less effective in resisting displacement, although the virtually identical behavior of sections no. 26 and no. 28 with asphalts of 59 and 81 penetration respectively, indicate that the hardness of the binder is certainly not the most potent factor affecting stability.

The mixtures containing blown asphalt showed an unexpected tendency to disintegrate under traffic and this tendency was confined solely to these mixtures. Disintegration was most marked in section 25, where the binder became brown and lifeless; the mixture crumbled, and wore down to the foundation as the test progressed. It is perhaps significant that the asphalt of this mixture had the lowest ductility, 13.5 cm, of any of the group. Section 24 showed slight indication of deterioration in a single small crack, but the other three of the group were beginning to show distress when the test was terminated.

The mixtures of this group were apparently not compacted to the degree to which they were susceptible. Although this may have rendered them liable to some displacement, the fact remains that perfectly stable sections in other groups were constructed with equally high voids. The outstanding indication is that only the blown asphalt mixtures showed deterioration through loss of binding power of the asphalt.

It is obvious that each of these groups of sheet asphalt mixtures had a certain resistance to displacement, which apparently depended to a great extent upon the amount of bitumen present. The limit of resistance was reached sooner on some sections than on others. The same medium sand combined with similar percentages of filler has long been used in the construction of Washington streets, and has successfully carried from 10.5 percent to 11.5 percent by weight of bitumen, which indicates the unusually severe conditions of these tests.

AGGREGATE VOIDS DETERMINED BY VARIOUS METHODS

The voids in the compacted mineral aggregate are considered by many to exert a controlling effect on mixture design. A considerable investigation was conducted, therefore, on the aggregate voids of these mixtures and the degree to which they were filled with bitumen. Determinations of aggregate voids were made in four different ways, as follows:

(1) By computation based on test results from cores taken from the various sections and representing the compression received by each mixture in construction.

(2) By direct determination using the cone method [3] on samples of aggregates extracted from the mixtures.

[3] A small metal cone of approximately 180 cc capacity was filled with the aggregate in four equal increments. After each addition of aggregate, the cone was beaten for 5 minutes on a brass plate mounted on a rigid base. After the last increment was added, beating was continued until no decrease in volume could be obtained. Percentage of voids is computed from weight of the known volume of aggregate contained in the cone and the specific gravity of the aggregate.

(3) By direct determination, using small cylinders in the vibrator developed by the Bureau of Public Roads [4] for this work, on samples of aggregates extracted from the mixtures.

(4) By computations using laboratory determinations on compressed specimens prepared by the method of tamping and compression prescribed for the Hubbard-Field stability test.[5]

Figure 8 and table 6 show the percentages of voids in the aggregates of the various mixtures as determined by these four methods. It will be noted that compaction by the Hubbard-Field method nearly always results in the lowest voids in the aggregate. There are a few exceptions to this where sufficient bitumen was present to prevent ultimate compaction of the aggregate particles. The use of the vibrator resulted in percentages of aggregate voids which were generally higher than those obtained by the Hubbard-Field method, and, in general, lower than those found by the cone method or in the pavement as actually compressed. The compaction obtained by the vibrator method was sufficient to wedge the aggregate particles tightly in the container and they had to be loosened by a pointed metal rod in removing.

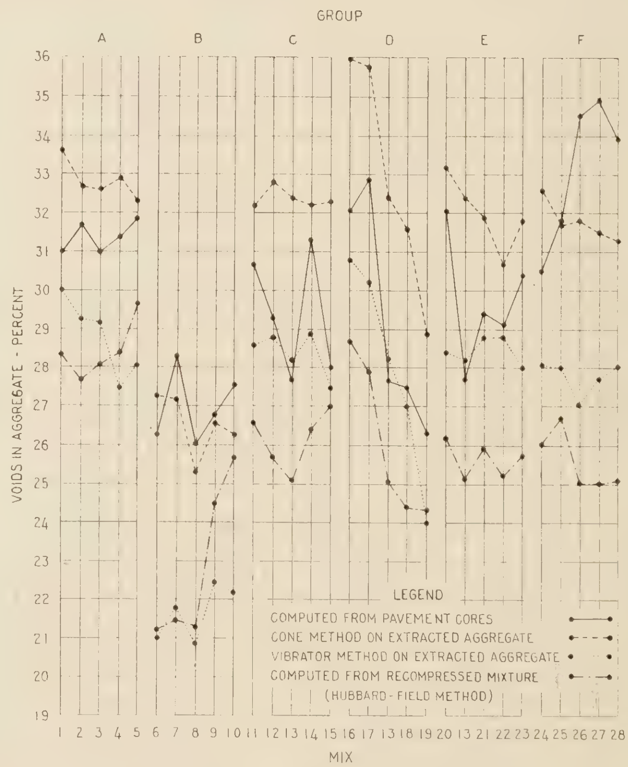

FIGURE 8.—VOIDS IN AGGREGATES OF SHEET-ASPHALT MIXTURES AS DETERMINED BY FOUR METHODS.

[4] Researches on Bituminous Paving Mixtures, Public Roads, vol. 7, no. 10, December 1926, p. 205.

[5] The material for molding the Hubbard-Filed specimens is first heated indirectly by means of an oil bath in order to prevent overheating, to the molding temperature of 250° F. The amount necessary to form a compacted specimen one inch high is then placed in the warmed cylinder and given 60 blows with the blade tamper and 15 blows with the blunt tamper. The cylinder containing the tamped material is then placed in a suitable container, the plunger inserted and placed under the head of the Hubbard-Field machine and pressure of 3,000 pounds per square inch is applied. Water is then poured in the container and around the cylinder in order to cool the specimen, maintaining the pressure while cooling. After remaining under pressure in water for 5 minutes the sample is removed from the mold, specific gravity determined by displacement method and percentage of aggregate voids computed.

The following formula is used:

$$\text{Percentage of voids in aggregate of compacted mixture} = 100 - \frac{MS}{A}$$

where M = density of compacted specimen,

S = percentage by weight of aggregate in mixture,

A = specific gravity of aggregate.

Table 6.—*Voids in aggregates of sheet-asphalt mixtures and degree to which voids were filled with bitumen*

Mixture or section number	Percentage of voids in aggregates				Percentage of aggregate voids filled with bitumen, assuming mixtures compacted to densities corresponding to voids of columns 1 to 4			
	Computed for track cores	Cone method	Vibrator method	Computed for recompressed mixture	Computed for track cores	Cone method	Vibrator method	Computed for recompressed mixture
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1	31.0	33.6	30.0	28.4	64.2	57.1	67.7	72.8
2	31.7	32.7	29.3	27.4	66.8	63.9	75.0	81.2
3	31.0	32.6	29.2	28.1	76.8	71.2	83.7	88.4
4	31.4	32.9	27.5	28.4	80.3	74.8	96.8	92.6
5	31.9	32.3	28.1	29.7	87.1	85.5	104.8	96.9
6	26.3	27.3	21.0	21.2	61.6	58.2	82.4	81.4
7	28.3	27.2	21.8	21.5	61.5	65.0	87.2	88.4
8	26.0	25.3	20.9	21.3	71.5	74.3	95.0	92.9
9	26.8	26.6	22.5	24.5	86.9	88.0	109.7	97.8
10	27.6	26.3	22.2	25.7	92.4	98.8	122.7	101.7
11	30.7	32.2	28.6	26.6	51.2	47.8	56.8	62.6
12	29.3	32.8	28.8	25.7	68.3	57.9	70.3	81.6
13	27.7	32.4	28.2	25.1	76.5	61.1	74.5	87.7
14	31.3	32.2	28.9	26.4	72.2	69.6	81.5	92.1
15	28.1	32.3	27.5	27.0	91.4	74.8	94.0	96.1
16	32.1	36.0	30.8	28.7	57.3	48.1	61.1	67.4
17	32.9	35.8	30.2	27.9	56.2	49.5	63.9	71.3
18	27.5	31.6	27.0	24.4	75.7	62.7	78.8	88.9
19	26.3	28.9	24.0	24.3	87.8	77.5	99.0	97.6
20	31.2	33.2	28.4	26.2	58.9	55.7	70.6	78.4
21	29.4	31.9	28.8	25.9	68.3	60.5	70.2	81.2
22	29.1	30.7	28.8	25.2	67.4	62.2	68.1	80.0
23	30.4	31.8	28.0	25.7	68.4	64.2	77.0	86.7
24	30.5	32.6	28.1	26.0	65.2	59.2	74.2	81.8
25	31.8	31.7	28.0	26.7	60.7	61.5	73.2	78.1
26	34.5	31.8	27.0	25.0	54.8	62.3	78.2	86.7
27	34.9	31.5	27.7	25.0	57.3	66.6	80.3	92.0
28	33.9	31.3	28.0	25.1	54.6	62.3	72.3	84.1

The cone method of determination gave aggregate voids which were generally higher than actually existed in the pavement. Some of the coarse-sand mixtures and some of those laid with the blown asphalts were exceptions. This reversal of the general tendency may indicate the relative incompressibility of these mixtures, although, in the case of the coarse-sand mixtures at least, it is more likely that they did not receive sufficient rolling.

TESTS SHOW A RELATION BETWEEN AGGREGATE VOIDS, PERCENTAGE OF BITUMEN AND STABILITY

The percentages of voids as determined by the various methods show the influence of gradation. The coarse group is characteristically low in voids. The variable filler series illustrates how aggregate density may be increased by the addition of dust. None of the laboratory methods uniformly reproduces the condition attained in the field.

Figure 9 shows the percentages to which the aggregate voids would be filled with bitumen were the several mixtures compressed to the degree corresponding to the aggregate voids determined by the various methods and shown in Figure 8. The data from which this chart is derived are given in Table 6. It appears that the fine-sand mixtures, as compacted in the test sections remain stable with approximately 77 percent of the aggregate voids filled with bitumen. The coarse-sand group seemed able to carry bitumen up to about 75 percent of the aggregate voids. Due probably to variations in sand supply, the behavior of the medium-sand mixtures was somewhat more erratic, due probably to variations in sand supply. Possibly a percentage of bitumen equivalent to 70 percent of the aggregate voids is about the maximum for stable mixtures and this maximum is dependent on the presence of approximately 12 or 13 percent of material passing the No. 200 sieve. There are indications that higher dust contents, such as in mixtures 13 and 18, which

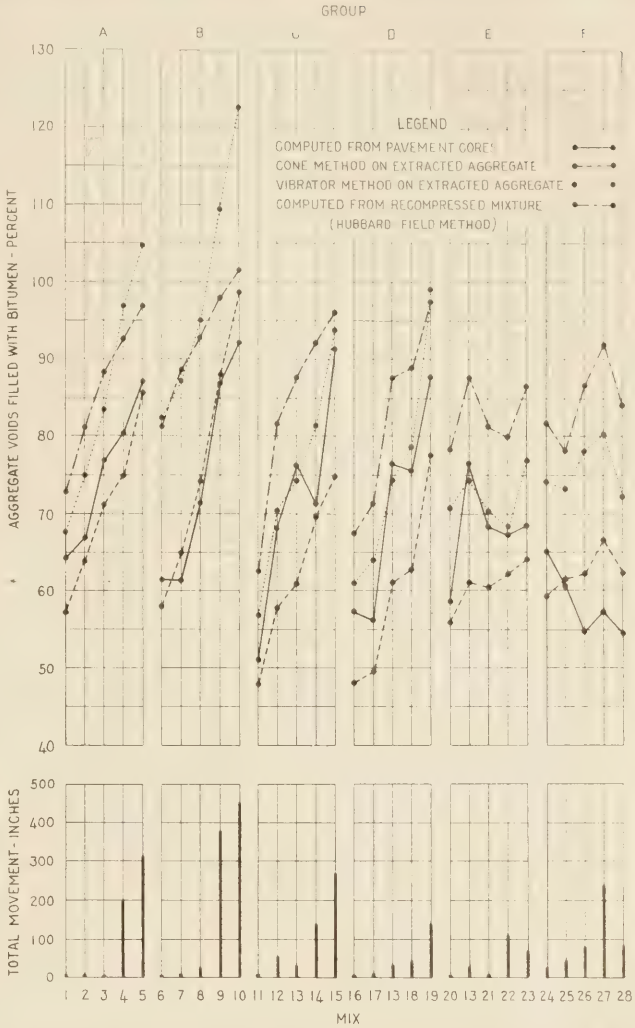

Figure 9.—Percentage of Voids in Aggregate Filled With Bitumen When Mixtures are Compressed to Densities Corresponding to Aggregate Voids Shown in Figure 8.

decrease the size as well as the volume of aggregate voids, will permit a somewhat greater percentage of voids to be filled with bitumen.

If other methods of void determination are used the results may be different and, the ratio of voids to volume of bitumen will be changed. If voids in the aggregate are taken to be those determined by the cone method, the degree to which the voids may be safely filled will be lower than that given in the preceding paragraph. Conversely, a greater percentage may be used on the basis of the lower voids determined by the vibrator method or the Hubbard-Field compression mixture method. Probably the greatest possible density of mixture is attained by the latter method and it is interesting to note that only the extremely low-void aggregates of the coarse-sand series carried, with satisfactory results, an amount of bitumen greater than 90 percent of the aggregate voids as computed by the Hubbard-Field method.

STABILITY OF TEST SECTIONS COMPARED WITH RESULTS OF LABORATORY STABILITY TESTS

Table 7 and figure 10 show comparisons of the observed field stabilities, the stabilities as determined by the Hubbard-Field method upon recompressed specimens, and approximations of Hubbard-Field stabilities of the mixtures at the densities of the sections as constructed. These approximations were made as in-

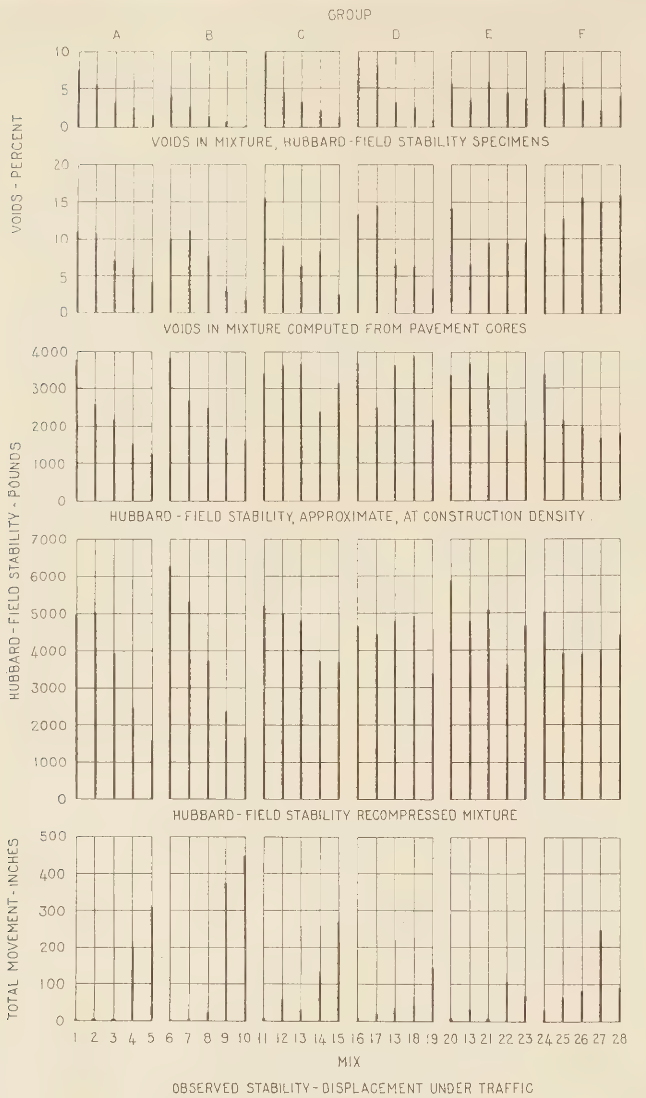

Figure 10.—Comparison of Stability and Voids of Sheet Asphalt Mixtures.

dicated in figure 11 by interpolating for the determined void content of pavement cores between results obtained in the Hubbard-Field test upon recompressed laboratory specimens and specimens taken from the upper 1 inch of removed pavement slabs. There is excellent agreement between displacement of test sections and the results of stability tests in the laboratory for the fine-sand and coarse-sand mixtures. The results indicate that a Hubbard-Field stability of over 3,000 pounds for laboratory compressed specimens is necessary to withstand the type of traffic to which these mixtures were subjected. At the densities to which the mixtures were compressed in the pavement a stability of about 2,000 is required.

The medium-sand mixtures gave decidedly less consistent agreement in results. All mixtures with a stability of 4,500 or more for laboratory compressed specimens gave reasonably good service. It will be noted that decidedly inferior mixtures such as numbers 14 and 15 have stabilities in laboratory tests approximately equal to that of the satisfactory mixture number 8. The soft-blown-asphalt mixtures, numbers 26, 27, and 28, also gave fairly high stability in laboratory tests. These mixtures were poorly

Table 7.—*Comparison of stability and voids in sheet asphalt mixtures*

Mixture or section number	Stability: Total displacement of 25 screws [1]	Stability: Hubbard-Field test on recompressed specimens [2]	Stability: Hubbard-Field test on specimens at approximately construction density	Voids in mixture: Computed for cores taken from pavement	Voids in mixture: Computed for recompressed specimens for Hubbard-Field stability test.	Rank according to—: Displacement by traffic	Rank according to—: Results of Hubbard-Field test on recompressed specimens	Rank according to—: Results of Hubbard-Field test on specimens at approximately construction density
	Inches	*Pounds*	*Pounds*	*Percent*	*Percent*			
1	1.4	5,008	3,800	11.0	7.7	1	9	3
2	4.0	5,019	2,600	10.7	5.2	5	8	13
3	2.7	3,986	2,200	7.2	3.2	3	17	17
4	199.8	2,445	1,500	6.2	2.2	23	25	26
5	311.2	1,638	1,250	4.0	1.1	26	28	28
6	1.6	6,269	3,850	10.2	4.0	2	1	2
7	8.3	5,372	2,700	11.0	2.6	9	3	12
8	23.4	3,741	2,500	7.4	1.6	11	21	15
9	379.0	2,355	1,700	3.5	.6	27	26	24
10	448.9	1,697	1,650	2.1	.1	28	27	27
11	6.7	5,244	3,450	15.1	10.0	7	4	7
12	56.1	5,044	3,700	9.3	4.8	16	7	6
13	32.8	4,824	3,700	6.5	3.1	13	11	5
14	137.6	3,723	2,400	8.7	2.1	21	22	16
15	267.4	3,759	3,200	2.4	1.0	25	20	11
16	7.1	4,687	3,750	13.6	9.4	8	12	4
17	15.6	4,466	2,525	14.4	8.0	10	15	14
13	32.8	4,824	3,700	6.5	3.1	13	11	5
18	46.9	4,908	3,900	6.5	2.7	14	10	1
19	140.8	3,376	2,175	3.1	.7	22	24	19
20	2.8	5,849	3,400	13.2	5.7	4	2	8
13	32.8	4,824	3,700	6.5	3.1	13	11	5
21	5.1	5,136	3,425	9.4	5.5	6	5	10
22	109.0	3,619	1,875	9.6	4.6	20	23	22
23	71.1	4,676	2,175	9.5	3.5	17	13	20
24	28.1	5,047	3,400	10.6	4.8	12	6	9
25	48.4	3,955	2,175	12.8	5.9	15	18	18
26	82.9	3,936	2,000	15.6	3.3	18	19	21
27	244.2	4,050	1,675	14.9	2.0	24	16	25
28	87.0	4,472	1,800	15.9	3.9	19	14	23

[1] Averaged from movement of 50 screws in two lines. In some cases results are approximate as a result of disintegration.
[2] Average of 3 specimens.

compacted in the field test and their estimated stability based on field voids is more in line with their field performance.

The voids in the mixtures (table 7 and fig. 10) both as laid and after laboratory recompression, are of interest in connection with the stabilities. In each group of mixtures, the highest voids of recompressed specimens are in mixtures having the greatest stabilities both in the Hubbard-Field test and under traffic.

Those mixtures which because of high bitumen content readily compacted to produce pavements having the lowest voids proved the least stable under traffic even though, as in the case of mixtures 14, 15, and 27, the Hubbard-Field stability test indicated good strengh. In general, it may be said that those mixtures, which, because of the high bitumen content were compressible by the indicated method to 2 percent or less of voids, were virtually certain to displace under the traffic used in these tests.

Mixture 8 may appear to be an exception to this statement, but it will be remembered that the aggregate of this group had low voids and the lower voids of the compressed mixtures do not represent greater density obtained by dangerously increased bitumen content.

The blown-asphalt group presents another phase of the density and stability relationship. As a result of inadequate rolling the voids in the pavement were high. Instead of behaving as did high-void mixtures in other groups, mixtures 26 and 28 were relatively unstable, reflecting to some extent perhaps the softer

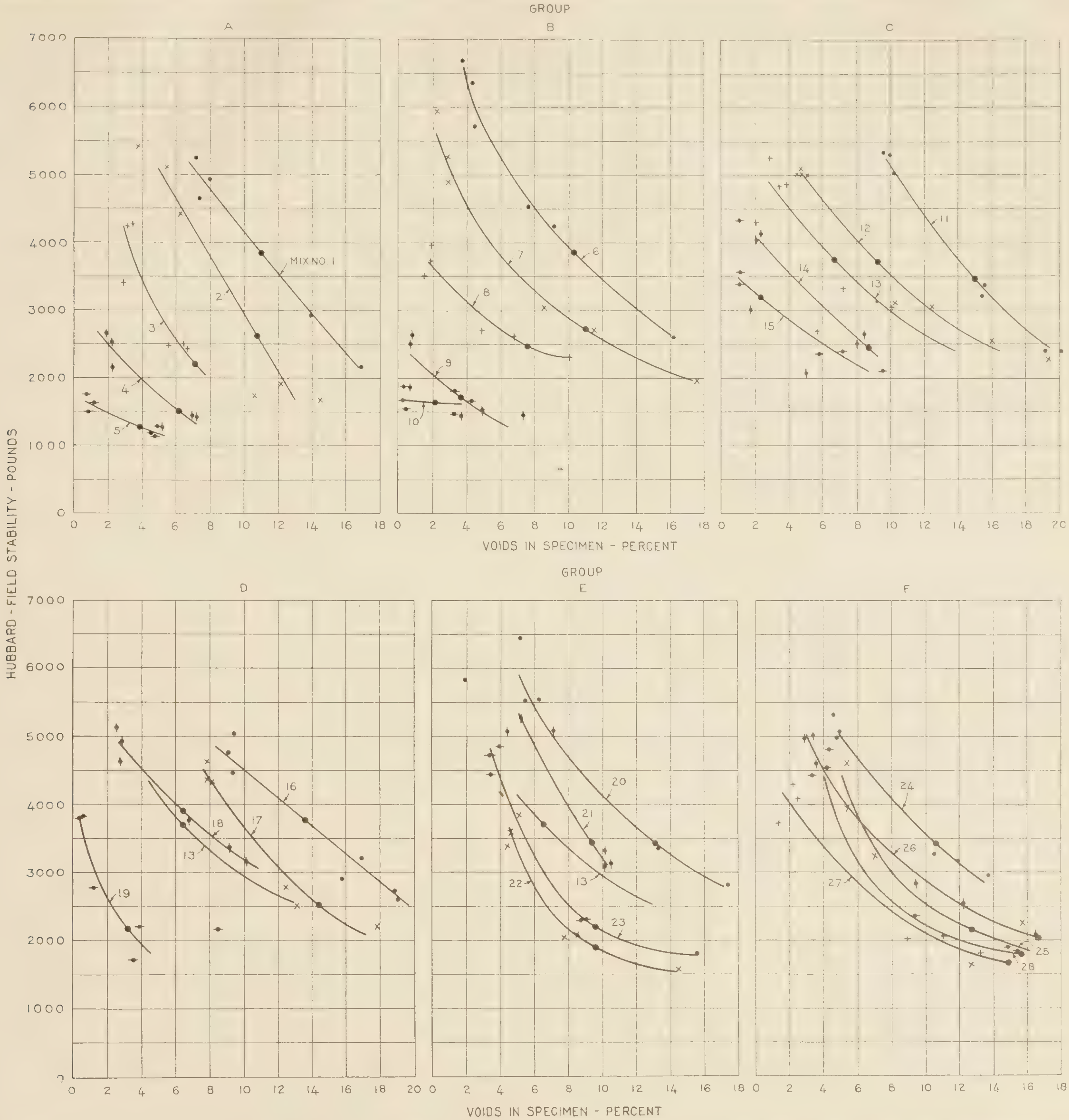

FIGURE 11.—CURVES SHOWING RELATION BETWEEN VOIDS IN SPECIMEN AND HUBBARD-FIELD STABILITY AS DETERMINED ON RECOMPRESSED SPECIMENS AND ON SPECIMENS TAKEN FROM TOP INCH OF PAVEMENT. THE LARGE DOT ON EACH CURVE CORRESPONDS TO THE PERCENTAGE OF VOIDS IN THE PAVEMENT AS CONSTRUCTED. DIFFERENT SYMBOLS ARE USED TO REPRESENT POINTS PERTAINING TO DIFFERENT MIXTURES.

bitumen which they contained, but more probably due to their susceptibility to compression to approximately the danger line of voids for their respective aggregates. Mixture 27 had a harder bitumen than mixture 28 and high voids as laid in the pavement. This mixture proved very unstable in laboratory tests after being compressed to 2.0 percent voids.

In this discussion of asphalt mixtures only stability or resistance to displacement has been considered. In reviewing this work and the results, it should be remembered that unusually severe conditions of traffic were imposed. The smooth base offered no keying action and the test was conducted during the hottest periods of the year when faults in proportioning are most likely to be developed.

ASPHALTIC CONCRETE MIXTURES DISCUSSED

In the earlier stability experiments with asphaltic concrete, those mixtures which contained approximately equal parts of coarse aggregate (retained on the no. 10 sieve) and fine aggregate (passing the no. 10 sieve) were very resistant to traffic displacement. Five asphaltic concrete sections of the second series of tests were patterned after these highly successful mixtures. All were planned to be of the same aggregate composition with the bitumen content as the only intended variable.

A clean hard limestone was used as the coarse aggregate. The sand and filler were the same as used in all of the sheet-asphalt sections except mixtures 1 to 10 inclusive. The asphalt cement, the analysis of which is given in table 4, had a penetration of 42. At the plant mixture 29 appeared to be extremely lean. Mixture 32 was so rich that it flowed freely and formed a uniform layer over the bottom of the truck, giving rise to the suspicion that it would prove unstable under traffic. Instead of using a still higher percentage of bitumen in mixture 33 as scheduled, a bitumen content between that of mixtures 31 and 32 was adopted.

Table 8 shows the plant formulas with which the asphaltic concretes were prepared, the analyses of large samples taken at the paving plant, their stabilities in the pavement as determined by the displacement of the reference screws, and laboratory determinations of stability with the roller stability machine.[6] No asphaltic-concrete section was displaced to an extreme degree. The maximum movement of a single reference screw in section 33, the least stable of the group, was 3.6 inches.

Table 9 shows the theoretical and actual specific gravities of the several mixtures, the calculated voids in the pavements and in the aggregates, and the extent to which the voids in the aggregates were filled with bitumen. It was recognized that small plugs as punched from the sheet asphalt sections would not be suitable for determining the density of the coarse-graded mixtures as laid in the pavement. Therefore, after the entire test was completed, a number of slabs, approximately 24 inches square, were chopped from each section. In general, eight slabs were taken from each section from areas over which the traffic had not passed, and the average of their specific gravities is regarded as the original density of the pavement section. Two slabs were taken from the wheel-track area and their average densities gives an indication of the increased compression generally resulting from the 64,000 passages of the truck.

The photographs of the asphaltic-concrete sections at the completion of the test, and the data in table 8 show that no mixture was displaced to an extreme degree, despite a variation in bitumen content of more than 50 percent between some of the mixtures. It is true that mixtures 32 and 33 were relatively unstable, reflecting the effect of the high ratio of volume of bitumen to volume of voids in the aggregate. Traffic on these sections during high summer temperatures produced a virtual absence of voids in the area under traffic. That a greater degree of instability did not develop, comparable with the complete failure of the richest sheet asphalt mixtures with voids practically filled with bitumen, must be due chiefly to the inherent rigidity of the interlocked particles of coarse aggregate.

RESULTS OF ROLLER TESTS IN AGREEMENT WITH PERFORMANCE OF MIXTURES IN FIELD TESTS

Specimens were cut from the large field samples and tested in the laboratory on the roller stability machine.[7] This machine was designed to reproduce the action of

[6] Researches on Bituminous Paving Mixtures, Public Roads, vol. 7, no. 10, December 1926.

[7] Researches on Bituminous Paving Mixtures, Public Roads, vol. 7, no. 10, December 1926.

TABLE 8.—*Composition, analyses, displacement and roller stability values of asphaltic-concrete mixtures of second series of stability tests*

Section or mix no.	Plant or mixing formula, percent by weight				Analyses of plant sample, passing and retained on next smaller size, percentages by weight														Displacement		Roller stability, number of passages
	Bitumen	Dust	Sand	Stone	Bitumen	1¼ inches	1 inch	¾ inch	½ inch	¼ inch	No. 10	No. 20	No. 30	No. 40	No. 50	No. 80	No. 100	No. 200	Maximum movement of single screw, inches	Movement of 25 screws, inches [1]	
29	5.1	3.5	43.7	47.7	4.8	0.0	14.6	17.0	8.6	3.0	3.3	4.3	3.4	8.2	12.0	6.8	8.5	5.5	0.2	0.8	355
30	6.0	3.4	43.2	47.4	5.8	4.7	12.7	16.1	10.4	4.8	3.0	4.0	3.0	6.6	9.2	5.2	8.6	5.9	.1	.5	297
31	7.1	3.4	42.8	46.7	7.1	3.6	9.6	11.7	10.9	5.7	3.7	4.5	3.4	7.2	10.5	6.2	9.1	6.8	1.2	5.1	244
32	8.1	3.3	42.3	46.3	7.6	4.3	10.0	13.9	10.9	5.6	4.5	4.6	3.3	6.9	8.7	6.1	7.6	6.0	2.9	21.1	88
33	7.6	3.4	42.5	46.5	7.3	2.7	11.8	16.1	9.1	7.0	4.8	5.0	3.3	6.5	8.4	4.8	6.5	6.7	3.3	21.5	99
Average gradation, coarse aggregate						6.8	26.1	33.2	22.3	11.6											
Average gradation fine aggregate											8.0	9.3	6.7	14.6	20.1	11.9	16.6	12.8			

[1] Averaged from movement of 50 screws in 2 lines.

TABLE 9.—*Density and voids of asphaltic-concrete mixtures in traveled and untraveled areas*

Mix no.	Theoretical maximum specific gravity		Average specific gravity of pavement slabs		Voids in compressed mixtures				Voids in aggregate				Percentage of aggregate voids filled with bitumen			
					Based on extraction composition		Based on plant formula composition		Based on extraction composition		Based on plant formula composition		Based on extraction composition		Based on plant formula composition	
	Based upon extraction	Based upon plant formula	No traffic	Subjected to traffic	No traffic	Subjected to traffic	No traffic	Subjected to traffic	No traffic	Subjected to traffic	No traffic	Subjected to traffic	No traffic	Subjected to traffic	No traffic	Subjected to traffic
					Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
29	2.490	2.480	2.309	2.277	7.3	8.6	6.9	8.2	17.8	19.0	17.9	19.2	59.0	54.8	62.6	59.2
30	2.456	2.450	2.332	2.404	5.1	2.1	4.8	1.9	18.0	15.4	18.1	15.6	71.6	86.3	73.4	87.8
31	2.417	2.417	2.329	2.403	3.6	.6	3.6	.6	19.3	16.8	19.3	16.8	81.3	96.6	81.3	96.6
32	2.393	2.379	2.378	2.390	.6	.1	0.04	.0	17.8	17.4	18.3	18.4	96.6	99.4	99.2	100.+
33	2.410	2.400	2.387	2.395	1.0	.6	.5	.2	17.6	17.2	17.8	17.5	94.3	96.6	97.2	98.9

traffic over a pavement surface. It consists essentially of a series of 11 steel cylinders or rollers, 4 inches in diameter and 3 inches long, mounted between and near the peripheries of two confining steel disks, which in turn are rotated by a motor. Beneath the rollers is a tank of water maintained at a temperature of 60° C. by an electric heater, and in which the specimen to be tested is placed. This bath is supported on 4 cams which rotate, raising and lowering the bath to compensate for the vertical component of the arc described by the roller in passing over the specimen, thereby eliminating impact.

The specimen is supported in a testing mold having one end and the top surface open. Rotation of the rollers is induced as they pass over the top surface of the specimen, tending to deform it longitudinally through the open end of the mold. The deformation is measured with an Ames dial. A trip indicator records the number of rolls passing over the specimen. The total weight of the bank of rollers is 450 pounds, giving a load of 150 pounds per inch width of roller as they pass over the specimen.

To prevent upward deformation at the sides of the specimen, a small section of angle-iron is clamped over its edges, extending ½ inch over the top at either side, leaving a 3-inch open surface over which the rollers pass. The specimen to be tested is brought to a temperature of 60° C. and is placed in the bath. The revolving rollers are lowered in contact with its surface. Rolling is continued until 0.3 inches longitudinal deformation has taken place. The number of rollers passing over the specimen in producing this deformation is read from the trip indicator and recorded as the roller stability value of the specimen. Results of this test were in general agreement with the service behavior of the asphaltic-concrete mixtures, those composing sections 32 and 33 deforming far more readily than did those which had more successfully resisted the truck traffic.

OBSERVATIONS ON SHEET ASPHALT MIXTURES

1. Equally stable sheet asphalt surface mixtures were laid with all three of the sands used. The fine and coarse sands were considerably outside the gradation limits of the usual specifications.

2. The effects of the individual characteristics of the constituent materials are so interrelated that in designing a sheet asphalt mixture it is necessary to consider the properties of various combinations of the materials.

3. The voids in the aggregate are of decided significance with respect to the amount of asphalt which the mixture can carry satisfactorily.

4. Aggregate voids may be determined by a number of methods but all of the several methods used in this investigation gave different results. The most satisfactory method was that of determination on compressed mixture specimens, prepared as for the Hubbard Field stability test. In general, this method gave lower voids than any other method.

5. No mixture having the voids completely filled with bitumen remained stable under the traffic imposed in these tests. In general, the maximum percentage of bitumen carried by stable mixtures amounted to between 85 percent and 90 percent of the aggregate voids as determined for Hubbard-Field stability specimens.

6. There are indications that mixtures containing aggregates with low percentages of voids may carry amounts of bitumen representing somewhat higher percentages of the aggregate voids. No mixture studied gave satisfactory results where the aggregate voids were filled in excess of 93 percent.

7. Stable mixtures resulted from the use of asphalts of 35 to 55 penetration. Softer steam-refined asphalts of 63 and 72 penetration gave somewhat more plastic mixtures, although but one section was laid with each of these consistencies.

8. The few sections in which blown asphalt was used were unsatisfactory due to the deterioration of the asphalt resulting in either actual or incipient disintegration of the pavement mixtures.

9. During construction the mixtures probably were not compressed to the maximum possible degree, although in general those which showed the highest voids in the pavement were also the least compressible in the laboratory. From the standpoint of stability, low voids in the compressed mixture appear to be undesirable if void reduction is accomplished by the addition of bitumen beyond the limits defined in 5 and 6. In each group of mixtures, except possibly the blown asphalt series, those with the higher percentages of voids were the more stable. This is further evidence of the importance of voids being filled with asphalt to the proper degree.

10. Since field compression is relatively nonuniform, the relative compressibility of mixtures can best be determined in the laboratory by standard test methods.

11. In general, service behavior of the mixtures was approximately proportional to stability values determined in the laboratory by the Hubbard-Field method. There were a few instances in which laboratory tests on mixtures found to be plastic in field tests indicated stabilities about as high as for mixtures found to be stable under traffic. In these cases, however, the voids in specimens compressed in the laboratory were dangerously low.

12. Air temperature prevailing during the daily traffic periods exerted a strong influence upon the behavior of the mixtures. With average air temperatures lower than 70° F. all sections were stable. At increased temperatures instability first became evident in those sections which were expected to prove least stable.

OBSERVATIONS ON ASPHALTIC-CONCRETE MIXTURES

14. Asphaltic concrete mixtures of dense graded aggregates appear well able to resist the heavy concentrated traffic imposed in these tests. The mixtures of the second series were outstandingly successful. They contained approximately equal parts of coarse and fine aggregate.

15. The limits of range in bitumen content which can be used without danger of extreme displacement are much less critical with the dense-graded asphaltic concretes than with sheet asphalt.

16. Although it is recognized that asphaltic concretes containing well over 60 percent of coarse aggregate have been successfully used in many localities, such mixtures behaved poorly in the first series of stability tests. An unintentional enrichment in bitumen caused by penetration of the seal coat was probably chiefly responsible for the failures which occurred, but it is believed that the more densely graded mixtures, successfully used in both series of tests, are inherently

(Continued on p. 218)

RELATIVE VISCOSITIES OF LIQUID ASPHALTIC ROAD MATERIALS AT VARIOUS TEST TEMPERATURES

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A REVIEW of the 1931 State specifications, submitted to the Bureau of Public Roads in connection with the cooperative work on the simplification of tests for liquid asphaltic materials, showed that there was considerable difference of opinion as to the temperatures at which viscosity determinations should be made. The following tabulation shows the various temperatures designated for the test and the number of specifications in which each temperature was designated

Designated temperature (°F.)	Number of specifications
77	39
104	12
122	87
140	16
194	2
210	1
212	7
302	1

The consistency of material for hot application was controlled not only by viscosities at the higher temperatures (194° F., 212° F., and 302° F.), but also by the float test at 90° or 122° F. and, in some cases, by both a viscosity and a float test.

WIDE DIFFERENCES OF OPINION EXPRESSED AS TO TEMPERATURE FOR VISCOSITY TESTS

At regional meetings of State testing engineers for consideration of the various tests, the Saybolt-Furol method of viscosity determination was adopted. The temperatures recommended by most of these groups for viscosity tests were 77° F., 122° F., and 210° F. There were, however, some who favored the float test at 122° F., as a single measure of consistency instead of the Furol viscosity test at 210° F.

There was a fairly general agreement on the choice of 122° F. as a test temperature with the following exceptions:

(a) Two States voted for the use of 104° F. as a test temperature instead of 122° F.

(b) Eight States of the Mississippi Valley region, although they did not vote for the Furol viscosity determinations at temperatures other than 77° F., 122° F., and 210° F., expressed the belief that some temperature between 122° F. and 210° F. was desirable.

(c) Seven States voted for the use of 140° F. as a control temperature for viscosities in addition to the three temperatures generally accepted.

The 1932 State specifications also show a wide difference of opinion as to the proper temperature to be used for viscosity determinations. In addition to the temperatures already listed, there have been specified during 1932 two others—180° F. and 200° F. It is apparent that there is considerable doubt as to the sufficiency of 122° F. as the single intermediate temperature; and there is a question as to whether a high-temperature viscosity test for the control of materials for hot application could not be discarded for some other consistency test.

The 1931 specifications gave the following maximum viscosity limits as determined with the Saybolt-Furol apparatus:

Temperature (°F.)	Maximum time, seconds
77	480
104	436
122	1,200
140	260
194	440
210	180
212	320

One of the important advantages of the Furol instrument over the Engler is the shorter time required to make a viscosity determination. This time advantage should be retained by the selection of test temperatures such as will insure accurate determinations in a reasonable time. In the above table the maximum time for the test at 122° F. is much greater than for the other temperatures. The maximum time of 1,200 seconds, or 20 minutes, shows the extent to which the time advantage of the Furol instrument may be lost.

VISCOSITY TESTS AT 210° F. WILL NOT FURNISH SHARP DIFFERENTIATION BETWEEN MATERIALS FOR RETREAD AND ROAD-MIX CONSTRUCTION

While there were only 12 specifications in 1931 having maximum viscosity limits of over 500 seconds, Furol at 122° F., there is nevertheless a trend toward the use of more viscous materials. Materials, considerably thinner than those for hot application, are used extensively in retread and oiled road-mix construction. The viscosities of these asphaltic products are very high at 122° F. The next higher temperature recommended at the regional meetings was 210° F. Tests were run at this temperature on a selected number of road oils and kerosene cut-back asphalts to determine the suitability of the temperature for control of such materials. Float tests at 122° F. were also made on the same samples.

The Furol viscosity at 122° F. for liquid asphaltic road materials of the slow-curing type may be compared with the Furol viscosity at 210° F. and the float test results at 122° F. from the data given in table 1 and plotted in figure 1. The materials tested were from many different fields and were, no doubt, produced by quite different methods of processing. No satisfactory curve could be drawn in figure 1 to show the relationship for the different consistency values, although a definite trend is indicated.

The same relationships for kerosene cut-backs are given in table 2 and figure 2. Although the sources of these cut-backs are all different, a fairly satisfactory curve can be drawn to show the relationship between viscosities at 122° F. and viscosities at 210° F. and between viscosities at 122° F. and float tests at 122° F.

These relationships may be expressed approximately as follows:

$$C = 0.03\,B + 18$$
$$D = 0.02\,B + 10$$

Where B = Furol viscosity at 122° F.
C = Furol viscosity at 210° F. and
D = Float test at 122° F.

The narrow range in viscosities at 210° F. and in float-test results at 122° F. for materials which had a wide range of viscosity at 122° F. indicates that these tests (viscosity at 210° F. and float test at 122° F.) will not furnish sharp differentiation between various consistencies.

140° F. SUITABLE TEMPERATURE FOR VISCOSITY TESTS FOR MATERIALS HAVING HIGH VISCOSITY AT 122° F.

To determine a suitable temperature for viscosity tests on materials for retread and road mix construction, five selected kerosene cut-backs, having appreciable differences in viscosity at 122° F., were tested for viscosity at the following temperatures: 122° F., 140° F. 158° F., 176° F., 194° F., and 210° F. The results of these determinations are given in table 3 and are shown graphically in figure 3.

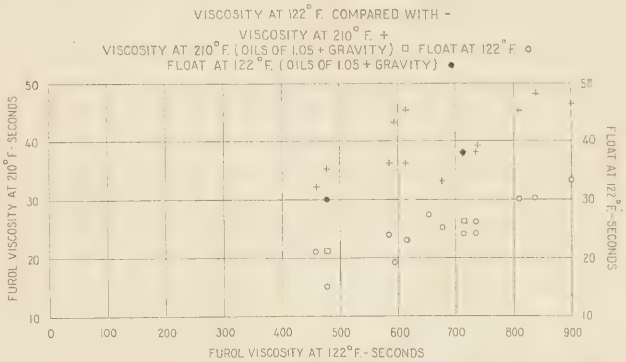


FIGURE 1.—VISCOSITY OF SLOW-CURING LIQUID ASPHALTS AT 122° F. COMPARED WITH VISCOSITY AT 210° F. AND FLOAT-TEST RESULTS AT 122° F.

TABLE 1.—*Furol viscosities of liquid asphaltic road materials of the slow-curing type at 122° F., 140° F., and 210° F., and float tests at 122° F.*

Sample no.	Producer	Specific gravity	122° F.		Viscosity at 140° F.	Viscosity at 210° F.	Float test at 122° F.	Results of other operators at 122° F.	
			Viscosity	Time of drip				Maximum viscosity	Minimum viscosity
			Sec.	*Sec.*	*Sec.*	*Sec.*	*Sec.*	*Sec.*	*Sec.*
34,453	Utah Refining Co.	1.079	68		36				
35,364	Shell, Wood River	.942	68		47			68	64
34,263	White Eagle	1.066	94		48			105	90
35,100	Shell, East Chicago	1.001	94		53			91	89
35,077	Shell, Martinez	.951	111		63			111	111
36,303	Standard Oil Co. of California, Bakersfield	.967	151		80			[1] 150	150
36,106	Standard Oil Co. of California, Richmond	.952	178		94			[1] 174	174
35,367	Shell, Wood River	.955	199		106			194	185
35,226	Gilmore, Los Angeles	.967	201		104			198	198
35,103	Shell, East Chicago	1.003	205		110			196	192
36,310	Standard Oil Co. of California, Bakersfield	.972	274		134			[1] 280	280
35,080	Shell, Martinez	.963	281		142			300	261
35,157	White Eagle	.976	294		152			308	245
34,454	Utah Refining Co.	1.101	297		123				
35,225	Gilmore, Los Angeles	.972	300		148			311	255
36,089	Standard Oil Co. of California, Richmond	.960	300		149			300	287
34,322	Lincoln Oil Co.	1.082	320		137			345	292
36,091	Standard Oil Co. of California, Richmond	.966	458		220	32	21	500	444
34,323	Lincoln Oil Co.	1.088	478		188	21	30	496	444
36,360	Berry	.981	478	405	243	35	15		
34,467	Union, Oleum		586	440	268	36	24	580	530
36,946	MacMillan	.979	596	480	301	43	19	566	564
35,081	Shell, Martinez	.972	613	(2)	288	36	23	630	616
35,221	Texas, Port Neches	.975	616	540	300	45	23	627	567
36,300	Standard Oil Co. of California, Bakersfield	.981	677	400	295	33	25	720	680
35,155	Standard Oil Co. of Indiana, Wood River	1.087	713	560	271	26	38	707	660
36,211	Standard Oil Co. of California, El Segundo		716	480	325	38	24	[1] 798	798
35,228	Gilmore, Los Angeles	.979	735	(2)	333	38	26	[1] 706	706
36,214	Standard Oil Co. of California, El Segundo		737	450	333	39	24	760	730
35,183	Standard Oil Co. of Indiana, Greybull	.982	808	540	384	45	30	831	790
34,279	White Eagle		838	600	392	48	30	852	805
36,426	Texas, Cody	1.003	898	600	410	46	33	908	850

[1] Only 1 determination.
[2] Not recorded.

TABLE 2.—*Furol viscosities of kerosene cut-backs at 122° F., 140° F., and 210° F., and float test at 122° F.*

Sample no.	Producer	Location	122° F.		140° F.		Viscosity at 210° F.	Float test at 122° F.	Results of other operators at 122° F.	
			Viscosity	Time of drip	Viscosity	Time of drip			Maximum viscosity	Minimum viscosity
			Sec.	*Sec.*	*Sec.*	*Sec.*	*Sec.*	*Sec.*	*Sec.*	*Sec.*
36,306	Standard Oil Co. of California	Bakersfield	488		238		33	20	550	530
36,095	do	Richmond	787	540	366		43	25	820	750
36,309	do	Bakersfield	999	550	432	410	48	30	1,080	991
35,114	Union Oil Co.	Oleum	1,020	500	445	420	49	33	964	806
35,113	do	do	1,023	600	449	420	47	30	1,105	950
36,097	Standard Oil Co. of California	Richmond	1,120	600	486	425	54	30	1,175	1,100
35,112	Union Oil Co.	Oleum	1,247	600	542	435	56	33	1,238	1,015
36,524	Associated	California	1,393	700	567	450	55	36	1,450	1,249
36,099	Standard Oil Co. of California	Richmond	1,400	600	600	435	62	35		
36,094	do	do	1,411	630	559	450	59	34	1,542	1,400

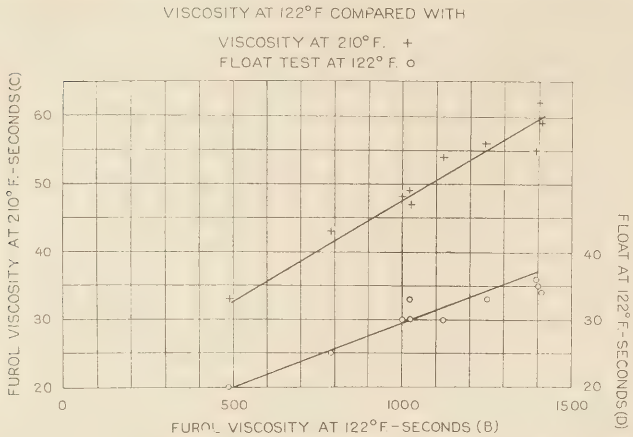


FIGURE 2.—VISCOSITY OF KEROSENE CUT-BACKS (MEDIUM-CURING TYPES) AT 122° F. COMPARED WITH VISCOSITY AT 210° F. AND FLOAT-TEST RESULTS AT 122° F.

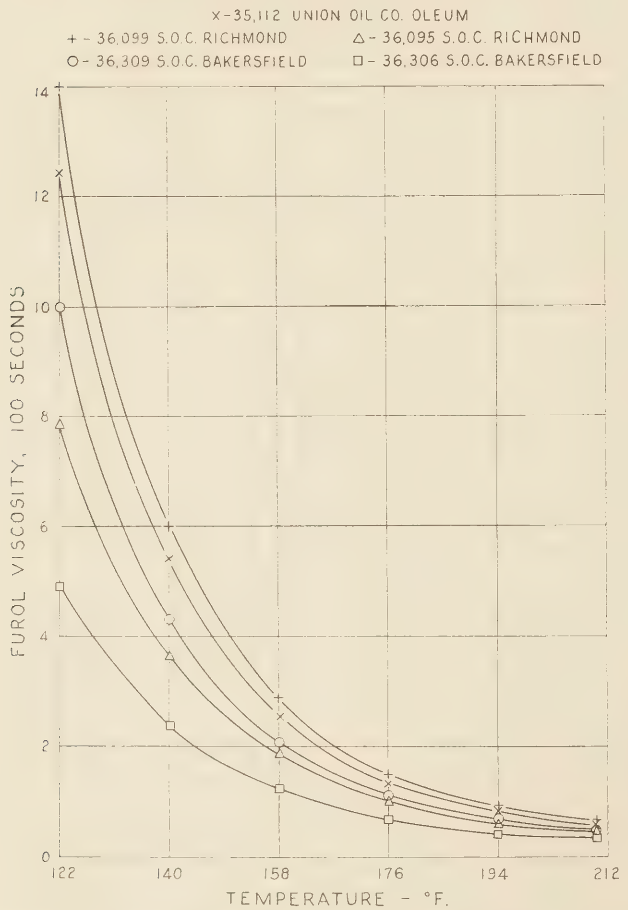


FIGURE 3.—TEMPERATURE-VISCOSITY CURVES FOR CALIFORNIA CUT-BACKS (MEDIUM-CURING TYPES).

An 18° increase in temperature from 122° F. to 140° F. decreases the time of the Furol viscosity more than one half even for the least viscous material. A 140° F. temperature has been used by some of the States and is being considered for use by other States, and it was thought desirable to investigate the viscosity values at this temperature of a considerable number of liquid asphaltic road materials of the slow, medium, and rapid-curing types.

The data for viscosities at 140° F. for materials of the slow-curing type are given in table 1, and for materials of the medium-curing type in table 2. Naphtha

TABLE 3.—*Furol viscosities of kerosene cut-backs at various temperatures*

Sample no.	Producer	Viscosity at— 122° F.	140° F.	158° F.	176° F.	194° F.	210° F.
		Sec.	*Sec.*	*Sec.*	*Sec.*	*Sec.*	*Sec.*
36,306	Standard Oil Co. of California, Bakersfield	488	238	121	69	44	33
36,095	Standard Oil Co. of California, Richmond	787	366	187	101	62	43
36,309	Standard Oil Co. of California, Bakersfield	999	432	207	111	67	48
35,112	Union Oil Co., Oleum	1,247	542	258	136	81	56
36,099	Standard Oil Co. of California	1,400	600	290	151	88	62

cut-backs (materials of the rapid-curing type) were not used in the preliminary work, but a number of samples of various consistencies from different producers have been tested at 140° F. as well as 122° F. The data on these naphtha cut-backs are given in table 4. The data for liquid asphaltic road materials of the slow-curing type are plotted in figure 4, for kerosene cut-backs in figure 5, and for naphtha cut-backs in figure 6.

In drawing the curve for materials of the slow-curing type in figure 4, it was found that some of the points differed greatly from the general trend of results, and, in checking the test reports on these samples, it wa found that these particular materials all had a hig specific gravity (greater than 1.05), while the gravities of the other materials were all less than 1.003. Tw curves were therefore drawn and the relationship may be expressed approximately as follows:

$A = 0.45\ B + 15$, for the low gravity group,
$A = 0.36\ B + 15$, for the high gravity group,
Where A = Furol viscosity at 140° F., and
B = Furol viscosity at 122° F.

The relationship for the kerosene cut-backs, as show in figure 4, may be expressed approximately as follows:

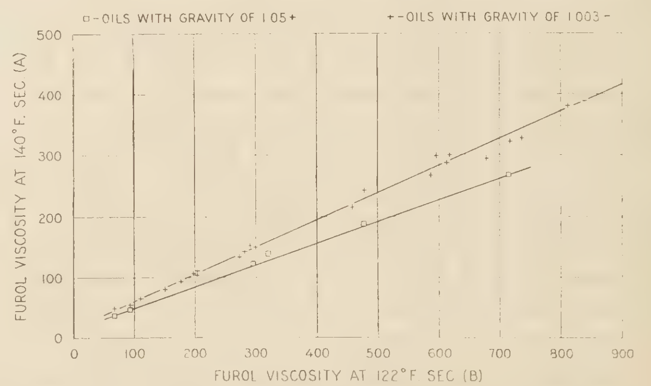


FIGURE 4.—VISCOSITY OF SLOW-CURING LIQUID ASPHALTS AT 122° F. COMPARED WITH VISCOSITY AT 140 F.

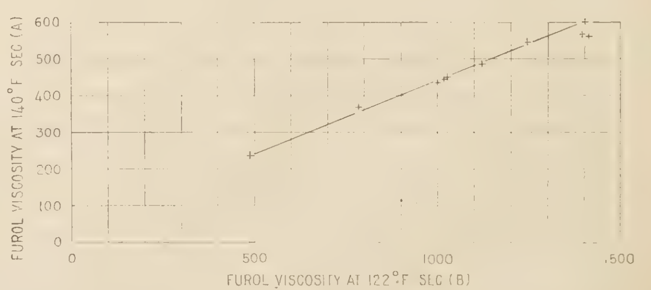


FIGURE 5.—VISCOSITY OF KEROSENE CUT-BACKS (MEDIUM-CURING TYPES) AT 122° F. COMPARED WITH VISCOSITY AT 140° F.

$$A = 0.4\,B + 40$$

Where A = Furol viscosity at 140° F., and
B = Furol viscosity at 122° F.

The relationship for naphtha cut-backs, as shown in figure 6, is approximately as follows:

$$A = 0.4\,B + 60$$

Where A = Furol viscosity at 140° F. and
B = Furol viscosity at 122° F.

TABLE 4.—*Furol viscosities of naphtha cut-backs at 122° F. and 140° F.*

Sample no.	Producer	Viscosity at—		Results of other operators at 122° F.	
		122° F.	140° F.	Minimum	Maximum
		Sec.	*Sec.*	*Sec.*	*Sec.*
35486	Shell, Norco	420	226	426	455
34286	Barber	662	338	651	725
34314	Standard Oil Co. of New Jersey, Baltimore	680	358	729	804
35208	Standard Oil Co. of Louisiana	681	325	685	702
34377	White Eagle	688	322	557	792
35347	Standard Oil Co. of Indiana, Whiting	689	327	676	809
35478	Texas, Port Neches	725	334	577	844
36059	Shell, Norco	730	360	604	875
36058	do	854	432	756	1,060
35207	Standard Oil Co. of Louisiana	919	418	895	914
34317	Standard Oil Co. of New Jersey, Baltimore	919	404	900	936
35474	Texas, Port Neches	960	430	844	1,060

It is believed that a continuous flow through the aperture of the viscosimeter is necessary to obtain an accurate viscosity determination. The control temperature should be so adjusted as to avoid the dripping which occurs when high viscosities are run. In table 1 there is dripping on all viscosities over 478 at 122° F., and when these same samples are run at 140° F. no dripping occurs.

A majority of the kerosene cut-backs listed in table 2 have much higher viscosities than the road oils listed in table 1, and drip at both 122° F. and 140° F., although the difference between the time of drip and the viscosity value is much smaller at 140° F. than at 122° F. The time of drip was not recorded in the naphtha cut-back determinations, but it is probable that dripping did occur on the more viscous materials at both temperatures.

It is evident from a study of the results of "other operators" in viscosity determinations at 122° F. (on same materials tested in other laboratories) that the difference in test values in cases of the heavier materials (tables 1, 2, and 4) are quire large. It is reasonable to suppose that, if accurate temperature control is obtained, the differences in viscosity values obtained by different operators should be materially reduced by the use of 140° F. as a temperature control. The total time for the viscosity determination is greatly shortened, and uncontrollable factors which affect the accuracy of the determinations have considerably less time to influence the flow of oil into the receiver. Since the time of running the test at 140° F. is greatly reduced and the dripping is less, check tests by different operators should be in closer agreement.

Since 140° F. also represents the approximate maximum temperature reached in the upper portion of the road under summer temperature, it is thought that it is a desirable intermediate temperature for the control of the more viscous products which have high Furol viscosities at 122° F.

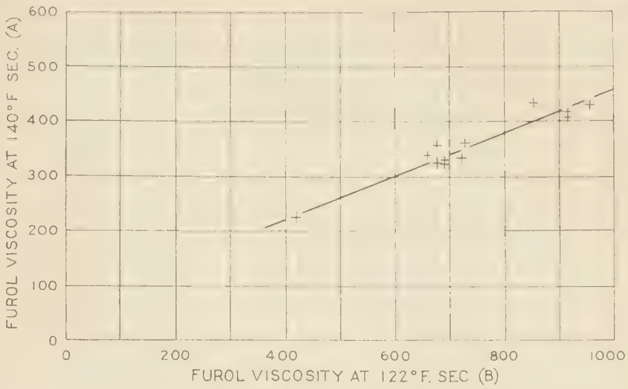


FIGURE 6.—VISCOSITY OF NAPHTHA CUT-BACKS (RAPID-CURING TYPE) AT 122° F. COMPARED WITH VISCOSITY AT 140° F.

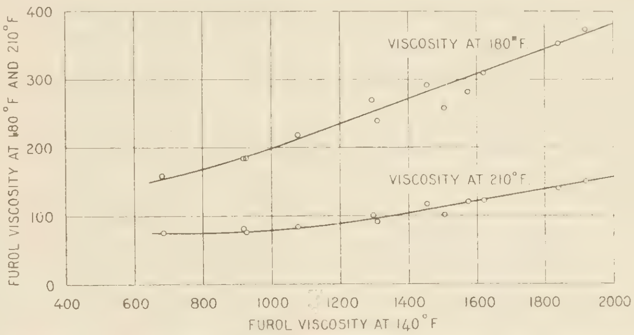


FIGURE 7.—VISCOSITY OF KEROSENE CUT-BACKS (MEDIUM-CURING TYPE) AT 140° F. COMPARED WITH VISCOSITIES AT 180° F. AND 210° F.

Recently very viscous liquid asphaltic road materials of the medium-curing and rapid-curing types have been used as binders in road mixes. Mixtures are prepared in a portable mixing plant and immediately laid and compacted. These materials have exceedingly high viscosities at 140° F. If naphtha has been used as the cutting agent in such products, it is not thought advisable to make the viscosity determination at a temperature above 140° F. because of the possible loss of volatile matter and subsequent stiffening of the material during the test. If kerosene is the solvent used in these cut-backs, a viscosity determination at some temperature higher than 140° F. is not only possible but is desirable because of the shorter time involved in making the test. Several states using material of this grade have designated 180° F. as the control temperature for the viscosity determination.

The Furol viscosities of extremely viscous medium-curing materials at 140° F., 180° F. and 210° F., as well as float tests at 122° F., are given in table 5. Curves showing the relationship existing are plotted in figure 7. It is evident that material having a high viscosity at 140° F. can be satisfactorily controlled by a Furol viscosity at 180° F. and that the use of 210° F. as a viscosity temperature control for this type of material is not necessary.

VISCOSITY TEST AT HIGH TEMPERATURE NOT NEEDED FOR MATERIALS FOR HOT SURFACE TREATMENT

The consistency requirements for materials to be used in hot surface treatments are generally specified by viscosities at one of the higher temperatures or by a float test at 90° F. or 122° F. In many cases the consistency is controlled by both a viscosity at a high temperature and a float test.

The method of applying these hot materials to the road surface is similar to that used in constructing

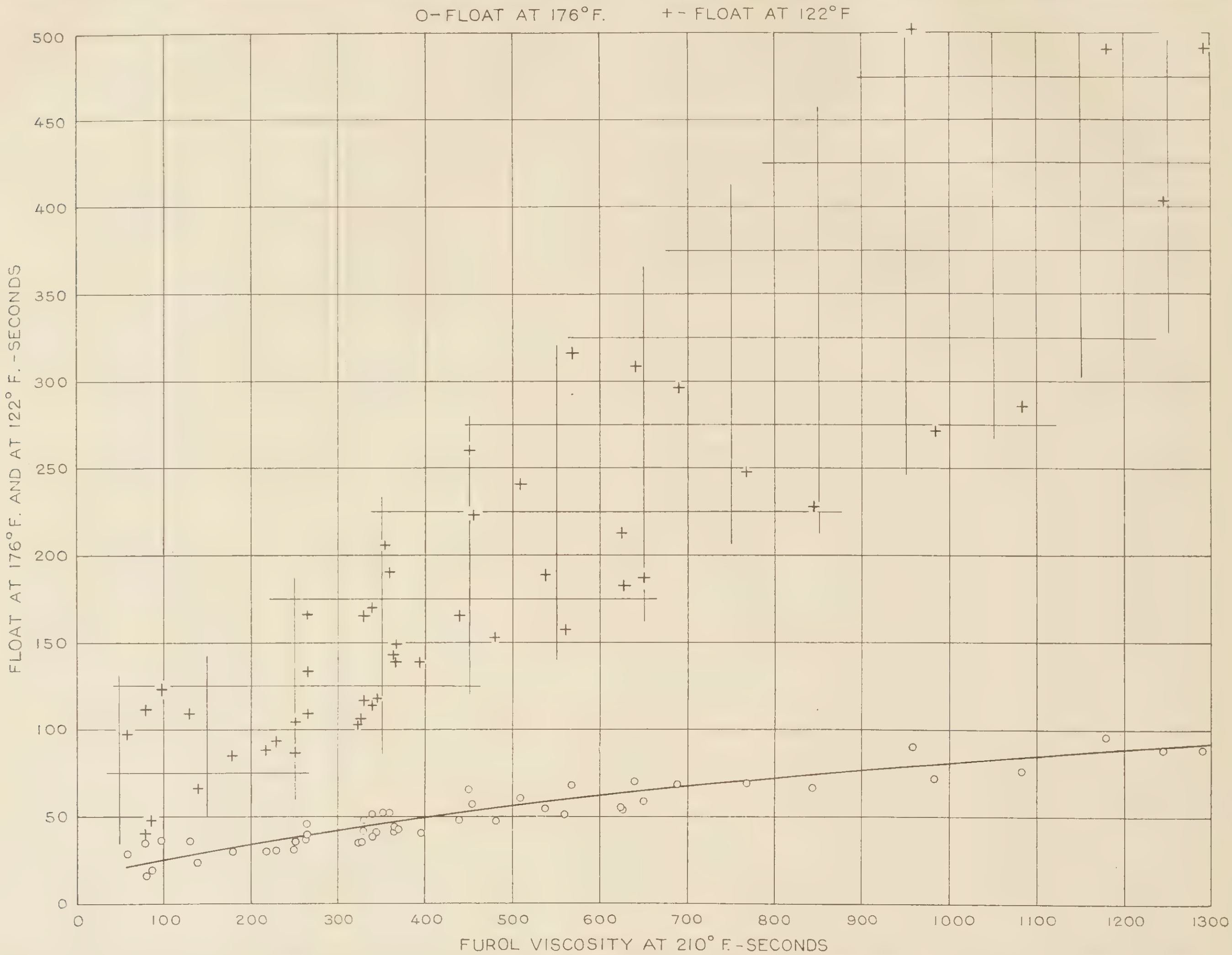


FIGURE 8.—VISCOSITY OF MATERIAL FOR HOT SURFACE TREATMENT AT 210° F. COMPARED WITH FLOAT-TEST VALUES AT 122° F. AND 176° F.

penetration macadam. No viscosity requirement is made in specifying materials for penetration macadam.

It is believed that a viscosity requirement at high temperature for material for hot surface-treatment material can be discarded from specifications and that some test measuring the consistency of the material at a lower temperature can be substituted.

TABLE 5.—*Furol viscosities and results of float tests on kerosene cut-backs at various temperatures*

Sample no.		Producer	Viscosity at—			Float test at 122° F.
Base	Flux		140° F.	180° F.	210° F.	
36, 769	36, 608	Mexican Petroleum Co. (Baltimore)	1, 920	372	148	75. 5
			1, 840	352	139	70. 0
			1, 455	290	116	58. 0
			682	159	75	40. 0
34, 326	(1)	White Eagle (Casper)	1, 622	309	122	78. 0
			1, 576	281	121	------
			1, 298	270	100	68. 5
			920	183	81	55. 0
35, 120	35, 122	Union Oil Co. of California	1, 506	256	100	67. 0
			1, 310	240	90	64. 5
			1, 077	218	84	59. 0
			925	183	76	50. 5

[1] Kerosene, no number.

With this idea in mind, a large number of liquid asphaltic road materials suitable for hot surface treatment were tested for Furol viscosities at 210° F., for float at 122° F., 140° F., 158° F., and 176° F., and penetration at 77° F. under variable load and time. The results of this series of tests are given in table 6 and the relationship between Furol at 210° F. and float test at 122° F. and 176° F. are shown in figure 8. The effect of change in temperatures on the float test values for a selected number of samples are shown in figure 9.

It is evident that the float test at 122° F. has no direct relationship to Furol viscosity at 210° F. for the materials tested, although data not included in this report indicate that there is a definite relation between time of float test at 122° F. and viscosities at 210° F. if the materials have been produced from the same base materials and by the same process. If the float tests are made at a higher temperature, the test becomes more nearly a measure of consistency for all types of material than is the case at low temperatures. As is shown in figure 8, a fairly satisfactory curve can be drawn to show the relationship between Furol viscosity at 210° F. and float test results at 176° F.

TABLE 6.—*Furol viscosities at 210° F., float tests at 122° F., 140° F., 158° F., 176° F., and penetrations at 77° F. for slow-curing materials suitable for hot surface treatment*

Sample no.	Producer	Viscosity at 210° F.	Float test at—				Penetrations at 77° F.			
							50 grams		100 grams	
			122° F.	140° F.	158° F.	176° F.	5 sec.	1 sec.	5 sec.	1 sec.
		Sec.	*Sec.*	*Sec.*	*Sec.*	*Sec.*				
35145	Standard Oil Co. of Indiana, Wood River	60	97	55	40	28	--------	249	--------	--------
35265	Standard Oil Co. of New Jersey, Bayonne	83	39	27	21	16	--------	(1)	--------	--------
35356	Standard Oil Co. of Indiana, Whiting	83	111	74	48	35	--------	180	--------	--------
34254	The Texas Co., Bayonne	89	48	32	23	18	--------	(1)	--------	--------
35196	Standard Oil Co. of New Jersey, Parkersburg	100	123	77	51	36	310+	189	--------	--------
35353	Standard Oil Co. of Indiana, Whiting	133	108	70	48	35	--------	191	--------	--------
36044	Jas. B. Berry	143	66	42	32	24	--------	(1)	--------	--------
35266	Standard Oil Co. of New Jersey, Bayonne	181	85	56	36	30	--------	(1)	--------	--------
35267	do	220	88	58	40	30	--------	(1)	--------	--------
34248	The Texas Co., Bayonne	232	93	60	41	30	300+	(1)	--------	--------
36343	Shell Oil Co., Norco	253	87	64	42	32	--------	(1)	--------	--------
35048	Standard Oil Co. of New Jersey, Baltimore	254	104	74	50	35	--------	242	--------	--------
36307	Standard Oil Co. of California, Bakersfield	266	166	107	68	46	294+	105	--------	--------
36863	Mexican Petroleum Corporation, Baltimore	267	109	74	50	37	--------	(1)	--------	--------
36216	Standard Oil Co. of California, El Segundo	267	133	83	57	39	283+	175	--------	--------
34497	The Texas Co., Port Neches	326	103	71	50	35	--------	(1)	--------	--------
36187	Atlantic Refining Co., Brunswick, Ga	328	105	76	50	35	--------	266	--------	--------
36085	Mexican Petroleum Corporation, Destrahan, La	331	117	79	57	41	--------	249	--------	--------
36093	Standard Oil Co., Richmond	333	165	104	69	48	285+	118	--------	--------
36759	The Texas Co., Bayonne	342	115	75	54	38	--------	(1)	--------	--------
35083	Shell Oil Co., Martinez, Calif	342	171	111	75	51	277+	111	303+	200
35335	Standard Oil Co. of New York, Riverside	347	118	80	56	40	--------	268	--------	--------
36311	Standard Oil Co. of California, Bakersfield	355	206	119	82	52	193	83	232	--------
35115	Union Oil Co., Oleum	362	191	114	76	52	242	115	291+	199
37323	The Texas Co., Bayonne	366	142	92	63	42	255+	149	--------	--------
35348	Standard Oil Co. of Indiana, Whiting	367	139	89	65	43	315+	263	--------	--------
35049	Standard Oil Co. of New Jersey, Baltimore	369	149	97	65	42	281+	172	--------	--------
35013	Standard Oil Co. of Ohio	398	139	92	61	41	290+	169	--------	--------
35192	Standard Oil Co. of New Jersey, Baltimore	442	166	103	72	48	283+	132	--------	--------
35082	Shell Oil Co., Martinez, Calif	452	260	140	96	67	157	63	210	--------
35229	Gilmore Oil Co	456	223	133	87	57	158	79	250	--------
36702	Mexican Petroleum Corporation, Baltimore	484	153	100	71	47	290+	155	--------	--------
36092	Standard Oil Co. of California, Richmond	512	242	136	88	61	146	63	194	--------
36320	Standard Oil Co. of Louisiana	540	189	120	84	55	237	120	295+	186
36344	Shell Oil Co., Norco	563	158	108	77	52	309+	167	--------	--------
36525	Associated Oil, Avon	570	316	159	105	68	126	52	160	--------
34496	The Texas Co., Port Neches	627	212	124	86	56	222	136	305+	199
34273	Colonial Beacon Oil Co	628	183	111	82	55	273	144	302+	240
35116	Union Oil Co., Oleum	644	308	159	97	70	127	69	175	--------
36086	Mexican Petroleum Corporation, Destrahan, La	653	187	115	83	59	266	136	292+	198
35156	White Eagle	693	297	157	102	69	120	60	180	--------
36319	Standard Oil Co. of Louisiana	770	248	147	99	69	165	92	235	--------
36345	Shell Oil Co., Norco	846	228	133	93	67	238	120	298+	195
36215	Standard Oil Co. of California, El Segundo	960	501	213	129	90	85	41	121	--------
36353	Atlantic Refining Co., Brunswick, Ga	985	271	147	100	72	188	95	264	--------
36087	Mexican Petroleum Corporation, Destrahan, La	1,085	285	165	109	76	159	85	220	--------
35230	Gilmore Oil Co	1,181	491	221	133	96	97	43	158	--------
36318	Standard Oil Co. of Louisiana	1,248	403	190	124	88	126	65	169	--------
34495	The Texas Co., Port Neches	1,293	491	221	131	88	110	61	162	--------

[1] Too soft for test.

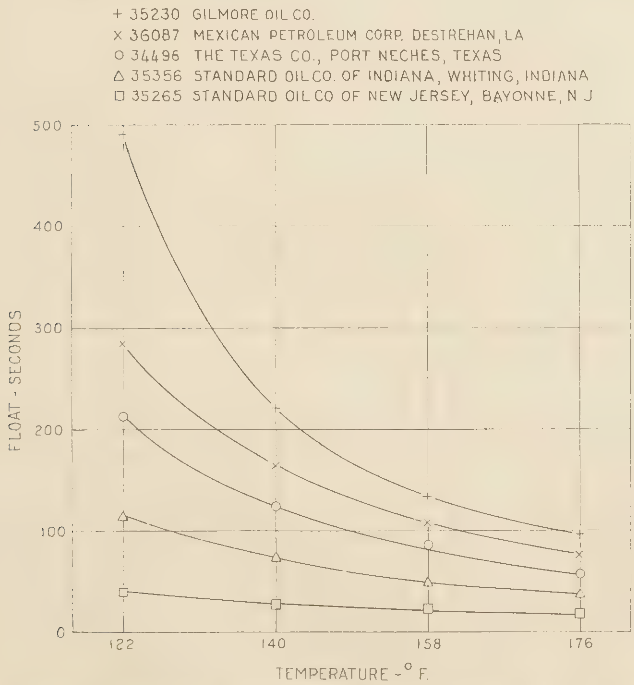


FIGURE 9.—RELATION BETWEEN TEMPERATURE AND FLOAT-TEST VALUES FOR MATERIALS FOR HOT SURFACE TREATMENT.

The one outstanding feature developed by these consistency tests is that although the materials tested are often considered as fluid asphaltic materials, 14 of the group tested gave penetrations at 77° F. (100 grams, 5 seconds), from 264 to as low as 121, and that only 10 may be considered as liquid bituminous materials as such materials are defined by the American Society for Testing Materials. According to this definition only those materials having a penetration at 77° F. under a load of 50 grams applied for 1 second, of more than 350, are liquids. The viscosity determination at high temperatures for materials for hot-surface treatment is unnecessary. A float test could be used, but for soft semisolid asphalts a penetration test at 77° F. under a load of either 50 or 100 grams seems to offer the better means for laboratory control and avoids the use of viscosity tests at high temperatures.

CONCLUSIONS

1. There is need for tests at a number of temperatures to adequately cover the wide range in consistency of liquid asphaltic road materials.

2. The use of 140° F. and 180° F. should prove satisfactory as control temperatures in Furol viscosity determinations for the more viscous liquid asphaltic road materials.

3. When practicable the temperature designated for control should give a Furol viscosity of less than 500 seconds. However, the more viscous materials of the rapid-curing type should not be tested for viscosity at a temperature higher than 140° F.

4. Many so-called liquid asphaltic road materials, which are used in hot surface-treatment work, and which are designated as hot oils, 90 and 95 percent road oils, etc., are in fact semisolid asphaltic materials. These grades of material probably can be controlled best for consistency by either a float test at a low temperature or by a penetration test at 77° F. under a load of 50 or 100 grams.

5. Tests at a temperature of approximately 210° F. can be omitted readily from specifications for liquid asphaltic road materials.

(Continued from p. 211)

stronger and better suited to severe traffic conditions such as were imposed.

17. Sand considerably coarser than is generally employed in asphalt pavement construction, was satisfactorily used in the stable, dense, coarse-graded asphaltic concretes of the first series of tests.

18. No influence of the consistency of the asphalt cement upon the service characteristics of the coarse-graded asphaltic concretes was apparent. Stable mixtures were laid with asphalts ranging from 45 to 75 penetration.

19. The five sections of asphaltic concrete of the second series of tests satisfactorily carried bitumen to the extent of 85 percent to 95 percent of the volume of the aggregate voids. Aggregate voids were determined for slabs removed from the traffic lanes after the service tests had been completed.

20. The stabilities determined by the roller stability machine conform to the observed service characteristics of the asphaltic concretes tested in the second series of tests. This apparatus constitutes a means by which mixtures of this type may be conveniently studied and compared.

LIMITED APPLICATION OF TEST RESULTS EMPHASIZED

Attention has been called to the reasons for these tests and to the conditions under which they were conducted, but it may be well to emphasize their limited scope and to warn against unconsidered application of the data and observations presented. At no time was it anticipated that a complete method of bituminous pavement design would be developed from these tests. It was desired merely to determine under controlled traffic conditions the relative traffic resisting capacities of a wide range of mixture compositions for correlation with laboratory stability tests which were in process of development. This aim was realized but the data collected were naturally studied with relation to existing theories.

There are several reasons why the indications of these experiments cannot be applied indiscriminately to all problems of mixture design. The property of stability is but one characteristic which a mixture should possess. To design for the highest possible stability may result in the adoption of a mixture which is unduly expensive, greatly deficient in bitumen or otherwise unbalanced, and consequently not the best suited to the specific conditions under which it is to be used. Very high stabilities, such as those apparently necessary to resist the exceptionally severe traffic of these tests, are not generally essential. It was found that a minimum stability of about 2,000 pounds was required on certain heavy-traffic New York City streets and under light to moderate traffic on country roads 1,000 pounds stability, or even a little less, has proved adequate to resist deformation. In such cases as the latter an attempt to attain unnecessarily high stabilities might yield pavements which would crack excessively or which would be undesirably high in voids.

Materials from different sources vary widely in their capacity to impart stability to mixtures. Since these tests were conducted, research has demonstrated that the sands used in these tests possess greater stabilities than those from many other sections of the country. This characteristic is not a function of gradation, so that sands selected at random but duplicating the mesh composition of these experimental mixtures cannot be similarly proportioned in surface mixtures and trusted to develop corresponding stabilities.

Therefore the value of this study does not rest in the actual pounds of stability, percentage of voids, or other characteristics of the mixtures described, but rather in the indications of interrelationship which exist and which must be considered, weighed, and evaluated with reference to the controlling factors of traffic, materials, climate, etc., involved in any individual design problem.

A LABORATORY TRAFFIC TEST FOR LOW-COST ROAD TYPES

THE Bureau of Public Roads has recently built and placed in operation at the Arlington Experiment Farm a small circular test track for applying, in the laboratory, traffic tests to sections of highway surfaces. The test was designed primarily for the study of low-cost bituminous types, but it is believed that it may be adapted for other studies such as, for instance, subgrade stabilization, motor-vehicle tire wear, etc.

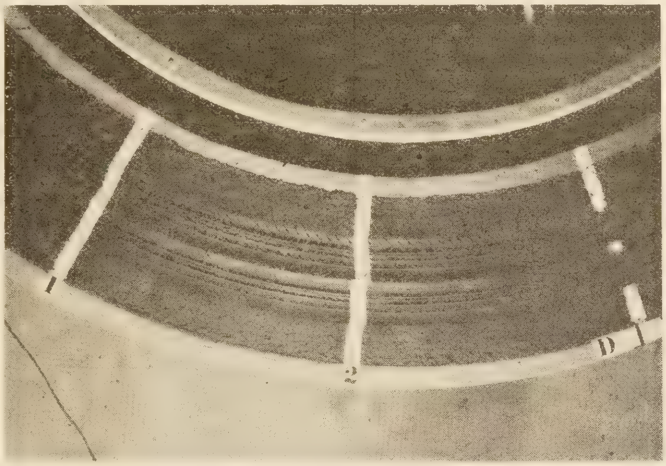
The track consists of an annular concrete trough 12 inches deep, 18 inches wide, and 12 feet in diameter at the center line. The depth is sufficient to permit the use of various combinations of base materials beneath the bituminous test surfaces. Along the smaller circumference of the trough in which the test sections are held, and cast intregally with it, is another trough 3 inches wide and 13 inches deep intended to be used as a reservoir for the introduction of water into the base material under the test surfaces through small openings at the base of the partition wall. By this arrangement the track may be flooded or the water may be introduced through capillarity.

Two full-size automobile wheels provide the traffic for the tests. These wheels are fixed to the two ends of a rigid structural member which is rotated in a horizontal plane by a vertical shaft in the pedestal at the center of the track. The upper end of this shaft is squared and on it rides a freely sliding square nut mounted in trunnions in the cross member. This arrangement causes a constant wheel load (that due to the weight of the wheels, tires, and cross member) to be applied at all times regardless of the irregularities of the test surfaces. At present this load amounts to about 800 pounds per tire. Although the distance between the two test wheels is fixed, a handwheel adjustment is provided which shifts the position of the square nut with respect to the midpoint of the cross member and enables the operator to place the path of either wheel at any point on the test surface.

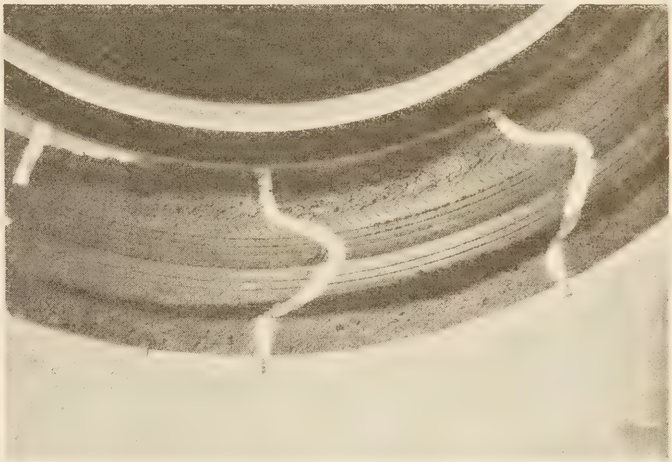
An electric motor operating through a 3-step cone pulley and a worm reduction drives the vertical shaft at the center of the track.

The test wheels may be operated at speeds of 4½, 6, or 9 miles per hour as desired. The low speed has proved to be most convenient when distributed traffic for compacting the surface is required. For the testing of the completed surfaces, concentrated traffic and the highest speed are used.

The number of trips made by each wheel is recorded by an electrical contact mechanism on the central vertical shaft operating a magnetic revolution counter at a point outside the track. In addition to this record, the data being collected include the corresponding behavior of the material under test, the density of the surface before and after test, oil migration, water content, and amount of material lost due to raveling. It is hoped that this information will make possible the evaluation of the important factors affecting the behavior of oiled aggregate mixtures.



The asphaltic material in this section has a Saybolt-Furol viscosity at 122° F. of 1,230.



The asphaltic material in this section has a Saybolt-Furol viscosity at 122° F. of 89.

Effect of Oil Consistency on the Stability of the Oiled Aggregate Surface Under Test Traffic.

The apparatus is now being used to investigate the effect of the percentage and consistency of the bituminous material on the durability and stability of mixtures with one type and grading of aggregate. A later phase of this first series of tests will involve a study of the effect of capillary water on the same mixtures. Various other factors influencing the behavior of different types of bituminous surface will be studied.

From the preliminary work which has been done up to this time it appears that the apparatus will provide a very useful method of studying some of the many factors involved in the performance of low-cost bituminous surfaces.

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS I—PROJECTS ON THE FEDERAL-AID HIGHWAY SYSTEM
OUTSIDE OF MUNICIPALITIES

AS OF DECEMBER 31,1933

STATE	PUBLIC WORKS FUNDS ASSIGNED FOR CLASS I PROJECTS ON THE FEDERAL AID HIGHWAY SYSTEM	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR NEW CLASS I PROJECTS
		Total cost	Public works funds	Regular Federal aid	Mileage	Estimated total cost	Public works funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	Public works funds allotted	Mileage	
Alabama	$ 4,185,067					$ 3,978,158.68	$ 2,190,403.56	$ 1,787,755.12	18.7	211.2	$ 525,638.08	57.8	$ 1,469,025.36
Arizona	3,804,731	$ 50,754.77	$ 50,754.77		.2	3,051,450.72	2,738,158.16		20.5	210.1	474,965.53	65.1	540,852.54
Arkansas	3,374,167					1,095,153.69	739,891.53	355,262.16	27.6	34.1	582,978.19	43.3	2,051,297.28
California	7,803,677					6,621,812.82	5,424,473.42		17.5	215.0	1,189,616.95	39.0	1,189,586.63
Colorado	3,437,265	198,864.32	198,864.32		14.8	1,652,364.39	1,652,364.39		36.9	83.4	223,152.15	4.8	1,362,884.14
Connecticut	1,404,213					186,337.04	186,337.04		19.3	3.0	1,217,874.96	27.7	1.00
Delaware	909,544	133,532.30	133,532.30		15.4	362,183.80	362,183.80		32.6	7.3	393,535.00	11.4	20,292.90
Florida	2,615,917					2,239,498.29	1,468,319.66	770,568.53	25.1	83.9	569,730.95	20.3	577,866.39
Georgia	5,045,592					781,431.96	781,431.96		32.1	30.8	1,728,129.94	109.0	2,536,030.10
Idaho	2,243,125	51,214.29	50,084.37		18.9	1,582,189.31	1,568,586.16		49.1	111.7	162,825.41	13.4	461,629.06
Illinois	4,431,348					630,863.60	630,863.60		4.1	17.0	571,573.50	9.9	3,228,910.90
Indiana	4,717,786					2,008,101.96	2,008,101.96		26.9	67.5	1,124,202.64	32.2	1,585,481.40
Iowa	5,027,830	729,099.63	704,400.00		30.1	3,944,046.72	3,716,000.00		29.2	209.4	465,830.00	22.6	141,600.00
Kansas	5,044,802	21,817.08	21,817.08		15.3	2,543,082.43	2,489,323.70		30.5	269.2	2,533,661.22	129.6	
Kentucky	3,608,332	51,941.61	51,941.61		1.6	1,554,677.14	1,554,677.14		29.8	123.3	1,070,787.76	78.8	930,925.49
Louisiana	2,914,295	12,626.07	12,626.07		.4	549,193.11	549,193.11		49.2	23.8	1,614,570.13	37.8	737,905.69
Maine	1,684,959					1,177,533.70	1,141,966.75		34.8	35.5	201,928.52	7.1	341,063.73
Maryland	1,782,263					156,305.93	146,657.18		21.1	4.8	644,837.85	10.0	990,767.97
Massachusetts	1,932,950	153,936.88	83,952.64	$ 69,984.24	4.2	1,155,771.39	860,635.58	295,135.81	14.5	32.8	66,479.60	.7	921,882.18
Michigan	5,094,491	35,500.00	35,500.00		1.2	1,636,150.00	1,636,150.00		37.3	106.1	2,618,841.00	96.0	804,000.00
Minnesota	5,115,153	996,229.46	996,229.46		202.0	1,740,834.16	1,740,834.14		41.1	258.9	817,287.40	205.6	1,560,802.00
Mississippi	3,489,337					796,451.01	401,955.65	355,863.42	26.4	42.1	1,614,345.35	151.9	1,473,036.00
Missouri	6,090,153	92,412.78	64,236.45		5.7	2,863,692.42	2,575,645.77		52.5	93.2	2,095,993.03	73.3	1,354,277.75
Montana	4,463,849	176,156.03	176,156.03		24.0	3,334,381.70	3,154,381.70	100,000.00	29.3	247.0	575,446.01	46.8	557,865.26
Nebraska	3,914,481	363,958.41	258,455.96		138.7	3,638,894.08	2,798,913.98	108,538.00	37.7	167.1	743,211.79	63.6	113,899.27
Nevada	2,909,387	84,617.72	84,617.72		32.2	1,552,731.71	1,552,731.71		21.6	154.6			1,272,037.57
New Hampshire	954,919					515,633.37	515,633.37		27.2	9.1	109,759.09	1.5	329,526.54
New Jersey	3,065,137					791,445.80	791,445.80		10.4	9.7	433,517.15	10.8	1,840,174.05
New Mexico	2,896,467	419,595.83	419,595.83		68.9	1,964,317.48	1,964,317.48		25.9	164.5			512,553.69
New York	10,830,099	117,100.00	117,100.00		3.8	8,552,060.00	7,989,020.00	315,000.00	20.0	186.9	2,228,660.00	44.1	495,319.00
North Carolina	4,761,147	320,365.45	160,182.74	160,182.71	25.9	1,263,292.01	867,475.34	395,816.67	23.5	135.2	1,227,887.28	207.2	2,505,601.64
North Dakota	2,902,224	503,828.71	503,828.71		249.7	773,152.37	773,152.37		8.4	212.7	1,237,048.11	355.1	388,194.81
Ohio	6,968,066	99,220.00	99,220.00		1.3	5,576,960.00	5,576,960.00		31.1	147.8	908,510.00	23.9	383,376.00
Oklahoma	4,608,399					3,151,965.53	3,151,965.53		28.7	183.9	800,793.16	76.3	655,640.31
Oregon	3,053,448	198,986.16	171,092.66		36.2	2,167,525.70	2,147,119.70		24.6	97.6	463,754.46	36.9	271,481.18
Pennsylvania	5,757,978					3,220,112.69	3,220,112.69		8.8	81.1	1,673,600.51	37.1	864,264.80
Rhode Island	999,354	18,830.63	18,830.63		.5	969,153.21	969,153.21		23.7	20.0			11,370.16
South Carolina	2,729,583	1,740.69	1,740.69			1,281,635.82	1,278,906.20	2,729.62	26.4	142.0	728,171.83	58.3	720,764.28
South Dakota	3,005,739	802,741.64	802,741.64		105.0	1,121,651.99	1,083,711.21	37,940.78	6.2	206.6	416,778.24	45.7	702,507.91
Tennessee	4,246,309	131,887.64	72,591.00	59,296.64	5.1	1,930,521.70	1,413,800.96	516,720.74	17.5	63.6	1,419,893.89	69.2	1,340,023.15
Texas	12,122,012	152,449.73	126,731.89		39.9	6,288,047.46	5,129,350.89		22.0	570.7	2,405,079.12	274.4	4,460,850.10
Utah	2,097,354	773,633.06	773,633.06		97.5	910,466.51	910,466.51		38.7	69.2	413,254.43	23.7	
Vermont	931,919	47,983.81	47,983.81		6.1	639,978.50	639,978.50		32.0	30.5	67,220.10	4.3	176,736.59
Virginia	3,708,379	94,272.69	83,502.98	10,769.71	8.2	1,345,621.49	1,339,208.33	6,413.16	33.4	51.9	781,353.01	24.1	1,504,314.68
Washington	3,057,934	187,015.09	187,015.09		13.2	1,532,794.71	1,529,437.89	3,356.82	23.2	53.0	878,645.11	31.2	462,835.91
West Virginia	2,013,405	116,220.82	116,220.82		4.2	1,511,877.13	1,511,877.13		25.7	64.9	164,976.32	1.9	220,330.73
Wisconsin	4,862,441	298,477.25	288,174.42	1,190.00	15.7	2,438,924.84	2,417,424.84	21,500.00	37.6	100.3	530,917.03	38.3	1,625,924.71
Wyoming	2,250,663	543,414.19	457,300.00	60,000.00	107.4	1,491,217.76	1,417,600.00		16.2	302.0	118,182.61	29.2	257,580.39
District of Columbia													
Hawaii	1,683,956	25,988.65	12,994.33	12,994.32	.9	842,351.10	634,770.27	157,581.08	3.2	17.8	684,305.08	13.4	351,886.32
TOTALS	186,551,646	8,006,413.39	7,383,649.08	374,417.62	1,294.2	101,113,998.93	91,343,069.87	5,230,181.91	26.3	5,763.8	41,519,749.49	2,764.3	46,305,177.56

CURRENT STATUS OF NATIONAL RECOVERY ROAD CONSTRUCTION

AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS II—NATIONAL RECOVERY MUNICIPAL HIGHWAY PROJECTS

(ON EXTENSIONS OF THE FEDERAL-AID HIGHWAY SYSTEM INTO AND THROUGH MUNICIPALITIES)

AS OF DECEMBER 31,1933

STATE	NATIONAL RECOVERY FUNDS ASSIGNED FOR CLASS II PROJECTS IN MUNICIPALITIES	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF NATIONAL RECOVERY FUNDS AVAILABLE FOR NEW CLASS II PROJECTS
		Total cost	National recovery funds	Regular Federal aid	Mileage	Estimated total cost	National recovery funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	National recovery funds allotted	Mileage	
Alabama	$ 2,092,533					$ 173,815.13	$ 137,692.99	$ 34,122.14	55.2	5.3	$ 155,402.61	3.5	$ 1,799,437.40
Arizona	781,794					11,448.25	11,448.25		78.6	1.0	38,841.83	1.2	731,503.92
Arkansas	1,687,084					201,302.27	153,847.30	47,454.97	13.6	5.5	325,045.22	9.0	1,208,191.48
California	3,901,839					1,014,064.74	873,360.38		12.9	17.7	1,302,468.65	15.2	1,726,009.97
Colorado	1,718,633	$ 128,443.81	$ 128,443.81		3.3	120,803.48	120,803.48		16.6	2.4	385,474.01	7.7	1,083,911.70
Connecticut	802,407					391,427.79	391,427.79		42.4	5.2	264,579.80	3.3	146,399.41
Delaware	454,772	77,384.90	77,384.90		1.5	149,600.00	149,600.00			1.1			227,787.10
Florida	1,307,959					476,134.13	298,238.21	177,895.92	15.1	5.9	482,541.59	3.8	527,179.20
Georgia	2,724,620					62,205.00	62,205.00		17.7	3.1	563,180.25	20.5	2,099,234.75
Idaho	1,121,562	1,750.35	1,750.35		.6	313,857.19	308,462.79		14.9	6.5	61,135.69	1.8	750,213.17
Illinois	6,877,199	45,749.72	45,749.72		.6	1,643,952.03	1,643,952.03		4.9	17.9	3,873,452.74	35.9	1,314,044.51
Indiana	4,818,165					24,698.54	24,698.54			.4	103,720.85	2.1	4,689,745.61
Iowa	2,815,585	181,286.46	169,850.00		5.5	708,430.92	665,450.00		25.8	17.0	626,200.00	16.1	1,354,085.00
Kansas	2,522,401	2,114.16	2,114.16		.8	359,636.94	359,636.94		7.2	5.3	2,160,649.90	30.6	
Kentucky	2,029,687					20,886.00	20,886.00		76.6	.6	166,239.55	3.7	1,842,561.45
Louisiana	1,457,148					329,960.57	329,960.57		39.7	6.5	116,195.40	5.7	1,010,992.03
Maine	842,479	78,746.59	78,746.59		1.6	340,046.25	340,046.25		30.3	8.5	88,884.90	1.6	334,801.26
Maryland	891,132					16,788.76	12,638.76		79.1	.6			878,493.24
Massachusetts	4,136,382	53,896.49	30,896.49	$ 23,000.00	1.4	1,779,131.94	1,765,031.94	14,100.00	15.0	11.2	355,184.22	1.9	1,985,269.35
Michigan	4,457,679	180,700.00	180,700.00		2.3	460,770.00	458,620.00		5.2	10.8	1,278,651.00	11.8	2,539,708.00
Minnesota	3,410,102	425,560.77	425,560.77		27.7	683,767.45	683,767.45		54.7	30.3	571,860.46	23.2	1,728,913.32
Mississippi	1,744,669					143,412.49	86,345.12	57,067.37	42.9	3.5	310,934.06	16.0	1,347,389.82
Missouri	3,045,077	2,596.65	1,948.52		.2	634,565.30	622,092.91		36.0	7.0	694,172.95	16.1	1,726,862.62
Montana	1,115,962	2,878.39	2,878.39		.1	91,327.03	91,327.03		46.0	3.7	325,883.34	11.6	695,873.24
Nebraska	1,957,240	29,267.12	28,577.83		3.6	906,132.56	906,132.56		37.3	15.6	107,331.55	3.6	915,198.06
Nevada	500,051					50,304.27	50,304.27		89.5	1.2			449,746.73
New Hampshire	477,460					301,104.78	301,104.78		11.3	6.4	79,020.67	1.6	97,334.55
New Jersey	3,217,442					1,718,880.86	1,718,880.86		21.8	13.3	125,110.11	1.1	1,373,451.03
New Mexico	1,448,234					173,201.60	173,201.60		19.6	4.8	413,671.68	6.7	861,360.72
New York	7,837,865	115,700.00	115,700.00		2.2	4,234,800.00	4,153,900.00		15.0	32.6	2,713,185.00	22.6	855,080.00
North Carolina	2,380,573	42,316.20	42,316.20		3.3	67,744.99	67,744.99		51.7	5.4	120,728.85	9.9	2,149,782.96
North Dakota	1,451,112	1,899.97	1,899.97		1.0	47,202.69	47,202.69		36.0	6.1	179,202.94	7.2	1,222,806.40
Ohio	4,645,378	47,460.00	39,985.00		1.6	1,109,420.00	1,026,920.00		23.3	12.9	507,065.00	8.1	3,071,408.00
Oklahoma	2,304,200					184,648.94	184,648.94		20.6	3.8	381,288.79	11.1	1,738,262.27
Oregon	1,526,724					494,913.03	494,913.03		10.1	7.8	286,849.73	8.6	744,961.24
Pennsylvania	5,416,051	46,325.04	46,325.04		1.7	683,149.42	680,938.71		18.7	15.6	1,900,306.64	30.0	2,788,480.61
Rhode Island	499,677					60,871.25	60,871.25		9.9	1.2	234,011.36	3.0	204,794.39
South Carolina	1,364,791	24,919.48	24,919.48		1.7	67,316.48	66,911.47	405.01	13.5	6.8	258,213.61	10.3	1,014,746.44
South Dakota	1,502,870	69,938.16	69,938.16		4.2	159,132.03	159,132.03		9.4	6.9	128,623.53	4.8	1,145,176.28
Tennessee	2,123,155	19,992.06	19,992.06		1.1	111,814.03	111,814.03		5.4	3.0	380,265.41	4.7	1,611,083.50
Texas	6,061,006	768.11	768.11		1.3	770,728.17	672,514.39		18.4	34.5	1,129,111.70	44.3	4,258,611.80
Utah	1,048,677	351,502.28	351,425.28		8.8	167,620.89	166,661.89		31.2	4.8	86,689.92	2.1	443,899.91
Vermont	470,628	3,854.07	3,854.07		.6	258,045.10	256,943.99		20.2	7.4	169,994.42	4.0	39,835.52
Virginia	1,854,189	20,431.84	11,671.35	8,760.49	1.4	310,853.87	310,853.87		29.9	6.3	682,529.12	8.7	849,134.66
Washington	1,877,571	65,733.19	65,733.19		2.3	312,368.37	312,368.37		17.9	6.6	1,006,446.77	15.4	493,022.67
West Virginia	1,342,270					103,573.64	103,573.64		25.1	1.9	407,243.17	7.8	831,453.19
Wisconsin	2,431,220	131,355.96	131,355.96		4.4	646,155.41	646,155.41		32.0	16.6	260,858.86	6.1	1,392,849.77
Wyoming	1,125,332	53,504.69	53,004.69		1.1	123,405.50	123,405.50			3.3	277,444.08	7.1	671,477.73
District of Columbia	959,235	508,017.51	508,017.51		3.2	278,337.00	278,337.00			.2	172,880.49	1.3	
Hawaii													
TOTALS	112,579,821	2,714,093.97	2,661,507.60	31,760.49	89.1	23,493,757.08	22,686,975.00	331,045.41	20.3	402.0	26,258,772.42	472.4	60,972,565.98

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS III—PROJECTS ON SECONDARY OR FEEDER ROADS

AS OF DECEMBER 31, 1933

STATE	PUBLIC WORKS FUNDS ASSIGNED FOR CLASS III PROJECTS ON SECONDARY HIGHWAYS	COMPLETED			UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR NEW CLASS III PROJECTS	STATE
		Total cost	Public works funds	Mileage	Estimated total cost	Public works funds allotted	Percentage completed	Mileage	Public works funds allotted	Mileage		
Alabama	$ 2,092,533								$ 139,527.30	12.7	$ 1,953,005.70	Alabama
Arizona	625,435				$ 40,700.00	$ 40,700.00	63.9	6.0	329,288.60	26.1	255,446.40	Arizona
Arkansas	1,687,084				9,586.90	9,586.90		.2	292,675.46	14.8	1,384,821.64	Arkansas
California	3,901,838				812,952.00	662,730.28	11.5	75.5	1,252,573.43	48.4	1,986,534.29	California
Colorado	1,718,632				689,262.42	689,262.42	41.9	139.9	234,602.89	42.5	794,766.69	Colorado
Connecticut	659,120				160,281.50	160,281.50	3.7	3.1	496,532.44	11.4	2,306.06	Connecticut
Delaware	454,772								138,752.00	1.8	316,020.00	Delaware
Florida	1,307,958				887,239.72	887,239.72	8.0	61.9	305,473.46	6.6	115,244.82	Florida
Georgia	2,320,973								319,912.59	26.2	2,001,060.41	Georgia
Idaho	1,121,562	$ 11,625.34	$ 7,523.08	2.8	813,681.26	783,778.86	33.8	121.7			330,260.06	Idaho
Illinois	6,262,223				464,038.89	464,038.89	11.4	69.9	2,120,622.90	141.4	3,677,561.21	Illinois
Indiana	501,892				165,000.00	165,000.00	2.4	48.1	151,464.91	11.6	185,427.09	Indiana
Iowa	2,212,245	46,834.43	44,100.00	7.6	450,456.45	429,700.00	42.6	36.9	322,100.00	42.8	1,416,345.00	Iowa
Kansas	2,522,401				395,339.02	395,339.02	2.8	23.8	2,127,061.98	148.7		Kansas
Kentucky	1,879,340				626,409.65	626,409.65	5.3	72.4	1,197,426.54	147.8	55,503.81	Kentucky
Louisiana	1,457,148				61,627.65	61,627.65	86.0	2.3	488,281.84	27.2	907,238.51	Louisiana
Maine	842,479	575,255.93	571,104.50	69.4	304,268.27	259,703.26	73.9	28.4	10,634.85	.2	1,036.39	Maine
Maryland	891,132				18,674.35	18,674.35		1.5	286,111.27	21.0	586,346.38	Maryland
Massachusetts	527,768				367,127.97	367,127.97	44.1	11.5	104,669.95	3.7	55,970.08	Massachusetts
Michigan	3,184,057				1,116,550.00	1,116,550.00	2.5	88.3	1,577,800.00	136.0	489,707.00	Michigan
Minnesota	2,131,314	28,119.39	28,119.39	21.2	1,195,395.21	1,107,217.08	21.0	168.6	399,097.39	76.1	596,880.14	Minnesota
Mississippi	1,744,669				435,000.00	435,000.00	22.1	54.6			1,309,669.00	Mississippi
Missouri	3,045,076				1,258,322.78	1,258,322.78	13.5	245.6	1,323,794.68	184.1	462,958.54	Missouri
Montana	1,859,937				231,205.86	231,205.86	7.4	23.9	691,116.50	83.2	937,614.64	Montana
Nebraska	1,957,240				1,605,898.84	1,604,036.52	17.2	249.7	36,078.47	4.5	317,125.01	Nebraska
Nevada	1,136,479				348,863.14	348,863.14	37.8	52.4	493,777.31	44.6	293,838.55	Nevada
New Hampshire	477,460				450,313.37	449,645.19	25.6	21.0			27,814.81	New Hampshire
New Jersey	63,460				56,550.52	56,550.52		.5			6,909.48	New Jersey
New Mexico	1,448,234				1,239,750.00	1,239,750.00	8.5	747.1	195,750.00	184.5	12,734.00	New Mexico
New York	3,662,137	88,300.00	88,300.00	1.9	3,383,863.19	2,994,388.19	14.9	80.2	534,575.00	17.5	44,873.81	New York
North Carolina	2,380,573	61,925.32	61,925.32	7.1	879,402.13	879,402.13	10.9	80.1	217,406.02	16.0	1,221,839.53	North Carolina
North Dakota	1,451,112								46,035.01	10.4	1,405,076.99	North Dakota
Ohio	3,871,148	40,500.00	40,500.00	16.2	2,118,510.00	2,013,074.90	15.3	223.4	1,283,690.00	57.9	533,883.10	Ohio
Oklahoma	2,304,199								98,609.03	.9	2,205,589.97	Oklahoma
Oregon	1,526,724				219,638.32	218,638.32	4.6	9.9	986,385.11	73.9	321,700.57	Oregon
Pennsylvania	7,716,975	6,570.00	6,570.00	.7	5,126,653.92	5,126,653.92	20.4	520.9	1,891,484.43	158.3	692,266.65	Pennsylvania
Rhode Island	499,677				7,631.36	7,631.36		1.1	172,014.08	15.2	320,031.56	Rhode Island
South Carolina	1,364,791				1,004,981.24	1,004,981.24	7.9	121.6	278,547.60	23.4	81,262.16	South Carolina
South Dakota	1,502,870				82,493.81	82,493.81	77.6	30.9	91,472.22	34.9	1,328,903.97	South Dakota
Tennessee	2,123,155				293,139.51	293,139.51	6.8	32.6	601,278.55	54.3	1,228,736.94	Tennessee
Texas	6,061,006				2,826,638.25	1,995,337.93	23.0	331.4	1,874,059.44	326.8	2,191,608.63	Texas
Utah	1,048,677	61,543.36	61,543.36	8.0	731,982.71	731,982.71	23.9	151.4	18,942.55	.2	236,208.38	Utah
Vermont	465,026				248,334.26	233,799.39	19.2	22.6	109,057.75	9.5	122,168.86	Vermont
Virginia	1,854,189	5,000.00	5,000.00	3.7	1,058,285.93	1,058,285.93	25.3	158.7	295,423.21	31.0	495,479.86	Virginia
Washington	1,180,362	36,856.60	36,856.00	11.6	622,532.39	612,532.39	12.9	33.6	351,075.32	16.5	179,898.29	Washington
West Virginia	1,118,559				142,992.63	142,992.63	13.3	5.4	308,837.33	20.3	666,729.04	West Virginia
Wisconsin	2,431,220	26,056.01	26,056.01	.6	1,192,443.23	1,153,427.95	22.1	88.6	604,150.17	38.8	647,585.87	Wisconsin
Wyoming	1,125,332				555,157.53	548,801.49	22.8	89.2	106,499.61	7.7	470,030.90	Wyoming
District of Columbia	187,106	62,897.00	62,897.00	1.5	268,004.12	268,004.12	52.6	1.9	575,491.35	4.3	52,841.53	District of Columbia
Hawaii	959,234								180,042.06	4.9	7,063.94	Hawaii
TOTALS	94,868,533	1,051,483.38	1,040,494.66	152.3	34,967,180.30	33,233,909.48	18.7	4,338.3	25,660,202.60	2,370.6	34,933,926.26	TOTALS