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BUREAU OF PUBLIC ROADS

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ROUNDED CUT SLOPES ON A CONNECTICUT HIGHWAY

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D. M. BEACH, *Editor*

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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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A STUDY OF SAND-CLAY-GRAVEL MATERIALS FOR BASE-COURSE CONSTRUCTION

BY THE DIVISION OF TESTS, BUREAU OF PUBLIC ROADS

Reported by C. A. CARPENTER, Associate Civil Engineer, and E. A. WILLIS, Associate Highway Engineer

THE RESULTS of an investigation of sand-clay materials for base-course construction were reported in the November 1938 issue of PUBLIC ROADS. A similar investigation of sand-clay-gravel materials for base courses has recently been concluded and the results of these tests are presented in this report.

Insofar as possible, the same general procedure was followed in making this study as was used in investigating the sand-clay materials. Two series, or a total of 11 mixtures, were prepared using water-worn Potomac River gravel, Potomac River sand, pulverized silica, and a red-clay soil from the same local source as that previously used.

The purpose of the study was to determine the effect of variations in plasticity index and aggregate grading on the stability and general serviceability of sand-clay-gravel materials when used as base courses for bituminous wearing surfaces. Such characteristics of the base-course mixtures as were known to have a direct bearing on their stability were investigated in conjunction with traffic tests in the circular track. These factors included compactibility, resistance to infiltration of water, and resistance to softening and loss of stability when exposed to the action of capillary water in conjunction with traffic.

To enable determination of the effect of variations in plasticity index, the five mixtures of series 1 were so designed that the fractions passing the No. 10 sieve were essentially the same as the five sand-clay materials used in series 1 of the previous tests. The plasticity indexes of the fractions passing the No. 40 sieve ranged from 0 to 16. The material retained on the No. 10 sieve was intended to have the same grading for all five mixtures, but mechanical analyses of samples from the track sections showed that there were minor variations in grading from section to section.

In order to determine the effect of variations in grading, the six mixtures of series 2 were designed to have a wide range of gradings and, with the exception of section 1, plasticity indexes of approximately 8. Section 1 was designed to have a plasticity index of 0.

The gradings and soil constants of the 11 materials used in the sand-clay-gravel studies are shown in table 1.

As in the studies of sand-clay mixtures, the indoor circular track was used to evaluate the serviceability of the various mixtures when used as base courses for a bituminous surface treatment and subjected to traffic under severe moisture conditions.

MIXTURES TESTED IN CIRCULAR TRACK

For the traffic tests on the materials of series 1, the track was divided into five, 7.5-foot sections, one for each of the five test mixtures, so that the traffic test could be made simultaneously on all five. The materials of series 2 were also tested as a group comprising six, 6.3-foot test sections. All the test sections were approximately 6 inches in depth when compacted and were laid over a porous, crushed-stone sub-base through which water introduced from below could pass. They were covered, after compaction, with a thin bituminous surface treatment, the purpose of which was to afford protection from the abrasive action of the test traffic and thus confine the test to a determination of the single factor of stability, or resistance to internal movement under traffic with the water table at various elevations in the base course.

The materials for each section of series 1 were prepared for laying by first thoroughly mixing the constituent aggregate fractions together dry and then adding sufficient water to bring the moisture content of the mortar portion, or material passing the No. 10 sieve, to its optimum moisture content as previously determined by Proctor tests on the sand-clay fractions. Because of this use of the fine fractions only as a basis for determining the moisture contents for consolidation, the mixtures proved to be somewhat deficient in moisture for maximum compaction in the track.

In order that there should be no such deficiency of moisture in series 2, it was necessary to devise a method

TABLE 1.—*Gradings and soil constants of sand-clay-gravel base-course materials*

	Series 1					Series 2					
	Section 1	Section 2	Section 3	Section 4	Section 5	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
Grading:	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Passing 1-inch sieve	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Passing ¾-inch sieve	93.4	93.2	89.2	93.8	90.3	88.1	98.5	79.4	97.5	87.1	93.9
Passing ⅜-inch sieve	81.3	76.7	74.0	82.8	73.2	70.6	89.3	58.9	90.2	70.7	83.3
Passing No. 4 sieve	68.2	62.5	61.6	67.3	60.8	58.5	83.9	41.9	82.5	56.4	75.6
Passing No. 10 sieve	53.5	47.9	47.9	50.3	47.0	42.3	65.0	31.9	41.9	38.0	66.1
Passing No. 20 sieve	44.2	40.3	40.3	43.9	39.5	35.5	58.4	27.3	37.0	32.5	57.2
Passing No. 40 sieve	34.0	31.0	31.0	34.4	30.3	25.9	48.5	19.8	30.5	24.0	41.4
Passing No. 100 sieve	20.7	18.1	18.9	19.9	18.8	2.6	26.0	14.4	22.9	16.9	28.5
Passing No. 200 sieve	16.9	15.1	16.0	16.7	16.1	1.2	24.6	12.4	22.1	16.2	27.5
Passing 0.005 mm	5.1	6.4	7.8	8.7	10.5	0	10.9	4.9	8.0	5.8	10.3
Passing 0.001 mm	2.0	3.0	5.0	6.0	9.0	0	7.0	3.0	5.0	4.0	7.0
Dust ratio [1]	50	49	52	49	53	5	51	63	72	67	66
Tests on material passing No. 40 sieve:											
Liquid limit	15	20	24	26	31	15	24	24	23	22	23
Plasticity index	0	5	9	11	16	0	9	8	7	6	7

[1] $\text{Dust ratio} = 100\left[\frac{\text{percentage passing No. 200 sieve}}{\text{percentage passing No. 40 sieve}}\right]$.

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that would take into account the coarse aggregate fraction. Since it was considered impractical to make the Proctor tests on materials containing 1-inch maximum-size stone, the moisture contents for the sections in series 2 were determined by vibratory compaction tests made on the dry aggregates, the volume of water used being that computed to be just sufficient to fill the voids in the vibrator-compacted aggregate. These moisture contents proved to be essentially correct for constructing the test sections since they did not render the material too wet to handle and yet were high enough to allow some drying during compaction operations without lowering the moisture content below the optimum.

The designed moisture contents and the actual moisture contents of samples from the uncompacted test sections immediately after laying are given in table 2. The required amounts of water were added to the aggregate mixtures on the basis of their air-dried weight whereas the actual moisture contents after laying were determined by oven drying. This accounts for the apparent increases shown for the more plastic sections of series 1 and the sections of series 2 having the higher soil-mortar contents.

TABLE 2.—*Designed moisture contents and actual moisture contents of track sections*

	Moisture content [1]		
	Designed by Proctor test	Designed to fill voids	Immediately after laying
SERIES 1	*Percent*	*Percent*	*Percent*
Section 1	4.5		4.3
2	4.8		4.4
3	4.9		4.9
4	5.4		5.5
5	5.5		6.5
SERIES 2			
Section 1		10.0	10.7
2		7.4	8.4
3		5.9	6.2
4		6.6	6.9
5		6.0	5.6
6		6.5	7.2

[1] Based on dry weight.

The procedure for preparing the materials and constructing the test sections was as follows:

1. The moistened sand-clay-gravel materials were thoroughly mixed to distribute the water uniformly and were then placed in the track in two approximately equal layers, each layer being compacted with pneumatic-tired traffic uniformly distributed over the surface.

2. Compaction was continued on the top layer until no perceptible subsidence could be produced in any section by additional wheel-trips. This required 30,000 wheel-trips for series 1[1] and 42,000 wheel-trips for series 2.

3. The sections were sprinkled with water to soften the surface slightly and were trimmed smooth with a blade.

4. After drying for a few days the surface was primed with light tar.

5. As soon as the prime had been absorbed and had cured sufficiently to be fairly dry, a ¾-inch surface treatment consisting of 0.4 gallon per square yard of hot-application bituminous material and 50 pounds of cover stone was constructed.

6. The surface treatment was consolidated by applying distributed traffic.

ADDITIONAL COMPACTION NECESSARY FOR THREE SECTIONS OF SERIES 1

Consolidation of the base and surface treatment appeared to be completed in series 1 after a total of 50,000 wheel-trips, and water was then admitted to the subbase and maintained at a height of ½ inch above the bottom of the base course being tested. After only 300 wheel-trips of distributed test traffic, sections 2, 3, and 4 began to move and displace so badly that traffic had to be discontinued. The loss of stability resulting from the introduction of water was accompanied by marked subsidence over the entire area of these three sections. Section 1, which was nonplastic, also showed marked subsidence although it remained highly stable. Tests showed that with the exception of section 5, which had a decrease in moisture of 1.5 percent, the materials had absorbed from 3 to 3.6 percent of moisture in addition to that contained at the time they were laid (see table 3). This absorption of moisture, together with the subsidence of the surface, definitely indicated that the moisture contents used for construction in sections 1, 2, 3, and 4 were too low to permit maximum compaction.

In an attempt to complete the compaction without reworking the materials in the weak sections, 7,700 wheel-trips of additional distributed traffic were applied. This additional traffic resulted in the complete failure of the surface treatments on sections 2, 3, and 4.

TABLE 3.—*Moisture contents of the track sections at various stages of the investigation*

Series 1	Moisture content expressed as a percentage of the dry weight of the aggregate				
	When laid	At 50,300 wheel-trips	At 58,000 wheel-trips	At 75,000 wheel-trips	At 425,000 wheel-trips
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Section 1	4.3	7.3			5.3
2	4.4	8.0	6.7	4.6	4.9
3	4.9	8.4	6.8	4.7	5.1
4	5.5	8.9	7.0	5.9	5.5
5	6.5	5.0			6.6

Series 2	Moisture content expressed as a percentage of the dry weight of the aggregate		
	When laid	At 145,000 wheel-trips	At 330,000 wheel-trips
	Percent	*Percent*	*Percent*
Section 1	10.7	6.2	6.9
2	8.4	6.6	6.9
3	6.2	3.9	4.0
4	6.9	4.9	4.9
5	5.6	4.5	4.1
6	7.2	5.8	6.2

Samples for moisture content and density determinations were taken and the surface treatment was removed to facilitate drying in conjunction with subsequent compacting operations. At this time the moisture contents of these three sections were approximately 2 percent higher than when they were originally constructed (see table 3). The condition of sections 2, 3, and 4 just prior to removal of the surface treatment is well illustrated by the photograph of section 3 shown in figure 1.

After removal of the surface treatment from sections 2, 3, and 4, 17,000 wheel-trips of additional compacting

[1] Introduction of water and application of a small amount of test traffic on the sections of series 1 later proved that thorough compaction had not been obtained.

traffic were applied in small daily increments, bringing the total to 75,000 wheel-trips. During this time the moisture contents of the three sections decreased to approximately those at which the sections were originally laid. A new surface treatment was then constructed and compacted with 25,000 wheel-trips of distributed traffic, bringing the total to 100,000 wheel-trips.

The behavior of the five sections under the regular traffic test from 100,000 to 425,000 wheel-trips will be discussed fully later. At this point the behavior of sections 2, 3, and 4 after recompaction, will be discussed in comparison with their above described earlier behavior when not fully compacted.

During the traffic test the water level was gradually raised until a height of 4½ inches above the top of the sub-base was reached at 370,000 wheel-trips and this water elevation was maintained to a total of 425,000 wheel-trips. Under these extreme conditions sections 2 and 3, because of their increased density, absorbed only 0.3 and 0.4 percent more moisture than they had contained at 75,000 wheel-trips and section 4 actually showed a loss of 0.4 percent moisture. All three sections were quite stable throughout the test in marked contrast to their behavior from 50,000 to 50,300 wheel-trips when each absorbed approximately 3½ percent of water and became highly unstable because of insufficient compaction.

No difficulties such as those encountered in connection with series 1 were encountered during the compaction of the materials of series 2 because, as previously stated, the original moisture contents were high enough to allow for appreciable drying during compaction. Thus compaction was able to proceed to the maximum density obtainable under traffic before the moisture content passed below the optimum.

ONLY ONE SECTION OF SERIES 1 FAILED DURING TRAFFIC TEST

Table 4 shows the procedure followed in testing the track sections in series 1 with notations on the behavior of each section during the test. Table 5 gives similar information for series 2.

Series 1.—After all construction and compaction operations had been completed at 100,000 wheel-trips, water was introduced into the sub-base and set at an elevation of ½ inch above the bottom of the test base course. Distributed traffic was applied to a total of 183,000 wheel-trips and then, without changing the water elevation, concentrated traffic was applied to 256,000 wheel-trips, making a net total of 156,000 wheel-trips of test traffic. Section 5 became unstable and was rated as having failed at 150,000 wheel-trips (50,000 wheel-trips of test traffic). Figure 2, left, shows the appearance of section 5 at 150,000 wheel-trips when its failure was recorded. On the right is shown the same section at 233,000 wheel-trips when measurements of its surface displacement were discontinued. The other four sections in the series, although showing some movement under traffic and slight cracking in section 4, were in good condition at 256,000 wheel-trips which marked the conclusion of that phase of the test in which the water was held at the ½-inch level.

As shown in table 4, the test with concentrated traffic was then continued with the moisture conditions being made progressively less favorable until the water level had reached an elevation of 4½ inches and a total of 425,000 wheel-trips had been applied. Sections 1, 2, and 3 remained in good condition. Section 4, although exhibiting a high degree of resistance to softening, considering the severity of the test, developed sufficient rutting and cracking to require its classification as a doubtful or border-line material.

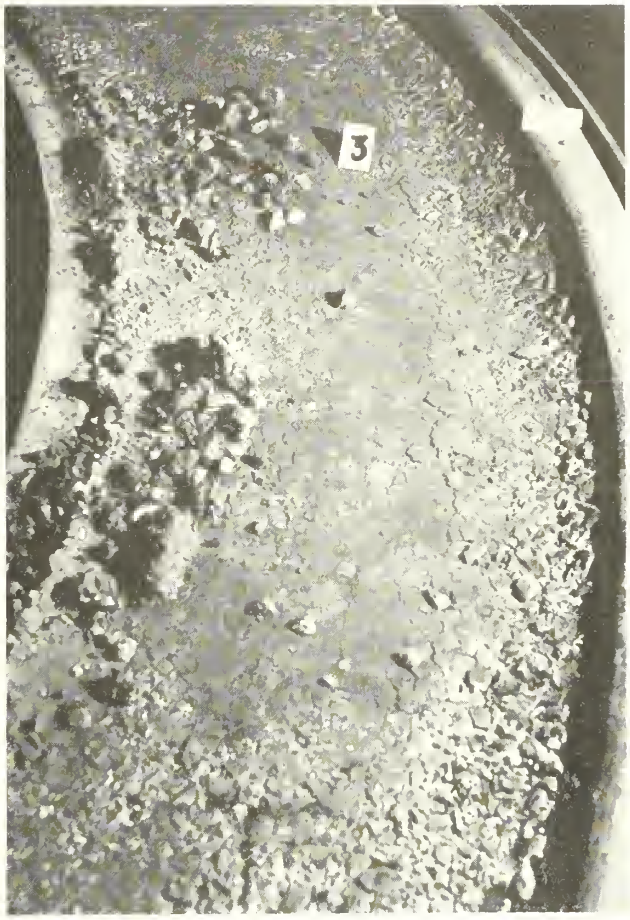

FIGURE 1.—APPEARANCE OF SECTION 3 OF SERIES 1 AFTER 58,000 WHEEL-TRIPS OF TRAFFIC.

Measurements of average vertical displacement made with the transverse profilometer at various stages of the test are shown graphically in figure 3. In the tests of sand-clay materials described in the previous report, it was found that unmistakable visual evidence of failure such as marked instability, breaking up of the surface treatment, and extrusion of mud through the surface was noted at about the time the average vertical displacement of the surface reached 0.25 inch. Section 5 of series 1 of the sand-clay-gravel materials showed an average vertical displacement of only 0.17 inch at the time failure became visually evident but the vertical displacement continued to increase rapidly, reaching 0.34 inch when measurements were discontinued on the section at 233,000 wheel-trips. The increase in average vertical displacement for the other four sections, none of which actually failed, was very gradual and the total displacement never reached more than 0.20 inch during the regular traffic test.

Section 2, judged by its rate and total amount of vertical displacement, was markedly superior to any of the other sections in series 1 and its general behavior in the track as judged by visual inspection confirmed the evidence of the displacement measurements. In this respect, it conformed to the behavior of the corresponding section of the sand-clay materials from which it differed physically only in having 46.5 percent of the sand-clay replaced with rounded gravel ranging in size from No. 10 to 1 inch.

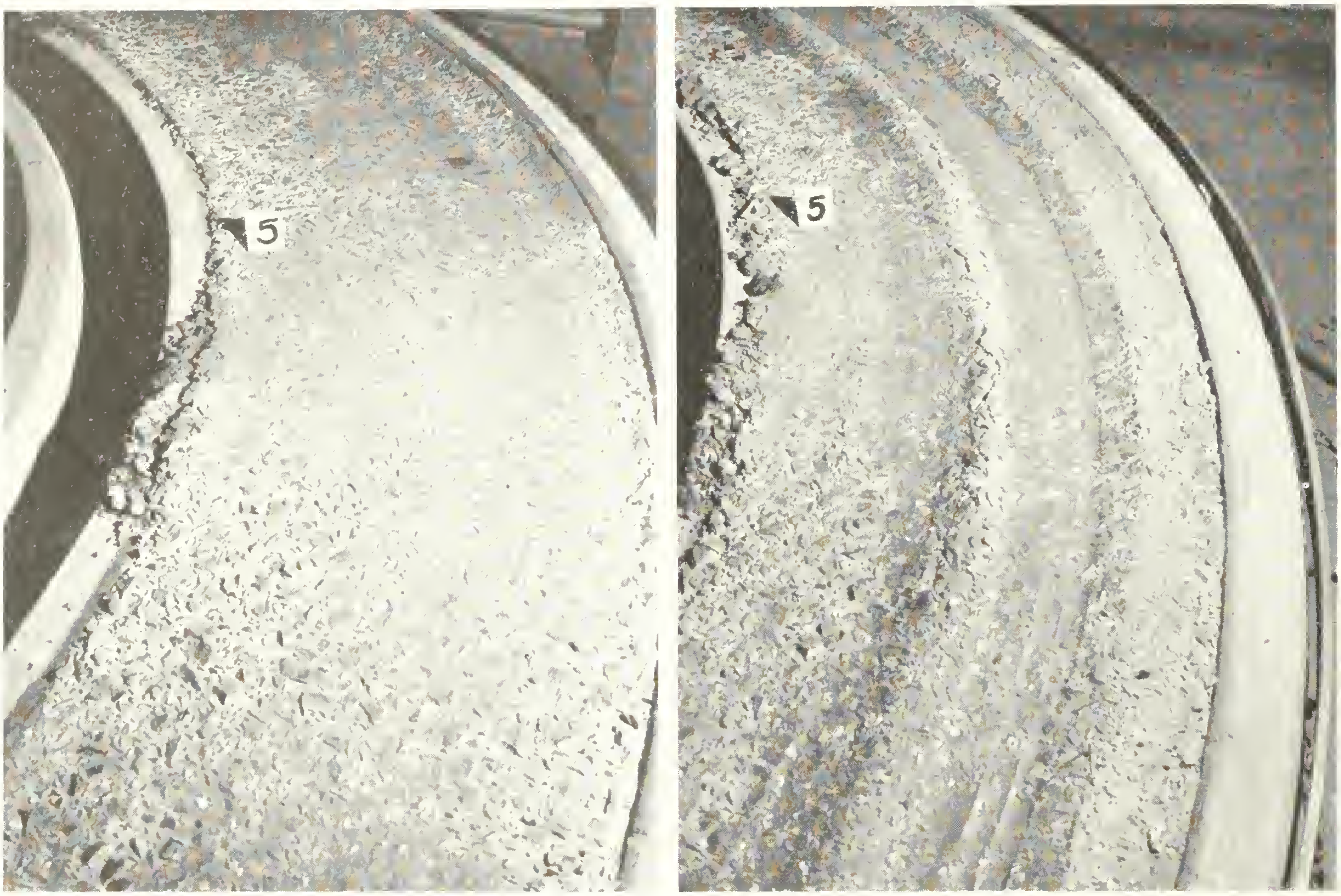

Figure 2.—Appearance of Section 5 of Series 1: Left, After 150,000 Wheel-Trips, When Its Failure Was Recorded. Right, After 233,000 Wheel-Trips, When Measurements of Its Surface Displacement Were Discontinued.

Table 4.—*Schedule of operations and behavior of test sections in circular track tests, series 1*

Operation	Traffic	Water level above top of sub-base	Behavior				
			Section 1 (plasticity index=0)	Section 2 (plasticity index=5)	Section 3 (plasticity index=9)	Section 4 (plasticity index=11)	Section 5 (plasticity index=16)
	Wheel-trips	*Inches*					
Compacting base course	0– 30,000	(1)	Stable but raveled [2]	Stable but raveled [2]	Stable but raveled [2]	Stable but raveled [2]	Unstable at first.[3]
Compacting base and surface treatment.	30,000– 50,000	(1)	Good	Good	Good	Good	Good.
Testing with distributed traffic.	50,000– 50,300	½	do.[4]	Unstable [5]	Unstable [5]	Unstable [5]	Do.
Compacting base course.	50,300– 58,000	(6)	do	Surface treatment destroyed.[7]	Surface treatment destroyed.[7]	Surface treatment badly damaged.[7]	Do.
Drying and recompacting base course.	58,000– 75,000	(1)	do	Unstable at first but improved rapidly.	Unstable at first but improved gradually.	Unstable at first but improved gradually.	Do.
Compacting base and new surface treatment.	75,000–100,000	(1)	do	Good	Good	Good	Do.
Testing with distributed traffic.	100,000–183,000	½	do	do	do	Good but moved slightly under traffic.	Quickly became unstable and failed at 150,000 wheel-trips.
Testing with concentrated traffic.	[8] 183,000–256,000	½	Good but developed slight rutting.	Good but developed slight movement.	Good but developed slight movement.	Good but developed some cracking.	
Do	256-000-320,000	2½	do	do	do	do	
Do	320,000–370,000	3½	Good but cracked somewhat along center line.	Good but cracked somewhat along center line.	Good but cracked somewhat along center line.	Movement increased appreciably.	
Do	370,000–425,000	4½	Good; some rutting and cracking.	Good	Good; some rutting and cracking.	Appreciable rutting and cracking.	
A 2-foot segment of each section was frozen with dry ice and tested after thawing.	425,000–445,000	4½	No change in behavior.	No change in behavior.	No change in behavior.	Frost heave, 0.03 inch; increased rutting and cracking.	Frost heave, 0.1 inch; extremely uustable.

[1] No water in sub-base.
[2] Raveling was caused by a deficiency of moisture.
[3] The early instability of sec. 5 indicated that its initial moisture content of 6.5 percent was sufficient to permit proper compaction.
[4] Sec. 1 was stable but its marked subsidence under traffic when water was admitted indicated a deficiency of moisture during compaction.
[5] This temporary loss of stability and the subsidence of the surface when water was admitted indicated a lack of compaction resulting from an initial deficiency of moisture.
[6] Water drained out of sub-base to allow unstable sections to dry and compact.
[7] Evaporation of the excess capillary moisture, admitted because of the incomplete early compaction, was so slow that the base course material had to be partially dried by remixing.
[8] Load on each wheel increased from 800 pounds to 1,000 pounds at 233,000 wheel-trips.

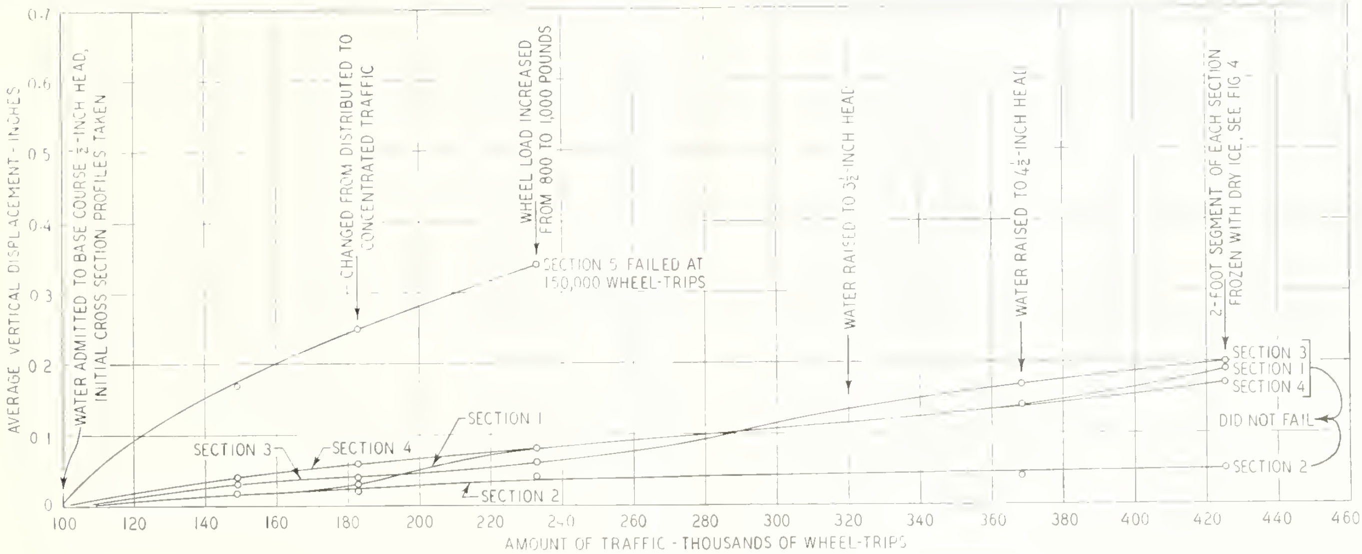

FIGURE 3.—RATE OF SURFACE DISPLACEMENT UNDER TRAFFIC, SERIES 1.

TABLE 5.—*Schedule of operations and behavior of test sections in circular track tests, series 2*

Operation	Traffic	Water level above top of sub-base	Behavior					
			Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
	Wheel-trips	*Inches*						
Compacting base course.	0- 40,000	(1)	Rutted badly at first but quickly became stable.[2]	Slightly unstable at first; stable later.	Very unstable at first; gradually became stable.	Stable at first; slightly unstable later.	Decidedly unstable at first; became stable later.	Stable at first; unstable and cracked later.
Compacting base and surface treatment.	40,000- 65,000	(1)	Good	Good	Good	Good	Good	Movement continued.
Testing with distributed traffic.	65,000-125,000	½	do	Good but developed slight movement under traffic.	do	do	do	Decidedly unstable.
Testing with concentrated traffic.	[3]125,000-205,000	½	Excellent	Developed more movement and failed.	do	do	Good but developed slight movement.	Rutted, corrugated and cracked.
Do	205,000-255,000	2½	do		do	do	Good	Failed.
Do	255,000-330,000	3½	do		do	Developed 2 chuck holes; section was near failure.	do	

[1] No water in sub-base.
[2] Because of its ability to drain readily, sec. 1 required frequent sprinkling during compaction.
[3] Load on each wheel increased from 800 pounds to 1,000 pounds at 175,000 wheel-trips.

After the conclusion of the regular traffic test on series 1 at 425,000 wheel-trips, the effect of freezing and thawing was investigated to a limited extent. A segment of each section about 2 feet long and 18 inches wide was frozen by placing a layer of crushed dry ice over it and covering the dry ice with blankets. Freezing to a minimum depth of 2½ inches was accomplished in about 5 hours. Measurements of surface elevations at this time revealed a heave of 0.1 inch on section 5 and 0.03 inch on section 4 with no change in surface elevation for the other three sections. After the frozen segments had thawed out, 20,000 additional wheel-trips of concentrated traffic were applied. Cross section profiles indicated additional average vertical displacements as shown in figure 4 from 425,000 wheel-trips to 445,000 wheel-trips.

WATER ELEVATION OF ½ INCH PROVED SEVERE TEST CONDITION

The average vertical displacements at 370,000 and 425,000 wheel-trips from figure 3 are repeated in figure 4 to show the effect of freezing and thawing on the rate of displacement. The nonplastic material of section 1 was apparently not affected, displacement caused by traffic continuing at the same rate after freezing and thawing as before. The plastic materials in sections 2, 3, and 4, were affected roughly in direct proportion to their plasticity indexes. Section 2, with a plasticity index of 5, showed only a slightly increased rate of displacement after freezing while section 4, with a plasticity index of 11, showed a very marked increase. Section 3 was intermediate between sections 2 and 4 in this respect.

Series 2.—As shown in table 5, preliminary compaction of the base and surface treatment was completed at 65,000 wheel-trips. Throughout the subsequent traffic test with water in the sub-base and at gradually increasing heights in the test base course, sections 1, 3, and 5 remained stable and showed no indications of failure. Section 2, as shown in figure 5, developed considerable movement and local depressions under concentrated traffic while the water level was still at ½ inch and was rated as having failed at 205,000 wheel-trips. Section 6 was decidedly unstable throughout the test period, indicating impending failure while the water level was at ½ inch, and was rated as having failed at 250,000 wheel-trips or shortly after the water level was raised from ½ inch to 2½ inches. Its appear-

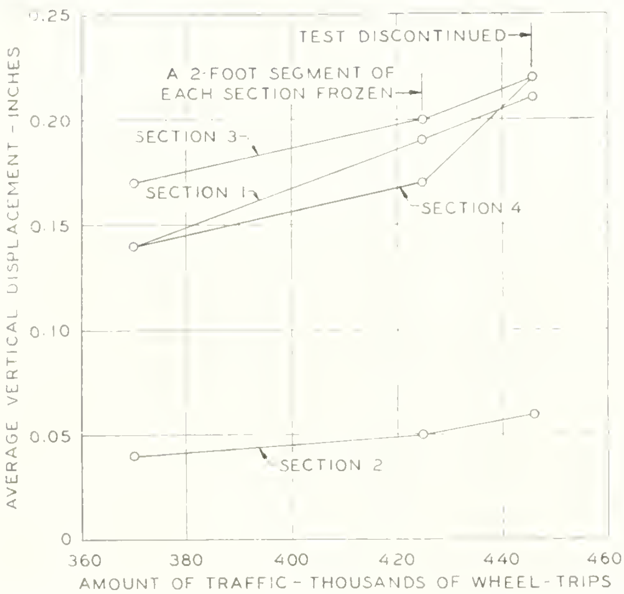

FIGURE 4.—EFFECT OF FREEZING AND THAWING CIRCULAR TRACK SECTIONS, SERIES 1. (THE DISPLACEMENTS AT 370,000 AND 425,000 WHEEL-TRIPS ARE REPLOTTED FROM FIGURE 3.)

ance shortly before complete failure is shown in figure 6.

Section 4 behaved well under the test traffic until after the water level had been held at 3½ inches for some time. It then developed two chuck holes and was definitely nearing failure when the test was discontinued at 330,000 wheel-trips. The latter circumstance necessitated its classification as a doubtful or borderline material.

The development of vertical displacement as measured with the transverse profilometer on the six sections of series 2 is shown in figure 7. For sections 2 and 6 the average vertical displacement at the time visual evidence of complete failure was noted was approximately 0.24 inch, which is in close agreement with the results of tests on sand-clay materials. Although section 4 showed only a slight increase in displacement up to 255,000 wheel-trips, the curve (fig. 7) broke abruptly upward after the water level was raised to 3½ inches and apparently would have passed 0.25 inch at about 350,000 wheel-trips had the test been continued.

In tests of both the sand-clay materials previously reported and the sand-clay-gravel materials here discussed, the definitely unsatisfactory materials were clearly distinguished from the rest by the fact that they either failed completely or showed unmistakable evidence of impending failure during the portion of the test when the water level was only ½ inch above the bottom of the test base course, and in no case were more than 140,000 wheel-trips of test traffic necessary to bring out this initial distinction. This initial classification was facilitated by the fact that the displacement curves of the unsatisfactory materials invariably

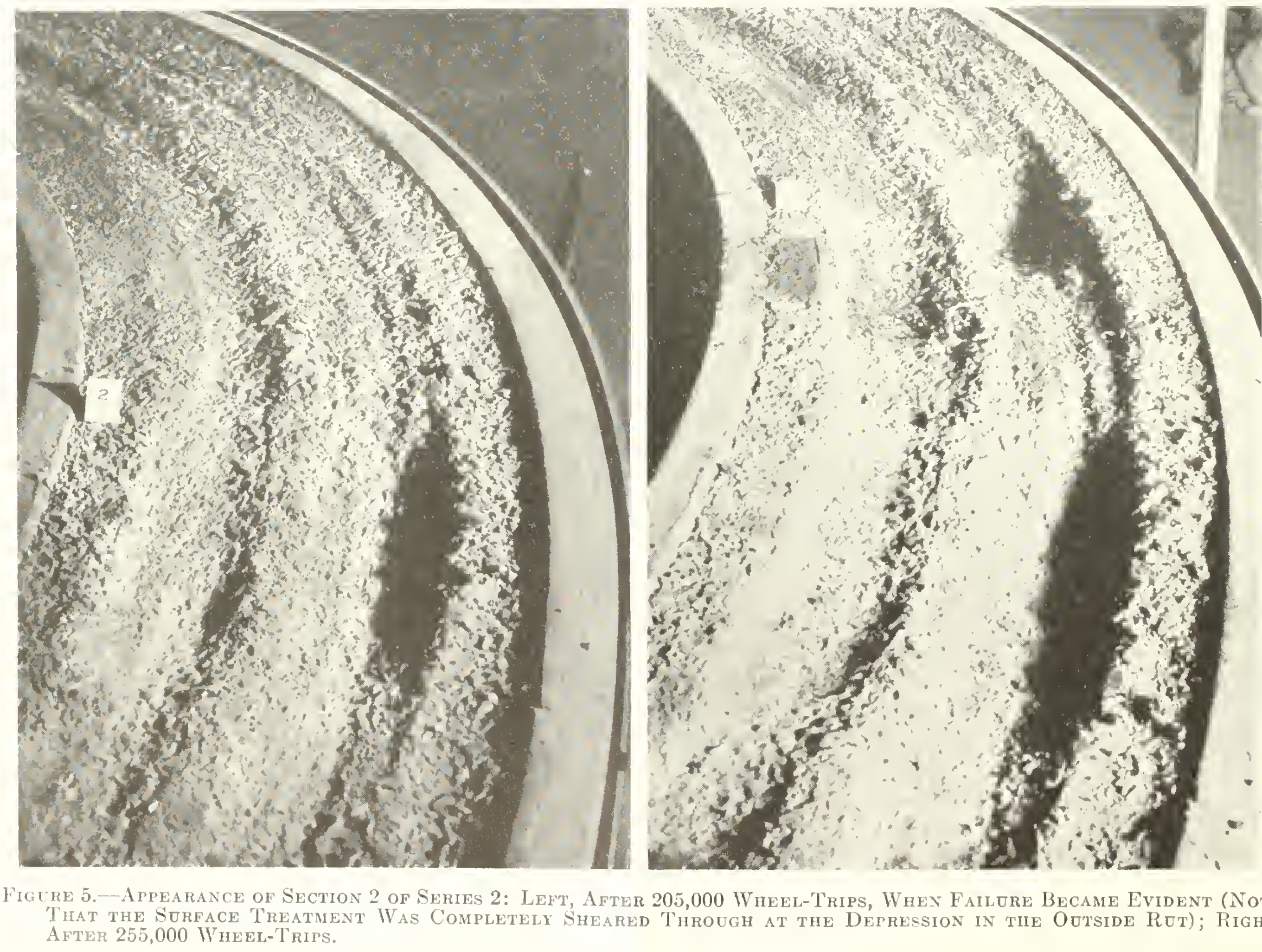

FIGURE 5.—APPEARANCE OF SECTION 2 OF SERIES 2: LEFT, AFTER 205,000 WHEEL-TRIPS, WHEN FAILURE BECAME EVIDENT (NOTE THAT THE SURFACE TREATMENT WAS COMPLETELY SHEARED THROUGH AT THE DEPRESSION IN THE OUTSIDE RUT); RIGHT, AFTER 255,000 WHEEL-TRIPS.

rose steeply or broke upward fairly early in the test whereas the displacement curves for the satisfactory and borderline materials tended to flatten out after the first few thousand wheel-trips of test traffic (see figs. 3 and 7).

Additional traffic and elevation of the water level were resorted to only after the definitely unsatisfactory materials had been identified, the purpose being to ascertain if any of the remaining sections were composed of borderline materials.

Figure 8, showing section 4 of series 1, well illustrates the appearance of one of the borderline materials at various stages of the test. The two upper views show the section in excellent condition after, respectively, 50,000 and 133,000 wheel-trips of test traffic. At these stages its condition was typical of that of any of the wholly satisfactory sections during the traffic test. The two lower views show the results of prolonged application of concentrated traffic under highly unfavorable conditions. Even at these stages the indications of failure, although sufficient to place the section in the border classification, were not extensive.

Figure 9 shows the condition of the other borderline material, section 4 of series 2, at 330,000 wheel-trips (the conclusion of the traffic test). Complete failure had not occurred but impending failure was clearly indicated by the deep depression in the inside wheel lane. The test conditions had been made so severe during the later stages of the tests of both series that not even the complete failure of a section could have been construed to indicate a seriously inferior material.

NEW INSTRUMENT USED TO TAKE LONGITUDINAL PROFILES

In addition to the transverse profiles which were taken at two stations on each section and from which the average vertical displacements of the surface were calculated (see figs. 3 and 7), longitudinal profiles were taken along the center lines of the wheel lanes with a new instrument designed especially for use on the circular track and used for the first time in these tests.

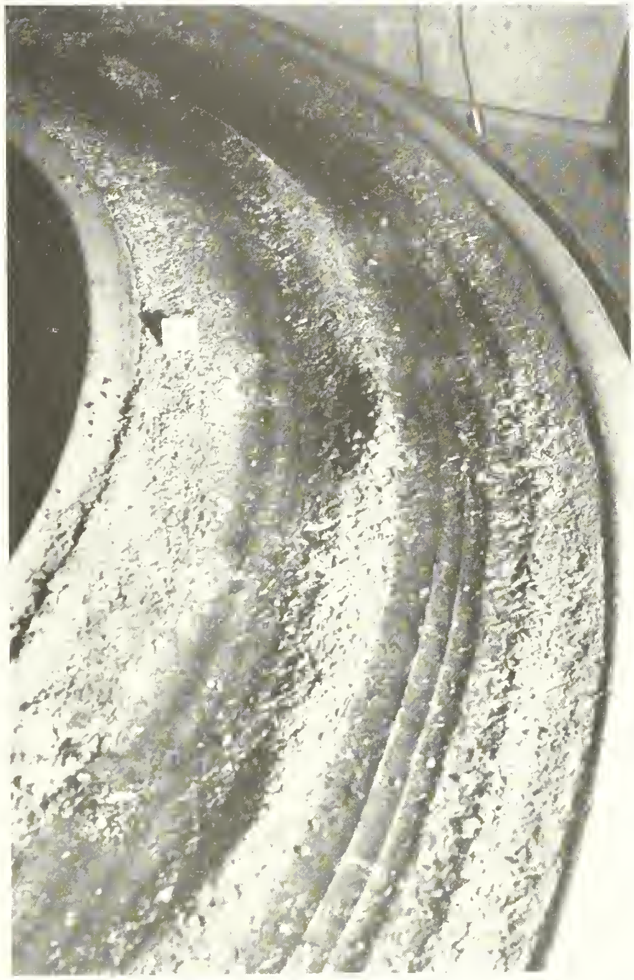

Figure 6.—Appearance of Section 6 of Series 2 After 255,000 Wheel-Trips of Traffic. Failure is Indicated by the General Roughness, Rutting, and Breaking of the Surface-Treatment.

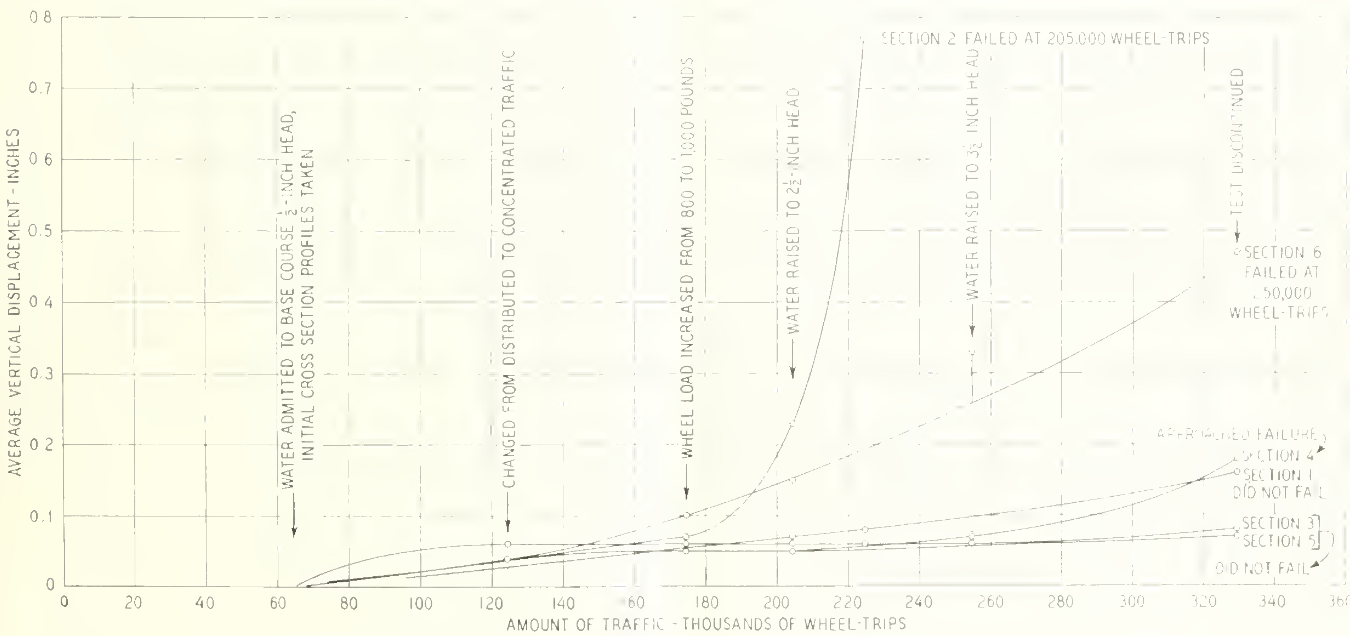

Figure 7.—Rate of Surface Displacement Under Traffic, Series 2.

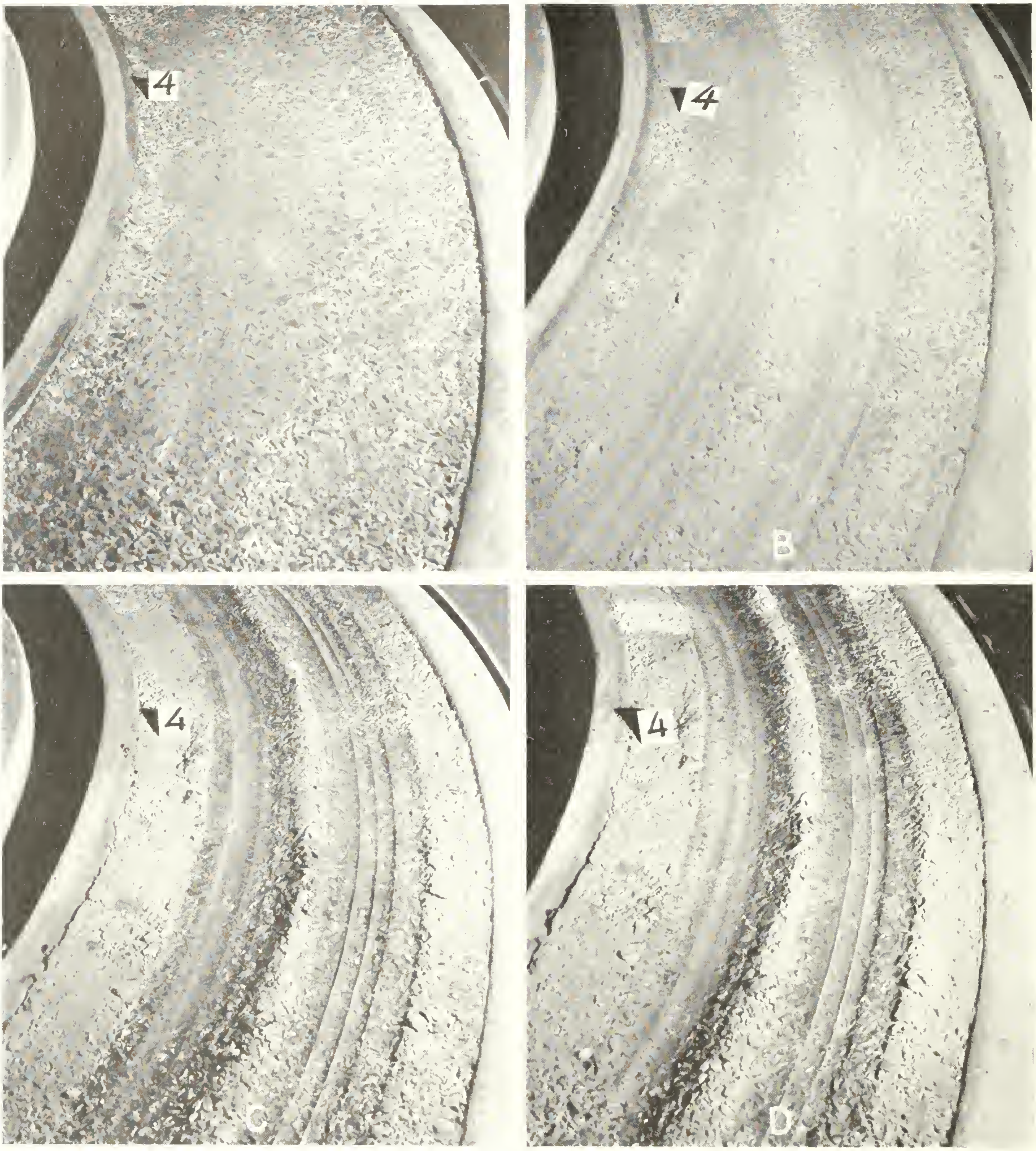

FIGURE 8.—APPEARANCE OF SECTION 4 OF SERIES 1 AFTER VARIOUS AMOUNTS OF TRAFFIC: A, AFTER 150,000 WHEEL-TRIPS; B, AFTER 233,000 WHEEL-TRIPS; C, AFTER 425,000 WHEEL-TRIPS; AND D, AFTER FREEZING, THAWING, AND THE APPLICATION OF 20,000 ADDITIONAL WHEEL-TRIPS, BRINGING THE TOTAL TRAFFIC TO 445,000 WHEEL-TRIPS.

Figure 10 is a photograph of the new longitudinal profilometer in position for making a recording of the profile of the track surface. It consists of a radial frame pivoted at the central pedestal of the track structure and supported at its outer end by two flanged wheels arranged in tandem and running on a peripheral steel track attached to the outer curb of the track. One of these wheels drives, through an appropriate transmission system, the vertical drum that carries the record sheet as shown in figure 10. This drum is mounted on a radially sliding cage that can be clamped at any desired radius within the width of the track. The cage also carries a vertical sliding measuring rod on the lower end of which is a small caster that rests on the track surface and moves up and down in conformity with the contour of the surface. At the rod's upper end is a stylus which draws the surface profile on the drum as it revolves when the instrument is moved around the track. The drum makes one revolution while the profilometer is making one trip around the track, so that a continuous profile of all the test sections in a track is made on one sheet 21¼ inches long.

Longitudinal profiles of the test sections in both series 1 and 2, taken on the wheel courses where concentrated traffic was applied, are shown in figure 11. The upper one of each pair of profiles shown was taken at the conclusion of the compaction period before any test traffic had been applied. The corresponding lower ones were taken at the conclusion of the traffic test and show the depth of the ruts that were formed.

These longitudinal profiles were found to be fully as satisfactory as the cross-section profiles as a means of evaluating the comparative quality of the materials. Tests with both instruments on this series of materials indicated that the average depth of rut was about 1.8 times the average vertical displacement as calculated from the cross-section profiles. While this factor might vary somewhat for different types of materials, the comparative results in a series of tests on similar materials are consistent and if desired the value of the factor is easily obtained for other types.

Compaction tests similar to those used in determining the moisture contents for constructing the sections of series 2 were made on the aggregates of both series. The vibratory compaction test was modified to the extent that about 5 percent by weight of kerosene was mixed with the aggregates before vibrating them, to prevent segregation of the coarse stone. It was found that this produced somewhat higher densities than were obtained by vibrating the dry aggregates as was done in setting the moisture contents.

A comparison of the densities obtained by the modified vibration method with those of the track sections at the conclusion of the traffic test is shown in table 6.

TABLE 6.—*Comparison of densities obtained by vibration and by testing in the circular track*

	Density (aggregate volume per unit of total compacted volume)		Behavior of section under traffic
	Compacted by vibration	Track section at end of test	
SERIES 1	*Percent*	*Percent*	
Section 1	89.7	82.8	Satisfactory.
2	88.0	87.0	Do.
3	87.5	86.4	Do.
4	87.1	86.2	Essentially satisfactory.
5	87.2	84.0	Failed.
SERIES 2			
Section 1	86.9	77.8	Satisfactory.
2	86.7	83.9	Failed.
3	89.9	89.1	Satisfactory.
4	87.5	87.3	Approached failure.
5	89.9	89.3	Satisfactory.
6	87.7	85.1	Failed.

SATISFACTORY PLASTIC MATERIALS HAD GREATEST COMPACTION IN TEST TRACK

The relations between service behavior and relative density, as shown in table 6, were consistent with those noted for the sand-clay materials of the previous investigation.

The satisfactory and borderline plastic sand-clay-gravels (sections 2, 3, and 4 of series 1 and sections 3, 4, and 5 of series 2) attained densities in the track within from 0.2 to 1.1 percent of the densities of the vibrated samples. The unsatisfactory materials (section 5 of series 1 and sections 2 and 6 of series 2) all of which were plastic, had densities that were 3.2, 2.8, and 2.6 percent, respectively, less in the track than in the vibrated samples. The two nonplastic materials, section 1 of series 1, and section 1 of series 2, because of their harshness, were the least compactible under

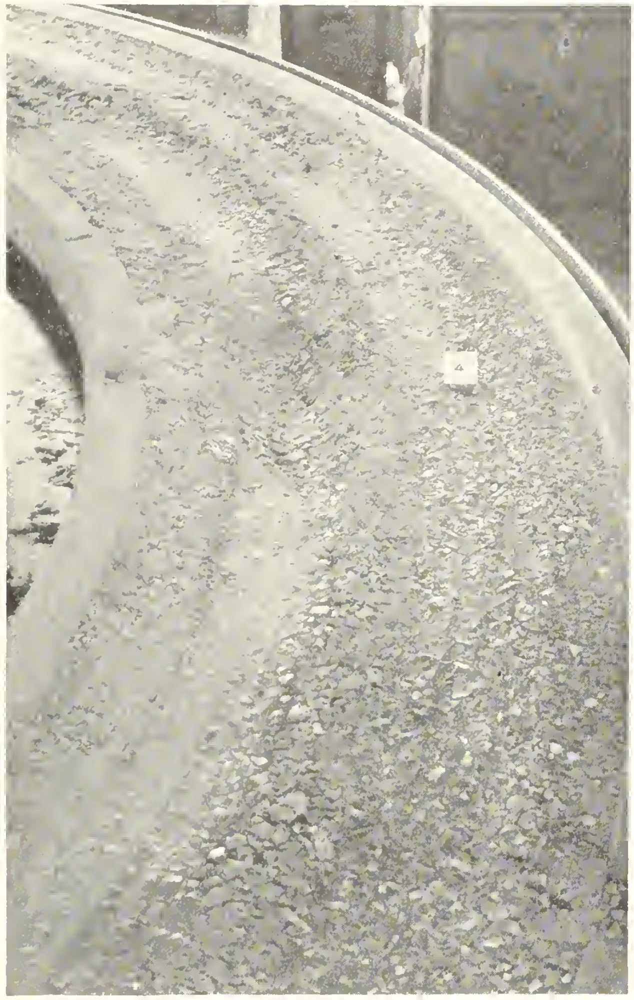

FIGURE 9.—APPEARANCE OF SECTION 4 OF SERIES 2 AT THE END OF THE TRAFFIC TEST (330,000 WHEEL-TRIPS). THE DEEP RUT IN THE INSIDE LANE INDICATED AN IMPENDING FAILURE.

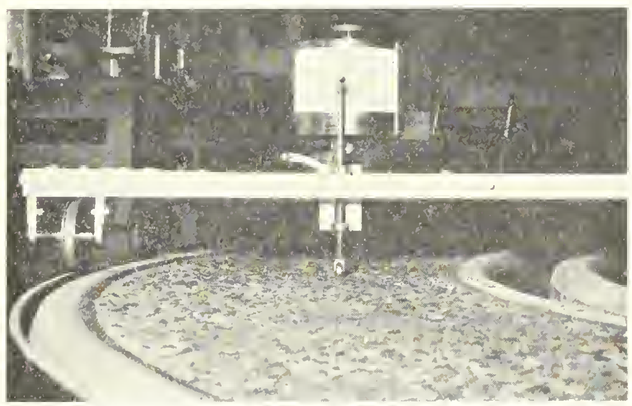

FIGURE 10.—LONGITUDINAL PROFILOMETER USED FOR RECORDING LONGITUDINAL PROFILES OF CIRCULAR TRACK SECTIONS.

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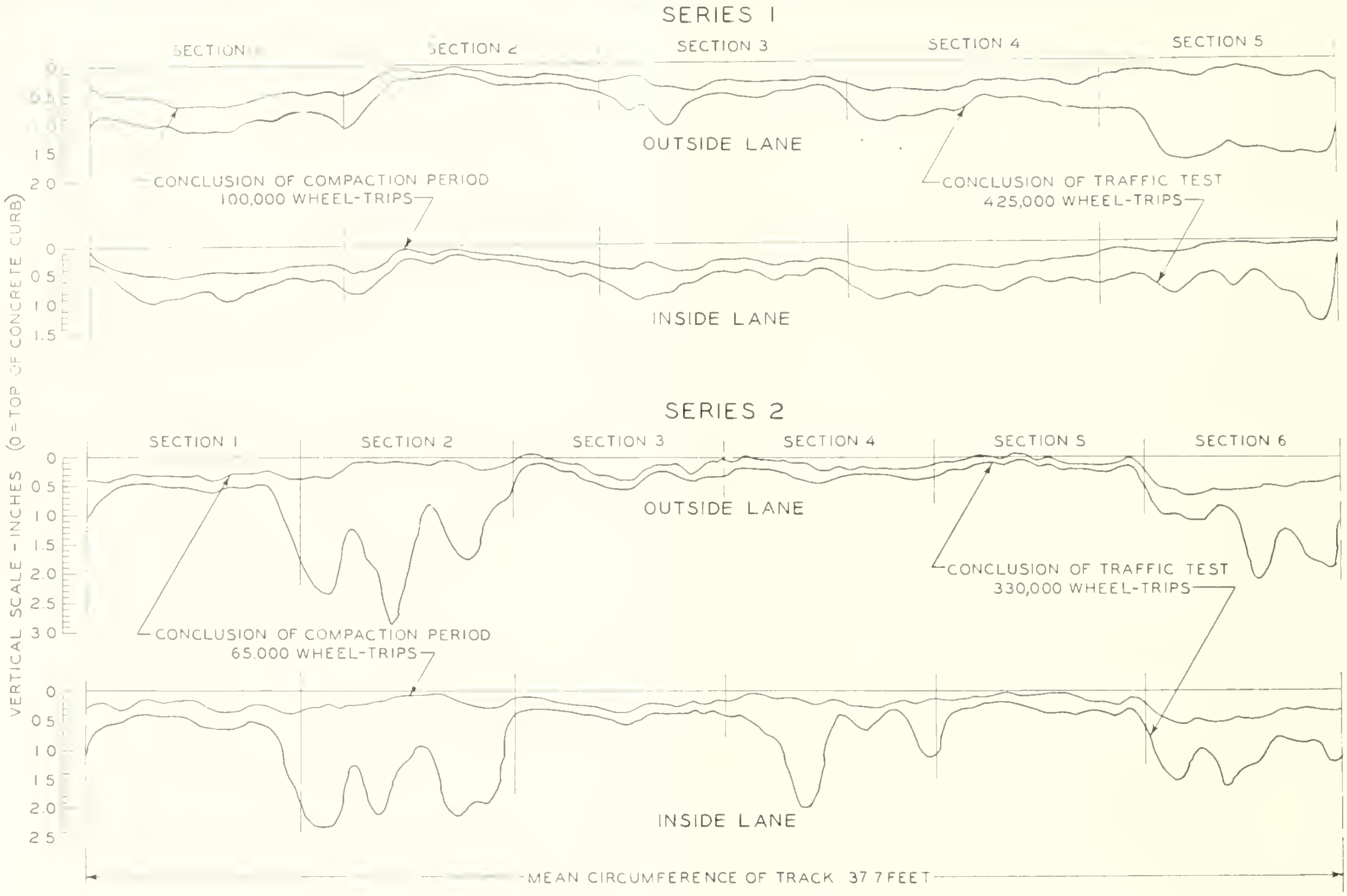

Figure 11.—Longitudinal Profiles of Circular Track Sections Showing Maximum Displacement or Rutting.

traffic, their densities in the track being 6.9 and 9.1 percent less than those of the vibrated samples. However, both gave satisfactory service because of the same inherent characteristic that caused their noncompactibility under traffic, namely their harshness.

The numerical differences in density of the plastic materials appear small and for that reason, their importance might easily be overlooked. To realize their importance where plastic materials are concerned, it is only necessary to analyze the data showing the densities of sections 2, 3, and 4 of series 1 at 58,000 wheel-trips when absorbed water had rendered them extremely unstable, the densities of the same sections at 425,000 wheel-trips after further compaction had made them highly resistant to the action of water, and their maximum obtainable densities as determined by vibration (see table 7).

Table 7.—*Comparison of densities obtained by vibration with densities of track sections after various amounts of traffic*

Series 1	Densities		
	In track at 58,000 wheel-trips (unstable)	In track at 425,000 wheel-trips (stable)	Samples compacted by vibration
	Percent	*Percent*	*Percent*
Section 2	83.2	87.0	88.0
3	84.5	86.4	87.5
4	84.3	86.2	87.1

At 58,000 wheel-trips, when the track sections were highly unstable, the densities of the three sections were respectively only 4.8, 3, and 2.8 percent less than the maximum densities obtained by vibration. The additional compaction obtained by the application of additional traffic in conjunction with the drying out of from 1.1 to 2.1 percent of moisture (see table 3) increased their densities by, respectively, 3.8, 1.9, and 1.9 percent or to within 1, 1.1, and 0.9 percent of their maximum densities obtained by vibration. This small increase in density accounted for their alteration from a condition in which they were highly susceptible to softening in the presence of capillary water to one in which they had a high resistance to the action of water.

A volumetric analysis of the composition of all of the test sections at the conclusion of the traffic tests is shown in table 8. The highly capillary nature of the plastic materials, comprising all of the test sections except section 1 of each series, is strikingly shown by the very low percentage of residual or air-filled voids. These residual or air-filled voids represent, in each section, less than 2 percent of the total volume of the traffic-compacted plastic materials. The water contents show considerable variation, being relatively low for the more compactible materials and high for the noncompactible ones. In other words, the capacity of these materials to absorb water seems to be limited only by the volume of pore space available with a small allowance for nondisplaceable air. Thus again is emphasized the importance of obtaining thorough compaction in plastic, highly capillary materials.

In contrast to the plastic sections, the nonplastic materials of section 1 of series 1, and section 1 of series 2, had higher percentages of residual or air-filled voids and water contents no higher than those of the plastic materials indicating low capillarity and a susceptibility to gravity drainage.

TABLE 8.—*Composition of track sections at conclusion of traffic tests, series 1 and 2*

	Water content by weight	Composition by volume		
		Aggregate	Water	Air
SERIES 1 [1]	*Percent*	*Percent*	*Percent*	*Percent*
Section 1	5.3	82.8	11.7	5.5
2	4.9	87.0	11.4	1.6
3	5.1	86.4	11.7	1.9
4	5.5	86.2	12.6	1.2
5	6.6	84.0	14.8	1.2
SERIES 2 [2]				
Section 1	6.9	77.8	14.2	8.0
2	6.9	83.9	15.4	.7
3	4.0	89.1	9.4	1.5
4	4.9	87.3	11.3	1.4
5	4.1	89.3	9.7	1.0
6	6.2	85.1	14.0	.9

[1] At 425,000 wheel-trips.
[2] At 330,000 wheel-trips.

TESTS SHOWED IMPORTANCE OF CONTROLLING PLASTICITY AND GRADING

As in the tests of sand-clay mixtures the delineation between good, serviceable materials and those of inferior quality was distinct. Again, the great importance of close control of both plasticity and grading was demonstrated and it was also shown that, where plastic materials are concerned, no amount of control of the quality of the materials will prevent failure if thorough compaction of the materials is not obtained during the construction operations.

Confirmation was found for the belief of some authorities that the behavior of a graded aggregate base-course material is largely dependent on the quality of the soil mortar or material passing the No. 10 sieve. The results of these tests indicate this to be true if more than about 40 percent of the total aggregate passes the No. 10 sieve, while if the total aggregate contains less than about 40 percent of soil mortar the effect of the quality of the soil fraction is modified or obscured by the coarser material. A discussion of the test results leading to this conclusion follows.

Figure 12 which was prepared from the data in table 1 shows the grading curves for the 11 sand-clay-gravel materials used in these tests. The shaded areas which are identical for both series were drawn to include the grading curves of all of the wholly satisfactory materials. They are limited on the left or fine side, as nearly as possible without introducing misleading undulations, by curves for the two borderline materials, section 4 of series 1 and section 4 of series 2. Their limit on the right or coarse side is established by curves for the materials of sections 1 and 3 of series 2, since theirs were the coarsest gradings used.

Figure 13 shows the gradings of the mortars or fractions passing the No. 10 sieve of the 11 sand-clay-gravel materials. The two identical shaded zones, reproduced from figure 13 of the report on sand-clay materials, include the gradings of all the wholly satisfactory sand-clay materials tested in the previous investigation and are limited on the left by curves for the borderline sand-clays and on the right by curves for the coarsest materials used in that investigation.

As shown in figure 13, the grading curves of the mortars of all but one of the sand-clay-gravel materials fall either partially or almost entirely outside the shaded area on the left or fine side. The amount of this divergence has no significance in the case of the nonplastic material of section 1 of series 1, and may not be sufficient for sections 2, 3, and 4 to impair seriously their quality as sand-clay materials for base courses. The divergence is extensive for section 5 of series 1, and sections 2, 3, 4, 5, and 6 of series 2, which leads to the conclusion that the mortars of these sections would be unsatisfactory for use as base-course materials by themselves. The mortar of section 4 of series 2 was the extreme example in this respect and yet because this inferior mortar comprised only 41.9 percent of the total base-course material as tested, the section withstood traffic well enough to be classed in the borderline group.

The mortars of the unsatisfactory sections 2 and 6 of series 2 were virtually identical in grading with those of the satisfactory sections 3 and 5 of series 2. The plasticity indexes of sections 2 and 6, which were respectively 9 and 7, did not differ sufficiently from those of sections 3 and 5, which were respectively 8 and 6, to account even in part for their difference in behavior. The only significant difference was in the percentage of the total aggregate passing the No. 10 sieve. For the unsatisfactory sections 2 and 6, these percentages were 65 and 66.1 as compared to 31.9 and 38 for the satisfactory sections 3 and 5.

FINDINGS USED IN DRAFTING SPECIFICATIONS FOR SOIL AND GRAVEL BASE COURSES

Section 5, series 1, in which a sand-clay material known to be unsatisfactory for use as a base course by itself comprised 47 percent of the sand-clay-gravel mixture, failed quite early in the traffic test.

Thus with definite failures recorded when poorly graded or highly plastic soil appreciably exceeded 40 percent of the aggregate and borderline behavior when 41.9 percent of an unsatisfactorily graded soil mortar was used, while satisfactory service was recorded for sand-clay-gravel mixtures containing 31.9 and 38 percent of poorly graded soil mortar, the critical percentage seems to be quite well established as being in the neighborhood of 40 with the rounded-gravel coarse aggregate used in these tests.

For convenience in studying these relationships, the percentages passing the No. 10 sieve and the plasticity indexes of all the sand-clay-gravel materials as shown in table 1 are repeated in table 9.

TABLE 9.—*Quantity and character of mortar fractions of track sections, and behavior under traffic*

	Fraction of total aggregate passing No. 10 sieve	Plasticity index of fraction passing No. 40 sieve	Behavior of section under traffic
SERIES 1	*Percent*		
Section 1	53.5	0	Satisfactory.
2	47.9	5	Do.
3	47.9	9	Do.
4	50.3	11	Essentially satisfactory.
5	47.0	16	Failed.
SERIES 2			
Section 1	42.3	0	Satisfactory.
2	65.0	9	Failed.
3	31.9	8	Satisfactory.
4	41.9	7	Approached failure.
5	38.0	6	Satisfactory.
6	66.1	7	Failed.

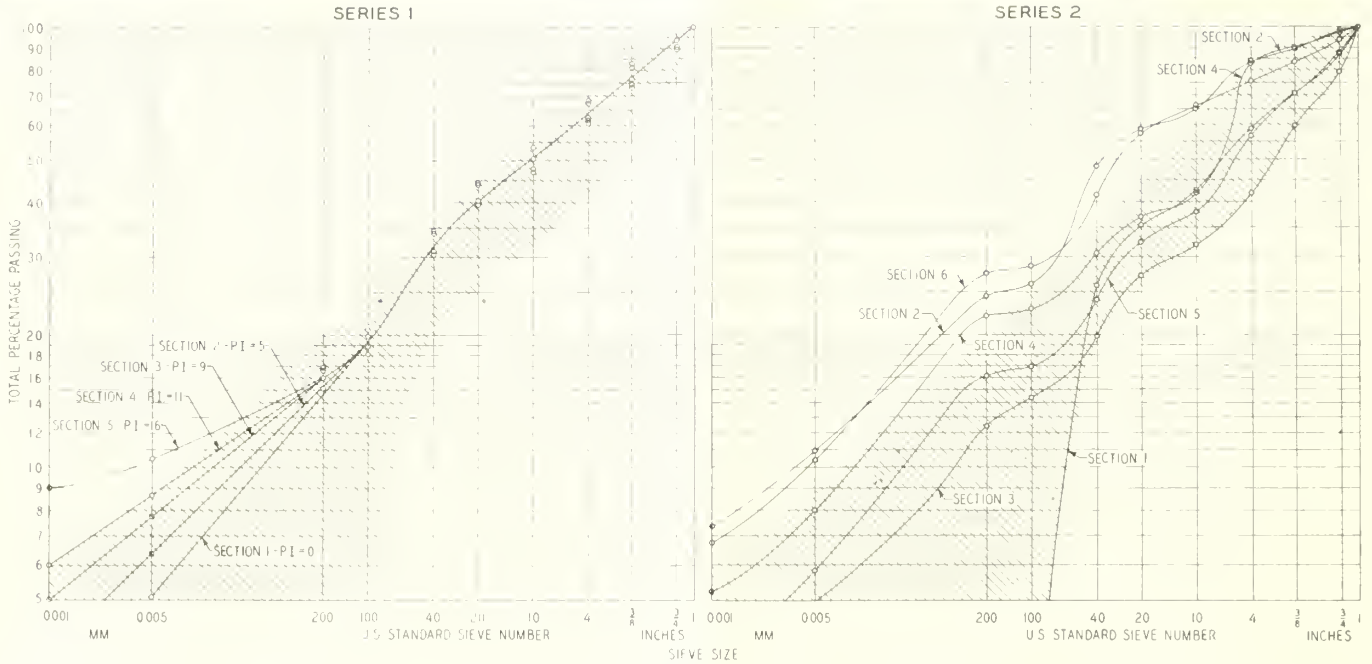

FIGURE 12.—GRADINGS OF MATERIALS IN SERIES 1 AND 2. SHADED AREA INDICATES ZONE WITHIN WHICH ALL THE WHOLLY SATISFACTORY MATERIALS ARE INCLUDED.

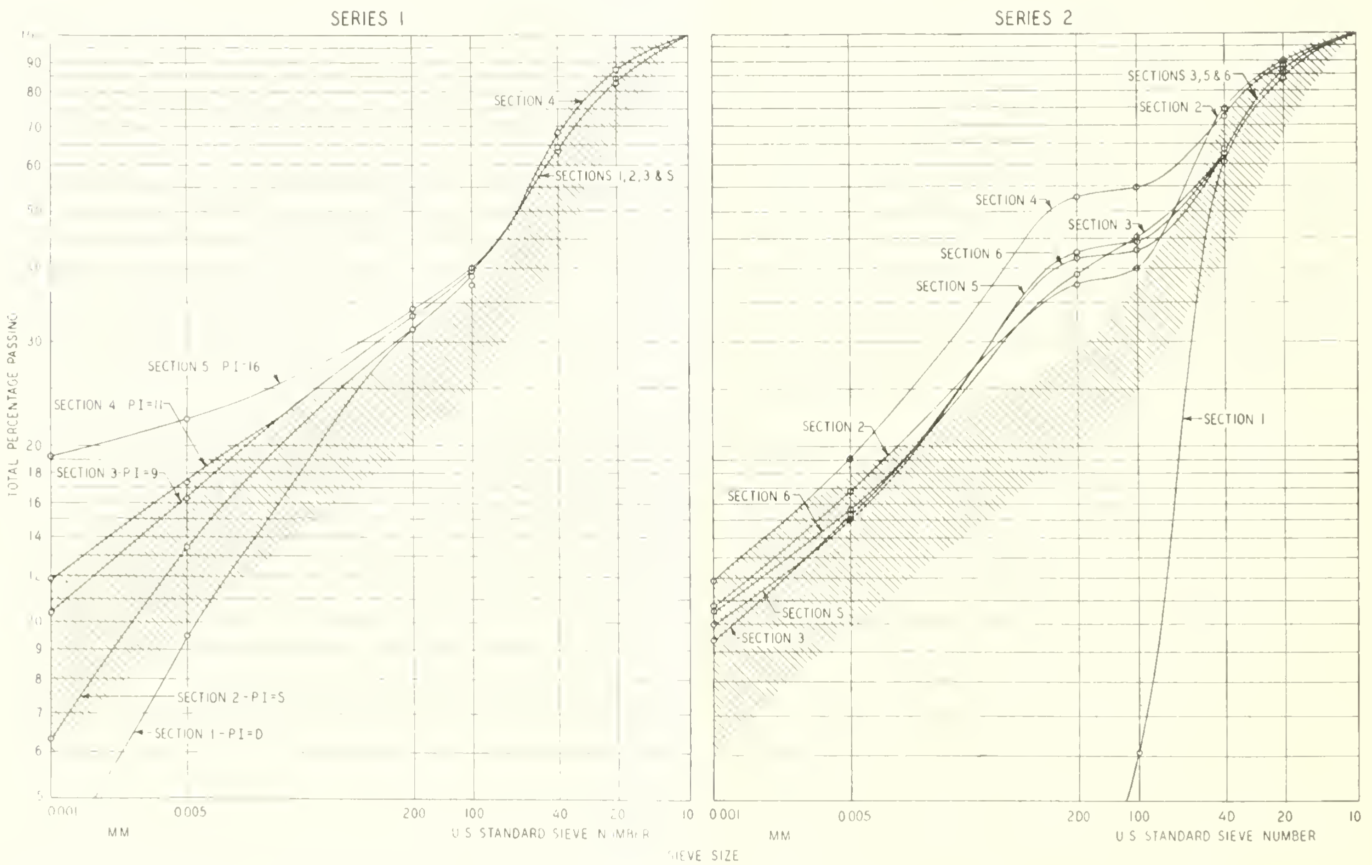

FIGURE 13.—GRADINGS OF THE MORTARS OF THE SAND-CLAY-GRAVEL MATERIALS TESTED. THE SHADED AREAS, REPRODUCED FROM FIGURE 13 OF THE PREVIOUS REPORT ON SAND-CLAY MATERIALS, SHOW THE GRADING RANGE OF THE SATISFACTORY SAND-CLAY BASE-COURSE MATERIALS.

None of the nonplastic materials, either in the tests of sand-clays or of the sand-clay-gravels, showed any indication of lack of stability and it is obvious that mixtures of nonplastic, satisfactorily graded sand-clay materials with coarser aggregate could be expected to make satisfactory base courses for bituminous surfaces regardless of whether 40 percent or even as much as 100

(*Continued on page 16*)

SIMPLIFIED COMPUTATION OF HYDROMETER TEST DATA FOR SOIL

BY THE DIVISION OF TESTS, BUREAU OF PUBLIC ROADS

Reported by EDWARD S. BARBER, Junior Highway Engineer

USE is made of a hydrometer in standard methods of test [1,2] to determine the size distribution of soil grains smaller than 0.05 millimeter in diameter. Direct computation of the size distribution by means of the formulas involved in interpreting the test data is rather laborious. To simplify this work a graphical method of computation was devised by the Bureau of Public Roads.[3] The present report describes a slide rule with special indicator and scales that is particularly useful in obtaining the size distribution when the data are reported as an accumulation curve. Each method has certain advantages and the choice of a particular one depends upon individual preference and the testing equipment used.

In the hydrometer method of mechanical analysis, Stokes' law for the velocity at which a small solid sphere falls through a liquid is used to determine equivalent grain size, which is the diameter of a sphere that would fall at the same velocity as the soil particle. Stokes' formula with reference to soil tests may be written

$$d=\sqrt{\frac{30nL}{980(G-G_1)t}} \qquad (1)$$

in which

d=equivalent grain size in millimeters.
n=viscosity of water, in poises, for any temperature T.
L=distance, in centimeters, through which the soil particles fall during a time, t, in minutes.
G=specific gravity of the soil.
G_1=specific gravity of water.

The numerical values presented in this paper are for a particular Bouyoucos hydrometer calibrated to read grams of soil per liter of suspension (in water) at 67° F. with a soil whose specific gravity is 2.65. However, the method of computation is applicable to any hydrometer of either the Bouyoucos or specific gravity type.

Table 1 gives the equivalent grain size in millimeters under standard conditions of temperature and specific gravity for various combinations of hydrometer reading and time. The viscosity of water is taken as 0.0102 poise at 67° F. and its specific gravity is taken as 1. Following the practice of the Bureau of Public Roads,[4] L in formula (1) is taken as 0.42 times the distance from the surface of the suspension to the bottom of the hydrometer.

Table 2 gives the factors by which the grain sizes of table 1 are multiplied to correct for variations in temperature and specific gravity. The temperature correction factor for G=2.65 is $\sqrt{\frac{n}{0.0102}}$ and the specific gravity correction factor for T=67° F. is $\sqrt{\frac{2.65-1}{G-1}}$.[4] The combined correction factor for both temperature and specific gravity is obtained by multiplying the temperature correction factor by the specific gravity correction factor.

For example, if t is 2 minutes and the hydrometer reading, H, is 34, table 1 gives a grain size of 0.0266 millimeters. Then for T=70° F. and G=2.6, table 2 gives a combined correction factor of 0.995. The product 0.0266 times 0.995 gives 0.0265 millimeters as the corrected value of the equivalent grain size.

TABLE 1.—*Equivalent grain sizes in millimeters under standard conditions* [1] *computed from Stokes' formula*

Bouyoucos hydrometer reading, H	Equivalent grain size for periods of sedimentation of—							
	1 minute	2 minutes	5 minutes	15 minutes	30 minutes	60 minutes	250 minutes	1,440 minutes
	Mm.	*Mm.*	*Mm.*	*Mm.*	*Mm.*	*Mm.*	*Mm.*	*Mm.*
0	0.0435	0.0307	0.0194	0.0112	0.0079	0.0056	0.00275	0.00115
2	.0432	.0305	.0193	.0111	.0079	.0056	.00273	.00114
4	.0428	.0303	.0192	.0111	.0078	.0055	.00271	.00113
6	.0425	.0300	.0190	.0110	.0078	.0055	.00269	.00112
8	.0422	.0298	.0189	.0109	.0077	.0054	.00267	.00111
10	.0418	.0296	.0187	.0108	.0076	.0054	.00265	.00110
12	.0415	.0293	.0186	.0107	.0076	.0054	.00262	.00109
14	.0411	.0291	.0184	.0106	.0075	.0053	.00260	.00108
16	.0408	.0288	.0182	.0105	.0074	.0053	.00258	.00107
18	.0404	.0285	.0180	.0104	.0074	.0052	.00255	.00106
20	.0400	.0283	.0179	.0103	.0073	.0052	.00253	.00105
22	.0397	.0281	.0178	.0102	.0072	.0051	.00251	.00105
24	.0394	.0279	.0176	.0102	.0072	.0051	.00249	.00104
26	.0391	.0276	.0175	.0101	.0071	.0050	.00247	.00103
28	.0387	.0274	.0173	.0100	.0071	.0050	.00245	.00102
30	.0383	.0271	.0172	.0099	.0070	.0050	.00243	.00101
32	.0380	.0269	.0170	.0098	.0069	.0049	.00240	.00100
34	.0377	.0266	.0168	.0097	.0069	.0049	.00238	.00099
36	.0373	.0264	.0167	.0096	.0068	.0048	.00236	.00098
38	.0369	.0261	.0165	.0095	.0067	.0048	.00234	.00097
40	.0366	.0259	.0164	.0094	.0067	.0047	.00231	.00096
42	.0362	.0256	.0162	.0094	.0066	.0047	.00229	.00095
44	.0359	.0254	.0160	.0093	.0066	.0046	.00227	.00094
46	.0355	.0251	.0159	.0092	.0065	.0046	.00224	.00093
48	.0351	.0248	.0157	.0091	.0064	.0045	.00222	.00092
50	.0347	.0245	.0155	.0090	.0063	.0045	.00220	.00091

[1] This table is for a particular hydrometer calibrated to read grams of soil per liter of suspension at 67° F. with a soil whose specific gravity is 2.65. The calibration for distance of fall is given in table 4.

TABLE 2.—*Combined correction factors* [1] *for temperature and specific gravity applied to Stokes' formula*

Temperature, degrees (F.)	Correction factor for soils having specific gravities of—								
	2.2	2.3	2.4	2.5	2.6	2.65	2.7	2.8	2.9
60	1.228	1.182	1.138	1.100	1.065	1.048	1.033	1.004	0.977
65	1.190	1.144	1.102	1.064	1.031	1.015	1.000	.972	.946
67	1.172	1.127	1.086	1.048	1.016	1.000	.985	.958	.932
70	1.149	1.104	1.064	1.027	.995	.980	.965	.939	.913
75	1.113	1.068	1.029	.995	.965	.949	.935	.909	.885
80	1.077	1.035	.998	.963	.934	.919	.905	.880	.857
85	1.044	1.004	.968	.934	.905	.891	.878	.853	.830
90	1.015	.976	.940	.908	.880	.866	.853	.829	.807

[1] A grain size under standard conditions as given in table 1 is multiplied by a correction factor from this table to correct for values of temperature and specific gravity other than 67° F. and 2.65, respectively.

[1] Tentative Method of Mechanical Analysis of Soils. Proceedings of the American Society for Testing Materials, vol. 35, pt. I, 1935, p. 953.

[2] Standard Method of Mechanical Analysis of Soils, Method T-88-38 Standard Specifications for Highway Materials and Methods of Sampling and Testing, 1938, p. 291. Published by the American Association of State Highway Officials.

[3] Graphical Solution of the Data Furnished by the Hydrometer Method of Analysis, by E. A. Willis, F. A. Robeson, and C. M. Johnston. PUBLIC ROADS, vol. 12, No. 8, October 1931.

[4] Procedures for Testing Soils for the Determination of the Subgrade Soil Constants, by A. M. Wintermyer, E. A. Willis, and R. C. Thoreen. PUBLIC ROADS, vol. 12, No. 8, October 1931.

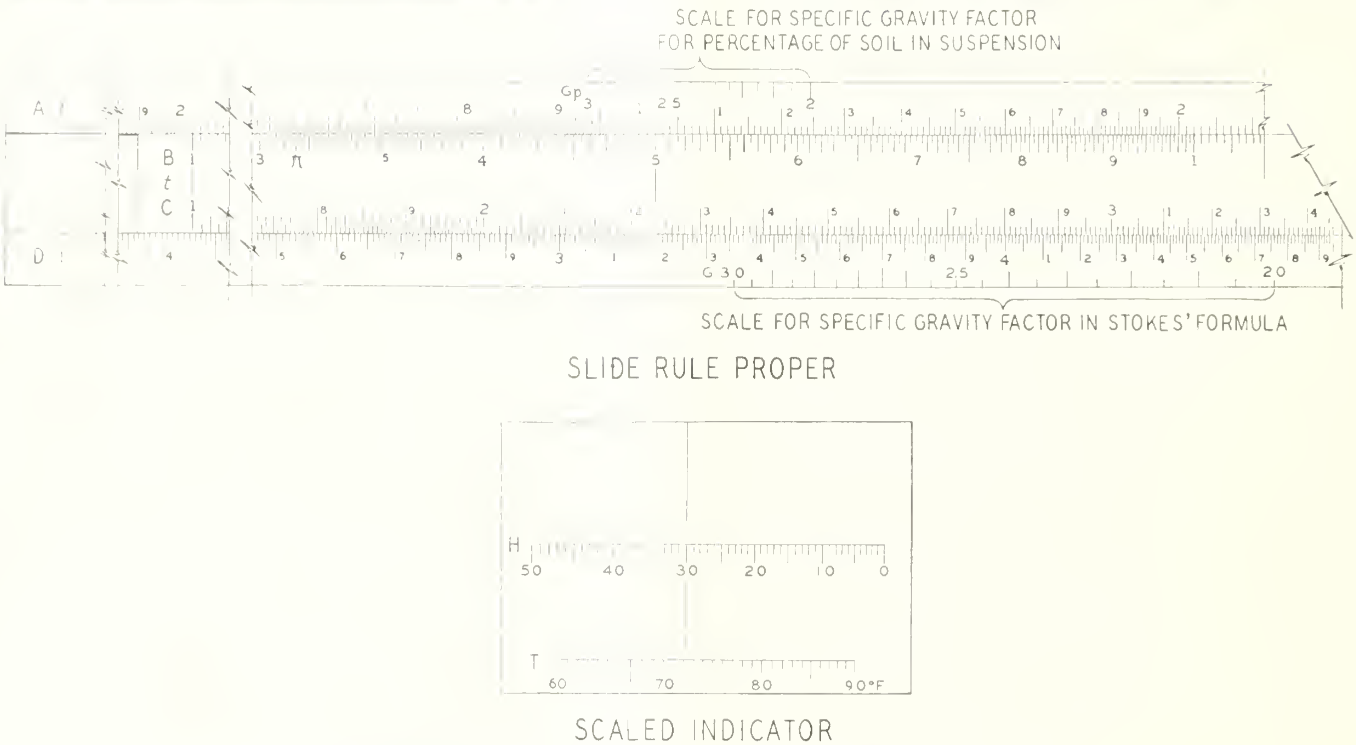

Figure 1.—Slide Rule and Indicator With Supplementary Scales.

SLIDE RULE USED TO COMPUTE PARTICLE SIZES

Values shown in tables 1 and 2 may be indicated graphically on a slide rule. The desirability of doing this will depend upon the operator's preference and the test method used. For example, if all tests are made at the same temperature and an average specific gravity is assumed, a tabulation such as table 1 gives the grain sizes directly. For the more general case, where both temperature and specific gravity are different for each test, the following method can be used for adapting a slide rule so as to facilitate computation of grain sizes by Stokes' formula without the use of tables or charts.

Stokes' formula may be written

$$d^2=\frac{30\times0.0102}{980\times1.65}\times\frac{n}{0.0102}\times\frac{1.65}{G-1}\times L\times\frac{1}{t}\text{------}(2)$$

with the specific gravity of water taken as 1.

Table 3 gives values of $\frac{n}{0.0102}$ for various temperatures. As shown in figure 1, marks for these temperatures are scribed as the T scale on the indicator so as to abut the lower edge of the face of the rule with the indicator in place (see fig. 2). The positions of the

Table 3.—*Temperature factors in Stokes' formula*

Temperature T	Temperature factor $\frac{n}{0.0102}$
Degrees F.	
60	1.10
65	1.03
67	1.000
70	.959
75	.899
80	.844
85	.794
90	.749

marks for the temperature values correspond to the positions of the values of $\frac{n}{0.0102}$ on the A (or B) scale with the direction reversed. To mark the T scale on the underside of the indicator, it is turned over in the direction of the length of the rule and the lower edge of the indicator is placed over the A scale so as to cover the interval from 0.749 to 1.10 on that scale, the range of values of table 3. The mark for $T=60°$ F. is then scribed on the indicator at 1.10 on the A scale, the mark for 65 at 1.03, the mark for 67 at 1.00, etc.,

Similarly, the hydrometer readings given in table 4 are scribed on the indicator, giving the H scale (see fig. 1). The positions of the marks on the H scale correspond to the positions of the values of L on the A (or B) scale, but this time with the indicator in its normal position. Thus, when the mark for zero hydrometer readings is opposite 1 (or 10) on the A scale, the mark for 50 grams per liter on the H scale will be opposite the corresponding height of fall which is 6.37.

Table 4.—*Distances of fall in Stokes' formula*

Hydrometer reading, H	Distance of fall,[1] L
Grams per liter	*Centimeters*
0	10.00
10	9.25
20	8.47
30	7.78
40	7.07
50	6.37

[1] These values, for a particular Bouyoucos hydrometer, are taken as 0.42 times the distance from each hydrometer reading or the surface of the suspension to the bottom of the hydrometer.

The values of the specific gravity factors, as given in table 5, are scribed below the D scale of the slide rule proper (see upper diagram of fig. 1). The position of

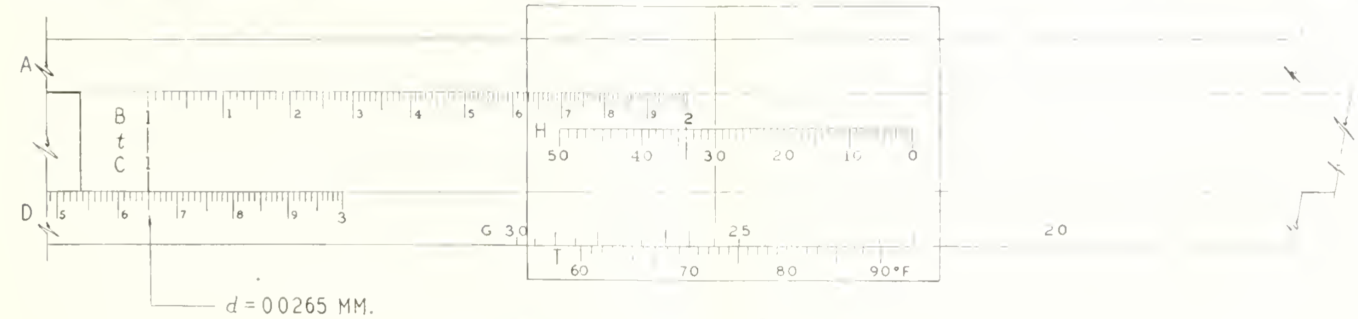

FIGURE 2.—SLIDE RULE SET FOR STOKES' FORMULA.

this scale in relation to the A scale is important. The value of the right-hand side of equation (2) is 0.001892 for $t=1$ minute, $G=2.65$, $n=0.0102$ poise ($T=67°$ F.), and $L=10$ centimeters ($H=0$). To determine the position of the mark for $G=2.65$, the indicator is placed so that the mark for $H=0$, is directly opposite 1892 on the A scale. The mark for 2.65 on the specific gravity scale (fig. 1) is scribed directly opposite the mark for $T=67°$ F. This corresponds to a specific gravity factor of 1.00. The remainder of the scale (2.0 to 3.0) is scribed relative to the mark for 2.65 using the A scale in its normal direction as the measure of length.

The time, t, is given directly on the B scale although it may be convenient to draw lines across the longitudinal center line of the slide (as shown in fig. 1 at 1 and 5 of upper diagram) corresponding to the usual time schedule for reading the hydrometer.

The solution of the same example as given in the description of the use of tables 1 and 2 is illustrated in figure 2. The indicator is moved to bring 70° F. on the T scale opposite 2.6 on the G scale. With the position of the indicator thus fixed, the slide is moved to bring 2 minutes on the B (or t) scale opposite 34 on the H scale. The grain size in millimeters is then read as 0.0265 on the D scale opposite 1 (or 10) on the C scale as shown in figure 2.

TABLE 5.—*Specific gravity factors in Stokes' formula*

Specific gravity G	Specific gravity factor $\frac{1.65}{G-1}$
2.0	1.650
2.1	1.500
2.2	1.375
2.3	1.268
2.4	1.178
2.5	1.099
2.6	1.031
2.65	1.000
2.7	.971
2.8	.917
2.9	.869
3.0	.825

COEFFICIENT OF PERMEABILITY COMPUTED USING SLIDE RULE

If the temperature is kept practically constant during a single test, one setting of the indicator will do for all of the time readings so that only one movement of the slide is required to determine each grain size, the indicator remaining fixed in position.

The slide rule may also be used conveniently for the reverse procedure of determining the time interval corresponding to a specific grain size for any temperature and specific gravity.

A slide rule method very similar to the one just described is now being used in computing the coefficient of permeability from test data in which the principle of the falling-head permeameter is used.

The formula used for determining the percentage of initially dispersed soil remaining in suspension [4] when the hydrometer method of mechanical analysis is used may be conveniently computed on a slide rule. This formula is

$$P=\frac{(H+\Delta H)f}{W_0}\times 100 \qquad (3)$$

in which

P = percentage of initially dispersed soil remaining in suspension at the time a hydrometer reading is taken.

W_0 = weight in grams of soil per liter of suspension initially dispersed.

H = hydrometer reading, grams of soil per liter of suspension.

ΔH = temperature correction for density of water (see values of table 6).

f = correction factor for G, the specific gravity of the soil (see table 7).

that is

$$f=\frac{2.65-1}{2.65}\times\frac{G}{G-1}$$

TABLE 6.—*Temperature corrections in formula for percentage of soil in suspension*

Temperature T	Temperature correction,[1] ΔH
Degrees F.	*Grams per liter*
60	−0.8
61	−.7
62	−.6
63	−.5
64	−.4
65	−.3
66	−.1
67	.0
68	.1
69	.2
70	.4
71	.5
72	.7
73	.8
74	1.0
75	1.2
76	1.4
77	1.6
78	1.8
79	2.0
80	2.2
81	2.4
82	2.6
83	2.8
84	3.0
85	3.2
90	4.3

[1] Experimental values.

[4] Procedures for Testing Soils for the Determination of the Subgrade Soil Constants, by A. M. Wintermyer, E. A. Willis, and R. C. Thoreen. PUBLIC ROADS, vol. 12, No. 8, October 1931.

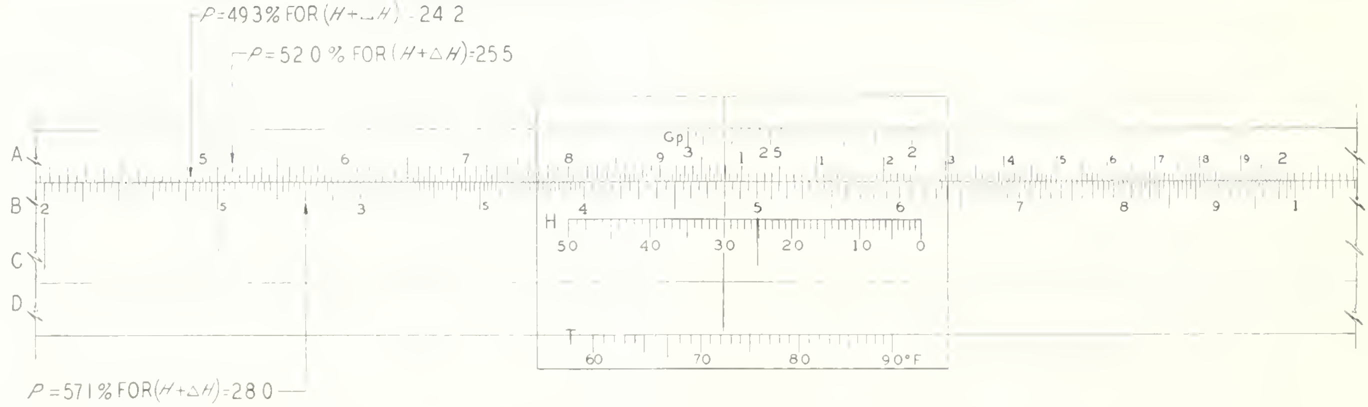

FIGURE 3.—SLIDE RULE SET FOR PERCENTAGE OF SOIL IN SUSPENSION.

TABLE 7.—*Specific gravity correction factors in formula for percentage of soil in suspension*

Specific gravity G	Specific gravity correction factor $f=\frac{2.65-1}{2.65}\times\frac{G}{G-1}$
2.0	1.242
2.1	1.187
2.2	1.141
2.3	1.101
2.4	1.067
2.5	1.038
2.6	1.012
2.65	1.000
2.7	.989
2.8	.969
2.9	.951
3.0	.934

Values of f are given in table 7. Marks for values of G are scribed above and opposite corresponding values of f on the A scale to make the G_p scale shown in figure 1. Thus, the mark for $G=3.0$ on the G_p scale is opposite a value of f of 0.934 on the A scale, $G=2.0$ is opposite 1.242 on the A scale, etc. The temperature correction is to be applied by mental arithmetic and to aid in this a table of values such as table 6 is fixed on the back of the slide rule or in any other convenient place. This completes the equipment.

To illustrate the method of computation, assume the specific gravity of the soil=2.75, the temperature=75° F., $W_0=48$ grams, and $H=26.8$, 24.3, 23.0, etc. As shown in figure 3, the indicator is moved to bring its index line to 2.75 on the G_p scale. The slide is moved to bring 48 on the B scale under the index line. The position of the slide remains thus set for the series of hydrometer readings. Referring to table 6 for values of T and ΔH, it is found that for $T=75°$ F., $\Delta H=1.2$. Opposite $H+\Delta H=26.8+1.2=28.0$ on the B scale, $P=57.1$ percent is read on the A scale as shown in figure 3. Similarly, opposite $24.3+1.2=25.5$ on the B scale, 52.0 percent is read on the A scale as the value of P; opposite 24.2, $P=49.3$ percent, etc.

The slide rule method of computing the percentage of soil in suspension is further simplified when used in conjunction with the calculating board[3] devised by the Bureau for computing, correcting, and interpolating values for specific grain sizes. In this case, the temperature correction in grams per liter is taken care of by moving vertically the transparent paper on which the uncorrected hydrometer readings in grams per liter are plotted on a vertical scale. In this way, both the temperature correction table and mental arithmetic are eliminated.

[3] Graphical Solution of the Data Furnished by the Hydrometer Method of Analysis, by E. A. Willis, F. A. Robeson, and C. M. Johnston. PUBLIC ROADS, vol. 12, No. 8, October 1931.

(*Continued from page 12*)

percent of the total aggregate passed the No. 10 sieve. The same should be true of plastic sand-clays that would be satisfactory as base-course materials without coarse aggregate.

The nonplastic materials raveled somewhat before the bituminous surface was applied and thorough compaction was difficult to obtain unless the material was kept very wet either by frequent sprinkling or by maintaining the ground water at a high elevation.

In general the materials having a plasticity index of 5 (section 2 of series 1 of the sand-clays and section 2 of series 1 of the sand-clay-gravels) were superior to either the nonplastic materials or the more plastic ones, both as to ease of compaction and resistance to displacement in the traffic tests.

The results of some of the early tests in this investigation were made available to the Committee on Materials of the American Association of State Highway Officials at the time it was considering requirements for soil and gravel base courses and were utilized in connection with the drafting of the specifications given in table 10.

TABLE 10.—*Specifications for soil and gravel base courses*

	Type B base courses			
	B-1, 1 inch maximum size		B-2, 2 inch maximum size	
	Minimum	Maximum	Minimum	Maximum
Percentage passing:				
2-inch sieve				100
1½-inch sieve			70	100
1-inch sieve		100	55	85
¾-inch sieve	70	100	50	80
⅜-inch sieve	50	80	40	70
No. 4 sieve	35	65	30	60
No. 10 sieve	25	50	20	50
No. 40 sieve	15	30	10	30
No. 200 sieve	5	15	5	15
Percentage of material finer than No. 40 sieve passing No. 200 sieve		50		50
Liquid limit (material finer than No. 40 sieve)		25		25
Plasticity index (material finer than No. 40 sieve)		6		6

Final analysis of the test data after completion of the entire test program indicated that the limits established are well drawn to insure that highly satisfactory base-course materials will be obtained. It was realized at the time the specifications were written that some fully satisfactory materials would be excluded.

Complete analysis of the test data indicates that if the amount of soil mortar in a well-graded sand-clay-gravel is low, the quality of the material depends largely on the grading of the coarse fraction and that the grading and plasticity of the soil mortar is of relatively less vital importance. Thus, for example, if a material fails to meet specification B-1 because less than 25 percent passes the No. 10 sieve, it is doubtful whether it should be classified as a sand-clay-gravel at all even though the plasticity index of the small amount of soil present might be quite high. It is believed that numerous materials of this type are likely to be encountered and that, provided they are well graded from the No. 10 sieve to the maximum size and that the aggregate particles are somewhat angular, they can be used successfully without applying such rigorous requirements to the character of the soil fraction as are set for the sand-clay-gravels. A special specification may be desirable to cover such materials.

Compaction and surface interlocking of such materials would be slow and somewhat difficult to obtain and the temptation would be great to add clay or other soil binder to hasten surface bonding. However, once compacted to a degree producing interlocking of the aggregate, and bonded at the top with a mixed or drag type of prime treatment, rolled to set the surface, they could be expected to provide a more satisfactory base structure than would be obtained by the addition of plastic soil binder in an attempt to bring these granular materials into conformity with the specifications for sand-clay-gravel base-course materials.

CONCLUSIONS

1. Control of grading is essential to insure satisfactory stability.

2. Control of plasticity index is essential, particularly when the aggregate contains as much as 40 percent of soil mortar.

3. As the amount of soil mortar decreases below 40 percent, the importance of the grading of the coarse material becomes relatively more important and the grading and plasticity index of the soil mortar becomes of relatively less vital importance.

4. Although there may be many instances where carefully mixed and placed aggregates having relatively low mortar contents would give satisfactory service even though the plasticity index of the soil mortar might be in excess of 6, the possibility of segregation and collection of the fine aggregate into rich spots or layers must not be overlooked and makes the limit of 6 for the plasticity index a desirable if not vitally necessary requirement.

5. A well-graded sand-clay-gravel material having a plasticity index of about 5 is to be preferred to absolutely nonplastic materials of comparable grading and is decidedly superior to those having appreciably higher plasticity indexes.

6. Thorough compaction of even the best plastic base-course materials to essentially the maximum density obtainable in the laboratory by the vibratory method of compaction is absolutely essential to prevent softening and loss of stability where water may reach the material after construction.

7. Thorough compaction of the plastic materials was obtained in the circular track tests by starting compaction operations with an excess of moisture of about 1.5 to 2 percent over the optimum as determined by the Proctor test on the portion of the aggregate passing the No. 10 sieve or, for the total sand-clay-gravel aggregate, sufficient moisture to fill the aggregate voids when compacted to the maximum obtainable density by vibration.

8. It is most important that compaction operations be continued during the drying out of the above-mentioned excess water since the combined action of compaction and drying is necessary to produce the required densities.

9. Some additional moisture may be required in handling plastic materials to provide for drying losses during mixing and leveling operations, but care should be taken not to surpass actual needs since any great excess of moisture will delay final compaction.

10. In connection with nonplastic materials, the term "optimum moisture content" has little or no significance. It is therefore not necessary to limit the amount of water used in mixing and compacting such materials since water drains out rapidly, making it difficult to maintain them in a wet enough condition to aid materially in obtaining compaction. The only precaution necessary is that softening of the subgrade shall not be caused by the excessive use of water.

11. Compaction should be as complete at the bottom of the base course as at the top, particularly with plastic materials, since even minor deficiencies in the compaction of plastic materials make them susceptible to softening and loss of stability when wet.

12. The tests on the materials of series 1, which contained from 47 to 53.5 percent of soil mortar, indicated that freezing and thawing would be likely to cause failure of the borderline material of section 4, the plasticity index of which was 11, and that serious damage might be done to section 3, with a plasticity index of 9, since the one cycle of freezing and thawing caused a marked increase in the rate of displacement under traffic in this section (see fig. 4). This is a further argument for placing a maximum limit on the plasticity index for base-course materials.

13. It might be desirable to promulgate a separate specification to cover essentially stone or gravel base-course materials having satisfactory gradings but very low soil-mortar contents to allow the use of such materials without too rigorous limitation of plasticity index. The design of such base courses would necessitate provision for a mixed type of prime using somewhat more viscous bituminous materials than are commonly used for the ordinary penetration prime treatment.

THE COVER PICTURE

Rounding cut slopes encourages vegetative growth and accomplishes the dual purpose of improving roadside appearance and preventing erosion along the Connecticut highway shown in the cover picture. Further flattening and rounding would have been desirable, but in this instance were prevented by limitations imposed by the right-of-way width. Establishing growth of native plants on cut slopes serves to reduce erosion and consequent clogging of drainage ditches, thereby reducing highway maintenance costs.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF FEBRUARY 28,1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 5,769,382	$ 2,600,520	174.4	$ 8,529,342	$ 4,252,875	343.0	$ 806,180	$ 402,485	22.0	$ 3,568,580
Arizona	2,046,153	1,529,291	109.0	1,341,336	930,543	57.8	376,056	232,676	7.8	2,036,457
Arkansas	1,152,857	1,138,638	81.4	3,467,312	3,463,658	212.4	529,044	526,119	34.9	1,865,756
California	9,322,768	5,103,695	207.5	5,322,952	2,866,205	80.8	1,486,000	792,749	37.7	4,897,534
Colorado	2,565,088	1,372,939	99.3	2,734,853	1,456,727	85.0	1,531,850	851,780	31.5	2,858,094
Connecticut	934,030	455,835	8.9	684,518	337,455	7.9				2,149,026
Delaware	485,437	241,521	14.1	708,131	349,289	9.9	259,491	125,600	6.6	1,625,207
Florida	2,046,100	1,020,204	47.6	3,208,138	1,604,069	57.5	618,200	309,100	13.0	3,781,441
Georgia	4,621,212	2,252,348	226.8	5,118,910	2,559,455	258.1	1,325,000	662,500	83.3	7,430,333
Idaho	2,089,523	1,214,155	200.7	1,189,537	710,407	38.9	270,262	161,091	10.9	2,174,894
Illinois	11,001,009	5,459,081	299.7	7,612,426	3,802,614	161.6	3,247,011	1,623,460	84.2	4,858,999
Indiana	5,587,556	2,743,351	146.5	2,798,314	1,399,157	51.6	2,580,951	1,238,090	63.7	4,129,102
Iowa	7,638,612	3,567,962	258.4	4,555,632	1,922,833	146.5	489,798	147,600	34.5	2,635,539
Kansas	5,044,710	2,510,360	708.5	4,202,257	2,101,103	192.0	3,165,218	1,575,609	166.9	4,692,076
Kentucky	5,592,733	2,776,123	209.3	2,743,030	1,371,515	59.0	1,117,168	558,782	33.3	3,883,951
Louisiana	1,294,187	646,891	38.2	11,144,987	2,656,864	31.7	1,479,276	634,892	27.1	3,229,166
Maine	2,797,885	1,394,536	65.5	1,570,121	785,059	32.8	174,108	87,054	1.4	1,015,391
Maryland	1,085,456	542,728	17.1	2,510,978	1,235,351	40.7	776,570	374,000	10.3	2,349,285
Massachusetts	1,870,824	935,409	9.0	2,953,970	1,476,526	19.4	1,233,099	612,765	15.5	3,295,441
Michigan	7,866,671	3,752,055	166.1	4,152,608	2,075,652	120.5	1,093,995	536,721	27.0	3,865,266
Minnesota	4,758,194	2,294,739	289.3	6,028,875	2,991,668	279.3	1,153,760	575,975	61.2	4,659,945
Mississippi	2,049,958	940,417	82.8	10,802,762	4,159,632	476.7	1,380,700	552,400	48.5	3,337,471
Missouri	5,731,742	2,730,036	151.9	2,868,298	1,405,166	66.8	5,000,559	2,375,530	223.2	4,871,311
Montana	1,658,740	932,323	83.6	826,978	464,810	17.6	994,424	559,348	69.9	5,906,459
Nebraska	3,472,047	1,684,635	329.6	5,453,601	2,749,345	411.1	4,027,832	1,906,864	377.9	2,888,027
Nevada	1,443,091	1,236,906	168.8	1,323,594	1,141,689	51.0	465,486	402,580	10.0	1,602,137
New Hampshire	983,828	487,160	22.4	382,110	190,095	3.3	93,802	46,900	1.4	1,635,822
New Jersey	1,897,135	939,155	16.7	2,497,376	1,246,963	15.9	1,513,280	755,810	14.5	2,860,028
New Mexico	2,034,273	1,239,748	241.8	2,153,263	1,380,102	85.7				2,006,595
New York	14,147,393	6,883,224	253.2	9,977,597	4,942,852	158.8	3,347,230	1,527,398	60.2	4,985,830
North Carolina	6,559,903	3,150,078	264.3	4,978,219	2,488,152	332.8	1,021,360	492,020	54.7	3,636,840
North Dakota	3,375,916	3,236,625	261.5	500,901	290,805	57.5	69,522	37,236	6.8	5,096,362
Ohio	8,337,363	4,082,883	99.0	7,235,262	3,608,602	72.8	2,502,440	1,190,920	27.0	8,758,868
Oklahoma	5,720,508	2,991,823	246.4	3,101,863	1,593,951	64.3	1,487,800	791,645	46.7	4,500,059
Oregon	3,154,227	1,849,620	110.7	1,524,478	930,767	83.5	810,717	494,470	39.0	2,791,357
Pennsylvania	8,432,467	4,174,025	140.6	7,652,531	3,807,110	78.3	3,791,468	1,744,007	31.4	5,627,630
Rhode Island	1,179,290	589,645	16.4	372,212	186,106	3.5	63,560	31,780	.6	1,516,518
South Carolina	4,841,632	2,134,948	249.2	3,074,433	1,377,876	92.3	278,191	133,400	11.1	2,493,886
South Dakota	1,928,842	1,080,884	245.7	4,674,676	2,585,190	441.5	339,070	187,490	27.8	4,182,927
Tennessee	5,454,830	2,699,117	176.7	2,802,829	1,402,136	42.8	1,305,780	652,890	42.8	5,475,339
Texas	12,315,146	6,090,670	803.1	13,549,227	6,686,521	611.7	3,792,062	1,765,950	244.3	8,485,063
Utah	1,063,013	740,744	102.1	2,139,223	1,517,650	73.4	109,720	78,320	4.2	1,605,990
Vermont	1,281,653	577,197	33.9	722,784	343,793	17.7	196,970	98,295	4.3	661,932
Virginia	5,951,973	2,973,806	205.5	2,940,176	1,466,528	89.0	849,440	424,135	10.5	2,169,747
Washington	4,011,591	2,090,017	99.6	2,679,151	1,402,216	35.8	537,730	282,400	3.0	2,023,120
West Virginia	1,772,403	1,273,366	64.6	1,296,682	649,266	26.5	715,470	357,735	25.4	3,091,790
Wisconsin	4,765,180	2,352,799	165.9	6,840,374	3,202,280	180.9	215,800	105,600	2.5	3,493,786
Wyoming	2,571,344	1,544,992	281.4	586,552	356,921	64.5	676,930	420,310	43.9	1,405,554
District of Columbia										487,500
Hawaii	821,861	408,514	18.0	845,010	414,380	9.3	303,340	149,310	7.7	1,546,517
Puerto Rico	189,737	92,320	4.4	1,502,199	747,055	30.2	217,822	107,520	6.8	774,980
TOTALS	202,717,473	104,760,058	8,317.1	187,882,588	93,086,984	5,981.6	59,821,542	29,701,311	2,218.9	170,920,928

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF FEBRUARY 28,1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 234,900	$ 117,450	18.4	$ 834,850	$ 412,050	38.6				$ 833,746
Arizona	321,282	204,457	20.6	140,213	98,773	15.5	$ 65,597	$ 47,235	6.2	519,527
Arkansas	13,126	6,563		268,515	266,169	25.8	231,550	230,808	30.3	615,955
California	1,319,726	752,403	87.3	822,633	456,223	52.1	745,703	361,702	14.7	846,581
Colorado	850,097	457,492	52.1	402,820	223,165	18.3	220,370	113,009	7.8	399,090
Connecticut	69,450	34,705	1.3	41,584	20,632	.2	201,480	71,130	3.0	285,414
Delaware	18,950	9,475	4.1	47,050	23,525	8.8	45,790	22,410	9.9	264,590
Florida				482,755	240,561	9.8	120,900	60,450	5.4	574,094
Georgia	297,712	139,781	39.7	629,686	314,843	78.2	120,360	60,180	16.6	1,120,199
Idaho	451,521	204,347	46.9	166,441	87,870	11.9	24,812	14,825	1.1	350,797
Illinois	1,622,533	809,530	139.9	1,386,732	639,366	71.5	384,500	183,750	28.0	1,002,676
Indiana	594,468	252,894	64.8	818,900	391,650	71.5	507,968	244,306	45.7	717,475
Iowa										1,679,807
Kansas	142,690	71,344	14.5	140,362	70,181	10.3	399,116	199,558	33.8	1,365,764
Kentucky	790,621	245,084	106.1	701,106	181,576	23.0	899,303	254,970	96.2	410,817
Louisiana	64,068	31,900	6.9	718,131	309,375	54.8	381,278	177,160	34.0	421,213
Maine	361,909	180,746	23.3	262,662	126,214	12.5	27,500	13,750	2.1	140,191
Maryland				145,174	67,987	11.2	220,800	82,855	14.2	384,839
Massachusetts				139,441	69,604	2.4	222,970	110,905	4.9	672,214
Michigan	409,561	203,281	37.0	684,304	341,402	42.2	553,500	269,800	29.1	1,173,005
Minnesota	280,091	131,160	42.2	581,324	288,618	42.9	232,968	116,484	19.6	1,248,943
Mississippi				299,000	149,500	23.8	44,700	22,350	16.9	979,016
Missouri	418,039	201,627	52.8	464,930	190,770	38.6	600,590	274,300	92.1	846,939
Montana				27,601	15,525					1,324,368
Nebraska	499,150	247,436	85.5	432,748	210,143	73.5	482,500	237,780	91.4	609,099
Nevada	424,798	354,271	68.8	120,241	104,184	15.5	26,563	23,035	1.6	205,756
New Hampshire	245,058	121,630	6.0	60,759	29,708	2.3				168,662
New Jersey	123,040	61,520	2.4	199,860	91,195	2.6	119,150	59,575	2.9	658,688
New Mexico	563,056	343,405	36.9	619,603	377,887	41.0				324,774
New York	2,311,517	1,131,920	166.3	1,899,000	949,500	99.6				1,007,964
North Carolina	630,222	314,580	74.8	924,144	462,050	74.3	125,500	58,130	18.0	595,623
North Dakota	51,622	27,362	9.0	169,910	90,999	26.1	42,770	22,907	8.2	875,809
Ohio	147,535	73,767	3.8	100,970	57,260	1.5	496,000	248,000	27.1	1,990,092
Oklahoma	249,090	131,417	29.8	213,642	113,676	13.1	602,040	297,148	32.4	990,263
Oregon	425,096	247,170	56.3	59,085	36,102	5.0	194,932	116,570	23.9	601,114
Pennsylvania	1,722,413	827,559	123.1	1,789,367	876,902	97.5	659,038	329,519	32.3	716,964
Rhode Island	66,840	33,420	3.5	162,675	81,314	4.8	74,070	37,035	.9	109,247
South Carolina	404,550	174,382	43.5	834,787	349,369	90.5	190,290	75,500	14.4	278,661
South Dakota				11,300	6,250					1,058,050
Tennessee	259,120	129,560	14.8	601,844	228,022	26.8	136,580	61,820	3.2	952,058
Texas	2,650,179	1,190,664	367.0	1,839,444	850,653	211.4	978,260	445,439	95.4	1,572,076
Utah	450,730	230,606	41.1	303,702	152,870	21.1	144,360	72,004	19.4	263,600
Vermont	238,385	109,790	13.8	90,306	45,153	4.0	43,300	20,500	.5	107,278
Virginia	571,647	246,135	61.5	705,620	340,343	48.5	163,862	81,931	24.0	496,316
Washington	549,807	286,426	63.7	509,591	268,196	23.4	312,037	163,900	19.0	246,606
West Virginia	245,806	122,025	21.4	117,096	58,548	6.2	36,200	18,100	2.1	513,306
Wisconsin	509,819	242,528	23.1	672,417	329,580	31.6	74,771	37,260	1.4	965,276
Wyoming	416,281	254,573	59.0	321,002	198,349	15.8	85,578	52,861	6.5	266,767
District of Columbia										73,125
Hawaii				56,250	28,125	2.4	135,550	67,775	3.5	224,100
Puerto Rico	224,621	110,876	11.3	131,604	64,530	8.8	65,450	32,135	2.1	112,459
TOTALS	22,241,126	11,067,261	2,144.3	23,153,181	11,386,487	1,611.5	11,440,556	5,490,861	941.8	34,160,993

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TECHNICAL BULLETINS

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MISCELLANEOUS PUBLICATIONS

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SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y. . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF FEBRUARY 28,1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	NUMBER			Estimated Total Cost	Federal Aid	NUMBER			Estimated Total Cost	Federal Aid	NUMBER			
			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 243,609	$ 243,410	6		4	$ 1,182,479	$ 1,180,624	13	1	1	$ 55,400	$ 55,400	2		2	$ 899,107
Arizona						203,898	201,694	3			27,976	27,976				526,004
Arkansas	279,639	278,482	7			415,591	415,311	8			104,053	104,053	2			1,304,654
California	669,417	669,417	2	2		1,338,720	1,338,095	8	1		1,028,262	1,027,330	4			1,394,223
Colorado	32,559	29,328	1		2	266,218	266,218	4			344,269	340,508	2		19	903,473
Connecticut						18,930	12,665									998,900
Delaware	39,000	39,000			13						47,420	47,420	2		10	504,830
Florida						445,510	445,510	3			75,900	75,900	1			1,161,858
Georgia						365,090	365,090	6			86,460	86,460	2		1	2,443,120
Idaho	174,973	174,800	4			280,682	249,386	4			131,898	120,500	1			439,963
Illinois	369,500	369,500	2		1	2,160,925	2,160,925	14	2	60	1,057,740	989,740	8		16	2,715,261
Indiana	690,166	599,203	3	5		894,563	867,663	3	1	90	164,640	164,640		1	8	1,297,142
Iowa	1,034,174	980,296	10	2	19	208,821	196,400	3		1	189,213	174,408	8		1	1,761,925
Kansas	457,727	457,622	7		4	1,035,551	1,035,551	13		1	164,005	164,005	2		8	1,422,289
Kentucky	145,000	145,000	1			344,297	344,297	4	1	31	462,882	462,882	6	3		1,218,712
Louisiana						447,201	440,458	4			429,826	398,090	10			1,049,379
Maine	48,590	48,590	2			332,396	332,396	3	2		69,630	69,630	1		1	375,778
Maryland						72,188	72,188	1			18,200	18,200			4	1,137,108
Massachusetts	54,710	54,710		1		176,639	176,028	1	1		386,385	385,425	1	3		1,705,411
Michigan	930,783	924,372	8	1	35	653,796	653,796	6	2	10	114,000	114,000				2,242,164
Minnesota	39,556	39,556		1	1	760,185	759,864	3	5		18,297	18,297			3	2,152,975
Mississippi	70,800	70,800	1			667,960	667,960	8			127,500	127,500	1	1		1,028,541
Missouri	297,091	295,552	4	1		436,130	436,130	2			703,130	703,130	3	3		2,186,625
Montana	355,586	350,704	4			634,520	634,520	6			235,773	235,773	3			364,726
Nebraska	150,374	150,374	4			729,307	729,307	15			757,668	757,668	18		33	474,750
Nevada	149,761	149,761		3	2	202,591	202,591	2		1	15,478	15,478			6	188,743
New Hampshire	70,205	69,765		1	2	87,856	87,797	5		1						433,688
New Jersey	116,891	111,665	1	1		229,856	229,856	1	1	2						2,018,350
New Mexico	168,984	168,984	4	1		118,994	118,994	3								730,470
New York	992,501	991,800	4	3	2	1,816,551	1,792,101	5	8		394,130	315,180		3	1	5,006,797
North Carolina	121,550	121,550	1	1	1	887,900	855,200	6	5		707,680	705,280	4	2	26	1,263,341
North Dakota	197,294	196,341		1		651,738	603,336	4	1							1,088,708
Ohio						478,020	478,020	6			476,990	435,990	5	1	1	4,149,011
Oklahoma						203,223	169,223	2			190,305	190,305			55	2,377,022
Oregon	308,391	307,742	1	2		384,601	250,243	1			167,455	167,455	2			484,121
Pennsylvania	213,129	197,923	2			1,039,656	827,757	2			864,536	864,536	3			4,970,935
Rhode Island						335,019	335,019	1	2		103,772	103,772		1		152,459
South Carolina	33,376	33,376		1		342,323	287,357	15	1		180,175	180,175	2		40	1,281,944
South Dakota	118,596	118,596	2		9	282,188	282,188	3	2		16,910	16,910	2		2	1,171,221
Tennessee	2,660	2,660			1	121,490	121,490		2		688,910	688,910	4		8	1,452,160
Texas	34,033	33,377	2			1,709,704	1,679,202	17	6		991,712	959,420	10	1		3,759,979
Utah	101,648	101,648	2		4	47,359	47,359	2			146,200	146,200			57	407,092
Vermont	237,610	232,253	6	2	6	7,406	7,406				23,000	23,000			7	315,293
Virginia	330,059	330,059	13	1		398,070	398,070	7	1		495,511	387,861	3	2	24	1,056,673
Washington	247,816	244,475	2	3	2	730,308	728,897	9	1		269,115	269,115	2		13	443,248
West Virginia	225,190	224,170	1		3	308,341	292,581	3			31,400	31,400	2		3	1,036,663
Wisconsin	215,236	214,831	3			1,186,812	1,145,988	11			4,917	4,917			1	1,593,847
Wyoming	164,200	164,200		2	4	10,150	10,150				214,610	135,190	1	1	7	499,324
District of Columbia						30,215	30,215		1							392,716
Hawaii						201,200	201,200	3	1		30,460	30,460	1			359,590
Puerto Rico						214,569	213,370	4			73,009	72,439	4			594,567
TOTALS	10,132,384	9,935,892	110	35	115	26,097,737	25,377,686	247	46	198	12,866,802	12,412,928	122	22	357	68,936,880

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PUBLIC ROADS

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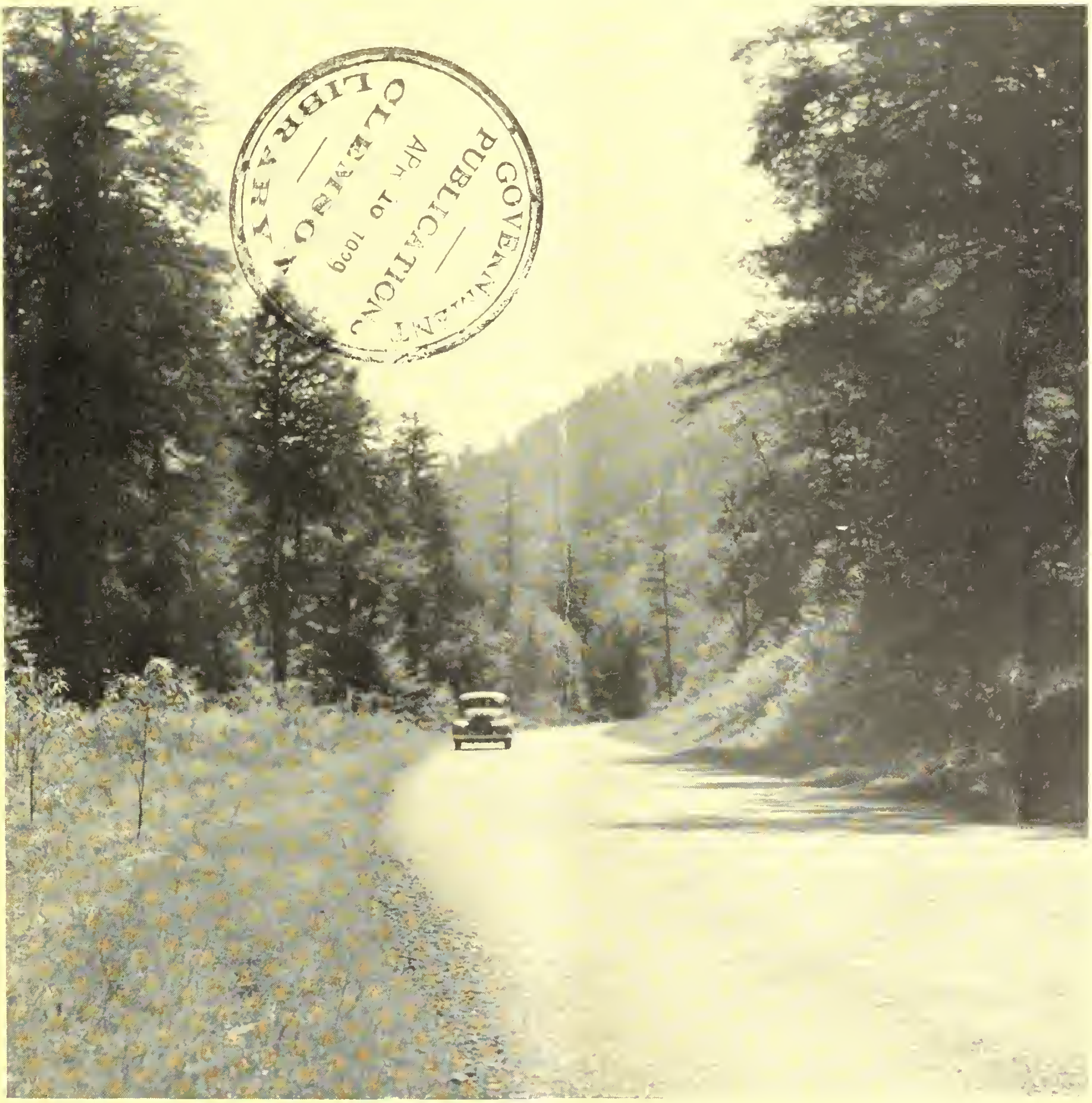
UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

VOL. 20, NO. 2 ▽ APRIL 1939



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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

D. M. BEACH, *Editor*

Volume 20, No. 2 April 1939

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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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

EXPERIMENTAL BITUMINOUS-TREATED SURFACES ON SAND-CLAY AND MARL BASES

REPORT ON EXPERIMENTS IN SOUTH CAROLINA

Reported by PAUL F. CRITZ, Associate Highway Engineer, U. S. Bureau of Public Roads, and
W. K. BECKHAM, Maintenance Engineer, South Carolina State Highway Department

A BITUMINOUS TREATED EXPERIMENTAL ROAD was built in Berkeley County, South Carolina, in 1929 by the State highway department and the Bureau of Public Roads to obtain information on sand-clay and marl as base materials and on the comparative value of various bituminous materials and methods of construction. This experimental road is still in service and a considerable portion of it is in good condition at the present time. It has been maintained with only such changes as have been brought about by the application of necessary re-treatments and maintenance that have been required to keep it in reasonably good condition at all times. The record of its construction, maintenance, and service behavior, provides an opportunity for the study of a number of factors upon which information was desired.

The experimental road, 4.48 miles long, was constructed on State route 46 and extends from the intersection with United States Highway No. 52 at Moncks Corner to the village of Pinopolis. It consists of eight different sections constructed upon a marl base and eight corresponding sections built upon a sand-clay base. Three methods of construction were employed in building the bituminous surfaces. One was the mixed-in-place or road-mix method in which the aggregates and bituminous materials were mixed together on the road with blade graders and then spread and rolled. The second was the penetration method in which the aggregate was spread and rolled and then penetrated with the bituminous material. The third method was the surface-treatment type, sometimes referred to as inverted penetration, in which the bituminous material was spread first and immediately covered with aggregate. Except on the road-mix sections, no manipulation other than that of light brooming to obtain a uniform cover of aggregate was done. Rolling completed the operations except where a seal was applied.

All of the bituminous materials used were liquid and were warmed slightly to insure uniform application. The material used for priming both the marl and sand-clay bases was a tar of 8 to 13 specific viscosity at 40° C. The materials used as binders in the mixed mats were a tar of 25 to 35 specific viscosity, Engler, at 40° C. and two asphalt cements of 60–70 and 85–100 penetration, respectively, cut-back with naphtha. These two asphaltic materials will be referred to as 60–70 or 85–100 "cut-backs" or "CB." A quick-breaking emulsion was used in the penetration sections and an 85–100 cut-back was used in the surface-treated sections. For the seal treatments, the same bituminous material used as a binder in a given section was used except in section 6A

TABLE 1.—*Analyses of subgrade and base course materials*

Identification of samples				Mechanical analysis						Characteristics of materials passing No. 40 sieve							Soil group
													Shrinkage		Moisture equivalent		
Section	Date sampled	Laboratory No.	Material represented	Particles larger than 2.0 mm.	Coarse sand, 2.0 to 0.25 mm.	Fine sand, 0.25 to 0.05 mm.	Silt, 0.05 to 0.005 mm.	Clay, smaller than 0.005 mm.	Colloids, smaller than 0.001 mm.	Per cent passing	Liquid limit	Plasticity index	Limit	Ratio	Centrifuge	Field	
				Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*								
1A and 1B.	1934	8166	Subgrade	0	33	39	13	15	17	94	13	3	12	2.0	12	12	A-2, feebly plastic.
2A	1936	10134	do	0	32	33	14	21		82	28	14	14	1.8	17	18	A-2, plastic.
2C	1934	8162	do	0	32	29	13	26	20	83	27	16	13	1.9	18	18	Do.
2C	1936	10139	do	0	9	50	22	19		96	34	18	15	1.8	19	21	A-4, very plastic.
3	1934	8163	do	0	35	49	5	11	10	89	16	0			6	19	A-3.
3	1936	10136	do	0	42	34	9	15		88	16	1	11	1.9	7	17	A-2, friable.
4	1934	8164	do	0	36	45	7	12	8	86	14	0			9	17	A-3.
2A	1936	10133	Marl base	0	12	45	21	22		95	40	9			23		A-5.
2C	1934	8161	do	0	11	44	21	24	15	95	40	17	31	1.4	29	31	A-5.
2C	1936	10138	do	0	14	48	19	19		95	41	6	31	1.4	23	42	A-5.
3	1936	10135	do	0	14	44	21	21		94	35	9	27	1.5	21	29	A-2, plastic.
3	1936	10137	do	0	14	48	19	19		95	36	5	27	1.5	19	33	A-4.
5	1936	10140	do	0	13	47	20	20		94	38	7	32	1.4	21	35	A-5.
6A [1]	During construction		Sand-clay base	1	17–24	47–53	10–11	19	14–15	93	25–26	9–12	17	1.8	14–16	19–22	A-2.
6B [2]	do		do	0–1	24–30	43–46	8–11	16–21	11–15	89–93	21–28	8–12	18	1.8	14–20	17–23	A-2.
7A [3]	do		do	0–1	21–30	46–60	7–8	11–17	7–14	92–93	17–24	0–10	17	1.8	8–14	16–20	A-2 and A-3.
7A	1934	8165	do	0	32	50	6	12	7	93	17	0			9	17	A-3.
7B [3]	During construction.		do	0–1	19–27	55–59	8	10–14	7–11	95	18–21	0–7	17–18	1.8	11–16	16–17	A-1 and A-2.
8	1934	8168	do	0	29	55	4	12	8	93	15	0			8	19	A-3.
10 [2]	During construction.		do	0	12–20	50–65	8–11	11–21	8–16	94–95	17–30	0–16	17–18	1.8	9–18	17–19	A-2 and A-3.
10	1934	8167	do	0	26	51	7	16	12	96	19	7	15	1.9	11	16	A-2 plastic.

[1] 2 samples. [2] 4 samples. [3] 3 samples.

where an 85–100 cut-back was used to seal the mat in which a 60–70 cut-back had been used as the binder. All of the asphaltic materials came from one refinery; all of the tar products came from another.

The aggregate was crushed granite graded as follows: 1¼ to ¼ inch, ¾ to ¼ inch, ⅝ to ¼ inch, and ¼ inch to dust.

ANALYSIS OF MATERIALS AND COST GIVEN FOR EACH SECTION

The location and description of the various sections comprising the experiment are shown in the charts in figures 1 and 2, which also show the character and extent of the re-treatments that have been applied since construction. Analyses of the base and subgrade materials are given in table 1. The grading of the aggregates and the analyses of the bituminous materials used are given in tables 2, 3, 4, and 5. The cost of maintaining the bituminous surfaces, including re-treatments, is given in table 6.

TABLE 2.—*Mechanical analysis of crushed stone used in construction*

Size designation	1¼ to ¼ inch	¾ to ¼ inch	⅝ to ¼ inch	¼ inch to dust
Retained on—	*Percent*	*Percent*	*Percent*	*Percent*
1¼-inch screen				
1-inch screen	16			
¾-inch screen	39			
½-inch screen	63	6	1	
¼-inch screen	88	59	71	
No. 10 sieve	100	88	96	15
No. 20 sieve		95	99	48
No. 30 sieve		100	100	59
No. 40 sieve				70
No. 50 sieve				79
No. 80 sieve				89
No. 100 sieve				92
No. 200 sieve				97

The experimental road extends west and north from the intersection with United States Highway No. 52 at Moncks Corner, where the stationing begins, but for easy reference in this report it will be considered as extending from east to west.

The route on which the improvement was made carries principally passenger cars and light trucks, but some heavier trucks with trailers carrying relatively heavy loads of logs or merchandise also use it. The average daily traffic is given in table 7. The area adjoining the experimental road is relatively flat and low with the water table close to the surface. The soil is sandy loam, suitable for agricultural purposes and forms the subgrade of the roads in this area. Unsurfaced roads in the vicinity contain the same type of soil and their service behavior depends upon the composition of the soil. The subgrade on the experimental road was composed of material of this character and, as shown in table 1, varied somewhat in composition and physical characteristics. Some natural drainage is afforded and this has been supplemented since construction by the installation of cross drains, French drains, and side ditches so that the drainage requirements have in general been fairly well met.

At the time this experimental road was constructed, the application of present-day soil analysis had not reached the stage of development that it has more recently attained. Methods of stabilizing loosely bonded, sandy soils had been developed and a considerable amount of work of this character had been done in South Carolina and elsewhere both experi-

TABLE 3.—*Composition of the cut-back asphalts used in construction*

ANALYSIS OF DISTILLATE

Specific gravity, 77°/77° F.	0.786
Initial boiling point ° F.	172
Total distillate to 200° F. percent	2
Total distillate to 300° F. do	16
Total distillate to 392° F. do	89
End point ° F.	522

ANALYSIS OF ASPHALT CEMENT BASE

	Penetration grade, 60 to 70	Penetration grade, 85 to 100
Penetration at 77° F.	67	100
Softening point ° F.	125	116
Ductility at 77° F. cm.	110+	110+
Loss at 325° F., 5 hours, 50 grams percent	.01	.02
Solubility in CS_2 do	99.88	99.88

ANALYSIS OF COMBINED DISTILLATE AND BASE

	Penetration grade, 60 to 70	Penetration grade, 85 to 100
Specific gravity at 77°/77° F.	0.947– 0.950	0.942– 0.947
Flash point ° F.	82 – 86	77 – 86
Specific viscosity, Engler, at 104° F.	59 – 71	41 – 62
Specific viscosity, Engler, at 122° F.	30 – 37	23 – 34
Loss, 325° F., 5 hours, 50 grams percent	25.4 – 28.3	24.5 – 28.2
Residue, penetration at 77° F.	84 –101	104 –119
Loss, 325° F., 5 hours, 20 grams percent	26.3 – 29.8	24.9 – 25.9
Residue, penetration at 77° F.	44 – 50	56 – 65
Solubility in CS_2 percent	99.79 – 99.91	99.79 – 99.88
Organic matter insoluble do	.18 – .08	.18 – .07
Inorganic matter insoluble do	.03 – .01	.04 – .02
Bitumen insoluble in 86° B. naphtha do	18.8 – 19.4	14.9 – 17.0
Residue of 100 penetration do	72 – 73	74 – 75
Penetration of residue at 77° F.	87 – 94	90 –104
Penetration of residue at 32° F.	14 – 23	12 – 22
Softening point of residue ° F.	115 –126	115 –119
Ductility of residue at 77° F. cm.	110+	110+
Ductility of residue at 38° F. do	4.0 – 5.2	4.9 – 6.4
Distillation by volume (A. S. T. M. D402-36):		
Total distillate to 374° F. percent	6.0 – 9.8	0.8 – 11.5
Total distillate to 437° F. do	19.0 – 25.0	20.0 – 22.0
Total distillate to 680° F. do	30.6 – 37.4	29.7 – 33.6
Residue, penetration at 77° F.	67 – 75	78 – 98
Residue, softening point ° F.	119 –122	110 –118

TABLE 4.—*Analysis of tars used in construction*

	Grade of material	
	8-13 viscosity [1]	25-35 viscosity [2]
Specific gravity, 77°/77° F.	1.142– 1.148	1.165
Specific viscosity, Engler, at 104° F.	11.1 – 11.7	27.5 – 28.5
Solubility in CS_2 percent	96.15 – 96.93	95.92 – 96.98
Organic matter insoluble do	2.65 – 3.81	2.04 – 3.61
Inorganic matter insoluble do	.02 – .04	.02 – .07
Water do	.04 – —	.40 – .96
Distillation by weight, water-free basis:		
To 338° F. percent	1.05 – 1.25	.53 – 1.07
338°–455° F. do	6.67 – 6.93	4.71 – 5.29
455°–518° F. do	11.57 – 11.89	8.89 – 10.26
518°–572° F. do	8.40 – 8.45	7.89 – 8.03
Residue do	71.49 – 72.10	76.07 – 77.26
Softening point of residue ° F.	109 –110	106 –107

[1] Used as a prime on all sections.
[2] Used on secs. 2B and 7B in the mixture and in the seal coat.

TABLE 5.—*Analysis of asphalt emulsion used in the construction of sections 2C and 7C*

Specific gravity, 77°/77° F.	1.009
Specific viscosity, Engler, at 122° F.	1.98
Distillation to 500° F., by weight:	
Water percent	47.3
Oil do	Trace
Residue do	52.6
Tests on residue from distillation:	
Specific gravity, 77°/77° F.	1.015
Penetration at 77° F.	113
Softening point ° F.	118
Ductility at 77° F. cm.	96.5
Solubility in CS_2 percent	98.94
Organic matter insoluble do	.40
Ash (by ignition) do	.66

TABLE 6.—*Cost of construction, re-treatments, and maintenance of the various experimental sections to July 1, 1937*

Section			Construction						Annual cost per square yard of re-treatments and maintenance of bituminous surfaces only																	
No.	Station		Type of base	Mat			Seal	Cost per square yard	1929-30		1930-31		1931-32		1932-33		1933-34		1934-35		1935-36		1936-37		Total	Average annual cost
	From	To		Bituminous material	Size of aggregate	Method of construction	Bituminous material [1]		Maintenance	Re-treatment	Maintenance	Re-treatment	Maintenance	Re-treatment	Maintenance	Re-treatment	Maintenance	Re-treatment	Maintenance	Re-treatment	Maintenance	Re-treatment	Maintenance	Re-treatment		
								Cts.	*Cts.*	*Cts.*	*Cts.*	*Cts.*	*Cts.*	*Cts.*	*Cts.*	*Cts.*	*Cts.*	*Cts.*	*Cts.*	*Cts.*	*Cts.*	*Cts.*	*Cts.*	*Cts.*	*Cts.*	*Cts.*
1-A	0+00	13+28	Marl	Asphalt, cut-back	1¼ inch to dust	Road-mix	Asphalt, cut-back	66.12	0.48		1.56		1.40	10.55	0.43		0.31		0.14		0.23	9.53	1.12		25.75	3.32
1-B	13+28	24+50	do	do	do	do	do	72.58	1.37		7.75	[3] 5.73	2.57	[3] 7.95	.60	[3] 1.76	.84		.29			10.03	.81	[3] 2.59	42.29	5.46
2-A	[2] 24+50	39+60	do	do	1¼-¼ inch	do	do	70.34	.18		3.86		5.46	7.82	.65	[3] 2.20	.63		.52		1.91	10.28	1.33	13.89	48.73	6.29
2-B	39+60	52+00	do	Tar	do	do	Tar	79.40	1.36		3.80	[3] 5.50	8.87	[3] 11.23	.96	[3] 3.33	.62		.82			15.25	.86	[3] 2.82	55.42	7.15
2-C	52+00	66+00	do	Asphalt emulsion	1¼ inch to dust	Penetration	Asphalt emulsion	84.62	2.61		8.95	[3] 13.28	12.43	[3] 21.45	2.57	[3] 4.09	.89	[3] 5.98	.07		2.73	[3] 6.49	2.71	19.61	103.86	13.40
3	79+20	92+40	do	Asphalt, cut-back	¾ inch to dust	Road-mix		61.07	.39		.08		.17		.87						1.36		.18	14.34	17.39	2.24
4	92+40	105+84	do	do	¾-¼ inch	do	Asphalt, cut-back	55.46	1.16		1.37		1.11	[3] 4.52	.57		1.20		1.26		1.06	10.55	.89		23.69	3.06
5	66+00	79+20	do	do	1¼-¼ inch	Surface treatment.	do	28.47	3.60	([4] [5])	4.06		1.97	[3] 4.51	2.29		.03	[3] 7.38	.72		.74		.13	14.34	39.77	5.13
6-A	105+84	119+99	Sand-clay.	do	1¼ inch to dust	Road-mix	do	72.53	.48		.05		.59		.65			12.20	.14						14.11	1.82
6-B	119+99	132+00	do	do	do	do	do	72.10	.62		1.68		2.93	9.51	.38										15.12	1.95
7-A	132+00	144+20	do	do	1¼-¼ inch	do	do	62.86	1.24		2.00		3.85	12.65	.20								.35		20.29	2.62
7-B	199+70	212+90	do	Tar	do	do	Tar	73.23	1.18	([4] [5])	1.45		1.77		.93						2.90		.70	17.98	26.91	3.47
7-C	144+20	158+40	do	Asphalt emulsion	1¼ inch to dust	Penetration	Asphalt emulsion	84.55	1.02		.54		3.45	18.10	.33						.90		.48	19.37	44.19	5.70
8	212+90	226+10	do	Asphalt, cut-back	¾ inch to dust	Road-mix		59.53	.37		.05		.80		.04										1.26	.16
9	226+10	238+80	do	do	¾-¼ inch	do	Asphalt, cut-back	66.74	.56		.42		1.61		.30						.03				2.92	.38
10	[6] 158+40	199+70	do	do	1¼-¼ inch	Surface treatment.		29.44	1.06	([5])	1.83		.40	12.50	.28		.04				.03				16.14	2.08

[1] Same type of material as used in the mixed mat, penetration or surface treatment course.
[2] Deduct 36 linear feet for railroad crossing.
[3] Treatment applied to part of section only but cost is prorated over entire section.
[4] Applied to part of section only.
[5] Cost included in construction cost.
[6] Deduct 170 linear feet for station correction.

EXPERIMENT 1 - SECTION A

STA 0+00 — CONSTRUCTION — 13+28

BASE	MARL, PRIMED WITH 0.27 GAL 8 TO 13 VISCOSITY TAR
MIXED MAT	1.8 INCHES USING 0.91 GAL 60-70 CUTBACK; 154 LB STONE, 1¼" TO ¼"; 38 LB SCREENINGS, ¼" TO DUST
SURFACE TREATMENT	0.42 GAL 60-70 CUTBACK, 30 LB STONE CHIPS, ⅝" TO ¼"

RE-TREATMENTS

NOV 1929	
JUNE 1931	
NOV 1931	0.37 GAL 60-70 CUTBACK; 30 LB. STONE, ¾" TO ¼"
DEC 1931	
SEPT 1932	
SEPT 1933	
SEPT 1935	0.45 GAL 60-70 CUTBACK; 40 LB. STONE, ½" TO ¼"
OCT 1936	

EXPERIMENT 1 - SECTION B

STA 13+28 — CONSTRUCTION — 24+50

BASE	MARL, PRIMED WITH 0.27 GAL 8 TO 13 VISCOSITY TAR
MIXED MAT	1.8 INCHES USING 1.03 GAL 85-100 CUTBACK; 156 LB STONE, 1¼" TO ¼"; 37 LB SCREENINGS, ¼" TO DUST
SURFACE TREATMENT	0.29 GAL. 85-100 CUTBACK; 15 LB STONE CHIPS, ⅝" TO ¼"

RE-TREATMENTS

NOV 1929	
JUNE 1931	0.33 GAL 85-100 CUTBACK, 33 LB STONE ⅝" TO ⅛"; 0.68 GAL 85-100 CUTBACK; 52 LB. STONE ⅝" TO ⅛"
NOV 1931	0.35 GAL 85-100 CUTBACK; 32 LB STONE, ¾" TO ¼" (ALSO ON A) — A
DEC. 1931	
SEPT. 1932	BASE AND OLD MAT REMIXED; PRIMED WITH 0.35 GAL TAR; SURFACE TREATED WITH 0.53 GAL 85-100 CUTBACK; 67 LB STONE, ¾" TO ¼"
SEPT 1933	
SEPT 1935	0.47 GAL. 85-100 CUTBACK; 40 LB. STONE, ½" TO ¼"
OCT 1936	0.58 GAL 85-100 CUTBACK, 25 LB STONE, ⅝" TO ¼"

EXPERIMENT 2 - SECTION A

STA 24+50 (1) — CONSTRUCTION — 39+60

BASE	MARL, PRIMED WITH 0.31 GAL 8 TO 13 VISCOSITY TAR
MIXED MAT	2.0 INCHES USING 0.69 GAL 85-100 CUTBACK, 180 LB STONE, 1¼" TO ¼"
SURFACE TREATMENT	0.30 GAL. 85-100 CUTBACK; 18 LB. STONE CHIPS, ⅝" TO ¼"

RE-TREATMENTS

NOV. 1929	
JUNE 1931	
NOV 1931	0.30 GAL 85-100 CUTBACK, 30 LB. STONE, ¾" TO ¼"
DEC. 1931	
SEPT. 1932	BASE AND OLD MAT REMIXED, PRIMED WITH 0.23 GAL TAR; SURFACE TREATED WITH 0.67 GAL 85-100 CUTBACK; 61 LB STONE, ¾" TO ¼"
SEPT 1933	
SEPT 1935	0.46 GAL. 85-100 CUTBACK; 40 LB. STONE, ½" TO ¼"
OCT 1936	0.60 GAL 85-100 CUTBACK; 25 LB STONE, ⅝" TO ¼"; 25 LB SAND

(1) DEDUCT 36 FEET FOR RR CROSSING

EXPERIMENT 2 - SECTION B

STA. 39+60 — CONSTRUCTION — 52+00

BASE	MARL, PRIMED WITH 0.29 GAL. 8 TO 13 VISCOSITY TAR
MIXED MAT	2.0 INCHES USING 0.89 GAL. 25 TO 35 VISCOSITY TAR; 175 LB. STONE, 1¼" TO ¼"
SURFACE TREATMENT	0.23 GAL 25 TO 35 VISCOSITY TAR; 15 LB. STONE CHIPS, ⅝" TO ¼"

RE-TREATMENTS

NOV. 1929	
JUNE 1931	0.34 GAL. 25-35 TAR; 33 LB STONE, ⅝" TO ⅛"
NOV 1931	
DEC. 1931	0.37 GAL. 25-35 TAR; 32 LB. STONE, ¾" TO ¼"
SEPT 1932	BASE AND OLD MAT REMIXED; PRIMED WITH 0.29 GAL TAR; SURFACE TREATED WITH 0.66 GAL. 85-100 CUTBACK; 74 LB STONE, ¾" TO ¼"
SEPT 1933	
SEPT 1935	0.44 GAL. 25-35 VISCOSITY TAR; 40 LB. STONE, ½" TO ¼"
OCT 1936	0.60 GAL. 25-35 VISCOSITY TAR; 25 LB. STONE, ⅝" TO ¼"; 25 LB SAND

EXPERIMENT 2 - SECTION C

STA. 52+00 — CONSTRUCTION — 66+00

BASE	MARL, PRIMED WITH 0.28 GAL 8 TO 13 VISCOSITY TAR
PENETRATION MAT	2.3 INCHES USING 37 LB SCREENINGS, ¼" TO DUST, 165 LB STONE 1¼" TO ¼"; 0.44 GAL EMULSION; 8 LB STONE CHIPS, ⅝" TO ¼"; 0.83 GAL EMULSION; 7 LB. STONE CHIPS, ⅝" TO ¼"
SURFACE TREATMENT	0.23 GAL EMULSION; 15 LB STONE CHIPS, ⅝" TO ¼"

RE-TREATMENTS

NOV 1929	REMIXED WITH 0.81 GAL. EMULSION; SEALED WITH 0.45 GAL. EMULSION AND 35 LB STONE, ⅝" TO ⅛"
JUNE 1931	0.45 GAL. EMULSION; 35 LB STONE, ⅝" TO ⅛"
NOV 1931	0.80 GAL EMULSION; 16 LB STONE, 1¼" TO ¼" AND 19 LB STONE, ¾" TO ¼"
DEC 1931	0.70 GAL. EMULSION; 10 LB STONE, 1¼" TO ¼" AND 15 LB STONE, ¾" TO ¼"
SEPT 1932	BASE AND OLD MAT REMIXED PRIMED WITH 0.75 GAL TAR; NEW MAT 1.93 GAL EMULSION; 138 LB. STONE, ¾" TO ¼"
SEPT 1933	0.62 GAL EMULSION AND 53 LB STONE, ⅝" TO ¼"
SEPT 1935	0.43 GAL EMULSION AND 31 LB STONE, ½" TO ¼"
OCT 1936	0.62 GAL EMULSION AND STONE, ⅝" TO ¼" 25 LB / 40 LB / 25 LB.

EXPERIMENT 3

STA. 79+20 — CONSTRUCTION — 92+40

BASE	MARL, PRIMED WITH 0.30 GAL. 8 TO 13 VISCOSITY TAR
MIXED MAT	1.8 INCHES USING 1.19 GAL 85-100 CUTBACK, 162 LB STONE, ¾" TO ¼"; 38 LB SCREENINGS, ¼" TO DUST
SURFACE TREATMENT	NONE

RE-TREATMENTS

NOV. 1929	
JUNE 1931	
NOV 1931	
DEC. 1931	
SEPT 1932	
SEPT 1933	
SEPT. 1935	
OCT 1936	0.58 GAL 85-100 CUTBACK; 25 LB. STONE, ⅝" TO ¼"; 25 LB. SAND

EXPERIMENT-4

STA. 92+40 — CONSTRUCTION — 105+84

BASE	MARL, PRIMED WITH 0.26 GAL. 8 TO 13 VISCOSITY TAR
MIXED MAT	1.8 INCHES USING 0.72 GAL. 85-100 CUTBACK; 157 LB. STONE, ¾" TO ¼"
SURFACE TREATMENT	0.22 GAL 85-100 CUTBACK; 15 LB. STONE CHIPS, ⅝" TO ¼"

RE-TREATMENTS

NOV 1929	
JUNE 1931	
NOV 1931	0.39 GAL 85-100 CUTBACK; 31 LB STONE, ¾" TO ¼"
DEC. 1931	
SEPT 1932	
SEPT 1933	
SEPT 1935	0.45 GAL 85-100 CUTBACK; 40 LB STONE, ½" TO ¼"
OCT 1936	

EXPERIMENT 5

STA 66+00 — CONSTRUCTION — 79+20

BASE	MARL, PRIMED WITH 0.33 GAL. 8 TO 13 VISCOSITY TAR
MIXED MAT	NONE
SURFACE TREATMENT	0.44 GAL. 85-100 CUTBACK; 50 LB. STONE, 1¼" TO ¼"

RE-TREATMENTS

NOV. 1929	0.36 GAL 85-100 CUTBACK; 15 LB STONE, ⅝" TO ¼"
JUNE 1931	
NOV. 1931	0.29 GAL. 85-100 CUTBACK; 21 LB. STONE, ¾" TO ¼"
DEC. 1931	
SEPT. 1932	
SEPT. 1933	0.46 GAL. 85-100 CUTBACK; 39 LB. STONE, ⅝" TO ¼"
SEPT. 1935	
OCT 1936.	0.56 GAL. 85-100 CUTBACK; 25 LB. STONE, ⅝" TO ¼"; 25 LB. SAND

FIGURE 1.—CHARACTER OF CONSTRUCTION AND RE-TREATMENTS APPLIED TO EXPERIMENTS 1 TO 5, INCLUSIVE. QUANTITIES SHOWN ARE AMOUNTS PER SQUARE YARD OF SURFACE AREA.

EXPERIMENT 6 - SECTION A

STA. 105+84 — CONSTRUCTION — 119+99

BASE	SAND-CLAY, PRIMED WITH 0.33 GAL. 8 TO 13 VISCOSITY TAR
MIXED MAT	2.0 INCHES USING 1.0 GAL. 60-70 CUTBACK; 171 LB. STONE, 1 1/4" TO 1/4"; 40 LB. SCREENINGS, 1/4" TO DUST
SURFACE TREATMENT	0.34 GAL. 85-100 CUTBACK; 15 LB. STONE CHIPS, 5/8" TO 1/4"

RE-TREATMENTS

NOV. 1929	
JUNE 1931	
NOV. 1931	
DEC. 1931	
SEPT. 1932	
SEPT. 1933	0.42 GAL. 60-70 CUTBACK; 40 LB. STONE, 5/8" TO 1/4"
SEPT. 1935	
OCT. 1936	

EXPERIMENT 6 - SECTION B

STA. 119+99 — CONSTRUCTION — 132+00

BASE	SAND-CLAY, PRIMED WITH 0.39 GAL. 8 TO 13 VISCOSITY TAR
MIXED MAT	2.0 INCHES USING 1.09 GAL. 85-100 CUTBACK; 170 LB. STONE, 1 1/4" TO 1/4"; 42 LB. SCREENINGS, 1/4" TO DUST
SURFACE TREATMENT	0.23 GAL. 85-100 CUTBACK; 15 LB. STONE CHIPS, 5/8" TO 1/4"

RE-TREATMENTS

NOV. 1929	
JUNE 1931	
NOV. 1931	0.41 GAL. 85-100 CUTBACK; 21 LB. STONE, 3/4" TO 1/4"
DEC. 1931	
SEPT. 1932	
SEPT. 1933	
SEPT. 1935	
OCT. 1936	

EXPERIMENT 7 - SECTION A

STA. 132+00 — CONSTRUCTION — 144+20

BASE	SAND-CLAY, PRIMED WITH 0.29 GAL. 8 TO 13 VISCOSITY TAR
MIXED MAT	2.0 INCHES USING 0.77 GAL. 85-100 CUTBACK; 178 LB. STONE, 1 1/4" TO 1/4"
SURFACE TREATMENT	0.27 GAL. 85-100 CUTBACK; 18 LB. STONE CHIPS, 5/8" TO 1/4"

RE-TREATMENTS

NOV. 1929	
JUNE 1931	
NOV. 1931	0.35 GAL. 85-100 CUTBACK; 35 LB. STONE, 3/4" TO 1/4"
DEC. 1931	
SEPT. 1932	
SEPT. 1933	
SEPT. 1935	
OCT. 1936	

EXPERIMENT 7 - SECTION B

STA. 199+70 — CONSTRUCTION — 212+90

BASE	SAND-CLAY, PRIMED WITH 0.25 GAL. 8 TO 13 VISCOSITY TAR
MIXED MAT	2.0 INCHES USING 0.77 GAL. 25 TO 35 VISCOSITY TAR; 169 LB. STONE, 1 1/4" TO 1/4"
SURFACE TREATMENT	← REMIXED WITH 0.30 GAL. 25 TO 35 VISCOSITY TAR → ← 0.42 GAL. 25 TO 35 VISCOSITY TAR; 15 LB. STONE CHIPS, 5/8" TO 1/4" →

RE-TREATMENTS

NOV. 1929	0.23 GAL. 25 TO 35 VISCOSITY TAR; 15 LB. STONE CHIPS, 5/8" TO 1/4"
JUNE 1931	
NOV. 1931	
DEC. 1931	
SEPT. 1932	
SEPT. 1933	
SEPT. 1935	
OCT. 1936	0.60 GAL. 25 TO 35 VISCOSITY TAR; 25 LB. STONE, 5/8" TO 1/4"; 25 LB. SAND

EXPERIMENT 7 - SECTION C

STA. 144+20 — CONSTRUCTION — 158+40

BASE	SAND-CLAY, PRIMED WITH 0.34 GAL. 8 TO 13 VISCOSITY TAR
PENETRATION MAT	2.3 INCHES USING 1.11 GAL. EMULSION; 30 LB. SCREENINGS, 1/4" TO DUST; 165 LB. STONE, 1 1/4" TO 1/4"
SURFACE TREATMENT	0.25 GAL. EMULSION; 15 LB. STONE CHIPS, 5/8" TO 1/4"

RE-TREATMENTS

NOV. 1929	
JUNE 1931	
NOV. 1931	0.63 GAL. EMULSION; 37 LB. STONE, 3/4" TO 1/4"
DEC. 1931	
SEPT. 1932	
SEPT. 1933	
SEPT. 1935	
OCT. 1936	0.89 GAL. EMULSION; 40 LB. STONE, 5/8" TO 1/4"; 27 LB. SAND

EXPERIMENT 8

STA. 212+90 — CONSTRUCTION — 226+10

BASE	SAND-CLAY, PRIMED WITH 0.26 GAL. 8 TO 13 VISCOSITY TAR
MIXED MAT	1.8 INCHES USING 1.25 GAL. 85-100 CUTBACK; 154 LB. STONE, 3/4" TO 1/4"; 38 LB. SCREENINGS, 1/4" TO DUST
SURFACE TREATMENT	NONE

RE-TREATMENTS

NOV. 1929	
JUNE 1931	
NOV. 1931	
DEC. 1931	
SEPT. 1932	
SEPT. 1933	
SEPT. 1935	
OCT. 1936	

EXPERIMENT 9

STA. 226+10 — CONSTRUCTION — 238+80

BASE	SAND-CLAY, PRIMED WITH 0.27 GAL. 8 TO 13 VISCOSITY TAR
MIXED MAT	1.8 INCHES USING 0.77 GAL. 85-100 CUTBACK; 152 LB. STONE, 3/4" TO 1/4"
SURFACE TREATMENT	0.29 GAL. 85-100 CUTBACK; 14 LB. STONE CHIPS, 5/8" TO 1/4"

RE-TREATMENTS

NOV. 1929	
JUNE 1931	
NOV. 1931	
DEC. 1931	
SEPT. 1932	
SEPT. 1933	
SEPT. 1935	
OCT. 1936	

EXPERIMENT 10

[1] STA. 158+40 — CONSTRUCTION — 199+70

BASE	SAND-CLAY, PRIMED WITH 0.29 GAL. 8 TO 13 VISCOSITY TAR
MIXED MAT	NONE
SURFACE TREATMENT	0.46 GAL. 85-100 CUTBACK; 52 LB. STONE, 1 1/4" TO 1/4"

RE-TREATMENTS

NOV. 1929	0.32 GAL. 85-100 CUTBACK; APPROXIMATELY 25 LB. LOOSE SURFACE STONE AND 10 TO 13 LB. STONE, 3/4" TO 1/4"
JUNE 1931	
NOV. 1931	0.50 GAL. 85-100 CUTBACK; 46 LB. STONE, 3/4" TO 1/4"
DEC. 1931	
SEPT. 1932	
SEPT. 1933	
SEPT. 1935	
OCT. 1936	

[1]-DEDUCT 170 FEET FOR STATION EQUATION

FIGURE 2.—CHARACTER OF CONSTRUCTION AND RE-TREATMENTS APPLIED TO EXPERIMENTS 6 TO 10, INCLUSIVE. QUANTITIES SHOWN ARE AMOUNTS PER SQUARE YARD OF SURFACE AREA.

TABLE 7.—*Average traffic on the experimental road, between 7 a. m. to 7 p. m.*

	Recording station	
	No. 1[1]	No. 2[2]
	Number	*Number*
Upon completion of project	587	320
1930 (average of 9 counts)	474	276
1931 (average of 12 counts)	537	325
1932 (average of 11 counts)	478	272
1933 (average of 12 counts)	572	320
1934 (average of 12 counts)	751	444
1935 (average of 12 counts)	834	425
1936 (average of 12 counts)	955	486
1937 (first 6 months, 6 counts)	898	480

[1] Between Moncks Corner business section and United States Highway No. 52 (formerly No. 17).
[2] Between Moncks Corner business section and Pinopolis.

mentally[1] and as routine construction. Results obtained in the use of sand-clay as a road-building material varied considerably. Because of its uncertain behavior, there were differences of opinion regarding the properties that a sand-clay should possess to be satisfactory as a base material for bituminous surfaces. These opinions were usually based upon the engineer's experience with the materials found in his particular locality. It was recognized that more definite information regarding the characteristics of sand-clay materials was essential if such a widespread and plentiful material were to be utilized to the greatest possible extent in building satisfactory low-cost roads.

MARL AND SAND-CLAY OBTAINED FROM LOCAL DEPOSITS

It was generally agreed that both the sand and the clay played important parts but it was not known what percentages of each would be most suitable or to what extent the characteristics of the component parts affected the behavior of the combination. While the methods of soil analysis now in general use had been developed at the time this experimental road was built, data sufficient for correlating laboratory tests with service behavior had not yet been obtained. Consequently, the only practical method for determining the suitability of a given material was by a service test. The same situation existed with respect to the marl.

The apparently successful results that had been obtained with limerock in base construction in Florida and Georgia led to the assumption that a somewhat similar material available in South Carolina might prove satisfactory. This material, known locally as marl, and used on 2 miles of the experimental road, was taken from a nearby deposit. Laboratory determination of the properties, which at that time were deemed the most significant, showed it to have the following characteristics:

Calcium carbonate, 84 to 87 percent.
Silica, alumina, and iron oxides, 10 to 14½ percent.
Magnesium carbonate, 1.3 to 1.7 percent.
Cementing values, 133 to 500 (plus).

The marl, when taken from the pit, was grayish white and contained some moisture. While in this condition it could be readily broken down with disks, harrows, and blade graders into a fine-grained homogeneous mass. It compacted uniformly without laminations, but developed small shrinkage cracks while drying. When dry, the surface became white, hard, and had an objectionable glare in the sunlight.

[1] Experimental Bituminous Treatment of Sandy-Soil Roads, by Paul F. Critz and H. L. Sligh. PUBLIC ROADS, vol. 17, No. 11, January 1937.

The marl base, which was 8 to 10 inches thick, was built by contract in the fall of 1928 and served as a wearing surface for traffic until August 1929. During this period its main disadvantages were its dazzling whiteness, a slight tendency to dust under steel-tired vehicles, and its tendency to soften in continued rainy weather. However, at the time the bituminous-treated surfaces were applied the base appeared to be in excellent condition.

The sand clay used in the base on 2.48 miles of the experiment was taken from a local pit and, although the best available, was not considered a good quality material largely because of its lack of uniformity and of binder. The base was built 6 to 7 inches thick and was constructed by State forces shortly before the bituminous treated surfaces were applied. Considerable work was done in manipulating the material on the road after each rain in an attempt to obtain consolidation and some degree of uniformity. Some bonding and consolidation were obtained but, at the time the experimental sections were built, the base was in only fair condition.

The bituminous surfaces were built by State forces during August, September, and October 1929. The three methods of construction used are indicated in figures 1 and 2. The details of constructing the various sections of the experiment were reported in PUBLIC ROADS in November 1931, but for convenience are briefly stated in this report, the main purpose of which is to present the data and information that have been accumulated from the time of construction to June 30, 1937, together with such discussions as appear warranted.

MARL BASE BLADED AND PRIMED BEFORE CONSTRUCTING SURFACES

In maintaining the sections since their construction an effort has been made to keep them in a uniformly satisfactory condition at all times. Maintenance has consisted of patching as needed and the application of re-treatments on sections in whole or in part as required. In applying the re-treatments, the same types of bituminous material were used that had been used in the original construction. The aggregate was one-size granite that varied slightly in maximum size as shown in figures 1 and 2.

Prior to applying the tar prime, the marl base was brought to a uniform cross section by blading after sprinkling with water. The tar prime was then applied and allowed to penetrate and dry before the bituminous treated surfaces were constructed. During this drying period it was observed that the tar penetrated readily and the base hardened in areas unshaded from the sun but that where the surface was shaded, the rate and extent of penetration were less and the surface hardened more slowly.

Experiment 1, section A, stations 0+00 to 13+28.—The method of construction and the amounts of material per square yard used on this section were as follows:

Prime: 0.27 gallon of tar.
Mix: 1.8 inches thick when compacted; 154 pounds of 1¼- to ¼-inch crushed stone, 38 pounds of ¼ inch to dust, and 0.91 gallon of 60–70 cut-back.
Seal: 0.42 gallon of the same bituminous material and 30 pounds of ⅝- to ¼-inch stone chips.

It was expected that the combined fine and coarse aggregate would produce a dense mat that would not require a seal. However, the resulting mat had a coarse texture and an open surface. During a month under traffic, some raveling occurred, and it was deemed advisable to seal the surface.

Prior to applying the seal on this section, as well as on several others where the mat was very open, the surface was choked with ⅝- to ¼-inch stone and subjected to traffic for 2 days before the bituminous material and cover materials were applied.

Considering the amount of traffic carried, this section has been one of the most satisfactory of those having a marl base. During most of its life the section appeared lean, dry, and somewhat porous. The small amount of raveling that occurred was confined mostly to the edges. Cracking of the mat was characteristic of the section, and most of the maintenance required was to seal these cracks. Little trouble that could be attributed to unsatisfactory base conditions was experienced. Test holes dug through the mat and base in 1934 showed the marl base to be dry, and the mat to be well bound by bitumen below the surface, although on the surface it appeared dry and lean. French drains, installed shortly after contructing the experiment, and deep side ditches apparently furnished adequate drainage.

Only two re-treatments were applied to this section, one in 1931 and one in 1935. Both were applied primarily to eliminate the dry, lean appearance of the surface, to seal cracks, and to eliminate the nonuniformity gradually resulting from the placing of numerous skin patches used in sealing the cracks.

When inspected in October 1936, the section was in very good condition. The edges were unbroken and showed no considerable tendency to ravel although they were not well supported by shoulder material. The surface had a rather mottled appearance but it was smooth, very dense, and had a nonskid, coarse-grained texture as shown in figure 3–A. The general appearance of the section at the time of inspection is illustrated by figure 3–B, a view taken near the west end of the section.

The cost of constructing this section was 66.12 cents per square yard and the average annual cost of maintaining the surface, including base repair and the two re-treatments, has been 3.32 cents per square yard.

Experiment 1, section B, stations 13+28 to 24+50.—The method of construction and the amounts of material per square yard used on this section were as follows:

Prime: 0.27 gallon of tar.
Mix: 1.8 inches thick when compacted; 156 pounds of 1¼- to ¼-inch crushed stone, 37 pounds of ¼ inch to dust, and 1.03 gallons of 85–100 cut-back.
Seal: 0.29 gallon of the same bituminous material was applied and covered with 15 pounds of ⅝- to ¼-inch stone chips.

This was the first section constructed and was necessarily quite experimental in character as the original plan of operations did not specify many of the details of construction.

UNSATISFACTORY BEHAVIOR ATTRIBUTED TO POOR DRAINAGE

To prevent segregation of the aggregate, the coarse material was first spread and given an application of bituminous material. The finer material was then spread and the remainder of the bituminous material applied. Mixing was begun immediately with a four-way drag, somewhat lighter and smaller than those commonly used at the present time. The results obtained were unsatisfactory, so blade graders were substituted and no further difficulty was experienced. No segregation occurred that could be attributed to the grading of the aggregate.

Mixing on this section did not proceed rapidly and the partly mixed materials laid in a windrow over Sunday. During this interval some stiffening of the cut-back asphalt occurred with the result that some segregation took place during spreading of the mixture so that the center 7 or 8 feet presented a more open appearance than the remainder of the section. As with section A of this experiment, it had not been expected that this section would require a seal. However, after about 3 weeks under traffic, the larger stone began to ravel in numerous places and a seal coat was applied.

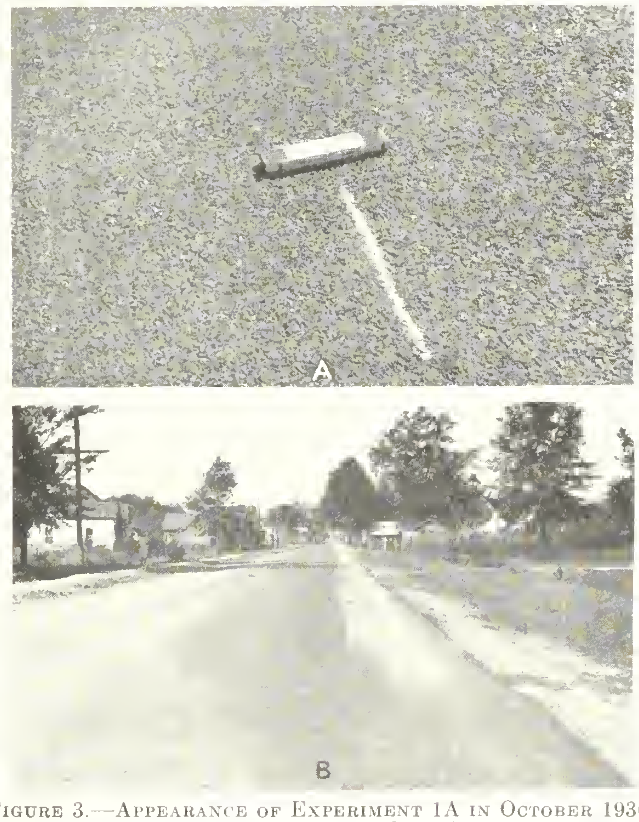

FIGURE 3.—APPEARANCE OF EXPERIMENT 1A IN OCTOBER 1936. A, CLOSE-UP VIEW SHOWING NON-SKID SURFACE TEXTURE; B, GENERAL APPEARANCE OF SECTION. NOTE EVEN EDGES, DESPITE LACK OF PROTECTION BY SHOULDERS.

Although nearly identical with experiment 1A in construction and in composition, except for the penetration of the base asphalt in the cut-back, section B was decidedly less satisfactory in service. In addition to the routine maintenance required to seal cracks and to prevent raveling, considerable maintenance was required near the west end where very unfavorable drainage conditions existed. At this location the right-of-way is little wider than the treated surface, with store buildings at the right-of-way line. Natural drainage is poor and open drainage is not practical. French drains were installed but did not aid materially in lowering the water table. Attempts to stabilize the marl base by scarifying the base and surface mats and mixing them together, and also by adding coarse aggregate, were of only temporary benefit.

Figure 1 shows that the section, excepting the west portion, has received only two re-treatments, and the west portion has been re-treated four times and virtually reconstructed twice. When inspected in October 1936, the section, with the exception of the west 200 feet, closely resembled section A in appearance and condition. The west 200 feet were badly cracked as a result of the moisture conditions already described. This part of the section was re-treated later in October. By March 1937, cracks had reappeared on this area and by July the west 50 feet were badly cracked and the marl base was exposed in places.

The cost of constructing section B was 72.58 cents per square yard and the average annual cost of maintenance including base modification and re-treatments has been 5.46 cents per square yard.

Experiment 2, section A, stations 24+50 to 39+60.[2]—The method of construction and the amounts of material per square yard used on this section were as follows:

- Prime: 0.31 gallon of tar.
- Mix: 2 inches thick when compacted; 180 pounds of 1¼- to ¼-inch stone and 0.69 gallon of 85–100 cut-back.
- Seal: An average of 0.30 gallon of the same bituminous material was applied and covered with an average of 18 pounds of ⅝- to ¼-inch stone chips. This section was sealed in two separate portions.

MOISTURE PENETRATED BITUMINOUS MAT DESPITE SEAL COAT

It was expected that the coarse-graded aggregate used in this section would produce an open mat and that a seal would be necessary. The seal was applied after the mat had been subjected to traffic for 3 weeks. On the east half of the section the seal was constructed by first applying the bituminous material and then spreading the cover stone. On the west half, a part of the stone was spread first and keyed into the surface by traffic, after which the seal was completed as on the east half.

This section extends through the business portion of Moncks Corner for a distance of approximately 1,000 feet and consequently is subjected to more severe usage than any other section of the experimental road.

French drains were installed shortly after construction of the surface and in the spring test holes were dug to determine the moisture conditions. It was observed that the marl base was damp near the top, but was dry lower down, indicating that the moisture had percolated through the surface in spite of the seal coat.

Maintenance the first 2 years consisted mainly of skin patching to seal cracks and to prevent raveling resulting from surface wear, but by the fall of 1931 the entire section had cracked so badly as to warrant a re-treatment which was applied in November. This re-treatment apparently left the section in good condition.

Later, however, a 100-foot section near the west end began to fail. The base became soft and spongy in spite of the French drains that had been installed. The bituminous mat cracked badly and patches placed to seal the cracks and prevent disintegration of the mat repeatedly failed until it became necessary practically to reconstruct this area. In August 1932, the marl base and bituminous mat were scarified, mixed together, and relaid for compaction under traffic. In September this base was primed and a surface treatment applied.

[2] The portion between stations 25+25 and 25+61 is occupied by railway tracks and was not a part of the experimental section.

Following this repair only a moderate amount of routine maintenance was required, primarily to seal cracks that developed on the end and central portions and that became more pronounced following freezing and thawing weather in February 1934. By the fall of 1935 the entire section, excepting about 300 feet in the center, had been skin-patched and had recracked in so many areas that a re-treatment was needed to seal all cracks, enliven the mat, eliminate worn areas, and provide a uniform appearance.

FIGURE 4.—SURFACE FAILURE ON EXPERIMENT 2A, PHOTOGRAPHED IN OCTOBER 1936.

The re-treatment was applied to the entire section in September 1935 but, after about 3 months, cracking again started and increased in amount and intensity so that another re-treatment was necessary. An inspection of the section made in October 1936, prior to applying the re-treatment, showed the section to be in poor condition. It had a number of badly broken areas of which one in the center of the business section of town was the worst. The mat in this area was not merely cracked but pieces had separated from each other and from the base. Sand had been washed into the cracks so that each piece appeared to be resting on a sand cushion. Figure 4 is an illustration of an area in this condition. Samples were taken of the marl base and of the subgrade and their analyses are given in table 1 under laboratory Nos. 10133 and 10134, respectively. The subgrade material was wet and sticky when sampled.

The portion of the section not cracked in the manner just described was in good condition. It had a smooth surface and appeared structurally sound. The re-treatment applied late in October was a heavy mixed-in-place seal. Prior to placing this treatment, 0.15 gallon of 85–100 cut-back was applied to the mat surface as a tack coat and to coat the sand particles that filled some of the cracks. The mixed-in-place mat or drag seal was composed of 0.45 gallon of cut-back asphalt with 25 pounds of ⅝- to ¼-inch stone and 25 pounds of sand per square yard. As a protective measure the treated portion of the roadway was widened 10 feet on each side through the business section of town (between stations 26+23 and 35+91). This treatment gave the section a smooth and uniform appearance and very little maintenance had been required on it up to July 1937, the end of the period of observation reported here.

The cost of constructing this section was 70.34 cents per square yard and the average annual cost of maintaining the surface has been 6.29 cents per square yard.

Experiment 2, section B, stations 39+60 to 52+00.—The method of construction and the amounts of material per square yard used on this section were as follows:

Prime: 0.29 gallon of tar.

Mix: 2 inches thick when compacted; 175 pounds of 1¼- to ¼-inch crushed stone and 0.89 gallon of 25–35 viscosity tar.

Seal: 0.23 gallon of the same bituminous material and 15 pounds of ⅝- to ¼-inch stone chips.

The surface of the mat on this section, like that of experiment 2A, was porous and open. The mat, however, was well bonded and did not ravel during the 2 months it was subjected to traffic before the seal coat was applied.

CONSIDERABLE MAINTENANCE NEEDED TO SEAL CRACKS

In the spring following construction, cracked areas appeared and were skin-patched after French drains were installed below the marl base. Test holes showed the bituminous mat to be rich in bitumen and to appear well sealed although water was observed below the mat and the marl base was damp at the top. By April 1930, the mat on the west 500 feet had become wet, spongy, and so badly cracked that it disintegrated in some areas and was removed and replaced with premixed material. On the remainder of this portion the cracks were sealed. With the advent of warm weather many of the cracks that had been observed elsewhere on the section earlier sealed themselves.

The entire section was given its first re-treatment during 1931. The east 400 feet were re-treated in June to seal the excessive amount of cracks that had appeared in that area. By fall, cracking became very pronounced on the remainder of the section and, in addition, the mat on the west 200 feet began to disintegrate as a result of the spongy condition of the marl base. Four French drains were installed and disintegrated areas totaling approximately 300 square feet, were replaced with premixed material.

This work at first appeared to have eliminated an unsatisfactory condition so the section, excepting the east 400 feet, was re-treated in December. However, by the following spring this area at the west end was again in an unsatisfactory condition. Cracks appeared through the patches placed to seal them. The marl base became spongy in spite of the four French drains placed during the previous fall. Only one of the four drains showed indications of functioning. Later in the summer a 60-foot section near the west end was scarified. The marl base and bituminous mat were mixed together, laid down as a new base and allowed to dry and compact under traffic. A new mat was placed on it in September. This reconstruction eliminated the instability and spongy subgrade condition. The east 400 feet, re-treated in June, had remained in good condition.

Although cracks appeared throughout the section at times, many of them closed in warm weather and the remainder were sealed by skin-patching as they occurred. The placing of numerous patches had gradually produced a nonuniform and unsightly mat, whose surface was not very smooth. To eliminate such conditions and also to enliven the surface, the entire section was given a fairly heavy re-treatment in September 1935.

Following this re-treatment the section apparently remained stable throughout except for small cracks that appeared. Many of these closed in warm weather but some of them had to be sealed. When inspected in October 1936, the section, with the exception of the east 200 feet, appeared to be in good condition. A few small cracked areas were observed as well as some surface roughness, especially near the west end. The east 200 feet were badly cracked and this portion was given a heavy re-treatment on October 30. Following this re-treatment little maintenance was required other than that of sealing a few cracks and of spreading some additional cover stone on the east end where bleeding occurred in hot weather.

The cost of constructing this section was 79.40 cents per square yard and the average annual cost of maintaining the bituminous mat, including the base reconstruction and re-treatments, was 7.15 cents per square yard.

Experiment 2, section C, stations 52+00 to 66+00.—The method of construction and the amounts of material per square yard used on this section were as follows:

Prime: 0.28 gallon of tar.

Penetration course: 2.3 inches thick when compacted; 165 pounds of 1¼- to ¼-inch stone, 37 pounds of ¼-inch to dust, and 1.27 gallons of asphalt emulsion.

Seal: 0.23 gallon of the same bituminous material and 15 pounds of ⅝- to ¼-inch stone chips.

It was planned originally to construct this section in the same manner as the two preceding sections but the producers of the emulsion objected and requested that a modified type of penetration construction be substituted. The construction which is here described therefore was done under their direction and with the approval of their representatives.

BASE FAILURE CAUSED HIGH MAINTENANCE COSTS

The screenings, ¼-inch to dust, were spread uniformly to a depth of ½ inch upon the primed marl base. The 1¼- to ¼-inch aggregate was then spread uniformly, sprinkled with water, and thoroughly rolled. Emulsion was then applied at the rate of 0.44 gallon per square yard and a strip 1 foot wide along each edge was given an additional amount of emulsion. Water was applied immediately to wash the emulsion down into the screenings and the surface was again rolled thoroughly. The second rolling was presumed to seat the coarse stone in the mortar of emulsion and screenings which would be forced up and around it. However, this expected action did not occur. Some of the water added to wash the emulsion down drained off at the edges, carrying an undetermined amount of emulsion with it. During the second rolling operation 8 pounds of ⅝- to ¼-inch stone per square yard were scattered over the surface and rolled into the surface voids. On the following day 0.83 gallon of emulsion was applied and about 7 pounds of ⅝- to ¼-inch stone per square yard were spread after which the surface was thoroughly rolled.

The seal was applied 1 week later, after the surface had been swept and loose or raveled areas had been patched. The treatment consisted of an application of 0.23 gallon of emulsion and 15 pounds of ⅝- to ¼-inch stone per square yard. Inspection showed that the first application of emulsion had not penetrated the cushion course at the bottom as expected, but, to a considerable extent, had been washed away by the water.

As shown in figure 1 and by the cost data in table 6, this section required a greater amount of maintenance than any other section and was the least satisfactory. The section lies partly in cut and partly on the highest fill on the experimental road. Trouble with moisture and failure of the marl base were experienced on this section almost from the start. First evidences of failure were surface cracks and sponginess in the base, especially near the center of the section and toward the west end where the section lies on a fill.

Surface patching was first done to seal cracks and prevent disintegration. By June 1931, however, these two areas were so badly cracked and disintegrated that the mat was scarified, remixed with additional emulsion, and relaid. A seal was then applied to these areas and also to the east 400 feet where surface cracking did not warrant remixing. In the following November, the entire section, excepting the east 400 feet, was given a re-treatment. One month later the area near the west end that had been remixed in June was given an additional re-treatment.

By September 1932, the mat on a 260-foot section near the center had become so badly cracked and the base appeared so unstable that reconstruction was deemed necessary. The marl base and the bituminous mat were scarified, mixed together, and relaid as a new base, which was then primed and covered with a heavy surface treatment. The area thus reconstructed included a portion of one of the areas whose mat had been remixed in 1931. In September 1933 the east 400 feet of the section was given its second re-treatment and two years later the remainder of the section was again re-treated. The entire section was again re-treated in October 1936.

Figure 1 shows that the entire section received three re-treatments in addition to the major surface and base repairs required on the central portion and on an area toward the west end.

FAULTY CONSTRUCTION METHODS PARTLY RESPONSIBLE FOR FAILURE

Prior to applying the re-treatment late in October of 1936, an inspection showed the east one-third of the section to be in good condition and to appear structurally sound. Approximately 350 feet adjoining it on the west were badly cracked and the mat appeared unstable. At the location examined, the mat was 3 inches thick but could be readily broken apart as it had little bond. Free water was observed in the bituminous mat and some of the aggregate had become uncoated. The area in question was on a slight grade where surface drainage was good and the location examined was but 5 feet down-grade from an intercepting lateral French drain. Samples of the marl base and of the subgrade were taken at this point and their analyses are given in table 1 under laboratory Nos. 10138 and 10139, respectively. The marl base was 7½ inches thick and while dense in appearance, was somewhat moist at the bottom. The sand-clay subgrade below the marl base was wet.

A re-treatment was applied October 30 and 31, 1936, to the entire section but by the following July the mat had cracked badly in the center of the section and had begun to ravel. Instability of the marl base again developed near the west end where it had been reconstructed in 1931. It is anticipated that this section very shortly will have to be entirely reconstructed by replacing the base or by stabilizing it in such a manner as to reduce its plastic properties.

The cost of constructing this section was 84.62 cents per square yard and the average annual maintenance cost was 13.40 cents per square yard.

The high maintenance cost of this section can be attributed in large measure to the unsatisfactory character of the marl base and the subgrade. Data shown in table 1 indicate that the base on this section possessed undesirable characteristics to a greater degree than any of the other marl base sections. The subgrade had the characteristics of soils of the A–2 plastic and A–4 very plastic groups and could be expected to be very unstable under unfavorable moisture conditions.

The method of constructing the bituminous mat, although at the time deemed satisfactory by the producers of the emulsion, would not be used at the present time. It is believed that the use of water to wash the emulsion down into the sand cushion was a mistake that was partially responsible for the excessive maintenance required, especially during the early life of this section.

Experiment 3, stations 79+20 to 92+40.—The method of construction and the amounts of material per square yard used on this section were as follows:

Prime: 0.30 gallon of tar.

Mix: 1.8 inches thick when compacted; 162 pounds of ¾- to ¼-inch stone and 38 pounds of stone ¼-inch to dust with 1.19 gallons of 85–100 cut-back.

Seal: None required.

This section was constructed in the same manner as experiments 1A and 1B, except that the coarse aggregate used was ¾ to ¼ inch in size. Approximately the same percentage of fine aggregate was used with this aggregate as had been used with the larger size stone. The resulting mat was apparently dense and well closed so that a seal was not applied.

EXPERIMENT 3 WAS BEST OF THOSE BUILT ON MARL BASE

Figure 1 and table 6 show this experiment to have been the best and the most economical to maintain, of the sections constructed on the marl base. Prior to the re-treatment in 1936 the total accumulated maintenance cost had been only 3 cents per square yard. The mat on this section, while appearing dry most of the time, remained in excellent condition up to the spring of 1936. The surface stayed smooth and unbroken. The mat remained dense and hard. No raveling occurred although a few cracks appeared, some of which closed in warm weather and the remainder of which were sealed with bituminous material.

In the spring of 1936, following a fairly severe winter, cracks in the surface appeared throughout the section. Skin-patching and sealing were not always effective in eliminating them. The section continued in this condition until October 1936, when it was given a re-treatment, the only one it received.

Prior to applying the re-treatment a fairly close examination was made of the section. It was noted that the mat was considerably cracked but that no raveling had occurred. The edges were in good condition and the surface was smooth. No base settlement was observed. Samples of the marl base and of the subgrade were taken from a cracked area near the center. At this location the bituminous mat was 2 inches thick and immediately below the surface appeared lifeless and very wet with some of the aggregate uncoated. The marl base was 7 inches thick and the core taken at this location was more moist at the bottom than at the top. The subgrade was sand-clay and it appeared more moist than the marl base.

The analyses of the marl base and of the sand-clay subgrade are given in table 1 under laboratory Nos. 10135 and 10136. Another area, approximately 300 feet west of the center, was also examined. There the bituminous mat was 2 inches thick, free from cracks, and was live and sticky. The marl base was 10 inches thick and appeared less dense than the base at the location previously examined. The subgrade was sand-clay that apparently contained little clay. The analysis of the marl base material at this location is given in table 1 under laboratory No. 10137.

The re-treatment applied the last of October 1936 was rather heavy, consisting of 0.58 gallon of 85–100 cut-back with 25 pounds of ⅝- to ¼-inch stone and 25 pounds of sand per square yard. Following this re-treatment, no maintenance was required and at the close of the observation period (July 1937) the section had a good appearance and seemed to be in excellent condition.

The cost of constructing this section was 61.07 cents per square yard and average annual cost of maintaining the bituminous surface was 2.24 cents per square yard.

Experiment 4, stations 92+40 to 105+84.—The method of construction and the amounts of material per square yard used on this section were as follows:

Prime: 0.26 gallon of tar.

Mix: 1.8 inches thick when compacted; 157 pounds of ¾- to ¼-inch stone, and 0.72 gallon of 85–100 cut-back.

Seal: An average of 0.22 gallon of the same bituminous material was applied and covered with 15 pounds of ⅝- to ¼-inch stone chips.

DENSE SURFACE OBTAINED BY APPLYING SEAL COAT

This section was similar to experiment 3 except that it contained no fine material. A dense surface was obtained by applying a seal coat instead of by mixing fine material in the mat.

This section, with two areas excepted, has been fairly satisfactory. The areas referred to were the east 400 feet and a small area near the center. Surface cracks appeared at times on all parts of the section but raveling or pot-holing was confined mostly to the two areas mentioned. Sealing with bituminous materials and warm weather eliminated most of the cracks. On the east 400 feet a re-treatment became necessary by the fall of 1931 to seal the cracks, enrich the surface, and eliminate the nonuniform appearance resulting from the patching done to repair small pot-holed areas.

Near the center, a small area became spongy and cracked badly when the marl base softened. On this area the bituminous mat was removed and the marl base was allowed to dry. A French drain was installed and a new mat of premixed material was placed. The re-treatment in 1931 and the base treatment just described constituted the major repairs required up to the early summer of 1934 when a spongy area of approximately 4 square yards developed. The marl base and mat on this area were scarified, mixed together, and relaid. After this new base had become dry and compacted, a surface of premixed material was placed.

Cracking continued in various amounts throughout the section as did a slight amount of pot-holing. Skin-patching and small premixed patches were generally satisfactory in preventing progressive failures but the numerous small patches reduced the smoothness of the surface considerably. To repair damaged areas and at the same time restore surface smoothness and uniformity, a re-treatment was applied to the entire section in September 1935. Except for the east 400 feet, this was the only re-treatment applied to this section during the period of observation. During the winter of 1935–36 no maintenance was required on the section but by the following spring some cracks appeared and some pot-holing occurred near the center of the section. This area was successfully repaired by skin-patching and by a small amount of patching with premixed material. One small area, however, repeatedly cracked where the marl base was spongy in spite of the French drain that had been installed.

Except for this small area, the section was in very good condition when inspected in October 1936. The surface was neither lean nor dry and it was smooth although somewhat mottled in appearance. The small unsatisfactory area in the center was badly cracked despite the numerous patches that had been placed. It was apparent that replacement of the marl base or stabilization by some suitable method would be necessary permanently to correct the unsatisfactory condition. The general condition of the section at the close of the period of observation was good except for the small unsatisfactory area.

The cost of constructing this section was 55.46 cents per square yard and the average annual cost of maintenance was 3.06 cents per square yard.

Experiment 5, stations 66+00 to 79+20.—The method of construction and the amounts of material per square yard used on this section were as follows:

Prime: 0.33 gallon of tar.

0.44 gallon of 85–100 cut-back was applied and covered with 50 pounds of 1¼- to ¼-inch crushed stone.

BASE MOVEMENT ATTRIBUTED TO CONSOLIDATION

Except for using a cut-back asphalt instead of the 150–200 penetration, hot-application material ordinarily used, this section was constructed by the surface-treatment method commonly used by the State at that time. Following the application of the cut-back asphalt, the 1¼- to ¼-inch cover stone was spread by hand from small stock piles previously placed at either side of the road. After the stone was spread and hand-broomed the surface was rolled. Traffic was not permitted on it for 24 hours, and during the first few days in service the stone displaced by traffic was respread and the surface was rolled intermittently.

Some patching was required on the portion of the section between stations 66+00 and 73+40 shortly after construction and this area was re-treated in November 1929. The treatment, which in reality was a seal, consisted of an application of 0.36 gallon of 85–100 cut-back and 15 pounds of ⅝- to ¼-inch stone.

During the next 2 years maintenance consisted mostly of sealing cracks and patching broken areas on the west half that had not been sealed. A seal was applied to this portion of the section in November 1931.

The east half of the section required some maintenance to seal the cracks that appeared at times. Sealing was generally effective for a considerable period but in a few instances cracks reappeared soon as a result of movement of the marl base. The base movement appeared to result from consolidation rather than from instability caused by detrimental amounts of moisture. In September 1933 cracking of the surface had become extensive on the east portion and it was re-treated. The west portion remained in good condition and was not re-treated in 1933.

The entire section required very little maintenance during the calendar years 1934 and 1935. The cracks that appeared either closed in warm weather or were sealed with bituminous material. A slight amount of raveling occurred along the edges at the west end and repairs were made by placing premixed material.

In the early months of 1936 cracks appeared throughout the section and numerous patches were required to prevent raveling. It was apparent that a re-treatment would be needed shortly. When inspected in October, before the re-treatment was applied, the section was considerably cracked but was not raveling. The surface was somewhat rough and had the typically dry, lean appearance of a bituminous surface in need of a re-treatment.

The mat and foundation were examined at station 68, approximately 200 feet from the east end. At this location the bituminous mat was 1 to 1¼ inches thick and appeared to contain water. The tar prime had penetrated ½ to ¾ inch into the marl base, which was 6½ inches thick at this point. Below the marl was a yellow sand subgrade. Directly below the bituminous mat at the point of examination free water was found at a depth of 11 to 12 inches from the road surface. On the adjacent shoulder, however, free water was found 29 inches below the surface. A sample of the marl base was taken at the location examined and its analysis is given in table 1 under laboratory No. 10140.

The re-treatment applied to the entire section later in October consisted of 0.56 gallon of 85–100 cut-back with 25 pounds of ⅝- to ¼-inch stone and 25 pounds of sand. This heavy re-treatment apparently placed the section in very good condition. Cracking and raveling were eliminated and surface smoothness was restored. It did become necessary, however, to spread small amounts of stone on the surface to prevent picking up in warm weather. Except for this richness the section was in very good condition at the close of the observation period, July 1937.

The cost of constructing this section was 28.47 cents per square yard and the average annual cost of maintenance was 5.13 cents per square yard.

PLASTIC CHARACTER OF MARL BASE CAUSED SURFACE CRACKS

In reviewing the service behavior of the sections on the marl base a number of facts appear to merit special comment. The most prevalent weakness displayed by the sections was their tendency to crack. The primary cause of cracking was the plastic character of the marl base and the fact that it was exceptionally difficult to drain even after numerous French drains had been installed. Leanness of the mixed mats and the lack of a tightly sealed surface on some of the sections permitted the entrance of moisture from the surface, thereby increasing the instability of the marl base.

Cracking also occurred on sections where there appeared to be no movement that indicated a lack of base stability. Such cracking was attributed directly to the leanness and openness of the bituminous mat.

All of the sections cracked considerably, but this characteristic was less pronounced on experiments 1A, 1B, and 3, which contained fine aggregate, than on experiments 2A, 2B, and 4, which did not contain such fine material but which were sealed in lieu of using fine aggregate in the mix. Raveling did not become especially serious at any time, as prompt maintenance prevented such failures.

Routine surface maintenance was generally effective, but where the defect lay in the base the only remedy was the removal of the base and replacement with satisfactory material.

LITTLE MAINTENANCE REQUIRED ON EXPERIMENT 6A

When the sand-clay base had been made as uniform and compact as possible under the conditions, a triangular trench was cut on each side to provide for a thickened edge for the mat. The trench was approximately 1 foot wide, and 4 inches deep on the outside edge. It was cut with a blade machine and was somewhat irregular in shape. Priming was beneficial in helping to bond the surface but during the mixing process the surface crust was considerably disturbed by construction equipment. Some of it broke up and was brought into the mix by the blade machines. Much of it was removed by hand before the mixes were placed. Under such circumstances the surface condition of the sand-clay base would obviously be quite variable when the mixed mats were completed but it was impracticable to determine the extent of such variation.

Experiment 6, section A, stations 105+84 to 119+99.—This section corresponds to experiment 1A. The method of construction and the amounts of material used per square yard were as follows:

- Prime: 0.33 gallon of tar.
- Mix: 2 inches thick when compacted; 171 pounds of 1¼- to ¼-inch stone, 40 pounds of ¼ inch to dust, and 1 gallon of 60–70 cut-back.
- Seal: 0.34 gallon of 85–100 cut-back was applied and covered with 15 pounds of ⅝- to ¼-inch stone chips.

The appearance and early behavior of the mat obtained with the above materials was very similar to that on experiment 1A, the corresponding section on the marl base. The surface was lean and open and raveled somewhat immediately after construction. A seal treatment, although not originally planned for this section, was applied 1 month after constructing the mixed mat. The bituminous material used in the seal was an 85- to 100-penetration cut-back asphalt instead of the 60- to 70-penetration material originally intended to be used.

The fact that the maintenance cost of this section, exclusive of the cost of the 1933 re-treatment, totaled only 1.91 cents per square yard for the entire period of observation, best indicates the satisfactory service behavior of this section. Only a slight amount of maintenance was required. Shortly after construction, a few small areas had to be built up with premix material to prevent water from collecting in low spots where the sand-clay base had settled slightly. A few cracks required sealing as did a few porous surface areas.

In the fall and winter of 1932 and 1933 quite a number of cracks appeared along the edges, especially on the low side of a curve. They were sealed by skin-patching but this was of only temporary benefit as the cracks reappeared shortly. By the fall of 1933 it was apparent that a re-treatment would be beneficial in enriching the surface to eliminate the characteristically lean appearance that had gradually developed and to seal all cracks and leave the section in a uniform condition.

FIGURE 5.—GENERAL VIEW OF THE EAST PORTION OF EXPERIMENT 6A, WHICH IS TYPICAL OF THE ENTIRE SECTION. NOTE THE EVEN EDGES AND UNIFORM APPEARANCE.

The re-treatment was applied in September 1933, and was the only one this section received.

When inspected in October 1936, the section was in excellent condition, probably better than any other section. The surface was smooth and free from cracked or raveled areas. The edges were in very good condition and there were no ruts or other surface inequalities. Practically no maintenance had been required since the 1933 re-treatment and none was needed at the time of this inspection. Figure 5 shows a general view of the east end of the section and figure 6 shows the texture of the mat. The analysis of the section of the mat illustrated is given in table 8.

The cost of constructing this section was 72.53 cents per square yard and the average annual cost of maintenance, including the re-treatment, was 1.82 cents per square yard.

Experiment 6, section B, stations 119+99 to 132+00. This section corresponds to experiment 1, section B. The method of construction and the amounts of material used per square yard were as follows:

Prime: 0.39 gallon of tar.
Mix: 2 inches thick when compacted; 170 pounds of 1¼- to ¼-inch crushed stone, 42 pounds of ¼ inch to dust, and 1.09 gallons 85–100 cut-back.
Seal: 0.23 gallon of the same bituminous material and 15 pounds of ⅝- to ¼-inch stone chips.

Construction of the bituminous surface on this section was similar to that on experiment 1B, the corresponding section on the marl base. After the mixture had been spread but before it could be consolidated by rolling, a heavy rain fell that soaked the mix and the base. During the rolling process a number of spongy areas appeared where the sand-clay base had been softened. These areas were removed and replaced with satisfactory materials.

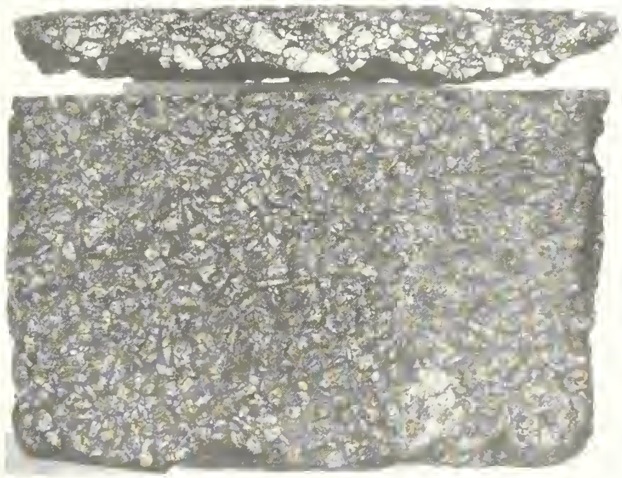

FIGURE 6. APPEARANCE OF A SAMPLE OF THE SURFACE OF EXPERIMENT 6A IN APRIL 1937.

The seal coat was not applied until the section had been under traffic for 3 months. During this period no raveling occurred although the surface appeared lean and open.

MAINTENANCE REQUIRED BECAUSE OF BASE SETTLEMENT

The service behavior of this section has been very good as may be inferred from its maintenance cost which, exclusive of the re-treatment applied in 1931, totaled only 5.61 cents per square yard for the period of observation.

TABLE 8.—*Analyses of samples of bituminous mats, taken 7½ years after construction*

	Section 6A (station 119+76; 6½ feet right of center-line)	Section 8 (station 225+78; 4 feet left of center-line)	Section 9 (station 238+52; 5 feet left of center-line)	Section 10 (station 199+44; 5 feet right of center-line)
Composition of mat.	(1)	(2)	(2)	(3)
Bitumen ... percent	5.6	3.6	3.7	4.7
Mechanical analysis:				
Passing 1¼, retained 1 inch ... do	1.9	----------	----------	3.4
Passing 1, retained ¾ inch ... do	5.8	----------	----------	6.7
Passing ¾, retained ½ inch ... do	14.1	3.7	6.3	8.1
Passing ½, retained ¼ inch ... do	28.5	32.0	38.3	25.0
Passing ¼, retained No. 10 ... do	20.1	27.2	30.0	25.5
Passing No. 10, retained No. 40 ... do	11.6	18.2	10.1	10.4
Passing No. 40, retained No. 80 ... do	5.2	7.1	4.6	6.9
Passing No. 80, retained No. 200 ... do	3.9	4.8	3.9	5.6
Passing No. 200 ... do	3.3	3.4	3.1	3.7
Volatile portion of bitumen [4] ... do	.8	.0	.4	.3
Tests on extracted bitumen recovered by the Dow method:				
Penetration at 77° F	32	35	29	27
Softening point ... ° F	151	147	154	163
Ductility at 77° F ... cm	14	15	8	5
Organic matter insol. in 86° B. naphtha ... percent	32.2	29.4	32.2	34.5

[1] Represents original mat plus 1 re-treatment (1933).
[2] Represents original mat.
[3] Represents original mat plus 1 re-treatment (1931).
[4] Determined by the crank-case dilution method, A. S. T. M. D322-35.

Prior to November 1931, maintenance on this section consisted of sealing surface cracks and eliminating low areas by patching. All of this maintenance was required because of base settlement that might be more properly termed consolidation.

The only re-treatment this section received was applied in November 1931, to eliminate the nonuniform appearance that gradually developed from the routine maintenance applied and to complete in one operation all necessary maintenance. After this re-treatment was applied, the bituminous surface required only a small amount of maintenance in 1932 and none thereafter. In October 1936, 5 years after the re-treatment had been applied, the mat appeared stable and in very good condition, although it was neither as smooth nor as uniform in appearance as experiment 6A. No cracks were observed nor had raveling occurred although in the traffic lanes the surface was somewhat open and ragged in appearance where some of the cover stone used in the 1931 re-treatment had been whipped from the surface shortly after the re-treatment was applied. At the close of the period of observation, June 30, 1937, the section was in very good condition.

The cost of constructing this section was 72.10 cents per square yard and the average annual cost of maintenance, including the re-treatment, was 1.95 cents per square yard.

Experiment 7, section A, stations 132+00 to 144+20.—This section corresponds to experiment 2A. The method of construction and the amounts of material used per square yard were as follows:

- Prime: 0.29 gallon of tar.
- Mix: 2 inches thick when compacted; 178 pounds of 1¼- to ¼-inch stone and 0.77 gallon of 85-100 cut-back; mixed, shaped, and rolled.
- Seal: 0.27 gallon of the same bituminous material was applied and covered with 18 pounds of ⅝- to ¼-inch stone chips.

Rain fell on this section about the time that the mixing operation was completed and because of the openness of the mixture, readily penetrated to the sand-clay base. Additional manipulation for drying purposes was not deemed advisable and the mixture was spread and rolled. After rolling, the surface appeared to be in good condition except for the moisture it contained. Traffic was not permitted on the surface until it had dried. The seal was applied about 1 week later after the surface had been choked with small stone under traffic. A few days after the section had been sealed the base was found to be dry and the surface apparently well sealed.

The behavior of this section was similar to that of experiment 6B except that it required somewhat more maintenance. Base settlement necessitated patching to eliminate numerous small low areas. Cracks which appeared most extensively at the west end were sealed. To reduce the routine maintenance being required and to restore surface smoothness and uniformity, the section was given its first and only re-treatment in November 1931. In the following 5¾ years, practically no maintenance was required except to patch and seal small areas where some of the cover stone used in the re-treatment had been whipped from the surface. When examined in October 1936, the section was in very good condition. The mat appeared stable and showed no evidence of impending failure. The surface was somewhat open in texture in spite of the seal and re-treatment it had received. On the east 50 feet, where equipment had turned during construction, the surface was rough.

The cost of constructing this section was 62.86 cents per square yard and the average annual cost of maintenance, including re-treatment, was 2.62 cents per square yard.

Experiment 7, section B, stations 199+70 to 212+90.—This section corresponds to experiment 2B. The method of construction and the amounts of material used per square yard were as follows:

- Prime: 0.25 gallon of tar.
- Mix: 2 inches thick when compacted; 169 pounds of 1¼- to ¼-inch crushed stone and 0.77 gallon of 25-35 viscosity tar.
- Re-treatment: Stations 206+50 to 212+90, seal coat applied, using 0.42 gallon of the same bituminous material and 15 pounds of ⅝- to ¼-inch stone chips. Station 199+70 to 206+50, remixed with 0.30 gallon of the same bituminous material. Sealed with 0.23 gallon of same bituminous material and 15 pounds of ⅝- to ¼-inch stone chips.

INSUFFICIENT TAR IN ORIGINAL MIXTURE MADE SEAL COAT NECESSARY

This section was planned as a duplicate of experiment 2B on the marl base and the method of constructing the mixed mat was the same. However, the amount of tar used was about 10 percent less. This difference was sufficient to affect seriously the richness of the mixture which, when spread and rolled, immediately began to ravel under traffic; whereas on experiment 2B the mat was well bonded and did not ravel during the 2 months it was subjected to traffic before it was sealed.

Because of the lateness of the season it was thought inadvisable to remix this section with additional tar and it was decided to compensate for this deficiency by placing a fairly heavy seal coat. Tar was applied at the rate of 0.42 gallon per square yard and covered with ⅝- to ¼-inch stone chips. The supply of tar on hand was sufficient for sealing only the west 640 feet;

and by the time additional tar was received, 10 days later, the unsealed portion of the section had raveled so badly that remixing was considered necessary. Approximately 0.30 gallon of tar was added and the surface was remixed and relaid. Two months later a re-treatment, which was in effect a seal coat, was applied to this portion in order to close the surface and prevent moisture from entering.

This section has been more expensive to maintain than any of the previously discussed sections on the sand-clay base. It has, however, continued in reasonably good condition at all times. Because of leanness and gradually developing brittleness, raveling did occur on the section, mostly along the edges. Throughout the period of observation the surface was open and appeared rough but was not rough riding. Routine maintenance prevented the small amount of raveling from developing into pot holes of serious proportions and, while the section most of the time appeared to be in need of a re-treatment, none was actually applied until October 1936.

Just prior to applying the re-treatment the section was in fairly good condition. The mat was hard and brittle. Some raveling had occurred and was becoming more pronounced. Patching had been required on the west end, especially along the edges. Many parts of the section, however, were in very good condition. The surface, while dry and coarse-textured, was well bonded. Figure 7 shows the condition of such an area. It appeared that a re-treatment would be beneficial and one was applied late in October. Following this re-treatment little maintenance was required other than the spreading of small amounts of stone where the surface became somewhat soft in hot weather. At the end of the period of observation on June 30, 1937, the section was in good condition and the surface had a smooth, rich appearance.

The cost of constructing this section was 73.23 cents per square yard and the average annual cost of maintenance, including the re-treatment, was 3.47 cents per square yard.

Experiment 7, section C, stations 144+20 to 158+40. This section corresponds to experiment 2C. The method of construction and the amounts of material used per square yard were as follows:

- Prime: 0.34 gallon of tar.
- Penetration course: 2.3 inches thick when compacted; 165 pounds of 1¼- to ¼-inch stone, 30 pounds of ¼-inch to dust and 1.11 gallons of asphalt emulsion.
- Seal: 0.25 gallon of the same bituminous material and 15 pounds of ⅝- to ¼-inch stone chips.

This section was constructed by the penetration method without mixing. Consequently the primed sand-clay base was not disturbed in any way by the construction of the mat.

As on experiment 2C, where similar materials and methods of construction were used, a considerable amount of emulsion was carried away by the water used to wash it down into the fine stone cushion. The completed mat was rough and, during a 10-day period under traffic before sealing, it raveled considerably. Before the seal was applied raveled areas were repaired and depressions were filled with ½-inch stone and emulsion. When the section was completed it was found that here too, as on experiment 2C, the emulsion had not penetrated into the cushion course below the coarse aggregate.

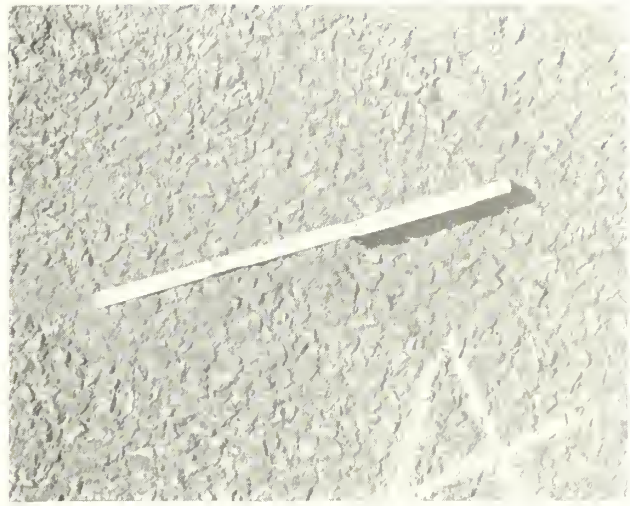

Figure 7.—Close-Up View of the Surface of Experiment 7B. Although Some Portions Raveled, Other Parts Retained a Closed, Non-Skid Surface.

Some early maintenance was required on this section to eliminate a few low areas produced by base settlement. This settlement was probably consolidation rather than movement resulting from loss of stability. Near the west end, in the vicinity of a culvert, small areas repeatedly cracked and broke where the base was somewhat spongy. French drains were installed and, when the base had dried, premixed patches were placed. This work constituted practically all of the maintenance that was made necessary by base weakness.

HIGH MAINTENANCE COST ATTRIBUTED TO METHOD OF CONSTRUCTION

To give the section a uniform appearance and to seal all cracks in a single operation, a re-treatment was applied to the entire section in November 1931. During the application of the emulsion in this re-treatment, the distributor nozzles became clogged and failed to deliver the emulsion uniformly, leaving uncoated streaks 2 to 4 inches wide and 10 to 50 feet long. Hand-pouring of emulsion on these areas did not provide a uniform cover and the amount of stone held by the treatment was consequently somewhat variable. The added emulsion benefitted the surface, however, and except for the loss of some cover stone the section remained in good condition.

Little maintenance was required after the 1931 re-treatment and consisted mainly in eliminating small pot holes resulting from the loss of stone used in the re-treatment. No further trouble was encountered with the base on the west end. Slight amounts of settlement on each side of the culvert made premixed patches necessary to maintain a reasonably smooth surface.

By the fall of 1936, the section had developed a lean, dry appearance, and its nonuniformity made a re-treatment desirable. A heavy re-treatment was applied to the entire section late in October 1936 and left the section in a uniformly good condition. In the summer following this re-treatment it became necessary to spread small amounts of stone to prevent the surface from picking up in hot weather, but no other maintenance was required. At the close of the period of observation the section was in a uniformly good condition, the surface being smooth and free from irregularities.

The cost of constructing this section was 84.55 cents per square yard and the average annual cost of maintenance, including the two re-treatments, was 5.70 cents per square yard. The maintenance cost of experiment 7C was relatively high in comparison with that of the other sections built on the sand-clay base. Unlike section 2C, its counterpart on the marl base, this relatively high cost could not be attributed, to any great extent, to the character of the base material.

The base on experiment 7C, at the time of construction, appeared very similar to that of the adjoining experiments 7A and 10 whose bases were group A 2 and A 3 materials. The high maintenance cost of experiment 7C is therefore believed to result primarily from the method of construction employed. It will be noted by reference to table 6 that the maintenance costs were relatively high up to the time of the 1931 re-treatment and were substantially lower after that time. Apparently the heavy re-treatment compensated for the lack of bituminous material resulting from the loss incurred during construction.

Experiment 8, stations 212+90 to 226+10.—This section corresponds to experiment 3. The method of construction and the amounts of material used per square yard were as follows:

- Prime: 0.26 gallon of tar.
- Mix: 1.8 inches thick when compacted; 154 pounds of ¾- to ¼-inch stone, 38 pounds ¼ inch to dust, and 1.25 gallons of 85–100 cut-back.
- Seal: None required.

The sand-clay base on this section was very nonuniform, and under traffic compacted into strata that sealed considerably, especially along the edges. The base, when primed, had a very ragged appearance. During the mixing operation, rain fell on this section. The mixture was windrowed until the exposed base and the mixture had dried. When mixing had been completed and the mat partially rolled, local traffic rutted the surface so badly that the mat was loosened, remixed, and relaid. Although the mix had become somewhat stiff, because of loss of the volatile portion of the cut-back, no difficulty was encountered in obtaining a well-compacted and well-closed mat. A seal was not considered necessary.

EXPERIMENTS 8 AND 9 HAD LOW MAINTENANCE COSTS

The fact that the total maintenance cost of this section for 7¾ years was only 1.26 cents per square yard, and that it is still in excellent condition, indicates the continued satisfactory service behavior. The portion of the road on which this section lies is flat and has poor drainage, but the base has remained stable at all times. A few transverse cracks made their appearance after a time but no detrimental effects were observed. In the spring of 1934 an examination of the section showed the mat to be 1½ inches thick and to be sufficiently rich below the surface although the surface itself was dry and hard. The sand-clay base was dry and hard.

When inspected in October 1936, the section was in excellent condition. The surface was smooth and showed no defects other than the presence of a few longitudinal and transverse cracks as previously mentioned. No raveling had occurred along the cracks or on any other area of the section. Figure 8 illustrates the texture and condition of the surface and also shows a side view of the mat. The analysis of the mat at the location illustrated is given in table 8. No expenditure for maintenance was made after 1932 and, at the close of the period of observation June 30, 1937, no maintenance was needed.

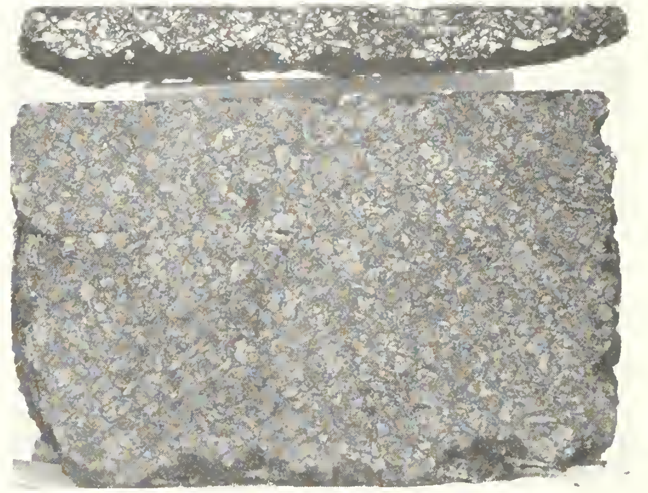

FIGURE 8.—APPEARANCE OF A SAMPLE OF THE SURFACE OF EXPERIMENT 8. THE CRACKS SHOWN WERE CAUSED BY HANDLING THE SAMPLE.

The cost of constructing this section was 59.53 cents per square yard and the average annual cost of maintenance was 0.16 cent per square yard.

Experiment 9, stations 226+10 to 238+80.—This section corresponds with experiment 4. The method of construction and the amounts of material used per square yard were as follows:

- Prime: 0.27 gallon of tar.
- Mix: 1.8 inches thick when compacted; 152 pounds of ¾- to ¼-inch stone and 0.77 gallon 85–100 cut-back.
- Seal: 0.29 gallon of the same bituminous material was applied and covered with 14 pounds of ⅝- to ¼-inch stone chips.

One month preceding construction of the bituminous surface on this section, the sand-clay base was scarified and additional clay binder added. It was then remixed, relaid, and opened to traffic for compaction. When construction of the surface was started the base was in fair condition.

The bituminous surface was the same as that on experiment 4. During the first 1½ months under traffic and before the surface was sealed, considerable raveling occurred, resulting probably from the lack of bituminous binder and of sufficient fine aggregate to provide a well-graded mixture. It had been anticipated that a seal coat would be required to obtain satisfactory surface density.

The service behavior of this section, like that of experiment 8, is best indicated by its low maintenance cost which, for the 7¾-year period, totaled only 2.92 cents per square yard. No re-treatments were applied to the section. Shortly after construction and also in the fall of 1932, some base settlement or consolidation necessitated the placing of a few patches to maintain a smooth surface. A very limited amount of skin-patching was required to seal small cracks. No transverse cracks, such as were found on experiment 8, appeared on this section. The surface throughout most of the 7¾ years appeared hard and dry but it neither pot-holed nor raveled. Practically no maintenance was required during the last 5 years of the period covered by this study.

When inspected in October 1936, the section was in excellent condition and was very similar to experiment 8 in appearance except that there were no transverse cracks. Near the edges there were light streaks, caused apparently by the failure of the end nozzles of the distributor to deliver sufficient bituminous material to hold all of the cover stone used in the seal treatment. Aside from its appearance the surface has been very satisfactory. Figure 9 shows the texture and condition of the surface. The analysis of the mat shown in figure 9 is given in table 8.

The cost of constructing this section was 66.74 cents per square yard and the average annual cost of maintenance was 0.38 cent per square yard.

Experiment 10, stations 158+40 to 199+70 (less 170 feet).—This section corresponds to experiment 5. The method of construction and the amounts of material used per square yard were as follows:

Prime: 0.29 gallon of tar.

0.46 gallon of 85–100 cut-back and 52 pounds of of 1¼- to ¼-inch crushed stone.

Re-treatment: 0.32 gallon of the same bituminous material covered with the loose stone that had been whipped off by traffic, plus 13 pounds of ¾- to ¼-inch crushed stone.

SAND-CLAY BASE GAVE GOOD SUPPORT DESPITE APPARENT LACK OF DRAINAGE

The sand-clay base on this section was very nonuniform and under traffic compacted in strata that separated when the base was swept prior to applying the prime. Some of the primed base flaked off and was removed and the untreated areas of base exposed were painted with cut-back. The cut-back and cover stone were then spread and the surface was rolled. During the rolling operation there was extensive failure of the sand-clay base. On many areas the upper portion of the base broke loose and worked up through the mat. Under traffic such areas quickly disintegrated and the surface was whipped off, leaving the base exposed. It was impractical to patch the numerous areas failing in this manner, but an immediate treatment was necessary. A treatment was applied consisting of an application of 0.32 gallon of 85–100 cut-back and a cover of approximately 38 pounds of stone, part of which was that swept from the surface before treatment and the remainder was new stone ¾ to ¼ inch in size.

Despite the difficulties encountered during construction this section has been surprisingly satisfactory and economical.

Since the bituminous surface was not constructed by the mixed-in-place method, it was expected that the irregular contour of the base would cause some surface unevenness. Such was the case and most of the maintenance applied up to November 1931 consisted of patching thin areas, filling depressions, and strengthening the edges.

The re-treatment applied in November 1931 was practically the same as the original construction except that the cover stone was ¾ to ¼ inch in size. Following this re-treatment the amount of maintenance required up to the close of the period of observation was practically negligible. When inspected in October 1936, 5 years after the re-treatment had been applied, the section was in excellent condition. The surface was smooth and dense. The edges were sound and no evidence of cracking, raveling, or other defects, was observed. Figure 10 shows the texture and surface appearance of a typical area of the section. The analysis of the bituminous mat illustrated is given in table 8.

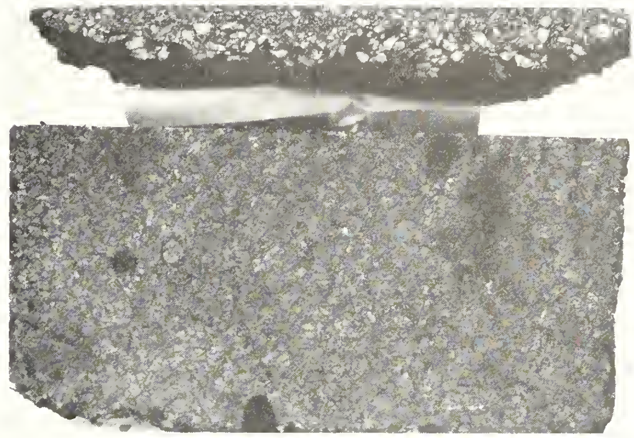

FIGURE 9.—APPEARANCE OF A SAMPLE OF THE SURFACE OF EXPERIMENT 9. THE CRACKS SHOWN WERE CAUSED BY HANDLING THE SAMPLE.

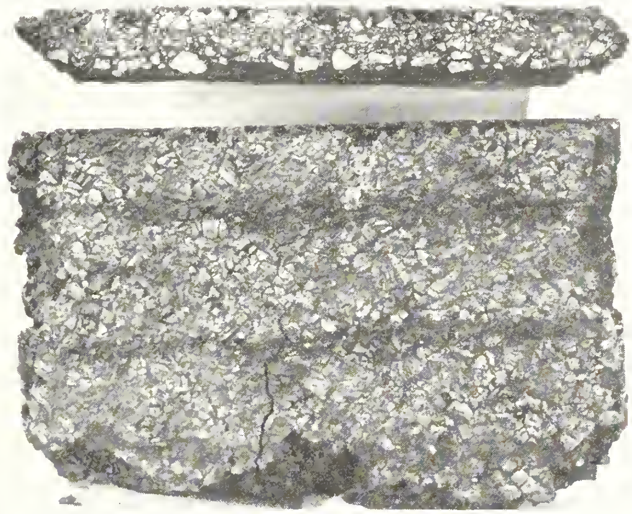

FIGURE 10.—APPEARANCE OF A SAMPLE OF THE SURFACE OF EXPERIMENT 10. THE CRACKS SHOWN WERE CAUSED BY HANDLING THE SAMPLE.

The cost of constructing this section was 29.44 cents per square yard and the average annual cost of maintenance, including the re-treatment, was 2.08 cents per square yard.

In reviewing the service behavior of the sections on the sand-clay base, probably the most outstanding fact was the excellent behavior of the sand-clay base in spite of an apparent lack of drainage since at the time of construction the sand-clay material was believed to be of inferior quality. No trouble was experienced because of moisture reaching the base and there was no reason to believe that the surfaces were more waterproof than were their counterparts on the marl base. The small amount of cracking that occurred on the sand-clay sections appeared to be caused in practically all instances by base settlement and compaction rather than by softening and loss of supporting power. Repeated maintenance of a given area was required only on experiment 7C, where the subgrade was unstable. Even at this location when the base was exposed and allowed to dry it acquired adequate stability and remained stable thereafter.

At the close of the period of observation, all of the sections were in good condition and gave every indication of long continued good behavior.

INFORMATION OBTAINED ALREADY PUT TO PRACTICAL USE

As stated in the original construction report and earlier in this report, the purpose of this experiment was to obtain information on a number of subjects of considerable importance in low-cost bituminous road construction among which were:

1. Information on the suitability of a local marl and a local sand-clay as base materials for bituminous surfaces.
2. The comparative value of various types of liquid bituminous materials.
3. The relative merits of variously graded aggregates.
4. The comparison of bituminous wearing surfaces produced by different methods of construction.

Since this experimental road was built, many hundreds of miles of bituminous surfaces have been built. All of the materials used in this road and most of the methods used are still employed in constructing bituminous surfaces. The successful results that are being obtained at the present time are based upon experience gained on previous construction as well as on the more carefully controlled and observed experimental section such as herein decribed.

Obviously the application of results obtained from experimental sections of this character must of necessity take place in advance of the time of publishing a report that covers a service behavior record for any extended period of time. For this reason most of the information developed by this experiment has already been put to practical use. However, the fact that the work has been closely observed and cost records carefully kept for 8 years and that it is still in service makes it of interest.

Discussion must obviously be confined to the experimental sections and any conclusions drawn or assumptions made would not necessarily be applicable to other sections that might, upon superficial examination, appear to be the same but which, in reality, might be widely different.

The fact that one of the subjects upon which information was sought in this experiment was the use of marl and sand-clay as base materials indicated the realization of the importance of bases for flexible pavements. As previously mentioned, the varied degrees of success that had been obtained with sand-clay were almost as numerous as were the various possible combinations of it. Little knowledge of the characteristics that affected its road-building properties was available and the only method of determining its suitability was by the relatively costly method of constructing an experimental road. Marl was used because it was available locally and because of its successful use in neighboring States. At the time the bituminous wearing surfaces were constructed the marl base appeared to be in excellent condition. Its surface was hard and smooth and apparently was not affected by occasional rains. Its behavior as an untreated surface gave no indication of the properties that were to result later in unsatisfactory behavior.

The sand-clay base, on the other hand, appeared to be in only fair condition when the bituminous treatments were applied. In spite of the blading, mixing, and shaping that was done on numerous occasions, it appeared impossible to obtain a uniform, well-compacted base that would remain undisturbed by traffic until it was protected by the bituminous wearing surface. Here, too, the properties that were to affect its service behavior were not recognized, solely because of the fact that soil-study methods had not reached their present stage of development. Table 1, which gives the properties of the soil comprising the sand-clay, the marl, and the sub-base under the marl, shows that those properties of the sand-clay that made it rather difficult to place and compact were very desirable characteristics so far as concerns the influence of moisture upon it; and that instead of being an inferior base material it was, except for lack of uniformity, an excellent one generally and its service behavior could have been anticipated had its characteristics been better understood.

MOISTURE BENEFITED SAND-CLAY BASES, HARMED MARL BASES

It will be observed that the sand-clay as represented by 19 tests had the characteristics of soil groups A–1, A–2, and A–3, and that in only one instance was it shown to have plastic properties sufficient to indicate possible unsatisfactory behavior in the presence of moisture. The marl, as represented by six tests, is shown to possess the characteristics of soils of the A–2 plastic, A–4, and A–5 groups, and consequently was adversely affected by moisture. In addition, group 5 soils possess undesirable elastic properties.

During the periods in which the marl and the sand-clay served as wearing surfaces the marl was decidedly more satisfactory than the sand-clay, but after the bituminous surfaces had been placed and the moisture content had increased there was an immediate reversal of behavior. The sand-clay base was benefited by the increase in moisture while the marl was detrimentally affected. Further reference to the analysis of the material underlying the marl shows that this material, in four out of seven instances, could have been expected to serve more satisfactorily as base material than the marl itself and in only one instance, judged by its analysis, would it have been expected to be much less satisfactory.

The better behavior of the bases of experiments 3 and 4 might be expected to have resulted from the better sub-base under the marl and the satisfactory behavior of the bases of experiments 1A and 1B might be expected to have resulted from the construction of the deep side ditches that apparently were of considerable benefit to the plastic and feebly plastic A–2 soil sub-base. The side drainage on the sand-clay base sections was not as ample as that on the marl base, and experiments 8 and 9 especially had practically no drainage. The characteristics of the soil composing the sand-clay indicated that drainage was not so vitally important and its service record substantiates the prediction that soils of this character would be stable even under rather unsatisfactory moisture conditions. It appears to have been definitely demonstrated by this experiment, therefore, that local designations for soils or chemical analyses of them alone are of little value in anticipating probable service behavior. The present method of determining the grading and physical properties of soils seems, in general, to provide the most reliable information thus far developed upon which probable service behavior can be predicted with reasonable accuracy.

This is further confirmed by the results of a study [3] conducted later by the Bureau of Public Roads on

[3] Road-Building Limerocks, by R. C. Thoreen, PUBLIC ROADS, Vol. 16, No. 8, October 1935.

TABLE 9.—*Analyses of limerocks or marls used in base construction in Alobama, Florida, and Georgia and on the experimental sections*

	Identification and service behavior									
	Group 1,[1] excellent		Group 2,[1] good		Group 3,[1] fair		Group 4,[1] poor		Experimental sections [2] mostly poor	
	Average	Range	Average	Range	Average	Range	Average	Range	Average	Range
Chemical composition:										
Silica, alumina, and iron oxide..percent..	0.78	0.40– 1.35	7.82	3.5 – 11.1	1.67	0.35– 3.20	8.29	6.65– 10.0	....	10.4 – 14.6
Calcium carbonate....do....	98.3	97.8 –98.7	91.1	87.5 – 96.1	97.1	95.3 – 98.8	89.7	87.7 – 91.6	....	83.9 – 86.9
Magnesium carbonate....do....	.80	.76– .87	.74	.61– .87	.96	.76– 1.51	.93	.68– 1.44	....	1.33– 1.65
Soil test constants:										
Liquid limit	21	17 –26	20	18 – 23	24	20 – 27	28	23 – 34	38	36 – 41
Plasticity index	0	0	4	0 – 6	4	0 – 7	11	8 – 17	9	5 – 17
Shrinkage limit	25	21 –35	20	16 – 25	25	22 – 29	21	16 – 24	30	27 – 32
Shrinkage ratio	1.6	1.4 – 1.7	1.7	1.6 – 1.8	1.6	1.5 – 1.7	1.7	1.6 – 1.8	1.4	1.4 – 1.5
Centrifuge moisture equivalent	17	14 –23	16	12 – 20	23	19 – 30	25	18 – 31	23	19 – 29
Field moisture equivalent	19	16 –24	19	15 – 22	24	21 – 30	21	17 – 24	34	29 – 42
Other physical tests:										
Cementing value	20	11 –53	83	50 –110	57	11 –168	180	67 –387	....	133 –500+

[1] From Road-Building Limerocks, by R. C. Thoreen, PUBLIC ROADS, October 1935. [2] From table 1.

road-building limerocks, which include marls. This study was made to correlate test analyses with service behavior. The materials studied were those actually used in base construction in Florida, Georgia, and Alabama. The test results are summarized in table 9 and are grouped according to the service behavior of the materials studied. For convenience of comparison, the analyses of the marl on the experimental sections are also included in this table.

NO FAILURES COULD BE ATTRIBUTED TO IMPROPER GRADING

The differences in the characteristics of the bituminous materials used on these sections were not reflected in their service behavior. The weaknesses that developed in the bituminous mats resulted primarily from unsatisfactory base conditions and the use of a relatively low percentage of bituminous material rather than from the type of bituminous material used. All of the mats were relatively lean and as a result were too rigid to adjust themselves to any appreciable base movement without cracking.

Moisture also had a deterimental effect on the lean mixtures. Had a greater amount of bituminous matetial been used it is very probable that the amount of cracking would have been greatly reduced. Raveling may follow cracking or may occur independently of it when the percentage of bituminous material is low or when the bituminous residue has hardened as a result of weathering. However, on this project, raveling was not extensive because of the prompt and continued maintenance. On experiment 7B, the tar section on the sand-clay base, a small amount of raveling occurred although it was not extensive enough to warrant a re-treatment until 1936. On experiment 2B, the corresponding section on the marl base, raveling was more pronounced. On this section the base movement and moisture caused a considerable amount of cracking that resulted in raveling in spite of maintenance.

No difference in behavior was observed that could be attributed to the penetration of the base asphalts used in the cut-back materials. The nonuniform appearance of experiment 7C, in which an emulsion was used, resulted from the mechanical difficulty encountered in applying the emulsion with the equipment available rather than from the character of the emulsion.

The viscosities of the materials used in mixing and in the seal and surface treatments were relatively low in comparison with those now generally considered suitable for these purposes. The initial viscosity greatly influences the amount of bituminous material that will be retained by the aggregate, and it is problematical whether additional bituminous material of as low viscosity as that originally used with the relatively open stone mixes would have produced a less rigid mat. It is probable that increasing the percentage of bituminous material would have resulted in a nonuniform mat with the bottom portion being excessively rich, unless the bituminous material was added in increments with manipulation and drying periods following each application. With such a procedure, however, there is considerable likelihood that excessive segregation of particles would occur.

The use of a higher viscosity material and a greater percentage of it might have provided a bituminous mat that would have been less susceptible to cracking or to raveling and that would have been more resistant to the effect of moisture. This would have been particularly beneficial for the sections constructed on the marl base. The service record and maintenance costs of the sections on the sand-clay base indicate that their design was satisfactory for the existing conditions.

One of the purposes of the experiment was to obtain information on the effect of the size and grading of the aggregate. Information was desired on the value of relatively dense-graded aggregate as compared with a more open grading and also upon the merits of various maximum-size aggregates. As shown in the report, aggregates graded from 1¼ to ¼ inch and from ¾ to ¼ inch were used with and without finer material added. It was expected that where material from ¼ inch to dust was used in the mix the mat would be sufficiently dense as not to require a seal but that where such material was not used a seal would be required.

Approximately 20 percent of material from ¼ inch to dust was used with both the 1¼ to ¼ inch and the ¾- to ¼-inch aggregates. It was observed during construction that the resulting mixtures were harsh and apparently would have been benefited had the percentage of finer material been increased. This was especially noticeable where the maximum-size aggregate was 1¼ inches. Because wide differences of opinion still exist regarding the grading of aggregates and successful performance has been obtained with a wide variety of gradings, it could hardly be said that better results would have been obtained had the gradings been changed, especially since the service behavior gave no indication of failures resulting from improper grading.

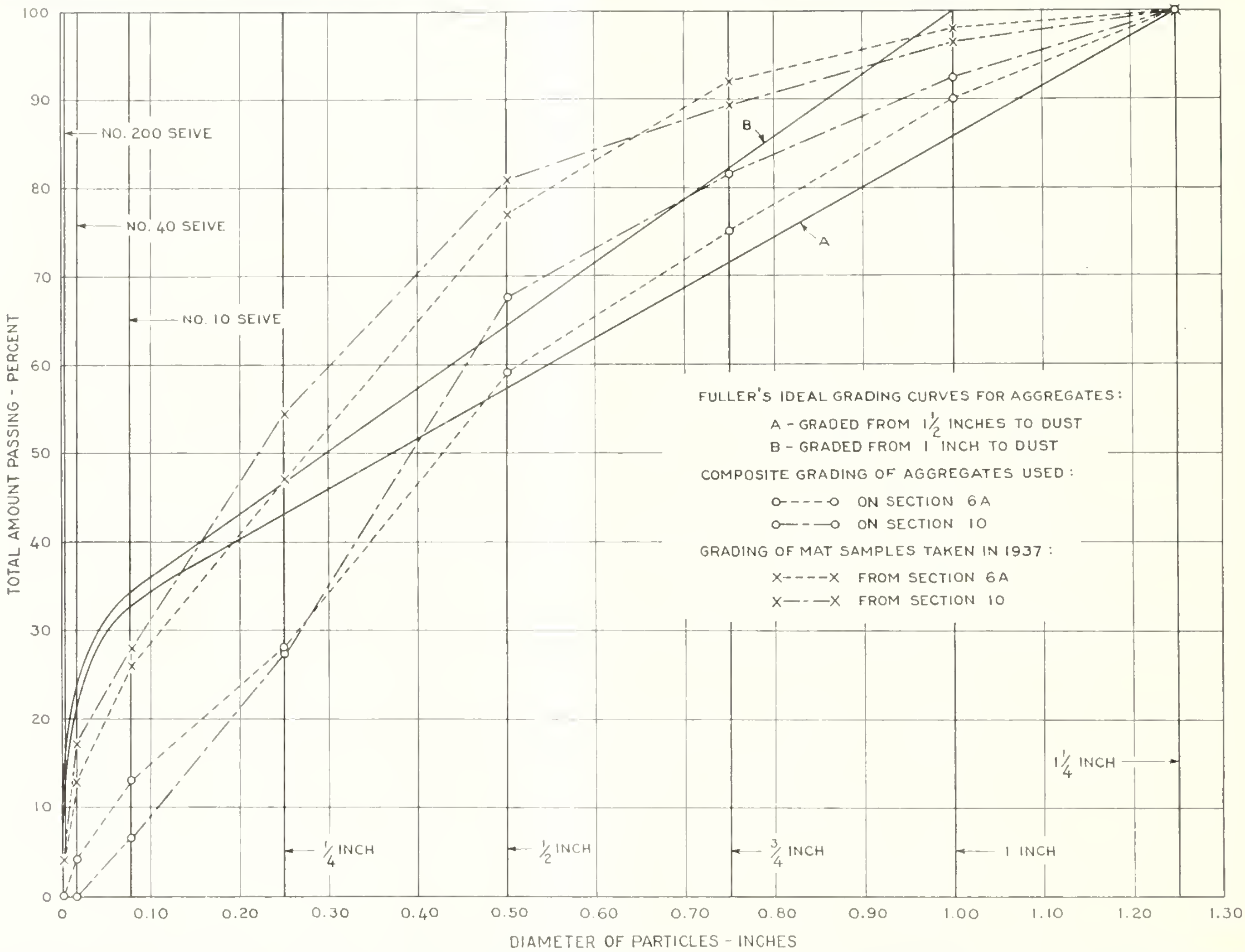

FIGURE 11.—COMPARISON OF ORIGINAL AND FINAL GRADINGS OF AGGREGATES USED IN EXPERIMENTS 6A AND 10 WITH FULLER'S IDEAL CURVE FOR MAXIMUM PRACTICAL DENSITY.

AGGREGATE GRADINGS COMPARED WITH FULLER'S CURVES

If it is assumed that a grading conforming substantially to that sometimes referred to as Fuller's Ideal Grading Curve for Maximum Practical Density [4] can be satisfactorily used as a basis of comparison, interesting information is developed by a study of the aggregates used and of the changes in grading that occurred in service. This discussion is limited to a consideration of those sections, the ultimate grading of whose aggregates was determined by extraction and mechanical analysis after 8 years of service. For convenience the gradings are plotted in figures 11 and 12. The original gradings are based upon the percentages and gradings of the aggregates used, and the final grading is the mechanical analysis of the aggregate extracted from mat samples taken in 1937. In the sample taken from experiment 6A the aggregate is the composite of the coarse and fine material in the mix and the ⅝- to ¼-inch aggregates used in the seal and in the 1933 re-treatment. Experiments 8 and 9 were not re-treated, consequently only the aggregate placed at the time of construction is involved.

The use of approximately 20 percent of the finer material, graded from ¼ inch to dust with the 1¼ to ¼-inch aggregate in experiments 1 and 6, produced a mixture that was apparently not dense enough to permit omitting the seal treatment. It was originally reported that "at least 30 percent of fines would have been necessary in order to produce a surface (density) similar to that obtained with (aggregate graded from) ¾ inch to dust." Considering the portion retained on the ¼-inch sieve as coarse and that passing it as fine material, it will be observed from the grading curve for experiment 6A in figure 11 that actually 28 percent of fine material was present although according to Fuller's curve A, 40 to 45 percent could have been used. Had 30 percent of the material graded from ¼ inch to dust been used as suggested in the original report, the resulting grading would have been in substantial agreement with that given by Fuller's curve.

The final grading of the aggregate as determined by an extraction test on a sample taken from experiment 6A in 1937 is shown in figure 11. As will be noted, the percentage passing any given sieve has greatly increased indicating that a considerable amount of crushing has occurred. Since practically all of the extracted aggregate passed the 1-inch sieve, Fuller's curve B, for 1-inch maximum-size aggregate, is more applicable for comparison. Therefore using curve B as a basis for comparison, it will be noted that as a result of the crushing that occurred the final grading has more nearly ap-

[4] Concrete Plain and Reinforced, by F. W. Taylor, S. E. Thompson, and E. Smulski. John Wiley & Sons, New York.

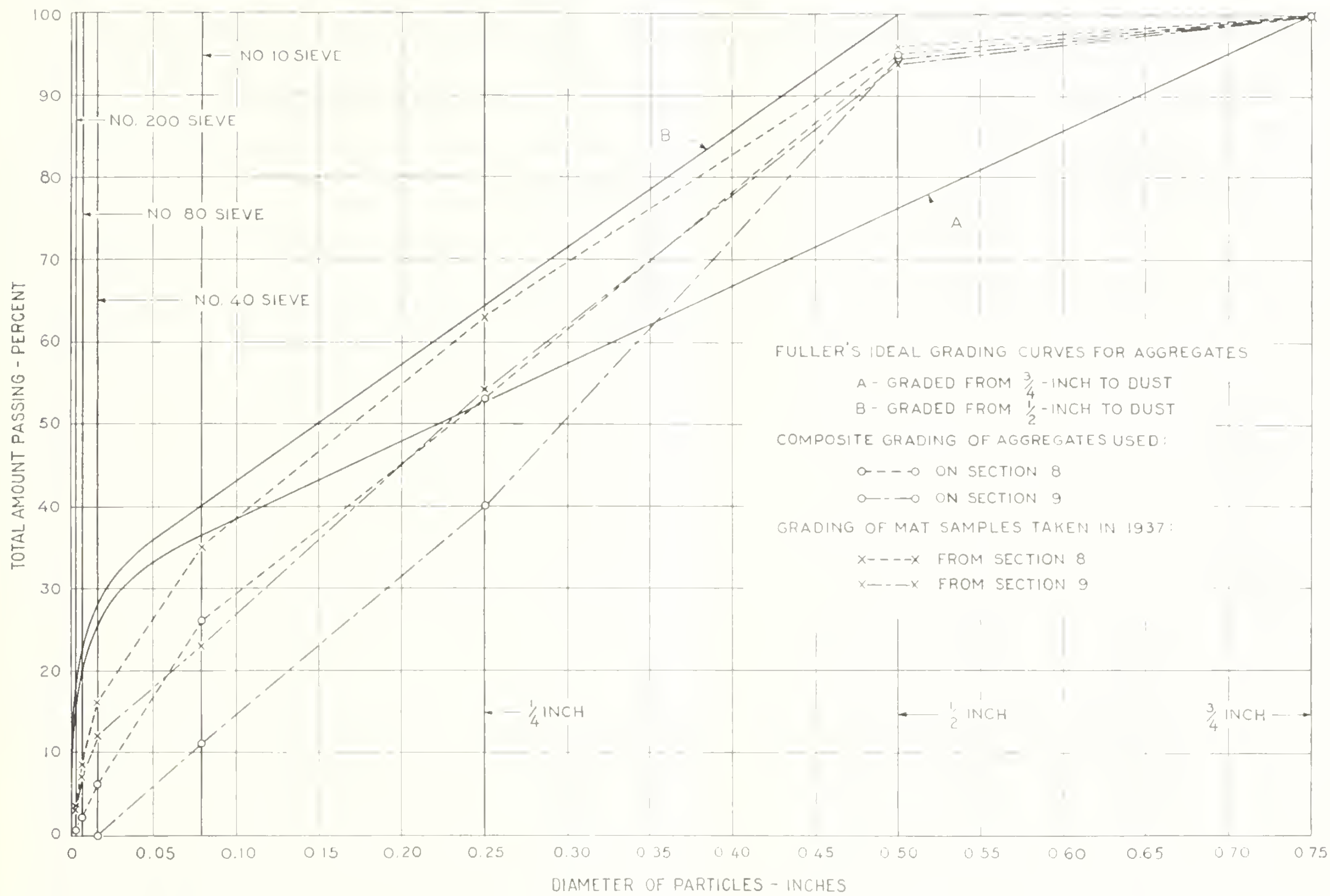

FIGURE 12.—COMPARISON OF ORIGINAL AND FINAL GRADING OF AGGREGATES USED IN EXPERIMENTS 8 AND 9 WITH FULLER'S IDEAL CURVE FOR MAXIMUM PRACTICAL DENSITY.

proached the maximum density curve especially for the ¼-inch and smaller sizes.

On experiments 3 and 8, where approximately 20 percent of the finer material was used with ¾- to ¼-inch stone, the mixture was not harsh and the resulting mat seemed sufficiently dense as not to require a seal. It appeared that the amount of finer material used with the smaller-sized coarse stone was much more nearly correct than that used with the 1¼- to ¼-inch stone and the grading curves in figure 12 verify this.

FINDINGS RELATIVE TO CHANGES IN GRADING SUMMARIZED

Although 20 percent of fine material was added to both the ¾- to ¼-inch and the 1¼- to ¼-inch stone, the grading of the ¾ inch to dust material actually contained approximately twice as much material passing the ¼-inch and No. 10 sieves as did the combined material graded from 1¼ inches to dust. However, since the smaller maximum-size material requires increased amounts of finer sizes for a given density and because the ¾ inch to dust material was in effect ½ inch to dust originally, it is apparent by reference to curve B in figure 12 that the percentages of material passing the ¼-inch and No. 10 sieves should have been 64 and 40, respectively, instead of 53 and 26 percent, the combination actually contained. Moreover, it is interesting to note that as a result of the crushing that later occurred, the final material actually contained 63 percent passing the ¼-inch sieve and 35 percent passing the No. 10 sieve.

Grading curves for the aggregate used and of that extracted from the mat in 1937 from experiment 9 also are shown in figure 12. On this section the ¾- to ¼-inch stone in the mixed mat contained no fine material, as surface density was to have been obtained by the seal treatment in which ⅝- to ¼-inch stone chips were used. It would not be expected that the original grading would conform to Fuller's curve and it will be noted that it was practically a straight-line grading from ½-inch to the No. 40 sieve. Its final grading, however, is very similar to Fuller's curve in form although it is deficient in all sizes smaller than ½ inch. The difference between the original and final gradings, which is a measure of the crushing that occurred, is greater on experiment 9 than on experiment 8. The increases in percentages passing the ¼-inch, No. 10 and No. 40 sieves are 14, 12, and 12, respectively, for experiment 9 while the corresponding percentages for experiment 8 are 10, 9, and 10. Some such difference might be expected in view of the fact that the graded mixture in experiment 8 was denser initially and therefore less susceptible to crushing.

Corresponding curves for the aggregates used on and extracted from the surface of experiment 10, which was a surface-treated section, are shown in figure 11. The aggregate used originally contained approximately the same percentage of material passing the ¼-inch sieve as did the combined aggregate of experiment 6A but substantially lesser amounts of material passing the No. 10 and No. 40 sieves. After 8 years in service, however, the percentages passing the ¼-inch, No. 10

and No. 40 sieves had increased to 54, 28, and 17 percent, respectively, as compared with the corresponding sizes of the material in experiment 6A which were 47, 26, and 13, respectively. Here also, as on experiment 9, considerably more crushing occurred than on the section where the mat was more dense originally.

Summarizing the findings relative to the changes in grading that occurred, the following facts appear to have been established:

1. Regardless of maximum size of stone used, crushing was more pronounced on the open type of mat than where greater density was provided initially.

2. On both open and closed types of mats, crushing was less where the smaller maximum-size aggregate was used.

3. For all gradings and on both open and closed types, crushing tended to produce increased density and the resulting grading approached that of Fuller's curve for maximum practical density.

Visual examination of the mat samples taken after 8 years in service showed them to be quite similar in appearance. All of them were hard and dense and well bonded despite the fact that the surface area of the aggregate had increased greatly because of the crushing that had occurred. The mats were so similar in appearance as to make detection of the method of construction used impossible.

Considering the character of the mats and the final gradings attained, it appears that the size and grading of the friable granite aggregate used on this project was of no great importance. It might even be inferred that crushing not only was not detrimental but was beneficial in providing greater density and stability than could have been obtained otherwise. As crushing occurred there was a corresponding increase in density and a reduction in voids. The low percentage of bituminous material used evidently became sufficient as the particles were brought into closer contact under the action of traffic. Had crushing not occurred it is quite likely that raveling would have been more pronounced.

With an aggregate so susceptible to crushing, it is quite possible that if a greater percentage of bituminous material had been used originally the mats might eventually have become too rich when greater density was obtained. This may explain the frequently observed tendency of bituminous mats to develop rich or fat spots after a considerable period of satisfactory behavior.

The road-mix and inverted penetration methods used on this work are in common use at the present time and are proving very satisfactory where the materials used have been properly selected. Direct penetration methods are also being used satisfactorily but the particular penetration method used in constructing experiments 2C and 7C would not now be considered good practice. Some unsatisfactory areas developed on sections constructed by each of the three methods but factors other than the construction method used were responsible for their unsatisfactory behavior, except that, as previously noted, the penetration method used on experiments 2C and 7C is believed to be at least partially responsible for their behavior.

CONSTRUCTION AND MAINTENANCE COSTS GIVEN FOR EACH SECTION

For the road-mix type of construction, densely graded aggregates, especially those containing appreciable amounts of material passing the number 200 sieve, are not generally used with rapid-curing materials. On this experimental road no difficulty was encountered in obtaining mixtures of uniformly coated aggregate either with the 1¼ inches to dust aggregate or with that graded from ¾ inch to dust. Although the air temperatures were relatively high, the loss of the volatile portion of the bituminous material was not sufficient to interfere with the manipulation and placing of the mixtures. The character of the distillate used in the cut-backs was such as to provide a material more nearly resembling the medium-curing type of cut-back; consequently, it is likely that a greater amount of fines could have been used without greatly increasing the work of mixing.

For a number of years the inverted penetration or surface-treatment method has been used satisfactorily on many miles of construction where a relatively thin mat was deemed adequate. Its low initial cost, ease of maintenance, and the fact that it serves excellently in stage construction show its economy and adaptability. Reference to the views of experiment 10 in figure 10, and to its analysis in table 8, show that the seal and re-treatment, together with the crushing of the aggregate that occurred in service, eventually produced a mat that was very similar to those originally obtained by the road-mix method. Although the liquid bituminous material was satisfactory in the surface-treated sections of this road, the use of a more viscous material is more generally favored in constructing surface-treated roads. Such material offers better protection against moisture, and, as it becomes very viscous almost immediately upon application, the cover stone is readily held in place.

For convenience of comparison, a summary of the cost data is given in table 10. It will be observed that little relation exists between the costs of construction and of maintenance. All of the sections on the sand-clay base were more economical to maintain than were the corresponding sections built on the marl base. Moreover, it is interesting to note that, in the order of their cost of maintenance, the corresponding sections on the two bases are almost identical, which fact might indicate that the character of the base was not the sole cause of the difference but that the type of structure had some effect on the maintenance cost.

Experiment 10 has been the most economical in total cost in spite of its annual maintenance cost of 2.08 cents. Experiments 6A, 6B, 8, and 9 have been more economical to maintain but their total costs are greater than that of experiment 10. The difference in maintenance cost between experiments 8 and 10 is sufficient to make experiment 8 the more economical after a period of 8 years providing, of course, that future maintenance costs continue in the same proportion as in the past. No other section would approach experiment 10 in economy within any reasonable period of time.

Based on their behavior, it is reasonable to expect that the sections on the sand-clay base will continue to give satisfactory service with little increase in maintenance cost under the conditions now existing. Moreover, the present structural soundness of the base and mat are apparently such that improvements to meet increased traffic demands could probably be made without sacrificing the present investment.

The same situation does not exist for the sections on the marl base, with the exception possibly of experiments 1A, part of 1B, 3, and 4 where good drainage exists. It is improbable that the marl bases on this road will ever be better than in the past, and there is little reason to expect that any bituminous surface placed on it will be satisfactory for any considerable period. Stabilization of the marl base to reduce its adverse reaction to moisture would be necessary in order to provide a foundation comparable with that provided at present by the sand-clay material.

TABLE 10.—*Summary of cost data*

SECTIONS ON MARL BASE

Section	Costs in cents per square yard				Order of maintenance cost
	Construction	Maintenance	Total to July 1, 1937	Average annual maintenance	
1A	66.12	25.75	91.87	3.32	6
1B	72.58	42.29	114.87	5.46	4
2A	70.34	48.73	119.07	6.29	3
2B	79.40	55.42	134.82	7.15	2
2C	84.62	103.86	188.48	13.40	1
3	61.07	17.39	78.46	2.24	8
4	55.46	23.69	79.15	3.06	7
5	28.47	39.77	68.24	5.13	5
Average	64.77			5.81	

SECTIONS ON SAND-CLAY BASE

Section	Construction	Maintenance	Total to July 1, 1937	Average annual maintenance	Order of maintenance cost
6A	72.53	14.11	86.64	1.82	6
6B	72.10	15.12	87.22	1.95	5
7A	62.86	20.29	83.15	2.62	3
7B	73.23	26.91	100.14	3.47	2
7C	84.55	44.19	128.74	5.70	1
8	59.53	1.26	60.79	.16	8
9	66.74	2.92	69.66	.38	7
10	29.44	16.14	45.58	2.08	4
Average	58.10			2.28	

CONCLUSIONS

The record obtained and the observations made on this road during the period covered by this report appear to warrant the following conclusions:

1. The service behavior of the bituminous surfaces was affected more by the character of the base than by the types of surfaces or the materials used in them.
2. The suitability of marl and of sand-clay as base materials was not indicated by their apparent similarity to other like materials or by their behavior before the bituminous surfaces were applied.
3. Definite knowledge of the characteristics that affect the service behavior of soils would have made possible the use of local materials to the best advantage and would have eliminated the likelihood of importing material that was inferior to that already at hand.
4. The present method of soil analysis and classification provides reliable information on the characteristics of soils and on their probable service behavior under given conditions.
5. Satisfactory bituminous surfaces can be constructed by various methods but the materials used should be suited to the method selected.
6. In the design of mixtures, consideration should be given to the possibility of the aggregate crushing under traffic, in order that the increase in density will not result in the voids being over-filled with bituminous material.
7. The construction of an adequate base will greatly reduce maintenance costs and will make possible the construction of a relatively thin and economical surface.
8. The construction of a thin mat that is satisfactory for current needs is most economical, providing adequate base support is provided originally, and such a surface can be strengthened to meet increased traffic demands.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF MARCH 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 6,865,382	$ 3,146,870	239.5	$ 7,605,982	$ 3,792,846	278.1	$ 915,180	$ 454,285	35.6	$ 3,430,460
Arizona	2,046,022	1,533,122	109.0	1,477,764	1,029,914	57.9	208,563	147,876	8.1	2,018,055
Arkansas	1,150,199	1,135,954	81.3	3,699,891	3,695,817	231.5	296,465	293,960	15.8	1,868,439
California	9,837,151	5,364,140	230.3	5,253,068	2,829,772	77.9	1,371,105	729,650	13.6	4,736,621
Colorado	2,565,088	1,368,707	99.3	2,734,853	1,456,727	85.0	1,531,850	851,780	31.5	2,862,326
Connecticut	934,030	455,835	8.9	684,518	337,455	7.9	470,630	233,815	5.7	1,915,211
Delaware	485,437	241,521	14.1	708,131	349,289	9.9	393,608	188,365	6.6	1,562,442
Florida	2,648,707	1,323,880	59.5	2,935,786	1,467,893	58.5	282,400	141,200	6.3	3,781,841
Georgia	5,142,252	2,472,651	254.8	5,005,390	2,502,695	254.1	1,705,150	852,585	98.9	7,076,705
Idaho	2,099,436	1,208,877	200.7	1,189,538	710,407	38.9	714,215	432,431	18.2	1,916,121
Illinois	11,503,316	5,703,172	307.3	7,550,526	3,771,709	164.3	3,142,509	1,571,209	77.4	4,698,237
Indiana	6,098,345	2,989,340	159.6	3,430,014	1,715,007	66.3	2,645,753	1,218,953	52.6	3,644,710
Iowa	7,846,834	3,664,762	263.5	4,426,917	1,826,033	141.5	504,669	147,600	34.5	2,635,539
Kansas	5,167,966	2,571,924	724.7	4,062,913	2,031,456	175.7	4,054,918	2,019,754	215.1	4,253,088
Kentucky	5,585,639	2,771,091	209.3	2,743,030	1,371,515	59.0	2,010,982	1,005,689	51.1	3,442,176
Louisiana	1,446,064	718,483	38.2	10,983,479	2,576,110	31.7	1,415,560	614,921	27.1	3,258,300
Maine	2,794,504	1,356,812	65.5	1,710,959	854,059	33.0	126,180	63,090	3.2	1,009,282
Maryland	1,085,456	542,728	17.1	2,467,978	1,222,851	40.7	822,470	397,000	11.2	2,338,785
Massachusetts	1,874,284	937,139	9.0	3,092,767	1,545,740	20.3	1,122,427	557,100	14.6	3,280,162
Michigan	7,875,485	3,752,043	166.1	4,200,328	2,099,512	120.6	1,397,095	688,621	32.2	3,689,518
Minnesota	4,862,727	2,351,927	301.6	5,854,665	2,904,563	267.4	1,137,912	568,051	61.2	4,697,786
Mississippi	4,966,478	2,079,413	210.8	7,963,242	3,027,236	355.3	1,486,800	594,000	43.6	3,289,271
Missouri	5,727,680	2,728,407	151.9	3,006,954	1,474,486	73.5	4,612,543	2,214,880	194.4	4,964,270
Montana	1,653,927	929,612	83.6	1,127,372	633,780	30.3	2,074,909	1,165,039	117.6	5,147,491
Nebraska	3,804,397	1,837,691	339.5	5,511,115	2,778,101	433.4	3,259,156	1,627,525	345.1	2,986,487
Nevada	1,407,318	1,180,891	168.8	1,728,043	1,491,468	61.0	127,111	109,181	2.6	1,601,772
New Hampshire	964,683	473,138	22.4	382,110	190,095	3.3	143,338	71,637	4.9	1,627,195
New Jersey	2,637,665	1,309,420	18.3	2,904,016	1,449,453	26.4	367,180	183,590	2.4	2,859,493
New Mexico	2,274,475	1,481,769	242.6	1,861,935	1,135,415	84.9	390,381	237,250	13.8	1,772,011
New York	14,165,418	6,789,694	253.2	10,758,427	5,294,689	168.9	2,519,000	1,186,100	50.4	5,083,103
North Carolina	6,732,813	3,171,622	259.1	5,354,259	2,676,252	358.1	1,701,440	816,830	74.7	3,102,386
North Dakota	3,442,748	3,261,988	261.5	427,380	237,794	57.5	69,522	37,236	6.8	5,124,010
Ohio	8,501,387	4,164,400	101.7	7,067,322	3,524,632	70.1	2,481,840	1,180,620	27.0	8,771,620
Oklahoma	6,925,221	3,634,955	247.6	1,782,024	941,272	54.9	1,487,800	791,645	46.7	4,509,927
Oregon	3,182,650	1,847,690	110.7	2,279,649	1,371,417	101.1	336,707	203,360	24.3	2,658,316
Pennsylvania	8,552,728	4,189,606	141.8	8,240,284	4,085,272	83.5	3,021,523	1,374,739	25.0	5,704,821
Rhode Island	1,179,290	589,645	16.4	390,482	195,241	3.5	63,560	31,780	.6	1,507,383
South Carolina	5,361,442	2,369,848	266.5	2,860,644	1,276,376	86.2				2,494,519
South Dakota	2,016,762	1,128,306	246.1	4,562,779	2,523,300	441.5	339,070	187,490	27.8	4,197,395
Tennessee	5,464,890	2,701,644	176.7	3,688,369	1,844,906	70.6	855,100	427,550	25.5	5,255,382
Texas	12,810,955	6,329,343	831.5	14,654,135	7,236,356	674.5	2,222,925	1,078,405	165.2	8,384,100
Utah	1,103,182	751,269	107.2	2,156,413	1,530,240	73.5	315,490	225,715	10.2	1,451,399
Vermont	1,285,741	592,143	33.9	722,784	343,793	17.7	200,670	100,095	4.3	643,793
Virginia	6,032,458	3,006,178	211.3	3,055,916	1,524,398	84.1	945,342	472,086	18.6	2,046,566
Washington	4,034,336	2,070,107	99.8	2,782,766	1,456,550	35.7	438,711	228,400	2.9	2,045,645
West Virginia	1,865,812	1,320,896	66.7	1,545,172	773,511	36.8	367,170	183,585	13.1	3,097,108
Wisconsin	5,069,889	2,502,304	176.2	6,981,719	3,272,880	183.4	81,482	37,000	.1	3,507,281
Wyoming	2,543,348	1,537,940	281.4	1,005,002	617,501	96.0	344,330	191,450	23.4	1,380,886
District of Columbia										487,500
Hawaii	809,490	396,078	18.0	856,440	419,895	9.4	484,577	239,928	8.9	1,462,820
Puerto Rico	189,737	92,320	4.4	1,591,031	790,985	32.4	568,259	281,430	10.8	557,140
TOTALS	214,621,244	110,079,295	8,708.9	189,066,281	94,208,664	6,027.7	57,575,577	28,656,481	2,111.2	165,836,894

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF MARCH 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 247,610	$ 247,411	6		4	$ 1,229,179	$ 1,227,324	15	1	1	$ 8,700	$ 8,700			2	$ 895,106
Arizona						231,874	229,670	3								526,004
Arkansas	405,664	403,606	9			296,955	296,675	6			104,053	104,053	2			1,298,166
California	1,086,722	1,086,147	4	2		1,652,737	1,652,137	8	1		444,118	443,623	4			1,247,158
Colorado	32,475	29,087	1		2	266,218	266,218	4			344,269	340,508	2		19	903,714
Connecticut						18,930	12,665									998,900
Delaware	39,000	39,000			13	47,420	47,420	2		10						504,830
Florida	10,616	10,616				434,894	434,894	3			79,700	79,700	1		1	1,158,058
Georgia						436,400	436,400	7			103,670	103,670	2	1	3	2,354,600
Idaho	174,973	174,800	4			280,682	249,386	4			131,898	120,500	1			439,963
Illinois	369,500	369,500	2		1	2,260,325	2,260,325	16	2	63	943,740	875,740	5	1	16	2,729,861
Indiana	690,434	597,473	3	5		895,391	868,491	3	1	90	164,640	164,640		1	8	1,298,044
Iowa	1,037,085	1,003,364	10	2	20	204,947	192,900	3			209,134	174,706	9		1	1,742,059
Kansas	532,239	532,134	8		4	959,403	959,403	12		1	157,405	157,405	2		8	1,430,525
Kentucky	145,000	145,000	1			449,315	449,315	5	1	31	357,864	357,864	5	3		1,218,712
Louisiana	11,980	11,980				435,221	428,478	4			429,826	398,090	10			1,049,379
Maine	48,590	48,590	2			332,396	332,396	3	2		147,150	147,150	1	1	1	298,258
Maryland						72,188	72,188	1			18,200	18,200			4	1,137,108
Massachusetts	54,710	54,710		1		296,258	295,577	1	2		282,055	281,205	1	2		1,690,082
Michigan	930,783	924,372	8	1	35	588,806	588,806	5	2	10	181,900	181,900	1			2,239,254
Minnesota	39,556	39,556		1	1	760,185	759,864	3	5		17,059	17,059			3	2,154,213
Mississippi	253,500	253,500	3			538,860	538,860	6	1		145,460	145,460	2			956,981
Missouri	297,091	295,552	4	1		447,800	447,800	2	1		1,026,220	959,930	4	2		1,918,155
Montana	355,586	350,704	4			634,520	634,520	6			235,773	235,773	3			364,726
Nebraska	147,408	147,319	4			757,788	757,788	16			618,287	618,287	16		33	588,705
Nevada	153,254	153,254		3	3	199,098	199,098	1	1		27,858	27,858			6	176,363
New Hampshire	70,205	69,765		1	2	87,856	87,797	5		1						433,688
New Jersey	116,891	111,665	1	1		229,856	229,856	1	1	2	373,470	351,050	1	1		1,667,300
New Mexico	168,984	168,984	4	1		118,994	118,994	3			87,240	87,240	2		1	643,230
New York	992,501	991,800	4	3	2	1,798,751	1,790,301	5	8		362,704	361,554		3	1	4,962,223
North Carolina	121,550	121,550	1	1	1	1,308,960	1,273,860	8	7		330,660	330,660	2		37	1,219,301
North Dakota	209,450	208,387		1		639,692	591,290	4	1							1,088,708
Ohio						539,740	539,740	6	1		412,640	371,640	5		2	4,151,641
Oklahoma	40,774	30,792			4	272,865	238,865	2		28	89,870	89,870			23	2,377,023
Oregon	308,391	307,742	1	2		384,601	250,242	1			167,455	167,455	2			484,121
Pennsylvania	213,129	197,923	2			1,301,403	1,089,504	2	1		690,222	690,222	2	1		4,883,502
Rhode Island						438,791	438,791	1	3							152,459
South Carolina	47,906	47,456	1	1		335,743	281,227	14	1	2	503,729	503,729	4	1	38	950,440
South Dakota	129,150	128,517	2		9	271,930	271,930	3	2		38,490	38,490	2		7	1,149,978
Tennessee	2,660	2,660			1	323,510	323,510		3	1	486,890	486,890	3		7	1,452,160
Texas	226,560	225,627	6	2		1,920,887	1,889,692	20	4		1,102,061	1,035,830	11	1		3,280,829
Utah	101,648	101,648	2		4	47,359	47,359	2			201,460	201,460			77	352,070
Vermont	237,610	226,469	6	2	6	10,176	10,176			1	25,490	25,300			8	316,007
Virginia	329,409	328,336	13	1		603,632	514,632	9	2	1	362,737	362,737	3	1	23	966,958
Washington	247,816	236,347	2	3	2	822,574	821,164	11	1		86,637	86,637		1	13	541,588
West Virginia	218,401	217,381	1		3	308,341	292,581	3			103,800	103,800	4		3	971,052
Wisconsin	202,131	200,987	3			1,186,812	1,145,988	11			18,119	18,119			4	1,594,489
Wyoming	164,037	164,037		2	4	10,150	10,150				214,610	135,190	1	1	7	499,487
District of Columbia						30,215	30,215		1		283,544	243,750	1			148,966
Hawaii						201,200	201,200	3	1		29,220	29,220	2			360,830
Puerto Rico	24,930	24,780	1			189,639	188,590	3			69,730	69,160	4			597,846
TOTALS	11,237,909	11,030,528	123	37	121	28,111,507	27,316,252	256	57	242	12,219,757	11,752,024	120	21	356	66,564,820

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF MARCH 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 234,900	$ 117,450	18.4	$ 834,850	$ 412,050	38.6				$ 833,746
Arizona	389,951	252,338	25.4	131,384	92,415	16.9	$ 65,551	$ 47,266	.1	477,973
Arkansas	13,126	6,563		309,841	307,495	32.8	252,870	252,096	35.7	553,341
California	1,507,713	850,893	104.6	984,513	511,412	38.8	372,927	194,930	11.4	859,674
Colorado	871,019	456,096	52.1	402,820	223,165	18.3	220,370	113,009	7.8	400,486
Connecticut	69,450	34,705	1.3	46,934	23,267	.2	196,130	68,495	2.9	285,414
Delaware	18,950	9,475	4.0	50,830	25,415	10.0	35,110	17,555	8.7	267,555
Florida	20,122	10,061		516,233	257,300	12.1	275,700	137,850	14.2	469,894
Georgia	374,181	176,800	50.6	579,226	289,613	71.6	170,780	85,390	23.4	1,083,775
Idaho	453,367	203,954	46.9	166,441	87,870	11.9	24,812	14,825	1.1	351,190
Illinois	1,667,341	829,836	146.2	1,381,632	636,816	70.4	412,500	197,750	29.4	970,920
Indiana	663,404	277,092	75.8	752,100	367,050	60.6	584,947	270,708	50.0	691,475
Iowa										1,679,807
Kansas	163,626	81,812	14.8	119,236	59,618	10.1	407,036	203,518	39.8	1,361,899
Kentucky	791,832	245,084	106.1	701,106	181,576	23.0	899,303	254,970	96.2	410,817
Louisiana	75,038	37,385	6.9	727,208	313,890	57.1	361,231	167,160	31.8	421,213
Maine	356,142	176,677	23.3	2[illegible],662	126,214	12.5	27,500	13,750	2.1	148,402
Maryland				137,974	68,987	11.2	197,900	74,855	14.2	391,839
Massachusetts				149,794	74,781	2.4	285,020	141,680	5.8	636,262
Michigan	409,561	203,281	37.0	834,004	417,002	49.7	589,700	285,150	28.6	1,082,055
Minnesota	280,898	131,160	42.2	602,574	299,243	43.0	204,248	102,124	19.5	1,252,678
Mississippi				299,000	149,500	23.8	44,700	22,350	17.0	979,016
Missouri	419,599	202,330	53.0	468,200	212,720	42.6	571,290	259,650	91.9	838,936
Montana				27,601	15,525		108,276	61,272	10.8	1,263,096
Nebraska	514,620	254,044	86.8	569,244	277,801	100.1	333,810	164,030	64.3	608,583
Nevada	425,929	347,472	68.8	120,169	104,184	15.5	26,563	23,035	1.6	212,555
New Hampshire	223,514	110,923	6.0	60,759	29,708	2.3				179,369
New Jersey	123,040	61,520	2.4	199,860	91,195	2.7	240,733	120,085	7.4	598,178
New Mexico	643,196	392,281	42.1	539,108	328,795	35.8	104,195	60,990	5.6	264,000
New York	2,309,275	1,125,396	166.3	1,899,000	949,500	99.6				1,014,487
North Carolina	699,564	349,170	77.3	904,564	452,260	80.1	180,740	77,950	20.9	551,003
North Dakota	51,622	27,362	9.0	169,910	90,999	26.1	42,770	22,907	8.2	875,809
Ohio	147,535	73,767	3.8	184,690	99,120	7.0	463,440	231,720	26.8	1,964,512
Oklahoma	302,203	160,942	35.8	158,054	84,098	7.1	602,040	297,148	32.4	990,316
Oregon	453,217	263,260	58.5	112,895	68,402	16.1	428,567	257,210	45.2	414,485
Pennsylvania	1,706,116	811,798	123.1	1,789,367	876,902	97.5	638,704	319,352	32.4	742,892
Rhode Island	66,840	33,420	3.5	194,923	97,438	5.8	74,070	37,035	.9	93,123
South Carolina	404,550	174,382	43.5	834,787	349,369	90.5	190,290	75,500	14.4	278,661
South Dakota	11,519	6,250								1,058,050
Tennessee	259,120	129,560	14.8	680,124	267,162	29.7	29,600	14,800	.3	959,938
Texas	2,877,833	1,301,941	398.4	2,115,674	1,008,898	225.1	793,300	355,476	100.7	1,392,517
Utah	450,730	230,606	41.1	335,512	170,870	27.0	98,150	53,444	13.5	264,160
Vermont	238,385	109,790	13.8	90,306	45,153	4.0	43,300	20,500	.5	107,278
Virginia	571,647	246,135	61.5	810,552	392,809	65.4	150,970	75,485	14.4	450,299
Washington	549,807	286,426	63.7	656,998	345,296	39.0	100,414	52,700	3.3	280,706
West Virginia	247,154	122,025	21.4	153,296	76,648	8.3				513,306
Wisconsin	557,666	265,848	23.1	656,279	322,660	32.0	146,298	69,640	2.5	916,888
Wyoming	416,281	254,565	59.0	321,002	198,349	15.8	85,578	52,861	6.5	266,775
District of Columbia										73,125
Hawaii				68,130	34,065	2.4	124,850	62,425	3.5	223,510
Puerto Rico	224,621	110,876	11.3	131,605	64,530	8.8	55,188	27,140	2.1	117,454
TOTALS	23,256,204	11,552,751	2,243.6	24,242,971	11,979,135	1,701.3	11,261,471	5,457,786	949.8	33,123,442

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PUBLIC ROADS

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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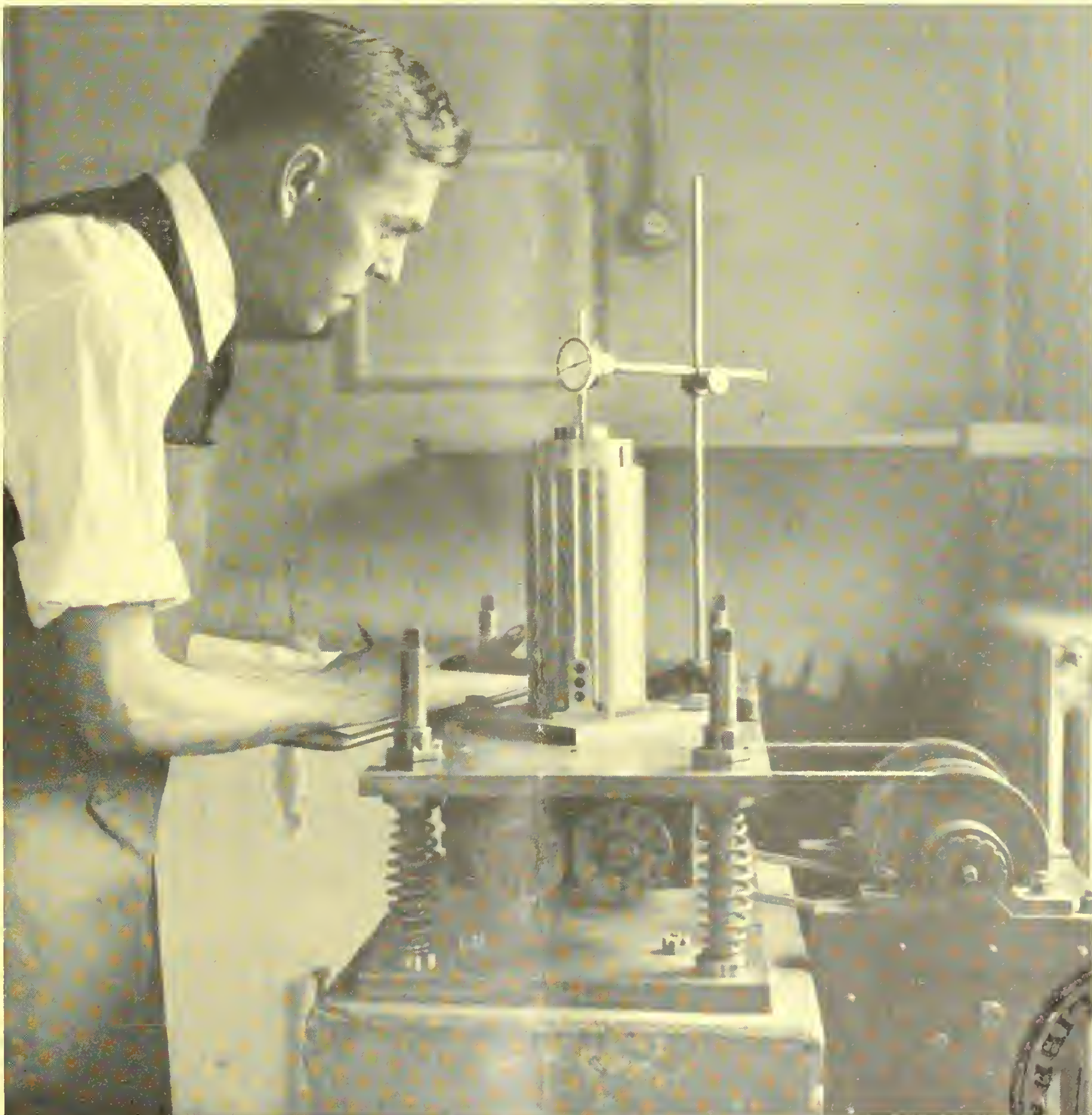

VIBRATORY MACHINE FOR DETERMINING THE COMPACTIBILITY OF AGGREGATES

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Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

D. M. BEACH, *Editor*

Volume 20, No. 3 May 1939

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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PRELIMINARY RESULTS OF ROAD-USE STUDIES

BY DIVISION OF CONTROL, BUREAU OF PUBLIC ROADS

Reported by ROBERT H. PADDOCK and ROE P. RODGERS, Associate Highway Engineer Economists

MOTOR VEHICLES annually travel approximately 250 billion vehicle-miles over the streets and highways of the United States. The benefits derived from such travel may be considered one measure of the returns received on the large capital investment in highway facilities. To finance the facilities necessary for the effective handling of such a great volume of travel, a large portion of the needed revenues is collected from vehicles owners largely on the basis of motor-vehicle use. However, highways also furnish other benefits than those received directly by motorists, and highway-user revenues are supplemented to a limited extent by revenues from other sources.

In planning highway programs an important problem is determination of where highway-user revenues should be spent to benefit the greatest number of motorists and to provide for the most essential needs on all classes of roads and streets. It is evident that a properly considered highway program should be intended, insofar as possible, to provide facilities for various classes of motorists so that the maximum benefits to the public will be realized.

Determinations of the type and extent of highway use as obtained from road-use studies will assist in formulating such a program. These studies, which are integral parts of the current State-wide highway planning surveys under way in 46 States, will provide factual bases for answering important questions concerning the nature of highway traffic. They will make possible an understanding of the relationships between highway expenditures and the travel of those who pay a large share of the transportation bill. The studies will also show the variations between the motoring habits of rural and urban residents and between the traffic of different types of motor vehicles.

The data presented in this summary are presented without the complementary material which will be available from other phases of the planning surveys, and which are essential in formulating integrated highway-development programs. However, a study of road-use data will assist in an understanding of highway-transportation problems.

ANALYSIS MADE OF INTERVIEWS FROM 17 REPRESENTATIVE STATES

Road-use information was obtained by means of a large number of personal interviews with motor-vehicle owners and drivers. These interviews were carefully selected to insure a proper representation of each geographical division of a State, of each group of governmental jurisdictions within similar population ranges, of various occupations, and of vehicles according to types and ages in operation. Information obtained from vehicle owners by survey interviewers made it possible to determine the extent of the owners' travel during the preceding year, and the routes of such travel for each trip. Experience has demonstrated that the year's driving of an individual can be accounted for reliably because of the numerous habitual trips, frequent local recreational trips, and unusual long trips that can be easily recalled.

By summarizing the data and expanding to the total State registration for each vehicle type—taking into account all known factors affecting the amount and kind of driving—information is obtained from which it is possible to estimate—

1. The total amount of travel on the various highway systems in a given area or in the State, and
2. The amount of travel performed on the various highway systems in the State by vehicle owners residing in the several governmental jurisdictions.

The two special analyses presented in this report are largely based upon preliminary road-use data obtained in the 17 States of Colorado, Florida, Iowa, Louisiana, Michigan, Minnesota, Missouri, Montana, New York, Ohio, Oklahoma, Oregon, Pennsylvania, South Dakota, Utah, Washington, and Wisconsin. Interviews covering a total of 198,809 passenger cars and 71,941 trucks were taken in these States during different periods, most of them during 1936, but some at an earlier date. All figures were adjusted to the year 1936 in proportion to the motor-vehicle registrations for the particular years under consideration. The 17-State sample was then expanded to obtain figures representing the entire United States by applying road-use data for a particular State to those surrounding or similar States for which data were not yet available.

Factors taken into consideration in these calculations included motor-vehicle registrations, the distribution of population by population groups (see table 1), motor-vehicle ownership per capita for various population groups, and existing mileages of the several highway systems in each State. A number of additional corrective factors were omitted in this preliminary analysis, but it is believed that the results are reliable.

The 17 States which formed the basis for this analysis represent:

> 45.4 percent of the estimated population of the United States in 1936.
> 47.8 percent of motor-vehicle registrations in the United States in 1936.
> 41.0 percent of the State primary road mileage in the United States in 1936.

Results of road-use studies indicate that these States were responsible for:

> 46.3 percent of estimated travel on all roads and streets in 1936.
> 44.9 percent of estimated travel on State-administered highways in 1936.

The close agreement of these figures indicates that for the purpose of this study, the 17 States were representative of the country as a whole.

That the estimate of total annual travel amounting to almost 250 billion vehicle-miles for all passenger cars, trucks, and busses in the United States is reasonable, can be demonstrated by comparison with the total

TABLE 1.—*Approximate distribution of population and of motor-vehicle registration in the United States in 1936 by population groups of residence*

Population group	Population[1]		Motor-vehicle registration[2]	
	Number	*Percent*	*Number*	*Percent*
Unincorporated areas	44, 636, 770	36. 4	8, 617, 876	30. 6
Incorporated places having a population of—				
1,000 or less	4, 362, 746	3. 6	1, 491, 044	5. 3
1,001 to 2,500	4, 820, 707	3. 9	1, 544, 370	5. 5
2,501 to 10,000	10, 614, 746	8. 6	3, 061, 979	10. 9
10,001 to 25,000	9, 097, 200	7. 4	2, 444, 929	8. 7
25,001 to 100,000	12, 917, 141	10. 5	3, 419, 713	12. 1
100,001 or more	36, 325, 736	29. 6	7, 585, 639	26. 9
Total	122, 775, 046	100. 0	28, 165, 550	100. 0

[1] Population data from 1930 census. Total midyear population for 1936 estimated by United States Census Bureau at 128,429,000.
[2] Includes passenger cars, trucks, and busses.

quantity of gasoline consumed in street and highway travel. The total estimated travel of 249,778,990,000 vehicle-miles in 1936, divided by the 17,855,454,000 gallons of gasoline used on highways in 1936, gives an average of 14 miles per gallon for all types of motor vehicles. This result is in close agreement with other estimates of average gasoline consumption per vehicle made in recent years. Both this figure and the estimated average annual travel of 8,870 miles for all types of motor vehicles also compare favorably with similar values determined from other highway planning survey data in a number of States.

OVER HALF OF ALL TRAVEL PERFORMED ON PRIMARY STATE HIGHWAYS

The highway systems over which total travel was distributed are classified as (1) primary rural highways and transcity connections, (2) secondary highways and local rural roads, and (3) city streets. Primary rural highways under State control consisted of 339,000 miles which, with the urban extensions and connecting links through municipalities of 20,000 miles, totaled 359,000 miles in the United States in 1936.

The secondary and local rural road classification includes 178,000 miles of rural highways under State control other than primary State highways, as well as an estimated 2,440,000 miles of county and township roads or a total for this classification of 2,618,000 miles for the United States.

City street mileage comprised 215,000 miles, of which 20,000 miles was urban extensions and connecting links of the primary systems and 195,000 miles was the estimated total of other streets in all incorporated places in 1936.

In determining the distribution of travel to these various systems all travel on streets of incorporated places incurred in going to or coming from rural portions of the primary highway system was summarized separately and for this particular presentation has been credited to the primary system. Similarly, travel on city streets incurred in going to or coming from rural portions of the secondary system was credited to the secondary system. Purely local city travel originating inside a municipality and not extending beyond the city limits was credited to the local street classification, even though some of that travel occurred on the urban extensions or connecting links of the primary system within the city.

Table 2 shows the distribution of estimated annual motor-vehicle travel in the United States in 1936 on the various highway systems, as performed by motor-vehicle owners resident in different population groups.

Table 3 shows for each population group of residence or vehicle ownership the percentage of total annual travel performed on each of the highway systems. The composition of the total annual motor-vehicle travel occurring on each highway system according to the various population groups in which the travel originated appears in table 4.

Average annual travel figures for each highway system by motor-vehicle owners resident in each population group appear in table 5.

TABLE 2.—*Estimated motor-vehicle travel on various highway systems in the United States in 1936* [1]

Travel by motor-vehicle owners resident in—	Total travel on—			
	Primary rural highways and transcity connections	Secondary highways and local rural roads	City streets	All systems
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*
Unincorporated areas	40, 846. 6	19, 453. 7	3, 333. 0	63, 633. 3
Incorporated places having a population of—				
1,000 or less	9, 869. 0	2, 942. 7	760. 4	13, 572. 1
1,001 to 2,500	10, 368. 9	2, 063. 6	1, 826. 8	14, 259. 3
2,501 to 10,000	19, 800. 8	2, 909. 3	6, 284. 5	28, 994. 6
10,001 to 25,000	15, 127. 6	1, 906. 8	6, 869. 8	23, 904. 2
25,001 to 100,000	18, 632. 8	2, 044. 5	12, 710. 2	33, 387. 5
100,001 or more	26, 328. 0	2, 113. 4	43, 586. 5	72, 027. 9
Total	140, 973. 7	33, 434. 0	75, 371. 2	249, 778. 9

[1] Based on preliminary data from road-use surveys in 17 representative States.

TABLE 3.—*Percentage of estimated motor-vehicle travel on the various highway systems in the United States in 1936*

Travel by motor-vehicle owners resident in—	Total travel on—			
	Primary rural highways and transcity connections	Secondary highways and local rural roads	City streets	All systems
	Percent	*Percent*	*Percent*	*Percent*
Unincorporated areas	64. 2	30. 6	5. 2	100. 0
Incorporated places having a population of—				
1,000 or less	72. 7	21. 7	5. 6	100. 0
1,001 to 2,500	72. 7	14. 5	12. 8	100. 0
2,501 to 10,000	68. 3	10. 0	21. 7	100. 0
10,001 to 25,000	63. 3	8. 0	28. 7	100. 0
25,001 to 100,000	55. 8	6. 2	38. 0	100. 0
100,001 or more	36. 6	2. 9	60. 5	100. 0
Total	56. 4	13. 4	30. 2	100 0

TABLE 4.—*Percentage of estimated motor-vehicle travel on each highway system by population groups of residence in which travel originated in the United States in 1936*

Travel by motor-vehicle owners resident in—	Total travel on—			
	Primary rural highways and transcity connections	Secondary highways and local rural roads	City streets	All systems
	Percent	*Percent*	*Percent*	*Percent*
Unincorporated areas	29. 0	58. 2	4. 4	25. 5
Incorporated places having a population of—				
1,000 or less	7. 0	8. 8	1 0	5. 4
1,001 to 2,500	7. 4	6. 2	2. 4	5. 7
2,501 to 10,000	14. 0	8. 7	8. 3	11. 6
10,001 to 25,000	10. 7	5. 7	9. 1	9. 6
25,001 to 100,000	13. 2	6. 1	16. 9	13. 4
100,001 or more	18. 7	6. 3	57. 9	28. 8
Total	100. 0	100. 0	100. 0	100. 0

The data presented in tables 2 and 3 indicate that of the nearly 250 billion vehicle-miles traveled in 1936 by passenger cars, trucks, and busses in the United States, 56.4 percent was travel on the primary rural highways and transcity connections, 13.4 percent on the secondary highways and local rural roads, and 30.2 percent on city streets. These figures may be more easily visualized by reference to table 5, which shows that the average motor vehicle traveled 8,870 miles during 1936, and that the division of this travel among the three classes of highways was 5,000, 1,190, and 2,680 miles, respectively.

TABLE 5.—*Estimated average travel per motor vehicle on the various highway systems of the United States in 1936*

Travel by motor-vehicle owners resident in—	Average travel on—			
	Primary rural highways and transcity connections	Secondary highways and local rural roads	City streets	All systems
	Vehicle-miles	*Vehicle-miles*	*Vehicle-miles*	*Vehicle-miles*
Unincorporated areas	4,740	2,250	390	7,380
Incorporated places having a population of—				
1,000 or less	6,620	1,970	510	9,100
1,001 to 2,500	6,710	1,340	1,180	9,230
2,501 to 10,000	6,470	950	2,050	9,470
10,001 to 25,000	6,190	780	2,810	9,780
25,001 to 100,000	5,450	600	3,710	9,760
100,001 or more	3,470	280	5,740	9,490
Total	5,000	1,190	2,680	8,870

Because the total average annual travel for motor vehicles registered in each population group was relatively uniform with the exception of those owned in unincorporated areas (table 5), the percentage of total annual travel on all highways and streets corresponded very closely to the percentage distribution of vehicle registrations within each population group. This fact is apparent from comparison of the figures in the last columns of tables 1 and 4.

MAJOR USE OF PRIMARY HIGHWAYS WAS BY CITY CAR OWNERS

There was considerable difference, however, in the relative use of the various highway systems by vehicles registered in the several population groups. These differences are indicated in tables 3 and 5. Vehicles owned in unincorporated areas performed 64.2 percent and 30.6 percent of their travel in 1936 on the primary highways and the secondary and local rural roads, respectively, and used city streets for only 5.2 percent of their total travel.

The use of the various highway systems by vehicles owned in the smaller incorporated places was somewhat similar to that for rural-owned vehicles. However, it is interesting to note the extent of the change in use of other highway systems with increase in the size of the place of vehicle ownership. Vehicles owned in the group of smallest incorporated places used the primary highways and the secondary and local rural roads for 72.7 percent and 21.7 percent, respectively, of their total annual driving, while vehicles owned in cities having populations over 100,000 used these same systems to the extent of 36.6 percent and 2.9 percent, respectively. Vehicles owned in the smallest incorporated places were used on city streets for only 5.6 percent of their total annual travel, but those owned in the largest cities performed 60.5 percent of their annual travel on streets of incorporated places. (See table 3.)

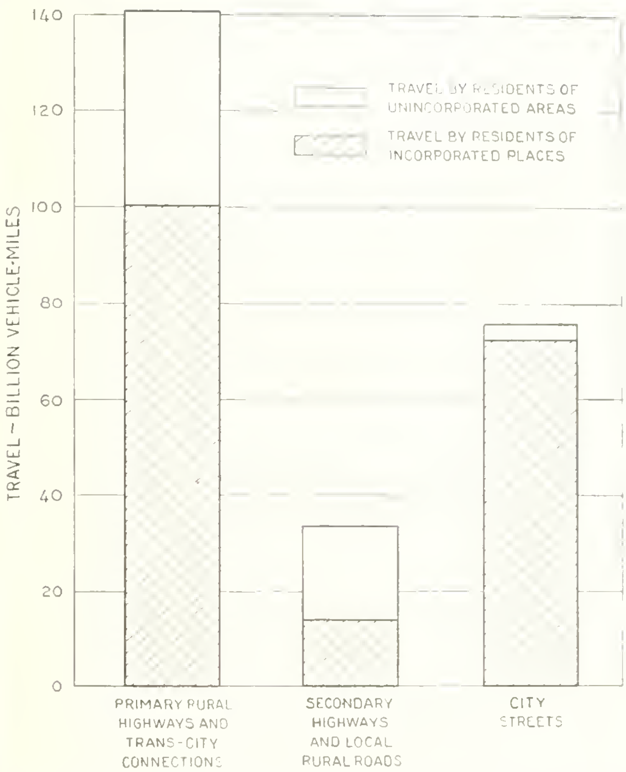

FIGURE 1.—DISTRIBUTION OF ESTIMATED ANNUAL MOTOR-VEHICLE TRAVEL IN THE UNITED STATES IN 1936 BY RESIDENTS OF UNINCORPORATED AREAS AND INCORPORATED PLACES.

This increase in the use of city streets by vehicles owned in the larger incorporated places was, of course, accompanied by a corresponding decrease in the use of other highway systems. It should be noted, however (table 3), that even for vehicles owned in the largest cities the primary rural highways and transcity connections were used for over one-third of the total annual travel. This use was sufficiently high to account for 18.7 percent (table 4) of the annual travel by all motor vehicles on the primary highway system.

As shown in table 4 and figure 1, the largest proportion of travel on the primary highway system was that of town and city residents. Motorists living in incorporated places accounted for 71 percent of the total travel on this system.

The importance of the primary highways to the city motorist is apparent. Though his use of the rural highway system decreased (see table 3) as the size of the place in which the motorist lived increased, the percentage of his travel on such highways was considerable. Only for vehicle owners resident in cities over 100,000 population did their travel on primary rural highways fall below 50 percent of their total travel.

Considering only residents of cities having populations of more than 10,000, table 2 shows that they accounted for more than 60 billion vehicle-miles of the 140,973,700,000 vehicle-miles traveled on primary highways in 1936. Residents of unincorporated areas accounted for only slightly more than 40 billion vehicle-miles of the primary highway travel, or less than that provided by vehicle owners from cities having more than 25,000 population.

In considering travel by residents of incorporated places having 10,000 population or less, it is significant that the percentage of their travel on primary highways as shown in table 3 was greater than that of any other group, even the residents of unincorporated areas. Practically all such incorporated places are located on or within a very short distance from primary roads. Highway development in this country has been such that generally it has been expected that all but the very smallest places would be given consideration in the location of primary routes. Vehicle owners living within these cities are located close to primary highways; they are much closer than many rural residents who live on township or county roads; and they are generally closer than residents of the larger cities who frequently must travel a considerable distance to reach primary routes.

It is apparent from this discussion and from the data shown in the accompanying tables that the major use of the primary highways is by city motor-vehicle owners, and that in spite of their large use of local city streets, the use of primary highways by all city-owned vehicles is greater than their use of all other highway facilities. It therefore follows that the provision of adequate rural highway facilities today is of major importance to the city motorist and that the required improvements in those facilities are largely occasioned by the city motorists' demands on the primary system.

TRIP-LENGTH DATA OBTAINED IN 11 STATES

Table 4 shows that only 4.4 percent of the total travel on city streets was contributed by motorists living in unincorporated areas, and that most of the remaining 95.6 percent of travel performed by residents of incorporated places represented the operations of residents in the larger cities. Of all travel on city streets, 57.9 percent was performed by residents of cities having populations of over 100,000, and drivers living in cities with over 10,000 inhabitants accounted for 83.9 percent of the total travel on local city streets.

Concerning motor-vehicle use on all streets and highways, approximately one-fourth was by residents of unincorporated areas, while twice that amount, or 51.8 percent of all travel, represents the driving of those living in cities having over 10,000 inhabitants. The largest percentage of vehicle travel accounted for by residents of any one group of governmental units as shown in table 4 was that originating in cities having populations over 100,000. Residents of these cities contributed 28.8 percent of all travel on all roads and streets.

These data on vehicle travel have also been expressed in terms of average 24-hour traffic volumes for each class of road and street. Table 6 shows that for the

TABLE 6.—*Approximate mileage of each highway system and average 24-hour traffic volume on each highway system in the United States in 1936*

Highway system	Approximate mileage	Estimated total annual motor-vehicle travel	Average 24-hour traffic volume
	Miles	*Million vehicle-miles*	*Vehicles*
Primary rural highways and transcity connections	359,000	140,973.7	1,076
Secondary highways and local rural roads	[1] 2,618,000	33,431.0	35
City streets	[2] 215,000	75,371.2	960
All systems	[3] 3,172,000	249,778.9	216

[1] Based on latest available estimates.
[2] Estimate includes 20,000 miles of transcity connections which are also included with primary system mileage, because exclusively local city travel includes travel over such connections.
[3] Excludes duplication of 20,000 miles of trans-city connections.

country as a whole, primary rural highways and their transcity connections carried an average daily volume of 1,076 vehicles, which was slightly higher than the 960 vehicles computed as the average for city streets. These volumes were about 30 times greater than the average daily volume on secondary and local rural roads combined. Average 24-hour traffic volume for the more than 3 million miles of roads and streets in the United States was estimated at 216 vehicles.

Another special study of considerable value was also made from road-use data concerning the radii of operation of motor vehicles. It was sought by this investigation to determine the length of vehicle trips that extend beyond the limits of cities; that is, of trips that are either partly or wholly on rural roads. Thus all trips by residents of unincorporated areas were included; but for motorists living in incorporated places, only those trips were counted that extended beyond the limits of the town or city in which the driver resided.

This special study was made in the 11 States of Florida, Kansas, Louisiana, Minnesota, New Hampshire, Pennsylvania, South Dakota, Utah, Vermont, Washington, and Wisconsin. In 1936 there were 4,862,541 passenger cars and 880,432 trucks registered in these 11 States, or a combined registration of 5,742,973. These figures are presented in table 7, together with information concerning the number of interviews taken in each State. The number of interviews totaled 129,407, and consisted of 94,167 for passenger cars and 35,240 for trucks. Trip-length information was not expanded to represent data for the entire country, but only to represent total registrations in each of these States.

For purposes of this analysis, all trips have been classified as one-way trips. If a motor-vehicle owner left his home and drove to some other point 10 miles distant, requiring a total travel of 20 miles from point of starting until return to that point, such a trip could be classified as two one-way trips of 10 miles each. The one-way trip classification has been used for all tabulations in this discussion.

TABLE 7.—*1936 motor-vehicle registrations and number of road use interviews used for basis of analysis of total number of one-way trips outside city limits in 11 States*

State	1936 registration			Number of interviews		
	Passenger cars	Trucks	Total	Passenger cars	Trucks	Total
Florida	321,467	63,885	385,352	7,015	3,010	10,025
Kansas	490,793	[1] 87,113	577,906	8,663	2,813	11,476
Louisiana	228,361	73,628	301,989	3,891	1,623	5,514
Minnesota	668,915	114,448	783,363	13,059	5,649	18,708
New Hampshire	97,361	[1] 24,875	122,236	1,936	914	2,850
Pennsylvania	1,615,955	235,834	1,851,789	23,783	10,567	34,350
South Dakota	158,192	28,216	186,408	3,608	1,533	5,141
Utah	96,768	19,397	116,165	2,148	1,097	3,245
Vermont	75,195	8,845	84,040	1,472	850	2,322
Washington	419,493	79,538	499,031	14,027	1,313	15,340
Wisconsin	690,041	144,653	834,694	14,565	5,871	20,436
Total	4,862,541	880,432	5,742,973	94,167	35,240	129,407

[1] Includes busses.

PASSENGER-CAR AND TRUCK TRIPS PREDOMINATELY OF SHORT LENGTH

Tables 8 and 9 contain analyses of the length of one-way trips partially or wholly traveled on roads in unincorporated areas. The numbers of these trips within designated length classifications are shown graphically in figure 2 for passenger cars and trucks combined.

The short length of travel of a large part of motor-

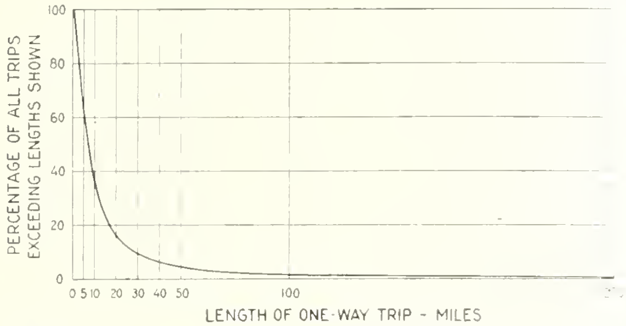

FIGURE 2.—PERCENTAGE OF ALL PASSENGER-CAR AND TRUCK TRIPS EXCEEDING VARIOUS LENGTHS.

vehicle operation is readily apparent. For passenger cars, trips of less than 5 miles constituted 38.4 percent of the number of all one-way trips traveled partly or wholly on highways in unincorporated areas. Trips of over 5 but less than 10 miles comprised 26.5 percent of the total. Of all the one-way trips tabulated, therefore, 64.9 percent of the total number were less than 10 miles long. Trips less than 20 miles long accounted for 85 percent of all passenger-car trips. Accordingly, only 15 percent of all trips extending beyond city limits or traveled entirely on rural roads were greater than 20 miles long. Passenger cars went over 100 miles from their starting point on only 1.5 percent of all their trips.

Analysis of truck movements gave fairly similar results, 34 percent of all trips being less than 5 miles long, 59.5 percent less than 10 miles, and 80.3 percent less than 20 miles. Trips over 100 miles were 2.0 percent of the total number of all trips, and truck trips above 50 miles and less than 250 miles long constituted 6.2 percent of the total number as compared with 4 percent for passenger cars.

Considering passenger cars and trucks combined, 37.5 percent of the number of all one-way trips involving travel on roads in unincorporated areas extended less than 5 miles from the point of origin. The fact that the many short trips made wholly within incorporated areas have been omitted from these trip-length data emphasizes still further the preponderant use of motor vehicles for short trips.

Tables 10 and 11 show the States of destination of one-way trips over 100 miles long made by passenger cars and by trucks registered in the 11 States. These data are summarized in table 12 to show the percentage of such trips having destinations in the State of origin,

TABLE 8.—*Frequency distribution of the length of all one-way trips made by passenger cars that extended outside city limits in 11 States* [1]

State	Length of one-way trips from point of origin in miles: Less than 5	5 to 9.9	10 to 19.9	20 to 29.9	30 to 39.9	40 to 49.9	50 to 99.9	100 to 249.9	250 to 499.9	500 to 999.9	1,000 and over	Total all trips
TOTAL NUMBER OF TRIPS												
	1,000 trips	*1,000 trips*	*1,000 trips*	*1,000 trips*	*1,000 trips*	*1,000 trips*	*1,000 trips*	*1,000 trips*	*1,000 trips*	*1,000 trips*	*1,000 trips*	*1,000 trips*
Florida	45,189	40,584	31,803	11,069	4,055	1,373	3,776	1,497	400	85	76	139,907
Kansas	124,109	69,011	54,600	17,267	6,526	2,282	6,634	2,591	458	154	89	283,721
Louisiana	43,984	25,005	18,019	6,970	2,986	1,335	3,842	946	182	35	10	103,314
Minnesota	97,533	62,426	51,591	16,477	6,477	3,680	8,224	5,045	556	89	45	252,143
New Hampshire	10,782	12,941	10,975	3,046	1,393	900	1,536	326	27	8	4	41,938
Pennsylvania	296,153	214,362	154,277	47,626	21,246	9,180	19,254	8,016	763	221	91	771,189
South Dakota	16,704	11,760	11,880	4,894	1,631	1,233	1,950	734	180	41	24	51,031
Utah	17,019	9,198	6,838	2,626	1,653	908	1,053	477	134	60	41	40,007
Vermont	14,763	10,650	6,178	2,079	718	562	757	223	28	1	1	35,960
Washington	73,201	42,913	33,562	11,020	5,700	2,208	4,882	2,062	359	49	59	176,015
Wisconsin	95,622	77,445	58,499	20,691	9,538	4,123	9,324	3,826	541	82	49	279,740
Total	835,059	576,295	438,222	143,765	61,923	27,784	61,232	25,743	3,628	825	489	2,174,965
PERCENTAGE OF TOTAL NUMBER OF TRIPS												
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Florida	32.3	29.0	22.7	7.9	2.9	1.0	2.7	1.1	0.3	0.1	(2)	100
Kansas	43.8	24.3	19.2	6.1	2.3	.8	2.3	.9	.2	.1	(2)	100
Louisiana	42.6	24.2	17.4	6.8	2.9	1.3	3.7	.9	.2	(2)	(2)	100
Minnesota	38.7	24.8	20.4	6.5	2.6	1.5	3.2	2.0	.2	.1	(2)	100
New Hampshire	25.7	30.9	26.1	7.3	3.3	2.2	3.6	.8	.1	(2)	(2)	100
Pennsylvania	38.4	27.8	20.0	6.2	2.7	1.2	2.5	1.0	.1	.1	(2)	100
South Dakota	32.7	23.0	23.3	9.6	3.2	2.4	3.8	1.4	.4	.1	.1	100
Utah	42.6	23.0	17.1	6.5	4.1	2.3	2.6	1.2	.3	.2	.1	100
Vermont	41.0	29.6	17.2	5.8	2.0	1.6	2.1	.6	.1	(2)	(2)	100
Washington	41.6	24.4	19.0	6.3	3.2	1.3	2.8	1.1	.2	.1	(2)	100
Wisconsin	34.2	27.7	20.9	7.4	3.4	1.5	3.3	1.4	.2	(2)	(2)	100
Total	38.4	26.5	20.1	6.6	2.8	1.3	2.8	1.2	.2	.1	(2)	100
CUMULATIVE PERCENTAGE OF TOTAL NUMBER OF TRIPS												
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Florida	32.3	61.3	84.0	91.9	94.8	95.8	98.5	99.6	99.9	100.0	100	----------
Kansas	43.8	68.1	87.3	93.4	95.7	96.5	98.8	99.7	99.9	100.0	100	----------
Louisiana	42.6	66.8	84.2	91.0	93.9	95.2	98.9	99.8	100.0	100.0	100	----------
Minnesota	38.7	63.5	83.9	90.4	93.0	94.5	97.7	99.7	99.9	100.0	100	----------
New Hampshire	25.7	56.6	82.7	90.0	93.3	95.5	99.1	99.9	100.0	100.0	100	----------
Pennsylvania	38.4	66.2	86.2	92.4	95.1	96.3	98.8	99.8	99.9	100.0	100	----------
South Dakota	32.7	55.7	79.0	88.6	91.8	94.2	98.0	99.4	99.8	99.9	100	----------
Utah	42.6	65.6	82.7	89.2	93.3	95.6	98.2	99.4	99.7	99.9	100	----------
Vermont	41.0	70.6	87.8	93.6	95.6	97.2	99.3	99.9	100.0	100.0	100	----------
Washington	41.6	66.0	85.0	91.3	94.5	95.8	98.6	99.7	99.9	100.0	100	----------
Wisconsin	34.2	61.9	82.8	90.2	93.6	95.1	98.4	99.8	100.0	100.0	100	----------
Total	38.4	64.9	85.0	91.6	94.4	95.7	98.5	99.7	99.9	100.0	100	----------

[1] Based on analysis of 42,407,204 one-way trips performed by 94,167 passenger cars in these States.
[2] Less than 0.1 percent.

TABLE 9.—*Frequency distribution of the length of all one-way trips made by trucks that extended outside city limits in 11 States* [1]

TOTAL NUMBER OF TRIPS

State	Length of one-way trips from point of origin in miles											Total all trips
	Less than 5	5 to 9.9	10 to 19.9	20 to 29.9	30 to 39.9	40 to 49.9	50 to 99.9	100 to 249.9	250 to 499.9	500 to 999.9	1,000 and over	
	1,000 trips	*1,000 trips*	*1,000 trips*	*1,000 trips*	*1,000 trips*	*1,000 trips*	*1,000 trips*	*1,000 trips*	*1,000 trips*	*1,000 trips*	*1,000 trips*	*1,000 trips*
Florida	11,145	11,433	8,504	3,616	1,456	514	1,358	842	179	19	14	39,080
Kansas	17,622	12,722	10,389	3,777	1,540	741	1,946	792	111	23	16	49,679
Louisiana	14,769	11,787	12,645	4,803	2,539	962	3,909	1,325	97	8	(2)	52,844
Minnesota	20,591	20,750	14,942	5,534	2,169	1,302	3,767	2,511	206	11	3	71,786
New Hampshire	2,708	3,504	2,584	863	414	195	602	75	3	(2)	(2)	10,948
Pennsylvania	53,717	33,446	24,704	7,291	3,789	1,897	4,077	1,130	59	20	(2)	130,130
South Dakota	2,824	2,528	3,104	1,239	516	542	1,181	505	77	6	2	12,524
Utah	4,579	1,960	1,616	659	395	247	462	314	45	5	2	10,284
Vermont	3,159	2,817	1,969	564	247	197	258	51	33	2	(2)	9,297
Washington	17,515	9,438	9,724	4,734	2,768	906	1,966	755	72	6	8	47,892
Wisconsin	36,323	28,531	23,340	8,775	4,197	2,759	4,382	1,611	152	10	(2)	110,080
Total	184,952	138,916	113,521	41,855	20,030	10,262	23,908	9,911	1,034	110	45	544,544

PERCENTAGE OF TOTAL NUMBER OF TRIPS

State	Less than 5	5 to 9.9	10 to 19.9	20 to 29.9	30 to 39.9	40 to 49.9	50 to 99.9	100 to 249.9	250 to 499.9	500 to 999.9	1,000 and over	Total all trips
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Florida	28.5	29.2	21.8	9.3	3.7	1.3	3.5	2.1	0.5	0.1	(3)	100
Kansas	35.5	25.6	20.9	7.6	3.1	1.5	3.9	1.6	.2	.1	(3)	100
Louisiana	27.9	22.3	23.9	9.1	4.8	1.9	7.4	2.5	.2	(3)	(3)	100
Minnesota	28.7	28.9	20.8	7.7	3.0	1.8	5.2	3.5	.3	.1	(3)	100
New Hampshire	24.7	32.0	23.6	7.9	3.8	1.8	5.5	.7	(3)	(3)	(3)	100
Pennsylvania	41.3	25.7	19.0	5.6	2.9	1.4	3.1	.9	.1	(3)	(3)	100
South Dakota	22.5	20.2	24.8	9.9	4.1	4.3	9.4	4.0	.6	.1	.1	100
Utah	44.5	19.1	15.7	6.4	3.8	2.4	4.5	3.1	.4	.1	(3)	100
Vermont	34.0	30.3	21.2	6.0	2.7	2.1	2.8	.5	.4	(3)	(3)	100
Washington	36.6	19.7	20.3	9.9	5.7	1.9	4.1	1.6	.2	(3)	(3)	100
Wisconsin	33.0	25.9	21.2	8.0	3.8	2.5	4.0	1.5	.1	(3)	(3)	100
Total	34.0	25.5	20.8	7.7	3.7	1.9	4.4	1.8	.2	(3)	(3)	100

CUMULATIVE PERCENTAGE OF TOTAL NUMBER OF TRIPS

State	Less than 5	5 to 9.9	10 to 19.9	20 to 29.9	30 to 39.9	40 to 49.9	50 to 99.9	100 to 249.9	250 to 499.9	500 to 999.9	1,000 and over	Total all trips
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Florida	28.5	57.7	79.5	88.8	92.5	93.8	97.3	99.4	99.9	100.0	100	....
Kansas	35.5	61.1	82.0	89.6	92.7	94.2	98.1	99.7	99.9	100.0	100	....
Louisiana	27.9	50.2	74.1	83.2	88.0	89.9	97.3	99.8	100.0	100.0	100	....
Minnesota	28.7	57.6	78.4	86.1	89.1	90.9	96.1	99.6	99.9	100.0	100	....
New Hampshire	24.7	56.7	80.3	88.2	92.0	93.8	99.3	100.0	100.0	100.0	100	....
Pennsylvania	41.3	67.0	86.0	91.6	94.5	95.9	99.0	99.0	100.0	100.0	100	....
South Dakota	22.5	42.9	67.5	77.4	81.5	85.8	95.2	99.2	99.8	99.9	100	....
Utah	44.5	63.6	79.3	85.7	89.5	91.9	96.4	99.5	99.9	100.0	100	....
Vermont	34.0	64.3	85.5	91.5	94.2	96.3	99.1	99.6	100.0	100.0	100	....
Washington	36.6	56.3	76.6	86.5	92.2	94.1	98.2	99.8	100.0	100.0	100	....
Wisconsin	33.0	58.9	80.1	88.1	91.9	94.4	98.4	99.9	100.0	100.0	100	....
Total	34.0	59.5	80.3	88.0	91.7	93.6	98.0	99.8	100.0	100.0	100	....

[1] Based on analysis of 22,268,882 one-way trips performed by 35,240 trucks in these States.
[2] Less than 500 trips.
[3] Less than 0.1 percent.

TABLE 10.—*Estimated total number of annual one-way trips over 100 miles long traveled by passenger cars registered in 11 States of origin, and classified by State of destination of individual trips*

State of destination	State of origin										
	Florida	Kansas	Louisiana	Minnesota	New Hampshire	Pennsylvania	South Dakota	Utah	Vermont	Washington	Wisconsin
Alabama	38,172	418	33,422	....	....	960	1,498	90	....	80	102
Arizona	....	764	72	546	....	108	430	3,474	....	1,272	172
Arkansas	1,368	38,180	43,030	726	....	594	86	....	....	46	496
California	870	16,122	1,556	8,994	84	3,516	5,710	33,084	....	39,830	5,716
Colorado	1,032	144,456	130	1,434	....	994	5,902	21,856	....	1,090	1,088
Connecticut	1,060	....	130	....	9,044	18,178	....	....		....	596
Delaware	168	....	....	....	280	124,146	86	....	7,424	....	....
Florida	1,746,624	2,048	8,518	2,156	1,514	30,266	456	....	....	160	5,838
Georgia	137,300	688	6,096	110	....	3,986	....	....	86	220	628
Idaho	....	1,204	....	216	....	....	1,620	115,152	86	123,418	260
Illinois	4,496	23,858	2,060	59,168	288	29,806	7,314	434	....	2,040	536,088
Indiana	4,590	3,582	292	2,398	....	12,360	1,442	90	....	494	34,820
Iowa	610	20,996	344	119,550	126	1,906	73,950	90	....	466	67,972
Kansas	296	1,827,206	330	842	....	950	3,090	260	....	312	990
Kentucky	6,094	2,462	726	436	....	6,564	198	120	....	126	2,916
Louisiana	7,826	4,026	873,324	612	84	904	....	150	....	....	926
Maine	1,140	278	....	110	47,766	10,444	....	....	9,366	....	286
Maryland	1,500	....	324	152	420	381,698	602	....	....	....	994
Massachusetts	2,570	690	....	536	124,144	36,346	....	150	79,258	96	1,918
Michigan	3,814	9,594	584	56,724	428	46,672	1,444	758	....	3,376	158,378
Minnesota	424	14,926	....	5,106,158	....	742	101,304	76	....	1,528	322,160
Mississippi	6,482	1,504	88,934	196	....	....	....	....	....	62	632
Missouri	1,156	505,022	990	5,554	84	2,610	2,502	400	116	684	8,436
Montana	....	1,570	....	5,232	....	360	3,292	21,496	....	32,826	1,222
Nebraska	134	98,148	....	6,592	....	916	24,628	544	....	392	4,402
Nevada	136	424	....	....	....	....	....	31,824	....	844	....
New Hampshire	538	....	....	....	100,766	7,056	....	....	22,576	....	....
New Jersey	2,044	98	....	....	856	1,100,928	....	....	1,560	....	528
New Mexico	....	8,696	924	76	....	....	100	256	....	164	206

TABLE 10.—*Estimated total number of annual one-way trips over 100 miles long traveled by passengercars registered in 11 States of origin, and classified by State of destination of individual trips*—Continued

State of destination	State of origin										
	Florida	Kansas	Louisiana	Minnesota	New Hampshire	Pennsylvania	South Dakota	Utah	Vermont	Washington	Wisconsin
New York	12,708	4,458	1,606	2,296	14,820	1,368,620	2,526	902	28,172	1,620	9,522
North Carolina	23,894	274	2,396		84	14,120			116	160	2,662
North Dakota	68	562	126	89,242		244	20,480			1,066	4,032
Ohio	6,056	4,124	396	1,786	534	837,310	186	492	232	204	8,896
Oklahoma	584	284,538	1,812	1,214		1,578	258	152		382	424
Oregon	100	2,042	256	640		114	952	4,188		343,286	586
Pennyslvania	4,482	1,076	162	720	560	4,419,820			1,284	160	2,256
Rhode Island	202	70	72		12,698	4,126			3,032		
South Carolina	8,336	98	200	660		2,620			116		68
South Dakota		3,048		62,812		228	698,396			904	11,084
Tennessee	13,486	1,872	6,868	1,048	224	4,620	184	90			1,650
Texas	2,894	41,380	93,052	2,308	84	1,682	796	570		774	2,268
Utah		962		196			272	314,246		2,356	462
Vermont	340				34,402	5,206			73,884		
Virginia	2,980	380	304		252	94,218	86			46	472
Washington	220	2,196		2,158		142	2,450	3,302		1,882,136	558
West Virginia	618	352	162	286		159,020					124
Wisconsin	976	1,906	388	157,866		3,694	5,870	90		416	3,277,310
Wyoming	168	9,136	130	1,748	84	490	10,748	148,118		4,876	1,534
District of Columbia	5,042	1,718	528	1,096	880	294,168		210	402	296	1,510
Canada	2,594	2,944	848	31,142	15,532	55,096	2,336	8,960	26,328	76,274	13,678
Mexico	220	2,610	1,756	1,284		366	794	428		3,618	748
Total	2,056,412	3,092,706	1,172,848	5,737,020	366,038	9,090,492	981,988	712,052	254,038	2,528,100	4,497,614

TABLE 11.—*Estimated total number of annual one-way trips over 100 miles long traveled by trucks registered in 11 States of origin, and classified by State of destination of individual trips*

State of destination	State of origin										
	Florida	Kansas	Louisiana	Minnesota	New Hampshire	Pennsylvania	South Dakota	Utah	Vermont	Washington	Wisconsin
Alabama	38,538		70	23							
Arizona								2,990			
Arkansas		2,134	34,250								
California		52			44	40		5,242		8,296	176
Colorado		13,748					10,704	7,958			
Connecticut					3,576	1,318			2,078		
Delaware						14,238					
Florida	930,621					190					
Georgia	49,568			44		544					
Idaho				50			68	59,912		52,786	
Illinois	220	2,504		12,609		2,294	924			110	361,321
Indiana	121	776		5,827		86	34				2,866
Iowa		2,512		97,133			43,992				36,182
Kansas		581,866					460			110	
Kentucky	137	652				4,224					176
Louisiana	126		1,266,240								
Maine					11,850				72		
Maryland	5,321					71,294					176
Massachusetts					38,142	4,114			12,604		
Michigan	133	15,566		378		764				204	75,605
Minnesota		154		2,489,410			28,570				201,889
Mississippi	250		30,856								
Missouri		233,474	136	38,565		132	306				176
Montana							8,020	598		17,472	
Nebraska		26,368		2,556			19,652			110	176
Nevada								11,122			
New Hampshire					13,990				9,426		
New Jersey	45				220	34,420			4,830		
New Mexico		716	66								
New York	2,504			56	2,156	371,576			36,822		176
North Carolina	4,251		60			394					
North Dakota		60		54,566			5,018				176
Ohio	180	60		277		154,188					1,078
Oklahoma		55,618	60								
Oregon						40		32		80,058	
Pennsylvania	2,077					497,144			4,026		
Rhode Island					6,600	54			86		
South Carolina	3,599										
South Dakota		60		7,689			463,604			110	176
Tennessee	4,921		546			32					
Texas	148	5,690	95,366					96			
Utah					2,260			235,788			
Vermont						228			11,862		
Virginia	2,070	60				6,246					176
Washington							936			676,740	
West Virginia	47					32,678					
Wisconsin				20,907							1,092,571
Wyoming		364					7,752	43,918		110	
District of Columbia	8,913					13,622					
Canada				516	132	332			3,651	3,132	
Mexico								64		228	
Total	1,053,790	942,434	1,427,650	2,730,606	78,970	1,210,192	590,040	367,720	85,460	839,466	1,773,096

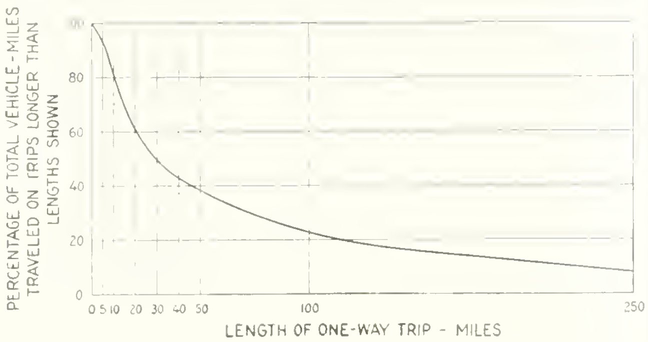

Figure 3.—Percentage of Total Vehicle-Miles by Passenger Cars and Trucks Traveled on Trips Exceeding Various Lengths.

adjoining States, and other States, providing another indication of the dispersion of motor-vehicle travel. It should be noted that even on these longer trips a very high percentage of the destinations was in the State of origin or in an adjoining State.

Table 12.—*Destination of motor-vehicle travel in 11 States on one-way trips over 100 miles long*

PASSENGER CARS

State of origin	Destination of trips in— State of origin	Adjoining States	Other States	Total
	Percent	*Percent*	*Percent*	*Percent*
Florida	84.9	8.5	6.6	100
Kansas	55.5	37.4	7.1	100
Louisiana	74.5	19.2	6.3	100
Minnesota	89.0	8.0	3.0	100
New Hampshire	27.5	60.6	11.9	100
Pennsylvania	48.6	43.7	7.7	100
South Dakota	71.1	23.9	5.0	100
Utah	44.1	45.0	10.9	100
Vermont	29.1	61.5	9.4	100
Washington	74.4	21.5	4.1	100
Wisconsin	72.9	24.1	3.0	100

TRUCKS

State of origin	State of origin	Adjoining States	Other States	Total
Florida	88.3	8.4	3.3	100
Kansas	61.7	35.0	3.3	100
Louisiana	88.7	11.2	.1	100
Minnesota	91.2	6.6	2.2	100
New Hampshire	17.7	63.5	18.8	100
Pennsylvania	41.1	56.1	2.8	100
South Dakota	78.6	19.2	2.2	100
Utah	64.1	34.2	1.7	100
Vermont	13.9	73.1	13.0	100
Washington	80.6	16.2	3.2	100
Wisconsin	61.6	38.1	.3	100

PASSENGER CARS AND TRUCKS

State of origin	State of origin	Adjoining States	Other States	Total
Florida	86.0	8.5	5.5	100
Kansas	56.9	37.1	6.0	100
Louisiana	81.7	15.1	3.2	100
Minnesota	89.7	7.6	2.7	100
New Hampshire	25.8	61.6	12.6	100
Pennsylvania	47.7	45.1	7.2	100
South Dakota	73.9	22.1	4.0	100
Utah	50.1	41.8	8.1	100
Vermont	25.3	64.5	10.2	100
Washington	75.7	20.4	3.9	100
Wisconsin	71.9	25.4	2.7	100

In addition to the distribution of the number of trips in various mileage ranges, the total vehicle-miles involved in these trips have also been computed and are presented in table 13 and figure 3. Here another aspect of motor-vehicle use is shown. For passenger cars, while trips of less than 5 miles constituted 38.4 percent of the total number of trips, they accounted for but 6.6 percent of the total vehicle-miles of travel partly or wholly on rural roads. Trips of less than 20 miles, accounting for 85.0 percent of all trips, involved but 40.9 percent of the total vehicle-miles of travel. Trips classified in mileage groups from 20 miles upward were responsible for a much larger percentage of travel than of total trips. In the higher mileage brackets, trips in the range from 50 to 249.9 miles were only 4.0 percent of the total number of trips, but they accounted for 28.6 percent of vehicle-miles traveled outside city limits.

These characteristics were similar but less pronounced for trucks. Thirty-four percent of the total number of one-way truck trips was classified as extending less than 5 miles, accounting for but 4.9 percent of the total vehicle-miles of travel; and trips of less than 20 miles, or 80.3 percent of all trips, constituted 33.9 percent of the mileage traveled wholly or partially on rural roads.

Table 13.—*Number of trips and vehicle-miles traveled by vehicles which went outside city limits in 11 States*

PASSENGER CARS

Length of one-way trip from point of origin (miles)	Number of trips		Travel	
	1,000 trips	*Percent*	*Million vehicle-miles*	*Percent*
0 to 4.9	835,059	38.4	2,087.6	6.6
5.0 to 9.9	576,295	26.5	4,322.2	13.6
10.0 to 19.9	438,222	20.1	6,573.3	20.7
20.0 to 29.9	143,765	6.6	3,594.1	11.3
30.0 to 39.9	61,923	2.8	2,167.3	6.8
40.0 to 49.9	27,784	1.3	1,250.3	3.9
50.0 to 99.9	61,232	2.8	4,592.4	14.4
100.0 to 249.9	25,743	1.2	4,505.0	14.2
250.0 to 499.9	3,628	.2	1,360.5	4.3
500.0 to 999.9	825	.1	618.8	1.9
1,000.0 and over	489	(1)	733.5	2.3
Total	2,174,965	100.0	31,805.0	100.0

TRUCKS

Length of one-way trip from point of origin (miles)	1,000 trips	Percent	Million vehicle-miles	Percent
0 to 4.9	184,952	34.0	462.4	4.9
5.0 to 9.9	138,916	25.5	1,041.9	11.0
10.0 to 19.9	113,521	20.8	1,702.8	18.0
20.0 to 29.9	41,855	7.7	1,046.4	11.0
30.0 to 39.9	20,030	3.7	701.0	7.4
40.0 to 49.9	10,262	1.9	461.8	4.9
50.0 to 99.9	23,908	4.4	1,793.1	18.8
100.0 to 249.9	9,911	1.8	1,734.4	18.3
250.0 to 499.9	1,034	.2	387.8	4.1
500.0 to 999.9	110	(1)	82.5	.9
1,000.0 and over	45	(1)	67.5	.7
Total	544,544	100.0	9,481.6	100.0

PASSENGER CARS AND TRUCKS

Length of one-way trip from point of origin (miles)	1,000 trips	Percent	Million vehicle-miles	Percent
0 to 4.9	1,020,011	37.5	2,550.0	6.2
5.0 to 9.9	715,211	26.3	5,364.1	13.1
10.0 to 19.9	551,743	20.3	8,276.1	20.1
20.0 to 29.9	185,620	6.8	4,640.5	11.2
30.0 to 39.9	81,953	3.0	2,868.3	6.9
40.0 to 49.9	38,046	1.4	1,712.1	4.1
50.0 to 99.9	85,140	3.1	6,385.5	15.5
100.0 to 249.9	35,654	1.3	6,239.4	15.1
250.0 to 499.9	4,662	.2	1,748.3	4.2
500.0 to 999.9	935	.1	701.3	1.7
1,000.0 and over	534	(1)	801.0	1.9
Total	2,719,509	100.0	41,286.6	100.0

[1] Less than 0.1 percent.

AVERAGE TRIP LENGTH ONLY 15.2 MILES

One-way truck trips less than 50 miles long constituted 93.6 percent of all truck trips outside city limits and accounted for 57.2 percent of all truck travel performed wholly or partially on rural roads. Trips less than 100 miles long accounted for 98.0 percent of such truck trips and 76.0 percent of all truck travel on rural roads. Corresponding figures for trips less than 250 miles were 99.8 percent of the number of trips and 94.3 percent of travel. It may be noted, however, that for distances over 250 miles, the passenger car was used relatively more than the truck. Thus, passenger-car and truck trips of 250 miles or more were 0.3 and 0.2 percent, respectively, of total number of trips, while the travel generated was 8.5 percent of total passenger-car vehicle-miles, and but 5.7 percent of all vehicle-miles of travel by trucks performed wholly or partially on rural roads.

Computations have also been made in this study of the mean and median lengths of trips involving the use of roads in unincorporated areas by residents of various governmental jurisdictions. Results are given in table 14. For the purpose of this particular presentation, unincorporated areas and incorporated places with a population of 2,500 or less have been grouped together, because motor-vehicle owners resident in these two classifications were considered to have travel characteristics sufficiently similar to warrant their combination. For motorists of these smaller cities, rural roads, either primary or purely local, are approximately as easily accessible as such roads are to strictly rural motorists.

Figure 4 shows that for both passenger cars and trucks the mean and median lengths of one-way trips that extended outside city limits were greatest for the largest place of residence of the owners. Thus the mean length of trips made by passenger cars owned by residents of unincorporated areas and places of 2,500 or less inhabitants was 10.6 miles, while for residents of cities having in excess of 100,000 persons the mean length was 37.1 miles. Corresponding values for median trip lengths were 5.9 and 16.3 miles. Figures for trip lengths for trucks were somewhat higher for all places of origin except the largest cities.

The mean one-way trip length for combined passenger-car and truck travel for all governmental jurisdictions was 15.2 miles, and the median trip, 7.4 miles.

The relative effect of the size of cities on highway use is also strikingly illustrated in tables 15 and 16 and figure 5, which show the average number of trips made outside cities by motor-vehicle owners of cities of various sizes. As in previous tables, a single round trip starting inside the city and going to some place outside the city limits was considered as two one-way trips for purposes of mileage classification. Thus, the average passenger-car owner resident in cities having from 2,501 to 10,000 population went outside the city of residence for 75 round trips less than 10 miles long, or as it has been expressed in table 15, for 150 one-way trips less than 5 miles long.

Trips which extended for one-way distances of 50 miles or more were made approximately the same number of times during the year by the average passenger-car operators resident in all sizes of cities. However, the average number of trips extending beyond city limits in the shorter trip-length ranges decreased rapidly with increased size of the city of residence. For example, table 15 shows that residents of cities having populations of over 100,000 made about one-half as many trips in the 20.0- to 29.9-mile trip-length range,

TABLE 14.—*Length of trips traveled outside city limits by vehicles registered in the various population groups in 11 States*

State	Length of trips traveled by vehicles registered in[1]—													
	Unincorporated areas and incorporated places having a population of 2,500 or less		Incorporated places having a population of—								All incorporated places having a population of more than 2,500		All places	
			2,501 to 10,000		10,001 to 25,000		25,001 to 100,000		More than 100,000					
	Mean[2]	Median[3]	Mean	Median	Mean	Median	Mean	Median	Mean	Median	Mean	Median	Mean	Median
PASSENGER CARS														
	Miles	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*
Florida	11.4	6.3	20.2	9.9	29.7	14.6	30.5	14.9	22.8	10.0	23.5	11.3	16.1	8.1
Kansas	9.6	5.0	23.2	11.6	29.7	15.6	34.6	16.0	41.0	18.5	29.6	14.6	13.3	6.3
Louisiana	9.9	5.5	17.9	7.8	22.1	11.2	26.6	13.5	74.2	57.5	28.3	12.6	14.2	6.5
Minnesota	11.4	6.1	21.9	11.6	27.4	12.3	...	...	54.3	19.7	34.7	15.1	16.4	7.3
New Hampshire	13.0	8.3	13.7	8.6	20.1	11.1	27.2	16.7	...	...	18.5	9.9	15.5	8.9
Pennsylvania	9.8	5.9	13.3	7.1	15.1	8.1	19.7	9.4	30.8	13.6	17.5	8.5	13.5	7.1
South Dakota	15.9	8.6	25.2	8.3	31.2	10.0	60.9	26.9	...	...	30.9	9.9	18.7	8.7
Utah	10.8	4.8	18.0	8.2	38.3	15.4	38.8	18.9	52.9	24.0	34.2	14.3	17.4	6.6
Vermont	9.3	5.7	20.2	9.5	24.5	11.4	...	...	...	...	21.6	9.8	11.7	6.5
Washington	11.5	5.8	20.2	8.1	30.6	13.8	26.2	14.7	40.8	20.0	30.6	14.1	14.6	6.7
Wisconsin	10.9	6.4	24.5	12.6	27.9	13.9	33.2	8.4	48.2	25.8	31.2	15.9	15.9	7.9
Average	10.6	5.9	17.2	8.3	20.5	9.9	24.4	11.7	37.1	16.3	22.9	10.0	14.6	7.2
TRUCKS														
Florida	15.3	7.7	18.1	9.5	32.3	[illegible]	21.8	15.2	30.6	9.5	25.8	10.1	19.4	8.7
Kansas	10.9	6.2	19.9	10.1	29.5	[illegible]	47.0	30.0	54.4	29.1	31.7	14.8	17.0	7.8
Louisiana	12.0	7.4	28.7	21.3	42.6	30.6	57.6	36.8	52.0	27.1	44.8	26.8	21.5	9.9
Minnesota	15.1	7.5	26.6	11.7	38.1	[illegible]	...	...	36.1	11.8	33.8	13.2	21.1	8.7
New Hampshire	12.1	8.2	11.4	7.4	24.6	[illegible]	32.9	19.3	...	...	20.6	10.0	16.1	8.9
Pennsylvania	11.9	6.5	10.6	5.0	12.6	6.8	[illegible]	8.4	19.9	9.4	14.2	6.9	13.0	6.7
South Dakota	21.7	10.5	37.3	17.6	64.8	[illegible]	66.9	45.3	...	...	52.5	28.0	28.7	12.9
Utah	16.0	5.2	14.4	5.4	20.3	12.0	41.8	22.5	58.4	20.0	28.8	9.7	19.9	6.4
Vermont	11.9	7.7	14.9	6.3	45.0	[illegible]	...	...	...	...	21.2	7.4	14.2	7.6
Washington	12.6	6.5	18.3	9.7	32.5	[illegible]	24.8	15.4	45.4	31.3	31.6	19.2	17.5	8.4
Wisconsin	11.6	6.9	21.2	11.6	31.0	16.1	33.0	16.5	35.9	19.0	29.2	15.0	16.5	8.3
Average	12.8	7.0	17.6	8.6	24.8	[illegible]	29.7	13.7	35.6	15.3	26.0	11.1	17.1	8.1
PASSENGER CARS AND TRUCKS														
Average	11.1	6.1	17.3	8.4	21.3	10.0	25.4	12.0	36.7	16.0	23.6	10.2	15.2	7.4

[1] This is the one-way distance of all trips. A trip from Washington, D. C., to Baltimore, Md., and return would be considered as 2 trips of 40 miles each.
[2] The mean shows the arithmetical average length of all trips.
[3] The median indicates the length of that trip below and above which equal numbers of trips occur.

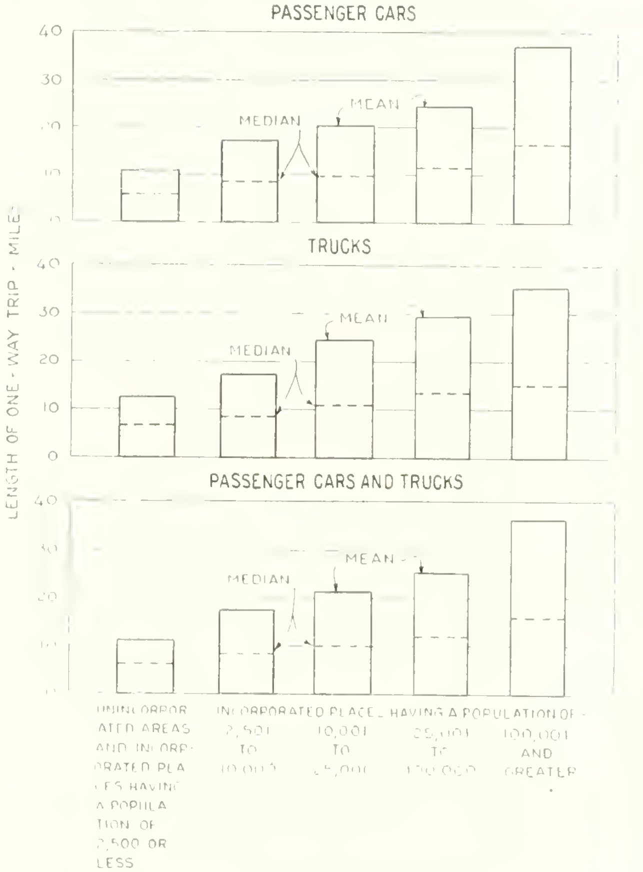

Figure 4.—Mean and Median Lengths of One-Way Trips That Went Outside of City Limits by Vehicles Registered in Various Population Groups.

Table 15.—*Average number of one-way trips of various lengths traveled outside city limits by passenger cars registered in various population groups*

Length of one-way trip in miles from point of origin	Average number of one-way trips traveled by passenger cars registered in cities having populations of—			
	2,501 to 10,000	10,001 to 25,000	25,001 to 100,000	More than 100,000
0 to 4.9	150	84	40	14
5.0 to 9.9	126	84	56	28
10.0 to 19.9	96	92	60	34
20.0 to 29.9	38	32	30	16
30.0 to 39.9	18	18	14	10
40.0 to 49.9	8	10	6	4
50.0 and over	26	28	28	22
Total	462	348	234	128

Table 16.—*Percentage of trips of various lengths traveled outside city limits by passenger cars registered in various population groups*

Length of one-way trip in miles from point of origin	Percentage of trips traveled by passenger cars registered in cities having populations of			
	2,501 to 10,000	10,001 to 25,000	25,001 to 100,000	More than 100,000
0 to 4.9	32.5	24.1	17.1	10.9
5.0 to 9.9	27.3	24.1	23.9	21.9
10.0 to 19.9	20.8	26.4	25.6	26.6
20.0 to 29.9	8.2	9.2	12.8	12.5
30.0 to 39.9	3.9	5.2	6.0	7.8
40.0 to 49.9	1.7	2.9	2.6	3.1
50.0 and over	5.6	8.1	12.0	17.2
Total	100.0	100.0	100.0	100.0

one-third the number in the 10.0- to 19.9-mile range, and one-tenth as many trips in the 0 to 4.9-mile range, as did residents of cities having populations of 2,501 to 10,000.

This smaller number of short trips by vehicles owned in the larger cities is to be expected because of the greater area covered by the larger cities. Since the analysis involved only those trips that extended beyond city limits, a large number of the shorter trips made by residents of large cities did not extend beyond the city limits and are not included in those trips shown here. It is probable that vehicle owners resident in the larger cities make as many, or possibly more, individual trips per year as do residents of the smaller cities. Many of those trips, however, are confined within the rather extensive city limits.

DATA EXPLAIN TRAFFIC CONGESTION NEAR LARGE CITIES

These data should not be considered as evidence that the vehicle owner in smaller cities makes more trips per year than does the owner resident in the larger cities. Rather, the data are an indication that the rural highway is of greater interest to the vehicle owner of large cities for long trips than for short ones, and that the rural highways serve vehicle owners resident in the smaller cities for local travel purposes to a much greater extent proportionally than they do residents of large cities. That is, for those trips extending to rural portions of the highway system there is proportionally a greater interest in longer trips by the residents of a large city than by the residents of small places. Table 16 illustrates this point. Passenger-car owners resident in cities of 2,501 to 10,000 population made 32.5 percent of their trips involving rural highways within the 0 to 4.9-mile trip-length range, while the residents of cities of over 100,000 population made only 10.9 percent of their out-of-city trips within that travel range. The percentages for trips of 50 miles or more one way were 5.6 and 17.2 percent, respectively.

These trip-length data indicate that much of the dense traffic often resulting in congestion on rural portions of highways near city limits is composed of a multitude of cars making short trips originating within the city. Heavily traveled sections of highway extend greater distances from the limits of large cities than from smaller cities because of the greater concentration of vehicles in the city and also because of the higher percentage of longer trips.

Facts derived from road-use data provide important guidance in outlining future highway policies, in regard to both physical and financial plans. The extent and location of the improvements made on the primary highway system are of considerable importance to all residents of the State. Except for those who live in the largest cities, all motorists in the State use the primary highway system more than any other class of roads. The condition of this sytem, therefore, is of comparable interest to all motorists except those residing in the largest cities. The latter do the greatest part of their traveling on city streets. On the other hand, it is significant that these motorists, resident in large cities, because of their large numbers, are responsible for a considerable amount of the total travel on primary highways. Therefore, their interest in such roads, although comparatively less per motorist than for other residents of the State, is still very large in the aggregate.

(*Continued on page 62*)

A NEW VIBRATORY MACHINE FOR DETERMINING THE COMPACTIBILITY OF AGGREGATES

BY THE DIVISION OF TESTS, BUREAU OF PUBLIC ROADS

Reported by J. T. PAULS, Senior Highway Engineer, and J. F. GOODE, Junior Highway Engineer

THE IMPORTANCE of compaction in highway construction has long been recognized. Recent laboratory and field investigations have repeatedly emphasized the value of thorough consolidation in both the base and surfacing courses. Thorough compaction is known to produce the following desirable results:

1. It increases interlocking of the aggregate particles, which is the primary factor in developing a high degree of stability.

2. It retards the entrance of moisture, thus preventing excessive loss of stability under adverse service conditions.

3. It reduces the flow of air and water through bituminous mixtures and is therefore an effective means of lessening damage from weathering and film stripping.

In order to obtain consistently a high degree of consolidation during construction, it is essential to know in advance the limits of compactibility of the materials used. Such tests as have been employed to determine the attainable density of materials, among which are dry rodding, shaking, and various molding tests involving tamping and direct compression, do not always give consistent results. Furthermore, as will be shown in this report, they fail to show the maximum compactibility limits of many aggregates.

The Bureau has been using for some time a small vibrator [1] called the voids determinator for the determination of voids in sheet asphalt aggregates. This vibrator, however, does not give consistent results for mixtures containing high percentages of dust; and, since the testing cylinder has a capacity of only 25 cubic centimeters, it is not suitable for testing aggregates containing large fragments. Accordingly, a new machine has been developed that produces more consistent results and higher densities, and which appears to be equally satisfactory for all gradations of aggregates commonly used in both base and surface construction.

APPARATUS CONSISTS ESSENTIALLY OF A VIBRATING TABLE

The general appearance of the newly developed test apparatus is shown in the cover illustration. The principle of its operation is more clearly brought out in figure 1.

The machine consists essentially of a floating table that is made to move vertically in periodic motion by rotating eccentric masses rigidly connected to its lower surface. The table is a steel plate 13 by 24 inches in size and ¾ inch thick. It is supported at each corner by a helical spring through which there is a vertical guidepost on which the table slides.

On the lower surface of the table, mounted parallel to the long axis of the plate, are two shafts running in ball bearings and geared to rotate at the same speed but in opposite directions. Four steel blocks of equal size and weight are symmetrically mounted at the ends of the two shafts, one at each end of each shaft. The size of these blocks and the speed at which they are rotated determine the magnitude of the unbalanced force. Since the two shafts rotate in opposite directions only vertical accelerations are imparted to the system.

The weight shafts are rotated at speeds of 4,300, 2,500, or 1,500 revolutions per minute by a 3-horsepower electric motor with a 3-speed, V-belt drive.

By trial it was found that the best compaction was obtained with a total eccentric weight of 1,100 grams located $1\frac{1}{16}$ inches off center and rotating at 4,300 revolutions per minute. For these particular conditions the maximum centrifugal force developed by each of the four eccentric masses is theoretically about 338 pounds. In the extreme upper and lower positions these forces add to give a theoretical total vertical resultant of about 1,350 pounds while at the midpoint between these positions the forces developed by the weights on one shaft exactly balance those of the other shaft and the total horizontal resultant is 0 pound.

At a frequency of 4,300 cycles per minute a powerful vibration is developed in the entire mass.

The assembly for holding the aggregate to be tested is bolted to the top of the vibrating plate or table. It is shown in section in figure 2. Its essential parts are a base plate and bottom plunger bolted to the table, a cylinder fitting over the bottom plunger and resting on a rubber support, and a top plunger which rests on the test material in the cylinder.

A micrometer dial mounted on a suitable base is used in conjunction with a series of calibrated gage blocks to measure the thickness of the compacted specimen without removing it from the cylinder.

The top plunger imposes a dead load of 1.75 pounds per square inch on the sample to be compacted. This dead load generally provides sufficient confinement to flatten the top of the specimen and to prevent segregation of the particle sizes. Both the top and bottom plungers have just sufficient clearance within the cylinder to allow free vertical movement during vibration, and each is fitted with three bronze guide strips to maintain it in a position parallel to the axis of the cylinder. The loss of fine aggregate is held to a minimum by the insertion of close-fitting pasteboard gaskets or pads above and below the test specimen. A suitable correction is made in the measured height of the specimen to allow for the final thickness of the pads.

EQUIPMENT ADAPTABLE FOR TESTING DIFFERENT AGGREGATES

In making a test, the first step is to obtain an initial or zero reading with the micrometer dial on the combined height of the two plungers with the two pasteboard pads compressed between the plungers by vibration for a short period. For this zero reading a steel spacer gage of the approximate thickness of a compacted specimen is inserted under the dial so that its ¾- or

[1] Research on Bituminous Paving Mixtures, by W. J. Emmons. PUBLIC ROADS, vol. 7, No. 10, December 1926.

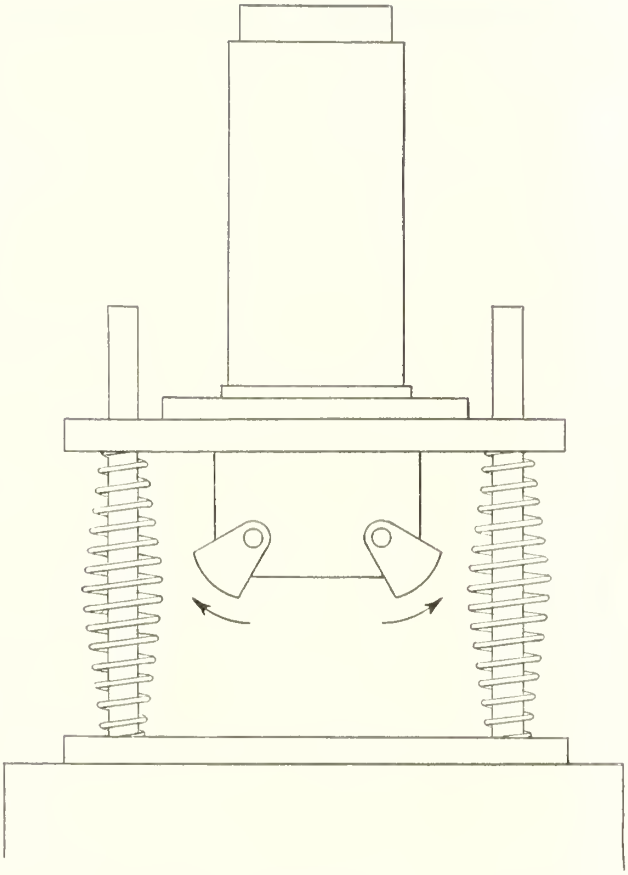

FIGURE 1. ESSENTIAL ELEMENTS OF THE VIBRATORY COMPACTOR.

1-inch range of travel will not be exceeded when the specimen is in place.

For tests in the 4-inch cylinder, which is the one used for aggregates up to about 1-inch maximum size, sufficient aggregate is used to produce a compacted specimen approximately 1½ to 1¾ inches high. This requires about 750 grams of aggregate.

If desired, a much smaller cylinder may be used when testing fine aggregates such as soil, sand, rock dust, or sheet asphalt aggregate, and the depth of the compacted specimen may be reduced to 1 inch or less and its weight to as little as 75 grams. For very large aggregates, a larger cylinder should be used and the thickness of the compacted specimen should be increased so that it is at least one-half to three-fourths inch more than the nominal diameter of the largest individual aggregate particle. The weight of the top plunger should be such that the dead load is approximately 1 pound per square inch per inch of depth of the compacted specimen.

It is essential that the loose aggregate be placed in the cylinder without segregation. When the aggregate to be tested has a large percentage retained on the No. 10 sieve it has been found that the addition of about 50 to 70 cubic centimeters of kerosene to 750 grams of aggregate aids greatly in preventing segregation and does not interfere with compaction. The most satisfactory amount of kerosene seems to be that which will just fill the voids in the compacted aggregate.

Materials such as fine soil, sand, clay, etc., are not particularly subject to segregation and, because of the greater difficulty with which air is forced out of them when wet, do not always compact as well with kerosene as without. They are therefore tested dry.

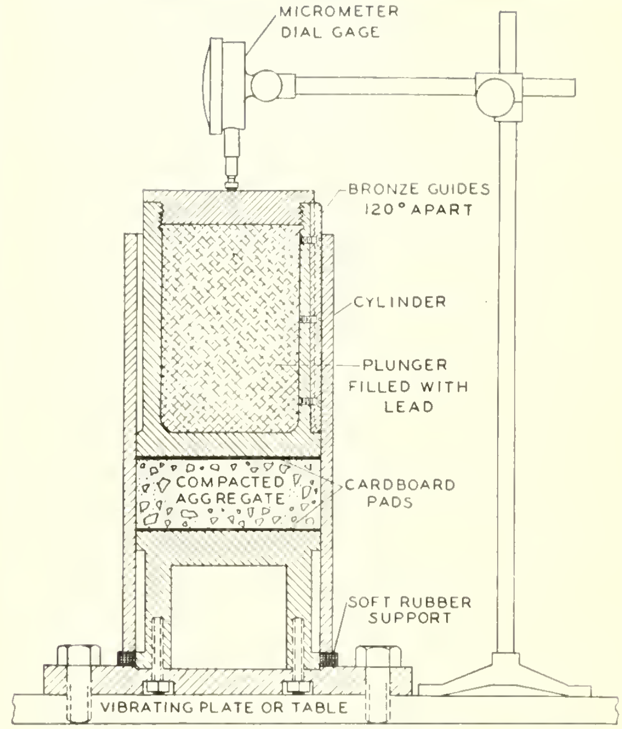

FIGURE 2. - CYLINDER AND PLUNGER ASSEMBLY WITH MEASURING DEVICE.

For determining whether or not to use kerosene it has been found that in general the following conditions will govern:

1. For aggregates having less than 35 percent passing the No. 10 sieve, use kerosene.
2. For aggregates having more than 50 percent passing the No. 10 sieve, test dry.
3. For aggregates having more than 35 percent and less than 50 percent passing the No. 10 sieve, test both with and without kerosene and report the higher density value obtained.

VIBRATION FOR 20 MINUTES ADOPTED AS STANDARD PROCEDURE

All aggregates should be oven-dried before testing, since very small amounts of water or other liquid, as distinguished from the relatively large amount of kerosene added in testing coarse materials, have a marked bulking effect which interferes with the obtaining of accurate test results. Drying is also necessary in order to obtain the true sample weights for use in calculating the density after vibration. Lumps or clods of clay in the aggregate impair the accuracy of the test and should be thoroughly broken down before placing the sample in the cylinder for compaction.

After the material is placed in the cylinder, with a pasteboard pad underneath and another on top, the upper plunger is inserted and the assembly is vibrated for a period of 20 minutes. The final reading is taken with the dial, and from this reading and the initial reading the over-all volume of the material in the cylinder is calculated. This volume, the dry weight, and the apparent specific gravity of the aggregate are used in calculating the density. In this report density is expressed as the percentage of aggregate volume per unit of total volume.

The method of determining this percentage is illustrated with a typical example:

Apparent specific gravity [2] of aggregate	2.67
Weight of aggregate sample, grams	736
Volume of vibrator-compacted sample, cubic centimeters	313.9
Unit weight of compacted sample (grams per cubic centimeter) 736/313.9	2.35
Density of compacted sample, (percent) $\frac{2.35}{2.67}\times 100$	88.0

The densities of a number of aggregates were determined for various periods of vibration up to a maximum of 60 minutes. The results of these tests are shown in figure 3. The asphaltic concrete aggregate, the sheet asphalt aggregate, and the fine sand showed practically no increase in density after 20 minutes of vibration. The sand-clay and the sand-clay-gravel each showed an apparent increase of 1.1 percent in density for the time increment from 20 to 60 minutes, and the micaceous soil showed an increase of 1.4 percent. It was found, however, that loss of dust, which became quite noticeable late in the test because of wear on the gaskets, accounted for most of the reduction in volume and consequent apparent increase in density after the initial 20 minutes of vibration. Vibration for a period of 20 minutes has, therefore, been adopted as regular procedure for the test.

The results of compaction tests on three different types of aggregate are shown in table 1 and demonstrate the ability of the apparatus to produce results that check. The maximum variation in results for these tests was slightly under 0.5 percent. However, for routine testing by various operators, this degree of accuracy probably could not be expected.

Table 1.—*Consistency of check tests using the vibratory compacting machine*

Type of aggregate	Density (aggregate volume per unit of total volume)				
	Test No. 1	Test No. 2	Test No. 3	Test No. 4	Average
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Sand-clay	79.4	79.6	79.6	79.6	79.6
Sand-clay-gravel	86.7	86.9	86.7	87.1	86.9
Sheet asphalt (sand and dust)	76.8	76.6			76.7

In the following tables and discussion, the results of a number of compaction tests using the vibratory machine and several other methods of compaction are shown. Table 2 shows the comparative effects of vibration and direct compression on the grading of the aggregate and demonstrates that little or no change in grading was produced by the vibratory method of compaction, whereas direct compression resulted in sufficient crushing to alter materially the grading of the aggregate samples.

[2] Standard Definitions of Terms Relating to Specific Gravity, A. S. T. M. Designation E12-27.

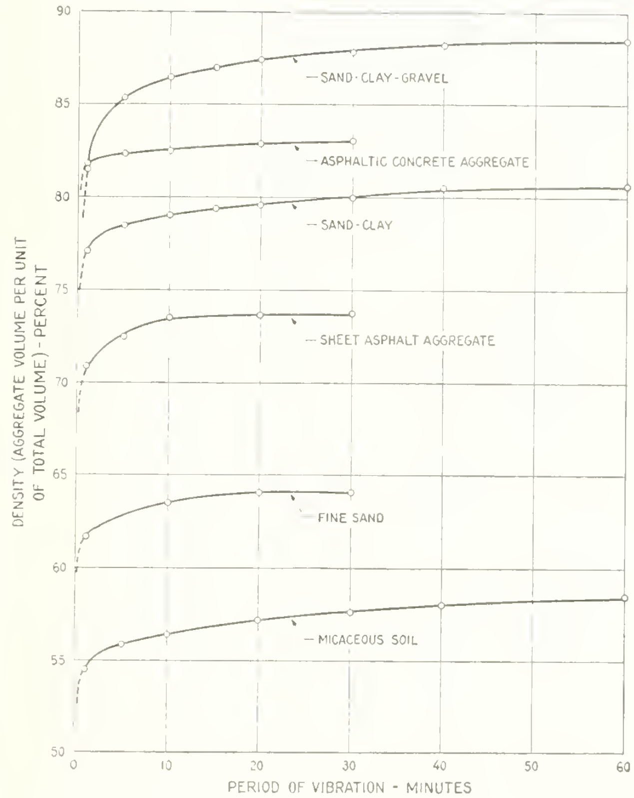

Figure 3.—Densities of Various Types of Aggregates and Their Rates of Consolidation.

HIGHER DENSITIES OBTAINED BY VIBRATORY METHOD THAN BY OTHER METHODS

A comparison of densities obtained by several methods of compaction on various types of aggregates is shown in table 3. In the upper section of the table dealing with the aggregates for base courses, the densities obtained from circular-track test sections built and compacted under the most favorable laboratory conditions agree closely with those obtained by the vibratory

Table 2.—*Effect of compaction by compression and vibration on breakage of various aggregates*

Type of aggregate	Method of compaction	Grading, total aggregate passing—							
		1-inch sieve	3/4-inch sieve	3/8-inch sieve	No. 4 sieve	No. 10 sieve	No. 40 sieve	No. 100 sieve	No. 200 sieve
		Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Graded, fine—high dust content	None				100.0	99.7	82.5		15.5
Do	Vibration				100.0	99.7	82.2		14.7
Graded, fine—low dust content	None				100.0	99.4	82.3		2.4
Do	Vibration				100.0	99.8	82.8		2.8
Do	None					100.0	75.2	18.9	2.2
Do	Compression, 3,000 lb./sq. in.					100.0	76.9	25.6	7.5
Graded, coarse—medium dust content	None	100	92.5	75.8	66.8	63.7	48.6		9.4
Do	Vibration	100	92.5	75.8	66.6	63.3	48.3		9.4
Graded, coarse—low dust content	None	100	98.3	76.7	55.9	50.0	38.5	11.4	1.8
Do	Compression, 3,000 lb./sq. in.	100	98.2	82.0	61.6	52.0	39.8	14.4	4.1
Graded, coarse—high dust content	None	100	90.8	77.0	66.3	47.2	31.7		17.3
Do	Compression, 3,000 lb./sq. in.	100	92.5	80.2	69.8	53.2	36.0		21.0

test. The other methods of compaction shown, with few exceptions, gave considerably lower densities. Direct compression appears to be quite effective for the fine-grained materials, but, as previously shown, the crushing of the aggregate in this test renders the results somewhat unsatisfactory.

TABLE 3.—*Comparison of aggregate densities obtained by various methods of compaction*

BASE COURSE MATERIALS [1]

Character of material tested			Density (aggregate volume per unit of total volume)			
				Aggregates compacted in laboratory		
Type	Plasticity-index	Passing No. 200 sieve	Samples cut from road or test track	Vibratory method	Compression 3,000 lb./sq. in.	Voids determinator
		Percent	Percent	Percent	Percent	Percent
Micaceous soil	0	62	53.4	57.3		
Micaceous soil with 3 percent cement	0	63	57.8	57.5		
Micaceous soil with 11 percent cement	0	65	57.0	57.0		
Sand-clay	0	25	74.8	80.6	75.3	80.2
Do	5	26	80.8	80.8	81.4	76.5
Do	6	20	81.2	81.4	78.8	75.6
Do	9	27	79.5	79.6	81.4	75.1
Do	13	29	78.1	78.5	82.4	73.1
Do	18	29	76.2	76.3	82.2	71.1
Sand-clay-gravel	0	17	82.8	89.7		
Do	0	1	77.8	86.9	76.7	
Do	5	15	87.0	88.0		
Do	6	16	89.3	89.9	82.9	
Do	7	22	87.3	87.5	83.2	
Do	8	12	89.1	89.9	84.2	
Do	9	25	83.9	86.7	84.1	
Do	11	17	86.2	87.1		
Do	16	16	84.0	87.2		
HOT, PLANT-MIX SURFACING MATERIALS [2]						
Sheet asphalt, D. C		14.9	70.7	76.5		71.4
Sheet asphalt, Ohio		12.7	69.3	75.0		
Fine bituminous concrete, Ohio		4.7	82.0	85.9		
Do		6.0	77.9	89.5		
Medium bituminous concrete, Ohio		4.1	76.4	80.6		
Do		6.7	81.3	88.1		
Coarse bituminous concrete, Ohio		3.3	89.7	86.9		
Do		4.2	81.6	89.2		

[1] Samples taken from circular track test sections.
[2] Field samples from pavements. Laboratory compaction tests made on extracted aggregates.

The densities obtained by means of the new vibratory machine are in general much higher than those obtained by the voids determinator.[3] The new apparatus has the further advantage of permitting the testing of large-size aggregates. The voids determinator used in previous work of the Bureau is not suitable for testing materials larger than those passing the No. 10 sieve.

The lower section of table 3 shows a comparison between the aggregate densities of asphaltic pavements of the hot-mix type and the densities obtained by vibrating the extracted aggregates from these pavements. The data shown indicate that construction operations and traffic may not generally produce as high densities in hot-mix pavements as are produced by vibrating the dry aggregates. The highly viscous binders apparently resist the free adjustment of the aggregate particles to form their densest possible arrangement. This resistance is known to be considerably less for the liquid binders than for the highly viscous ones. Mixtures containing the liquid materials often attain densities closely agreeing with the vibratory test results, which accounts for the fact that such mixtures cannot safely be made as rich in bituminous material as hot paving mixtures of comparable aggregate grading.

[3] Research on Bituminous Paving Mixtures, by W. J. Emmons. Public Roads, vol 7, No 10, December 1926.

To attain consistently during construction a satisfactory degree of compaction for any particular material it is necessary to know in advance its compactibility limit, to have an idea of how closely this limit may be approached by practical construction methods, and how closely it needs to be approached to insure satisfactory behavior provided the materials are otherwise satisfactory. Tests have been made on a large number of materials. The few typical results given in table 4 illustrate the relations between field densities and compactibility limits, as determined by the vibratory machine.

For the plastic sand-clay and sand-clay-gravel materials that have been found by various tests to be suitable for base-course construction, the compaction obtained during construction appears to be the deciding factor influencing service behavior. The importance of consolidation is particularly well illustrated in the behavior of the plastic sand-clay-gravel referred to in the footnote of table 4. This material, which is representative of a group of materials that showed similar behavior, was placed in the test section as a base course with insufficient moisture to permit compaction to the density obtained in the vibratory test. It failed in service as soon as unfavorable sub-base moisture conditions were imposed. It was later scarified and recompacted with a higher moisture content. It was then easily compacted to essentially the same density as was obtained in the vibratory test and gave excellent service under very adverse moisture conditions.

TABLE 4.—*Relation of density of soil-type bases to service behavior for base course materials*

Character of material tested					Density (aggregate volume per unit of total volume)		Behavior in test sections
		Grading, total aggregate passing—					
Type	Plasticity index	No. 10 sieve	No. 40 sieve	No. 200 sieve	Vibratory compaction	Field compaction	
		Percent	Percent	Percent	Percent	Percent	
Micaceous soil	0	98	76	62	57.3	53.4	Unsatisfactory.
Micaceous soil with 3 percent cement.	0	98	77	63	57.5	57.8	Satisfactory.
Micaceous soil with 11 percent cement.	0	98	78	65	57.0	57.0	Do.
Sand-clay	0	100	71	25	80.6	74.8	Do.
Do	6	100	51	20	81.4	81.2	Do.
Do	9	100	68	27	79.6	79.5	Fairly satisfactory.
Sand-clay-gravel	0	54	34	17	89.7	82.8	Satisfactory.
Do (Same material)[1]	5	48	31	15	88.0	83.2	Unsatisfactory.
Do (Same material)[1]	5	48	31	15	88.0	87.0	Satisfactory.

[1] When this material was placed in the roadway it had so low a moisture content that it did not compact to a satisfactory density. It failed early in service but when remixed and relaid with the correct moisture content, it compacted to within 1 percent of the density obtained by vibration and gave satisfactory service.

VIBRATORY METHOD USEFUL IN BLENDING AGGREGATES TO OBTAIN DENSE MIXTURE

In highway base- and surface-course construction it is frequently necessary to blend two or more aggregates to provide a material suitable for the intended use. The vibratory compactor provides a means by which the best combination of two or more available aggregates may be determined. The application of this test to the design of aggregate blends and bituminous mixtures will be discussed in connection with figures 4, 5, 6, and 7.

Figures 4 and 5 illustrate two methods of using the vibratory compactor to obtain the densest combination

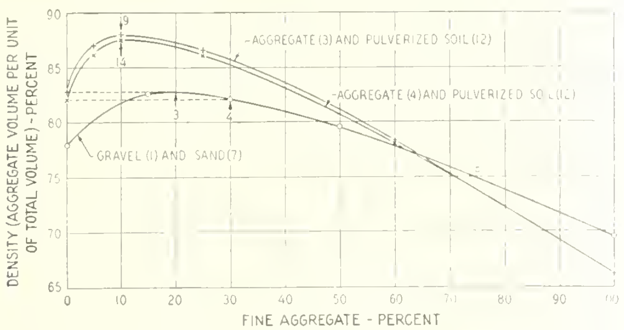

FIGURE 4.—BLENDING CURVES FOR CRUSHED GRAVEL, SAND, AND PULVERIZED SOIL. AGGREGATE NUMBERS CORRESPOND TO THOSE IN TABLE 5.

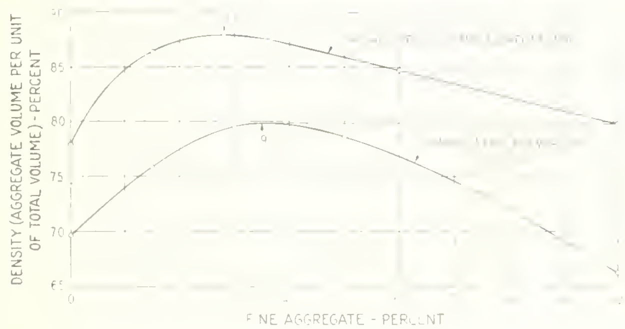

FIGURE 5.—BLENDING CURVES FOR CRUSHED GRAVEL, SAND, AND PULVERIZED SOIL. AGGREGATE NUMBERS CORRESPOND TO THOSE IN TABLE 5.

of three different aggregates for use as a base-course material. For this type of construction the combination of the available materials that gives the densest mixture is generally the most desirable. The three aggregates used in producing the blending curves of figures 4 and 5 were crushed gravel (1-inch maximum size), fine sand, and a pulverized soil. The densities and the gradings of the individual constituents and the various blends are shown in table 5.

In the first method illustrated in figure 4, an initial series of blends was made of the gravel and sand and the densest blend of these two materials was determined. This blend, designated as aggregate 3 in table 5, was then blended in various proportions with the pulverized soil, the densest blend in this series being presumably the densest possible blend of the three materials. This blend, designated aggregate 9 in table 5, had a density of 87.9 percent aggregate solids and the following composition: Gravel 72 percent, sand 18 percent, and pulverized soil 10 percent.

A less dense blend of the gravel and sand, selected at random and designated aggregate 4 in table 5, was also blended with the pulverized soil as shown in figure 4. The highest density obtained by blending with aggregate 4 was lower than that obtained with aggregate 3, indicating that the procedure of selecting the densest combination of the coarse materials for blending with the fines was the correct method.

In the second method, illustrated in figure 5, the order of the tests was reversed. The initial series of blends was made with the sand and pulverized soil. The densest blend of these, designated aggregate 19 in table 5, was then blended with the gravel. The two methods gave identical final results both as to maximum density and proportions of the three constituents, the density at the high point of the second curve being again 87.9 percent and the proportions of material being: Gravel 72 percent, sand 18 percent, and soil 10 percent.

It is of interest to note that the grading of the densest blend of these three materials, which were selected more

TABLE 5.—*Densities and gradings of blended aggregates, sand-clay-gravel base-course type*

Identification	Composition of aggregate				Density (aggregate volume per unit of total volume)	Grading, total aggregate passing—					
	Coarse		Fine			¾-inch sieve	⅜-inch sieve	No. 4 sieve	No. 10 sieve	No. 40 sieve	No. 200 sieve
	Type	Amount	Type	Amount							
		Percent		*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1	Crushed gravel	100	Sand	0	77.9	82.0	48.5	24.9	0.8	0.3	0
2	do	85	do	15	82.6	84.7	56.2	36.2	15.7	12.0	.6
3	do	80	do	20	82.8	85.6	58.8	39.9	20.6	15.9	.9
4	do	70	do	30	82.1	87.4	64.0	47.4	30.6	23.8	1.3
5	do	50	do	50	79.6	91.0	74.3	62.5	50.4	39.4	2.2
6	do	25	do	75	75.6	95.5	87.1	81.2	75.2	59.0	3.2
7	do	0	do	100	69.6				100.0	78.5	4.3
3	Aggregate No. 3	100	Pulverized soil	0	82.8	85.6	58.8	39.9	20.6	15.9	.9
8	do	95	do	5	87.0	86.3	60.9	42.9	24.6	20.1	5.1
9	do	90	do	10	87.9	87.0	62.9	45.9	28.5	24.3	9.2
10	do	75	do	25	86.6	89.2	69.1	54.9	40.5	36.9	22.1
11	do	40	do	60	78.3	94.2	83.5	76.0	68.2	66.4	51.7
12	do	0	do	100	66.2					100.0	85.6
4	Aggregate No. 4	100	do	0	82.1	87.4	64.0	47.4	30.6	23.8	1.3
13	do	95	do	5	86.2	88.0	65.8	50.0	34.1	27.6	5.5
14	do	90	do	10	87.6	88.7	67.6	52.7	37.5	31.4	9.7
15	do	75	do	25	86.1	90.6	73.0	60.6	48.0	42.9	22.4
16	do	40	do	60	78.1	95.0	85.6	79.0	72.2	69.5	51.9
12	do	0	do	100	66.2					100.0	85.6
7	Sand	100	do	0	69.6				100.0	78.5	4.3
17	do	85	do	15	75.8				100.0	81.7	16.5
18	do	70	do	30	79.5				100.0	85.0	28.7
19	do	65	do	35	79.8				100.0	86.0	32.7
20	do	50	do	50	78.6				100.0	89.3	45.0
21	do	25	do	75	73.3				100.0	94.6	65.3
12	do	0	do	100	66.2					100.0	85.6
1	Crushed gravel	100	Aggregate No. 19	0	77.9	82.0	48.5	24.9	.8	.3	0
22	do	85	do	15	86.3	84.7	56.2	36.2	15.7	13.2	4.9
23	do	72	do	28	87.9	87.0	62.9	45.9	28.5	24.3	9.2
24	do	40	do	60	84.7	92.8	79.4	70.0	60.3	51.7	19.6
19	do	0	do	100	79.8				100.0	86.0	32.7

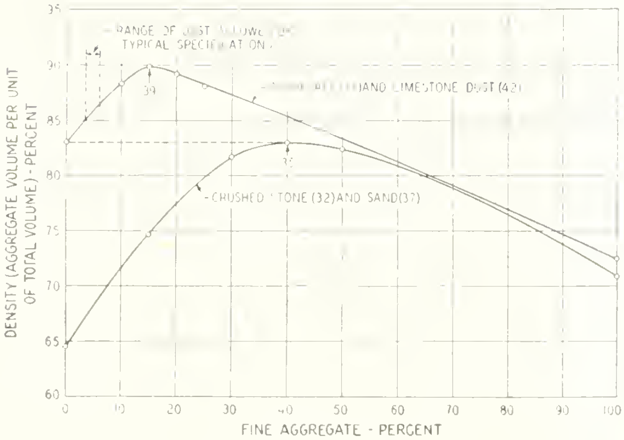

FIGURE 6.—BLENDING CURVES FOR BITUMINOUS CONCRETE. AGGREGATE NUMBERS CORRESPOND TO THOSE IN TABLE 7.

or less at random, conformed to the grading requirements now recommended for base-course construction. This relationship is shown in table 6.

In figure 6 is shown the application of method 1 in blending crushed stone, sand, and limestone dust for a typical bituminous concrete aggregate. The densities and gradings of the various blends are shown in the lower section of table 7. As shown in figure 6 a maximum density of 89.8 percent solids was obtained, using

TABLE 6.—*Comparison of grading obtained by blending sand, clay, and gravel for maximum density, with recommended grading requirements for base-course construction*

	Grading, total aggregate passing—						
	1-inch sieve	3/4-inch sieve	3/8-inch sieve	No. 4 sieve	No. 10 sieve	No. 40 sieve	No. 200 sieve
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Maximum-density blend	100	87.0	62.9	45.9	28.5	24.3	9.2
A. A. S. H. O. specification for type B, sand-clay-gravel base	100	70-100	50-80	35-65	25-50	15-30	5-15

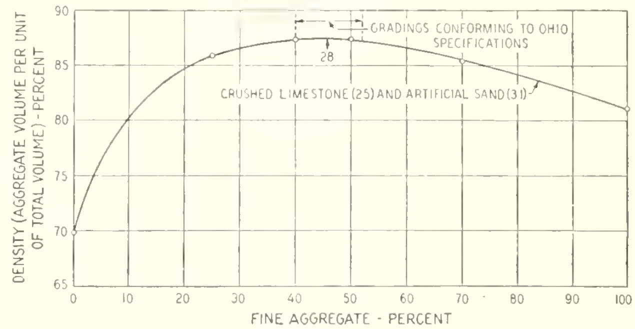

FIGURE 7.—BLENDING CURVE FOR CRUSHED LIMESTONE AND ARTIFICIAL LIMESTONE SAND. AGGREGATE NUMBERS CORRESPOND TO THOSE IN TABLE 7.

15 percent of limestone dust with the densest blend of the stone and sand (aggregate 35, table 7).

VIBRATOR ENABLES DESIGN OF MIXTURES WITHOUT OVERFILLING VOIDS

Here is an example where too dense an aggregate for practical use in bituminous concrete was obtained since the voids remaining would only permit the use of about 10 percent by volume or approximately 5 percent by weight of asphalt. To produce a practical aggregate it would be necessary to reduce its density. This would best be accomplished by reducing the dust content since the densest possible combination of the coarse fractions is always desirable. Reduction of the dust content to range between 3½ and 6 percent would reduce the density to between 85 and 86.5 percent solids, thus permitting the use of approximately 6 to 7 percent asphalt by weight and bringing the design into line with established practice.

Figure 7 shows a blending curve for bituminous concrete aggregate composed of crushed stone and artificial limestone sand without dust. This type of aggregate is used extensively in Ohio. The densities and gradings of the constituents and blends are shown in the upper section of table 7. This type differs from the previous

TABLE 7.—*Densities and gradings of blended aggregates of the type used in bituminous concrete*

Identification	Composition of aggregate				Density (aggregate volume per unit of total volume)	Grading, total aggregate passing—					
	Coarse		Fine								
	Type	Amount	Type	Amount		3/4-inch sieve	3/8-inch sieve	No. 4 sieve	No. 10 sieve	No 40 sieve	No. 200 sieve
WITHOUT MINERAL FILLER											
		Percent		*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
25	Crushed stone	100	Artificial sand	0	69.8	100	90.0	5.0	0	0	0
26	do	75	do	25	85.9	100	92.5	28.8	23.3	9.3	.9
27	do	60	do	40	87.3	100	94.0	43.0	37.3	14.8	1.4
28	do	54	do	46	87.5	100	94.6	48.7	42.9	17.0	1.6
29	do	50	do	50	87.4	100	95.0	52.5	46.6	18.5	1.8
30	do	30	do	70	85.4	100	97.0	71.5	65.2	25.9	2.5
31	do	0	do	100	81.1			100.0	93.2	37.0	3.5
WITH MINERAL FILLER											
32	Crushed stone	100	Sand	0	64.5	100	0	0	0	0	0
33	do	85	do	15	74.7	100	15.0	15.0	11.7	4.5	.3
34	do	70	do	30	81.7	100	30.0	30.0	23.4	9.0	.6
35	do	60	do	40	83.0	100	40.0	40.0	31.2	12.0	.8
36	do	50	do	50	82.4	100	50.0	50.0	39.0	15.1	1.0
37	do	0	do	100	70.8			100.0	78.0	30.1	2.0
35	Aggregate No. 35	100	Limestone dust	0	83.0	100	40.0	40.0	31.2	12.0	.8
38	do	90	do	10	88.3	100	46.0	46.0	38.1	20.8	10.4
39	do	85	do	15	89.8	100	49.0	49.0	41.5	25.2	15.2
40	do	80	do	20	89.2	100	52.0	52.0	45.0	29.6	19.9
41	do	75	do	25	88.1	100	55.0	55.0	48.4	34.0	24.7
42	do	0	do	100	72.4					100.0	96.4

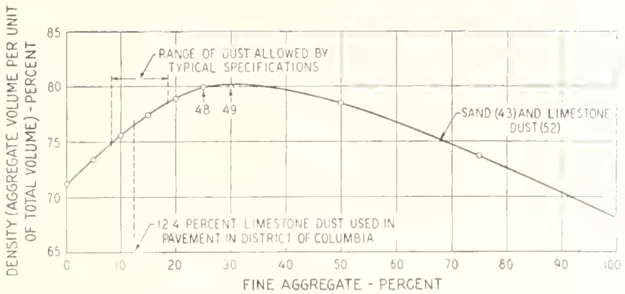

FIGURE 8.—BLENDING CURVE FOR SHEET ASPHALT SAND AND COMMERCIAL LIMESTONE DUST. AGGREGATE NUMBERS CORRESPOND TO THOSE IN TABLE 8.

example in that the densest combination of the two aggregate constituents provides sufficient void space for the bituminous material. It is therefore desirable to use the densest blend, this being easily found by means of the vibratory compactor.

The aggregate void space in the densest blend shown in figure 7 would permit the use of about 6 percent asphalt, which conforms approximately to the design used successfully in Ohio with the same type of aggregate.

The results of vibratory compaction tests on blends of fine sand and limestone dust to give a dense aggregate for sheet asphalt are shown in figure 8. The densities and gradings of the two constituents and the blends are given in table 8. A maximum density of 79.9 percent solids was obtained with the blends consisting of 70 percent fine sand and 30 percent dust and 75 percent sand and 25 percent dust. Again the 20 percent voids in this blend provide insufficient space for the proper amount of asphalt and the high dust content would produce an aggregate that would be difficult to mix and handle.

TABLE 8.—*Densities and gradings of blended aggregates of the type used in sheet asphalt*

Sample identification	Composition of aggregate		Density (aggregate volume per unit of total volume)	Grading, total aggregate passing—			
	Sand	Limestone dust		No. 10 sieve	No. 40 sieve	No. 80 sieve	No. 200 sieve
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
43	100	0	71.2	100	81.3	33.8	3.5
44	95	5	73.4	100	82.2	37.1	8.1
45	90	10	75.6	100	83.2	40.4	12.7
46	85	15	77.4	100	84.2	43.7	17.3
47	80	20	78.9	100	85.1	47.0	21.9
48	75	25	79.9	100	86.0	50.4	26.4
49	70	30	79.9	100	86.9	53.7	31.0
50	50	50	78.5	100	90.7	66.9	49.4
51	25	75	73.7	100	95.3	83.5	72.4
52	0	100	68.1	100	100.0	100.0	95.3

In this type of construction the problem of design utilizing the vibratory compactor might be attacked from either of two angles:

1. The amount of dust could be set on the basis of well-established practice, which would call for considerably less than 25 percent dust, and the asphalt content required to fill the void space could then be determined by vibratory tests on the fixed aggregate blend.

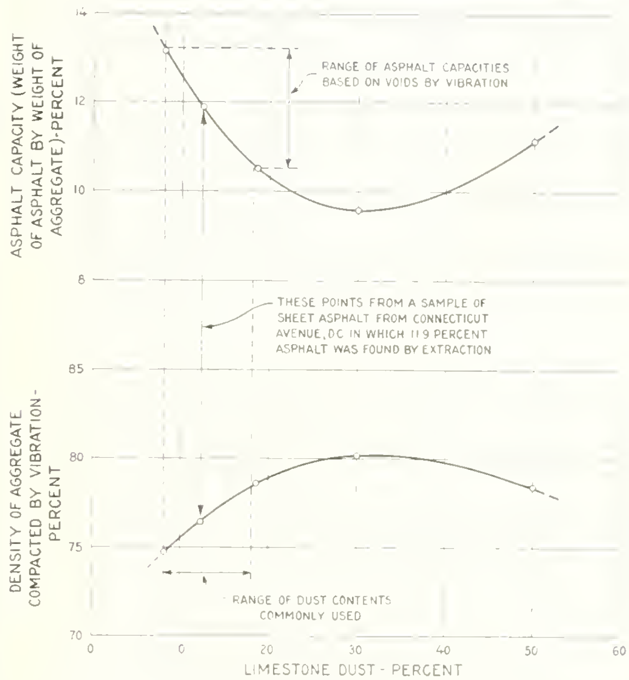

FIGURE 9.—VARIATION IN DENSITY OBTAINED BY VIBRATION AND CORRESPONDING ASPHALT CAPACITY OF SHEET ASPHALT AGGREGATES CONTAINING VARIOUS PERCENTAGES OF LIMESTONE DUST FILLER.

2. The amount of asphalt could be set also on the basis of well-established practice, and the amount of dust to be used could then be adjusted by vibratory compaction tests on a series of blends covering a narrow range of dust contents to produce an aggregate that would hold the fixed amount of asphalt.

Figure 9 illustrates the relation between asphalt capacity as determined by vibratory compaction tests and the dust content of the aggregate.

The use of the compaction test to coordinate content of bituminous materials and capacity for them appears to offer special possibilities in the design of dense surfacing mixtures where overfilling of the voids might seriously impair stability.

SUMMARY

As shown in the preceding discussion the vibratory test appears to offer valuable aid in connection with the following problems of design and construction:

1. Establishment of a definite optimum degree of compaction toward which field compaction may be aimed.

2. Determination of the best combination of two or more available aggregates for base-course or surface construction.

3. Investigation of the capacity for bituminous materials of certain aggregates to insure against over-bituminization.

4. Modification of aggregate blends to permit the use of sufficient bituminous material for workability and surface sealing without overfilling the void spaces and destroying stability.

(*Continued from page 54*)

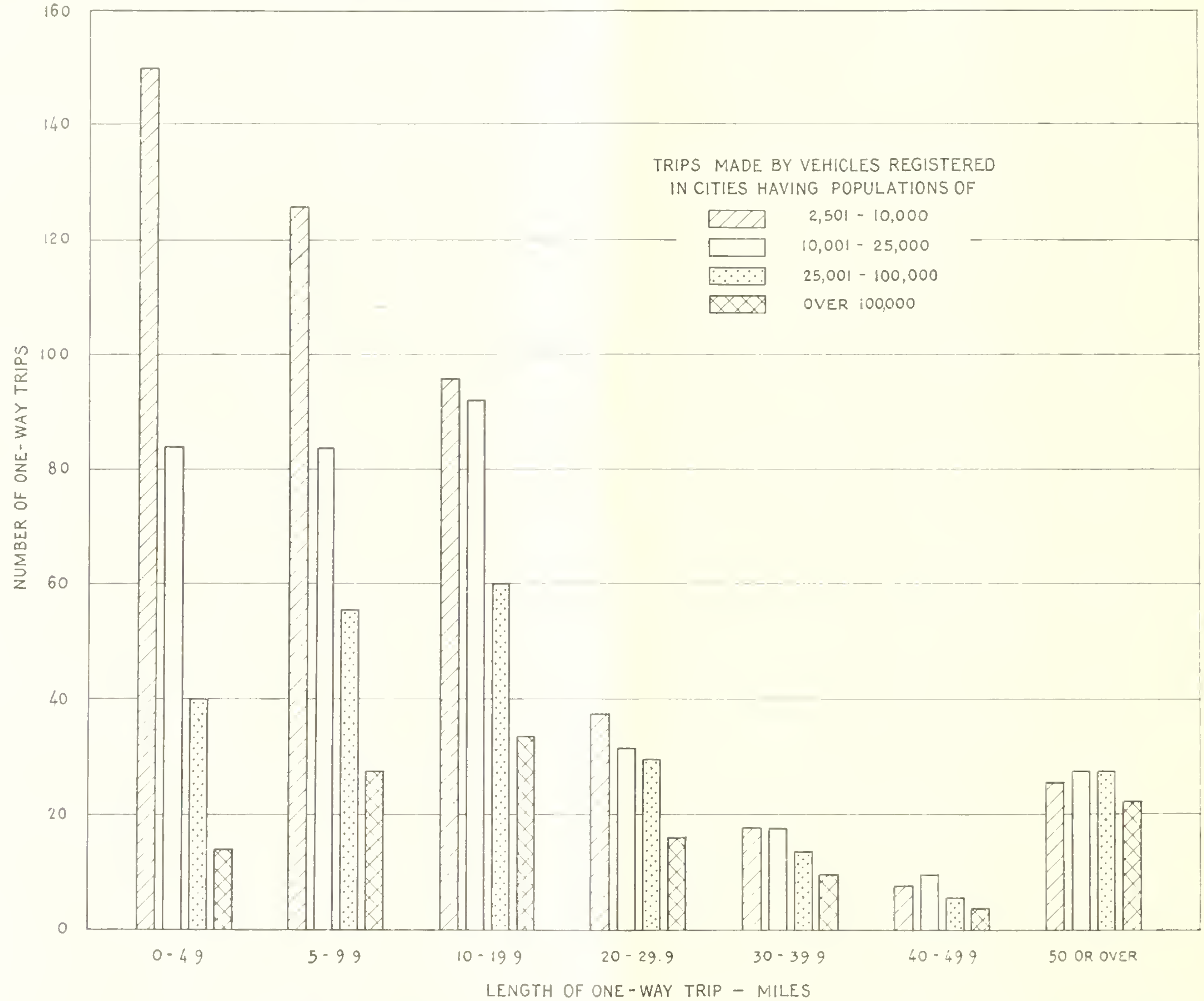

FIGURE 5.—NUMBER OF TRIPS MADE BY VEHICLES REGISTERED IN VARIOUS INCORPORATED PLACES ACCORDING TO LENGTH OF ONE-WAY TRIPS THAT WENT OUTSIDE OF CITY LIMITS.

The expenditure of motor-vehicle tax revenue on secondary highways and local roads does not create highway user benefits as widespread as those created by primary road expenditures, because these roads are used to a much smaller extent than the primary system or city streets. The use of secondary highways and local roads by residents of unincorporated areas and small towns is comparable with the use of local city streets by city residents.

The preceding data show the extensive use of motor vehicles for local travel and the self-imposed limitations on their use which results in a large percentage of their travel being performed within a surprisingly small area around their place of ownership. Accordingly, those roads radiating from centers of population are very important links in the highway system. It is apparent, then, that appreciable portions of the expenditures of motor-vehicle tax revenue on the primary system, in order to benefit the large cities properly, must be so applied as to alleviate the conditions of congestion and accompanying danger that exist within short distances of population centers.

Data on the use of rural roads and city streets and the extent of such use cannot be used alone to determine adequate plans for a highway program. Road-use data must be supplemented by data regarding the condition of existing roads, by other types of traffic data, and by financial data. For example, road-use information might point to the desirability of improving primary highway conditions in the vicinity of large cities but special traffic studies would be necessary to determine whether improvement at a particular location should consist of a by-pass route to accommodate an existing high percentage of through traffic or whether it should consist of extensions to main city thoroughfares of adequate width and design to accommodate a high percentage of local traffic together with a relatively small amount of through traffic. Studied alone, however, road-use information presents an essential picture of highway operations and a background of travel characteristics which are extremely valuable in projecting comprehensive plans for a highway system to serve the best interests of all motorists.

SUMMARY

These preliminary analyses of road-use data indicate:

1. Use of the rural road facilities by urban motorists decreases with increase in size of the city in which they reside.
2. Motorists residing in incorporated places perform 71 percent of all travel occurring on primary highways.
3. In the case of all motorists except those resident in cities of more than 100,000 population, more than half their annual travel occurs on primary highways.
4. Motor-vehicle use is largely comprised of short trips for passenger cars as well as for trucks.
5. A large amount of rural highway travel is occasioned by the travel of city motor-vehicle owners within short distances of their residences.
6. The proportional amount of such travel by urban residents decreases with increase in the size of the cities in which the vehicle owners reside.
7. Expenditures for rural highway facilities in the vicinity of cities, especially the larger ones, will provide proportionally greater benefits for urban than for rural motor-vehicle owners.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF APRIL 30, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 6,865,382	$ 3,146,870	239.4	$ 7,712,955	$ 3,845,646	282.2	$ 1,307,250	$ 650,570	50.3	$ 3,181,375
Arizona	2,283,575	1,679,245	118.2	1,231,929	874,131	48.7	350,012	223,396	13.8	1,952,195
Arkansas	1,280,707	1,266,330	86.8	3,629,785	3,625,843	228.2	343,467	340,912	15.5	1,761,085
California	10,198,246	5,559,953	240.0	5,154,134	2,804,228	72.9	1,058,749	558,364	8.1	4,717,588
Colorado	2,578,157	1,368,526	99.3	2,734,853	1,456,727	85.0	2,166,750	1,124,009	38.8	2,590,278
Connecticut	934,030	455,835	8.9	1,021,668	504,690	11.9	644,160	321,920	6.5	1,659,871
Delaware	510,437	254,021	14.2	695,941	343,194	9.9	695,890	329,690	20.0	1,418,868
Florida	2,898,007	1,414,222	64.1	2,686,486	1,343,243	53.9	283,900	141,950	6.3	3,817,233
Georgia	5,205,428	2,491,320	260.9	5,259,210	2,629,605	270.7	1,577,150	788,585	89.0	7,032,558
Idaho	2,120,236	1,202,494	200.7	1,378,664	822,939	45.0	917,503	559,414	23.5	1,682,989
Illinois	11,478,998	5,682,378	307.3	8,025,824	4,009,358	170.2	3,263,211	1,631,560	86.2	4,428,536
Indiana	6,077,173	2,948,099	154.6	3,521,276	1,760,638	69.3	3,119,099	1,455,490	56.3	3,403,782
Iowa	7,727,570	3,651,245	263.5	4,465,034	1,843,033	146.0	672,451	226,300	35.9	2,553,356
Kansas	5,172,155	2,570,857	727.9	4,238,567	2,119,283	184.5	3,855,402	1,919,996	207.3	4,266,095
Kentucky	5,556,100	2,747,346	209.2	2,750,562	1,375,281	59.0	2,348,952	1,174,674	59.9	3,294,273
Louisiana	1,516,025	753,365	38.3	11,115,551	2,642,130	42.2	1,380,075	671,501	17.4	3,100,818
Maine	2,860,227	1,388,934	65.0	1,641,459	820,728	33.6	119,110	59,555	2.0	996,965
Maryland	1,085,456	542,728	17.1	2,729,978	1,353,851	45.2	1,486,470	635,000	23.8	1,969,785
Massachusetts	1,877,599	938,724	9.0	3,392,414	1,694,505	24.5	1,451,005	721,555	12.0	2,965,358
Michigan	8,295,221	3,909,644	172.8	4,103,724	2,051,862	125.8	1,169,755	581,180	21.6	3,687,008
Minnesota	4,902,005	2,356,216	304.8	5,796,123	2,875,292	264.1	1,235,614	616,902	61.2	4,673,917
Mississippi	4,975,778	2,122,923	211.1	9,047,642	3,435,536	394.6	414,300	185,150	4.1	3,246,311
Missouri	5,764,313	2,734,844	156.9	3,260,364	1,601,196	76.2	4,309,223	2,063,220	186.7	4,980,107
Montana	1,653,927	929,612	83.6	1,928,464	1,086,932	74.9	1,808,947	1,023,955	92.5	4,835,423
Nebraska	4,119,631	1,984,688	362.0	5,631,473	2,834,730	463.6	2,848,104	1,425,550	296.6	3,004,158
Nevada	1,404,575	1,178,382	168.8	1,799,236	1,552,165	63.6	10,264	8,827	.3	1,643,938
New Hampshire	1,178,535	579,858	23.7	155,856	77,222	2.0	502,882	247,842	11.8	1,457,890
New Jersey	2,637,665	1,309,420	18.3	2,913,986	1,454,438	26.4	357,210	178,605	2.4	2,859,493
New Mexico	2,253,381	1,468,905	242.6	2,300,174	1,396,594	98.7				1,764,412
New York	14,230,225	6,738,841	253.3	11,983,150	5,866,776	192.1	1,233,200	575,600	27.2	5,172,368
North Carolina	6,763,450	3,189,099	259.6	5,992,519	2,992,402	389.6	1,453,190	692,690	69.6	2,893,893
North Dakota	3,437,527	3,233,548	260.9	434,490	243,744	57.5	1,451,312	777,848	167.3	4,405,888
Ohio	8,727,886	4,282,814	103.3	8,079,402	4,030,672	78.9	1,569,600	724,500	20.8	8,603,703
Oklahoma	7,093,181	3,720,600	254.7	1,679,375	890,385	47.7	1,487,800	791,645	46.7	4,475,169
Oregon	3,183,317	1,845,945	110.7	2,246,158	1,370,327	100.7	260,297	157,460	24.1	2,712,800
Pennsylvania	8,575,988	4,201,896	141.8	8,691,991	4,310,954	88.9	2,442,148	1,100,850	20.4	5,740,739
Rhode Island	1,179,290	589,645	16.4	390,482	195,241	3.5	331,030	165,515	3.1	1,373,648
South Carolina	5,361,442	2,369,848	266.5	2,860,644	1,276,376	86.2				2,494,519
South Dakota	2,016,762	1,128,306	246.1	4,650,859	2,571,990	453.7	1,062,780	585,650	68.0	3,750,545
Tennessee	5,741,748	2,825,832	182.2	3,626,389	1,813,916	72.0	1,831,380	915,690	59.5	4,674,044
Texas	14,143,847	6,970,620	898.4	14,871,392	7,322,071	696.6	1,775,072	861,970	116.2	7,873,543
Utah	1,121,537	749,894	107.2	2,406,783	1,710,420	80.8	282,010	208,327	14.0	1,289,982
Vermont	1,295,969	614,434	33.9	722,784	343,793	17.7	200,670	100,095	4.3	621,501
Virginia	6,022,569	3,005,159	211.4	3,155,558	1,574,219	85.8	727,688	363,844	16.4	2,106,061
Washington	4,036,518	2,070,943	99.8	3,044,447	1,593,650	38.4	308,064	159,600	8.0	1,976,533
West Virginia	1,851,632	1,309,939	66.7	1,636,172	819,011	39.1	387,070	193,535	12.4	3,057,039
Wisconsin	5,069,889	2,502,304	176.2	6,694,362	3,272,880	183.3	366,002	173,965	11.2	3,370,316
Wyoming	2,526,025	1,526,619	281.4	1,186,722	729,771	108.4	307,000	141,186	22.0	1,337,327
District of Columbia							76,000	38,000	.4	449,500
Hawaii	809,490	396,078	18.0	859,420	421,240	9.5	484,577	239,928	8.8	1,461,475
Puerto Rico	408,596	199,770	6.4	1,711,401	851,255	39.5	338,549	167,985	3.5	502,865
TOTALS	219,007,427	112,109,118	8,863.9	197,247,832	98,436,113	6,342.7	58,072,344	29,001,555	2,173.7	159,289,123

U. S. GOVERNMENT PRINTING OFFICE 1939

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF APRIL 30, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 234,900	$ 117,450	18.4	$ 834,850	$ 412,050	38.6				$ 833,746
Arizona	453,523	295,507	36.1	110,104	79,348	6.3	$ 187,308	$ 93,916	23.9	401,221
Arkansas	13,126	6,563		371,315	368,899	41.5	268,172	267,120	39.3	476,913
California	1,501,456	846,807	104.6	1,335,124	696,042	50.2				874,060
Colorado	959,959	501,483	56.3	398,410	211,194	16.8	143,050	79,530	5.2	400,549
Connecticut	69,450	34,705	1.3	46,934	23,267	.2	163,280	67,660	2.9	286,249
Delaware	22,730	11,365	5.3	80,840	40,420	17.5	56,990	28,495	7.6	239,720
Florida	20,122	10,061		516,233	257,300	12.1	287,900	143,950	14.8	463,794
Georgia	374,181	176,800	50.6	579,226	289,613	71.6	170,780	85,390	23.4	1,083,775
Idaho	448,491	203,954	46.9	191,253	102,695	13.0	35,913	19,682	3.4	331,508
Illinois	1,787,015	879,463	156.4	1,358,632	625,316	66.4	501,300	242,150	36.0	888,393
Indiana	661,294	275,217	75.8	808,750	395,375	68.7	460,097	208,283	35.1	727,450
Iowa										1,679,807
Kansas	163,438	81,718	14.8	121,922	60,961	17.3	392,736	196,368	19.0	1,367,800
Kentucky	799,835	245,084	106.1	701,106	181,576	23.0	899,303	254,970	96.2	410,817
Louisiana	144,531	67,635	11.6	657,715	283,640	52.4	361,231	167,160	31.8	421,213
Maine	368,580	180,745	23.3	259,316	124,811	12.5	27,500	13,750	2.1	145,106
Maryland				188,974	94,487	15.1	142,000	52,355	10.2	388,839
Massachusetts				149,795	74,781	2.3	599,990	296,430	11.8	481,512
Michigan	409,561	203,281	37.0	903,204	451,602	53.1	705,200	342,900	35.1	989,705
Minnesota	278,383	129,758	42.2	625,022	310,467	49.0	190,036	95,018	13.5	1,249,962
Mississippi				325,662	162,831	23.7	148,000	73,950	25.0	914,085
Missouri	420,585	201,478	53.0	594,560	256,400	48.5	506,280	240,970	90.4	814,788
Montana				76,186	43,082	4.2	449,526	252,544	34.2	1,044,267
Nebraska	582,032	287,750	96.0	690,138	336,113	126.1	126,024	63,012	26.2	617,583
Nevada	427,436	345,390	68.8	120,169	104,184	15.5	26,563	23,035	1.6	214,637
New Hampshire	218,766	108,446	6.0	60,759	29,708	2.4				181,847
New Jersey	123,040	61,520	2.4	293,890	138,210	5.5	182,553	90,995	9.3	580,253
New Mexico	625,191	381,292	42.1	546,286	332,384	35.8	137,262	79,009	7.5	253,381
New York	2,311,062	1,121,381	167.4	1,899,000	949,500	99.6				1,018,502
North Carolina	695,412	346,576	77.2	945,624	472,790	85.3	235,280	114,200	29.5	496,817
North Dakota	53,630	27,222	9.0	169,910	90,999	26.1	42,770	22,907	8.2	875,949
Ohio	147,535	73,767	3.8	273,610	143,580	10.7	524,440	262,220	29.0	1,889,552
Oklahoma	302,203	160,942	35.8	167,850	89,311	7.1	602,040	297,148	32.4	985,103
Oregon	453,217	263,260	58.5	207,851	125,392	25.3	324,711	194,880	36.0	419,825
Pennsylvania	1,765,882	835,010	125.7	1,869,669	917,053	101.3	471,430	235,715	26.0	763,166
Rhode Island	66,840	33,420	3.5	194,923	97,438	5.8	74,070	37,035	.9	93,123
South Carolina	461,400	200,382	55.8	798,427	332,669	80.2	169,800	66,200	12.4	278,661
South Dakota	11,519	6,250								1,058,050
Tennessee	259,120	129,560	14.8	766,884	310,542	34.2	137,080	68,540	.7	862,818
Texas	2,964,192	1,345,731	423.4	2,441,554	1,170,258	256.7	476,245	220,041	58.3	1,322,802
Utah	485,961	241,038	45.2	357,567	180,073	29.9	54,585	32,751	6.4	265,218
Vermont	238,385	109,790	13.8	90,306	45,153	4.0	43,300	20,500	.5	107,278
Virginia	571,443	244,241	61.4	868,214	421,640	69.2	171,950	77,767	13.6	421,080
Washington	550,138	286,426	63.7	715,043	375,696	41.1	42,369	22,300	1.2	280,706
West Virginia	242,491	119,673	21.4	153,296	76,648	8.3				515,658
Wisconsin	557,666	265,848	23.1	667,079	328,060	32.0	425,639	206,822	4.6	774,306
Wyoming	416,281	254,565	59.0	356,182	220,069	20.2	112,098	70,081	5.8	227,835
District of Columbia										73,125
Hawaii				68,130	34,065	2.4	124,850	62,425	3.5	223,510
Puerto Rico	224,621	110,876	11.3	131,605	64,530	8.8	88,818	43,395	4.3	101,199
TOTALS	23,886,623	11,829,430	2,328.8	26,089,099	12,932,222	1,837.5	11,290,469	5,533,569	878.8	31,817,263

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF APRIL 30, 1939

STATE	Completed During Current Fiscal Year: Estimated Total Cost	Completed During Current Fiscal Year: Federal Aid	Completed: Number: Grade Crossings Eliminated by Separation or Relocation	Completed: Number: Grade Crossing Structures Reconstructed	Completed: Number: Grade Crossings Protected by Signals or Otherwise	Under Construction: Estimated Total Cost	Under Construction: Federal Aid	Under Construction: Number: Grade Crossings Eliminated by Separation or Relocation	Under Construction: Number: Grade Crossing Structures Reconstructed	Under Construction: Number: Grade Crossings Protected by Signals or Otherwise	Approved for Construction: Estimated Total Cost	Approved for Construction: Federal Aid	Approved: Number: Grade Crossings Eliminated by Separation or Relocation	Approved: Number: Grade Crossing Structures Reconstructed	Approved: Number: Grade Crossings Protected by Signals or Otherwise	Balance of Funds Available for Programmed Projects
Alabama	$ 251,110	$ 250,911	6		5	$ 1,229,179	$ 1,227,324	15	1	1	$ 4,800	$ 4,800			1	$ 895,506
Arizona						229,905	227,701	3								527,973
Arkansas	492,462	490,404	10			210,157	209,877	5			104,053	104,053	2			1,298,166
California	1,362,358	1,361,783	5	3		1,726,257	1,725,162	10			80,272	80,272	1			1,261,848
Colorado	84,715	81,327	2		2	440,348	440,348	3			117,900	114,138	1		19	903,714
Connecticut						18,930	12,665				171,920	171,520		1		827,380
Delaware	35,784	35,305			14	45,420	45,420	2		9						510,525
Florida	10,616	10,616				434,894	434,894	3			79,700	79,700	1		1	1,158,058
Georgia						436,950	436,950	7			112,970	112,970	2	1	5	2,344,750
Idaho	180,246	172,543	4			280,682	249,386	4			131,898	120,500	1			442,220
Illinois	400,280	400,280	2		11	2,362,545	2,362,545	17	2	53	1,036,590	979,590	5	1	23	2,493,011
Indiana	690,037	584,751	4	4		894,116	867,216	3	1	89	169,040	169,040		1	10	1,307,641
Iowa	1,038,978	1,001,309	12	2	20	247,534	230,900	4			166,547	136,706	8		1	1,744,114
Kansas	535,159	535,054	5		5	955,333	955,333	12			164,619	164,619	2		8	1,424,461
Kentucky	145,000	145,000	1			449,315	449,315	5	1	31	434,574	386,625	6	3		1,189,951
Louisiana	11,980	11,980				435,221	428,478	4			394,361	393,570	10			1,053,899
Maine	53,997	53,877	2			329,136	329,136	3	2		147,150	147,150	1	1	1	296,231
Maryland						72,188	72,188	1			18,200	18,200			4	1,137,108
Massachusetts	54,710	54,710		1		316,093	315,372	1	3		262,260	261,410	1	1		1,690,082
Michigan	932,761	924,372	8	1	35	588,806	588,806	5	2	10	292,820	292,820	1		9	2,128,334
Minnesota	38,606	38,332		1	1	780,054	779,733	3	5	2	290,819	290,819		1	7	1,862,250
Mississippi	253,500	253,500	3			576,014	576,014	7			126,800	126,800	2			938,487
Missouri	296,960	295,421	4	1		447,800	447,800	2	1		1,026,220	959,930	4	2		1,918,286
Montana	355,586	350,704	4			870,293	870,293	9								364,726
Nebraska	162,252	156,459	6			743,908	743,908	14			618,287	618,287	16		33	593,445
Nevada	158,241	158,241		3	5	209,031	209,031	1	1		56,717	46,408			7	142,893
New Hampshire	70,205	69,765		1	2	87,856	87,797	5		1	102,775	102,302	3			331,386
New Jersey	116,891	111,665	1	1		431,161	431,161	1	2	2	151,050	151,050	1			1,665,995
New Mexico	275,206	275,206	7	1		15,276	15,276				87,240	87,240	2		1	640,726
New York	992,501	991,800	4	3	2	2,020,155	2,011,005	5	10		141,300	140,850		1	1	4,962,223
North Carolina	154,540	154,540	2	1	1	1,287,640	1,252,540	7	7	3	367,770	367,770	2	1	35	1,170,521
North Dakota	209,450	208,387		1		639,692	591,290	4	1		221,220	221,220	4			867,488
Ohio						642,800	642,800	7	1		490,180	449,180	6		2	3,971,041
Oklahoma	40,774	30,792			4	322,590	288,590	2		40	46,970	46,970			11	2,370,198
Oregon	308,391	307,742	1	2		422,059	287,701	1			129,997	129,997	2			484,121
Pennsylvania	213,129	197,923	2			1,735,939	1,524,040	4	1		255,686	255,686		1		4,883,502
Rhode Island						438,791	438,791	1	3							152,459
South Carolina	55,856	55,406	1	1	2	456,943	402,427	15	2	2	355,054	355,054	3		36	969,965
South Dakota	129,150	128,517	2		9	281,970	281,970	5	2		35,050	35,050			8	1,143,378
Tennessee	7,360	7,360			2	323,910	323,910		3	2	472,400	472,400	3		5	1,461,550
Texas	482,860	481,127	9	3		2,199,487	2,168,782	18	4		1,084,101	994,375	10			2,787,694
Utah	101,648	101,648	2		4	47,359	47,359	2			262,300	262,300			98	291,230
Vermont	243,221	229,239	6	2	7	7,406	7,406				25,490	25,300			8	316,385
Virginia	476,532	475,459	15	2	1	461,604	372,604	7	1	2	406,404	406,404	4	2	21	918,196
Washington	247,816	236,347	2	3	2	822,574	821,164	10	1		86,637	86,637		1	13	541,588
West Virginia	218,401	217,381	1		3	337,441	321,681	5		2	64,400	64,400	2		1	981,352
Wisconsin	202,131	200,987	3			1,194,012	1,153,188	11			316,850	295,726	4		4	1,309,682
Wyoming	154,992	154,992		2	4	207,460	128,040	1	1		17,010	17,010			7	508,822
District of Columbia	30,215	30,215		1							283,544	243,750	1			134,198
Hawaii						201,200	201,200	3	1		29,220	29,220	2			360,830
Puerto Rico	61,900	61,550	2			222,399	220,980	6			180,009	179,127	3			418,719
TOTALS	12,338,507	12,094,927	138	40	141	30,137,833	29,257,497	263	59	249	11,621,174	11,198,945	116	18	380	64,098,308

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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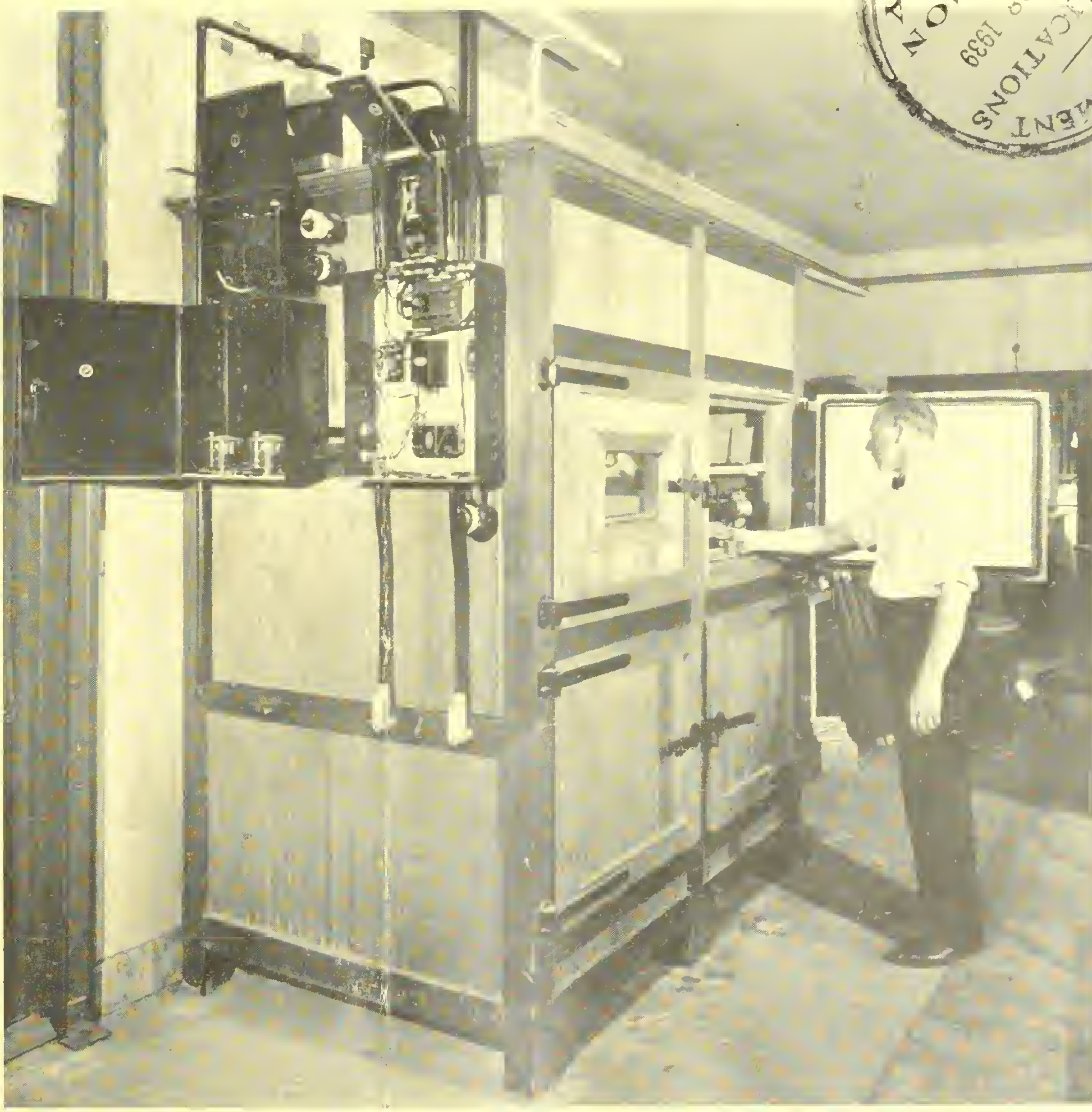

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PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

D. M. BEACH, *Editor*

Volume 20, No. 4 June 1939

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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Because of the necessarily limited edition of this publication it is impossible to distribute it free to any person or institution other than State and county officials actually engaged in planning or constructing public highways, instructors in highway engineering, and periodicals upon an exchange basis. At the present time additions to the free mailing list can be made only as vacancies occur. Those desiring to obtain Public Roads can do so by sending $1 per year (foreign subscription $1.50), or 10 cents per single copy, to the Superintendent of Documents, United States Government Printing Office, Washington, D. C.

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

TOLL ROADS AND FREE ROADS

A SPECIAL REPORT by the Bureau of Public Roads on the feasibility of constructing and operating as a toll facility a system of six transcontinental highways and on needed highway improvments was transmitted to Congress by the President on April 27, 1939, with a message recommending the report for the consideration of Congress.

The report shows that a system of transcontinental superhighways cannot be supported by tolls and will not solve any considerable part of the problem of providing adequate highway facilities.

The report was made in accordance with the Federal Aid Highway Act of 1938, approved June 8, 1938, which directed the Chief of the Bureau of Public Roads to investigate and report his findings "with respect to the feasibility of building, and cost of, superhighways not exceeding three in number, running in a general direction from the eastern to the western portion of the United States, and not exceeding three in number, running in a general direction from the northern to the southern portion of the United States, including the feasibility of a toll system on such roads."

The report states that the building of such a system is entirely feasable from a physical standpoint but the system would not come within 50 percent of being self-supporting if operated as a toll facility. The report adds, however, that a system of toll roads such as the Bureau was required to report on does not meet the most urgent highway needs, and presents a master plan designed to meet these needs.

In this plan five classes of improvement are listed. A bold attack on the congestion and delays on main arteries by constructing express highways through cities, belt-line distribution routes around them, and bypasses around small towns, is proposed. It is also proposed to create a national system of interregional highways, approximately 27,000 miles in extent, by modernizing and improving existing routes of travel and building new roads where necessary to provide more direct travel.

In studying the feasibility of a toll system, the Bureau selected six routes, located in accordance with the terms of the act, aggregating 14,336 miles. Its detailed studies show that the cost of constructing this system for fast-moving traffic, without crossing other highways or railroads at grade, would be about $2,899,800,000, which is at the average rate of $202,270 per mile.

The average estimated annual expenditure for the period 1945–60, required for financing the construction, maintaining the property, and operating the facility would be $184,054,000, which is at the average rate of $12,840 per mile per year.

Estimates of the potential traffic on the proposed toll system were based on actual traffic counts on the main highways of the country and studies of the character of highway travel. A number of facts led to the conclusion that only a small portion of present traffic could be attracted to the toll system. Long-distance travel constitutes only a small fraction of the total travel. Counts made on east-west highways at stations established on a line extending from Canada to Mexico showed only 300 passenger vehicles crossing the line daily in transcontinental travel. The system could be expected to serve adequately only that portion of the traffic having origin and destination close to one of the six routes. Access to the highways would have to be controlled both for collection of tolls and to prevent interference with flow of traffic by entering vehicles. Vehicles traveling distances less than the spacing of access points would not use the toll roads.

It is estimated that the utilization of the system would average, during the period 1945–60, 12,450,000 vehicle-miles per day. Assuming toll charges of 1 cent per vehicle-mile for passenger vehicles and an average of 3.5 cents per vehicle-mile for trucks and busses, this

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TO THE CONGRESS OF THE UNITED STATES:

I transmit herewith a letter from the Secretary of Agriculture, concurred in by the Secretary of War, enclosing a report of the Bureau of Public Roads, United States Department of Agriculture, on the Feasibility of a System of Transcontinental Toll Roads and a Master Plan for Free Highway Development.

The report, prepared at the request of the Congress, is the first complete assembly of data on the use being made of our national highway network. It points definitely to the corrective measures of greatest urgency and shows that existing improvements may be fully utilized in meeting ultimate highway needs.

It emphasizes the need of a special system of direct interregional highways, with all necessary connections through and around cities, designed to meet the requirements of the national defense and the needs of a growing peace-time traffic of longer range.

It shows that there is need for superhighways, but makes it clear that this need exists only where there is congestion on the existing roads, and mainly in metropolitan areas. Improved facilities, needed for the solution of city street congestion, are shown to occupy a fundamental place in the general replanning of the cities indicated as necessary in the report "Our Cities", issued in September 1937 by the National Resources Committee.

The report also points definitely to difficulties of right-of-way acquisition as obstacles to a proper development of both rural highways and city streets, and makes important and useful recommendations for dealing with these difficulties.

I call the special attention of the Congress to the discussion of the principle of "excess-taking" of land for highways. I lay great emphasis on this because by adopting the principle of "excess-taking" of land, the ultimate cost to the Government of a great national system of highways will be greatly reduced.

For instance, we all know that it is largely a matter of chance if a new highway is located through one man's land and misses another man's land a few miles away. Yet the man who, by good fortune, sells a narrow right-of-way for a new highway makes, in most cases, a handsome profit through the increase in value of all of the rest of his land. That represents an unearned increment of profit—a profit which comes to a mere handful of lucky citizens and which is denied to the vast majority.

Under the exercise of the principle of "excess-taking" of land, the Government, which puts up the cost of the highway, buys a wide strip on each side of the highway itself, uses it for the rental of concessions and sells it off over a period of years to home builders and others who wish to live near a main artery of travel. Thus the Government gets the unearned increment and reimburses itself in large part for the building of the road.

In its full discussion of the whole highway problem and the wealth of exact data it supplies, the report indicates the broad outlines of what might be regarded as a master plan for the development of all of the highway and street facilities of the Nation.

I recommend the report for the consideration of the Congress as a basis for needed action to solve our highway problems.

FRANKLIN D. ROOSEVELT

THE WHITE HOUSE,
April 27, 1939.

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travel would produce an average annual revenue of $72,140,000. This is considerably less than the $184,054,000 estimated average annual cost and leads the Bureau to conclude that the system studied could not be supported by toll collections.

The portion of the proposed system estimated to be most nearly self-supporting is the 172 miles from a point near Philadelphia, Pa., to a point near New Haven, Conn. With the increase in traffic expected by 1960, this portion of the system would earn slightly more than the estimated cost for that year.

The report states, "If, as an actual test of the feasibility of a limited mileage of toll roads, it is the desire of the Congress to make provision for the construction of a section of highway of substantial length upon which there is a reasonable prospect of the recovery of the costs through tolls, it is recommended that such provision be made applicable to a section of highway, properly located, and extending from an appropriate point near Washington, D. C., to an appropriate point near Boston, Mass."

The report recommends the construction of a special system of direct interregional highways, with all necessary connections through and around cities, designed to meet the requirements of the national defense in time of war and the needs of a growing peace-time traffic of longer range. A system of such roads, including 26,700 miles, has been tentatively selected on the basis of the detailed traffic data available. Existing main highways can be modernized to form a large part of the system but some new highways will be needed to provide directness of travel. Although these roads represent less than 1 percent of the total mileage of rural roads, the Bureau estimates they would serve, when improved as indicated, at least 12.5 percent of the total vehicle travel on rural highways.

More complete information on the character of traffic in and near cities than has heretofore been available is presented. Traffic maps in the report show that about 90 percent of the traffic on main highways near the entrances to large cities is bound to or from points in cities themselves and cannot be bypassed around them. It is found also that a large part of the traffic is destined to or bound from points in the very heart of the city or points most conveniently reached by going through the center of the city.

There is great need, the report indicates, for express highways cut directly into and through the center of the big cities. These are needed not only for service of the through traffic delivered by the main rural highways but also for the daily in-and-out movement of local traffic between the downtown section and suburbs centering about the main highways at the periphery of the city.

The West Side Highway and Henry Hudson Parkway in New York, and the recently constructed express highway in St. Louis are cited as early examples of such facilities. The provision of similar facilities in Pittsburgh is now receiving serious consideration.

By preference such express highways should be constructed as attractively landscaped depressed thoroughfares passing under all cross streets.

Bypasses—the remedy usually proposed for the relief of congestion on through streets in cities—are said to be only a partial and, by themselves, a not very effective remedy. They are recommended around the smaller towns and a new type of belt-line distribution road around cities is proposed. For maximum effectiveness, both the bypass and distribution highways must be free of cross traffic, parked vehicles and developments immediately adjacent, to preserve their initial advantage against the encroaching growth of the urban community, which otherwise soon converts them into ordinary local streets.

Outside of city limits on the main highways the report shows there is need of modernization of the existing roads to ease curvature, reduce gradients, and extend sight distance in order more safely to serve fast-moving traffic. Near the cities, also, a steadily increasing mileage of four-lane divided highways is believed to be required.

According to the report, such improvements are required on most of the mileage of the Federal-aid and State highway systems, especially those parts built before the recent considerable increase in the travel speed of motor vehicles. For the most part they involve only local changes in the existing roads. By such changes the bulk of the highway traffic that moves between adjacent cities will be amply served.

The report sketches the general outlines of a Master Plan for the improvement of roads and streets to meet the real needs of highway transportation. In addition to the several classes of improvements previously mentioned, the plan includes improvement of a carefully selected mileage of secondary and feeder roads to give direct service to a larger number of rural dwellers. The selection would be made from among the 2,618,000 miles of roads outside of the Federal-aid and primary State highway systems. Constituting about 83 percent of the country's total road and street mileage, these lesser roads serve at present only about 13 percent of the total vehicle mileage of traffic. Located on them, however, are the homes and working places of about 75 percent of the rural population. The purpose of the improvement of an additional mileage of these roads, therefore, is shown to be that of affording better access to rural property rather than the service of a large increment of traffic. The choice of the roads to be improved should be made in close conformity with a program looking to the promotion of economically and socially beneficial land use.

The report discusses at length the limitations hitherto placed upon road improvement by difficulties of right-of-way acquisition, and shows that similar difficulties are now the principal obstacle standing in the way of needed improvements of the several types described, especially within and in the vicinity of cities.

Taking the city of Baltimore as an example of a universal condition, it shows, by spotting the location of properties on which the city holds tax liens and properties being acquired for Federal slum clearance projects, that a wide belt of decadent property surrounds the central business section. Decay of values within this zone (the result of the outward movement of the homes of the more well-to-do citizens) is rapidly approaching a critical point. Creation of new values is beginning to occur, generally without regard to any well-conceived future street plan. In Baltimore, proposed slum clearance projects are shown to lie in the path of desirable express highway locations. All of which indicates the great importance of early consideration of the new street plans which must form the framework upon which the cities of the future will be erected. It also indicates the need and present timeliness of effective measures for the acquisition of land in the

(Continued on page 75)

TESTS OF CONCRETE CURING MATERIALS

BY THE DIVISION OF TESTS, BUREAU OF PUBLIC ROADS

Reported by F. H. JACKSON, Senior Engineer of Tests, and W. F. KELLERMANN, Associate Materials Engineer

NUMEROUS methods and procedures used for curing concrete pavements are included in the scope of the investigation herein reported. However, the investigation did not include the use of cotton mats which have proven highly effective as a curing medium, not only on account of their ability to retain moisture over a considerable period of time but also because they protect the concrete from large fluctuations in temperature at early ages when its ability to resist temperature stresses is low.

The reason for the omission of cotton mats was twofold. First, the investigation did not involve any study of thermal insulation but was for the purpose of determining the ability of various curing agents to retain moisture and thereby promote the development of strength. In the burlap curing that was used as a basis for the comparison of the other methods, the burlap was kept in a saturated condition at all times and therefore, insofar as moisture loss is concerned, the results were the same as would have been obtained with saturated cotton mats. Second, the technical and practical advantages of cotton mats have been demonstrated conclusively by previous investigations, both in the laboratory and in the field.

Thorough and complete curing has always been recognized as one of the most important single factors involved in the construction of a concrete pavement. The importance of delaying moisture loss until the concrete has attained sufficient strength to furnish high resistance to the shrinkage stresses resulting from drying is self-evident. For this reason, provisions for curing form a very important part of every concrete pavement specification.

For many years concrete pavements were cured almost entirely by means of a thorough and continuous application of water for periods up to 10 days after placing. Curing began by covering the concrete with wet burlap applied just as soon after finishing as possible. This was kept continuously wet until the following day when it was replaced by a covering of earth or straw kept continuously wet for periods of from 7 to 10 days. It has always been pretty generally agreed that, theoretically at least, the above method is ideal. However, it requires continuous wetting over a considerable period of time, a procedure which is not only expensive but requires constant and efficient supervision to insure full compliance.

TESTS MADE TO COMPARE CURING MATERIALS AND TO DEVELOP STANDARD TEST PROCEDURE

So long as only small amounts of pavement were involved and daily yardages were limited, it was possible to enforce such curing provisions without great difficulty. However, as methods of construction became more efficient, and daily yardages increased, the cost of curing by water as well as the difficulty of enforcing the requirements mounted rapidly. As was bound to happen, this condition has resulted within the last several years in the introduction of numerous substitute methods of curing, designed to accomplish the same purpose without the use of water. Most of these methods, involving the use of such materials as various grades of waterproof paper coverings, sodium silicate, liquid bituminous products, rubber emulsion, etc., depend entirely for their efficiency on the ability of the materials to retain water within the concrete. The materials seal the surface and their use is justified on the theory that adequate curing can be accomplished by retaining the contained water.

Many attempts have been made from time to time to study the effectiveness of different methods of curing concrete through the construction of experimental roads, curing different sections by different methods. Such procedure would seem to be a very logical method of ascertaining the comparative value of different curing materials. Actually, however, the impossibility of controlling other variables that may affect the result, particularly weather conditions, make it of distinctly questionable value.

There are many problems regarding concrete pavement construction that may be studied with profit through the construction of experimental roads. However, in the authors' opinion curing is not one of them. It is believed that such comparisons should be made only in the laboratory under closely controlled temperature and humidity conditions, using a test procedure that will permit direct comparisons of the efficiency of different curing materials. Having determined, by means of a series of tests of this type, the degree of compliance that may reasonably be expected, suitable requirements could be written into standard specifications and the test procedure used as a standard routine laboratory method of evaluating the various materials and processes offered for use.

The tests reported herein were made with the twofold purpose of obtaining comparative data on the effectiveness of various curing materials and methods now in common use and of developing a standard laboratory test procedure for use in specifications. The procedure followed has been made available to Committee C–9 of the A. S. T. M. in developing a tentative method for testing curing agents. The curing materials that were investigated included, in addition to burlap, calcium chloride, used both as a surface application and as an admixture; sodium silicate; six waterproof papers; a special curing blanket consisting of two layers of burlap with a jute bat between; an asphalt emulsion; an asphalt cutback; a straw-colored lacquerlike liquid; and a rubber (latex) emulsion. The last four materials were proprietary liquid curing compounds applied in the form of a spray. A brief description of each of the materials investigated is given in table 1.

STUDIES MADE OF 38 DIFFERENT CURING PROCEDURES INVOLVING 14 MATERIALS

Several of the surface-sealing materials were used both with and without a preliminary 24-hour application of wet burlap. In addition, the time elapsing between the molding of the specimen and the application of the curing material was varied. The comparative effects of burlap curing for 1, 2, and 3 days without subsequent curing were also investigated. In all, 38 different curing procedures involving 14 materials were studied, the results being compared with the results obtained with specimens cured continuously with wet burlap in sealed containers (ideal curing) as well as with specimens exposed to the air without protection of any kind. To provide ideal curing the specimens were first covered with two layers of wet burlap. Over this was placed a metal cover that was sealed around the edges to prevent any loss of moisture.

Table 1.—*Description of curing materials*

Type	Description
Burlap	Weight, 9 ounces per square yard.
Paper A	2 layers of paper cemented together with bitumen and reinforced with sisal fibers.
Paper B	2 layers of paper, reinforced in both directions at about ½-inch intervals and cemented together with bitumen, bitumen applied to 1 layer only.
Paper C	Same as paper B, except bitumen applied to both layers of paper.
Paper D	Single layer of unreinforced paper, treated with a white emulsion.
Paper E	Same as paper D, except treated with a brown emulsion.
Paper F	Same as paper D, except treated with a brown-white emulsion
Sodium silicate	Commercial grade as used for curing concrete. Applied with a brush.
Calcium chloride	Standard commercial product (flake) as used for curing concrete.
Curing blanket	Consists of 2 layers of burlap with jute bat between. Weight 22 ounces per square yard.
Liquid curing material A.[1]	Special asphalt emulsion used for curing concrete. Applied with a spray gun.
Liquid curing material B.[1]	Special asphalt cut-back used for curing concrete. Applied with a spray gun.
Liquid curing material C.[1]	A straw-colored lacquerlike liquid. Applied with a spray gun
Liquid curing material D.[1]	A rubber emulsion (latex). Applied with a spray gun.

[1] The liquid curing materials are all proprietary compounds, the exact composition of which was not determined.

The results of five series of tests, four after 7 days of exposure under the temperature and humidity conditions described below, and one after 28 days of exposure, are reported. A brief description of each procedure, including the type of curing material involved, whether used with or without an initial application of burlap, the time of application, and the duration of application, is shown in table 2. This table also indicates the series in which each procedure was used.

In series A to D, inclusive, the specimens were exposed for 7 days in an atmosphere maintained at 100° F.±2° F. with a relative humidity of 32 percent ±2 percent, using for this purpose a specially designed curing cabinet in which the temperature and humidity were controlled automatically within the limits indicated. In series E, the specimens were exposed for 28 days. Each result reported in series A to D, inclusive, with certain exceptions noted in series B, is the average of either five or six individual determinations made on different days. The results of the tests after 28 days, series E, are the averages for from two to five specimens, as noted in subsequent tables.

In series A, 19 methods in addition to the standard or ideal method and the method involving no curing treatment, were investigated. These included burlap for 1, 2, and 3 days; paper A and liquid curing material A with and without burlap; the other papers and liquid curing materials without burlap, that is, as recommended by the manufacturers; calcium chloride, both as a surface application and as an admixture; sodium silicate; and the curing blanket. It will be noted that in this series, surface sealing materials when used without burlap were applied 3 hours after molding. This would represent about the maximum time that might be required in the field. Burlap in this series, however, was applied immediately after molding.

Table 2.—*Description of curing procedures*

Method No.	Curing procedure	Used in series A	B	C	D	E
1a	Wet burlap, sealed with metal cover continuously	x	x	x	x	x
2	No treatment	x	x		x	x
3a	Wet burlap for 1 day, applied immediately after molding	x			x	x
4a	Wet burlap for 2 days, applied immediately after molding	x	x		x	x
4b	Wet burlap for 2 days, applied 1 hour after molding		x			
4c	Wet burlap for 2 days, applied 3 hours after molding		x			x
4c-1	Wet burlap for 2 days, applied 3 hours after molding [1]		x			
4c-2	Wet burlap for 2 days, applied 3 hours after molding [2]		x			
5a	Wet burlap for 3 days, applied immediately after molding	x	x		x	x
5b	Wet burlap for 3 days, applied 1 hour after molding		x			
5c	Wet burlap for 3 days, applied 3 hours after molding		x			x
5c-1	Wet burlap for 3 days, applied 3 hours after molding [1]		x			
5c-2	Wet burlap for 3 days, applied 3 hours after molding [2]		x			
6a	Wet burlap for 1 day, applied immediately after molding followed by paper A for 6 days	x	x		x	x
6b	Wet burlap for 1 day, applied 1 hour after molding, followed by paper A for 6 days		x			
6c	Wet burlap for 1 day, applied 3 hours after molding, followed by paper A for 6 days		x	x		x
7a	Wet burlap for 1 day, applied immediately after molding, followed by sodium silicate	x			x	x
7c	Wet burlap for 1 day, applied 3 hours after molding, followed by sodium silicate					x
8a	Wet burlap for 1 day, applied immediately after molding, followed by calcium chloride (surface application)	x			x	x
8c	Wet burlap for 1 day, applied 3 hours after molding, followed by calcium chloride (surface application)					x
9a	Wet burlap for 1 day, applied immediately after molding, followed by liquid curing material A	x			x	x
9c	Wet burlap for 1 day, applied 3 hours after molding, followed by liquid curing material A			x		x
10c	Wet burlap for 1 day, applied 3 hours after molding, followed by liquid curing material B			x		
11a	Wet burlap for 1 day, applied immediately after molding, calcium chloride admixture	x			x	x
11c	Wet burlap for 1 day, applied 3 hours after molding, calcium chloride admixture					x
12c	Curing blanket, for 3 days, applied 3 hours after molding	x			x	x
13b	Waterproof paper A, for 7 days, applied 1 hour after molding		x			
13c	Waterproof paper A, for 7 days, applied 3 hours after molding	x	x	x	x	x
14c	Waterproof paper B for 7 days, applied 3 hours after molding	x			x	x
15c	Waterproof paper C for 7 days, applied 3 hours after molding	x			x	x
16c	Waterproof paper D, for 7 days, applied 3 hours after molding	x			x	x
17c	Waterproof paper E, for 7 days, applied 3 hours after molding	x			x	x
18c	Waterproof paper F, for 7 days, applied 3 hours after molding	x			x	x
19b	Liquid curing material A, applied 1 hour after molding		x			
19c	Liquid curing material A, applied 3 hours after molding	x	x	x	x	x
20b	Liquid curing material B, applied 1 hour after molding		x			
20c	Liquid curing material B, applied 3 hours after molding	x	x	x	x	x
21b	Liquid curing material C, applied 1 hour after molding		x			
21c	Liquid curing material C, applied 3 hours after molding	x	x		x	x
22c	Liquid curing material D, applied 3 hours after molding	x			x	x

[1] The burlap was sprinkled intermittently in such manner as to keep it continuously wet.
[2] The burlap was sprinkled intermittently and allowed to become dry between wettings.

SPECIMENS SEALED TO PERMIT MOISTURE LOSS ONLY THROUGH CURING MEDIUM

Series D was a duplication of series A, run several months later. In series B the effect of varying the time of application of the curing agent was investigated. For burlap, the effects of delaying the application 1 hour and 3 hours are shown as well as the effects of continuous sprinkling and of intermittent sprinkling. The relative effects of applying paper A and liquid curing materials A, B, and C, 1 hour after molding, as well as 3 hours, are also shown. Series C was run in order to obtain additional data on the effect of using waterproof paper and liquid curing material with a preliminary 24-hour application of wet burlap for comparison with the usual method which does not require burlap. Series E gives the results of a series of tests after 28 days of exposure, using, in general, the same methods as used in series A and D in which the specimens were exposed 7 days.

The effectiveness of each curing method was measured both in terms of relative moisture loss and relative strength, using test specimens of 1 : 2 mortar, 6½ inches wide by 12 inches long by 2 inches deep. The curing material was applied to the top or molded surface of the specimen, and sealed around the edges in such a manner that moisture could escape only through the curing medium itself. The rate of moisture loss was measured for each method by determining the loss in weight at various intervals during the exposure period.

The specimens were molded in watertight sheet metal pans, the bottoms of which were reinforced with angle sections for a stiffening effect. This was done because it was frequently necessary to handle them at about the time initial set was taking place and it was felt that molds that were not rigid might allow stresses to be set up within the specimen.

The procedure followed in fabricating the specimens was to mix just sufficient mortar for one test specimen at a time. A well-graded concrete sand was used in the mortar together with sufficient water to produce a plastic consistency. In each series, the water-cement ratio was maintained constant. However, because of slight differences in grading of sand used in the different series, it was necessary to vary slightly the water-cement ratio from series to series. The maximum difference did not exceed 0.02 by weight. The mortar was puddled into the molds with the gloved fingers, after which the surface was struck off with a single stroke of a steel blade. No troweling was done. Immediately after molding, the specimens were weighed, these weights being taken as the initial weights from which the moisture losses were computed.

In all instances where burlap was applied immediately after molding (except where it was sealed in as in method 1), the specimens were not placed immediately in the humidity-controlled curing cabinet, but were placed in an oven maintained at 100° F. ±2° F. but without humidity control. Where burlap was applied 1 or 3 hours after molding, the specimens were placed in the humidity-controlled cabinet until the burlap was applied after which they were placed in the oven. This was necessary because the procedure for burlap curing required that the material be kept saturated for the entire time of application. This would have made it impossible to maintain a constant humidity in the cabinet.

The burlap cover was kept saturated by immersing an overhanging end in a pan of water. It was found that in order to insure even and continuous saturation over the entire surface of the specimen it was necessary to use three layers of burlap. This method of keeping the specimens wet was used in order to avoid the necessity of opening the oven doors frequently for the purpose of sprinkling.

RELATIVE EFFICIENCY OF EACH CURING METHOD DETERMINED

At the conclusion of the period of burlap curing the specimens were removed from the oven, the burlap removed and the specimens immediately placed in the curing cabinet (where burlap curing only was involved) or covered with the final curing material and then placed in the cabinet.

In instances where burlap curing was not involved, the procedure was to place the specimen in the humidity-controlled cabinet immediately after molding. After the concrete had set, or after passage of a prescribed interval of time, the specimen was removed for the purpose of applying the curing material, after which it was replaced in the cabinet for the duration of the test.

At the conclusion of the exposure period the specimens were removed from the cabinet, the molds removed and the specimens immersed in water for 2 days prior to testing for flexural strength. For series A to D, inclusive, the age at test was therefore 9 days whereas for series E it was 30 days. To facilitate absorption of water, the upper and lower surfaces of each specimen

Table 3.—*Series A; results of tests after 7 days* [1]

Method No.	Type of curing	Procedure				Water remaining in specimens at age indicated [2]						Flexural strength	Relative efficiency, based on—	
		Burlap		Final curing material										
		Applied after molding	Duration of application	Applied after molding	Duration of application	3 hours	1 day	2 days	3 days	4 days	7 days		Water loss	Strength
		Hours	*Days*	*Hours*	*Days*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Lb. per sq. in.*		
1a	Burlap	0	7								102	1,018	100	100
2	None					97	79	77	76	74	73	561	0	0
3a	Burlap	0	1				101	95	93	92	90	861	59	66
4a	do	0	2					102	98	96	94	988	72	93
5a	do	0	3						102	99	97	996	83	95
6a	Paper A with burlap	0	1	24	6		101	101	100	100	100	1,025	93	102
7a	Sodium silicate with burlap	0	1	24	6		101	98	96	95	94	908	72	76
8a	Calcium chloride with burlap	0	1	24	6		101	96	95	94	90	1,022	59	101
9a	Liquid material A with burlap	0	1	24	6		101	101	100	100	100	1,055	93	108
11a	Calcium chloride admixture with burlap	0	1	(3)	(3)		101	97	96	95	92	947	66	84
12c	Curing blanket			3	3	96			90	89	86	740	45	39
13c	Paper A			3	7	96	96	96	95	95	95	769	76	46
14c	Paper B			3	7	97	96	95	94	94	94	768	72	45
15c	Paper C			3	7	97	97	97	96	95	95	808	76	54
16c	Paper D			3	7	96	84	82	80	80	77	571	14	2
17c	Paper E			3	7	96	84	82	81	80	78	634	17	16
18c	Paper F			3	7	95	84	82	80	79	77	610	14	11
19c	Liquid material A			3	7	95	96	96	96	95	94	791	72	50
20c	Liquid material B			3	7	96	95	95	93	93	92	762	66	44
21c	Liquid material C			3	7	96	90	89	88	87	85	673	41	25
22c	Liquid material D			3	7	96	94	93	93	92	92	724	66	36

[1] All results average of 5 tests. [2] Based on total water in specimens after molding. [3] Mixing water contained 2 percent calcium chloride.

were rubbed with a carborundum stone prior to immersion. This procedure was followed in an effort to place all specimens in a uniform condition, insofar as contained moisture was concerned, prior to test. As will be discussed in detail later, this apparently was not accomplished under all conditions, possibly accounting for certain discrepancies in the strength results that were observed.

Flexure tests were made at the conclusion of the 2-day resaturation period, the load being applied at the center of a 9-inch span, with the top surface as molded in tension.

The rate at which specimens cured by the different methods gave up water at various periods from time of molding up to and including 7 days of exposure, the flexural strengths at 9 days, and the "relative efficiency," from the standpoint of both water retention and strength, are shown in tables 3 to 6, inclusive; except that table 5 (series C) contains no data on relative efficiency because in this series no values were obtained on the specimens receiving no curing treatment (method no. 2). Corresponding values for 28-day exposure are shown in table 7.

Relative efficiency as used in this report is a value that represents the comparative effectiveness of the particular method involved on the basis of 100 for specimens cured by the ideal method (method 1a) and 0 for specimens receiving no curing treatment (method 2). Thus, in series A, table 3, the strength of the ideally cured specimens averaged 1,018 pounds per square inch, whereas, the specimens receiving no curing treatment averaged 561 pounds per square inch. The difference, 457 pounds per square inch, may be considered as representing the gain in strength that was attained through ideal curing. On this basis, method 3a, 24-hour burlap curing, with a strength of 861 pounds per square inch, had a relative efficiency of 66. Values for relative efficiency based on water loss were computed in the same manner.

Thus, from table 3 it will be noted that the specimens given no curing treatment (method 2) averaged 27 percent moisture loss at 7 days, whereas, the specimens

TABLE 4.—*Series B; results of tests after 7 days* [1]

Method No.	Type of curing	Procedure				Water remaining in specimens at age indicated [2]							Flexural strength	Relative efficiency based on—	
		Burlap		Final curing material											
		Applied after molding	Duration of application	Applied after molding	Duration of application	1 hour	3 hours	1 day	2 days	3 days	4 days	7 days		Water loss	Strength
		Hours	*Days*	*Hours*	*Days*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Lb. per sq. in.*		
1a	Burlap	0	7									103	924	100	100
2	None							73	71	69	68	65	574	0	0
4a	Burlap	0	2						102	96	94	91	879	68	87
4b	do	1	2			99			99	92	90	87	841	58	76
4c	do	3	2				96		103	95	93	90	827	66	72
[3] 4c-1	Burlap sprinkled intermittently	3	2				96		99	95	93	90	798	66	66
[4] 4c-2	do	3	2				96		90	86	85	82	714	45	37
5a	Burlap	0	3							104	99	95	910	79	96
5b	do	1	3			99				99	94	90	907	66	95
5c	do	3	3				96			102	96	92	859	71	81
[3] 5c-1	Burlap sprinkled intermittently	3	3				97			99	94	91	885	68	92
[4] 5c-2	do	3	3				96			90	89	85	698	53	32
6a	Paper A with burlap	0	1	24	6			102	102	101	101	100	894	92	91
6b	do	1	1	24	6	99		99	99	99	98	97	956	84	109
6c	do	3	1	24	6		96	103	102	101	101	100	925	92	100
13b	Paper A			1	7	99		97	96	96	95	94	855	76	80
13c	do			3	7		96	95	95	94	94	93	838	74	75
19b	Liquid material A			1	7	99		92	89	88	87	85	773	53	57
19c	do			3	7		96	95	95	94	94	93	849	74	79
20b	Liquid material B			1	7	99		92	90	88	87	84	807	50	67
20c	do			3	7		95	94	94	93	92	92	919	71	99
21b	Liquid material C			1	7	99		85	82	80	79	76	657	29	24
21c	do			3	7		95	88	86	84	83	81	733	42	45

[1] All results average of 5 tests except as noted.
[2] Based on total water in specimens after molding.
[3] Average of 3 tests. Burlap kept constantly in a moist condition.
[4] Average of 2 tests. Burlap allowed to become practically dry before each sprinkling.

TABLE 5.—*Series C; results of tests after 7 days* [1]

Method No.	Type of curing	Procedure				Water remaining in specimens at age indicated [2]						Flexural strength
		Burlap		Final curing material								
		Applied after molding	Duration of application	Applied after molding	Duration of application	3 hours	1 day	2 days	3 days	4 days	7 days	
		Hours	*Days*	*Hours*	*Days*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Lb. per sq. in.*
1a	Burlap	0	7								104	843
6c	Paper A with burlap	3	1	24	6	96	102	102	102	102	100	810
9c	Liquid material A with burlap	3	1	24	6	96	104	103	102	102	100	823
10c	Liquid material B with burlap	3	1	24	6	95	102	101	101	100	98	826
13c	Paper A			3	7	95	94	94	94	93	92	786
19c	Liquid material A			3	7	95	95	95	94	94	94	818
20c	Liquid material B			3	7	95	94	93	92	92	90	786

[1] All results average of 6 tests.
[2] Based on total water in specimens after molding.

TABLE 6.—*Series D; results of tests after 7 days* [1]

Method No.	Type of curing	Procedure: Burlap, Applied after molding	Procedure: Burlap, Duration of application	Procedure: Final curing material, Applied after molding	Procedure: Final curing material, Duration of application	Water remaining in specimens at age indicated [2]: 3 hours	1 day	2 days	3 days	4 days	7 days	Flexural strength	Relative efficiency based on—: Water loss	Relative efficiency based on—: Strength
		Hours	*Days*	*Hours*	*Days*	*Percent*	*Per cent*	*Percent*	*Percent*	*Percent*	*Percent*	*Lb. per sq. in.*		
1a	Burlap	0	7								104	897	100	100
2	None					96	76	73	71	71	67	597	0	0
3a	Burlap	0	1				103	94	91	90	86	808	51	70
4a	do	0	2					104	98	96	92	875	68	93
5a	do	0	3						104	99	95	909	76	104
6a	Paper A with burlap	0	1	24	6		103	102	102	102	100	943	89	115
7a	Sodium silicate with burlap	0	1	24	6		103	96	94	92	89	865	59	89
8a	Calcium chloride with burlap	0	1	24	6		100	95	93	91	88	867	57	90
9a	Liquid material A with burlap	0	1	24	6		102	101	101	101	99	943	86	115
11a	Calcium chloride admixture with burlap	0	1	(3)	(3)		103	95	93	91	88	845	57	83
12c	Curing blanket			3	3	96			89	87	84	741	46	48
13c	Paper A			3	7	96	95	95	94	94	93	785	70	63
14c	Paper B			3	7	95	95	94	94	94	93	780	70	61
15c	Paper C			3	7	96	95	95	95	95	94	805	73	69
16c	Paper D			3	7	96	81	78	76	75	71	623	11	9
17c	Paper E			3	7	95	83	80	78	77	74	621	19	8
18c	Paper F			3	7	95	83	79	78	76	73	600	16	1
19c	Liquid material A			3	7	96	96	95	95	95	94	822	73	75
20c	Liquid material B			3	7	96	94	93	92	92	90	759	62	54
21c	Liquid material C			3	7	96	92	89	88	87	84	678	46	27
22c	Liquid material D			3	7	95	93	91	91	90	89	719	59	41

[1] All results average of 5 tests.
[2] Based on total water in specimens after molding.
[3] Mixing water contained 2 percent calcium chloride.

TABLE 7.—*Series E; results of tests after 28 days* [1]

Method No.	Type of curing	Procedure: Burlap, Applied after molding	Procedure: Burlap, Duration of application	Procedure: Final curing material, Applied after molding	Procedure: Final curing material, Duration of application	Water remaining in specimens at age indicated [2]: 3 hours	1 day	2 days	3 days	4 days	7 days	14 days	21 days	28 days	Flexural strength	Relative efficiency based on—: Water loss	Relative efficiency based on—: Strength [3]
		Hours	*Days*	*Hours*	*Days*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Lb. per sq. in.*		
1a	Burlap	0	28											102	1,156	100	100
2	None					95	75	72	72	71	68	64	62	59	558	0	0
[4] 3a	Burlap	0	1				101	94	92	91	90	84	83	81	756	51	42
[5] 3c	do	3	1			96	103	93	92	92	87	84	82	79	773	47	31
[4] 4a	do	0	2					103	98	97	95	91	89	86	834	63	56
[5] 4c	do	3	2			97		102	97	96	92	89	87	83	822	56	38
[4] 5a	do	0	3						102	99	97	91	90	88	810	67	52
[5] 5c	do	3	3			96			103	98	95	91	88	86	841	63	42
[4] 6a	Paper A with burlap	0	1	24	27		102	101	101	100	100	99	97	96	1,216	86	125
[5] 6c	do	3	1	24	27	96	103	102	102	102	100	99	98	96	1,185	86	96
[4] 7a	Sodium silicate with burlap	0	1	24	27		101	97	96	94	93	88	87	85	773	60	46
[5] 7c	do	3	1	24	27	96	104	98	97	96	93	90	88	85	819	60	38
[4] 8a	Calcium chloride with burlap.	0	1	24	27		99	97	95	95	93	89	87	85	909	60	69
[5] 8c	do	3	1	24	27	96	103	98	97	95	92	89	85	81	908	51	52
[4] 9a	Liquid material A with burlap.	0	1	24	27		101	100	100	99	99	96	96	94	1,106	81	106
[5] 9c	do	3	1	24	27	96	101	100	98	98	96	94	93	91	953	74	60
[4] 11a	Calcium chloride admixture with burlap.	0	1	(6)	(6)		104	98	96	95	93	89	87	84	770	58	46
[5] 11c	do	3	1	(6)	(6)	96	104	96	95	94	90	87	85	83	848	56	44
12c	Curing blanket			3	3	96			89	88	85	81	79	76	682	40	21
13c	Paper A			3	28	97	96	95	94	94	93	92	91	90	799	72	40
14c	Paper B			3	28	96	95	93	93	93	92	90	88	86	780	63	37
15c	Paper C			3	28	96	95	95	95	95	94	93	93	92	866	77	52
16c	Paper D			3	28	95	82	78	78	76	74	70	67	65	589	14	6
17c	Paper E			3	28	95	83	80	79	78	75	72	69	66	563	16	2
18c	Paper F			3	28	95	81	78	77	76	74	70	67	65	564	14	2
19c	Liquid material A			3	28	96	94	94	94	93	92	91	90	89	891	70	56
20c	Liquid material B			3	28	95	93	90	91	90	88	85	83	80	692	49	23
21c	Liquid material C			3	28	95	88	84	83	82	79	75	73	71	610	28	10
22c	Liquid material D			3	28	95	89	88	88	88	86	85	85	82	726	53	29

[1] Results average of 5 tests except as noted.
[2] Based on total water in specimens after molding.
[3] The relative efficiency based on strength was computed for each test separately from the strength of the ideally cured specimens and the specimens given no curing treatment in that test, and the average of these values is the relative efficiency shown.
[4] Average of 2 tests.
[5] Average of 3 tests.
[6] Mixing water contained 2 percent calcium chloride.

cured continuously under burlap (method 1a) gained 2 percent in weight. By giving this value a rating of 100 and that for specimens given no curing treatment a rating of 0, the relative efficiency of, say, method 3a, with a moisture loss of 10 percent, was found to be 59. This method of rating the efficiency of the various curing procedures is considered more satisfactory than expressing the result as a ratio of the value for ideal curing because it is a measure based on the difference in result between ideal curing and no curing treatment, whereas the latter expresses the result in terms of ideal curing only.

SURFACE SEALING MATERIALS DID NOT COMPLETELY PREVENT MOISTURE LOSS

The relative efficiencies of the several curing procedures are also shown in tables 8 to 12, inclusive, to facilitate comparisons between similar methods as well as to provide ready comparison of the results obtained for the same procedure in different series.

TABLE 8.—*Effect of time of application and duration of curing using wet burlap*

Method No.	Procedure	Relative efficiency based on— Water loss: Series A	Series B	Series D	Average	Strength: Series A	Series B	Series D	Average
3a	Burlap for 1 day; applied immediately after molding	59		51	55	66		70	68
4a	Burlap for 2 days; applied immediately after molding	72	68	68	69	93	87	93	91
5a	Burlap for 3 days; applied immediately after molding	83	79	76	79	95	96	104	98
4b	Burlap for 2 days; applied 1 hour after molding		58		58		76		76
5b	Burlap for 3 days; applied 1 hour after molding		66		66		95		95
4c	Burlap for 2 days; applied 3 hours after molding		66		66		72		72
5c	Burlap for 3 days; applied 3 hours after molding		71		71		81		81
4c-1	Burlap for 2 days; applied 3 hours after molding; sprinkled intermittently; kept continuously wet		66		66		66		66
5c-1	Burlap for 3 days; applied 3 hours after molding; sprinkled intermittently; kept continuously wet		68		68		92		92
4c-2	Burlap for 2 days; applied 3 hours after molding; sprinkled intermittently; allowed to become dry between wettings		45		45		37		37
5c-2	Burlap for 3 days; applied 3 hours after molding; sprinkled intermittently; allowed to become dry between wettings		53		53		32		32

TABLE 9.—*Comparison of various curing papers and effect of time of application*

Method No.	Type of paper	Procedure	Relative efficiency based on— Water loss: Series A	Series B	Series D	Average	Strength: Series A	Series B	Series D	Average
13b	A	Paper for 7 days; applied 1 hour after molding		76		76		80		80
13c	A	Paper for 7 days; applied 3 hours after molding	76	71	70	73	46	75	63	61
14c	B	Paper for 7 days, applied 3 hours after molding	72		70	71	45		61	5[illegible]
15c	C	Paper for 7 days; applied 3 hours after molding	76		73	74	54		69	62
16c	D	Paper for 7 days; applied 3 hours after molding	14		11	12	2		9	6
17c	E	Paper for 7 days; applied 3 hours after molding	17		19	18	1[illegible]		8	12
18c	F	Paper for 7 days, applied 3 hours after molding	14		16	15	11		1	6

TABLE 10.—*Comparison of various liquid curing materials and effect of time of application*

Method No.	Type liquid curing material	Procedure	Relative efficiency based on— Water loss: Series A	Series B	Series D	Average	Strength: Series A	Series B	Series D	Average
19b	A	Liquid for 7 days; applied 1 hour after molding.		53		53		57		57
20b	B	Liquid for 7 days; applied 1 hour after molding.		50		50		67		67
21b	C	Liquid for 7 days; applied 1 hour after molding.		29		29		24		24
19c	A	Liquid for 7 days; applied 3 hours after molding.	72	74	73	73	50	79	75	68
20c	B	Liquid for 7 days; applied 3 hours after molding.	66	71	62	66	44	99	54	6[illegible]
21c	C	Liquid for 7 days; applied 3 hours after molding.	41	42	46	43	25	45	27	32
22c	D	Liquid for 7 days; applied 3 hours after molding.	66		59	62	36		41	38

TABLE 11.—*Comparison of paper and liquid curing materials with and without preliminary curing with burlap, and effect of time of application*

Method No.	Procedure	Relative efficiency based on— Water loss: Series A	Series B	Series D	Average	Strength: Series A	Series B	Series D	Average
6a	Burlap applied immediately, followed in 24 hours by paper A for 6 days.	93	92	89	91	102	91	115	103
6b	Burlap applied 1 hour after molding, followed in 24 hours by paper A for 6 days.		84		84		109		109
6c	Burlap applied 3 hours after molding, followed in 24 hours by paper A for 6 days.		92		92		100		100
13b	Paper A for 7 days; applied 1 hour after molding.		76		76		80		80
13c	Paper A for 7 days; applied 3 hours after molding.	76	74	70	73	46	75	63	61
9a	Burlap applied immediately followed in 24 hours by liquid material A.	93		86	90	108		115	112
19b	Liquid material A applied 1 hour after molding		53		53		57		57
19c	Liquid material A applied 3 hours after molding.	72	74	73	73	50	79	75	68

TABLE 12.—*Relative efficiencies of miscellaneous curing materials*

Method No.	Type of curing	Procedure	Relative efficiency based on— Water loss: Series A	Series D	Average	Strength: Series A	Series D	Average
7a	Sodium silicate with burlap.	Burlap applied immediately followed in 24 hours by sodium silicate.	72	59	66	76	89	82
8a	Calcium chloride with burlap.	Burlap applied immediately followed in 24 hours by a surface application of calcium chloride.	59	57	58	101	90	96
11a	Calcium chloride admixture with burlap.	Burlap applied immediately for 1 day; 2 percent calcium chloride admixture added to mixing water.	66	57	62	84	83	84
12c	Curing blanket	Curing blanket for 3 days; applied 3 hours after molding.	45	46	46	39	48	44

In the following discussion of the results shown in tables 3 to 7, inclusive, consideration will be given first to the rate at which specimens cured in various ways lost water during the exposure period. In general, the values obtained for the various methods checked very closely from series to series. The data also show that the different methods varied considerably in their ability to retain moisture. As would be expected, burlap covering applied immediately and kept saturated (methods 3a, 4a, and 5a) not only retained all of the mixing water during the entire period of application, but also added water in amounts of from 1 to 4 percent. However, as soon as the burlap was removed, the specimens started losing water, the amount retained at the end of 7 days depending upon the duration of the burlap curing.

It will also be observed that, where the initial curing material was not applied until 3 hours after molding (methods in which the letter c follows the numeral), the specimens lost from 3 to 5 percent of the mixing water before they were covered. Where burlap was used and removed at the end of 24 hours (as, in methods 6c and 9c), all of this water was regained during the first day after application. However, where no burlap was used (methods 13c to 22c, inclusive), the loss was permanent, the surface sealing materials being unable to supply moisture lost during this period. In series B, table 4, tests were run with burlap applied 1 hour after molding as well as after 3 hours (methods 4b, 5b, and 6b). For some unknown reason this procedure did not result in adding water to the specimens, the amount of contained moisture being exactly the same at the end of the burlap curing period as when it was applied.

It will be noted that none of the surface sealing materials was completely effective in retaining all of the mixing water throughout the 7-day exposure period. For these materials moisture losses varied from 1 to 3 percent for the most effective materials to as much as 25 percent for the poorer materials. Furthermore, for a given material, this loss was about the same whether the material was used with or without burlap. When exposure in the curing cabinet was carried to 28 days (table 7), further loss in moisture was observed in every instance, the amounts ranging from 2 to 11 percent, depending upon the material. In general, papers A, B, and C and liquid curing materials A and B were the most efficient of the surface seals in retaining water; sodium silicate, the curing blanket, and liquid curing materials C and D were intermediate; and papers D, E, and F were the least efficient. However, as stated above, none of the surface sealing materials studied was completely effective in retaining moisture during the 7-day exposure period.

STRENGTH DETERMINATIONS AFFECTED BY NONUNIFORM MOISTURE DISTRIBUTION WITHIN SPECIMENS

The relative efficiencies of the several curing procedures based on both water loss and flexural strength after 7 days of exposure, as given in tables 3 to 6, inclusive, have been regrouped in tables 8 to 12, inclusive, in order more readily to compare the effect of varying the details of similar methods of application as well as to facilitate comparisons of the results of each method from series to series.

In studying these data the reader is cautioned against drawing conclusions regarding the comparative values of the different methods based on comparisons of individual relative efficiencies. This applies particularly to efficiencies based on flexural strength results. As will be noted from the tables, these values for a given method varied considerably from series to series. The variations were more pronounced for the surface sealing materials such as paper and the liquid curing materials, than where methods involving burlap only were used. Furthermore, they seem to follow a general trend in that the efficiencies calculated from the results of tests made in series A are, in general, low; those obtained from series B, are, in general, high; while the results obtained from series D are, as a rule, intermediate. As previously mentioned, these discrepancies may possibly be the result of variations in the moisture condition of the specimens at the time of test.

As is well known, the distribution of moisture within a flexure specimen at the time of test will appreciably affect its strength. In general, if the shell of the specimen contains more moisture than the core (a condition usually resulting from incomplete saturation after drying), the extreme fibers will be in compression and the observed breaking load will be higher than the true value. On the other hand, if the shell contains less moisture than the core (a condition usually associated with incomplete drying) the observed value will be lower than the true value. Because of the fact that these specimens were tested after an immersion period during which they may not have absorbed sufficient water to become completely saturated, it is possible that the comparatively high relative strengths obtained in certain series may have resulted from incomplete saturation of the specimens.

In preparing the specimens for test, every effort was made to insure uniform distribution of moisture. This, of course, is the only condition under which flexure tests of concrete should be made. However, inspection of the fractured specimens indicated that in many instances complete saturation was not accomplished even after 48 hours of immersion. The ideally cured specimens (method 1a) were, of course, thoroughly saturated when tested. The specimens that were given no curing treatment, as well as those cured with the least efficient surface sealing materials, because of their lack of density, absorbed water more readily upon immersion than the specimens cured by the more efficient surface sealing materials. Therefore, if the low ratings for the various curing materials were obtained because of more complete saturation, these ratings may possibly be considered to represent more nearly the true curing effect than where high ratings for the same method are shown.

In spite of wide variations in strength results in the different series, it is felt that the strength data are significant in that they indicate definite trends insofar as the general effectiveness of the several classes of curing materials are concerned. These trends will be pointed out in the following discussion of tables 8 to 12, inclusive.

EFFICIENCY OF LIQUID CURING MATERIALS INCREASED BY DELAYING APPLICATION FOR 3 HOURS

In table 8 the results of varying the time of application after molding and the duration of curing with wet burlap are given. It will be noted that, regardless of the time elapsing before the application of the burlap, the efficiency of this method of curing is increased as the length of the period of application is increased. This is true for both water loss and strength. For instance, method 3a, where the burlap was applied immediately and remained in place for 1 day, had a

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relative efficiency based on water loss of 55 and on strength of 68. When the same material was allowed to remain in place 3 days (method 5a) the efficiency based on water loss was raised to 79 and that based on strength to 98. The effect of delaying application of the burlap was to lower the efficiency as measured by strength (methods 4a, 4b, and 4c, for 2-day curing compared with methods 5a, 5b, and 5c, for 3-day curing). The same trends appear when the efficiency is measured by water loss, except that for both 2-day and 3-day curing the amount of water remaining at the end of the 7-day period was somewhat less when the burlap was applied 1 hour after molding than when applied 3 hours after molding. This reversal of trend has already been commented upon.

The results for methods 4c–1, 5c–1, 4c–2, and 5c–2 show the effects of continuous and intermittent sprinkling. Comparing 4c with 4c–1 and 5c with 5c–1, it will be noted that about the same results were obtained when the burlap was kept wet by sprinkling as when continuously saturated by keeping an end of the covering immersed in water. The effectiveness based on both water loss and strength was, however, seriously affected when the burlap was allowed to dry between the sprinklings (results for method 4c compared with 4c–2 and 5c with 5c–2). These data illustrate the importance of maintaining a continuously wet covering when burlap is used.

The results obtained with the six curing papers are shown in table 9. Papers A, B, and C, seem to be about equally effective as is also true for papers D, E, and F, except that the latter three papers show much poorer results. Papers D, E, and F, in fact, gave strengths little better than those for specimens receiving no curing treatment. The effect of period of application for paper A may be noted by comparing methods 13b and 13c. It will be observed that the efficiency of the paper, especially from the standpoint of strength, is somewhat less when the time of application is delayed.

Comparisons of the effectiveness of the various liquid curing compounds when used without burlap, that is, as recommended by the manufacturers, may be made from table 10. It will be observed that liquid materials A and B were considerably more effective than materials C and D. However, in no instance except one does the average efficiency approach that obtained by, say, the 3-day burlap curing shown in table 8, method 5a. The exception is method 20c, series B. This is an instance where an unusually high value may have resulted from incomplete saturation of the specimens.

It will be observed also that in every instance except one, the relative efficiency of the liquid curing materials is increased by delaying the application until 3 hours after molding. This is just the reverse of the trend shown for curing with paper A (table 9). This increased efficiency may possibly be accounted for by the fact that when the liquid material was sprayed on at the end of 3 hours, surface moisture had disappeared to an extent which permitted a more perfect seal than when the material was applied at the end of 1 hour. The results emphasize the necessity of watching this detail carefully when applying such materials in the field.

PRELIMINARY CURING WITH BURLAP BENEFITED SPECIMENS LATER CURED WITH OTHER MATERIALS

Table 11 permits a comparison of the results obtained with paper A and liquid curing material A when used with and without an initial curing of wet burlap. It will be observed from the data that for both methods the efficiency of the surface sealing material is materially increased by the prior use of burlap. Additional data along this line are shown in table 5 (series C). The results of these tests were not included in table 11 because, due to the omission of the method involving no curing treatment, no calculations of relative efficiency could be made.

The results indicate that when application of the burlap is delayed for 3 hours, the strengths of the specimens cured without burlap (13c, 19c, and 20c) are very nearly as high as when burlap was used. However, because the saturated burlap returned to the specimen water lost during the first 3 hours, the total water retained at the end of 7 days was somewhat higher when burlap was used than when the paper and liquid curing materials were used as recommended by the manufacturers. In general, the conclusion is that for best results such surface sealing materials as paper, liquid asphalt, etc., should be used following application of wet burlap for 24 hours.

In table 12 are shown the results of tests with sodium silicate, calcium chloride, and the special curing blanket.

In testing these materials the general practice as used in the field was followed. For sodium silicate the results indicate an effectiveness somewhat less, in general, than for a 3-day application of burlap and considerably less than the best waterproof paper or liquid curing materials used with burlap.

The results with calcium chloride are rather conflicting. For instance, the strengths obtained in the surface application method are somewhat higher than would be expected from the water losses indicated. It is apparent that, at the low relative humidity to which these specimens were subjected (32 percent) the calcium chloride withdrew water from the specimen rather than from the air. The strengths, however, are quite high. The admixture did not seem to provide any better water-retaining properties than many of the surface seals. Moreover, under these conditions, the strengths of the specimens containing the admixture were quite low. This also may have been due to the low humidity and high temperature (100° F.) to which the specimens were exposed.

PROTECTION AGAINST MOISTURE LOSS OF GREATEST IMPORTANCE

The special curing blanket, which was wet once when applied and remained in place 3 days, was quite low in efficiency as measured by both strength and water loss. Attention is directed to the fact that this blanket was of burlap and jute and it should not be confused with the cotton mats which, as previously stated, have proven highly effective for curing purposes. Neither should the results obtained with the jute blanket be regarded as representative of what would have been obtained had the blanket been wet at sufficiently frequent intervals to have kept it in a continuously moist condition.

Relative efficiencies of the various curing materials based on water retention and strength at the end of 28 days, are shown in table 7. Attention is called to the fact that, for methods 3 to 11, inclusive (methods involving the use of burlap), the results are the average of only two tests for the "a" methods and three tests for the "c" methods, instead of five tests as in all other instances.

With the above limitation in mind, it may be noted that all of the methods involving burlap only, that is methods 3a to 5c, inclusive, had low ratings after 28 days as compared to the corresponding results at 7 days. Furthermore, the beneficial effects of burlap curing for 3 days as compared to curing for 1 day appear to be somewhat less pronounced. Specimens cured with waterproof paper A following burlap curing (methods 6a and 6c) developed high strength at 28 days. Attention is called to the fact that the paper remained in place for the full 28-day period. The same material without initial burlap curing (method 13c) showed a relative efficiency of only 40 as regards strength.

Liquid curing material A gave high strengths when the burlap was applied immediately (method 9a) but showed a comparatively low relative efficiency when application of the burlap was delayed 3 hours (method 9c). Without burlap, liquid material A (method 19c) showed a rating of 56, only slightly lower than the combination in which the burlap was applied after 3 hours. These trends seem to parallel in general the indications at 7 days (table 5, series C and table 11). With respect to burlap curing as compared with the sealing materials, it might be pointed out that in this high-temperature, low-humidity atmosphere, the curing with burlap was discontinued at 3 days, whereas, curing continued to some extent under the seals that were effective.

The relative efficiencies of the methods employing sodium silicate, calcium chloride, and the curing blanket (methods 7, 8, 11, and 12) are about the same at 28 days as at 7 days, when judged from the standpoint of water retention, but are much lower when considered from the standpoint of strength. However, the small number of specimens represented make any comparisons involving these methods of doubtful value.

The most significant point in connection with the 28-day test data lies in the fact that in only two instances did the strength ratings anywhere near approach that of method 1a. These methods, burlap applied immediately followed by waterproof paper A (method 6a), and burlap applied immediately followed by liquid bituminous material A (method 9a), provide the most nearly perfect continuous seals of any of the methods tested, thus emphasizing the conclusion that the greatest curing efficiency is provided by those methods that protect the concrete against moisture loss to the greatest extent.

The results obtained in this investigation seem to warrant the following general conclusions:

A. As regards burlap used alone:

1. The effectiveness of burlap is increased by lengthening the duration of application.

2. The effectiveness of burlap is decreased by increasing the time elapsing between the placing of the concrete and the application of the burlap.

3. Burlap is not as effective when sprinkled intermittently as when kept continuously saturated.

B. As regards surface sealing materials:

1. The effectiveness of such materials as waterproof paper and liquid curing materials applied with a spray gun is materially increased when preceded by application of wet burlap for 24 hours.

2. The effectiveness of such membrane coverings as liquid curing materials A and B is materially improved by applying the covering 3 hours after molding as compared to an application made 1 hour after molding.

TOLL ROADS AND FREE ROADS

(Continued from page 66)

cities, for future street developments and also for other kinds of public works and developments.

As one of its most important recommendations, the report suggests the creation of a Federal Land Authority with adequate capitalization and authority to issue obligations, which would be empowered to acquire, hold, sell, and lease lands, in connection with all sorts of public improvements, in ways designed to accomplish (1) the total or partial self-liquidation of such improvements, (2) the coordination of the various classes of improvements by the establishment of a proper relation in their use of land, and (3) the elimination of embarrassing delays in the accomplishment of desirable improvements, and of restriction likely to warp the form, and partially to defeat the purpose, of the improvements.

The report, entitled "Toll Roads and Free Roads," has been printed as House Document No. 272, Seventy-sixth Congress, first Session. Single copies can be obtained without charge from the Bureau of Public Roads, United States Department of Agriculture, Washington, D. C.

MOTOR-FUEL CONSUMPTION, 1938

[Compiled for calendar year from reports of State authorities [1]]

State	Tax rate per gallon [2]	Gross amount reported [3]	Amount exempted from payment of tax [4]	Gross amount assessed for taxation	Amount subject to refund of entire tax	Net amount taxed: Total	Net amount taxed: At prevailing rate	Net amount taxed: At other rates: Rate per gallon	Net amount taxed: At other rates: Amount
	Cents	*1,000 gallons*	*1,000 gallons*	*1,000 gallons*	*1,000 gallons*	*1,000 gallons*	*1,000 gallons*	*Cents*	*1,000 gallons*
Alabama	6	226, 838		226, 838		226, 838	226, 838		
Arizona	5	102, 711	5, 487	97, 224	12, 690	84, 534	84, 534		
Arkansas	6½	166, 200	6, 256	159, 944		159, 944	143, 479	(5)	16, 465
California	3	1, 763, 625	33, 284	1, 730, 341	158, 413	1, 571, 928	1, 571, 928		
Colorado	4	227, 258	10, 115	216, 813	28, 869	187, 944	187, 941		
Connecticut	3	326, 263	7, 377	318, 886	6, 176	312, 710	312, 710		
Delaware	4	56, 638	1, 256	55, 382	2, 892	52, 490	52, 490		
Florida	7	338, 650	11, 812	326, 838		326, 838	326, 838		
Georgia	6	339, 392	10, 471	328, 921		328, 921	328, 921		
Idaho	5	95, 077	3, 870	91, 207	9, 130	82, 077	81, 888	2½	[6] 189
Illinois	3	1, 358, 680		1, 358, 680	102, 664	1, 256, 016	1, 256, 016		
Indiana	4	612, 714	2, 057	610, 657	47, 778	562, 879	562, 879		
Iowa	3	521, 535		524, 535	78, 629	445, 906	415, 906		
Kansas	3	459, 433	121, 906	337, 527		337, 527	337, 527		
Kentucky	5	256, 516		256, 516		256, 516	256, 516		
Louisiana	7	247, 176	4, 965	242, 211	4	242, 207	234, 941	2	[7] 7, 266
Maine	4	144, 866	882	143, 984		143, 984	137, 106	1	[8] 6, 578
Maryland	4	271, 434	4, 226	267, 208	18, 600	248, 608	246, 433	3	[9] 2, 175
Massachusetts	3	690, 203	2, 702	687, 501	25, 247	662, 254	662, 254		
Michigan	3	1, 053, 961	81, 484	972, 477	43, 184	929, 293	928, 920	1½	[10] 373
Minnesota	4	536, 861	24, 719	512, 112	64, 668	447, 444	447, 444		
Mississippi	6	190, 248	9, 147	181, 101		181, 101	171, 044	1	[11] 10, 057
Missouri	2	608, 472		608, 472	27, 386	581, 086	581, 086		
Montana	5	117, 164	6, 455	110, 709	21, 259	89, 450	89, 450		
Nebraska	5	232, 817	9, 469	223, 348	39	223, 309	223, 309		
Nevada	4	34, 771	2, 886	31, 885	1, 927	29, 958	29, 958		
New Hampshire	4	85, 157		85, 157	2, 443	82, 714	82, 714		
New Jersey	3	812, 804	3, 257	809, 547	67, 112	742, 435	742, 435		
New Mexico	5	96, 150	6, 110	90, 040	8, 390	81, 650	81, 521	7½	[12] 129
New York	4	1, 802, 216	64, 987	1, 737, 229	52, 557	1, 684, 672	1, 684, 672		
North Carolina	6	403, 333	6, 294	397, 039		397, 039	385, 834	1	[11] 11, 205
North Dakota	3	122, 866	1, 353	121, 513	35, 738	85, 775	85, 775		
Ohio [13]	4	1, 278, 825	63, 190	1, 215, 635	8, 797	1, 206, 838	1, 157, 015	1	[8] 49, 823
Oklahoma	4	403, 795	12, 311	391, 481	41, 294	350, 190	350, 190		
Oregon	5	230, 187	4, 927	225, 260	26, 616	198, 644	197, 797	1	[14] 847
Pennsylvania	4	1, 403, 587	6, 519	1, 397, 068		1, 397, 068	1, 397, 068		
Rhode Island	3	120, 886	1, 023	119, 863	2, 989	116, 874	116, 874		
South Carolina	6	192, 170		192, 170	3, 387	188, 783	188, 783		
South Dakota	4	132, 002	7, 353	124, 649	24, 981	99, 668	99, 668		
Tennessee	7	280, 862	14, 976	265, 886	1, 723	264, 163	264, 163		
Texas	4	1, 267, 298	23, 886	1, 243, 412	167, 561	1, 075, 851	1, 075, 851		
Utah	4	92, 950	5, 100	87, 850		87, 850	87, 850		
Vermont	4	61, 324	1, 024	63, 300		63, 300	63, 300		
Virginia	5	355, 150		355, 150	20, 823	334, 327	334, 327		
Washington	5	341, 023	6, 044	334, 979	25, 282	309, 697	309, 697		
West Virginia	5	190, 397		190, 397	1, 482	188, 915	188, 915		
Wisconsin	4	512, 883	16, 884	525, 999	41, 187	484, 812	484, 812		
Wyoming	4	65, 356	1, 980	63, 376		63, 376	63, 376		
District of Columbia	2	139, 612	5, 586	134, 026	701	133, 325	133, 325		
Total	[15] 3.96	21, 106, 636	614, 290	20, 792, 346	1, 182, 618	19, 609, 728	19, 504, 621		105, 107

[1] An analysis of motor-fuel usage, similar to that given in the right-hand portion of table Motor-Fuel Consumption, 1937, previously issued will be published in a subsequent table.

[2] No changes in tax rates reported during 1938.

[3] Export sales and other amounts not representing consumption in State have been eliminated as far as possible. In cases where States failed to report amounts exempted from taxation, the gross amount taxed is shown in this column.

[4] Includes allowances for evaporation and other losses, Federal use, other public use, and nonhighway use, where initial exemptions rather than refunds are made.

[5] Within 300 feet of border, tax is reduced to that of adjacent State. Gallons taxed at 2 cents, 3,787,000; at 4 cents, 12,678,000.

[6] Motor fuel used in aviation.

[7] Represents evaporation or loss allowance under 5-cent tax not allowed under additional 2-cent tax, which is administered under a separate law.

[8] 3 cents per gallon refunded on nonhighway uses.

[9] 1 cent per gallon refunded on motor fuel used in vehicles licensed to operate exclusively in cities.

[10] 1½ cents per gallon refunded on motor fuel used in interstate aviation.

[11] 5 cents per gallon refunded on nonhighway uses.

[12] Diesel oil taxed at 7½ cents per gallon.

[13] Amounts given do not include 66,240,000 gallons of liquid fuel (kerosene, fuel oil, etc.) taxed at 1 cent per gallon but not subject to the 3-cent tax on motor-vehicle fuel.

[14] 4 cents per gallon refunded on motor fuel used in aviation.

[15] Weighted average rate.

STATE MOTOR-FUEL TAX RECEIPTS, 1938

[Compiled for calendar year from reports of State authorities]

State	Tax rate per gallon[1]	Receipts from taxation of motor fuel					Other receipts in connection with motor-fuel tax					Net total receipts	Less tax on aviation gasoline	Adjusted net total receipts
		Gross tax collections	Deductions by distributors for expenses[2]	Gross receipts by State	Refunds paid	Net receipts by State	Distributors' and dealers' licenses	Inspection fees[3]	Fines and penalties	Miscellaneous receipts[4]	Total			
	Cents	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars
Alabama	6	13,523		13,523		13,523		56			56	13,579		13,579
Arizona	5	5,016		5,016	773	4,243						4,243		4,243
Arkansas	6½	10,004		10,004		10,004		88			88	10,092		10,092
California	3	51,853		51,853	4,752	47,101	14			2	16	47,117		47,117
Colorado	4	8,623		8,623	1,158	7,465						7,465		7,465
Connecticut	3	9,471	94	9,377	185	9,192	50				50	9,242		9,242
Delaware	4	2,211		2,211	142	2,069	3		1		4	2,073		2,073
Florida	7	22,801		22,801		22,801	28	403			431	23,232		23,232
Georgia	6	19,831	198	19,633		19,633						19,633		19,633
Idaho	5	4,543		4,543	455	4,088				2	2	4,090	5	4,085
Illinois	3	40,325	806	39,519	3,038	36,481		405	2		407	36,888		36,888
Indiana	4	24,170		24,170	1,911	22,259		511			511	22,770		22,770
Iowa	3	15,504		15,504	2,271	13,233	1				1	13,234		13,231
Kansas	3	10,017		10,017		10,017	13	105		33	151	10,168		10,168
Kentucky	5	12,655	127	12,528		12,528			3		3	12,531		12,531
Louisiana	7	16,543		16,543		16,543		77	7		[5] 84	16,627		16,627
Maine	4	5,755		5,755	197	5,558						5,558		5,558
Maryland	4	10,695		10,695	766	9,929						9,929		9,929
Massachusetts	3	20,951		20,951	757	20,194						20,194		20,194
Michigan	3	29,025		29,025	1,301	27,724	4				4	27,728	45	27,683
Minnesota	4	22,048		22,048	2,668	19,380	1	187		2	190	19,570		19,570
Mississippi[6]	6	10,696		10,696	515	10,181						10,181		10,181
Missouri	2	12,059		12,059	557	11,502		125	9		134	11,636		11,636
Montana	5	5,491		5,491	1,039	4,452						4,452		4,452
Nebraska	5	11,365	86	11,279	253	11,026	7	107		30	144	11,170	31	11,139
Nevada	4	1,304	26	1,278	77	1,201			1		1	1,202		1,202
New Hampshire	4	3,395		3,395	98	3,297			1		1	3,298		3,298
New Jersey	3	24,348		24,348	2,051	22,294	68				68	22,362		22,362
New Mexico	5	4,486		4,486	420	4,066	24				24	4,090		4,090
New York	4	68,917	689	68,228	2,096	66,132	63				63	66,195		66,195
North Carolina	6	23,860		23,860	560	23,300		1,002		6	1,008	24,308		24,308
North Dakota	3	3,632	55	3,577	1,323	2,254		64			64	2,318		2,318
Ohio	4	[7] 48,031		48,031	2,049	45,982						45,982		45,982
Oklahoma	4	15,855	317	15,538	1,633	13,905			5		5	13,910		13,910
Oregon	5	11,246		11,246	1,100	9,846						9,846	8	9,838
Pennsylvania	4	52,574	653	51,921	7	51,914	80		7		87	52,001		52,001
Rhode Island	3	3,754		3,754	262	3,492	3				3	3,495		3,495
South Carolina	6	11,451		11,451	197	11,254		240			240	11,494	32	11,462
South Dakota	4	4,986	100	4,886	838	4,048		64			64	4,112	10	4,102
Tennessee	7	18,375		18,375	99	18,276		1,044			1,044	19,320	89	19,231
Texas	4	50,041	500	49,541	6,821	42,720				27	27	42,747		42,747
Utah	4	3,576	54	3,522		3,522	1		1		2	3,524	46	3,478
Vermont	4	2,530		2,530		2,530						2,530		2,530
Virginia	5	17,661		17,661	1,041	16,620			1		1	16,621		16,621
Washington	5	16,684		16,684	1,263	15,421	1			9	10	15,431		15,431
West Virginia	5	9,470		9,470	84	9,386	11				11	9,397		9,397
Wisconsin	4	20,902		20,902	1,649	19,253		194			194	19,447		19,447
Wyoming	4	2,505		2,505		2,505	3				3	2,508	30	2,478
District of Columbia	2	2,523		2,523	14	2,509	11				11	2,520		2,520
Total	[8] 3.96	817,281	3,705	813,576	46,723	766,853	386	4,672	38	111	5,207	772,060	296	771,764

[1] No changes in tax rates reported during 1938.

[2] The indicated States make allowances to distributors for expense of collecting the tax. In Kentucky, South Dakota, and Utah allowances of 2¼, 4, and 3 percent, respectively, of the tax otherwise due are made in consideration of both expense of collection and gallonage losses in handling. In these States the allowances for expenses only have been estimated as 1, 2, and 1½ percent, respectively.

[3] Fees for inspection of motor-vehicle fuel. Wherever possible, fees for inspection of kerosene and other nonmotor-vehicle fuels have been eliminated.

[4] Includes fees for motor-fuel carrier permits, refund or exemption permits, and miscellaneous unclassified receipts.

[5] Receipts from tax on lubricating oil, $784,060, not included in this table.

[6] Special county taxes of 3 cents per gallon in Hancock County and 2 cents per gallon in Harrison County, amounting to $163,000 in 1938, are imposed for sea-wall protection and are not included in this table.

[7] Ohio imposes a 3-cent tax on motor-vehicle fuel and a 1-cent tax on all liquid fuels. The receipts from the 1-cent tax applicable to nonmotor-vehicle fuels (kerosene, fuel oil, etc.) were $633,000. These receipts have been eliminated from the total given, which represents a 4-cent tax on motor-vehicle fuel.

[8] Weighted average rate.

STATE MOTOR-VEHICLE REGISTRATIONS, 1938

[Compiled for calendar year from reports of State authorities [1]]

State	Registered motor vehicles, private and commercial [2]: Total motor vehicles	Passenger motor vehicles: Total	Passenger motor vehicles: Automobiles (including taxicabs)	Passenger motor vehicles: Motor busses [3]	Motor trucks, tractor trucks, etc.	Other registered vehicles: Trailers and semitrailers [4]	Other registered vehicles: Motorcycles	Publicly owned vehicles, Federal [5]: Motor vehicles	Federal: Trailers and semitrailers	Federal: Motorcycles	State, county, and municipal [6]: Motor vehicles	State, county, and municipal: Trailers and semitrailers	State, county, and municipal: Motorcycles	Dealers' registrations and plates [7]: Regular registrations	Dealers': Extra sets of plates	1937 total registered motor vehicles	Year's change in motor-vehicle registrations: Increase or decrease	Year's change: Percentage change
Alabama	301,990	250,074	243,745	6,329	51,916	3,890	816	1,992	55	4	3,755	------	143	2,781	------	313,359	−11,369	−3.6
Arizona	128,791	105,793	105,354	439	22,998	4,567	452	2,374	97	4	2,054	181	23	1,704	------	129,210	−419	−.3
Arkansas	220,391	167,045	166,687	358	53,346	10,162	517	2,067	25	1	3,193	13	31	489	------	229,867	−9,476	−4.1
California	2,510,867	2,213,152	2,213,152	(8)	297,715	142,268	11,802	7,539	276	75	24,502	1,646	1,157	4,532	------	2,484,653	26,214	1.1
Colorado	332,774	277,860	276,767	1,093	54,914	1,422	1,271	2,347	23	7	------	------	------	3,559	------	337,217	−4,443	−1.3
Connecticut	440,335	369,693	368,664	1,029	70,642	5,356	1,883	655	6	1	3,891	87	294	2,848	7,731	436,564	3,771	.9
Delaware	64,078	53,559	53,559	(8)	10,519	2,772	231	312	4	------	896	25	62	729	------	63,599	479	.8
Florida	423,021	352,978	351,209	1,769	70,043	17,324	1,496	1,787	25	15	5,072	317	168	2,638	------	418,145	4,876	1.2
Georgia	432,360	359,204	356,609	2,595	73,156	12,684	1,233	2,665	45	33	4,019	63	138	2,164	------	441,847	−9,487	−2.1
Idaho	137,851	109,716	109,595	121	28,135	18,172	548	1,541	80	------	1,548	89	11	419	------	142,110	−4,259	−3.0
Illinois	1,780,865	1,565,202	1,565,202	(8)	215,663	23,073	6,194	3,317	98	19	9,492	323	654	4,199	------	1,768,946	11,919	.7
Indiana	922,788	795,118	793,969	1,149	127,670	62,914	4,543	1,631	86	15	6,169	------	195	2,467	------	956,016	−33,228	−3.5
Iowa	740,021	650,534	650,534	------	89,487	[9] 87,447	2,558	1,412	22	6	6,054	357	66	1,887	------	745,602	−5,581	−.7
Kansas	573,985	476,587	476,241	346	97,398	6,728	1,084	1,504	61	9	------	------	------	2,045	------	586,685	−12,700	−2.2
Kentucky	414,207	350,531	349,940	591	63,676	(10)	1,116	1,571	11	109	4,325	------	------	859	------	404,455	9,752	2.4
Louisiana	326,199	248,754	248,287	467	77,445	14,586	1,020	1,827	27	13	4,725	380	63	321	------	323,498	2,701	.8
Maine	196,690	154,027	153,861	166	42,663	[11] 10,144	834	552	10	5	2,087	164	31	735	------	200,907	−4,217	−2.1
Maryland	395,347	339,896	338,885	1,011	55,451	4,208	1,460	2,366	61	21	------	------	------	8,462	------	387,410	7,937	2.0
Massachusetts	843,789	739,323	734,585	4,738	104,466	13,122	765	2,717	31	11	[12] 5,700	------	------	2,981	20,315	846,556	−2,767	−.3
Michigan	1,408,835	1,269,894	[13] 1,269,894	(8)	138,941	141,647	4,294	2,542	81	17	------	------	------	1,957	------	1,505,111	−96,276	−6.4
Minnesota	821,241	705,271	705,019	252	115,970	31,033	2,226	2,509	56	10	4,790	------	------	------	------	822,598	−1,357	−.2
Mississippi [14]	215,195	163,709	161,015	2,694	51,486	1,771	318	1,475	57	1	------	------	------	2,380	------	226,286	−11,091	−4.9
Missouri	837,118	703,457	702,941	516	133,661	33,368	1,792	2,085	27	6	2,177	------	9	2,114	------	835,895	1,223	.1
Montana	171,326	130,188	130,188	(8)	41,138	2,953	456	2,090	27	------	2,201	------	------	986	------	173,892	−2,566	−1.5
Nebraska	407,330	342,275	342,047	228	65,055	41,294	1,125	1,199	14	8	2,462	------	46	1,492	------	412,726	−5,396	−1.3
Nevada	38,424	30,899	30,695	204	7,525	1,218	109	696	21	2	639	48	8	81	336	40,655	−2,231	−5.5
New Hampshire	124,379	97,635	97,635	(8)	26,744	4,767	896	655	19	1	------	------	------	556	------	125,939	−1,560	−1.2
New Jersey	1,000,684	868,734	863,665	5,069	131,950	7,276	4,767	2,763	26	14	10,477	------	545	2,493	------	994,497	6,187	.6
New Mexico	116,537	89,592	89,123	469	26,945	2,575	357	2,178	67	------	911	------	41	405	------	118,106	−1,569	−1.3
New York	2,584,123	2,259,468	2,259,468	------	324,655	40,771	10,391	5,629	49	81	26,083	881	1,036	5,381	------	2,561,703	22,420	.9
North Carolina	537,242	461,141	460,298	843	76,101	42,317	1,705	2,181	42	3	11,671	------	------	8,529	------	525,350	11,892	2.3
North Dakota	174,256	141,195	141,111	84	33,061	849	296	788	12	------	699	------	------	585	522	173,188	1,068	.6
Ohio	1,870,249	1,686,555	1,686,555	(8)	183,694	105,249	9,073	2,476	85	6	18,768	1,125	576	4,054	18,445	1,876,132	−5,883	−.3
Oklahoma	535,399	441,184	438,979	2,205	94,215	36,498	1,090	2,435	42	14	6,719	------	------	3,733	------	547,263	−11,864	−2.2
Oregon	357,321	297,492	296,837	655	59,829	(8)	1,531	2,821	33	4	4,071	------	------	653	838	360,348	−3,027	−.8
Pennsylvania	1,976,466	1,730,893	1,725,674	5,219	245,573	26,225	10,561	3,983	123	24	17,800	463	1,226	29,614	------	1,984,821	−8,355	−.4
Rhode Island	168,888	149,634	149,223	411	19,254	631	704	460	16	1	1,409	23	105	388	------	167,586	1,302	.8
South Carolina	287,913	246,585	245,117	1,468	41,328	5,396	1,011	1,570	14	5	4,468	------	146	792	1,586	296,224	−8,311	−2.8
South Dakota	180,632	152,138	152,040	98	28,494	19,080	436	1,241	17	2	1,132	152	12	709	------	184,743	−4,111	−2.2
Tennessee	398,624	337,584	336,900	684	61,040	(8)	1,502	2,168	45	2	7,105	------	------	582	------	400,384	−1,760	−.4
Texas	1,548,343	1,231,424	1,230,548	876	316,919	50,944	3,980	5,368	148	75	15,905	1,193	378	3,307	------	1,552,114	−3,771	−.2
Utah	127,004	107,038	106,373	665	19,966	569	428	1,758	56	10	1,378	64	46	385	------	126,692	312	.2
Vermont	87,402	78,360	78,265	95	[15] 9,042	1,853	450	648	14	------	------	------	------	602	------	88,958	−1,556	−1.7
Virginia	441,462	373,896	373,896	------	67,566	9,350	1,743	3,231	217	52	5,470	165	183	5,268	------	440,713	749	.2
Washington	523,328	440,128	439,328	800	83,200	17,826	2,025	3,215	77	27	6,950	336	164	1,503	3,291	535,483	−12,155	−2.3
West Virginia	275,691	230,637	230,024	613	45,054	3,058	1,212	1,188	6	------	5,109	148	70	10,209	------	290,837	−15,146	−5.2
Wisconsin	840,291	703,807	703,227	580	136,484	5,047	3,346	2,300	39	5	8,546	204	360	2,428	------	854,374	−14,083	−1.6
Wyoming	80,765	63,176	63,176	------	17,589	10,209	272	1,300	26	9	681	57	------	368	------	81,837	−1,072	−1.3
District of Columbia	162,863	148,614	147,495	1,119	14,249	809	622	1,262	20	62	[16] 2,366	106	94	2,308	------	184,119	−21,256	−11.5
At large	------	------	------	------	------	------	------	5,371	45	10	------	------	------	------	------	------	------	------
Total	29,485,680	25,261,649	25,213,601	48,048	4,224,031	1,085,422	108,541	109,716	2,564	799	257,469	8,610	8,081	139,681	53,064	29,705,220	−219,540	−.7

[1] Registration periods ending not earlier than Nov. 30 and not later than Jan. 31 are considered calendar-year periods. In those States where the registration period is definitely removed from the calendar year, registration figures were obtained for the calendar-year period.

[2] Wherever possible publicly owned vehicles and vehicles not for highway use have been eliminated from these columns.

[3] A complete segregation of motor busses from other vehicles is not available. The figures given represent common-carrier busses in most cases, although in some States contract busses and contract school busses are included. In some cases city busses are not included. Where no buses are tabulated, they are included with automobiles, unless otherwise noted.

[4] Figures for trailers and semitrailers are as reported. Apparent inconsistencies are due to the fact that some States require the registration of tourist trailers, light work trailers, and similar vehicles, whereas other States register only freight-carrying trailers and semitrailers.

[5] Data on Federal vehicles obtained through agency of Procurement Division, Department of the Treasury.

[6] State, county, and municipal vehicles are included with private and commercial registrations in Colorado, Kansas, Maryland, Michigan, Mississippi, New Hampshire, and Vermont. An unknown number of Federal vehicles are included in the figures for Indiana, Iowa, Kentucky, Louisiana, Montana, New York, and Virginia. Some States give State-owned vehicles only; others exclude certain classes from registration, such as fire apparatus and police vehicles.

[7] Figures include new-car, used-car, and motorcycle dealer registrations and some wrecker and repairer registrations. Data on dealers' extra plates are incomplete, although they are apparently included with dealer registrations in some States.

[8] Included with motor trucks.

[9] Includes 63,000 light trailers registered without charge.

[10] Trailers of 1,000 pounds capacity or more prohibited on highways, although permitted in cities under city licenses. Tractor-semitrailers registered as motor trucks. Light trailers permitted but not registered.

[11] Includes light trailers and commercial semitrailers. Commercial full trailers included with motor trucks.

[12] Of these vehicles approximately 1,700 are also included with private and commercial registrations.

[13] Taxicabs included with motor trucks.

[14] License year changed to Nov. 1 during 1938. Registrations recorded on this table are for 10-month period through Oct. 1938. Registrations for 1939 in Nov. and Dec. 1938 were: Automobiles, 150,822; motor busses, 1,367; motor trucks, 41,012; total motor vehicles, 193,201.

[15] Trucks under 1,500 pounds capacity included with passenger cars.

[16] Includes 405 automobiles of the diplomatic corps.

STATE MOTOR-VEHICLE RECEIPTS, 1938

[Compiled for calendar year from reports of State authorities

State	Total receipts, registration and other fees	Motor-vehicle registration fees: Total[2]	Motor-vehicle registration fees: Passenger motor vehicles: Total	Motor-vehicle registration fees: Passenger motor vehicles: Automobiles (including taxicabs)	Motor-vehicle registration fees: Passenger motor vehicles: Motor busses[3]	Motor-vehicle registration fees: Motor trucks, tractor trucks, etc.	Registration fees, other vehicles: Trailers and semitrailers	Registration fees, other vehicles: Motorcycles	Total registration fees, all vehicles	Miscellaneous receipts: Total	Miscellaneous receipts: Dealers' licenses and plates	Miscellaneous receipts: Operators' and chauffeurs' permits	Miscellaneous receipts: Certificates of title	Miscellaneous receipts: Special titling taxes[4]	Miscellaneous receipts: Fines and penalties	Miscellaneous receipts: Transfer or reregistration fees	Miscellaneous receipts: Other receipts	Miscellaneous receipts: Unclassified refunds	Miscellaneous receipts: Estimated service charges, local collectors[5]
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Alabama	4,314	3,858							3,858	456	3	251			193		9		
Arizona	1,076	809	398	371	27	411	62	2	873	203	4	25	93				81		
Arkansas	2,908	2,495	1,575	1,499	76	920	220	3	2,718	190	57	124			2		2		
California [6]	23,930	21,049	16,996	16,996	(7)	4,053	989	60	22,098	1,832	74	150			130	1,263	215		
Colorado	2,544	2,049	1,571	1,571		[8] 478	30	2	2,081	463	27	113	159		27	66	80	−9	
Connecticut	6,611	4,249	2,822	2,698	124	1,427	20	6	4,275	2,336	106	1,663			169	104	300	−6	
Delaware	1,216	894	607	607	(7)	287	32	1	927	289	7	135	33		80	13	21		
Florida	6,432	5,815	4,044	3,853	191	1,771	259	8	6,082	350	22	10	206			1	22	−20	109
Georgia	1,974	1,465	985	875	110	480	148	2	1,615	359	12	319				7	21		
Idaho	2,380	2,269	1,638	1,614	24	631	33	3	2,305	75	25	38					12		
Illinois	21,591	19,925	14,022	14,022	(7)	5,903	175	16	20,116	1,475	91	400	429		16	425	132	−18	
Indiana	9,635	7,599	5,905	5,822	83	1,694	316	8	7,923	1,712	50	694	274		36	277	264	−112	229
Iowa	11,797	11,188	8,415	8,415		2,773	93	9	11,290	507	49	162			2	190	104		
Kansas	3,823	3,463	2,543	2,543	(7)	920	109	4	3,576	247	21	46	134			35	11		
Kentucky	4,599	2,971	1,745	1,666	79	1,226	(9)	2	2,973	1,626	25	522		865	3	196	15		
Louisiana	4,892	4,215	2,936	2,868	68	1,279	587	4	4,806	86	12	103			77		1	−37	
Maine	3,582	2,725	1,898	1,882	16	827	[10] 66	4	2,795	787	44	559			30	119	35		
Maryland	5,069	3,584	2,963	2,810	153	621	119	7	3,710	1,359	45	348	171	390	329	97	31	−52	
Massachusetts	6,759	4,121	2,740	2,609	131	1,381	28	1	4,150	2,609	70	2,532			3		66	−62	
Michigan	20,856	17,801	12,609	[11] 12,609	(7)	5,192	1,002	16	18,819	2,037	91	943	664			259	80		
Minnesota	9,377	8,832	6,889	6,734	155	1,943	259	7	9,098	279	58	122			7		22		
Mississippi [12]	4,001	3,947							3,947	54	5				43		6		
Missouri	9,439	8,273	6,936	6,936	(7)	1,337	140	9	8,422	1,017	78	290	392		2	238	17		
Montana [13]	1,546	1,169	852	852	(7)	317	13	1	1,183	363	22	172	62		56	41	10		
Nebraska	2,442	2,135	1,242	1,220	22	893	58	2	2,195	247	30	68				66	83		
Nevada	265	245	155	154	1	90	5		250	15	2				1	11	1		
New Hampshire	2,711	2,262						5	2,267	444	23	343			30		48		
New Jersey	20,204	13,806	9,804	9,509	295	4,002	284	10	14,100	6,104	62	3,875	464		219	532	961	−9	
New Mexico	1,643	1,505	955	929	26	550	34	1	1,540	103	11				1	29	62		
New York	47,124	43,418	31,675	31,675		11,743	573	46	44,037	3,087	208	1,067			544	947	360	−39	
North Carolina	7,211	6,871	4,113	4,056	57	2,758	(7)	7	6,878	333	32	121	147		56		12	−35	
North Dakota	1,523	1,404	1,073	1,062	11	331	3	2	1,409	114	17	14	57	24	1		2	−1	
Ohio	27,204	22,370	14,610	14,610	(7)	7,760	1,148	(14)	23,518	3,686	235	753	418			586	44		1,650
Oklahoma	5,779	4,401	2,643	2,544	99	1,758	218	5	4,624	1,155	48	207	292		186		125		297
Oregon	2,922	2,598	1,523	1,484	39	1,075	(7)	4	2,602	320	25	74	175		24		22		
Pennsylvania	34,513	27,710	19,145	18,612	533	8,565	390	29	28,129	6,384	377	2,998	1,668			872	578	−109	
Rhode Island	2,778	2,217	1,732	1,692	40	485	2	3	2,222	556	17	426				68	45		
South Carolina	1,633	1,316	732	732		584	71	1	1,388	245	19	191			27	21	31	−44	
South Dakota	1,983	1,583	1,359	1,355	4	224	30	1	1,614	369	17		23	319			10		
Tennessee	4,173	3,907					(7)		3,907	266	(15)	259			5		7	−5	
Texas	20,263	18,716	12,015	11,865	150	6,701	520	17	19,253	1,010	41	417				398	154		
Utah	1,097	921	510	510	(7)	411	34	1	956	141	14	19	66		1	16	25		
Vermont	2,365	1,994	1,420	[16] 1,407	13	[16] 574	25	3	2,022	343	26	280				42	7	−12	
Virginia	6,134	5,449	4,092	4,092		1,357	134	6	5,589	545	59	80	245			141	38	−18	
Washington	3,262	2,487	1,389	1,389		1,098	207	6	2,700	562	14	165	219		18		10		136
West Virginia	5,498	4,524	3,478	3,458	20	1,046	57	5	4,586	912	43	144	136	525		47	17		
Wisconsin	13,001	12,377	9,375	9,168	207	3,002	322	16	12,715	286	23	35				215	13		
Wyoming	601	552	360	360		192	34	1	587	14	8					4	2		
District of Columbia	2,145	1,333							1,333	812	1	298	70		406	29	8		
Partial totals [17]			224,489	221,735	2,754	91,070	8,849	346											
Full totals	388,825	330,866							340,061	48,764	2,350	21,555	6,597	2,123	2,724	7,360	4,222	−588	2,421

[1] Receipts for registration periods ending not earlier than Nov. 30 and not later than Jan. 31 are considered calendar-year receipts. In those States where the registration period is definitely removed from the calendar year, registration receipts were obtained for the calendar-year period.

[2] Segregation of registration fees by type of vehicle was not available for Alabama, Mississippi, New Hampshire, Tennessee, and the District of Columbia. Total motor-vehicle registration fees in those States include trailer and motorcycle fees, except in New Hampshire, for which motorcycle fees were reported separately. Dealers' license fees in Tennessee are also included in this column.

[3] The motor-bus registration fees are incomplete (see footnote 3 of preceding table). Where no fees are tabulated, the fees of busses are included with those of automobiles, unless otherwise noted.

[4] Proceeds of special excise and privilege taxes on new-car sales have been segregated and entered in this column. Receipts from a 2-percent motor-vehicle excise tax in Oklahoma, imposed as part of a general sales tax, are not included in this table. Proceeds of this tax were $1,104,000 in 1938.

[5] In many States county or local officers are allowed service charges for issuing registrations, operators' licenses, etc. In the majority of cases these charges are included in registration and other fees. The amounts shown in this column are estimates of service charges collected and retained by local officials and not reported elsewhere in the table.

[6] Registration fees include proceeds of State "vehicle license fees", $10,854,000, imposed in addition to the regular registration fees of $11,244,000.

[7] Included with motor-truck fees.

[8] Fees of 23,978 light trucks included with those of passenger vehicles.

[9] Trailers of 1,000 pounds capacity or more prohibited on highways, although permitted in cities under city licenses. Tractor-semitrailers registered as motor trucks. Light trailers permitted but not registered.

[10] Fees of light trailers and commercial semitrailers only. Fees of commercial full trailers included with those of motor trucks.

[11] Fees of taxicabs included with those of motor trucks.

[12] License year changed to Nov. 1 during 1938. Receipts recorded in this table are for calendar year and include fees for 1939 registrations received from Oct. 1 through Dec. 31.

[13] Registration fees are collected by counties and State does not maintain complete record. Figures given are estimates supplied by State.

[14] Included with fees of automobiles.

[15] Included with motor-vehicle registration fees.

[16] Fees of trucks under 1,500 pounds capacity included with those of passenger cars.

[17] Totals of columns for which full classified data were not available for all States

STATE MOTOR-CARRIER TAX RECEIPTS, 1938

[Compiled for calendar year from reports of State authorities]

State	Proceeds of State imposts on motor vehicles operated for hire and other motor carriers [1]							Total
	Gross-receipts taxes [2]	Mileage, ton-mile, and passenger-mile taxes	Special license fees and franchise taxes [3]		Certificate or permit fees [3]	Caravan taxes	Miscellaneous receipts	
			On weight or capacity basis	On flat rate basis				
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Alabama		195			6			201
Arizona	166							166
Arkansas					1			1
California	2,595			83		57		2,735
Colorado		583			11			594
Connecticut	253							253
Delaware [4]								
Florida		272		1	2			275
Georgia		4		67	3			74
Idaho	24		46		1	9		80
Illinois [4]								
Indiana			619	138	10			767
Iowa		473		64				537
Kansas		1,152			15			1,167
Kentucky		273			39		[5] 18	330
Louisiana					8		3	11
Maine				5	12		2	19
Maryland [6]								
Massachusetts				84	11		[5] 4	99
Michigan		427						427
Minnesota					40			40
Mississippi		44	73		6		6	129
Missouri			492					492
Montana	26			13	3			42
Nebraska				24	23			47
Nevada			152	38		3		193
New Hampshire				3				3
New Jersey		74						74
New Mexico		174			3			177
New York [4]								
North Carolina	253							253
North Dakota		3			13		1	17
Ohio			469					469
Oklahoma		1,456			8			1,464
Oregon	292	504		248			25	1,069
Pennsylvania	13							13
Rhode Island				9	1			10
South Carolina		80	147				[5] 3	230
South Dakota		12	443		22			477
Tennessee		338	58		2			398
Texas				100	8			108
Utah		9						9
Vermont [4]								
Virginia	248				5			253
Washington	16		117	19	37			189
West Virginia		79						79
Wisconsin		334	1,246		431		3	2,014
Wyoming		181			25	14		220
District of Columbia		114		102				216
Total	3,886	6,781	3,862	998	746	83	65	16,421

[1] Complete classification of motor-carrier tax receipts is not available in all States. The classified receipts, in some cases, include miscellaneous small receipts not classified.

[2] Numerous States impose taxes on the gross receipts of motor carriers in connection with general State sales taxes or taxes on all transportation companies or public utilities. This column includes only the proceeds of gross-receipts taxes reported by the States as special taxes on motor carriers.

[3] It is often difficult to make a distinction between the 3 classes of receipts listed in the third, fourth, and fifth columns of figures. In general, the proceeds of special weight or capacity taxes and taxes imposed at a flat rate per vehicle are included under special license fees and franchise taxes, application or filing fees required for the issuance of certificates of convenience and necessity to common carriers and corresponding permits to contract and other motor carriers are included under certificate or permit fees.

[4] No special taxes on motor carriers reported.

[5] Motor-carrier drivers' licenses.

[6] Ton-mile and passenger-mile taxes paid by motor carriers in lieu of registration fees included in table, State Motor-Vehicle Receipts, 1938.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF MAY 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	NUMBER: Grade Crossings Eliminated by Separation or Relocation	NUMBER: Grade Crossing Structures Reconstructed	NUMBER: Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	NUMBER: Grade Crossings Eliminated by Separation or Relocation	NUMBER: Grade Crossing Structures Reconstructed	NUMBER: Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	NUMBER: Grade Crossings Eliminated by Separation or Relocation	NUMBER: Grade Crossing Structures Reconstructed	NUMBER: Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 253,090	$ 252,891	6		6	$ 1,229,062	$ 1,227,124	15	1		$ 62,800	$ 55,800	2		1	$ 842,726
Arizona						229,905	227,701	3			268,471	245,000	2			282,973
Arkansas	575,492	573,236	13			127,127	127,045	2			166,256	166,256	3			1,235,963
California	1,362,358	1,361,783	5	3		1,691,373	1,690,278	10			80,272	80,272	1			1,296,732
Colorado	84,715	81,327	2		2	487,708	487,708	4			46,030	42,268			19	928,224
Connecticut						18,930	12,665				171,920	166,540		1		832,360
Delaware	33,995	33,516			14	45,420	45,420	2		9	2,320	2,320			1	509,994
Florida	17,416	17,416	1			428,094	428,094	2			79,700	79,700	1		1	1,158,058
Georgia						436,950	436,950	7			138,600	138,600	2	2	5	2,319,120
Idaho	180,246	172,543	4			388,794	357,136	5								454,970
Illinois	534,280	534,280	4		11	2,577,545	2,520,545	17	3	53	713,450	713,450	3		24	2,467,151
Indiana	688,790	578,620	4	4		894,116	867,216	3	1	89	169,040	169,040		1	10	1,313,772
Iowa	1,038,701	1,001,200	12	2	20	311,091	272,806	6			176,113	165,100	7		11	1,673,923
Kansas	552,846	552,740	5		5	978,673	978,678	12			121,659	121,659	2		8	1,426,390
Kentucky	165,688	165,688	1		8	667,203	667,203	9	4	23	277,438	229,489	2	1		1,108,511
Louisiana	11,980	11,980				435,221	428,478	4			394,361	393,570	10			1,053,899
Maine	53,997	53,877	2			409,266	409,266	3	3	1	67,020	67,020	1			296,231
Maryland						72,189	72,189	1			228,200	131,407	1		4	1,023,901
Massachusetts	54,710	54,710		1		540,425	539,162	4	3							1,727,702
Michigan	957,084	915,797	8	1	45	628,626	628,626	5	2		252,690	252,690	1		19	2,137,219
Minnesota	38,606	38,332		1	1	780,054	779,733	3	5	2	552,469	551,149	4	1	11	1,601,920
Mississippi	356,600	356,600	4			603,614	603,614	8								934,587
Missouri	296,960	295,421	4	1		1,082,570	1,082,570	5	1		567,910	564,120	4	2		1,679,326
Montana	365,654	360,772	4			860,225	860,225	9			29,070	29,070	1			335,656
Nebraska	156,731	156,459	6			938,073	938,073	26			436,342	436,342	4		33	581,225
Nevada	161,386	161,033		3	6	237,364	237,364	1	1		30,558	30,558			11	127,618
New Hampshire	70,205	69,765		1	2	67,609	67,562	5		1	102,775	102,302	3			351,621
New Jersey	125,381	120,155	1	1	1	557,101	557,101	2	2	1						1,682,615
New Mexico	264,915	264,649	7	1		99,655	99,655	2			2,861	2,861			1	651,283
New York	1,032,101	1,027,600	5	3	2	1,980,555	1,975,205	4	10		141,300	140,850		1	1	4,962,223
North Carolina	154,540	154,540	2	1	1	1,316,400	1,281,300	7	7	12	344,210	344,210	2	1	29	1,165,321
North Dakota	209,450	208,387		1		865,312	816,910	8	1		225,990	225,990	4			637,098
Ohio						844,902	808,140	10	1		890,980	890,980	6		2	3,363,901
Oklahoma	40,774	30,792			4	330,960	296,960	2		43	38,600	38,600			8	2,370,197
Oregon	675,679	540,671	2	2		39,002	39,002				129,997	129,997	2			499,890
Pennsylvania	213,129	197,923	2			1,967,294	1,755,395	3	3		6,200	6,200			1	4,901,633
Rhode Island						438,791	438,791	1	3							152,459
South Carolina	71,586	71,136	1	1	2	648,088	593,572	17	2	2	148,179	148,179			36	969,965
South Dakota	128,909	128,276	2		9	281,970	281,970	5	2		45,750	45,750	1		11	1,132,919
Tennessee	12,460	12,460			4	690,870	690,870	2	3	2	181,800	181,800	1	1	3	1,380,090
Texas	907,616	905,342	15	3		2,461,147	2,430,362	22	2		775,513	747,615	6	1		2,348,659
Utah	111,307	111,307	2		4	37,700	37,700	2			314,590	314,590			118	238,940
Vermont	245,681	230,614	6	2	8	9,806	9,806			1	20,630	20,440			6	317,470
Virginia	506,768	505,695	17	3	4	489,013	400,013	6		2	368,462	368,462	4	2	18	900,508
Washington	403,227	391,758	5	3	2	667,163	665,753	7	1		86,637	86,637		1	13	541,588
West Virginia	221,081	217,381	1		3	399,541	383,781	7		2	18,800	18,800	1		1	964,852
Wisconsin	202,131	200,987	3			1,194,012	1,153,188	11			486,783	466,619	4	2	4	1,138,789
Wyoming	154,992	154,992		2	4	207,460	128,040	1	1		17,010	17,010			7	508,822
District of Columbia	30,215	30,215		1							283,544	243,750	1			134,436
Hawaii	3,820	3,650				226,770	226,770	5	1							360,830
Puerto Rico	61,900	61,550	2			222,399	220,980	6			180,009	179,127	3			413,719
TOTALS	13,789,192	13,370,066	158	41	168	33,143,143	32,284,695	301	63	243	9,843,309	9,552,189	89	17	417	61,444,979

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF MAY 31,1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 6,865,382	$ 3,146,870	239.4	$ 8,201,012	$ 4,087,296	307.8	$ 790,650	$ 394,870	25.1	$ 3,195,425
Arizona	2,478,830	1,791,914	125.5	1,087,983	770,477	44.4	387,516	275,336	24.2	1,891,240
Arkansas	1,807,728	1,792,834	107.1	3,180,413	3,176,988	207.9	287,230	284,675	16.5	1,739,673
California	10,660,759	5,749,563	242.7	5,446,871	2,990,017	73.8	708,497	373,994	7.4	4,526,559
Colorado	2,668,693	1,418,126	104.0	4,482,913	2,491,726	115.1	526,310	294,780	13.4	2,334,908
Connecticut	1,108,390	540,911	11.0	991,398	491,639	11.5	1,175,632	584,078	10.5	1,331,528
Delaware	737,221	363,900	17.8	469,157	233,315	6.3	959,430	439,005	28.1	1,309,553
Florida	3,187,007	1,554,568	65.6	2,686,920	1,343,460	58.8	360,000	180,000	6.4	3,604,620
Georgia	5,264,142	2,520,321	266.6	5,184,610	2,592,305	265.0	2,798,090	1,399,045	151.4	6,430,397
Idaho	2,217,937	1,258,987	207.3	1,615,454	968,318	51.0	585,952	357,952	11.3	1,677,410
Illinois	11,694,439	5,779,935	313.9	9,045,107	4,519,658	194.3	2,785,837	1,402,873	69.2	4,058,327
Indiana	6,100,734	2,944,390	154.4	4,637,526	2,267,413	93.3	3,382,522	1,638,509	83.8	2,717,697
Iowa	7,697,389	3,649,025	263.5	4,749,799	2,060,133	169.8	992,251	466,100	33.4	2,098,676
Kansas	5,450,953	2,695,427	729.9	4,159,141	2,071,865	182.2	3,949,338	1,974,669	214.7	4,129,725
Kentucky	5,577,596	2,755,411	209.2	3,387,868	1,693,934	76.9	1,712,357	854,622	45.6	3,290,557
Louisiana	1,520,282	750,988	38.3	11,249,627	2,716,380	52.5	2,034,965	996,706	28.5	2,703,740
Maine	2,852,507	1,392,847	65.0	1,746,024	873,011	35.6	229,340	114,670	8.3	885,654
Maryland	1,085,456	540,462	17.1	2,809,978	1,393,851	46.1	1,478,470	601,000	23.4	1,966,544
Massachusetts	2,224,695	1,112,271	12.7	3,597,796	1,796,208	27.4	1,690,757	842,320	11.5	2,569,343
Michigan	8,367,645	3,935,500	174.1	4,606,814	2,302,912	135.7	1,044,005	518,800	27.1	3,472,482
Minnesota	5,016,326	2,403,668	311.8	5,958,771	2,957,196	290.0	1,932,467	964,549	138.1	4,196,914
Mississippi	6,571,088	2,879,023	284.0	7,506,132	2,704,636	322.7	964,960	389,026	24.6	3,017,235
Missouri	5,874,711	2,821,779	163.2	4,066,976	2,004,496	120.5	3,515,532	1,732,020	141.6	4,821,352
Montana	1,976,018	1,109,693	91.0	2,961,682	1,672,769	151.3	1,048,187	545,933	55.3	4,547,527
Nebraska	4,596,076	2,191,021	389.2	5,314,067	2,676,027	455.0	2,972,212	1,487,606	297.4	2,894,348
Nevada	1,605,062	1,359,894	181.3	1,547,105	1,333,852	51.1	5,525	4,752	.3	1,685,095
New Hampshire	1,178,535	579,858	23.7	155,856	77,222	1.9	1,499,682	641,278	41.2	1,064,453
New Jersey	2,637,665	1,305,520	18.3	3,238,336	1,616,613	28.8	651,440	324,905	2.9	2,554,918
New Mexico	2,663,604	1,718,927	284.9	1,738,989	1,059,524	56.8	254,901	159,078	52.3	1,700,652
New York	14,674,939	6,929,962	260.4	11,788,050	5,761,000	192.7	1,651,100	784,550	29.7	4,878,073
North Carolina	7,425,054	3,518,610	312.1	6,178,259	3,085,072	381.5	1,219,850	593,015	61.6	2,571,387
North Dakota	3,437,179	3,230,743	260.9	434,490	243,744	57.5	2,695,724	1,444,843	280.4	3,741,698
Ohio	8,688,354	4,269,480	103.3	8,965,962	4,413,652	90.1	2,478,020	1,238,680	35.8	7,719,985
Oklahoma	6,512,822	3,402,954	259.8	2,277,875	1,206,176	42.7	1,695,511	902,011	48.2	4,367,419
Oregon	3,193,688	1,851,995	111.0	2,302,916	1,404,057	123.7	848,583	508,595	50.5	2,325,013
Pennsylvania	8,606,088	4,219,429	142.1	9,873,562	4,763,267	92.5	2,449,516	1,212,229	28.3	5,159,514
Rhode Island	1,303,817	643,270	17.3	283,232	141,616	2.6	808,450	404,105	9.2	1,135,058
South Carolina	5,344,560	2,368,578	266.4	2,936,804	1,310,986	86.4	12,800	5,800		2,455,379
South Dakota	2,098,918	1,173,486	259.0	4,561,349	2,522,510	440.8	1,355,820	774,810	103.5	3,565,685
Tennessee	6,342,829	3,144,817	199.3	3,586,924	1,793,462	75.2	1,076,200	538,100	42.6	4,753,103
Texas	15,787,639	7,781,722	997.8	13,691,525	6,734,943	616.7	2,189,313	1,082,315	159.7	7,433,747
Utah	1,381,405	924,538	110.9	2,334,010	1,642,830	80.0	338,825	213,007	22.4	1,220,382
Vermont	1,295,915	610,413	33.9	726,484	345,593	17.7	196,970	98,295	4.3	625,523
Virginia	6,996,034	3,490,418	240.3	2,410,592	1,202,986	65.2	1,360,156	678,228	34.4	1,682,467
Washington	4,755,291	2,454,520	113.7	2,417,868	1,263,050	24.6	1,344,410	687,800	15.2	1,395,555
West Virginia	1,851,636	1,309,930	66.7	1,638,812	822,136	39.1	2,032,752	997,715	46.9	2,249,743
Wisconsin	5,061,870	2,498,459	176.2	6,772,589	3,311,880	183.3	2,333,622	1,129,715	81.2	2,379,411
Wyoming	2,515,959	1,526,600	281.3	1,318,352	813,064	119.5	405,066	226,576	41.4	1,178,481
District of Columbia							200,400	100,200	1.2	387,300
Hawaii	1,107,990	544,953	21.9	937,620	460,715	13.3	436,537	200,128	5.7	1,312,925
Puerto Rico	598,026	294,485	11.9	1,680,341	835,295	34.5	180,179	89,230	3.1	502,865
TOTALS	230,666,883	118,252,995	9,358.7	202,941,941	101,021,275	6,413.1	68,033,859	34,153,098	2,698.8	145,457,265

U. S. GOVERNMENT PRINTING OFFICE: 1939

PUBLICATIONS of the BUREAU OF PUBLIC ROADS

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Department and as the Department does not sell publications, please send no remittance to the United States Department of Agriculture.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1938. 10 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 25 cents.

No. 191MP . . Roadside Improvement. 10 cents.

No. 272MP . . Construction of Private Driveways. 10 cents.

No. 279MP . . Bibliography on Highway Lighting. 5 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

Federal Legislation and Rules and Regulations Relating to Highway Construction. 15 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. 60 cents.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.

No. 265T. . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Bureau of Public Roads upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP. . Bibliography on Highway Safety.

House Document No. 272 . . . Toll Roads and Free Roads.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF MAY 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 234,900	$ 117,450	18.4	$ 834,850	$ 412,050	38.6	$ 281,200	$ 57,000	.3	$ 776,746
Arizona	453,523	295,507	36.1	178,710	125,948	17.3	156,461	95,882	18.2	352,655
Arkansas	84,303	77,740	7.8	365,195	362,504	40.2	216,817	215,819	35.5	463,432
California	1,829,287	1,020,549	106.5	1,011,858	516,201	48.4	144,022	83,604	7.6	796,555
Colorado	1,013,189	531,073	58.9	430,110	227,544	17.2	411,300	198,662	12.6	235,477
Connecticut	89,160	44,560	1.4	190,504	81,072	3.1				286,249
Delaware	22,730	11,365	5.3	80,840	40,420	17.5	56,990	28,495	7.6	239,720
Florida	20,122	10,061		762,533	380,450	26.3	112,300	51,950	5.6	432,644
Georgia	452,681	216,050	54.1	500,726	250,363	68.0	170,780	85,390	23.4	1,083,775
Idaho	497,893	222,141	57.2	141,851	84,508	2.6	104,744	55,679	8.6	295,511
Illinois	1,869,237	919,055	161.1	1,426,632	659,316	74.1	482,300	232,650	33.1	824,301
Indiana	686,994	288,067	75.9	1,013,370	500,585	85.2	306,177	143,452	22.1	674,221
Iowa							47,751	22,015	35.4	1,657,792
Kansas	223,100	111,544	23.1	76,778	38,389	18.5	337,700	168,850	8.9	1,388,064
Kentucky	798,767	243,871	106.1	827,302	237,026	34.3	817,962	236,063	90.4	375,487
Louisiana	173,954	82,335	15.8	628,292	268,940	48.1	420,276	189,660	37.0	398,713
Maine	367,659	178,826	23.3	259,316	124,811	12.5	26,634	13,317	2.1	147,458
Maryland				188,974	94,487	15.1	142,000	52,355	10.2	388,839
Massachusetts	57,625	28,490	.9	248,093	123,561	4.7	425,290	209,605	8.6	491,067
Michigan	390,291	191,920	34.2	1,008,104	504,052	62.7	682,200	317,700	37.1	973,816
Minnesota	273,069	126,708	42.2	702,410	349,161	61.1	114,184	57,092	1.4	1,252,244
Mississippi				325,662	162,831	23.8	379,100	189,450	37.5	798,585
Missouri	460,053	219,093	57.8	698,120	325,640	62.1	510,000	228,015	99.4	740,888
Montana	14,071	7,865		125,531	71,135	10.8	595,970	335,609	42.0	925,284
Nebraska	583,291	282,472	95.6	756,444	369,266	143.3	221,378	101,375	44.3	551,345
Nevada	427,436	34[illegible],390	68.8	120,169	104,184	15.5	26,563	23,035	1.6	214,637
New Hampshire	218,767	108,445	6.0	60,759	29,708	2.4				181,847
New Jersey	171,820	79,020	2.5	332,120	164,010	8.3	134,520	66,375	9.4	561,573
New Mexico	625,191	380,030	42.1	550,676	335,062	36.0	137,262	79,009	7.5	251,965
New York	2,306,430	1,115,917	167.4	1,899,000	949,500	99.6	127,000	55,000	7.3	968,966
North Carolina	695,412	346,576	77.2	1,077,044	538,500	102.4	308,800	147,960	32.7	397,347
North Dakota	108,510	56,615	26.8	115,030	61,606	8.3	42,770	22,907	8.2	875,949
Ohio	147,535	73,767	3.8	357,610	185,580	14.5	435,680	217,840	25.2	1,891,932
Oklahoma	304,728	160,942	35.8	167,850	89,311	7.1	602,040	297,148	32.4	985,103
Oregon	471,113	274,000	63.2	337,945	203,492	38.9	248,846	149,320	22.7	376,545
Pennsylvania	1,818,211	858,708	128.4	2,128,643	1,046,540	118.1	155,992	77,996	6.4	767,700
Rhode Island	70,486	33,379	3.5	194,923	97,438	5.8				130,199
South Carolina	587,550	254,282	68.1	672,277	278,769	67.8	169,800	66,200	12.4	278,661
South Dakota	11,519	6,250								1,058,050
Tennessee	273,975	129,470	14.8	762,064	308,132	34.2	125,640	62,820	.6	871,038
Texas	3,244,257	1,536,859	466.9	2,332,640	1,107,801	233.8	344,079	168,536	41.1	1,238,037
Utah	603,604	308,008	53.2	242,850	112,763	22.0	112,625	60,401	10.0	237,908
Vermont	232,410	106,201	13.8	90,306	45,153	4.0	43,300	20,500	.5	110,867
Virginia	864,925	378,562	74.6	650,974	323,724	63.9	111,660	47,622	10.1	415,340
Washington	560,544	291,226	64.5	705,929	370,896	40.4	42,369	22,300	1.2	280,706
West Virginia	242,491	119,483	21.4	153,296	76,648	8.3				515,848
Wisconsin	548,482	263,669	23.4	725,191	357,540	33.2	428,654	202,717	5.0	751,110
Wyoming	416,758	254,565	59.0	356,182	220,069	20.2	112,098	70,081	5.8	227,835
District of Columbia										73,125
Hawaii				170,080	85,040	4.6	22,900	11,450	1.3	223,510
Puerto Rico	224,621	110,876	11.3	131,604	64,530	8.8	140,883	68,620	6.1	75,974
TOTALS	25,772,674	12,818,982	2,478.2	27,117,367	13,466,256	1,933.6	11,037,017	5,307,526	876.4	30,512,640

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PUBLIC ROADS

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FEDERAL WORKS AGENCY

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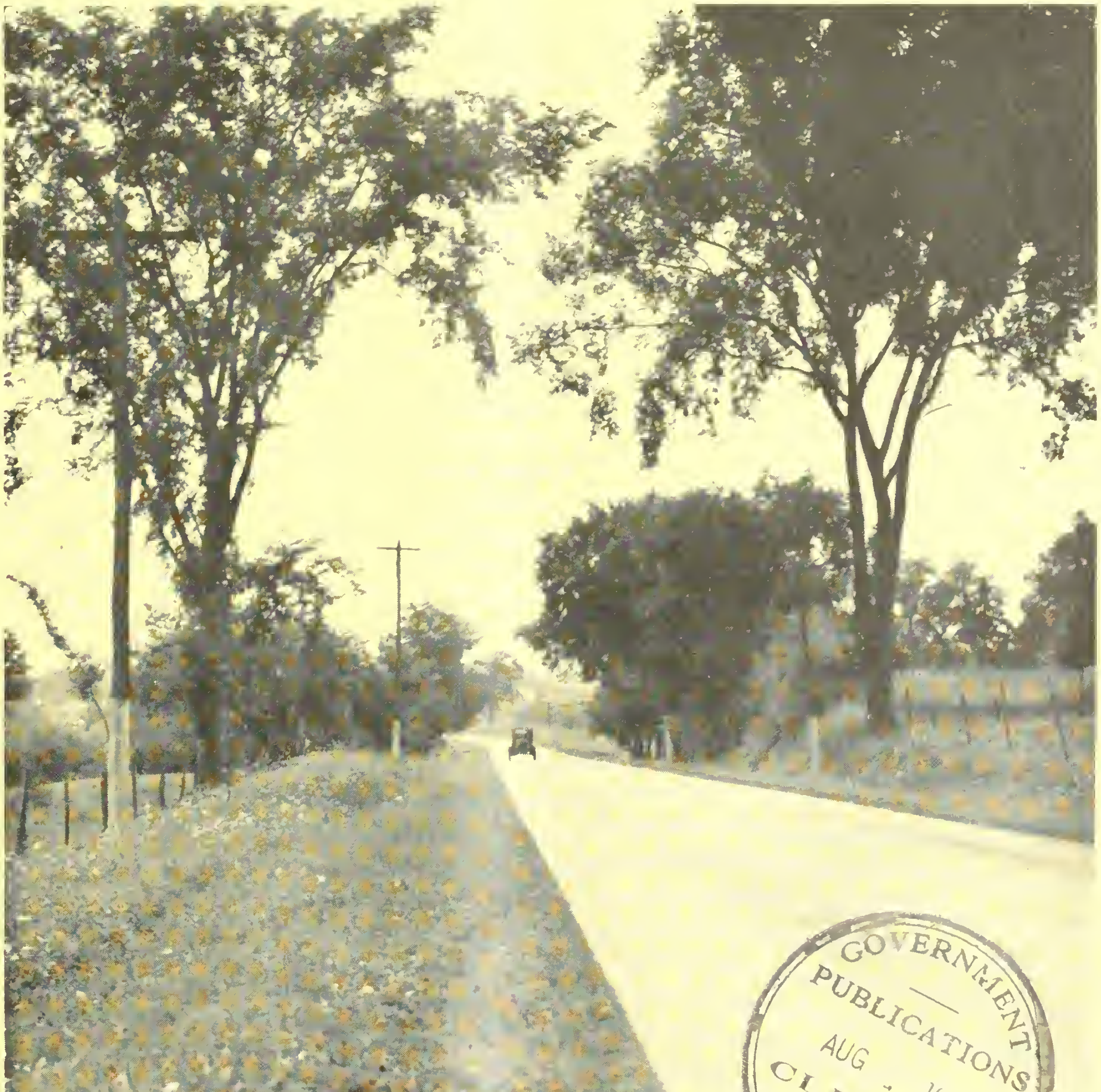
VOL. 20, NO. 5 JULY 1939



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PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 20, No. 5 July 1939

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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APPLICATION OF THE RESULTS OF RESEARCH TO THE STRUCTURAL DESIGN OF CONCRETE PAVEMENTS[1]

Reported by E. F. KELLEY, Chief, Division of Tests, Public Roads Administration

DURING the past 20 years many studies have been made of the various factors that influence the structural performance of concrete pavement slabs and the numerous reports of these investigations are scattered through the technical literature. Most of these reports, of necessity, are highly technical and the mass of data presented and the detailed descriptions that are included, both as a matter of record and in order that the reader might have confidence in the validity of the results, frequently tend to obscure the value and importance of the conclusions.

In addition, individual reports frequently cover but a single phase of a given subject and are useful only when considered in connection with the available reports dealing with the remaining phases of the same subject. The net result of this situation is that many facts that have been well established by research are little appreciated and too frequently are given scant consideration in the practical design of pavements. It is the purpose of this paper to bring together under one head and make available for the practical use of the designing engineer the important facts that have been developed thus far in research work relating to the structural design of concrete pavements.

In the field of bridges and buildings the basic principles of design have become so well established that, to many engineers, the term "structural design" conveys the idea of a rather exact and accurate mathematical procedure to be followed in proportioning the several parts of a structure. No such presumed accuracy exists in connection with the structural design of concrete pavements.

From the standpoint of stress analysis the concrete pavement is a highly complex structure. It is supported by soil whose physical properties vary appreciably at different locations, at different points in the same general location, and even at different times at the same point. It is subjected to the action of external forces produced by the wheels of vehicles and the magnitude of these forces and their effect on pavement stresses are influenced by a number of variables. In addition, it is constantly subjected to high internal stresses produced by changes in temperature and moisture. Much has been learned concerning the influence of the different variables on pavement stresses but a great deal of additional research is still needed. However, on the basis of available information, reasonable assumptions of sufficient accuracy can be made to insure a pavement structure that will function in a satisfactory manner.

Structural design, in general, is distinguished by the use of conservative unit stresses which, for structural steel, are well below the elastic limit and, for concrete, well below the ultimate strength. This results in the so-called factor of safety which is depended upon to provide for all the unknown conditions for which it is impossible to make definite provision. In contrast to this the current designs of concrete pavements are generally such that the factor of safety, if any, is so small as to be almost negligible.

The maximum combined stresses due to external loads and to temperature in pavement slabs of the dimensions commonly used will very frequently be found to be so close to the ultimate strength of the concrete that there is little or no margin left to provide for unknown or unforseen conditions. In making this statement there is no intention to imply any general criticism of present practice since the present standards of design have proven reasonably adequate. When the need for the great mileage of existing pavements and the fact that structural failures of these pavements do not generally endanger human life are considered, it seems probable that any significant increase in cost to provide a margin of safety comparable to that provided in bridges, could not have been justified from the economic standpoint. However, it is important to recognize that the low or negligible factor of safety that is provided in designing concrete pavements makes it highly desirable to be somewhat conservative in assuming design values for the different variables that must be considered.

IMPACT REACTION DEPENDENT ON FOUR VARIABLES

Wheel loads and impact.—Neglecting the unpredictable forces caused by localized differential heaving or subsidence of the subgrade soil, the external forces that create stress in the pavement slab are produced by vehicles. Naturally, the heavier vehicles are the more important.

One of the earlier investigations (*1*)[2] developed the important fact that for heavy vehicles of the usual type, that is, four- or six-wheel trucks or trailers, the critical stress developed in a concrete pavement, when the axle spacing is in excess of about 3 feet, is primarily a function of the wheel load and not a function of the gross load on the vehicle or the axle spacing. By means of his theoretical analysis, Westergaard (*2*) subsequently arrived at the same conclusion and this has been confirmed by later tests (*3*). This finding, which permits attention to be confined to wheel loads rather than gross loads, greatly simplifies a problem already sufficiently complicated.

The magnitude of the vertical force exerted on a pavement by the wheel of a moving vehicle may be considered to be the sum of the static weight of the loaded wheel and the additional impact or dynamic force created by the movement of the wheel over the irregularities that exist in the pavement surface. The researches of the Bureau of Public Roads have demonstrated conclusively that the impact reaction of a moving wheel is sufficiently in excess of the static wheel load to make it an important factor in pavement design.

The impact reaction of a moving wheel depends upon four major variables—wheel load, vehicle speed, tire

[1] Paper presented at the annual meeting of the American Concrete Institute, March 1939. Because of its length, this report will be presented in two issues of PUBLIC ROADS. The second installment will appear in the August issue.

[2] Italic figures in parentheses refer to the bibliography, p. 102.

equipment, and road roughness (*4*). Other variables exert some influence but, in general, these four are the important ones. An increase in wheel load or pavement roughness; a decrease in the cushioning qualities of the tires; and, within limits, an increase in vehicle speed; all result in increased impact reactions.

The tests that have been made have amply demonstrated the fact that the magnitude of the impact reaction is a function of the wheel load. Also, these tests have brought out important facts, not previously known, regarding the relation between wheel load and the impact reaction that it produces. In bridge design it is customary to express impact as a percentage of the static live load. Therefore it is important to observe that while the total impact reactions of the wheels of motor vehicles increase with increase in wheel load, the percentage of impact, or the ratio of the dynamic increment to the static load, actually decreases as the wheel load is increased. This fact may be attributed largely to the relative effects of sprung and unsprung weights, and to the relation between size of tire and its cushioning properties.

The force which the wheel of a vehicle delivers to the road surface is made up of two component forces. One of these is caused by the unsprung weight on the wheel (that is, the weight of the parts not supported by the springs), and the other is caused by the spring pressure on the axle at the instant of impact. The part of the total impact reaction caused by the unsprung weight is, in general, considerably greater than the part caused by the sprung weight. However, the ratio of unsprung weight to total weight is not a constant but decreases as the total or gross weight is increased. Also, as the wheel load is increased the tire size is increased and with it the ability of the tire to minimize the effect of surface irregularities. The result is that for a given condition of road roughness an increase in wheel load is not accompanied by a corresponding percentage increase in the dynamic component of the impact reaction.

The magnitude of the impact force is greatly dependent on the type and condition of the tire equipment. Solid, cushion, and pneumatic tires, in the order named, produce impact reactions of decreasing magnitude. The tests that developed this information were made at a time when rubber tires of the solid and cushion types were commonly used. Fortunately, these types are no longer in general use. The relatively few solid tires that are now used must be operated at such low speeds that, in comparison with the pneumatic tires used on high-speed trucks and busses, they need be given no consideration from the standpoint of impact. Therefore attention may be confined to pneumatic tires.

With respect to pneumatic tires it has been found (*5*) that, other conditions being the same, the dynamic increment of the impact reaction of high-pressure and balloon tires is closely proportional to their inflation pressures. Therefore, it follows that for a given wheel load the impact reaction created by low-pressure balloon tires is appreciably less than that caused by high-pressure tires. From the standpoint of pavement protection the balloon tire offers the additional important advantage that it applies the load to the pavement over a larger area of contact, a condition that results in a lower slab stress. This relation will be discussed in detail later.

INTENSITY OF IMPACT DECREASES AS FREQUENCY OF OCCURRENCE INCREASES

Another fact with respect to the effect of tire equipment is that dual tires generally give somewhat higher impact reactions than do single tires of the same type and same load capacity. The difference is a variable which, from the practical standpoint, may safely be ignored since the increased stress in a concrete pavement slab resulting from the greater impact effect of dual tires may generally be expected to be more than offset by the reduction in stress resulting from their greater area of load application. For example, if it be assumed that a certain wheel load on dual high-pressure tires produces an impact reaction of 10,000 pounds then the minimum reaction that may reasonably be expected from the same load on a single high-pressure tire of comparable capacity would be of the order of 9,000 pounds. With reasonable assumptions as to area of tire contact and other variables the computed stresses, by the original Westergaard analysis (*2*), for loads applied at the interior of a 6-inch slab, are about 330 pounds per square inch for the 9,000-pound load on the single tire and about 315 pounds per square inch for the 10,000-pound load on the dual tires.

When a wheel runs over an obstruction, such as an inclined plane or a rectangular block, two types of vertical impact reactions are developed. One is caused by shock as the wheel strikes the obstruction and the other is caused by the drop of the wheel from the obstruction to the pavement. In the earlier investigations involving pneumatic tires operated over artificial obstructions at speeds up to about 55 miles per hour (*5*), it was found that the shock reactions increased approximately in direct proportion to speed. It was also found that drop reactions reached maximum values at relatively low speeds, of the order of 25 to 35 miles per hour, and that these were not exceeded by the shock reactions except at speeds of the order of 50 miles per hour. In a subsequent investigation (*6*) involving only balloon tires, it was found that the use of artificial obstructions resulted in maximum drop impacts at speeds of from 20 to 40 miles per hour and that these were not exceeded by shock impacts at speeds up to 70 miles per hour.

From these tests with artificial obstructions it might be concluded that the effect of speed on impact reactions is not important for speeds in excess of 40 miles per hour. However, such a conclusion would require some modification as a result of the tests (*6*) that have been made to determine impact reactions resulting from the natural roughness of road surfaces. These tests were made at 28 locations where the natural roughness was as severe as would permit the safe operation of a heavy vehicle at high speed. In each of these 28 locations the shape of the curve of impact reaction versus speed was different depending on the characteristics of the particular roughness condition.

In some cases the maximum impacts were observed at relatively low speeds but in the majority of cases the impact reactions showed a general tendency to increase with increases in speed up to the maximum of 70 miles per hour. However, this statement applies to individual locations. When all the maximum impact reactions were plotted against speed it was found that a general maximum was reached at about 50 miles per hour and that this remained constant up to 70 miles per hour, the maximum speed attained in the tests (fig. 24, Public Roads, Nov. 1932). Therefore, it seems reasonable to conclude that the effect of speed on impact reaction may be neglected for speeds in excess of 50 miles per hour.

Two investigations have been made to determine the effect of conditions of general road roughness on the magnitude of impact reactions (*6*, *7*). This is in contrast to the study of extreme conditions of roughness already described. In these tests, roads of various degrees of roughness, as determined by the relative roughness indicator (*8*), were selected for study and the test vehicles with different wheel loads and different tire equipments were operated over them at various speeds.

It was found that, other conditions being the same, there was a rather definite relation between the magnitude of the impact reaction and the frequency of its occurrence. Of the great number of impacts that may occur on a given section of road, those of the greatest magnitude occur only a few times while those of lesser intensity occur a greater number of times and the intensity decreases as the frequency of occurrence increases. For example, in the tests with a motor bus equipped with balloon tires and operated at a speed of 40 miles per hour over a very rough concrete road, it was found that the impact factors (ratio of total impact reaction to static wheel load) for frequencies of 1, 40, 80, and 100 times per mile were approximately 2.20, 1.65, 1.55, and 1.50, respectively. However, the magnitude of the impact factor for a given frequency becomes less as the roughness of the pavement decreases. The impact factors for the same vehicle as described above, operated at the same speed of 40 miles per hour over a smooth concrete pavement, were approximately 1.25 and 1.18 for frequencies of 1 and 100 per mile, respectively.

It is immediately apparent from this relation between frequency and magnitude of impact factors that, from the standpoint of pavement design, it is necessary to select some reasonable frequency and to compute dynamic loads on the basis of the impact factor corresponding to this frequency. Designing a pavement for a maximum load that may occur only once per mile would certainly be open to serious question and it is necessary to select an impact force that occurs with sufficient frequency to be of practical importance. A frequency of 100 per mile, corresponding to the maximum impact reaction that may be expected to occur on an average of once every 50 feet, is suggested as a reasonable assumption.

The existing data do not permit the evaluation, from any single series of tests, of all the variables that have been discussed. However, some of the variables have been studied in each series of tests and it is possible, by interpolation and extrapolation, to combine the data in the reports that have been mentioned (*4*, *5*, *6*, *7*) so as to give impact factors that are in agreement with our present knowledge of the subject and which are sufficiently accurate for purposes of design. Such impact factors for a range of static loads on wheels equipped with dual high-pressure and balloon tires, a speed of 50 miles per hour on a pavement having a reasonable degree of smoothness (neither extremely rough nor extremely smooth), and a frequency of 100 per mile, are given in table 1.

The pavements on which impact-frequency studies were made were rated with respect to degree of roughness with the relative roughness indicator (*8*) and it is interesting to observe that, with minor exceptions, the order of rating would have been the same had they been rated for roughness by means of the impact-frequency curves. In other words, the roughness indicator gave a qualitative measure of the characteristics of the pavement surface that determine the magnitude of impact. However, while the roughness indicator is a useful instrument, it is not one of precision. As it has commonly been used the motor vehicle on which it is mounted becomes an integral part of the instrument and the results are reproducible only with the same car operated under the same conditions. Therefore, while a given instrument mounted on a given car gives a qualitative measure of the relative roughness of different road surfaces, it is not possible to express these results in absolute figures.

Table 1.—*Impact factors and total impact-road reactions*

Speed—50 miles per hour.
Frequency—100 per mile.
Condition of pavement surface—reasonably smooth.

Static wheel load, pounds	Dual high-pressure tires		Dual balloon tires	
	Impact factor	Total impact reaction	Impact factor	Total impact reaction
		Pounds		*Pounds*
4,000	2.05	8,200	1.70	6,800
5,000	1.80	9,000	1.54	7,700
6,000	1.67	10,000	1.43	8,600
7,000	1.56	10,900	1.37	9,600
8,000	1.48	11,800	1.31	10,500
9,000	1.41	12,700	1.27	11,400
10,000	1.36	13,600	1.24	12,400

The tests that form the basis for the data given in table 1 were made on pavements that appeared to represent reasonable average conditions of surface roughness, intermediate between extremely smooth and extremely rough surfaces. A more precise definition cannot be given. On account of this variable and the others that affect the magnitude of the impact reactions, the data given in table 1 can be considered only as approximate. They represent the best estimate that can be made, on the basis of existing data, of the maximum impact reactions, important with respect to design, that can reasonably be expected to occur as the result of the normal operation of the heavier motor vehicles. The digit in the second decimal place in the figures for impact factors is without significance. It is included merely for the purpose of making the impact factors agree with the total impact reactions which are given to the nearest hundred pounds.

IMPACT FACTOR USED SHOULD BE INDEPENDENT OF POSITION OF LOAD

As will be shown later, in a concrete pavement slab of uniform thickness the magnitude of the critical stress is greatly influenced by the position of the wheel load; that is, whether it is near an edge, a corner, or in the center of the slab. Since the higher impact reactions will be produced at the points where the surface irregularities are greatest, it follows that higher impact reactions may be expected in the vicinity of transverse joints and cracks than in the interior of the slab. In view of this consideration Bradbury (*9*) has suggested that a higher allowance for impact be made in the computation of stresses at transverse joints than in other portions of the slab. However, in plain (nonreinforced) pavements transverse open cracks are quite likely to develop at random, except in very short slabs, and thereby create a roughness condition similar to that at formed joints. When this takes place in a thickened-edge slab a condition of weakness is created at the broken edge of the slab along the crack that makes it desirable to overdesign rather than underdesign the thickness of the pavement.

Also when a truck wheel leaves the edge of the pavement and then rolls back on the slab from a shoulder that frequently is not at the same elevation, an impact reaction of considerable magnitude may be developed. These considerations lead to the conclusion that nice distinctions with respect to the position of the load on the pavement are unwarranted and that the same impact factor should be used irrespective of the position of the load.

DESIGN STRESS EQUAL TO 50 PERCENT OF ULTIMATE STRENGTH IS CONSERVATIVE

Fatigue limit of concrete.—Concrete, like other structural materials, will fail under repeated loads at unit stresses which are much less than the ultimate strength as determined by the stress at failure produced by one application of static load. The stress at which failure takes place under a very large number of loadings is known as the fatigue limit or the endurance limit and, for concrete, it is expressed as a percentage of the ultimate strength.

Investigations of the fatigue limit in flexure under static load (*10*, *11*, *12*) have shown that concrete may be subjected to an almost unlimited number of applications of a stress equal to about 55 percent of its ultimate strength without danger of failure. A similar study of the fatigue limit of concrete under impact loads (*13*) gave similar results although the maximum number of load applications was only about 83,000 as compared with the one or more million that are usually considered desirable in fatigue studies. From this study it was concluded that, with respect to fatigue, the behavior of concrete may be assumed to be very similar under both static and impact loads and that the same fatigue limit is applicable to both.

On the basis of these investigations it has become rather general practice to assume about 50 percent of the ultimate flexural strength as a safe value of the working stress for use in designing pavements to resist wheel loads. However, the fatigue limit of the order of 50 percent of the ultimate flexural strength of the concrete has been established by tests in which the load applications were repeated at relatively short time intervals, as many as 40 per minute in tests in which the loads were applied without shock. In contrast to this, under normal conditions of traffic the heavy wheel loads that produce maximum stress are applied to the pavement slab at relatively long time intervals.

Hatt concluded (*11*) that the fatigue limit is about the same for beams under continuous fatigue loading as for those under fatigue loading with short rest periods. This is based on tests in which the stress cycles were at the rate of 10 per minute and in which the rest periods were not between individual load applications but were at intervals of several hundred or several thousand stress cycles. It is by no means certain that the fatigue limit might not be considerably different, and possibly higher, for stresses applied at time intervals corresponding to those which occur between successive applications of heavy wheel loads to a pavement under traffic.

It is a well-known fact that stresses above the fatigue limit cause progressive inelastic deformation and final failure. However, the relation between intensity of stress above the fatigue limit and the number of repetitions of this stress that will cause failure is not well established even for rapid repetitions of stress. For less frequent repetitions nothing is known concerning it.

On the majority of highways the heavier vehicles constitute a small percentage of the total traffic and therefore the occurrence of maximum load stresses is relatively infrequent. It appears therefore that the present practice of assuming the design stress to be approximately 50 percent of the ultimate strength of the concrete is a conservative one insofar as the stresses due to maximum wheel loads are concerned. In view of the possibility that the fatigue limit for these infrequent repetitions of stress may be higher than is indicated by available data, this practice may introduce some factor of safety of unknown magnitude.

However, the limitation of the design stress to 50 percent of the ultimate strength is believed to be unduly conservative when the pavement slab is designed for the combined effect of stresses due to load and those due to temperature warping since, as will be shown later, the maximum combined stresses due to load and temperature occur only in the daytime during the spring and summer months. It is apparent, therefore, that the frequency of occurrence of maximum load stresses in combination with maximum temperature stresses is considerably less than the frequency of passage of the truck wheels that produce maximum load stresses. This is particularly true on those highways where the movement of heavy trucks is principally at night.

In attempting to establish safe unit stresses for use in the design of concrete pavement slabs several factors in addition to fatigue should be considered and these will be discussed later. It is sufficient here to point out that the many uncertainties regarding the fatigue characteristics of concrete render of doubtful value any refinements in the use of existing data.

STATIC LOAD STRESSES MAY EXCEED IMPACT LOAD STRESSES

Static stress versus impact stress.—With respect to the relative stress effects of static and impact loads, exhaustive tests by the Bureau of Public Roads (as yet unpublished) have shown that static and impact forces of the same magnitude, applied through rubber-tired truck wheels, produce approximately equal strains in concrete cantilever beams that are free to deflect. The procedure followed in making these tests has been described (*14*). However, it does not follow from this that the same relationship will exist in a concrete pavement slab resting on a subgrade. In fact, there is some evidence to indicate that it may not.

A very limited series of exploratory tests of the effect of impact loads on pavement slabs has indicated the possibility that the stresses due to impact loads may be somewhat less than those due to static loads and that the difference between the two may not be the same in all portions of the slab. Any differences of this character that may exist undoubtedly result from the complex interrelation between pavement slab and subgrade and from the difference in time duration of the load application. The maximum impact reaction due to a wheel load is effective only for a small fraction of a second while static loads must be applied to the pavement for several minutes before an equilibrium of load and strain is obtained.

In the Arlington tests [3] it was found that in a pavement slab the time duration of the load application had a very important influence on the observed fiber deformation. From the time a static load was fully applied to the slab the observed fiber deformations increased at a fairly uniform rate for a period of several minutes before equilibrium was reached. The increase in deformation during this period amounted to as much as 15 percent. As a result (*15*), in all the studies of the effect of static loads, the loads were held constant for a period of 5 minutes after application before deformation measurements were made. The measured strains were therefore larger than would be caused by the momentary application of loads of the same magnitude.

However, even if significant differences are eventually found to exist between static and impact stresses in a pavement slab, there are no means for evaluating them at this time and therefore the assumption must be made that impact forces create the same stresses as static forces of the same magnitude. It appears that this is a safe practice and one which may introduce some factor of safety that at present is unknown.

Mathematical analysis of stress.—In 1919 Goldbeck (*20*) suggested approximate formulas for computing the stresses in concrete pavement slabs under certain assumed conditions of loading and subgrade support. Among these approximate formulas is one which has since become generally known as the "corner formula". This be expressed in the form

$$\sigma_c=\frac{3P}{h^2} \quad (1)$$

where σ_c = maximum tensile stress, in pounds per square inch, in a diagonal direction in the top of the slab near a rectangular corner;

P = load, in pounds, applied at a point at the corner;

h = depth of slab in inches.

This simple formula is derived on the assumption that the load is applied at a point at the extreme corner of the slab; that the corner receives no support from the subgrade and acts as a simple cantilever; and that the fiber stresses in the slab are uniform on any section at right angles to a line bisecting the corner angle.

Some years later, in the analysis of the data from the Bates Road tests (*21*), it was found that there was a reasonably good agreement between the wheel loads that caused corner failure and loads computed by the corner formula. However, it is now quite definitely known that the corner formula gives stresses considerably higher than the actual stresses in pavement slabs, even under extreme conditions of warping. The agreement between computed loads and measured loads in the Bates Road report may be explained by the fact that the latter were static wheel loads while the loads that actually caused corner failures were the impact reactions due to these wheel loads. In view of the fact that the truck wheels were equipped with solid rubber tires, the impact loads were undoubtedly considerably higher than the static wheel loads.

In 1925 the analysis by Westergaard (*2*) made available for the first time a logical and scientific basis for evaluating the stresses in concrete pavements. This analysis concerns itself with the determination of maximum stresses in slabs of uniform thickness resulting from the following three conditions of loading:

1. Load applied close to the rectangular corner of a large slab.
2. Load applied in the interior of a large slab at a considerable distance from the edges.
3. Load applied at the edge of the slab at a considerable distance from any corner.

WESTERGAARD EQUATIONS GIVEN

The anlysis involves the following important assumptions:

1. That the concrete slab acts as a homogeneous, isotropic, elastic solid in equilibrium.
2. That the reactions of the subgrade are vertical only and that they are proportional to the deflections of the slab.
3. That the reaction of the subgrade per unit of area at any given point is equal to a constant, k, multiplied by the deflection at that point. The constant, k, is termed the "modulus of subgrade reaction" or "subgrade modulus" and is assumed to be constant at each point, independent of the deflections, and to be the same at all points within the area under consideration.
4. That the thickness of the slab is uniform.
5. That the load at the interior and at the corner of the slab are distributed uniformly over a circular area of contact. For the corner loading, the circumference of this circular area is tangent to the edges of the slab.
6. That the load at the edge of the slab is distributed uniformly over a semicircular area of contact, the center of the circle being on the edge of the slab.

For the three positions of load, the analysis results in equations which may be expressed as follows:

$$\sigma_c=\frac{3P}{h^2}\left[1-\left(\frac{12(1-\mu^2)k}{Eh^3}\right)^{0.15}(a\sqrt{2})^{0.6}\right] \quad (2)$$

$$\sigma_i=0.275(1+\mu)\frac{P}{h^2}\log_{10}\left(\frac{Eh^3}{kb^4}\right) \quad (3)$$

$$\sigma_e=0.529(1+0.54\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{Eh^3}{kb^4}\right)-0.71\right] \quad (4)$$

in which

P = load, in pounds;

σ_c = maximum tensile stress in pounds per square inch at the top of the slab, in a direction parallel to the bisector of the corner angle, due to a load P at the corner;

σ_i = maximum tensile stress in pounds per square inch at the bottom of the slab directly under the load P, when P is at a point in the interior of the slab at a considerable distance from the edges;

σ_e = maximum tensile stress in pounds per square inch at the bottom of the slab directly under the load P at the edge, and in a direction parallel to the edge;

h = thickness of the concrete slab, in inches;

μ = Poisson's ratio for concrete;

E = modulus of elasticity of the concrete, in pounds per square inch;

k = subgrade modulus, in pounds per cubic inch;

a = radius of area of load contact, in inches. The area is circular in the case of corner and interior loads and semicircular for edge loads;

b = radius of equivalent distribution of pressure

[3] The term "Arlington tests" will be used to designate the investigation of concrete pavement design made by the Bureau of Public Roads at the Arlington Experiment Farm and described in reports listed in the bibliography (*15, 16, 17, 18, 19*).

$$b=\sqrt{1.6a^2+h^2}-0.675h \text{ when } a<1.724h \quad (5)$$
$$b=a \text{ when } a>1.724h$$

Values of b for various values of a and h are given in table 2.

Value of Poisson's ratio.—If an isotropic, elastic material is subjected to stress in one direction a unit deformation is produced in the direction of the force and, in addition, a smaller deformation is produced in the direction perpendicular to the force. The relation between these two deformations, expressed as the ratio of the smaller to the larger, is known as Poisson's ratio. It appears in the Westergaard equations and therefore a value must be assigned to it.

The results of several investigations to determine the magnitude of Poisson's ratio are available (*22*, *23*, *24*). The general conclusion from these investigations is that there is no definite relationship between the strength of concrete and Poisson's ratio. With respect to other variables, such as age, the trends are not very definite and the conclusions reached by different investigators are not always in agreement. It is apparent that Poisson's ratio for a given concrete cannot be foretold and that for purposes of design it is necessary to select some reasonable and safe value.

TABLE 2.—*Values of b for various values of a and h, computed by equation 5*

Ratio a/h	Values of b in inches for different values of h in inches								
	$h=4$	$h=5$	$h=6$	$h=7$	$h=8$	$h=9$	$h=10$	$h=11$	$h=12$
	Inches	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*
0	1.30	1.63	1.95	2.28	2.60	2.93	3.25	3.58	3.90
.1	1.33	1.66	2.00	2.33	2.66	3.00	3.33	3.66	4.00
.2	1.43	1.78	2.14	2.50	2.85	3.21	3.57	3.92	4.28
.3	1.58	1.97	2.37	2.76	3.16	3.55	3.95	4.34	4.73
.4	1.78	2.23	2.67	3.12	3.57	4.01	4.46	4.90	5.35
.5	2.03	2.54	3.05	3.56	4.07	4.57	5.08	5.59	6.10
.6	2.32	2.90	3.48	4.06	4.64	5.22	5.80	6.38	6.96
.7	2.64	3.30	3.96	4.62	5.29	5.95	6.61	7.27	7.93
.8	2.99	3.74	4.49	5.23	5.98	6.73	7 48	8.22	8.97
.9	3.36	4.20	5.04	5.88	6.72	7.56	8.40	9.24	10.08
1.0	3.75	4.69	5.62	6.56	7.50	8.44	9.37	10.31	11.25
1.1	4.15	5.19	6.23	7.27	8.31	9.35	10.38	11.42	12.46
1.2	4.57	5.71	6.86	8.00	9 14	10.28	11.43	12.57	13.71
1.3	5.00	6.25	7.50	8.75	10.00	11.25	12.50	13.75	14.99
1.4	5.43	6.79	8.15	9.51	10.87	12.23	13.59	14.95	16.30
1.5	5.88	7.35	8.82	10.29	11.76	13 23	14.70	16.17	17.64
1.6	6.33	7.91	9.49	11.08	12.66	14.24	15.82	17.41	18.99
1.7	6.79	8.48	10.18	11.88	13.57	15.27	16.97	18.66	20.36
1.724 [1]	6.90	8.62	10.34	12.07	13.79	15.52	17.24	18.96	20.69

[1] When a/h is greater than 1.724, $b=a$

The digest by Richart and Roy (*22*) shows values of Poisson's ratio, obtained by several investigators and involving a number of variables, ranging from 0.08 to 0.28. Koenitzer (*24*) reports about 250 values for a range of conditions, of which the minimum is 0.08, the maximum is 0.40, and the average is 0.18. Approximately 20 percent of the values reported by Koenitzer do not exceed 0.15, 78 percent do not exceed 0.20 and 95 percent do not exceed 0.25.

If it be assumed, on the basis of these data, that a range of Poisson's ratio to be reasonably expected is from 0.10 to 0.20 and an average figure of 0.15 is assumed for design purposes, then the maximum error in computed stresses within this range will be plus or minus 4.3 percent for interior stresses and plus or minus 2.5 percent for edge stresses. The effect of Poisson's ratio on corner stresses is negligible. Even if Poisson's ratio happens to have the rather high value of 0.25 the error involved in assuming it equal to 0.15 will be only 8.7 percent for interior stresses and 5 percent for edge stresses, the effect on corner stresses still being negligible. It appears, therefore, that the general practice, first suggested by Westergaard, of assuming for the purpose of pavement design that Poisson's ratio is equal to 0.15, is an entirely reasonable one, and that value will be used hereafter in this paper.

In addition to the quantities that appear directly in the three stress equations, there is the radius of relative stiffness, l, which is defined by the equation

$$l=\sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}} \quad (6)$$

Values of l for various values of E, h, and k are given in table 3.

Westergaard has expressed equation 2 in terms of l, as follows:

Corner loading

$$\sigma_c=\frac{3P}{h^2}\left[1-\left(\frac{a\sqrt{2}}{l}\right)^{0.6}\right] \quad (7)$$

and Bradbury (*9*) has shown that, when $\mu=0.15$, equations 3 and 4 may be expressed in the form:

Interior loading

$$\sigma_i=0.31625\frac{P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right)+1.0693\right] \quad (8)$$

Edge loading

$$\sigma_e=0.57185\frac{P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right)+0.3593\right] \quad (9)$$

NEW FORMULA FOR CORNER STRESSES IN AGREEMENT WITH TEST RESULTS

Modified equations for corner loading.—If, in equation 2, for corner loading, the radius of contact area, a, is assumed equal to zero then the influence of the subgrade modulus, k, and the modulus of elasticity, E, are eliminated and the equation reduces to the corner formula

$$\sigma_c=\frac{3P}{h^2} \quad (1)$$

TABLE 3.—*Radius of relative stiffness, l, computed by equation 6* $\mu=0.15$

Modulus of elasticity of concrete E	Subgrade modulus k	Radius of relative stiffness, l, in inches for different values of h, in inches								
		$h=4$	$h=5$	$h=6$	$h=7$	$h=8$	$h=9$	$h=10$	$h=11$	$h=12$
Lb. per sq. in.	*Lb. per cu. in.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*
3,000,000	50	23.9	28.3	32.4	36.4	40.2	43.9	47.6	51.1	54.5
	100	20.1	23.8	27.3	30.6	33.8	37.0	40.0	43.0	45.9
	150	18.2	21.5	24.6	27.7	30.6	33.4	36.1	38.8	41.4
	200	16.9	20.0	22.9	25.7	28.4	31.1	33.6	36.1	38.6
	300	15.3	18.1	20.7	23.3	25.7	28.1	30.4	32.6	34.8
	400	14.2	16.8	19.3	21.6	23.9	26 1	28.3	30.4	32.4
4,000,000	50	25.7	30.4	34.8	39.1	43.2	47.2	51.1	54.9	58.6
	100	21.6	25.6	29.3	32.9	36.4	39.7	43.0	46.2	49.3
	150	19.5	23.1	26.5	29.7	32.8	35.9	38.8	41 7	44.5
	200	18.2	21 5	24.6	27.7	30.6	33.4	36.1	38.8	41.4
	300	16.4	19.4	22.3	25.0	27.6	30.2	32.7	35.1	37.4
	400	15.3	18.1	20.7	23.3	25.7	28.1	30.4	32.6	34.8
5,000,000	50	27.2	32.1	36.8	41.4	45.7	49.9	54.0	58.0	62.0
	100	22.9	27.0	31.0	34.8	38 4	42.0	45.4	48.8	52.1
	150	20.7	24.4	28.0	31.4	34.7	37.9	41.1	44.1	47.1
	200	19.2	22.7	26.0	29.2	32.3	35.3	38.2	41.0	43.8
	300	17.4	20.5	23.5	26.4	29.2	31.9	34.5	37.1	39.6
	400	16.2	19.1	21.9	24.6	27.2	29.7	32.1	34.5	36.8
6,000,000	50	28.4	33.6	38.6	43.3	47.8	52.3	56.6	60.7	64.8
	100	23.9	28.3	32.4	36.4	40.2	43.9	47.6	51.1	54.5
	150	21.6	25.6	29.3	32.9	36.4	39.7	43.0	46.2	49.3
	200	20.1	23.8	27.3	30.6	33.8	37.0	40.0	43.0	45.9
	300	18.2	21.5	24.6	27.7	30.6	33.4	36.1	38.8	41.4
	400	16.9	20.0	22.9	25 7	28.4	31.1	33.6	36.1	38.6

The derivation of the corner formula (equation 1), involves two assumptions, of which one is manifestly incorrect and the other is very questionable. When the radius of contact area is zero the load is assumed to be concentrated at a point at the extreme corner of the slab. This is an impossible condition since a rubber-tired wheel distributes its load over an area of contact of appreciable size. The second assumption is, in effect, that when a load is applied to the corner of a slab which is warped upward the effect of subgrade support is completely eliminated. The combination of these two assumptions results in computed stresses that are much higher than have been observed in carefully conducted tests.

When the corner of the slab is warped upward there may be a complete lack of subgrade support immediately beneath the corner and to this extent the original Westergaard analysis (equations 2 or 7), which involves the assumption of uniform subgrade support, is incorrect. Westergaard has recognized this and has suggested a modification of the analysis which takes account of this condition (*25*). This modification involves assumptions as to the reduction in subgrade support which cannot be readily evaluated at the present time. However, it does recognize the fact, which is corroborated by test data, that while there may be no contact between slab and subgrade immediately beneath a corner load, nevertheless the subgrade support in the vicinity of the corner is effective in reducing the maximum stress by a considerable percentage below that computed by the corner formula.

In a somewhat limited but carefully conducted series of tests on large slabs under laboratory conditions, Spangler and Lightburn (*26*, *27*) observed corner stresses appreciably greater than those computed by the Westergaard equation.

As a result of these observations Bradbury (*9*) has suggested the modified equation

$$\sigma_c=\frac{3P}{h^2}\left[1-\left(\frac{a}{l}\right)^{0.6}\right] \qquad (10)$$

In effect this equation represents the assumption that the subgrade modulus in the vicinity of the corner is only one-fourth of the modulus that is effective under the other portions of the slab.

In the Arlington tests (*19*), in which the slabs were exposed to normal weather conditions, it has been found that in the daytime, when the corner is warped downward and has contact with the subgrade, there is very good agreement between observed stresses and those computed by the Westergaard formula (equation 7). However, at night, when the corner is warped upward, the observed stresses, while lower than those given by the corner formula, are much higher than those computed either by the Westergaard equation or by Bradbury's formula (equation 10).

Westergaard has shown that for the conditions assumed in his analysis the maximum corner stress occurs at a distance from the corner, measured along the diagonal bisector of the corner angle, equal to X_1 where

$$X_1=2\sqrt[4]{2}\,\sqrt{al}$$

In the Arlington tests it has been found that when the slab is warped upward the maximum stress occurs at a distance from the corner several inches greater than the computed value of X_1. It has also been found that observed stresses are in good agreement with stresses computed by the equation

$$\sigma_c=\frac{3P}{h^2}\left[1-\left(\frac{a\sqrt{2}}{l}\right)^{1.2}\right] \qquad (11)$$

It will be observed that this equation has the same general form as the Westergaard formula (equation 7) and Bradbury's formula (equation 10). However, it is purely empirical and has no theoretical background. Its only virtue is its algebraic simplicity and the fact that it gives results that are in reasonably good agreement with a considerable number of tests on pavement slabs exposed to normal fluctuations of temperature and moisture. Its use is suggested pending the time when more exact information may be available.

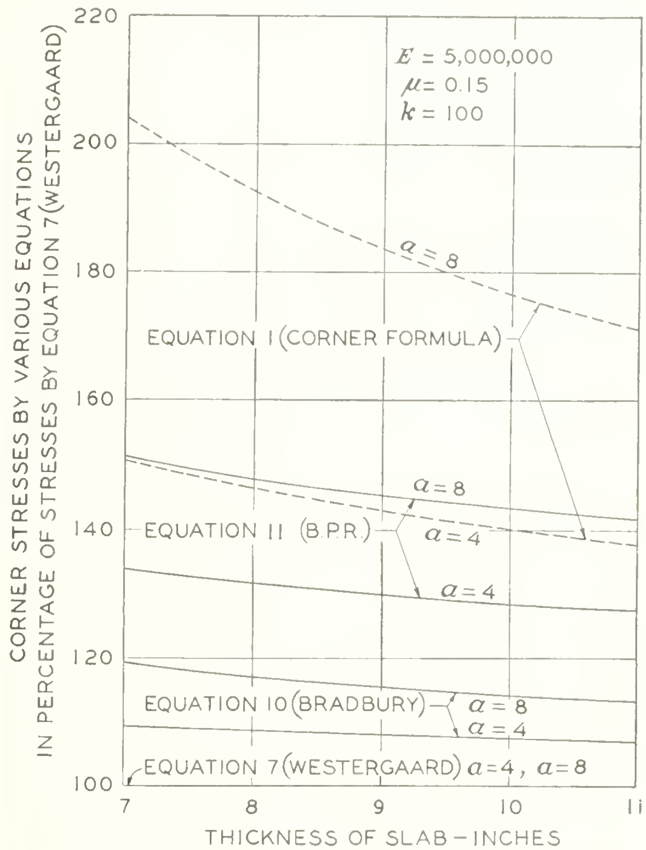

FIGURE 1.—COMPARISON OF CORNER STRESSES COMPUTED BY VARIOUS EQUATIONS.

A comparison of the results given by equations 1, 7, 10, and 11 is shown in figure 1. For the range of conditions assumed, the corner stresses computed by Westergaard's formula (equation 7) are exceeded by those computed by Bradbury's formula (equation 10) by 7 to 20 percent, by those computed by equation 11 by 27 to 51 percent, and by those computed by the corner formula, equation 1, by 38 to 104 percent.

MODIFIED EQUATIONS FOR INTERIOR AND EDGE LOADING GIVEN

Modified equations for interior loading.—Early in the Arlington tests it was found that the observed stresses due to loads in the interior of the slab were not as great as those computed by equation 3 and as a result Westergaard modified his original analysis (*28*). The modified equation for stress due to interior loading is

$$\sigma_i=0.275\,(1+\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{Eh^3}{kb^4}\right)-54.54\left(\frac{l}{L}\right)^2 Z\right] \qquad (12)$$

in which

L=maximum value of the radius of the circular area, with center at the point of load application, within which a redistribution of subgrade reactions is made;

Z = ratio of reduction of the maximum deflection.

Westergaard has stated that, under actual conditions, Z may be expected to vary between 0 and 0.39. When $Z=0$, equation 12 reduces to equation 3. He has also suggested as a reasonable assumption that $L=5l$. It is immediately apparent that the values assigned to Z and L and the relation of these values to each other have a major effect on the computed stresses. Moreover, reasonably exact values can be developed only from the data obtained in tests of large slabs.

As an approximation Bradbury (*9*) has suggested that an average value of $Z=0.20$ be assumed and this, and the further assumption that $L=5l$ and $\mu=0.15$, leads to the equation:

$$\sigma_i=0.31625\frac{P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right)+0.6330\right] \quad (13)$$

For the conditions which obtained in the Arlington tests, values of $L=1.75l$ and $Z=0.05$ were quite well established and these values, with $\mu=0.15$, lead to the equation

$$\sigma_i=0.31625\frac{P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right)+0.1788\right] \quad (14)$$

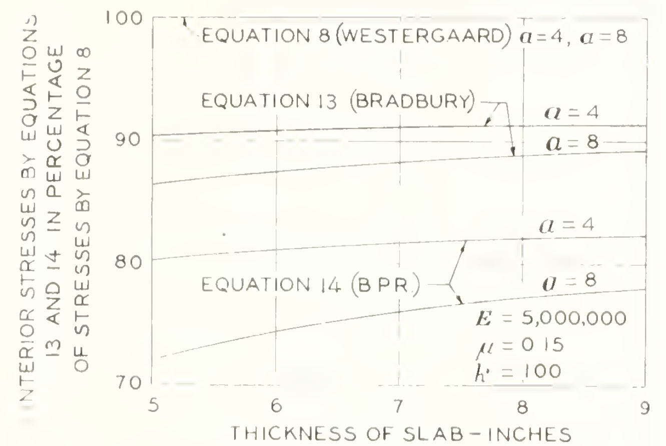

FIGURE 2.—COMPARISON OF INTERIOR STRESSES COMPUTED BY VARIOUS EQUATIONS.

A comparison of the results given by equations 8, 13, and 14, is shown in figure 2. For the range of conditions assumed, the interior stresses computed by equation 14 are from 72 to 82 percent, and those computed by equation 13 are from 86 to 91 percent, of those computed by Westergaard's original formula (equation 8).

The reduction of interior stresses, as expressed by equation 12, is dependent on the characteristics of the subgrade and the slab and the complex reaction between them. Equation 14 is representative of what may be expected under the conditions obtaining in the Arlington tests but these were concerned with only one type of subgrade and one class of concrete. In view of this it is believed that equation 14, with its rather large stress reductions, is not suitable for general use as representative of average conditions. In the light of present knowledge it will be conservative, and not uneconomical, to continue to use the results given by the original Westergaard analysis, equation 8.

Modified equation for edge loading.—In the Arlington tests it has been found that for what may be considered

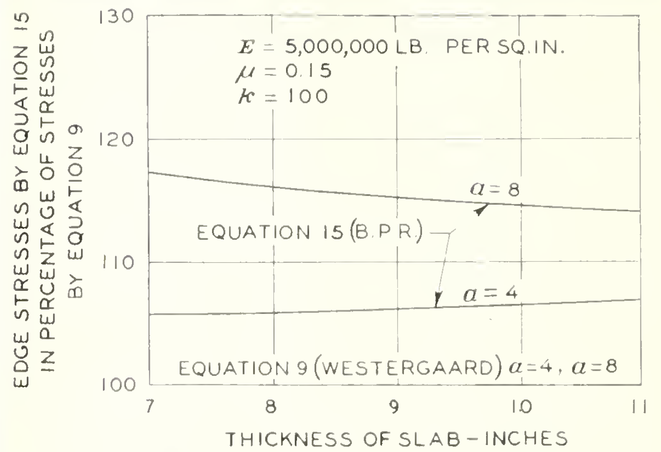

FIGURE 3.—COMPARISON OF EDGE STRESSES COMPUTED BY EQUATIONS 9 AND 15.

as average values of a, the radius of contact area, there is good agreement between observed edge stresses and those computed by Westergaard's formula (equation 9) when the slab is in an unwarped condition. For smaller values of a the observed stresses are somewhat less than the theoretical stresses and for larger values of a the observed stresses are somewhat greater than the theoretical stresses. However, the differences are not great and no serious errors will result from the use of equation 9 for the computation of edge stresses in a slab which is not warped. The same equation is also applicable when the edges of the slab are warped downward during the daytime, although in this case the computed stresses may generally be expected to be slightly less than the actual stresses.

When the edges of the slab are warped upward at night the observed load stresses exceed the theoretical stresses, as in the case of corner loading although not to the same extent. It has been found that the observed stresses under the conditions of nighttime warping are in reasonably good agreement with the empirical equation

$$\sigma_e=0.57185\frac{P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right)+\log b\right] \quad (15)$$

A comparison of the results given by equations 9 and 15 is shown in figure 3. For the range of conditions assumed, the edge stresses computed by equation 15 exceed those computed by equation 9 by 6 to 17 percent.

SIMPLIFIED METHOD OF COMPUTING STRESSES PRESENTED

Simplification of Stress Computations.—The equations of Westergaard and the modified equations that have been discussed are simple algebraic expressions but their solution requires a considerable amount of tedious labor. However, Bradbury (*9*) has suggested a simplified method of computation which reduces the determination of stress by means of these equations to a simple slide-rule operation.

He has pointed out that all the equations have the general form,

$$\sigma=\frac{CP}{h^2} \quad (16)$$

in which C is a quantity that may be termed a stress coefficient. The coefficients C_i and C_e, for interior and edge stresses, respectively, are fixed by the ratio l/b; while the coefficient C_c for corner stresses is fixed by the ratio a/l.

Values of stress coefficients are given in tables 4, 5, 6, and 7. Table 4 gives the coefficients for corner loading by the Westergaard equation 7. Table 5 gives coefficients for corner loading by the modified equation 11. Table 6 gives coefficients for interior loading by equation 8. Coefficients for interior loading by equation 13 may be obtained by subtracting 0.138, and those corresponding to equation 14 by subtracting 0.282, from the values given in table 6. Table 7 gives the coefficients for edge loading by equation 9. Table 8 gives a correction factor to be added algebraically to the coefficients of table 7 to obtain the stress coefficients corresponding to equation 15.

TABLE 4.—*Stress coefficients, C_c, for corner loading, computed by equation 7 (Westergaard), $\mu=0.15$*

Ratio a/l	C_c	Ratio a/l	C_c	Ratio a/l	C_c
0	3.000	0.20	1.594	0.40	0.869
0.01	2.767	0.21	1.552	0.41	.837
0.02	2.647	0.22	1.511	0.42	.805
0.03	2.549	0.23	1.171	0.43	.774
0.04	2.465	0.24	1.431	0.44	.743
0.05	2.388	0.25	1.392	0.45	.713
0.06	2.317	0.26	1.354	0.46	.682
0.07	2.251	0.27	1.316	0.47	.652
0.08	2.189	0.28	1.279	0.48	.622
0.09	2.129	0.29	1.243	0.49	.593
0.10	2.072	0.30	1.206	0.50	.563
0.11	2.018	0.31	1.171	0.51	.534
0.12	1.965	0.32	1.136	0.52	.505
0.13	1.914	0.33	1.101	0.53	.477
0.14	1.865	0.34	1.067	0.54	.448
0.15	1.817	0.35	1.033	0.55	.420
0.16	1.770	0.36	.999	0.56	.392
0.17	1.724	0.37	.966	0.57	.364
0.18	1.680	0.38	.933	0.58	.336
0.19	1.636	0.39	.901	0.59	.309
0.20	1.591	0.40	.869	0.60	.282

TABLE 5.—*Stress coefficients, C_c, for corner loading, computed by equation 11 (Bureau of Public Roads) $\mu=0.15$*

Ratio a/l	C_c	Ratio a/l	C_c	Ratio a/l	C_c
0	3.000	0.20	2.341	0.40	1.486
0.01	2.982	0.21	2.301	0.41	1.440
0.02	2.958	0.22	2.261	0.42	1.394
0.03	2.932	0.23	2.221	0.43	1.348
0.04	2.904	0.24	2.180	0.44	1.302
0.05	2.875	0.25	2.138	0.45	1.256
0.06	2.845	0.26	2.097	0.46	1.209
0.07	2.813	0.27	2.055	0.47	1.162
0.08	2.780	0.28	2.013	0.18	1.115
0.09	2.747	0.29	1.971	0.49	1.068
0.10	2.713	0.30	1.928	0.50	1.021
0.11	2.678	0.31	1.885	0.51	.973
0.12	2.643	0.32	1.841	0.52	.925
0.13	2.607	0.33	1.798	0.53	.877
0.14	2.570	0.34	1.754	0.51	.829
0.15	2.533	0.35	1.710	0.55	.781
0.16	2.496	0.36	1.666	0.56	.732
0.17	2.458	0.37	1.621	0.57	.684
0.18	2.419	0.38	1.576	0.58	.635
0.19	2.380	0.39	1.531	0.59	.586
0.20	2.341	0.40	1.486	0.60	.537

The procedure to be followed in using these tables is very simple. By means of the ratio a/h, b is determined, by interpolation if necessary, from table 2, and l is obtained from table 3. Then the ratios a/l and l/b are computed. Using the ratio a/l, the coefficient C_c is obtained from table 4 or table 5. Using the ratio l/b, the coefficient C_i is obtained from table 6 and the coefficient C_e from table 7. To obtain the stress coefficient, C''_e, corresponding to equation 15, the correction factor K_e corresponding to the value of a/h is obtained from table 8 and is added algebraically to the value of C_e obtained from table 7.

Effect of variables on computed stresses.—For a specific pavement design to be used in a specific location it is not possible at present to predetermine, with any degree of precision, the values to be assigned to several of the variables which appear in the stress equations. Therefore it is necessary, both when the design is for a particular project and when it is a general design to be used on a number of projects, to assign reasonable and rather conservative values to these variables. In order to do this it is necessary to have some knowledge of their relative effects on computed stressses.

It is apparent from the equations that the computed stress varies directly with the magnitude of the wheel load. The effect of variations in Poisson's ratio has already been discussed.

TABLE 6.—*Stress coefficients, C_i, for interior loading,[1] computed by equation 8, $\mu=0.15$*

Ratio l/b	C_i	Ratio l/b	C_i	Ratio l/b	C_i
1.0	0.338	6.0	1.323	11.0	1.656
1.1	.391	6.1	1.332	11.1	1.660
1.2	.438	6.2	1.341	11.2	1.665
1.3	.482	6.3	1.349	11.3	1.670
1.4	.523	6.4	1.358	11.4	1.675
1.5	.561	6.5	1.367	11.5	1.680
1.6	.596	6.6	1.375	11.6	1.685
1.7	.630	6.7	1.383	11.7	1.689
1.8	.661	6.8	1.391	11.8	1.694
1.9	.691	6.9	1.399	11.9	1.699
2.0	.719	7.0	1.407	12.0	1.703
2.1	.716	7.1	1.415	12.1	1.708
2.2	.771	7.2	1.423	12.2	1.712
2.3	.796	7.3	1.430	12.3	1.717
2.4	.819	7.4	1.438	12.1	1.721
2.5	.842	7.5	1.445	12.5	1.726
2.6	.863	7.6	1.452	12.6	1.730
2.7	.884	7.7	1.460	12.7	1.734
2.8	.904	7.8	1.467	12.8	1.739
2.9	.923	7.9	1.474	12.9	1.743
3.0	.942	8.0	1.481	13.0	1.747
3.1	.960	8.1	1.487	13.1	1.752
3.2	.977	8.2	1.494	13.2	1.756
3.3	.994	8.3	1.501	13.3	1.760
3.4	1.010	8.4	1.507	13.4	1.764
3.5	1.026	8.5	1.514	13.5	1.768
3.6	1.042	8.6	1.520	13.6	1.772
3.7	1.057	8.7	1.527	13.7	1.776
3.8	1.072	8.8	1.533	13.8	1.780
3.9	1.086	8.9	1.539	13.9	1.784
1.0	1.100	9.0	1.545	14.0	1.788
4.1	1.113	9.1	1.551	14.1	1.792
4.2	1.127	9.2	1.557	14.2	1.796
4.3	1.140	9.3	1.563	14.3	1.800
4.4	1.152	9.4	1.569	14.4	1.803
4.5	1.164	9.5	1.575	14.5	1.807
4.6	1.177	9.6	1.581	14.6	1.811
4.7	1.188	9.7	1.586	14.7	1.815
1.8	1.200	9.8	1.592	14.8	1.819
4.9	1.211	9.9	1.598	14.9	1.822
5.0	1.222	10.0	1.603		
5.1	1.233	10.1	1.609		
5.2	1.244	10.2	1.614		
5.3	1.254	10.3	1.619		
5.4	1.265	10.4	1.625		
5.5	1.275	10.5	1.630		
5.6	1.285	10.6	1.635		
5.7	1.294	10.7	1.640		
5.8	1.304	10.8	1.645		
5.9	1.313	10.9	1.651		
6.0	1.323	11.0	1.656		

[1] For values of C_i corresponding to equation 13, subtract 0.138 from the values given in this table.
For values of C_i corresponding to equation 14, subtract 0.282 from the values given in this table.

CONSERVATIVE VALUE OF SUBGRADE MODULUS RECOMMENDED

Effect of variations in subgrade modulus, k.—It has been stated repeatedly in the literature that variations in the modulus of subgrade reaction have a minor effect on the computed stresses. The accuracy of this statement appears to depend on the range of conditions that are under consideration and the degree of error in computed stresses that can be tolerated.

Figure 4 shows the effect of variations in subgrade modulus between 50 and 300 pounds per cubic inch on stresses computed for interior, corner, and edge loadings for a reasonable range in values of a, the radius of

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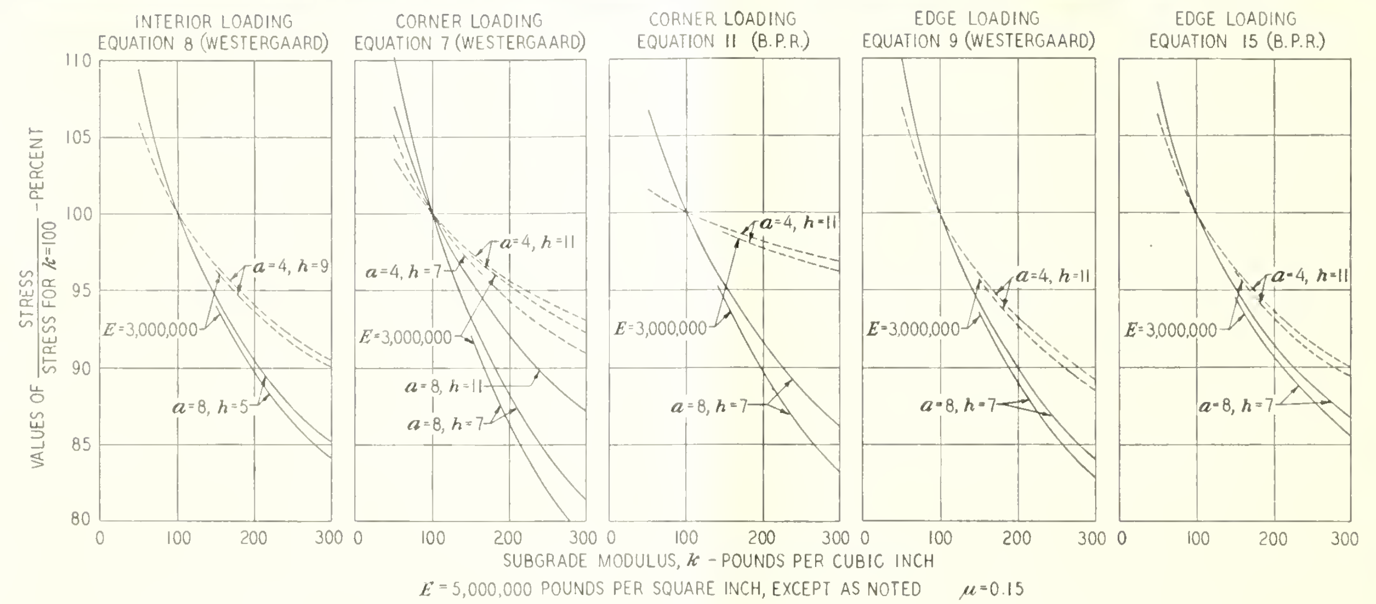

FIGURE 4.—EFFECT ON COMPUTED STRESSES OF VARIATIONS IN SUBGRADE MODULUS, k.

TABLE 7.—*Stress coefficients, C_e, for edge loading, computed by equation 9, $\mu=0.15$*

Ratio l/b	C_e	Ratio l/b	C_e	Ratio l/b	C_e
1.0	0.205	6.0	1.985	11.0	2.588
1.1	.300	6.1	2.002	11.1	2.597
1.2	.387	6.2	2.018	11.2	2.605
1.3	.466	6.3	2.034	11.3	2.614
1.4	.540	6.4	2.050	11.4	2.623
1.5	.608	6.5	2.065	11.5	2.632
1.6	.672	6.6	2.080	11.6	2.640
1.7	.733	6.7	2.095	11.7	2.649
1.8	.789	6.8	2.110	11.8	2.657
1.9	.843	6.9	2.124	11.9	2.666
2.0	.894	7.0	2.139	12.0	2.674
2.1	.943	7.1	2.153	12.1	2.682
2.2	.989	7.2	2.167	12.2	2.690
2.3	1.033	7.3	2.180	12.3	2.699
2.4	1.075	7.4	2.194	12.4	2.707
2.5	1.116	7.5	2.207	12.5	2.715
2.6	1.155	7.6	2.220	12.6	2.722
2.7	1.192	7.7	2.233	12.7	2.730
2.8	1.228	7.8	2.246	12.8	2.738
2.9	1.263	7.9	2.259	12.9	2.746
3.0	1.297	8.0	2.271	13.0	2.753
3.1	1.329	8.1	2.284	13.1	2.761
3.2	1.361	8.2	2.296	13.2	2.769
3.3	1.392	8.3	2.308	13.3	2.776
3.4	1.421	8.4	2.320	13.4	2.784
3.5	1.450	8.5	2.331	13.5	2.791
3.6	1.478	8.6	2.343	13.6	2.798
3.7	1.505	8.7	2.355	13.7	2.806
3.8	1.532	8.8	2.366	13.8	2.813
3.9	1.557	8.9	2.377	13.9	2.820
4.0	1.583	9.0	2.388	14.0	2.827
4.1	1.607	9.1	2.399	14.1	2.834
4.2	1.631	9.2	2.410	14.2	2.841
4.3	1.654	9.3	2.421	14.3	2.848
4.4	1.677	9.4	2.431	14.4	2.855
4.5	1.700	9.5	2.442	14.5	2.862
4.6	1.721	9.6	2.452	14.6	2.869
4.7	1.743	9.7	2.463	14.7	2.876
4.8	1.764	9.8	2.473	14.8	2.882
4.9	1.784	9.9	2.483	14.9	2.889
5.0	1.804	10.0	2.493		
5.1	1.824	10.1	2.503		
5.2	1.843	10.2	2.513		
5.3	1.862	10.3	2.522		
5.4	1.881	10.4	2.532		
5.5	1.899	10.5	2.541		
5.6	1.917	10.6	2.551		
5.7	1.934	10.7	2.560		
5.8	1.952	10.8	2.569		
5.9	1.969	10.9	2.578		
6.0	1.985	11.0	2.588		

contact area, and h, the depth of the slab. All stresses are expressed as percentages of the stresses computed for $k=100$. The curves that are continuous from $k=50$ to $k=300$ are for stresses computed with the modulus of elasticity, E, equal to 5,000,000 pounds per square inch.

TABLE 8.—*Values of correction factor,[1] K.*

Ratio a/h	Values of K_e for different values of h in inches								
	$h=4$	$h=5$	$h=6$	$h=7$	$h=8$	$h=9$	$h=10$	$h=11$	$h=12$
0	−0.140	−0.085	−0.040	−0.001	0.032	0.062	0.087	0.111	0.133
0.1	−.134	−.079	−.034	.005	.038	.067	.093	.117	.139
0.2	−.117	−.062	−.017	.022	.055	.084	.110	.134	.156
0.3	−.092	−.037	.009	.047	.080	.109	.135	.159	.181
0.4	−.062	−.006	.039	.077	.110	.140	.166	.189	.211
0.5	−.029	.026	.071	.110	.143	.172	.198	.222	.244
0.6	.004	.059	.104	.143	.176	.205	.231	.255	.277
0.7	.036	.091	.137	.175	.208	.237	.263	.287	.309
0.8	.067	.122	.167	.206	.239	.268	.294	.318	.339
0.9	.096	.151	.196	.235	.268	.297	.323	.347	.368
1.0	.123	.178	.223	.262	.295	.324	.350	.374	.396
1.1	.148	.204	.249	.287	.320	.350	.376	.399	.421
1.2	.172	.227	.273	.311	.344	.373	.400	.423	.445
1.3	.194	.250	.295	.333	.366	.396	.422	.445	.467
1.4	.215	.270	.316	.354	.387	.416	.443	.466	.488
1.5	.234	.290	.335	.373	.407	.436	.462	.486	.507
1.6	.253	.308	.353	.392	.425	.454	.480	.504	.526
1.7	.270	.326	.371	.409	.442	.471	.498	.521	.543
1.724 [2]	.274	.330	.375	.413	.446	.475	.502	.525	.547

[1] To be added algebraically to the edge coefficient, C_e, obtained from table 7, to obtain the edge coefficient, C_e', corresponding to equation 15_{10}.
[2] When a/h is greater than 1.724, $b=a$ and $K_e=0.57185$ (log 10 $a-0.3593$).

The curves that are only partially complete are for stresses based on a value of E equal to 3,000,000 pounds per square inch. The upper portions of these curves are omitted since they so nearly coincide with the upper portions of the curves for $E=5,000,000$ that their inclusion would detract from the clarity of the charts.

It is evident from these curves that the value of E has no significant influence on the relation between subgrade modulus and stress when, as in this case, stresses are expressed as percentages of a basic stress which is different for each curve. Therefore, the subsequent discussion of the effect on stress of variations in the subgrade modulus will be confined to the curves for $E=5,000,000$.

It will be observed in the second chart from the left in figure 4 that the two curves, one for the minimum value of a in combination with the maximum value of h, and the other for the maximum value of a in combination with the minimum value of h, form an envelope for the curves for all intermediate values of a and h. In order to clarify the presentation, only these envelope curves are shown in the other charts of this and succeeding figures of similar character.

Before discussing figure 4 it will be well to examine the available data regarding observed values of the subgrade modulus. Unfortunately, these data are very meager. It is not known if a value of 50 pounds per cubic inch is the minimum that may be expected but there is reason to believe that the maximum may exceed 300 pounds per cubic inch, at least in some cases. Therefore the range that may be encountered in practice is not known.

In corner-loading tests and working with what may be termed synthetic subgrades, that is, earth subgrades consolidated in the laboratory by tamping, Spangler (*26*) observed in one very stiff clay subgrade (probably very dry) a subgrade modulus of the order of 1,000 pounds per cubic inch. In another test, with a subgrade of more normal characteristics, he observed that the apparent subgrade modulus was reduced by repeated corner loading from about 275 to about 40 pounds per cubic inch.

In still another corner-loading test Spangler and Lightburn (*27*) found that the subgrade modulus was constant at a given point in the slab but varied with the distance of the point from the corner, being about 300 pounds per cubic inch at the corner and about 75 pounds per cubic inch at distances of 4.5 feet from the corner. They concluded, however, that the assumption of a uniform value of the subgrade modulus appears to be justifiable for analytical solutions since stresses computed with a modulus equal to about the average of the two extreme values were in good agreement with observed stresses.

In considering the values of subgrade modulus obtained in the tests by Spangler and Lightburn it is well to remember that the subgrades with which they worked were protected from the weather and were not exposed to natural fluctuations of moisture.

In the Arlington tests the pavement slabs were exposed to the weather but it is necessary to bear in mind that only one subgrade was involved. In these tests the values of the subgrade modulus observed under normal conditions of subgrade support varied from about 170 to about 280 pounds per cubic inch.

These meager data indicate that the subgrade modulus may vary over a rather wide range, the limits of which are unknown; that its value may be affected by repeated loading of the slab; and that, at the same location, it is likely to be different at different times. The development of additional data is hampered by the present lack of any simple method of making the required tests over the wide range of conditions that merit study. The situation makes it highly desirable to be conservative in the selection of values of the modulus for use in stress computations.

Examination of figure 4 shows that variations in subgrade modulus have little effect on stresses computed by the modified equation for corner loading, equation 11, for small values of a and large values of h. The effect of variations in the modulus on interior, corner, and edge stresses computed by the Westergaard equations, on edge stresses computed by equation 15, and on corner stresses by equation 11 for large values of a and small values of h, is very similar.

On the assumption that a range in subgrade modulus from 50 to 300 pounds per cubic inch can reasonably be expected in practice, figure 4 shows that stresses computed on the basis of $k=300$, may be too low by as much as 25 percent if the modulus happens to have a value of 50. On the other hand, stresses computed on the assumption that $k=100$ will be too low by less than 10 percent if k happens to equal 50.

In view of all the uncertainties, a value of the subgrade modulus equal to 100 pounds per cubic inch is suggested as a reasonable figure for general use, pending the development of more exact information than is now available.

VALUE OF $E=5$ MILLION POUNDS PER SQUARE INCH SUGGESTED FOR GENERAL USE

Effect of variations in modulus of elasticity of concrete.—In contrast to the lack of data concerning the subgrade modulus, there is a wealth of information with respect to the modulus of elasticity of concrete. Numerous investigations have demonstrated that, in general, the modulus of elasticity increases with age, with increase in strength of the concrete, and with increase in temperature; that it may be higher in wet concrete than in dry; and that it is influenced by the character of the aggregate.

Thirty-five reports on the subject, published during the period 1928 to 1938, inclusive, and involving many variables such as type of aggregate, type of cement, water-cement ratio, and age, give values of the modulus of elasticity ranging from about 1,000,000 to 7,000,000 pounds per square inch for concrete ranging in compressive strength from about 1,000 to 7,000 pounds per square inch. For nearly all of the specimens involved in these investigations the ratio of the modulus of elasticity to the compressive strength falls between the values of 650 and 1,500 and a fair average value of this ratio for all the specimens is 1,000. This is in agreement with the building regulations of the American Concrete Institute (*29*) which recommend that for design purposes the modulus of elasticity of concrete be taken as 1,000 times its compressive strength.

For concrete of the character generally used in pavement construction a range in the value of the modulus of elasticity from 3,000,000 to 6,000,000 pounds per square inch may reasonably be expected. Within this range it is believed that the tendency will be for the values to be high rather than low and the use of relatively high values in design is on the side of safety. The concrete used in the Arlington tests, with flexural and compressive strengths at 28 days of 765 and 3,525 pounds per square inch, respectively, is believed to be fairly representative of the average run of paving concrete. The modulus of elasticity of this concrete, as determined by flexure tests of beams, was about 4,500,000 pounds per square inch for air-dry beams and about 5,500,000 pounds per square inch for beams in a moist condition. The same range in values was observed in tests on the pavement slabs themselves, the higher values being obtained in winter and the lower values in summer.

Figure 5 shows the effect of variations in modulus of elasticity between 3,000,000 and 6,000,000 pounds per square inch on stresses computed for interior, corner, and edge loadings for the same range in values of a and h as in figure 4 and for values of $k=100$ and $k=300$. All stresses are expressed as percentages of the stresses computed for $E=5,000,000$. It may be concluded from these curves that variations in the modulus of elasticity between 3,000,000 and 6,000,000 pounds per square inch do not have a major influence on computed stresses and that the effect of these variations is not

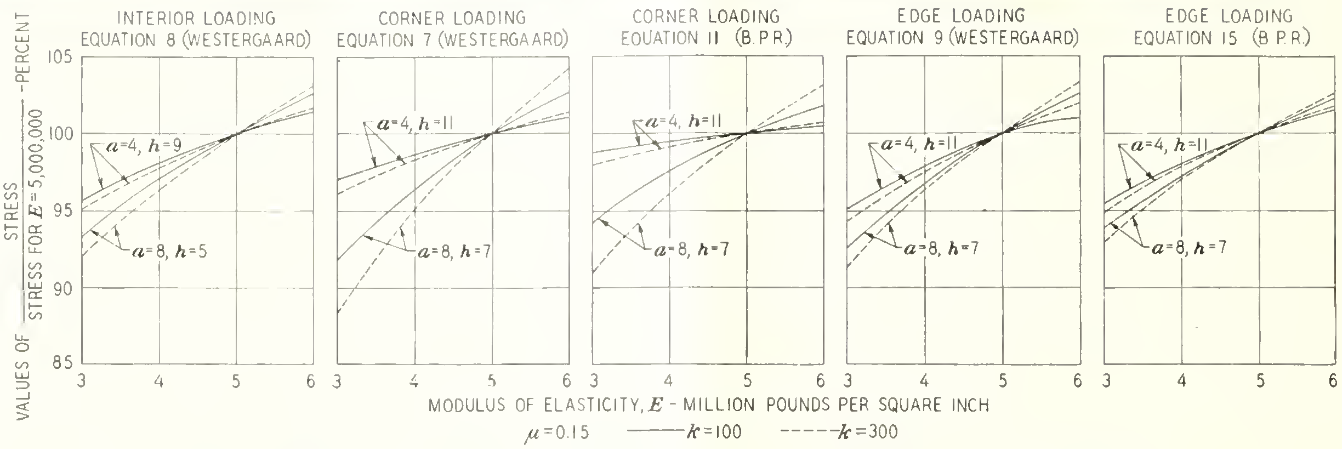

FIGURE 5.—EFFECT ON COMPUTED STRESSES OF VARIATIONS IN MODULUS OF ELASTICITY, E.

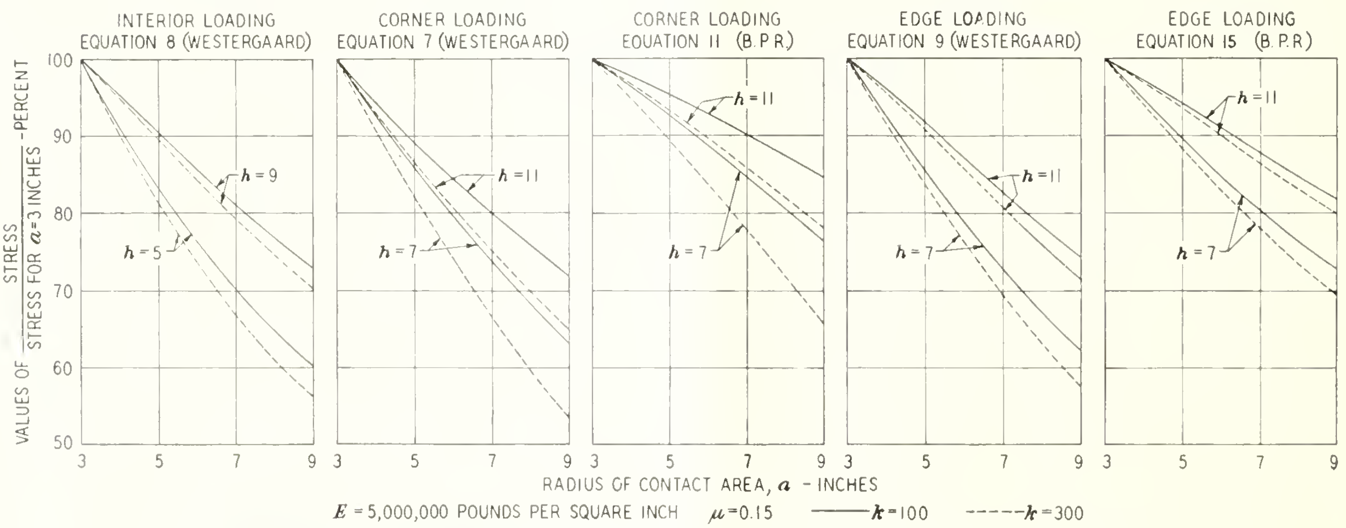

FIGURE 6.—EFFECT ON COMPUTED STRESSES OF VARIATIONS IN RADIUS OF CONTACT AREA, a.

greatly influenced by variations in the subgrade modulus.

Since it is on the side of safety to use relatively high values of the modulus of elasticity and since it is believed that it is representative of what may be expected in practice, the value of $E=5,000,000$ pounds per square inch is suggested for general use.

Variations in radius of contact area.—The radius of contact area, a, appears directly in the equations for corner loading and, through the radius, b, indirectly in the equations for interior and edge loading. Its marked effect on computed stresses is not readily apparent except by some such means as the charts of figure 6.

This figure shows the effect of variations in the radius of contact area between 3 and 9 inches on stresses computed for interior, corner and edge loadings for the same range in values of h as in figures 4 and 5 and for values of $k=100$ and $k=300$. It will be observed that an increase in the radius, a, from 3 to 9 inches may reduce the computed stress by more than 40 percent. It will also be observed that variations in the value of a have less effect on corner stresses and edge stresses computed by equations 11 and 15 than on those computed by equations 7 and 9.

Values of the radius of contact area.—Figure 7 shows the relation between static load and contact area for single and dual high-pressure and balloon tires. The curves are based on data developed by the Bureau of Public Roads in tests of single high-pressure and balloon tires, each in a range of sizes, subjected to static loads ranging from rated tire capacity to more than twice the rated capacity. The curves for single tires shown in figure 7 are closely representative of individual test results throughout the entire range of loadings, indicating that the relation between load and contact area is not appreciably affected by loads in excess of the rated tire capacity.

The curves of figure 7 for dual tires were developed from the data for single tires by assuming the tires to be spaced in accordance with the recommendations of the Tire and Rim Association, and adding to twice the contact area of one tire the area between the two tire impressions.

Figure 8 shows the relation between the wheel load and the radius of tire contact area. These curves were developed from those of figure 7 by assuming the tire contact area to be circular. The further assumption is made in connection with these data that they apply to both static and impact wheel loads.

All the assumptions that have been mentioned, and the additional one that the load is uniformly distributed over the contact area, require discussion.

ASSUMPTIONS REGARDING CONTACT AREAS OF TIRES DISCUSSED

It is known that the distribution of load under a pneumatic tire is not uniform (*30*) and that the shape of the tire impression tends to be elliptical rather than

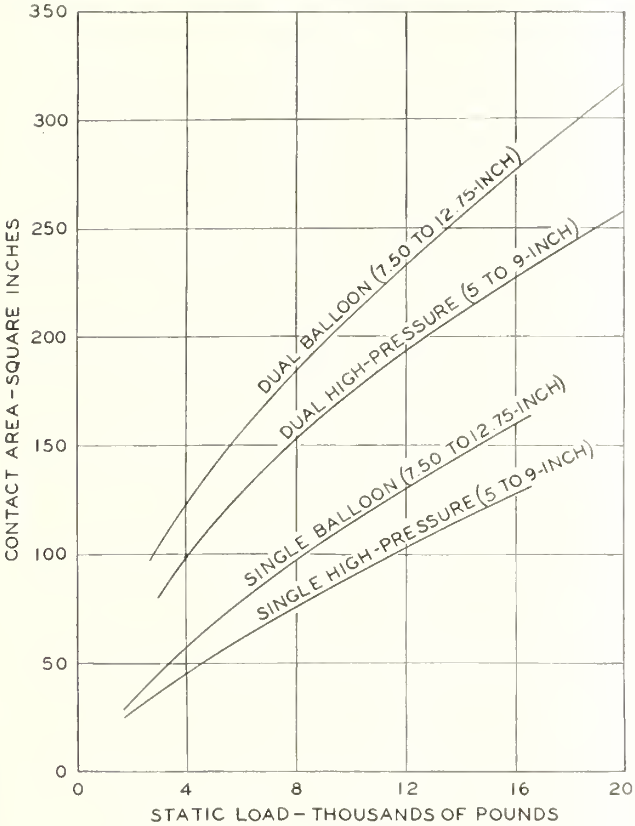

FIGURE 7.—RELATION BETWEEN STATIC LOAD AND AREA OF CONTACT FOR PNEUMATIC TIRES; AVERAGE RESULTS OF TESTS WITH STATIC LOADS RANGING FROM RATED TIRE CAPACITY TO MORE THAN TWICE THE RATED CAPACITY. (AREAS OF CONTACT FOR DUAL TIRES COMPUTED FROM TESTS WITH SINGLE TIRES AND INCLUDE THE AREAS BETWEEN THE TWO TIRE IMPRESSIONS WITH THE TIRES SPACED IN ACCORDANCE WITH THE RECOMMENDATIONS OF THE TIRE AND RIM ASSOCIATION.)

circular. Nevertheless, it is believed that the assumption of uniform loading over a circular area equivalent to the measured contact area will lead to no serious error.

In computing the contact area for dual tires from the data for single tires, the area between the tire contacts is included. Since the area between the tire contacts actually receives no load, this procedure has been questioned. No tests have been made to determine the correctness of the assumption but very limited analysis of certain data developed in the Arlington tests indicate that it is not wholly unreasonable.

Unreported tests by the Bureau of Public Roads indicate that contact areas under impact and equivalent static loads are not greatly different for pneumatic tires of the high-pressure and balloon types. There are also data (*31*) indicating that the vertical deflections of solid and cushion tires are practically the same for the two types of load. While not conclusive, this information appears to justify the assumption that the curves of figure 8 are applicable to impact loads as well as to static loads.

Much additional research work is necessary to prove or disprove the validity of the assumptions that have been discussed. In the absence of such investigations it is necessary to make some assumptions and it is believed that those suggested are reasonable. Also, in the absence of more information than is now available, it is believed that further refinement in the use of existing data is unwarranted.

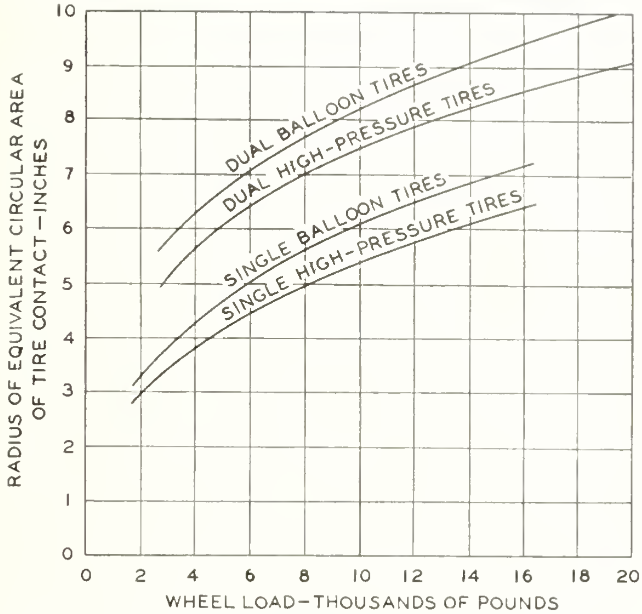

FIGURE 8.—RELATION BETWEEN WHEEL LOAD (STATIC OR IMPACT) AND RADIUS OF EQUIVALENT CIRCULAR AREA OF TIRE CONTACT. RADII CORRESPOND TO CONTACT AREAS SHOWN IN FIGURE 7.

Radius of contact area for edge loading.—The Westergaard analysis assumes that interior and corner loads are applied on circular bearing areas and that edge loads are applied on semicircular bearing areas. Therefore it is necessary to decide: (1) If the semicircle used for edge loading is to have the same area as the circle used for interior and corner loading, or (2) if the semicircle is to have the same radius as the circle. The first procedure involves the assumption of equal unit pressure on the circular and semicircular areas and the second involves the assumption that the unit pressure on the semicircular area is twice as great as on the circular area.

When a wheel equipped with a single pneumatic tire moves along the edge of a pavement slab with depressed shoulders in such manner that only a part of the tire tread is in contact with the slab, the shape of the area of tire contact is undoubtedly changed but the effect on its area is unknown. For this case either assumption as to radius of contact area might be justified.

However, the situation is somewhat different with respect to the dual tires that are common equipment for the heavier wheel loads. It is not uncommon to see wheels with dual tires operated so close to the edge of the pavement that the entire wheel load is carried by the inside tire. In this case the tire load is doubled without a corresponding increase in contact area. For example, assuming an 8,000 pound static wheel load on dual high-pressure tires, table 1 shows that 11,800 pounds is the total impact reaction for this wheel load, and figure 7 shows a corresponding contact area of approximately 194 square inches. Also from figure 7 it is found that for this same load on a single tire the contact area is approximately 102 square inches. The corresponding unit pressures are about 61 and 116 pounds per square inch respectively. In the same manner it may be shown that the same wheel load on dual balloon tires may be expected to develop unit pressures of approximately 49 pounds per square inch over the full area of contact and 88 pounds per square inch when the load is concentrated on one tire.

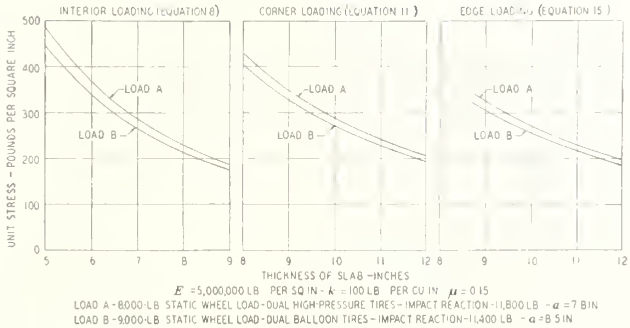

FIGURE 9.—COMPARISON OF STRESSES DUE TO 8,000-POUND WHEEL LOAD ON HIGH-PRESSURE TIRES AND 9,000-POUND WHEEL LOAD ON BALLOON TIRES.

In view of these facts it is recommended that, when the design is based on dual-tire equipment, the radius of area of contact for edge loadings be the same as for interior and corner loadings. Also, in view of the uncertainty regarding single tires, it is suggested that when the design is based on single-tire equipment, the area of contact for edge loadings be the same as for interior and corner loadings. If r is the radius of a circle then the radius of a semicircle of equivalent area equals $r\sqrt{2}$.

Variations in thickness of slab, h.—The fact that the thickness of the slab, h, exerts a major influence on computed stresses is evident from the stress equations. Since an exponential value of h appears twice in each stress equation and, in the equations for interior and edge loading an exponential value of h is also involved in the derivation of the radius, b, the relation between slab thickness and computed stress is not a simple one.

The relation between slab thickness and load stresses is shown graphically in figure 9 for two loads; one a static load of 8,000 pounds on a wheel equipped with dual high-pressure pneumatic tires, and the other a static load of 9,000 pounds on a wheel equipped with dual balloon tires. The impact reactions corresponding to these wheel loads are taken from table 1 and the corresponding radii of contact areas from figure 8. For the slab thicknesses ordinarily encountered in practice, the heavier wheel load on balloon tires gives stresses lower than those for the lighter wheel load on high-pressure tires by about 20 pounds per square inch. Here is justification for the requirement of the Uniform Vehicle Code (*32*) that the maximum wheel load on high-pressure tires be limited to 8,000 pounds and that on balloon tires to 9,000 pounds. It may also be noted that, for slabs of equal thickness, the stress due to corner loading is only slightly in excess of that due to edge loading.

EQUATIONS FOR COMPUTING TEMPERATURE WARPING STRESSES PRESENTED

Warping stresses due to temperature differential.—Changes in the temperature of concrete produce corresponding changes in its volume. A rise in temperature causes expansion of the concrete and a drop in temperature causes it to contract.

The temperature of a concrete pavement is constantly changing owing to variations in air temperature and during these changes in air temperature, which take place at a relatively rapid rate, the temperature in the slab does not remain constant throughout its depth. During the heat of the day in summer the top of the slab is warmer than the bottom while at night the reverse may be true. This differential in temperature between the two surfaces of the slab causes it to warp or curl and, since free warping is prevented by the weight of the slab, bending stresses are developed.

As early as 1926 Westergaard (*33*) presented a theoretical analysis of warping stresses due to temperature but their importance has not been generally recognized, possibly owing to the fact that in his stress computations he assumed a rather low value for the temperature differential. It remained for the Arlington tests (*16*) to demonstrate that these warping stresses may be as great as those produced by heavy wheel loads.

Westergaard's analysis covers slabs of infinite length and width, those of finite width and infinite length, and suggests a procedure to be followed in slabs having finite dimensions in both directions. On the basis of this analysis Bradbury (*9*) has developed general equations for the computation of temperature-warping stresses in the edge and interior of pavement slabs of the usual dimensions.

The following equations are not in exactly the same form as Bradbury's but they give identical results:

Edge Stresses

$$\sigma_{xe}=\frac{C_xEet}{2} \qquad (17)$$

Interior Stresses

$$\sigma_x=\frac{Eet}{2}\left(\frac{C_x+\mu C_y}{1-\mu^2}\right) \qquad (18)$$

$$\sigma_y=\frac{Eet}{2}\left(\frac{C_y+\mu C_x}{1-\mu^2}\right) \qquad (19)$$

in which

σ_{xe} = maximum stress, in pounds per square inch, in the extreme fiber at the edge of the slab, in the direction of slab length. At the extreme edge the stress at right angles to the edge is zero;

σ_x = maximum stress, in pounds per square inch, in the extreme fiber at the interior of the slab, in the direction of slab length;

σ_y = maximum stress, in pounds per square inch, in the extreme fiber at the interior of the slab, in the direction of slab width;

E = modulus of elasticity of concrete, in pounds per square inch;

e = thermal coefficient of expansion and contraction of concrete per degree Fahrenheit;

t = difference in temperature between top and bottom of slab, in degrees Fahrenheit;

C_x and C_y are coefficients determined from the curve in figure 10.

In figure 10:

L_x = length of slab in inches;

L_y = width of slab in inches;

l = radius of relative stiffness in inches (equation 6);

C_x corresponds to the value of $\frac{L_x}{l}$

C_y corresponds to the value of $\frac{L_y}{l}$

The data in figure 10 are also given in table 9.

The direction of slab warping is determined by the relation between the temperature in the top of the slab and that in the bottom and this in turn determines whether the resulting stress is a tensile stress in the top

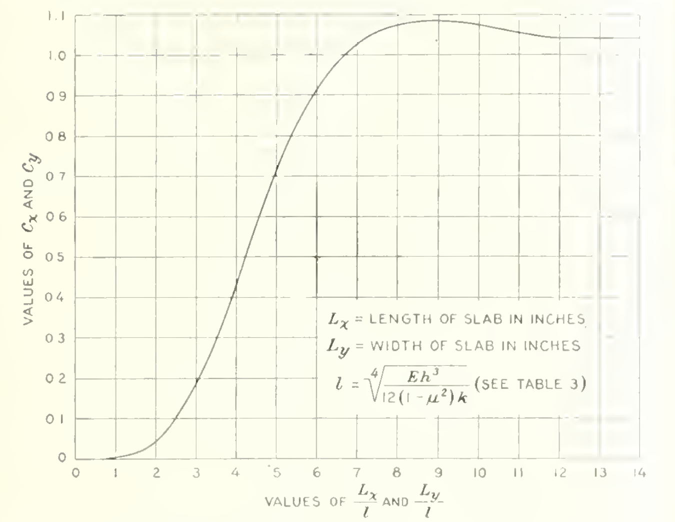

FIGURE 10.—COEFFICIENTS FOR WARPING STRESSES DUE TO TEMPERATURE.

of the slab or a tensile stress in the bottom of the slab. Of course, in either case an equal compressive stress is created in the opposite surface. For convenience the temperature differential will be considered positive when the top of the slab is at a higher temperature than the bottom and negative when the top of the slab is at a lower temperature than the bottom. A positive differential creates tensile stress in the bottom of the slab and a negative differential creates tensile stress in the top of the slab.

TABLE 9.—*Coordinates of curve of figure 10*

$\frac{L_x}{l}$ or $\frac{L_y}{l}$	C_x or C_y	$\frac{L_x}{l}$ or $\frac{L_y}{l}$	C_x or C_y	$\frac{L_x}{l}$ or $\frac{L_y}{l}$	C_x or C_y
1.41	0.010	4.95	.701	7.78	1.069
2.12	.051	5.66	.856	8.49	1.084
2.83	.148	6.37	.964	9.90	1.078
3.54	.309	6.69	1.000	11.31 [1]	1.052
4.24	.508	7.07	1.032		

[1] For values of $\frac{L_x}{l}$ or $\frac{L_y}{l}$ greater than 11.31, the values of C_x and C_y are determined by a composite curve constructed as follows:

Extend the curve plotted from the data in the above table from $\left(\frac{L_x}{l}=11.31,\ C_x=1.052\right)$ toward $\left(\frac{L_x}{l}=14.14,\ C_x=1.009\right)$ until it intersects a horizontal line drawn through $C_x=1.043$. C_x or C_y for all values of $\frac{L_x}{l}$ or $\frac{L_y}{l}$ to the right of this intersection is equal to 1.043.

Value of temperature differential.—The data developed in the Arlington tests (*16*) showed that the maximum temperature differential varies with the depth of the slab, being greater in thick slabs than in thin ones. The maximum positive differential occurs in the daytime and is greater in summer than in winter. The maximum negative differential occurs at night and is much the same in both winter and summer. The published data are summerized in tables 10 and 11.

From these data Bradbury (*9*) concluded that, for purposes of design computations, the maximum positive temperature differential might be assumed as 3.0° F. per inch of slab thickness and the maximum negative differential as 1.0° F. per inch of slab thickness. These appear to be reasonable figures for general use but it should be recognized that they are merely average figures and will result in computed stresses that may be appreciably lower than the stresses that will occur at times in the pavement.

TABLE 10.—*Summary of values of maximum positive temperature differentials observed in Arlington tests on 27 days between April 3 and June 4, 1934* [1]

	At edge of slab of uniform thickness		Thickened-edge section 9-6-9 inch		
	6-inch slab	9-inch slab	Edge	18 inches from edge	36 inches from edge
	° F.	° F.	° F.	° F.	° F.
Maximum	+24	+33	+33	+31	+28
Minimum	+14	+20	+18	+17	+15
Average	+19	+27	+27	+25	+22

[1] Data from table 2, PUBLIC ROADS, November 1935.

TABLE 11.—*Summary of values of maximum temperature differentials observed in Arlington tests on 17 days during 1931, 1932 and 1933* [1]

	6-inch slab				9-inch slab	
	April to August, inclusive		September to February, inclusive		April to August, inclusive	
	Day	Night	Day	Night	Day	Night
	° F.	° F.	° F.	° F.	° F.	° F.
Maximum	+24.3	−6.5	+15.6	−6.7	+31.0	−9.2
Minimum	+18.7	−4.5	+8.2	−1.3	+22.3	−5.7
Average	+21.2	−5.8	+11.8	−4.1	+26.9	−7.5

[1] Data from table 1, PUBLIC ROADS, November 1935.

FOR TEMPERATURE WARPING, INTERIOR STRESSES EXCEED EDGE STRESSES

Value of the thermal coefficient of expansion.—The thermal coefficient of expansion and contraction of concrete depends on a number of factors, among which the character of the aggregate appears to be the most important. Data from a number of investigations indicate that in general the highest thermal coefficient will be found in concrete containing siliceous aggregates and that considerably lower values may be expected in concrete made with granite, limestone, or diabase aggregates. A summary of data given by various authorities (*34*) shows values of the thermal coefficient ranging from about 0.000004 to about 0.000007 per degree Fahrenheit for concrete having a cement content comparable to that used in pavement construction.

The concrete used in the Arlington tests, with a limestone coarse aggregate and a siliceous fine aggregate, had a coefficient of approximately 0.000005 per degree Fahrenheit and this value appears to be a satisfactory one for general use. However, when the circumstances are such as to make this possible, it will be well to select a value appropriate for the character of concrete that is under consideration.

Computed warping stresses.—The Arlington tests were all made on slabs that varied in dimensions only in depth. Within these limitations the observed warping stresses due to temperature differential were in reasonably good agreement with computed stresses.

Stresses computed by the Bradbury equations are shown graphically in figure 11 for the interior, and in figure 12 for the edge, of slabs 10 feet wide and of various lengths, depths of 6 and 9 inches, and values of the subgrade modulus of 100 and 300 pounds per cubic inch.

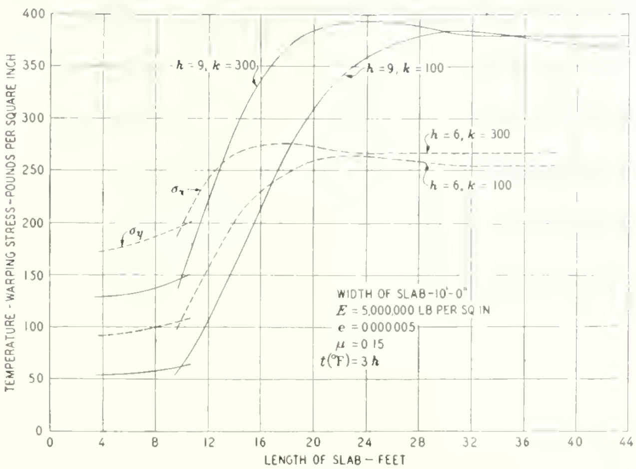

FIGURE 11.—TEMPERATURE-WARPING STRESSES, INTERIOR OF SLAB.

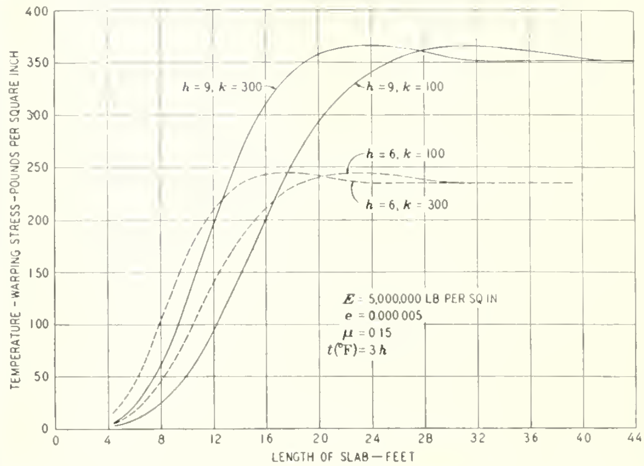

FIGURE 12.—TEMPERATURE-WARPING STRESSES, EDGE OF SLAB.

The most striking fact shown by these curves is the magnitude of the maximum temperature-warping stresses, which are of the order of 275 and 375 pounds per square inch, respectively, for the 6-inch and 9-inch slabs. Other interesting observations that may be made are enumerated as follows:

1. A comparison of figures 11 and 12 shows that maximum edge stresses are always lower than maximum interior stresses but the difference is not great except in slabs having a length less than the width. (In this discussion the length of the slab is considered as the dimension in the direction of the longitudinal axis of the pavement even though it may be less than the width of the slab.)

2. Increases in the length of the slab beyond about 18 feet for the 6-inch slab, and about 24 feet for the 9-inch slab, have no great influence on maximum edge or interior stresses. Below these limits, decreases in slab length result in rapid reduction in stress.

3. In the interior of the slab, $\sigma_x = \sigma_y$ when the slab is square. When the length exceeds the width, σ_x is greater than σ_y and when the length is less than the width the reverse is true. Between the upper limits of slab length that have been mentioned and the point at which the length equals the width, reduction in slab length results in rapid reduction in maximum interior stresses. When the length is less than the width the critical warping stress is influenced primarily by the width and variations in length have little effect on its magnitude. In contrast to this, edge stresses decrease continuously with decreasing slab length.

4. For the longer slabs the maximum stresses in the 9-inch slab exceed those in the 6-inch slab by 40 to 50 percent. However, for slab lengths less than about 17 feet for $k = 100$ and 13 feet for $k = 300$, the stresses in the 6-inch slab exceed those in the 9-inch slab by as much as 50 pounds per square inch.

5. Variations in the value of the subgrade modulus have no significant influence on the stresses in long slabs. However, for short slabs increases in the value of the subgrade modulus result in considerable increases in the computed stresses. Figures 11 and 12 show that the stresses in the 9-inch slab for $k = 300$ may exceed those for $k = 100$ by more than 100 pounds per square inch. The difference is somewhat less in the case of the 6-inch slab.

This effect of subgrade modulus on temperature stresses is the reverse of its effect on stresses due to wheel loads where low values of the modulus give higher stresses than do high values. In the case of combined stresses due to load and temperature warping this reversal of influence tends to compensate somewhat for possible errors in computed stresses owing to the assumption of a subgrade modulus different from that which may actually exist.

For example, assuming an 8,000-pound static wheel load on high-pressure dual tires, table 1 shows the total impact reaction to be 11,800 pounds and figure 8 gives a value of a equal to 7.8 inches. For $\mu = 0.15$ and $E = 5{,}000{,}000$, equation 8 gives interior stresses in a 6-inch slab of approximately 365 pounds per square inch for $k = 100$ and 315 pounds per square inch for $k = 300$. From figure 11 the corresponding warping stresses in a slab 14 feet long are 200 and 265 pounds per square inch. The combined stresses due to load and temperature are then 565 pounds per square inch for $k = 100$ and 580 pounds per square inch for $k = 300$.

Thus it appears that, for short slabs, variations in the subgrade modulus may be expected to have a minor influence on combined stresses. However, for slabs of the length commonly used in pavements, the effect of subgrade modulus on warping stresses is slight, with the result that it will have a noticeable effect on combined stresses. Therefore, the value of $k = 100$ pounds per cubic inch appears to be a desirable figure for general use in the computation of combined stresses as well as for stresses due to wheel loads only.

TEMPERATURE WARPING STRESSES CAUSE MUCH CRACKING OF CONCRETE PAVEMENTS

Table 12 is presented to show the effect of width of pavement on transverse warping stresses. The figures indicate that the warping stresses in a slab 20 feet wide may exceed 300 pounds per square inch and may be more than twice as great as the stresses in a slab 10 feet wide. Figures such as these show the reason for the use of longitudinal joints in concrete pavements, the necessity for which has been thoroughly demonstrated by practical experience.

It is evident from equations 17, 18, and 19 that the computed warping stress due to temperature differential varies directly with values of the modulus of elasticity, E, the thermal coefficient, e, and the temperature differential, t. The stress values shown in figures 11 and 12 are based on assumed values of E, e and t that may be considered as average rather than maximum.

The value of E may exceed 5,000,000 pounds per square inch, the value of e may exceed 0.000005 per degree Fahrenheit and, at times, the value of t is very likely to exceed 3° F. per inch of slab thickness. In the Arlington tests (tables 10 and 11) values of the temperature differential as high as 4° F. per inch of slab thickness were observed occasionally. Therefore the warping stresses that may exist at certain times in concrete pavements having a high modulus of elasticity and a high thermal coefficient may be more than twice as great as the stresses shown in figures 11 and 12.

TABLE 12.—*Transverse temperature-warping stresses in slabs 30 feet long*

$\mu = 0.15$.
$E = 5{,}000{,}000$ pounds per square inch.
$e = 0.000005$.
$t(°F.) = 3h$ (inches).

Subgrade modulus k	Width of slab	Depth of slab: 6 inches	Depth of slab: 7 inches	Depth of slab: 8 inches
Lb. per cu. in.	*Feet*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*
100	10	130	120	115
	20	280	320	340
300	10	210	200	190
	20	285	335	380

It should be noted also that the assumption of a 10-foot width of slab for the computation of the longitudinal interior warping stresses shown in figure 12 involves also the assumption that the longitudinal joint offers no restraint to warping. Actually the types of longitudinal joints in common use may be expected to develop some restraint to warping and such restraint as may exist serves to increase the computed interior warping stresses, both in the longitudinal and transverse directions.

It seems reasonable to conclude that the magnitude of the stress that may be induced by temperature warping explains much of the cracking that takes place in concrete pavements which, in the past, has frequently been attributed to other causes. The possible magnitude of these stresses indicates the importance of the use of curing methods that will protect the concrete from extreme changes of temperature during its early life when its strength is low.

Corner warping stresses.—An exact mathematical analysis of stresses produced by temperature warping near the corner of a slab is not available and an approximate solution must be used for stress computation. Both theory and experiment (*16*) indicate that the warping stress increases as the distance from the corner along the diagonal bisector increases. The warping stress that is important is that which occurs at the point of maximum load stress. Bradbury (*9*) has developed an approximate equation for this stress, which is

$$\sigma_{cw} = \frac{Eet}{3(1-\mu)}\sqrt{\frac{a}{l}} \qquad (20)$$

Combinations of simultaneous stresses due to load and temperature:

Corner.—When the temperature differential is positive it produces compressive stress in the top of the slab, whereas corner loading produces tensile stress. Therefore, since the combined stress due to warping and load is less than stress due to load alone, this condition requires no further consideration. At night, when the slab is warped upward, the two stresses are of the same sign

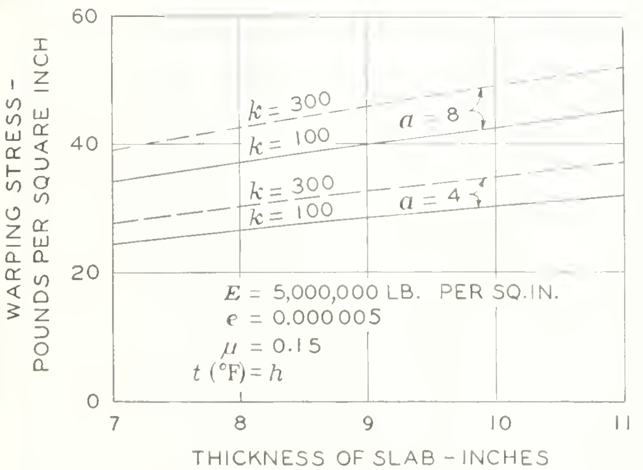

FIGURE 13.—TEMPERATURE-WARPING STRESSES, CORNER OF SLAB.

and therefore the warping stress tends to increase the combined stress. However, the effect is not great since at night the temperature differential, and the resultant warping stress, are small.

Corner-warping stresses computed by equation 20 are shown in figure 13 for an assumed temperature differential of 1° F. per inch of slab thickness. The curves show no great effect of any of the variables considered and the assumption of a flat value for the warping stress of about 40 pounds per square inch would probably be sufficiently accurate for all practical purposes. This value is in good agreement with observed values ((*18*), table 14).

Edge.—When temperature-warping stresses in the edge of the slab are combined with load stresses, two combinations require consideration. In the daytime, when the edge of the slab is warped down so that it is in contact with the subgrade, the load stresses are computed by Westergaard's formula (equation 9) and these should be combined with warping stresses computed for the daytime temperature differential of 3° F. per inch of slab depth. In this case both load and temperature create tensile stress in the bottom of the slab.

The second combination is that of maximum load stresses, which occur at night when the edge of the slab is warped upward, with the warping stresses computed for the nighttime temperature differential of 1° F. per inch of slab thickness. For these assumed temperature differentials the warping stress at night is one-third as large as that which occurs during the day and it is of opposite sign from stress due to load. Therefore, the combined stress at night is less than the stress due to load alone.

MOISTURE WARPING STRESSES CAN BE SAFELY IGNORED IN DESIGN

Interior.—In the Arlington tests (*16*) it was found that the condition of slab warping had a negligible effect on the magnitude of the maximum stress produced by a load applied at the interior of the slab. The maximum load stress at the interior is about the same at night when the edges of the slab are warped upward as in the daytime when the edges are warped down. Therefore, in the determination of the maximum combined stress due to load and temperature warping, the maximum load stress should be combined with the warping stress produced by the temperature differential that occurs in the daytime.

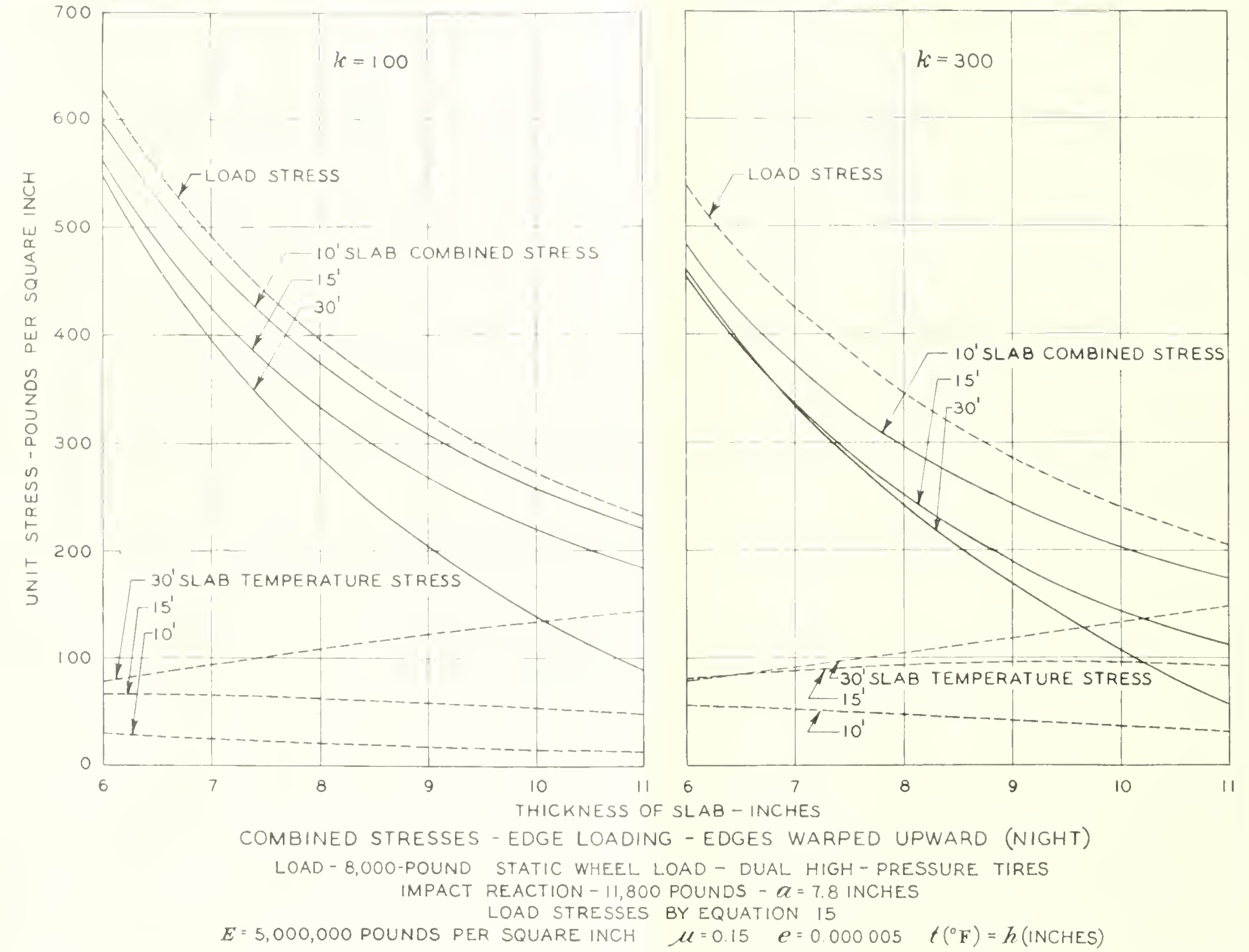

FIGURE 14.—EFFECT OF SLAB THICKNESS, SUBGRADE MODULUS, AND SLAB LENGTH ON COMBINED STRESSES DUE TO LOAD AND TEMPERATURE WARPING IN THE EDGE OF A SLAB 10 FEET WIDE.

Moisture warping.—Since concrete expands and contracts with changes in moisture content, it follows that a difference in the moisture content between the top and bottom of a concrete pavement slab causes the slab to warp or curl in much the same manner as does a differential in temperature. When the top of the slab is dryer than the bottom the edges of the slab curl upward and when the moisture differential is in the opposite direction the edges of the slab curl downward.

As a result of the extensive observations made in the Arlington tests (*16*) it was concluded that, for the climatic conditions that prevailed, the moisture content of a pavement slab is at a maximum, and the moisture gradient that causes warping is at a minimum, during the period from January to March. As compared with the conditions that prevailed during this period, it was found that the edges of the slab were curled upward during the summer months, when the top of the slab was dryer than the bottom, and began to curl downward again during the fall.

Thus the warping of the slab caused by moisture differential is a seasonal change which takes place slowly over a considerable period of time during which there is opportunity for plastic yield of the concrete to take place. Also it was observed in the Arlington tests that as the seasonal warping takes place the slab settles into the subgrade, thus reducing the restraint to warping due to the weight of the slab. Because of the time element and its effect on the adjustment between slab and subgrade and on the plastic flow of the concrete, it seems very probable that stresses due to moisture warping are not as great as the deformations in the concrete would indicate.

For these reasons the strains due to moisture warping that have been measured in connection with the Arlington tests cannot be translated into stress with any certainty. However, the observations made indicate that the curvature caused by moisture is principally an upward warping of the edges caused by moisture loss from the top of the slab during the warm season of the year, and that the downward warping that takes place when the moisture in the top of the slab exceeds that in the bottom may be expected to be considerably smaller. Thus, during hot summer days when moisture and temperature differentials are both a maximum, the curvature caused by one is in the opposite direction to that caused by the other and such stress as may be caused by moisture serves to reduce rather than to increase the stress due to temperature warping. Since the stresses due to moisture warping cannot be evaluated, it is fortunate that the evidence indicates that they may be disregarded with safety in computing the stresses in pavement slabs. To ignore them appears

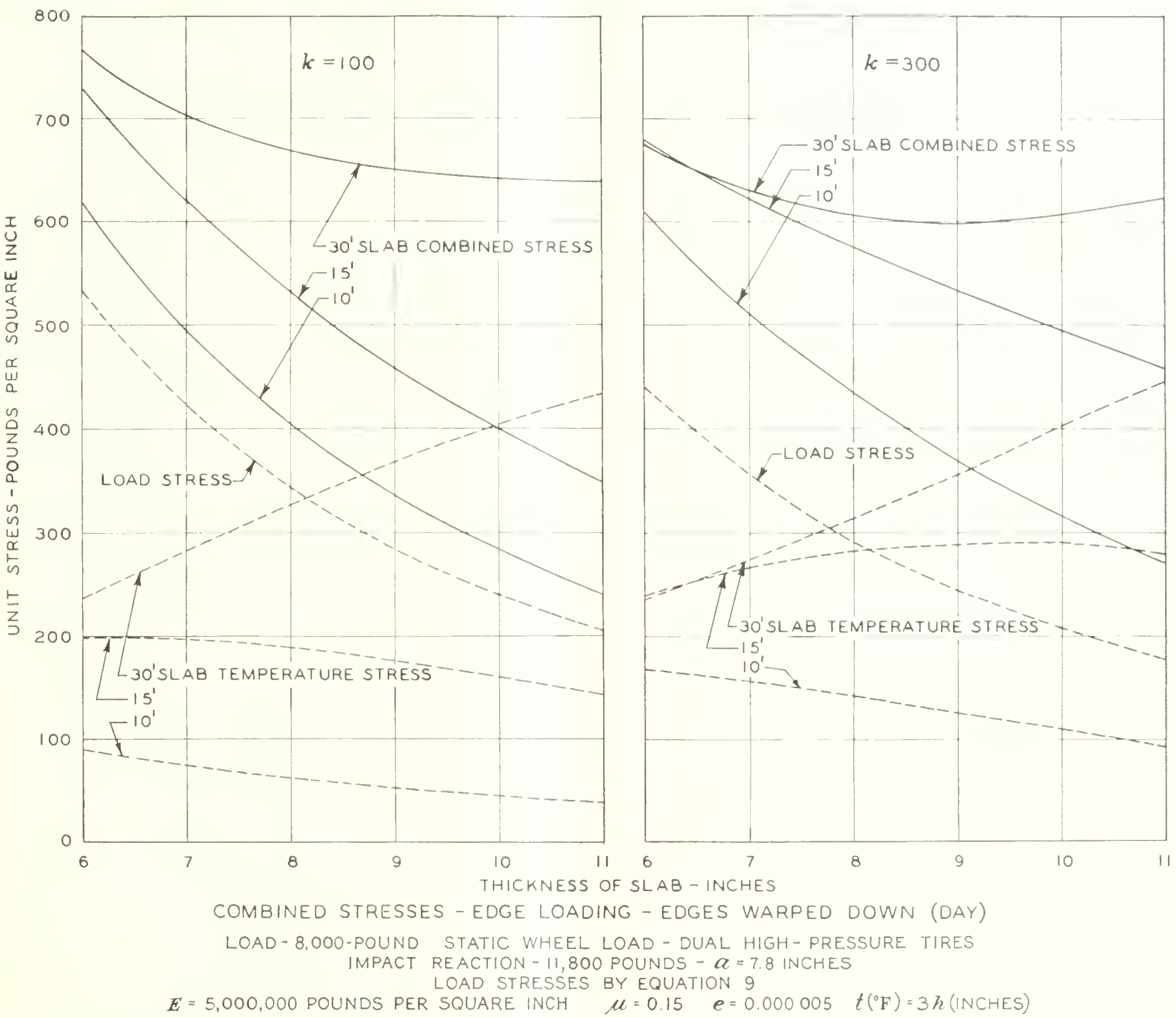

FIGURE 15.—EFFECT OF SLAB THICKNESS, SUBGRADE MODULUS, AND SLAB LENGTH ON COMBINED STRESSES DUE TO LOAD AND TEMPERATURE WARPING IN THE EDGE OF A SLAB 10 FEET WIDE.

to add some factor of safety of unknown magnitude and importance.

Combined stresses.—Total combined stresses due to load and temperature warping are shown in figures 14, 15, and 16 for the edge and interior of slabs of different depths, a width of 10 feet and lengths of 10, 15, and 30 feet. Combined corner stresses, which are not influenced by the dimensions of the slab other than depth, are shown in the left part of figure 17. The assumed load is an 8,000-pound wheel load on dual high-pressure tires. The edge-load stresses of figure 14 are computed by equation 15 for the nighttime condition of upward warping and therefore the assumed temperature differential for the warping stresses is taken as 1° F. per inch of slab thickness. Since the warping stresses and load stresses are of opposite sign, the combined edge stresses of figure 14 are less than the load stresses. For the reasons that have been given, the assumed temperature differential for the corner warping stresses of figure 17 is also taken as 1° F. per inch of slab thickness. The edge-load stresses of figure 15 are computed by equation 9 for daytime conditions and therefore the assumed temperature differential for the warping stresses is taken as 3° F. per inch of slab thickness. The same differential is also used for computing interior warping stresses to be combined with interior load stresses in figure 16.

As would be expected from the previous discussion, the computed corner warping stresses are small, ranging from about 30 to 50 pounds per square inch for the range of variables assumed, and their effect on combined corner stresses is practically negligible.

REDUCING SLAB LENGTH TO 10 FEET GREATLY REDUCES COMBINED STRESSES

It may be observed that in all cases, for a given thickness of slab and the same value of the subgrade modulus, the combined edge stresses of figure 15 are larger than those of figure 14. The somewhat larger load stresses that may occur at night (equation 15), when reduced by the warping stresses, are less than the lower load stresses of equation 9 in combination with the high warping stresses that occur during the day. Except in slabs 10 feet long the differences are of considerable

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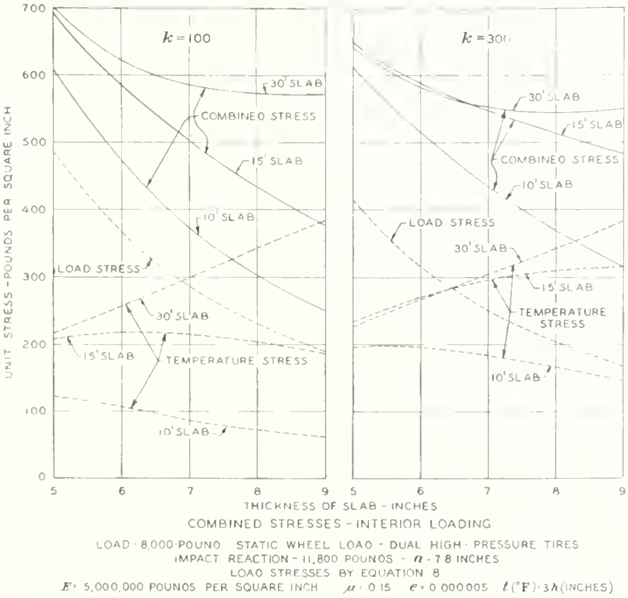

FIGURE 16.—EFFECT OF SLAB THICKNESS, SUBGRADE MODULUS, AND SLAB LENGTH ON COMBINED STRESSES DUE TO LOAD AND TEMPERATURE WARPING IN THE INTERIOR OF A SLAB 10 FEET WIDE.

magnitude. In view of this, the combined stresses of figure 14 will be disregarded in the subsequent discussion although it should be recognized that other assumptions than those which determine the curves of figures 14 and 15 might lead to different relative values.

Bearing in mind that the temperature warping stresses shown in figures 15 and 16 may be regarded as average rather than probable maximum values, the following interesting observations may be made with respect to the combined edge stresses of figure 15 and the combined interior stresses of figure 16, both being for a slab 10 feet wide.

1. In slabs 30 feet long an increase in the depth of slab does not effect any marked decrease in the total combined stress. In fact, for $k=300$, there is a slight increase in interior stress as the slab thickness is increased beyond 8 inches and in the edge stress as the thickness is increased beyond 9 inches.

2. In slabs 30 feet long a high value of the subgrade modulus results in a lower combined stress than a low value of the modulus, but for values between $k=100$ and $k=300$ the difference is not great enough to be significant.

3. In slabs 30 feet long the combined edge stresses are somewhat higher than those in the interior of the slab. For an 8-inch slab the difference is about 100 pounds per square inch for $k=100$ and 60 pounds per square inch for $k=300$.

4. Reducing the slab length from 30 to 15 feet results in some reduction in interior stress when $k=100$ but has very little effect when $k=300$. In general, this reduction in slab length has a greater effect on combined edge stresses than on combined interior stresses and the reduction in stress is considerably greater when $k=100$ than when $k=300$.

5. In slabs 15 feet long in contrast to those 30 feet long, a high value of the subgrade modulus generally results in a higher combined stress than does a low value of the modulus. In an 8-inch slab, interior and edge stresses for $k=300$ exceed those for $k=100$ by about 80 pounds per square inch and 40 pounds per square inch, respectively.

6. Reducing the slab length from 30 to 10 feet results in an appreciable reduction in combined interior and edge stresses. The combined stresses in an 8-inch slab, as shown in figures 15 and 16, are given in table 13.

The combined stresses which may occur in the daytime in the free edge of a transverse joint in a slab 10 feet wide are shown in the second chart of figure 17. The curves show that the depth of slab has a marked influence on combined stresses but that the effect of variations in the subgrade modulus between $k=100$ and $k=300$ is negligible.

From the above discussion it may be concluded, for the stress-producing conditions assumed, that:

1. In slabs as long as 30 feet, the depth of slab has very little influence on the magnitude of combined interior and edge stresses.

2. In slabs as long as 30 feet, combined edge stresses and combined interior stresses of the order of 600 pounds per square inch are to be expected under what may be considered average conditions. When the concrete has a higher thermal coefficient and a higher modulus of elasticity than the values used in these computations and when the temperature differential is higher than that assumed, these combined stresses may be greatly increased.

TABLE 13.—*Combined edge and interior stresses in a slab 10 feet wide and 8 inches thick* [1]

Slab length	Combined edge stress		Combined interior stress	
	$k=100$	$k=300$	$k=100$	$k=300$
Feet	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*
30	670	610	570	550
15	530	570	430	510
10	400	430	300	370

[1] From figs. 15 and 16.

3. In order to effect any significant reduction in combined stresses in the edge and interior of the slab it is necessary to reduce the slab length to about 10 feet. In a slab 10 feet long and 8 inches thick the combined stresses will be of the order of 400 pounds per square inch as compared with 600 pounds per square inch in a slab 30 feet long.

4. In short slabs the depth of the slab has a very marked influence on combined stresses at the edge and interior. In slabs of any length the depth of slab has a marked influence on combined stresses at the corners and edges of free transverse joints.

5. The character of the subgrade, as measured by variations in the subgrade modulus between $k=100$ and $k=300$, does not have a great effect or a consistent effect on the magnitude of combined stresses. In long slabs the higher interior and edge stresses are associated with the lower values of the modulus while in short slabs the reverse is true.

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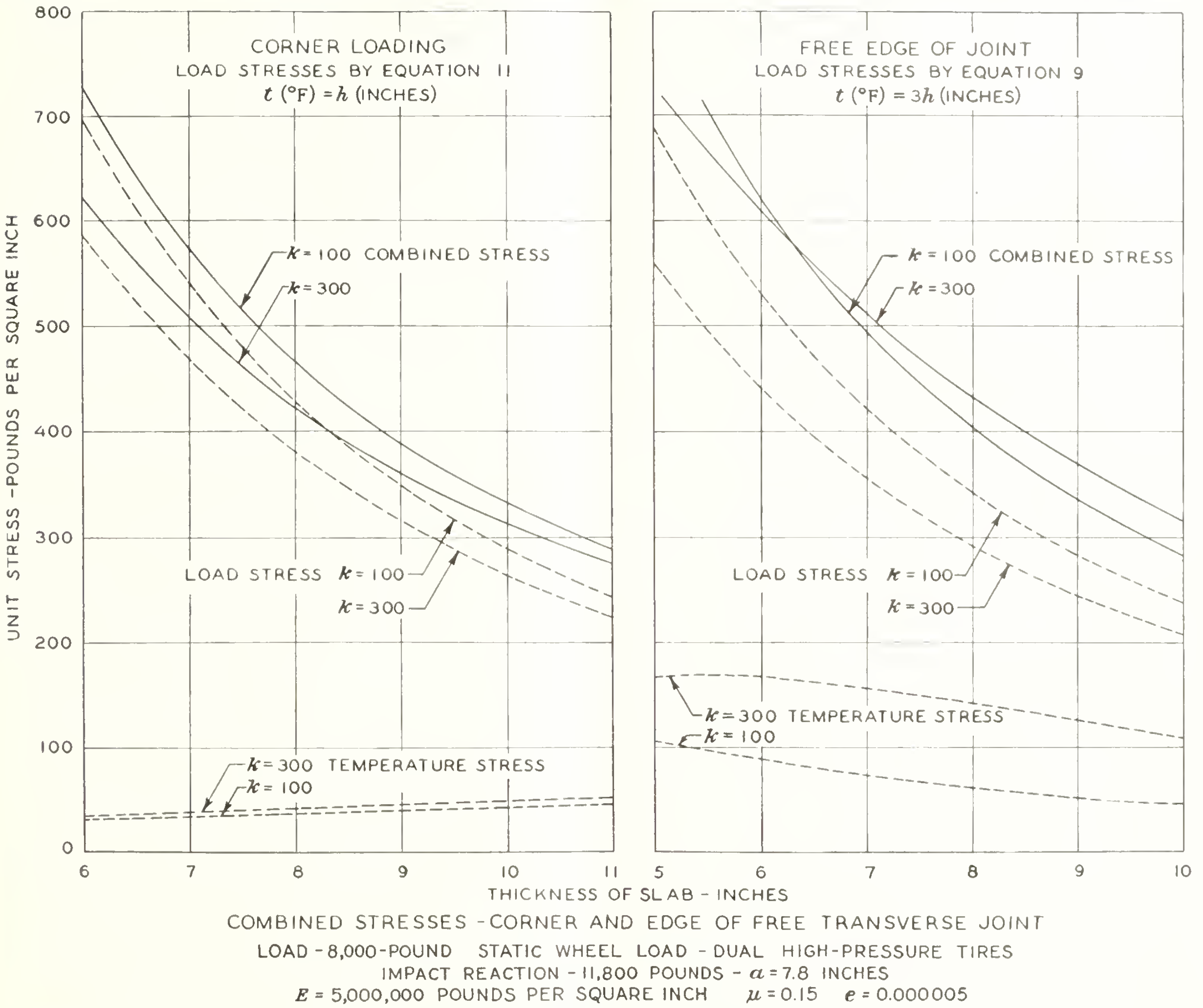

FIGURE 17.—EFFECT OF SLAB THICKNESS AND SUBGRADE MODULUS ON COMBINED STRESSES DUE TO LOAD AND TEMPERATURE WARPING IN THE CORNER AND EDGE OF A FREE TRANSVERSE JOINT IN A SLAB 10 FEET WIDE.

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STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF JUNE 30, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 7,005,608	$ 3,212,670	246.6	$ 8,323,267	$ 4,148,643	308.6	$ 727,350	$ 361,820	21.9	$ 3,101,328
Arizona	2,516,890	1,799,919	125.5	1,261,511	895,630	50.9	289,535	204,830	19.4	1,825,489
Arkansas	1,810,161	1,792,683	107.1	3,271,382	3,267,487	220.1	196,261	194,176	4.3	1,738,388
California	11,119,638	5,996,198	256.4	5,294,381	2,908,628	63.8	858,878	431,709	9.7	4,293,753
Colorado	3,615,134	1,936,720	139.0	3,857,872	2,145,923	88.4	454,385	254,526	8.6	2,202,372
Connecticut	1,191,520	582,221	11.6	1,389,368	690,879	15.7	694,532	343,528	5.6	1,331,528
Delaware	743,081	366,830	17.8	524,921	257,380	12.7	1,451,557	712,821	25.8	1,008,742
Florida	3,584,507	1,747,762	83.4	2,357,420	1,178,710	41.0	1,771,868	885,709	32.3	2,904,467
Georgia	5,625,422	2,700,961	281.2	5,545,490	2,772,745	285.1	3,630,798	1,815,399	220.9	5,652,962
Idaho	2,207,117	1,250,690	198.2	2,034,693	1,227,773	55.3				1,662,248
Illinois	13,131,577	6,485,708	330.2	8,199,507	4,096,859	193.6	3,570,527	1,794,623	66.9	3,545,138
Indiana	6,372,925	3,076,689	163.3	5,012,566	2,454,775	90.5	3,400,506	1,647,659	86.5	2,268,286
Iowa	8,459,615	4,016,131	290.0	4,297,566	1,874,633	158.9	1,598,718	747,200	81.4	1,533,270
Kansas	6,098,327	3,017,327	757.6	3,533,265	1,758,927	155.1	3,798,690	1,898,465	219.3	4,195,785
Kentucky	5,801,316	2,867,271	222.7	4,052,874	2,024,881	84.0	1,271,322	635,661	70.4	3,017,531
Louisiana	1,520,382	750,988	38.3	11,671,198	2,929,730	52.3	1,497,266	727,906	28.0	2,703,740
Maine	3,104,397	1,518,792	71.2	1,504,944	752,471	29.4	1,177,536	588,768	31.4	406,151
Maryland	1,223,576	609,522	19.9	2,799,245	1,388,291	45.7	1,376,636	679,505	21.0	1,824,539
Massachusetts	2,616,317	1,307,565	16.3	3,348,765	1,671,693	25.1	1,537,039	765,997	10.3	2,574,887
Michigan	8,416,730	3,949,240	174.1	4,962,839	2,478,972	145.8	1,498,700	651,700	32.8	3,145,182
Minnesota	5,172,346	2,480,193	317.9	6,165,319	3,066,676	305.3	2,305,132	1,149,598	175.5	3,815,595
Mississippi	6,571,088	2,879,023	284.0	7,579,632	2,710,048	315.6	1,303,300	480,134	54.9	2,824,715
Missouri	6,295,359	3,026,127	165.5	4,927,490	2,451,293	182.9	2,638,432	1,238,973	83.4	4,663,254
Montana	2,305,566	1,295,090	103.8	3,561,604	2,014,552	184.1	212,688	120,632	11.2	4,445,648
Nebraska	4,854,308	2,274,896	413.9	5,297,999	2,667,993	445.4	2,828,638	1,415,819	308.2	2,890,294
Nevada	2,255,396	1,914,681	202.5	963,736	833,163	42.0	46,009	39,572	3.3	1,603,560
New Hampshire	1,316,109	647,939	25.7	620,591	305,385	14.1	964,343	476,703	29.8	932,785
New Jersey	2,999,495	1,481,172	20.0	3,058,686	1,527,793	28.9	1,193,400	546,700	2.5	2,246,292
New Mexico	2,827,851	1,819,345	294.9	1,652,917	1,007,894	80.6	458,711	286,898	41.5	1,524,043
New York	16,218,410	7,724,629	273.1	10,708,900	5,231,287	190.3	3,033,410	1,286,710	41.3	4,110,959
North Carolina	7,815,839	3,713,373	336.7	6,506,843	3,247,822	391.1	1,512,260	694,460	64.2	2,112,429
North Dakota	3,622,515	3,334,141	292.3	256,090	137,089	26.1	3,282,864	1,759,544	323.0	3,430,254
Ohio	9,429,801	4,649,998	112.3	9,715,552	4,789,387	104.1	1,757,150	883,535	25.4	7,319,148
Oklahoma	6,819,168	3,562,656	272.7	1,977,680	1,046,474	29.7	2,693,791	1,428,185	93.3	3,841,245
Oregon	3,452,899	2,010,145	159.0	2,951,521	1,789,322	127.0	68,256	40,480	1.1	2,249,713
Pennsylvania	8,886,117	4,359,444	144.2	10,192,359	4,922,665	99.4	1,706,868	840,917	20.7	5,231,412
Rhode Island	1,382,093	681,850	17.9	508,476	254,051	6.1	629,776	314,400	8.0	1,073,748
South Carolina	5,396,142	2,404,078	267.1	2,862,034	1,277,487	86.3	117,200	53,000	23.7	2,408,179
South Dakota	2,363,488	1,319,786	300.5	4,323,949	2,391,170	404.8	1,557,870	898,070	145.3	3,427,465
Tennessee	6,950,643	3,442,913	207.5	3,852,210	1,926,105	106.3	512,280	256,140	14.8	4,604,324
Texas	18,600,528	9,149,390	1,156.4	12,501,058	6,176,003	548.1	1,264,449	570,515	133.8	7,136,819
Utah	1,854,605	1,258,418	133.1	2,333,900	1,564,310	87.4	190,825	129,487	28.8	1,076,602
Vermont	1,334,290	610,413	33.9	726,484	345,593	17.7	172,110	85,865	5.2	637,953
Virginia	7,545,084	3,764,699	255.8	2,385,237	1,190,554	67.0	2,178,394	1,031,904	48.1	1,066,943
Washington	4,882,817	2,538,963	114.3	2,752,950	1,440,116	30.0	1,649,892	741,242	16.2	1,080,604
West Virginia	2,220,412	1,460,013	71.3	1,465,862	757,011	38.1	1,805,740	899,168	43.2	2,263,331
Wisconsin	5,348,323	2,640,259	188.9	6,761,936	3,307,780	183.4	3,348,370	1,601,700	146.5	1,769,726
Wyoming	2,974,801	1,807,345	304.2	1,570,645	972,995	148.5	298,147	188,208	37.9	1,026,385
District of Columbia							297,500	148,750	2.0	338,750
Hawaii	1,287,547	637,043	23.3	808,350	395,200	12.7	892,407	427,968	14.3	1,058,510
Puerto Rico	703,366	347,080	14.2	1,632,798	811,330	33.4	171,477	80,925	2.3	482,540
TOTALS	249,850,646	128,220,989	10,057.3	203,865,882	101,483,578	6,458.4	71,893,793	35,458,204	2,945.9	133,629,011

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF JUNE 30, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 284,756	$ 139,712	24.3	$ 778,250	$ 383,750	32.7	$ 281,200	$ 57,000	0.3	$ 782,784
Arizona	506,576	329,737	42.3	263,967	173,408	29.3	15,912	11,475		355,372
Arkansas	108,487	101,817	9.9	398,390	395,311	44.9	181,860	181,422	32.6	440,945
California	1,918,647	1,065,263	117.9	1,055,708	542,287	44.7	86,988	50,895	3.7	758,464
Colorado	1,160,418	606,547	64.8	540,850	274,564	21.5	161,270	76,292	4.2	235,353
Connecticut	106,870	53,215	1.6	172,794	72,417	2.9				286,249
Delaware	22,730	11,365	5.3	80,840	40,420	17.5	73,930	36,965	7.8	231,250
Florida	20,122	10,061		762,533	380,450	26.3	227,500	109,850	11.4	374,744
Georgia	525,621	252,520	70.1	442,406	221,203	55.2	155,980	77,990	20.3	1,083,865
Idaho	497,893	222,141	57.2	246,595	140,187	11.2				295,511
Illinois	2,011,457	977,930	167.4	1,501,632	696,816	83.8	380,000	190,000	22.8	770,576
Indiana	759,194	318,067	80.5	941,170	470,585	80.5	386,177	173,298	22.5	644,375
Iowa							47,751	22,015	35.4	1,657,792
Kansas	251,257	125,622	29.9	47,588	23,794	11.7	408,515	204,257	21.8	1,353,173
Kentucky	798,767	243,871	106.1	1,076,701	290,910	66.2	730,604	239,763	67.0	317,903
Louisiana	241,327	107,635	20.0	675,599	290,180	54.4	307,416	143,120	26.6	398,713
Maine	423,420	205,994	25.7	218,250	109,395	11.5	224,590	111,265	14.4	37,761
Maryland				188,974	94,487	15.1	142,000	52,355	10.2	388,839
Massachusetts	126,988	63,055	1.8	300,011	148,856	6.4	288,754	142,443	6.0	498,369
Michigan	512,333	251,636	37.4	1,165,104	580,052	68.8	404,300	188,450	42.4	967,350
Minnesota	273,069	126,700	42.2	702,410	349,161	61.1	213,446	106,723	16.8	1,202,621
Mississippi				399,262	199,631	31.8	305,500	152,650	29.5	798,585
Missouri	523,807	250,970	72.8	705,966	342,573	76.6	515,824	218,755	87.3	701,338
Montana	14,071	7,865		452,945	256,840	37.5	461,668	261,854	38.4	813,334
Nebraska	617,650	292,326	101.3	743,466	362,777	143.8	426,966	202,489	67.0	446,867
Nevada	427,436	345,390	68.8	120,169	104,184	15.5	51,737	44,685	9.5	192,987
New Hampshire	206,978	102,285	6.0	60,759	29,708	2.4				188,007
New Jersey	171,820	79,020	2.5	349,350	172,625	10.0	155,240	76,735	9.8	542,598
New Mexico	857,449	521,681	57.5	450,024	271,508	28.1				252,877
New York	2,400,190	1,161,481	167.9	1,807,100	903,550	99.1	464,000	172,900	13.0	851,452
North Carolina	769,492	383,616	89.3	1,253,454	622,055	111.9	162,570	75,110	17.5	349,602
North Dakota	108,510	56,615	26.8	115,030	61,606	8.3	42,770	22,907	8.2	875,949
Ohio	147,535	73,767	3.8	632,270	322,910	32.0	243,200	121,600	13.4	1,850,842
Oklahoma	394,585	205,059	42.1	82,986	44,156	.8	625,440	309,598	37.4	973,691
Oregon	471,113	274,000	63.2	509,332	306,332	55.0	261,327	153,035	20.6	269,990
Pennsylvania	1,903,132	900,337	133.5	2,121,525	1,042,981	116.3	185,900	92,950	7.6	714,676
Rhode Island	166,074	81,173	7.2	99,335	49,644	2.2				134,171
South Carolina	673,849	292,852	79.1	583,907	239,069	56.9	169,800	66,200	12.4	279,791
South Dakota	11,519	6,250					13,880	7,640		1,050,410
Tennessee	420,621	185,123	17.6	719,438	304,489	32.0				881,848
Texas	3,654,802	1,732,636	515.2	2,028,005	966,441	199.7	397,069	191,406	46.5	1,160,749
Utah	780,439	387,018	65.9	103,735	56,018	18.3	143,020	66,846	17.2	209,198
Vermont	232,410	106,201	13.8	90,306	45,153	4.0	109,100	53,400	3.1	77,967
Virginia	1,086,502	488,862	90.8	493,334	239,171	50.6	172,096	69,912	15.4	367,303
Washington	571,884	297,126	64.3	694,589	364,996	40.1	70,770	37,000	16.3	266,006
West Virginia	243,696	119,483	21.4	153,296	76,648	8.3				515,848
Wisconsin	682,683	326,289	28.9	846,640	419,575	29.5	312,992	135,550	7.0	693,622
Wyoming	416,758	254,565	59.0	356,182	220,069	20.2	333,324	209,724	31.6	88,192
District of Columbia										73,125
Hawaii				170,080	85,040	4.6	22,900	11,450	1.3	223,510
Puerto Rico	250,901	123,966	13.7	135,545	65,880	8.6	98,148	48,085	3.9	82,069
TOTALS	28,755,838	14,268,844	2,716.8	27,837,802	13,853,862	1,989.8	10,463,434	4,978,059	880.1	29,008,613

PUBLICATIONS of the PUBLIC ROADS ADMINISTRATION

(Formerly the *BUREAU OF PUBLIC ROADS*)

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Agency and as the Agency does not sell publications, please send no remittance to the Federal Works Agency.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1938. 10 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 25 cents.

No. 191MP . . Roadside Improvement. 10 cents.

No. 272MP . . Construction of Private Driveways. 10 cents.

No. 279MP . . Bibliography on Highway Lighting. 5 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

Federal Legislation and Rules and Regulations Relating to Highway Construction. 15 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. 60 cents.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.

No. 265T. . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP . . Bibliography on Highway Safety.

House Document No. 272 . . . Toll Roads and Free Roads.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration, (formerly the *Bureau of Public Roads*) classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF JUNE 30, 1939

STATE	Completed During Current Fiscal Year: Estimated Total Cost	Completed: Federal Aid	Completed: Number: Grade Crossings Eliminated by Separation or Relocation	Completed: Number: Grade Crossing Structures Reconstructed	Completed: Number: Grade Crossings Protected by Signals or Otherwise	Under Construction: Estimated Total Cost	Under Construction: Federal Aid	Under Construction: Number: Grade Crossings Eliminated by Separation or Relocation	Under Construction: Number: Grade Crossing Structures Reconstructed	Under Construction: Number: Grade Crossings Protected by Signals or Otherwise	Approved for Construction: Estimated Total Cost	Approved: Federal Aid	Approved: Number: Grade Crossings Eliminated by Separation or Relocation	Approved: Number: Grade Crossing Structures Reconstructed	Approved: Number: Grade Crossings Protected by Signals or Otherwise	Balance of Funds Available for Programmed Projects
Alabama	$ 278,490	$ 278,291	7		6	$1,203,662	$ 1,201,724	14	1		$ 62,800	$ 55,800	2		1	$ 842,733
Arizona	30,741	30,741				469,516	443,841	5								281,092
Arkansas	618,179	615,817	14			189,891	189,891	3			71,693	71,693	1		4	1,225,099
California	1,362,358	1,361,783	5	3		1,691,373	1,690,278	10			80,272	80,272	1			1,296,732
Colorado	93,136	89,748	2		6	489,958	489,958	4		1	69,722	65,962			24	893,860
Connecticut						18,930	12,665				171,920	166,540		1		832,360
Delaware	66,368	65,889			23	9,150	9,150	2			2,320	2,320			1	513,891
Florida	17,416	17,416	1			428,094	428,094	2			79,700	79,700	1		1	1,158,058
Georgia	28,650	28,650	1			427,280	427,280	7			132,120	132,120	1	2	9	2,306,620
Idaho	256,974	249,144	5			312,066	280,535	4								454,970
Illinois	563,280	563,280	4		22	2,830,545	2,773,545	18	3	49	583,450	544,450	2		18	2,354,151
Indiana	736,388	626,218	4	4	21	848,183	848,183	3	1	69	484,475	484,475	2	1	9	969,772
Iowa	1,047,880	1,011,085	13	2	20	314,382	276,106	6			487,414	456,600	8		89	1,369,238
Kansas	596,658	596,533	6		5	934,612	934,612	11			473,030	473,030	5		8	1,075,292
Kentucky	249,688	249,688	1		31	586,597	586,597	9	4		277,438	226,991	2	1		1,107,615
Louisiana	98,714	91,980	1			441,463	440,678	4			328,603	328,570	11			1,026,699
Maine	54,882	54,247	2		1	473,676	473,676	4	3		90,800	90,800	1			207,671
Maryland						72,188	72,188	1			258,200	161,407	1		14	993,901
Massachusetts	74,505	74,505		2		520,631	519,367	4	2							1,727,702
Michigan	957,084	915,797	8	1	45	822,526	822,526	6	2		110,950	110,950			20	2,085,059
Minnesota	50,605	50,331		1	3	1,042,247	1,025,806	3	6	6	358,889	357,569	5		5	1,537,428
Mississippi	356,600	356,600	4			603,614	603,614	8								934,587
Missouri	319,890	318,351	4	1		1,059,640	1,059,640	5	1		567,910	564,120	4	2		1,679,326
Montana	365,654	360,772	4			860,225	860,225	9			37,469	37,469	1			327,257
Nebraska	175,709	172,676	8			931,327	931,327	24			457,389	457,389	4		35	550,707
Nevada	246,425	245,178	1	3	6	151,935	151,935		1		46,951	46,951	1		11	112,509
New Hampshire	70,205	69,765		1	2	100,927	100,459	7		1	105,035	104,987	2			316,039
New Jersey	188,941	183,715	2	1	1	493,541	493,541	1	2	1	255,740	255,740	1	1		1,426,875
New Mexico	264,915	264,649	7	1		75,081	75,081	2			2,861	2,861			1	675,857
New York	1,147,444	1,142,543	5	4	3	2,006,512	2,001,112	4	10		913,951	673,500	6	1		4,288,723
North Carolina	419,861	419,861	3	4	8	1,146,500	1,111,400	7	4		423,615	423,615	5	1	36	990,495
North Dakota	546,750	545,687	1	1		676,052	627,650	10	1	7	345,860	345,860	3			369,188
Ohio						844,902	808,140	10	1		1,049,680	1,000,490	6	1	2	3,254,391
Oklahoma	73,654	63,672			5	298,080	264,080	2		42	217,400	217,400	3		8	2,191,397
Oregon	595,514	594,865	3	2		39,002	39,002				266,457	261,372	3			314,891
Pennsylvania	213,129	197,923	2			1,967,294	1,755,395	3	3		570,653	362,200	3		2	4,545,633
Rhode Island						438,791	438,791	1	3							152,459
South Carolina	110,771	110,321	11	1	4	623,703	569,187	8	2		143,479	143,479		1	35	959,865
South Dakota	138,949	138,316	4		9	278,800	278,800	3	2	2	61,260	61,260			15	1,110,539
Tennessee	54,421	54,421		1	6	651,609	651,609	2	2	1	185,940	185,940	1	1	4	1,373,250
Texas	1,040,905	1,038,613	17	3		2,690,625	2,659,822	22	3		554,259	525,030	5		6	2,208,513
Utah	109,143	108,908	2		4	39,638	39,638	2			336,619	336,619	1		127	217,372
Vermont	245,681	230,614	6	2	8	14,256	14,256			2	16,180	15,990			5	317,471
Virginia	511,925	510,852	17	3	7	608,282	519,282	8	1	14	241,327	235,327	1	2	3	912,147
Washington	803,854	788,804	9	4	3	277,733	276,324	3		3	117,743	117,743		2	9	502,865
West Virginia	249,681	245,981	1		5	370,941	355,181	7			18,800	18,800	1		1	964,852
Wisconsin	202,131	200,987	3			1,432,157	1,387,305	15		1	208,478	208,462		2	3	1,162,829
Wyoming	155,409	154,992		2	4	202,010	122,590	1	1		17,010	17,010			7	514,272
District of Columbia	30,215	30,215		1							283,544	250,000	1			128,186
Hawaii	50,098	48,630	1			181,790	181,790	4	1							360,830
Puerto Rico	61,900	61,550	2			394,352	392,150	9								426,676
TOTALS	15,931,770	15,630,604	191	48	258	33,586,289	32,716,026	297	60	199	11,569,406	10,758,863	95	19	513	57,549,944

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PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 20, No. 6 — August 1939

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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APPLICATION OF THE RESULTS OF RESEARCH TO THE STRUCTURAL DESIGN OF CONCRETE PAVEMENTS[a]

Reported by E. F. KELLEY, Chief, Division of Tests, Public Roads Administration

Shape of cross section of slab.—Two types of cross section of the pavement slab are in general use: the cross section of uniform thickness, and the cross section in which the edges of the slab are thicker than the central portion. An appreciable number of State highway departments use slabs of uniform thickness but the majority use the thickened-edge design.

Since the thickened-edge pavement design is used so extensively at the present time, the history of its development is of interest.

So far as is known, the thickened-edge section in essentially its present form was first utilized by the California Highway Commission, as an alternate to a section of uniform thickness, in the construction of concrete bases. In this design the edge depth of the slab was 2 inches greater than the interior depth, the slab thickness being reduced from the edge depth to the interior depth at a uniform rate in the outer 18 inches of pavement width. This alternate design is shown in the May 1, 1913, issue of the California Highway Bulletin and it is shown subsequently in the first and second biennial reports of the California Highway Commission (Dec. 31, 1918, and Dec. 31, 1920). In the biennial report for 1921–22 (Nov. 1, 1922) the thickened-edge cross section appears as a standard rather than an alternate design.

According to T. E. Stanton [4] the alternate thickened-edge section was officially adopted in November 1912, for base construction and was used for this purpose from time to time until 1921 after which it was made standard for all concrete pavement construction.

In 1920 Maricopa County, Ariz., undertook a very extensive paving program and on November 12 of that year construction was started on a contract involving 141 miles of concrete pavement, all with thickened edges (*35*).[b] The design provided for a uniform interior thickness of either 5 or 6 inches and an edge thickness 3 inches greater than the interior thickness. The edge thickness was reduced to the interior thickness at a uniform rate in a distance of 2 feet. Thus the section was identical with that which is used today by a number of States and was similar to that now used by a majority of the States. The stated purpose of the design was to "strengthen the edge and at the same time permit simple construction of the subgrade" and to secure "a paving slab with a more uniform resisting strength" (*36*).

The Pittsburg Test Road at Pittsburg, Calif., was built during the summer of 1921. Traffic tests were begun that year and were finally discontinued in July 1922. The test road contained one thickened-edge section, similar to the 9–6–9-inch section used previously in Maricopa County, and in the final report (*37*), issued January 1, 1923, this section was given the highest rating of any of the sections included in the investigation.

The sections of the Bates Road (*21*) that were built in 1920 and 1921 did not include any thickened-edge design. However, sections of this design were built in the fall of 1922 and were subjected to traffic tests during 1923. The results corroborated the earlier findings of the Pittsburg tests that thickening the edges of a relatively thin pavement slab greatly increases its resistance to concentrations of heavy wheel loads.

In general, two types of thickened-edge cross sections are used. In one, the upper and lower boundaries of the section are parabolic curves so arranged that the thickness gradually increases from a minimum at the center to a maximum at the edge, the edge thickness being from 2 to 3 inches greater than the center thickness. The second type, which is used by a majority of the State highway departments, is the same as that used originally by the California Highway Commission. The central portion of the slab is of uniform thickness and the edge thickness exceeds this by 2 to 3 inches. The edge section is a trapezoid, the edge thickening taking place at a uniform rate over the outer 2 to 4 feet of slab width. In the Arlington tests (*17*) it has been found that with this type of cross section the greatest uniformity of load stresses throughout the section may be obtained.

Another type of thickened-edge section that is used to a considerable extent is the lip-curb design. In this design a low curb of approximately wedge shape is formed along the edge of the slab. The base of the curb is generally about 12 inches wide and the height is about 3 inches. When such a curb is superimposed on a slab of uniform thickness the stress diagram for loads is very similar to that for slabs of the conventional thickened-edge type in which the edge thickening is on the underside of the slab (*17*). However, the lip-curb design is not used primarily to strengthen the slab edge but rather as a drainage measure to prevent erosion of the road shoulders by storm water.

EFFECT OF LOAD STRESSES ON SLAB DESIGN DISCUSSED

Use of stress analysis in design.—In introducing the discussion of the application of stress analysis to the design of pavement slabs it is well to emphasize that one of the basic assumptions of the Westergaard analyses, both for load stresses and temperature warping stresses, is that the thickness of the slab is uniform. The equations for edge stress and corner stress are not directly applicable to slabs of thickened-edge design.

With respect to interior stresses the situation is somewhat different. In the Arlington tests (*17*) it was found that in slabs of uniform thickness the critical stress under a load in the interior of the slab was practically

[4] Materials and Research Engineer, Division of Highways, California Department of Public Works.

[a] Because of its length, this report is presented in two issues of PUBLIC ROADS. The first installment appeared in the July issue.

[b] Italic figures in parenthesis refer to the bibliography, p. 102, of the preceding issue.

the same from the center of the slab to a point about $2\frac{1}{2}$ feet from the edge. A similar condition was found to exist, over an even greater portion of the slab width, in thickened-edge slabs in which the edge thickness was reduced to a uniform interior thickness in a short distance and at a uniform rate. Therefore, it appears appropriate to use the equation for interior load stress both for slabs of uniform thickness and for those with thickened edges since, in the latter case, the maximum interior stresses are not affected appreciably by the edge thickening. Although test data are not available considerations of similar character lead to the conclusion that it will be approximately correct to consider interior warping stresses in a slab of uniform thickness to be the same as in a thickened-edge slab in which the interior portion is of equal uniform thickness.

In applying stress analysis to the design of slabs of uniform thickness, curves similar to those of figure 9 may be used to determine the thickness required to resist load stresses. For example, assume that it is desired to determine the required thickness of a slab having a modulus of rupture of 700 pounds per square inch for load A, an 8,000-pound wheel equipped with high-pressure pneumatic tires. If the conservative working unit stress of 350 pounds per square inch is used, figure 9 shows that the required thicknesses for the interior, corner and edge are approximately 6.2 inches, 9 inches, and 8.6 inches, respectively. These figures indicate that if the allowable unit stress is to be limited to 350 pounds per square inch the slab should have a uniform thickness of 9 inches. However, the load stresses will not be equal in the several portions of the slab. The indicated stresses at the interior, corner, and edge of this 9-inch slab are approximately 190, 350, and 330 pounds per square inch, respectively. On the other hand, if a less conservative unit stress is used, say 400 pounds per square inch, then the required thickness of slab, as determined by the corner stress, is approximately 8.3 inches. In this case the computed load stresses at the interior, corner, and edge of the slab are approximately 220, 400, and 370 pounds per square inch, respectively.

In the Arlington tests *(17)* it has been found that the thickened-edge cross section gives the nearest approach to a design that is balanced for load stresses; that is, one in which the stresses in a cross section of the slab are approximately equal for all positions of the load. It has also been found that the section which most nearly accomplishes this is of uniform thickness in the interior and has an edge thickness about 1.67 times the interior thickness, the edge thickness being reduced to the interior thickness at a uniform rate over a distance of 2 to $2\frac{1}{2}$ feet.

At present, the only means of applying stress analysis to the design of thickened-edge slabs is to determine the interior thickness in the same manner as for slabs of uniform depth and to determine the edge thickness by the empirical relation between edge and center thickness that has been indicated by the Arlington tests.

On the basis of the same assumptions that have been made for the slabs of uniform thickness, the interior thickness required to resist load A in a thickened-edge slab is indicated to be approximately 6.2 inches if the allowable unit stress is 350 pounds per square inch and 5.7 inches if the allowable unit stress is 400 pounds per square inch. Since these dimensions are based on Westergaard's original analysis rather than on the modified analysis of interior stresses, it will be sufficiently accurate to use interior thicknesses of 6 inches and 5.5 inches, respectively.

Multiplying these figures by 1.67 gives an edge thickness of 10 inches for the first design and 9.2 inches for the second. The data obtained in the Arlington tests indicate that the load stresses in the edge and interior of the 10–6–10-inch cross section will be approximately balanced and equal to about 350 pounds per square inch and that the edge and interior load stresses in the 9.2–5.5–9.2-inch cross section will be approximately balanced and equal to about 400 pounds per square inch.

Permissible unit stresses.—Before discussing the design of pavement slabs to resist the combined stresses due to load and temperature warping it is desirable to consider the factors that should influence the selection of permissible maximum unit stresses. Most of these factors have been mentioned in the previous discussion.

As has been stated, consideration of the available data concerning the fatigue limit of concrete has led to the rather general practice of assuming about 50 percent of the ultimate flexural strength as a safe value of the unit stress to be used in designing pavements to resist wheel loads. In general the probable strength of paving concrete at ages greater than 28 days is not definitely known and therefore the design stress has usually been based on the 28-day strength. Since concrete of the character used in pavements may be expected to have a flexural strength at 28 days of from 600 to 700 pounds per square inch, the customary design stress has been of the order of 300 to 350 pounds per square inch.

FOR COMBINED STRESSES, ALLOWABLE STRESS MAY EXCEED 400 POUNDS PER SQUARE INCH

As applied to load stresses this practice is a conservative one and the considerations that lead to this conclusion are:

1. The possibility that the fatigue limit of concrete, for the loading conditions that obtain in pavements, is greater than 50 percent of the ultimate strength.
2. The possibility that the stresses in pavement slabs caused by impact forces are less than those caused by static loads of the same magnitude.
3. The fact that concrete increases in strength with age and the probability that by the time the pavement has been subjected to enough repetitions of stress due to maximum wheel loads to require consideration of the fatigue limit, the concrete will have attained a strength appreciably in excess of its strength at 28 days.

The numerous investigations that have been made indicate that the rate at which concrete increases in strength after the age of 28 days is a variable that depends on several factors. The averages of the results obtained in a number of these investigations give values of the moduli of rupture at the age of 1 year that exceed the average moduli at the age of 28 days by amounts ranging from about 20 to 45 percent. Since these are average figures it is apparent that under some conditions the 1-year strength will exceed the 28-day strength by less than 20 percent.

It must be recognized that, for a given concrete, the 1-year strength cannot be predicted with any certainty from test results obtained at 28 days. However, when all the factors are considered, it does not seem unreasonable to believe that in general there may be expected a minimum increase in strength between the ages of 28 days and 1 year of the general order of 20 percent.

If the practice of limiting load stresses to about 50 percent of the 28-day strength of the concrete is a conservative one, then the same practice would certainly be unduly conservative if applied to the design of slabs proportioned to resist the combined stresses due to load and temperature warping. The additional considerations that lead to this conclusion have been discussed previously and are:

4. The fact that vehicles having maximum wheel loads constitute a small percentage of the traffic on most roads. The occurrence of maximum stress due to load is therefore relatively infrequent and the occurrence of maximum load stress in combination with maximum warping stress is much less frequent. This is particularly true in those localities where the movement of heavy trucks is principally at night when the warping stresses that are of consequence are generally such that the combined stresses are less than the load stresses.

5. The fact that the unknown stresses due to moisture warping appear to reduce, rather than to increase, the maximum stresses due to temperature warping.

On the basis of present knowledge the five factors that have been mentioned cannot be definitely evaluated. However, when all of them are considered, it does not appear unreasonable to conclude that, when the design is based on combined stresses due to load and temperature, the safe allowable unit stress is in excess of 400 pounds per square inch and may be as high as 500 pounds per square inch.

Design of cross section for combined load and temperature-warping stresses.—A consideration of slab design on the basis of combined load and warping stresses leads to the conclusion that there must be either an increase in permissible unit stresses even beyond the limits that have been suggested or an acknowledgment that current practice with respect to joint spacing in nonreinforced concrete slabs is incorrect.

In the previous discussion it has been shown that, for the assumed conditions, a slab of 9-inch uniform thickness is required if the unit load stress is limited to 350 pounds per square inch and that the thickness should be about 8.3 inches if the unit load stress is limited to 400 pounds per square inch. The combined interior and edge stresses (from figures 15 and 16) in these same slabs are shown in table 14. It will be observed that the edge stresses are always greater than the interior stresses; that in a 30-foot slab the edge stresses are equal to or greater than 600 pounds per square inch; that in a 15-foot slab they exceed 500 pounds per square inch except when the slab

TABLE 14.—*Combined edge and interior stresses in slabs 10 feet wide and of uniform thickness* [1]

Depth of slab (inches)	Position	Length of slab: 30 feet, $k=100$	30 feet, $k=300$	15 feet, $k=100$	15 feet, $k=300$	10 feet, $k=100$	10 feet, $k=300$
		Lb per sq. in.	*Lb per sq. in.*	*Lb. per sq. in*	*Lb. per sq. in.*	*Lb. per sq in*	*Lb. per sq in*
9	Interior	570	550	380	480	250	320
	Edge	650	600	460	530	330	370
8.3	Interior	570	550	420	500	290	350
	Edge	660	600	510	560	380	410

[1] From figs. 15 and 16.

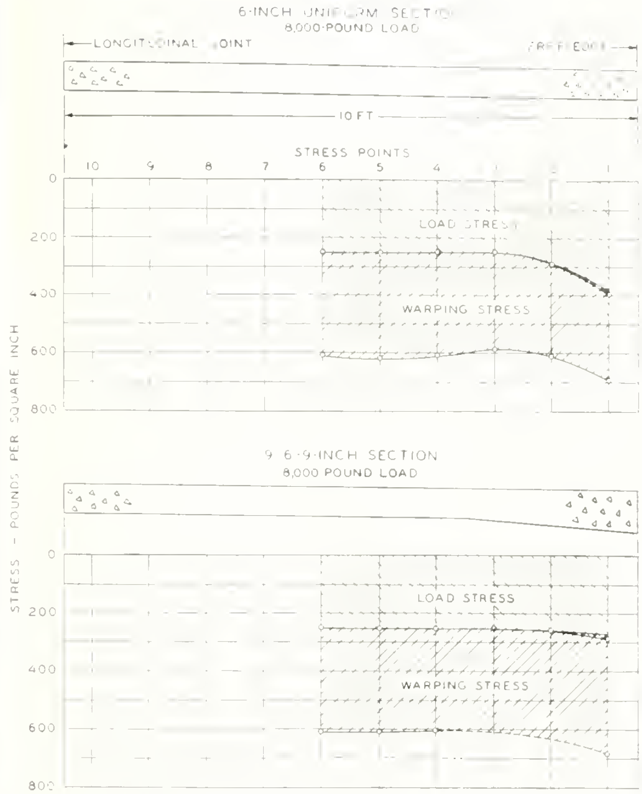

FIGURE 18.—MAXIMUM STRESS DIAGRAMS FOR COMBINED LOAD AND WARPING STRESSES FOR TWO TYPICAL CROSS SECTIONS; SLAB LENGTH 20 FEET; BASED ON DATA FROM THE ARLINGTON TESTS. DOUBLE HATCHED AREA SHOWS THE SMALL REDUCTION APPLIED TO THE OBSERVED LOAD STRESS VALUES TO CORRECT FOR THE EFFECT OF WARPING.

thickness is 9 inches and k—100; and that it is not until the slab length is reduced to 10 feet that the edge stresses are reduced to values equal to or less than about 400 pounds per square inch.

Since, as has been stated, only the interior stresses can be computed in a thickened-edge slab, it is necessary to depend on the data from the Arlington tests for information concerning balanced design of cross section for slabs with thickened edges. Figure 18 shows such data for a 6-inch uniform section and a 9–6–9-inch section, the load stresses in both being the stresses observed under a load of 8,000 pounds and the slab length being 20 feet.

ASSUMPTIONS NECESSARY IN APPLYING WESTERGAARD ANALYSIS TO THICKENED-EDGE SLABS

In the 6-inch uniform-thickness slab the observed load stresses of figure 18 are somewhat less than the computed stresses shown in figures 15 and 16. This is to be expected since the loads are not the same. However, the observed warping stresses of figure 18 are greater than the computed warping stresses of figures 15 and 16 even for a slab length of 30 feet. The net result is that the observed combined stresses in the 6-inch slab, 20 feet long, of figure 18 are of about the same order of magnitude as the average values, for $k=100$ and $k=300$, of the computed combined stresses in the 6-inch slab, 30 feet long, of figures 15 and 16. This is merely a demonstration of the fact that observed stresses are of the same order of magnitude as the maximum stresses obtained by theoretical analysis.

The real importance of figure 18 lies in the fact that, from the standpoint both of maximum stress and of uniformity of stress, there is no significant difference between the thickened-edge section and the section of uniform thickness. The maximum combined stresses are approximately the same for both slabs and the stress diagrams are of approximately the same shape. Therefore, it may be concluded that for long slabs (20 feet or more) there is no particular advantage, from the standpoint of combined stresses at the edge and interior, of thickening the slab edges. This conclusion does not apply to the slab corners where the load stresses are greatly reduced by edge thickening and where the combined stresses do not exceed the load stresses by any great amount. With respect to short slabs (length about 10 feet) a further analysis is necessary before a conclusion can be reached.

As has already been pointed out, the Westergaard analyses for load and warping stresses do not apply to slabs with thickened edges. Therefore there is no exact analytical method available on which to base a comparison of maximum combined stresses in short slabs of uniform thickness with those in slabs with thickened edges. However, by making certain assumptions, which the data from the Arlington tests appear to justify, it is possible to make an approximate computation of stresses in thickened-edge slabs for comparison with stresses, computed by the Westergaard analyses, in slabs of uniform thickness. These assumptions are as follows:

1. That the Westergaard analyses for load and warping stresses are applicable to the interior of thickened-edge slabs in which the interior portion of the slab is of uniform thickness.

2. That when the edge thickness of a thickened-edge slab is 1.67 times the interior thickness the maximum load stress at the edge is approximately the same as the maximum interior load stress.

These two assumptions have been discussed previously.

3. That the edge-warping stress in a thickened-edge slab is approximately the same as the edge-warping stress in a slab having a uniform thickness equal to the edge thickness of the thickened-edge slab.

In the Arlington tests ((*16*), table 4) it was found that the average observed warping stresses in the edges of slabs 20 feet long and of uniform thickness were not much greater in a 9-inch slab than in a 6-inch slab. This result is not in accord with theory and cannot be fully explained. However, the average edge-warping stresses in a 9–6–9-inch section exceeded the average edge stresses in a slab of 6-inch uniform thickness by about 30 percent.

By using the same assumptions that have been used previously in the computation of warping stresses, it may be shown that in a slab 20 feet long the edge-warping stresses in a 6-inch slab of uniform thickness are approximately 240 pounds per square inch both for $k=100$ and $k=300$ and that the edge stresses in a 9-inch slab of uniform thickness are approximately 290 pounds per square inch for $k=100$ and 360 pounds per square inch for $k=300$. The average value of 325 pounds per square inch for the 9-inch slab exceeds the average value of 240 pounds per square inch for the 6-inch slab by about 35 percent.

The average computed stress and the average observed stress in the 6-inch slab of uniform thickness are of about the same order of magnitude. The same is true of the computed stress in the 9-inch slab of uniform thickness as compared with the average observed stress in the 9–6–9-inch section. Also the ratio of the computed edge stress in a 9-inch slab to that in a 6-inch slab is approximately the same as the ratio of the observed stress in the edge of the 9–6–9-inch section to that in the edge of the 6-inch section. Therefore, it appears that it is a reasonable approximation to assume that in a thickened-edge slab the edge warping stress is of the same order of magnitude as in a uniform-thickness slab having the same edge depth.

Approximate interior and edge stresses, computed on the basis of these three assumptions, are shown in table 15 for three thickened-edge sections. Also shown in this table are the stresses in slabs of uniform thickness that are approximately comparable, with respect to maximum stress, with the thickened-edge designs. The three pairs of cross sections are designed for maximum combined stresses of approximately 500, 425, and 350 pounds per square inch.

TABLE 15.—*Combined stresses in thickened-edge slabs and slabs of uniform thickness; for slabs 10 feet wide and 10 feet long* [1]

	9–6–9-inch section				7.1-inch uniform section			
	Interior		Edge		Interior		Edge	
	$k=100$	$k=300$	$k=100$	$k=300$	$k=100$	$k=300$	$k=100$	$k=300$
	Lb. per sq. in.	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*
Load stress	370	320	430	370	280	250	410	350
Warping stress	110	200	50	130	90	180	70	150
Combined stress	480	520	480	500	370	430	480	500
Average	500		490		400		490	
	10–6.8–10-inch section				8-inch uniform section			
	Interior		Edge		Interior		Edge	
	$k=100$	$k=300$	$k=100$	$k=300$	$k=100$	$k=300$	$k=100$	$k=300$
	Lb. per sq. in.	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*
Load stress	300	260	370	320	230	200	340	290
Warping stress	90	180	50	110	70	170	60	140
Combined stress	390	440	420	430	300	370	400	430
Average	415		425		335		415	
	11.2–7.5–11.2-inch section				9-inch uniform section			
	Interior		Edge		Interior		Edge	
	$k=100$	$k=300$	$k=100$	$k=300$	$k=100$	$k=300$	$k=100$	$k=300$
	Lb. per sq. in.	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*
Load stress	240	210	310	270	190	170	280	240
Warping stress	70	170	40	90	60	150	50	130
Combined stress	310	380	350	360	250	320	330	370
Average	345		355		285		350	

[1] Assumptions with respect to load and other variables same as in figs. 15 and 16.

THICKENED-EDGE SLAB HAS NO MARKED SUPERIORITY OVER UNIFORM-THICKNESS SLAB

It will be observed that in all cases, for slabs of this length, the maximum combined stress is less when $k=100$ than when $k=300$. The difference is not great in any case and, since the value of the subgrade modulus cannot be predetermined, it is considered reasonable to average the stresses for the two subgrade conditions. On the basis of these average stresses the 9–6–9-inch thickened-edge section is comparable with the section of 7.1-inch uniform thickness; the 10–6.8–10-inch section may be compared with the 8-inch uniform section; and the 11.2–7.8–11.2-inch section may be compared with the 9-inch uniform section.

Since these pairs of slabs are comparable with respect to stress they may also be compared on the basis of probable cost. In making this comparison the depth of the thickened-edge slabs will be assumed to be increased at a uniform rate from the interior thickness to the edge thickness in the outer 2 feet of slab width. Then in a mile of 20-foot pavement the amount of concrete required by the slabs of uniform thickness exceeds that required by the comparable thickened-edge slabs by approximately 260, 290 and 280 cubic yards, respectively, for the slabs having uniform thicknesses of 7.1, 8, and 9 inches. When consideration is given to the additional expense involved in the construction of thickened-edge slabs, such as shaping the subgrade, shaping joint fillers, the more expensive side forms that are required, and the expense of strengthening the edges of transverse joints, it appears that there is no great difference in cost between the thickened-edge slab and the slab of uniform thickness.

In the above comparison of thickened-edge and uniform-thickness slabs no consideration has been given to stresses due to corner loading. There are two reasons for this, the first being the very practical one that there is no accurate method available for computing either the load stresses or the warping stresses in the corner of a thickened-edge slab.

The second reason is that in slabs of uniform thickness the corner stresses will not exceed the edge stresses except at transverse joints not provided with load-transfer devices and at transverse cracks in nonreinforced pavements. For the uniform-thickness slabs shown in table 15 the average maximum combined corner stresses (average for $k=100$ and $k=300$) are 530, 445, and 375 pounds per square inch, respectively, for the 7.1-, 8-, and 9-inch slabs. These corner stresses exceed the comparable edge stresses by a maximum of 40 pounds per square inch. As will be shown later, any of the common types of load-transfer devices used in transverse joints may be expected to reduce corner stresses by much greater amounts than this and therefore the neglect of corner stresses in slabs of uniform thickness will not result in any overstress at transverse cracks or joints in properly reinforced slabs in which the joints are provided with some means for load transfer. The overstresses that may occur at free transverse joints or at transverse cracks in nonreinforced pavements are so small as to be negligible.

While no figures can be produced to support the argument, it is believed that the same reasoning is applicable to thickened-edge slabs and that the designs of table 15 are truly comparable even though they cannot be compared on the basis of corner stresses.

On the basis of the foregoing discussion it is concluded that, when pavement slabs are designed for wheel loads such as are commonly permitted by regulatory laws and when the combined stresses due to load and temperature warping are kept within safe limits, the thickened-edge cross section has no marked advantage over the cross section of uniform thickness.

Edge strengthening at free transverse joints.—When a free transverse joint is introduced in a thickened-edge slab, or when a transverse crack develops in a thickened-edge slab that is not reinforced, a condition of relative weakness is created at the edges of the joint or crack. This is because the central portion of the joint or crack has the same thickness as the interior of the slab but is subjected to the higher stresses which are associated with edge loading.

In table 16 are shown the maximum combined stresses at the interior, the longitudinal edge and the edge of a free transverse joint in each of the three thickened-edge slabs that have already been shown in table 15. However, in table 16 the slabs are assumed to be 30 feet long instead of 10 feet as in table 15.

In table 15, for slabs 10 feet long, the maximum stresses were shown to be approximately 500 pounds per square inch for the 9–6–9-inch section, 425 pounds per square inch for the 10–6.8–10-inch section and 350 pounds per square inch for the 11.2–7.8–11.2-inch section. It will be noted at once, from table 16, that increasing the slab length from 10 to 30 feet has increased the stresses in the 9–6–9-inch section from a maximum of 500 pounds per square inch to 600 pounds per square inch in the interior and 760 pounds per square inch in the longitudinal edge. It will also be noted that the stresses at the interior and edge of the two heavier slabs are almost as large as in the 9–6–9-inch section. Thus, as has already been shown, the magnitude of combined interior and edge stresses in slabs as long as 30 feet is not greatly affected by variations in the depth of the slab.

TABLE 16.—*Combined stresses in thickened-edge slabs having a width of 10 feet and a length of 30 feet* [1]

	9–6–9-inch section					
	Interior		Edge		Edge of free transverse joint	
	$k=100$	$k=300$	$k=100$	$k=300$	$k=100$	$k=300$
	Lb. per sq. in.	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*
Load stress	370	320	430	370	530	440
Warping stress	250	260	370	350	90	170
Combined stress	620	580	800	720	620	610
Average	600		760		615	
	10–6.8–10-inch section					
	Interior		Edge		Edge of free transverse joint	
	$k=100$	$k=300$	$k=100$	$k=300$	$k=100$	$k=300$
	Lb. per sq. in.	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*
Load stress	300	260	370	320	440	370
Warping stress	290	300	400	400	80	160
Combined stress	590	560	770	720	520	530
Average	575		745		525	
	11.2–7.8–11.2-inch section					
	Interior		Edge		Edge of free transverse joint	
	$k=100$	$k=300$	$k=100$	$k=300$	$k=100$	$k=300$
	Lb. per sq. in.	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*
Load stress	240	210	310	270	360	300
Warping stress	330	330	440	450	60	140
Combined stress	570	540	750	720	420	440
Average	555		735		430	

[1] Assumptions with respect to load and other variables same as in figs. 15, 16 and 17.

EDGES OF TRANSVERSE JOINTS MUST BE STRENGTHENED

Table 16 shows that in these 30-foot slabs the stress at the edge of a free transverse joint is approximately equal to or less than the stress in the interior of the slab. This condition might be considered as evidence that there is no necessity for strengthening the edges of transverse joints in thickened-edge pavements. However, the figures presented indicate that combined edge stresses of the order of 750 pounds per square inch may be expected in slabs of this length and it may be anticipated that stresses of this magnitude will eventually result in the formation of transverse cracks. When these cracks develop, the slab length will be reduced and the combined stresses at the interior and edge will also be reduced but the reduction in slab length will have no effect on the combined stress at the edge of free transverse joints. The joint stresses are then likely to be much higher than the edge and interior stresses and should be reduced, by edge strengthening, to safe values and to values which are not excessive as compared with the stresses in other portions of the slab.

If the initial design of the slab is to be balanced so that the stresses are approximately the same in all portions of the slab, then it is necessary to reduce the slab length to about 10 feet. In order to have a balanced design it will then be necessary to strengthen the joint edges sufficiently to reduce the joint stresses from 615, 525 and 430 pounds per square inch, as shown in table 16, to 500, 425 and 350 pounds per square inch, respectively, the maximum values of the edge and interior stresses shown in table 15.

Thus far the discussion has been confined to combined stresses due to load and temperature but the question of the edge strengthening at joints should also involve a consideration of load stresses only, since maximum load stresses occur much more frequently than do maximum combined stresses due to load and temperature. If the average load stresses at transverse joints of table 16 (average for $k=100$ and $k=300$) are compared with the average interior load stresses in table 15 it is found that the load stresses at the edges of free transverse joints exceed the interior load stresses by 105 to 140 pounds per square inch. Thus edge strengthening at the transverse joints is required if the stresses due to load are not to be more severe at joints than at the interior of the slab.

Still another reason for strengthening the edges of transverse joints is the fact, already pointed out, that wheel loads may be expected to develop higher impact reactions in the vicinity of transverse joints than in other portions of the slab.

The discussion that has been presented indicates quite definitely that, when the interior of a thickened-edge slab is designed to resist either load stresses or combined stresses due to load and temperature, a condition of relative weakness will be created at the transverse joints if the edges of the joints are not strengthened.

When pavement slabs of uniform thickness are adequately designed to resist edge stresses, no edge strengthening at transverse joints or cracks is necessary. When the thickened-edge design is used the edges of joints may be strengthened by methods which will be described later. But, when a transverse crack develops in a thickened-edge pavement that is not reinforced there is developed a condition of weakness for which there is no remedy and which may eventually lead to complete failure. This possibility may be avoided by proper design and there are two methods of design available. The first, applicable to nonreinforced pavements, requires the use of a joint spacing of the general order of 10 feet. It is probable that the expense of edge strengthening for so many joints as would be required by this design would lead to the abandonment of the thickened-edge section or the adoption of the second, or alternate, method.

The second method is to use properly designed steel reinforcement. Reinforced slabs can safely be made of any length consistent with the economical use of reinforcement suitably designed to prevent the formation of open cracks. If the design of the reinforcement is such that the stresses to which it is subjected cause either rupture or excessive elongation at the cracks which inevitably will develop, then the edge weakness at cracks will not have been remedied. However, if the reinforcement is adequate to hold the edges of the fractured slab in close contact, the crack will tend to act as a hinged joint thereby relieving the warping stresses at the edge and interior; and the interlocking of the irregular surfaces of fracture may be expected to furnish the required edge strengthening along the crack.

Longitudinal and lateral expansion and contraction.—The preceding discussion of stresses due to changes in temperature and moisture content has dealt entirely with warping stresses due to a temperature or moisture gradient between the top and bottom of the slab. It is now necessary to consider general increases or decreases in temperature and moisture that are effective throughout the depth of the slab and which tend to cause corresponding changes in its horizontal dimensions.

If the slab were perfectly free to move, changes in volume would take place without restraint and no stress would be created. However, the subgrade offers considerable resistance to the horizontal movement of the slab. If the slab is attempting to contract as the result of a drop in temperature or a lowering of the moisture content, the subgrade resistance creates tensile stress. If the slab is attempting to expand, the subgrade resistance creates compressive stress. The magnitude of the tensile stress is dependent on the length of slab that is free to contract and the magnitude of the compressive stress is dependent on the distance between free expansion joints.

It has been amply demonstrated by experience that, in pavements not provided with transverse joints, both tensile and compressive failures develop. The tensile failures are evidenced by transverse cracking and the compressive failures by "blow-ups".

COMPRESSIVE FAILURES DUE PRIMARILY TO COLUMN ACTION

It is apparent from the discussion of temperature warping that many of the transverse cracks that develop in long slabs are due to warping stress but theoretical analysis indicates definitely that some of them are due to contraction of the slab as a whole. For example, assume a pavement slab of such length that the subgrade resistance is sufficient to prevent any movement of the slab in the vicinity of its mid-length. If the concrete has a modulus of elasticity of 5,000,000 pounds per square inch and a thermal coefficient of 0.000005 per degree Fahrenheit, a drop in temperature of only 20° F. will create a tensile stress of 500 pounds per square inch, which exceeds by a considerable amount the probable tensile strength of the concrete.

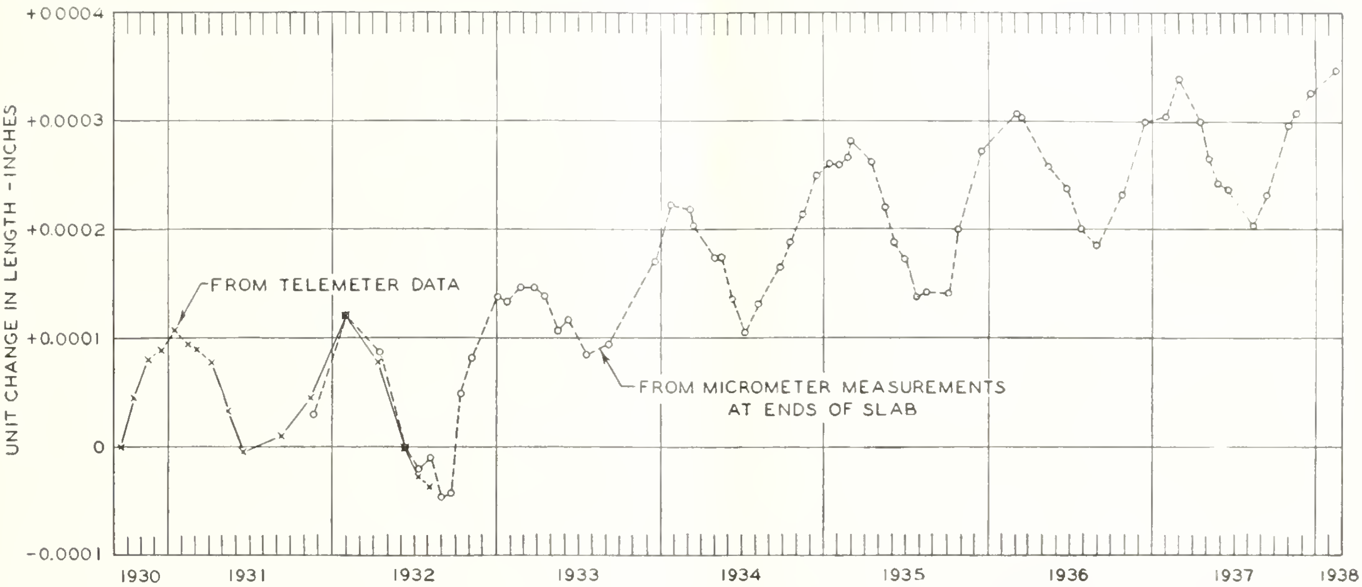

FIGURE 19.—ANNUAL VARIATION IN PAVEMENT LENGTH CAUSED BY CHANGES IN MOISTURE CONTENT.

In the same slab a rise in temperature as great as 100° F. would create a compressive stress of only 2,500 pounds per square inch. A direct compressive stress of this magnitude should cause no distress in concrete of the quality commonly used in pavements. Also, such a large change in temperature generally can be expected to take place only over a relatively long period of time and therefore it may be expected that the indicated stress will be reduced somewhat by the plastic flow of the concrete. However, the slab undoubtedly acts to some extent as a long column and its ultimate strength as a column is considerably less than its compressive strength as measured by tests on short specimens. It is believed that compressive failures are due primarily to column action rather than to direct compression and observations of pavement failures support this conclusion. Also, to the compressive stress caused by a rise in temperature must be added the unknown stresses caused by the slow "growth" of the slab that takes place over long periods of time. This growth, and the fact that changes in moisture content probably do not increase compressive stresses, will be discussed later.

Neither the magnitude of the compressive stress that may be developed in a long slab nor the stress to which it may safely be subjected are known. It is probable that both are variables depending on conditions. However, it is definitely known from experience that compressive failures may be expected in long slabs. The fact that these usually do not occur until the pavement is several years old is an indication that the slow growth of the concrete with age is a contributing factor.

All the facts point definitely to the conclusion that, if failures are to be avoided, joints must be provided in concrete pavements to reduce to safe values the stresses due to expansion and contraction.

Spacing and width of expansion joints.—Theoretically, the spacing of expansion joints should be dependent on the allowable compressive stress in the concrete and on the maximum compressive stress created by the expansion of the slab. However, in practice the maximum spacing of joints is influenced primarily by the desirability of using a rather narrow joint opening. The practice of the various States is not uniform but, in general, expansion joints are spaced at intervals not greater than 100 feet and, for this spacing, joint openings are usually either ¾ inch or 1 inch wide.

Open transverse cracks may be expected to develop in nonreinforced slabs of this length and usually it is not considered economical to provide sufficient longitudinal reinforcement to prevent the formation of such cracks. Therefore, it is customary to introduce contraction joints at intervals between the expansion joints and it is convenient to make the spacing of expansion joints some multiple of the spacing of contraction joints.

In general it may be assumed that concrete pavements will be built during periods when the temperature is not more than 60° F. below the maximum temperature to be expected. In concrete of the character that has been assumed, a rise in temperature of 60° F. will cause an increase of approximately ⅜ inch in the length of a slab 100 feet long. In a slab of this length the expansion will be restrained to some extent by the subgrade resistance and cause some reduction, probably negligible, in this computed movement of the slab ends. Also after the concrete has been placed there will be some reduction in slab length as a result of contraction due to moisture loss. Thus it might be concluded that a ¾-inch joint opening would be more than ample.

However, there are two other factors that have an influence on the required joint opening. If intermediate contraction joints, or open cracks that may have developed, are not maintained in such a manner as to exclude all foreign material, the joints or cracks will gradually become filled with incompressible soil material. This action operates to increase the length of the slab and results in a reduction in the effective width of the expansion joint.

SUBGRADE RESISTANCE AFFECTS SPACING OF CONTRACTION JOINTS

Also, in arriving at a decision as to the required width of joint opening, consideration should be given to the gradual increase in length, or "growth," of the slab that takes place over long periods of time. Figure 19 presents data obtained in the Arlington tests showing the annual variations in pavement length caused by changes other than temperature. The data cover the period from September 1930 to February 1938. The graph indicates that there is an annual cyclic variation in length caused by variations in moisture content and that the pavement slabs were longest (for a given temperature) during the winter and shortest during the summer. This would indicate that, in climates similar to that of Washington, D. C., the compressive stresses developed by high summer temperatures may be relieved somewhat by contraction due to loss of moisture and that the same action may result in some slight reduction in the width of joint opening theoretically required to provide for increase in slab length due to increase in temperature.

However, figure 19 also shows that, since the summer of 1932, there has been a definite, progressive yearly increase in the length of the pavement. In the summer of 1937 the length of the pavement exceeded its length during the summer of 1931 by approximately 0.0002 inch per inch. It is not known how long this growth will continue or at what rate. Neither is it known if the same degree of growth would take place in other concrete under other climatic conditions. However, it is known that all concrete has a tendency to increase permanently in volume in the presence of moisture.

The permanent increase in slab length that has taken place in the Arlington tests in a period of 6 years amounts to approximately ¼ inch per 100 feet. The sum of this increased length and the computed expansion due to a temperature rise of 60° F. equals approximately ⅝ inch. This indicates rather definitely that a provision for expansion of ¾ inch per 100 feet is not excessive. It may even prove to be inadequate, particularly in view of the fact that a certain portion of the joint width is frequently occupied by incompressible joint filler.

Subgrade resistance.—The required spacing of transverse contraction joints in concrete pavements is dependent on the allowable tensile stress in the pavement and on the subgrade resistance which prevents its free contraction.

Included in the investigations by the Bureau of Public Roads have been three studies undertaken to determine the probable magnitude of the resistance offered by the subgrade to the horizontal movement of a concrete slab (*16, 38, 39*). In all these investigations slabs of concrete, cast on prepared subgrades of various characteristics, were displaced horizontally over small distances and the relation between the horizontal force required to produce movement and the weight of the slab was determined. This relation is known as the coefficient of subgrade resistance. Of necessity the slabs used in all of these tests were of relatively small size as compared with pavement slabs. These studies have revealed the following facts:

1. The coefficient of subgrade resistance is not a constant but increases with increasing displacement of the slab until a maximum value is reached. This maximum corresponds to the force required to produce free sliding.
2. The resistance to movement on a very wet subgrade, which is not frozen, is less than on a dry or damp subgrade.
3. The resistance is much greater on a frozen subgrade than on one which is not frozen. This fact is probably not of great importance, at least in climates similar to that of Washington, D. C. The temperature observations made in connection with the Arlington tests showed relatively small changes in average concrete temperature during periods of cold weather. This suggests that the movements due to contraction during cold periods may be so small that the stresses in the pavement will not be increased to an important degree by a frozen subgrade.
4. For each of the first few successive applications of a given horizontal force, in repeated tests on the same slab, there is a reduction in the coefficient of resistance until an approximately constant value is reached. This indicates that the subgrade resistance may be greater for the first movement of a newly constructed pavement than it is at later ages when the concrete has expanded and contracted a number of times.
5. When a slab is subjected to a horizontal thrusting force a part of the resistance developed is due to the elastic or semielastic action of the soil. If the thrusting force is removed, even after a considerable period of time, there is a partial return of the slab to its original position.
6. The thrusting force is not directly proportional to the weight of the slab and it appears that this is due to the resistance to deformation of the subgrade. It has been concluded (*16*) that the subgrade resistance is composed of two elements: A resistance caused by the deformation of the soil; and a resistance that approximates that of simple sliding friction. While data are available only for the one soil involved in the Arlington tests, it seems probable that the relative magnitude of the two components of the subgrade resistance will vary with different subgrade soils.

LIMITED DATA AVAILABLE ON RELATION BETWEEN THRUSTING FORCE AND SLAB DISPLACEMENT

In tables 17 and 18 are given values of the coefficient of subgrade resistance obtained in the first investigation by the Bureau of Public Roads (*38*) and in the Arlington tests (*16*), respectively. Both tables show the increase in the coefficient of resistance with an increase in the displacement of the slab. In addition, table 18 shows that, because of the resistance of the subgrade to deformation, the coefficient is not directly proportional to the weight of the slab but increases as the thickness of slab decreases.

TABLE 17.—*Coefficients of subgrade resistance for concrete slabs of 6-inch thickness on various kinds of bases in damp but firm condition* [1]

Kind of base	Coefficients of resistance for displacements of—		
	0.001 inch	0.01 inch	0.05 inch
Level clay	0.55	1.30	2.07
Uneven clay	.57	1.29	2.07
Loam	.34	1.18	2.07
Level sand	.69	1.24	1.38
¾-inch gravel	.52	1.10	1.26
¾-inch crushed stone	.44	.92	1.09
3-inch crushed stone	1.84	1.78	2.18

[1] Data from table 1, p. 20, PUBLIC ROADS, July 1924.

TABLE 18.—*Coefficients of subgrade resistance for concrete slabs of different thicknesses on a silt loam soil (class A-4)* [1]

Slab thickness (inches)	Coefficients of resistance for displacements of—					
	0.01 inch	0.02 inch	0.03 inch	0.04 inch	0.07 inch	0.10 inch [2]
8	0.8	1.2	1.5	1.8	2.1	2.2
6	.9	1.3	1.6	2.0	2.4	2.5
4	1.1	1.5	1.8	2.2	2.8	3.1
2	1.3	1.7	2.1	2.5	3.3	3.5

[1] Data from table 3, PUBLIC ROADS, November 1935

[2] Displacement of 0.10 inch corresponds to maximum horizontal resisting force that could be developed.

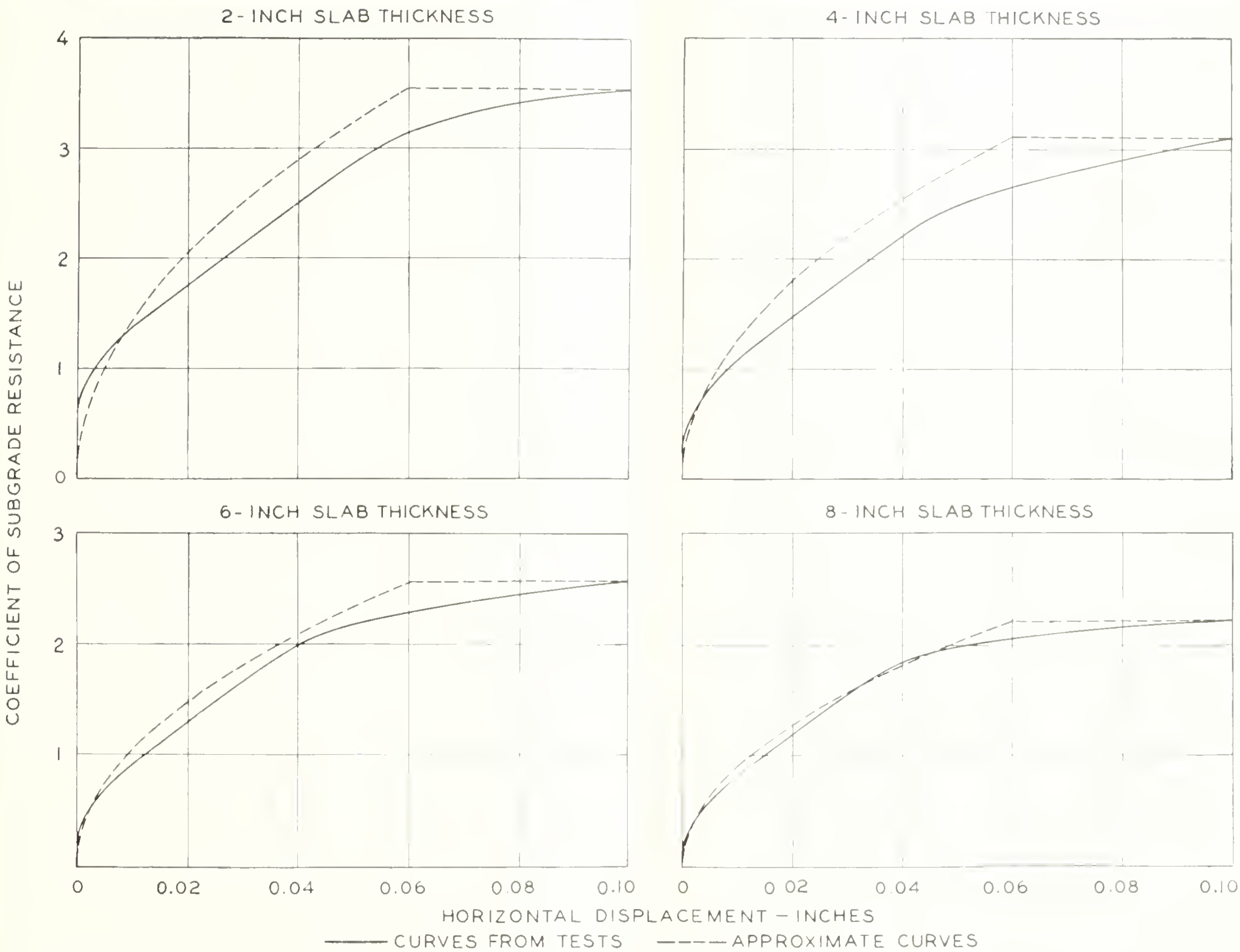

FIGURE 20.—COMPARISON OF ACTUAL AND APPROXIMATE CURVES SHOWING RELATION BETWEEN COEFFICIENT OF SUBGRADE RESISTANCE AND HORIZONTAL DISPLACEMENT.

Stresses due to contraction.[5]—When a pavement slab contracts, the total forces developed by the resistance of a uniform subgrade will be equal and opposite in each half of the slab and theoretically no movement will take place at the center line. The total displacement due to contraction then increases at a nearly uniform rate from zero at the center line to a maximum at the end of the slab. Since the subgrade resistance varies with the displacement it is apparent that an accurate analysis of slab stress should take account of the subgrade resistance corresponding to the total displacement of each increment of slab length.

Utilizing the data obtained in the tests with the 6-inch slab of table 18, such a method of analysis is illustrated in the report of the Arlington tests (*16*), the stresses being those due to an assumed change in temperature of 100° F. As will be shown later this temperature change is excessive when applied to the computation of stresses in slabs provided with joints at reasonable intervals but the principles of the analysis are correct.

An exact analysis of this character requires the use of test data showing the relation between thrusting force and slab displacement and therefore is applicable only when such data are available. However, if it may be assumed that the general shape of the force-displacement curve will be similar under all conditions, then a simple approximate method of analysis may be developed for general use. The available data are limited and it is recognized that the relation between thrusting force and slab displacement may be different at different locations, depending largely on the character of the subgrade. However, the approximate method that will be presented gives results that appear to be reasonable and it is believed that its use will not involve any serious errors.

The solid curves of figure 20 show the force-displacement relation, as developed in the Arlington tests, for slabs of four thicknesses. The curves are the same as those of figure 20, PUBLIC ROADS, November 1935. The dotted lines represent an approximation of the actual force-displacement relation. The curved portion of each dotted line is a parabola, with vertex at the origin, passing through the point having an ordinate equal to the maximum coefficient of subgrade resistance which, in these tests, was developed at a displacement of approximately 0.10 inch, and having an abscissa equal to a displacement of 0.06 inch. In comparison with these test results the approximate force-

[5] The original manuscript of this section on stresses due to contraction has been completely rewritten as a result of suggestions made by Mr. R. D. Bradbury, to whom credit is due for the development of the method for computing the average value of the coefficient of subgrade resistance.

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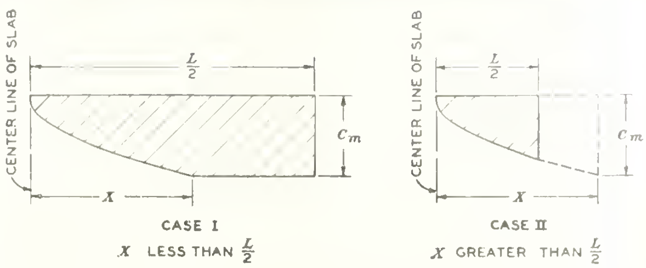

Figure 21.—Approximate Variation in Value of the Coefficient of Subgrade Resistance From the Center to the End of a Pavement Slab.

displacement curves are conservative since, in general, they give values of the subgrade coefficient that are greater than the test values.

At a given distance from the center of a pavement slab, a given drop in temperature will result in a certain movement due to contraction and, theoretically, the subgrade resistance which is developed should be that corresponding to this movement. At the center of the slab the movement and the corresponding resistance are zero. As the distance from the center of the slab is gradually increased the contraction movement, due to a given drop in temperature, and the corresponding coefficient of subgrade resistance are also gradually increased until, if the slab is long enough, a point is reached at which the subgrade coefficient reaches a maximum and constant value. An average value of this variable subgrade coefficient may be determined and, for the computation of the maximum contraction stress at the center of the slab, this average value may be considered as applied over the entire length of slab.

MAXIMUM CONTRACTION STRESSES OCCUR DURING A PERIOD OF CONTINUOUSLY FALLING TEMPERATURE

On the assumption that the force-displacement relation is as shown by the dotted lines of figure 20, figure 21 shows the variation in the value of the coefficient of subgrade resistance along the length of a pavement slab. In this figure, X equals the distance from the center of the slab to the point where the transition from the parabolic variation to a constant value occurs. Case I is that in which the distance X is less than half the slab length and Case II is that in which X is greater than half the slab length.

The distance X, in feet, is determined by the equation

$$X=\frac{D}{12Te} \qquad (21)$$

in which

D=assumed minimum displacement, in inches, at which the maximum value of the coefficient of subgrade resistance is developed;

T=the temperature drop, in degrees F.;

e=thermal coefficient of contraction per degree.

D has already been assumed as 0.06 inch and if, as in previous examples, e is assumed equal to 0.000005, then

$$X=\frac{1{,}000}{T} \text{ (feet)} \qquad (22)$$

The equations for the average value of the coefficient of subgrade resistance are as follows:

Case I, X less than $\frac{L}{2}$

$$C_a=C_m\left(1-\frac{2X}{3L}\right) \qquad (23)$$

Case II, X greater than $\frac{L}{2}$

$$C_a=\frac{2C_m}{3}\sqrt{\frac{L}{2X}} \qquad (24)$$

in which

C_a=average value of the coefficient of subgrade resistance;

C_m=maximum value of the coefficient of subgrade resistance;

L=free length of slab, in feet, for computation of longitudinal forces and free width of slab, in feet, for computation of transverse forces.

With respect to the type of resistance to slab movement that is offered by the subgrade, it appears that subgrades may be divided into two general classes: those which have some elasticity, such as the subgrades involved in the Arlington tests, and those which have no elasticity as, for example, sand.

When a pavement slab on a partially elastic subgrade contracts as a result of a decrease in temperature, the tensile stress that is created may be considered as being developed in three successive increments. The first increment of stress is due to the resistance of the subgrade to elastic deformation, the second is due to the resistance to inelastic deformation, and the third is due to the resistance developed by sliding friction. If the slab displacement is small, only the resistance to elastic deformation may be developed, but large displacements will develop all three increments of stress. If the subgrade has no elasticity the stress developed is due only to the resistance to inelastic deformation and to frictional resistance.

When the temperature has reached a minimum the slab ceases to shorten and, since the movement ceases, the stress due to inelastic deformation and frictional resistance is immediately reduced to zero. In the case of the semielastic subgrade, that portion of the stress caused by resistance to elastic deformation remains in the slab until it is relieved by expansion due to an increase in temperature. As the temperature gradually increases from the minimum, the tensile stress created by the resistance to elastic deformation is gradually reduced and is completely relieved when the temperature reaches its initial level.

If the temperature does not return to its initial upper level, a residual tensile stress remains in the slab. The total stress in the slab, after another drop in temperature equal to that which occurred during the first cycle, may therefore be somewhat greater than that which was developed during the first cycle. Also, if the slab length is such that large changes in temperature produce small displacements, the resistance of the subgrade to elastic deformation may not be exceeded until there have occurred several cycles of temperature change during which the level of the minimum temperature has decreased.

It is apparent from this discussion that the maximum contraction stress in a pavement slab is not dependent on the annual change in temperature. Rather it is dependent on the subgrade resistance that can be developed during a single period of continuously falling temperature or, at most, during a relatively few cycles of temperature change in which the general level of the minimum temperatures is decreasing. Since many subgrade soils are not elastic and since the degree of elasticity that has been observed is rather small, it is believed that the changes in slab temperature that take place during successive cycles are of considerably less importance than the drop in slab temperature which may take place during any one day.

MAXIMUM DAILY RANGE IN AVERAGE SLAB TEMPERATURE ASSUMED AS 40° F.

The daily change in average slab temperature is dependent on the daily change in air temperature and the relation between the two is influenced by the season of the year and by the particular climatic conditions that happen to obtain when the comparison is made.

In the Arlington tests it was found that, in general, the maximum daily change in the average temperature of the slab was considerably less during the cold months of the year than during the warm months. However, there were numerous occasions during the winter when the daily change in air temperature was as great as during the summer. Therefore, the lower daily change in slab temperature during the winter may be attributed to a lesser absorption of solar heat, since during this period the rays of the sun strike the pavement at a relatively low angle of incidence. This is a matter of importance when the attempt is made, on the basis of daily changes in air temperature, to establish for design purposes the maximum daily change in slab temperature.

Unpublished data obtained in the Arlington tests during the period from April to September, inclusive, on a number of selected days when the change in average slab temperature was relatively high, show that the daily change in the average temperature of a 6-inch slab was generally less than the daily change in air temperature. However, in a number of cases the difference was so small as to be negligible and in a few cases the change in slab temperature exceeded the change in air temperature by as much as 5° F. The maximum observed daily change in the average temperature of a 6-inch slab was 32° F. on a day when the change in air temperature was 47° F. Very little information is available concerning the relation between slab temperature and air temperature in slabs having a thickness greater than 6 inches. Apparently the daily change in the average temperature of thick slabs is always less than in thin ones and the few data that are available from the Arlington tests indicate that the daily range in average temperature in a 9-inch slab is about 80 percent of that in a 6-inch slab.

In table 19 are given the maximum ranges in air temperature that occurred during the years 1936 to 1938, inclusive, at selected cities in the United States. Excluding a few extremely high values that were observed during the winter months, it will be seen that a maximum daily range in air temperature of the order of 45° F. is of rather general occurrence except along the Pacific Coast, in some of the southern States, and in certain areas in the northeastern States. In the light of these data and the preceding discussion it is concluded that it will be conservative to assume, for general use in the United States, a maximum daily range in average slab temperature of 40° F. and that the climatic conditions in certain areas justify the use of a somewhat lower value.

TABLE 19.—*Greatest daily range in air temperature for selected cities, 1936 to 1938, inclusive* [1]

City	Greatest daily temperature range for year						
	1936		1937		1938		Average
	°F.	*Month*	*°F.*	*Month*	*°F.*	*Month*	*°F.*
Seattle, Wash	31	Aug	34	Sept	33	Feb.	33
Portland, Oreg	37	Apr	36	May	36	Sept	36
San Francisco, Calif	32	Sept.-Oct.	34	Sept	35	Sept	34
Los Angeles, Calif	39	Oct	32	Oct	31	Aug	34
Reno, Nev	44	July-Aug	45	July	42	Sept Oct	44
Phoenix, Ariz	42	June-Oct-Nov.	43	May	43	Apr	43
Salt Lake City, Utah	46	May	44	Aug.-Sept	41	July	44
Helena, Mont	58	Dec	45	Feb	48	Dec	50
Bismarck, N. Dak	46	Oct	52	Jan	45	Aug	48
Denver, Colo	60	Feb	41	Apr	48	Jan	50
Albuquerque, N. Mex	47	Apr	45	Apr	49	Mar	47
Omaha, Nebr	46	Apr	44	Oct	46	Feb.-Oct	45
St. Louis, Mo	48	Jan	44	Jan	45	Jan	46
Chicago, Ill	47	Apr	39	Apr	39	Apr	42
Indianapolis, Ind	49	Jan	39	Feb	35	Mar	41
Washington, D. C	44	Mar	45	Apr	37	May	42
Rochester, N. Y	37	May-June	35	Jan	36	Feb	36
Portland, Maine	42	May	36	May	40	Apr	39
Little Rock, Ark	40	Apr	38	Jan	37	Jan	38
Atlanta, Ga	44	Apr	42	Sept	37	Mar	41
Houston, Tex	41	Nov	40	Jan	33	Jan	38
Mobile, Ala	31	Feb.-July-Oct.-Dec.	32	Mar	34	Nov	32
Miami, Fla	25	Mar.-Nov	27	Dec	27	Oct	26

[1] Data obtained from the U. S. Weather Bureau.

Having established a basis for computing the value of the average coefficient of subgrade resistance, an analysis may be made to determine the maximum contraction stress in a pavement slab.

For a slab without reinforcement the maximum contraction stress is given by the equation

$$\sigma_s = \frac{WLC_a}{24h} \qquad (25)$$

in which

σ_s = tensile stress in concrete in pounds per square inch;
W = weight of slab in pounds per square foot;
L = length of slab in feet;
h = depth of slab in inches;
C_a = average value of the coefficient of subgrade resistance as determined by equation 23 or equation 24.

For an assumed drop in average slab temperature of 40° F., the distance X as determined by equation 22 is 25 feet. For a value of L = 100 feet the calculated value of C_a (equation 23) is 0.83 C_m. In table 18 the maximum observed value of the coefficient of subgrade resistance, C_m, for the 6-inch slab is shown to be 2.5. Then for a 6-inch slab having a length of 100 feet and a weight of 75 pounds per square foot,

$$\sigma_s = \frac{75 \times 100 \times 0.83 \times 2.5}{24 \times 6} = 108 \text{ pounds per square inch.}$$

CONSTRUCTION PRACTICES TO REDUCE SUBGRADE RESISTANCE NOT EFFECTIVE IN REDUCING TRANSVERSE CRACKING

One of the more recent investigations of the tensile strength of concrete (*40*) indicates that concrete of the quality used in pavements, if thoroughly cured for a period of 28 days, may be expected to have a tensile strength at that age of the order of 200 to 250 pounds per square inch. When the computed contraction stress of 108 pounds per square inch in a slab 100 feet long is compared with a probable 28-day tensile strength of at least 200 pounds per square inch, it seems very probable that, in pavements provided with transverse joints at reasonable intervals, any transverse cracking, except that which may occur at very early ages, must be attributed primarily to the effect of warping stresses.

If this is true, it follows that the difference in degree of cracking that is observed in pavements constructed with different aggregates is due not so much to differences in the strength of the concrete as to differences in modulus of elasticity, thermal coefficient of expansion and, possibly, to differences in thermal conductivity that may affect the magnitude of the temperature differentials.

Some evidence of this is found in the records of the old Ohio Post Road which was constructed in 1914 and 1915 (*41*). In a part of the project the concrete aggregate was gravel and in the remainder it was crushed stone. Samples of concrete were taken from the pavement in 1932 and the compressive and flexural strengths determined. Both the gravel concrete and the crushed-stone concrete had compressive strengths of approximately 6,600 pounds per square inch. The modulus of rupture of the specimens of gravel concrete was 1,150 pounds per square inch and that of the specimens of crushed-stone concrete was 1,030 pounds per square inch. Yet, in a given length of pavement, the transverse cracks in the gravel concrete were much more numerous than in the crushed-stone concrete. Tests made in recent months indicate that the gravel concrete has a higher modulus of elasticity and higher thermal coefficient of expansion than the stone concrete. On the assumption that the temperature differential is the same in both kinds of concrete, the differences in the values of modulus of elasticity and thermal coefficient are sufficient to account for warping stresses 25 percent higher in the gravel concrete than in the stone concrete.

In the light of the foregoing discussion it also seems very probable that any special construction practices designed to reduce the subgrade resistance, and thereby reduce or eliminate transverse cracking, will not be particularly effective for the purpose. The limited experimental data that are available support this conclusion.

Some years ago it was observed in western Iowa that extensive hair cracking developed during the curing period in concrete pavements constructed on the loess soils that are prevalent in that area and in other portions of the valleys of the Missouri and Mississippi Rivers (*42*). These loess soils, unless saturated, are highly water absorbent. The hair cracking, which is caused by contraction, was attributed to the rapid drying of the concrete owing to excessive water absorption by the subgrade soil. It was found that a layer of tar paper, placed on the subgrade before the placing of the concrete, was quite effective in preventing this excessive loss of water and in eliminating the formation of hair cracks.

Since the development of the tar-paper subgrade treatment in Iowa it has been used extensively in other States. In some cases it has been used rather generally on all soils without regard to their capacity to absorb water from the concrete and apparently this practice has been influenced somewhat by the belief that the treatment would lower the subgrade resistance sufficiently to have a beneficial effect in the reduction of transverse cracking.

The effect of the tar-paper treatment was studied to a very limited extent in one of the investigations by the Bureau of Public Roads (*39*). This investigation, made primarily to study methods of curing concrete, involved the construction of a number of long concrete slabs. Included in these were two slabs, each 6 inches deep, 2 feet wide, and 200 feet long, that were cured in the same manner. The only difference between them was that one was placed on a dry soil and the other was placed on tar paper. The slabs were constructed during the summer of 1926.

In connection with the same investigation a determination was made of the effect of the tar-paper treatment on subgrade resistance. It was found that for small displacements of the test slabs the resistance was about the same for a slab on a dry subgrade as for one on tar paper. However, for displacements of the order of 0.05 inch it was found that the resistance developed by the dry subgade was about twice that which was developed with the tar-paper treatment.

In spite of this difference in subgrade resistance the 200-foot slab on the dry subgrade contained only 4 transverse cracks at the age of 5 days while at the age of 2 days the 200-foot slab on tar paper contained 6 transverse cracks. A survey made during the summer of 1938, when the slabs were about 12 years old, showed 11 cracks in the slab built on the dry subgrade and 15 cracks in the slab built on tar paper.

Thus, while the tar-paper treatment of the subgrade is undoubtedly effective for the purpose for which it was originally used, both theory and experiment point to the conclusion that it has no merit as a means for preventing the transverse cracking of pavements.

STEEL REINFORCEMENT BENEFICIAL IN CONCRETE PAVEMENT SLABS

Use of steel reinforcement.—It has been pointed out previously that, if detrimental cracking is to be prevented in thickened-edge pavements, the use of steel reinforcement is an alternate to the use of very short slabs with edge strengthening at all transverse joints. It has also been stated that in slabs of uniform thickness, adequately designed to resist edge stresses, no edge strengthening at transverse joints or cracks is required. While this is true, it should not lead to the conclusion that it will necessarily be safe to build long slabs of uniform thickness with the idea that the formation of open transverse cracks will not be detrimental.

In New Jersey (*43*) and elsewhere it has been observed that, even when the edge strength at transverse joints is adequate, trouble may develop at the joint from other causes unless the two slab ends are connected in such manner that the deflection of each will be approximately equal under the action of heavy wheel loads. In the absence of such a connection between the slab ends it has been found that, under certain conditions of soil and drainage, the end of the slab which is on the side of the joint opposite the approaching wheel load is gradually forced permanently below the level of the adjacent slab. This results in poor riding quality, increased impact reactions, and the eventual development of pavement failure in the vicinity of the joint. While this experience does not appear to be universal, it suggests that, at least under some conditions, the use of steel reinforcement in long slabs of uniform thickness may be beneficial in preventing the faulting that might otherwise develop at transverse cracks.

Design of reinforcement.—For a reinforced slab the same assumptions that are used in the derivation of equation 25 leads to the equation

$$A=\frac{WLC_a}{2f_s} \qquad (26)$$

in which W and C_a are the same as in equation 25, and

L=distance in feet between free joints (spacing of free transverse joints for computing longitudinal steel, and spacing of free longitudinal joints for computing transverse steel);

A_s=effective cross-sectional area of steel in square inches per foot of slab width;

f_s=allowable unit tensile stress in the reinforcement, in pounds per square inch.

If the steel reinforcement is to maintain in a tightly closed condition the warping cracks that will develop, it is necessary to limit its elongation at cracks to a very small amount. The total elongation of steel subjected to tensile stress is dependent on the length that is free to elongate. The reinforcement in a concrete pavement initially is in bond with the concrete and, when a crack forms, the bond is destroyed over a certain length of steel. This length is then free to elongate under the stress induced by the subgrade resistance. However, the length over which the bond is destroyed is not known and, therefore, it is impossible to compute accurately the total elongation corresponding to a given stress. This, in turn, makes it impossible to determine with accuracy the maximum allowable stress in the steel that will insure the maintenance of tightly closed cracks.

It is common practice to base the design of steel members on an allowable unit stress which is considerably less than the yield point of the steel. This is to minimize the possibility of elastic failure due to the occurrence of unforeseen stresses greater than those used in design. The practice is a logical one to follow but, in the case of slab reinforcement, the maximum permissible elongation should also be considered.

Slab reinforcement should be designed to limit the maximum width of cracks that may develop to a small dimension. But the crack width is dependent on the elongation of a certain length of steel and this elongation is in turn dependent, not on the strength of the steel, but on its modulus of elasticity and the unit stress to which it is subjected. Since all grades of reinforcing steel have approximately the same modulus of elasticity, it follows that the elongation in a given length is independent of the grade and varies only with the unit stress. Therefore, in the determination of a safe allowable unit stress, consideration should be given both to the yield point and to the maximum permissible elongation. However, as has been stated, the elongation corresponding to a given unit stress cannot be determined because the length of reinforcement that is free to elongate is not known. In addition, nothing definite is known concerning the maximum width of crack that can be permitted without the development of edge weakness.

In view of these considerations the best that can be done, until more information becomes available, is to select maximum allowable unit stresses that appear to be reasonably conservative when considered in relation to the yield point of the steel. Having done this, it is then possible to compute elongations that may be developed under certain assumed conditions.

SAMPLE CALCULATION OF AMOUNT OF REINFORCEMENT REQUIRED IN A PAVEMENT SLAB

The standard specifications of the American Society for Testing Materials require minimum yield points in the various grades of reinforcing steel, as follows:

	Pounds per square inch
Structural grade	33,000
Intermediate grade	40,000
Hard grade and rail steel	50,000
Cold-drawn steel wire	56,000

There is precedent for the use of an allowable working unit stress in steel equal to 50 percent of its minimum allowable yield point and the adoption of this value is suggested, pending the development of the information that is required for a more logical determination. In table 20 are shown computed elongations for the different grades of reinforcing steel, on the basis of this suggested unit stress, for assumed lengths of free elongation of 12, 18, and 24 inches.

The figures of table 20 indicate that if the steel is free to elongate over a length as great as 24 inches, the stresses permitted in the higher-strength steels are likely to result in the formation of open cracks having a width as great as 0.02 inch. On the other hand, the elongation in this length will not greatly exceed 0.01 inch for a unit stress of the order of 16,000 pounds per square inch. The data from the Arlington tests give some indication that an opening of 0.02 inch may result in some reduction in edge strength at a crack in a reinforced slab but the evidence is by no means conclusive.

TABLE 20.—*Elongation of steel reinforcement* [1]

Grade of steel	Unit stress 50 percent of yield point	Elongation in a length of— 12 inches	18 inches	24 inches
	Lb. per sq. in.	*Inches*	*Inches*	*Inches*
Structural	16,500	0.007	0.010	0.013
Intermediate	20,000	.008	.012	.016
Hard and rail steel	25,000	.010	.015	.020
Cold-drawn wire	28,000	.011	.017	.022

[1] Modulus of elasticity of steel=30,000,000 pounds per square inch.

Certainly a crack opening of 0.01 inch is less likely to create edge weakness than an opening of 0.02 inch, but the adoption of the lower limitation would require the use of a low unit stress for all grades of steel. This, in turn, would require the use of much greater amounts of steel than are commonly used and, since the necessity for it is not definitely indicated, the adoption of the low unit stresses would hardly be justified at the present time.

It will now be of interest to determine, from the preceding equations, the amount of reinforcement required in a pavement slab. The following assumptions will be made. The pavement is 20 feet wide with a longitudinal joint with bonded tie bars; the transverse joints are 50 feet apart; the slab is 8 inches thick and weighs 100 pounds per square foot; the maximum drop in temperature is 40° F.; the value of C_m (table 18) is 2.2; and the reinforcement will be welded wire fabric with an allowable unit stress of 28,000 pounds per square inch.

X=25 feet, and for the stress in the longitudinal direction C_a, as determined either by equation 23 or equation 24, equals 0.67 C_m. By the use of equation 26 it is then found that the required cross-sectional area of longitudinal steel is 0.132 square inch per foot of slab width. For stress in the transverse direction L=20 and C_a, as determined by equation 24, equals 0.42 C_m. Then the required cross-sectional area of the transverse steel, as determined by equation 26, equals 0.033 square inch per foot of slab width. These requirements may be met by No. 3-gage longitudinal wires on 4-inch centers ($A_s=0.140$) and No. 5-gage transverse wires on 12-inch centers ($A_s=0.034$), resulting in a fabric weighing about 63 pounds per 100 square feet.[6] Similar calculations for a slab 30 feet long indicate that wire fabric weighing about 37 pounds per 100 square feet is required.

In the above examples the transverse steel has been designed on the assumption that $L=20$ feet which, in turn, involves the assumption that the reinforcement is continuous through the longitudinal joint. This is not a usual condition since in common practice tie bars constitute the only reinforcement extending through the longitudinal joint.

When tie bars are used and the transverse reinforcement is interrupted at the longitudinal joint, the maximum tensile stress in the transverse steel is developed at the end of the tie bars and not at the joint. Therefore the effective value of L is less than the width of pavement by an amount equal to the length of the tie bars. Since this is the case, the amount of transverse steel computed as in the foregoing examples is somewhat excessive.

Also, since longitudinal cracks in slabs 10 feet wide are the exception rather than the rule, it is believed to be entirely safe to reduce the transverse reinforcement to the minimum practicable amount. The minimum might be established as No. 6-gage wires at 12-inch centers. The substitution of No. 6-gage wire for the No. 5-gage wire would reduce the weight of the fabric by a little less than 2 pounds per 100 square feet.

The above calculations to determine the required amount of reinforcement are for purposes of illustration only. The results should not be considered as necessarily applicable to all conditions.

Since the total cost of transverse joints in a given length of pavement increases as the required amount of steel reinforcement decreases, it is evident that the economical design of reinforced pavements requires consideration of both factors.

JOINTS NEEDED TO PREVENT CRACKING AND TO PROVIDE FOR EXPANSION AND CONTRACTION

Longitudinal and transverse joints.—The need for longitudinal and transverse joints in concrete pavements is demonstrated both by theory and by extensive experience. Longitudinal joints which divide the slab into lanes 10 to 12 feet in width are required to prevent the unsightly and detrimental longitudinal cracks that otherwise may be expected to develop. Transverse expansion joints are required at reasonable intervals, consistent with a rather narrow joint opening, to prevent compressive failures or blow-ups. In nonreinforced pavements, intermediate transverse contraction or warping joints are required at frequent intervals if cracks due to warping stresses are to be eliminated. In reinforced pavements the need for contraction joints is dependent on the spacing of expansion joints. The expansion joints may be placed at the ends of each reinforced slab, in which case no other transverse joints are required, or the distance between expansion joints may be made some multiple of the slab length in which case the intermediate joints are contraction joints.

Joints of numerous types and design are in use but no attempt will be made to describe all of them here. The discussion will be confined to the more common types of joints that were investigated in the Arlington tests. These are shown in figure 22.

The devices used to connect adjoining slabs either at transverse or longitudinal joints are required for several purposes. In the case of longitudinal joints in the interior of thickened-edge slabs the joint edges require strengthening and the joint designs shown in figure 22—A, B, and C are frequently used for this purpose. The transverse tie bars are bonded to the concrete and are required to prevent the separation of the slabs and the consequent loss of joint efficiency. The butt joint of figure 22—D and the thickened-edge joint of figure 22—E are suitable only for the so-called lane-at-a-time construction in which each width of slab is constructed separately. The butt joint may be used in the interior of thickened-edge slabs in which case the bonded tie bars are required to prevent loss of joint efficiency.

The longitudinal butt joint of figure 22—D may also be used in slabs of uniform thickness. In this case, and also in the case of the longitudinal thickened-edge joint of figure 22—E, the tie bars are not required for the purpose of edge strengthening but they are needed to prevent the separation of the slabs and the development of an unsightly appearance. The tarred felt shown in the butt and thickened-edge longitudinal joints is desirable to prevent any bond between the concrete in adjacent slabs and also to provide the play in the joint needed to relieve warping stresses.

All of the transverse expansion and contraction joints of figure 22, with the exception of the thickened-edge joint (fig. 22–G), when used in thickened-edge slabs require the use of dowels or other devices for the purpose of edge strengthening. When these joints are used in pavements of uniform thickness, or when the thickened-edge joint is used, the dowels are not needed for edge strengthening but, as has already been indicated, they may be needed under certain conditions to prevent the development of faults at the joints.

Provision for slab movement must be made in transverse joints and, in order that the dowels may be free to move, it is necessary to prevent the formation of a bond between the dowels and the concrete at least on one side of the joint. This is usually accomplished by painting or greasing the dowels, or both. Also, in expansion joints, caps or sleeves are required on one end of each dowel in order to provide space for the movement of the dowel into the slab when the joint closes. These dowel caps are not required in contraction joints.

IDEAL LONGITUDINAL JOINT WOULD ACT AS A HINGE

Design of tie bars.—The purpose of tie bars is to hold the edges of longitudinal joints in close contact and they may be designed in the same manner as steel reinforcement. For example, in a two-lane pavement the tie bars may be designed by means of equation 26 in which L is taken as the width of pavement. If intermediate grade bars, with an allowable unit stress of 20,000 pounds per square inch, are used in the center joint of the 8-inch uniform thickness slab for which the steel reinforcement has already been designed, the required area of steel is found to be 0.046 square inch per foot of joint. This requirement may be met by ½-inch round bars spaced 51 inches apart.

It should be noted that tie bars designed in this manner are intended only to hold the edges of the joint in close contact and they may not be adequate in all cases to furnish the edge strengthening that is required

[6] Gage numbers are those of the Standard Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement of the American Society for Testing Materials, Designation A82-34.

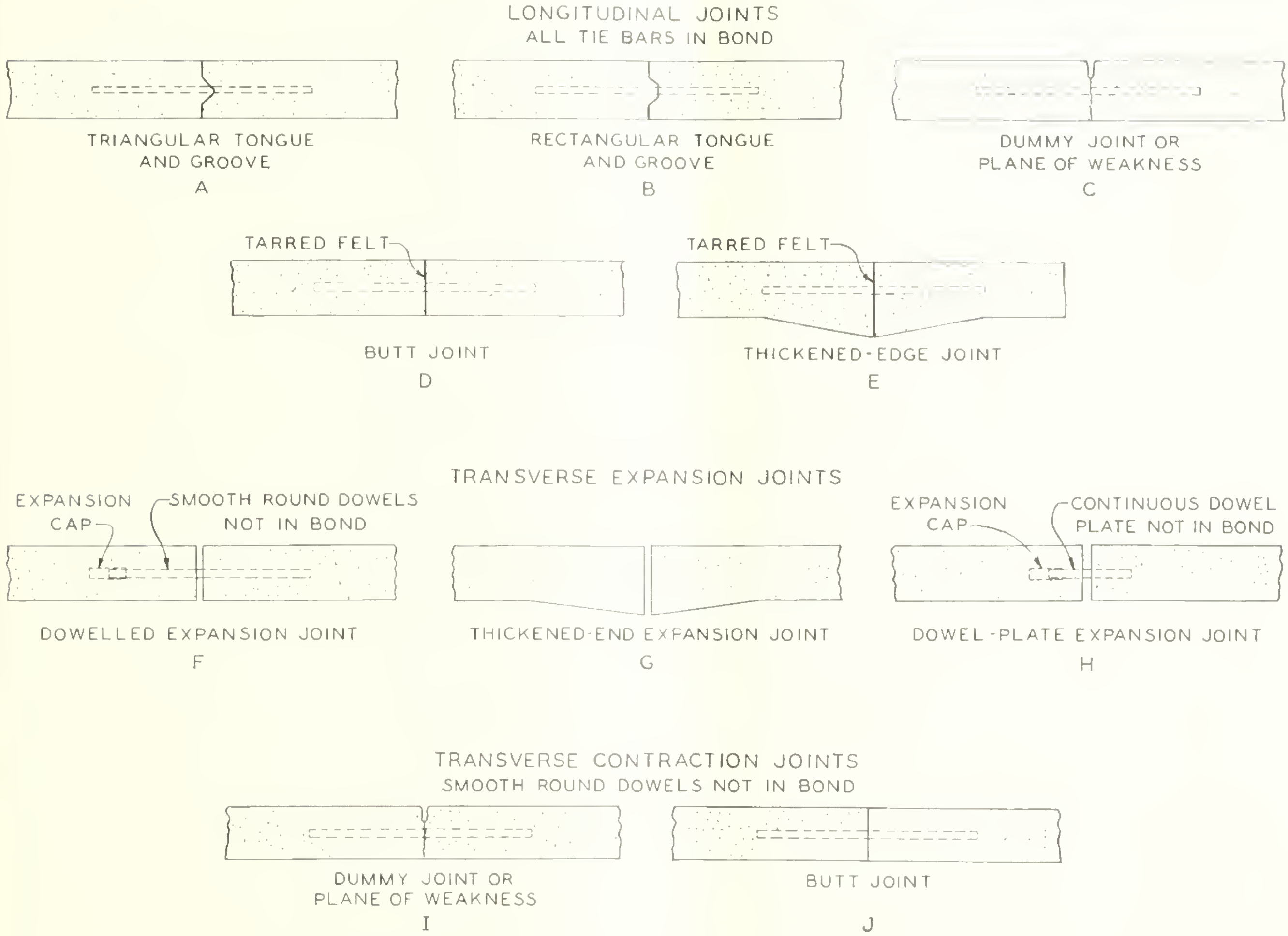

Figure 22.—Types of Joints for Concrete Pavements.

in the longitudinal joints of thickened-edge slabs. As will be shown later, the Arlington tests indicate that longitudinal tongue-and-groove joints, provided with ½-inch round tie bars spaced 60 inches apart, are quite effective in furnishing the necessary edge strengthening but that in longitudinal joints of the butt and dummy types it would be desirable to increase the size and number of the bars.

The depth of embedment of the tie bars in each slab should be sufficient to develop their strength in bond. The depth of embedment required to accomplish this is dependent on the allowable unit tensile stress in the steel and the allowable unit bond stress, and may be expressed by the equation.

$$D=\frac{f_s d}{4u} \qquad (27)$$

in which

D=depth of embedment in inches;
f_s=allowable unit tensile stress in the steel, in pounds per square inch;
u=allowable unit bond stress in pounds per square inch;
d=diameter of a round bar, or side of a square bar, in inches.

The 1937 Progress Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete recommends for plain bars a unit bond stress equal to 4 percent of the ultimate compressive strength of the concrete but not to exceed 160 pounds per square inch, and for deformed bars a unit bond stress equal to 5 percent of the ultimate compressive strength of the concrete but not to exceed 200 pounds per square inch.

For intermediate grade steel with an allowable unit stress of 20,000 pounds per square inch the required depths of embedment for the maximum bond stresses of 160 and 200 pounds per square inch are, respectively, 31¼ diameters for plain bars and 25 diameters for deformed bars. If deformed bars are used, the maximum bond stress of 200 pounds per square inch would require the total length of a ½-inch round tie bar to be 25 inches. A lower permissible unit bond stress or a higher permissible unit stress in the steel would require the use of longer bars.

The above method for designing tie bars is predicated on the assumption that the joint is of a type that will act as a hinge and will be incapable of developing any appreciable resistance to warping. If the design is such as to permit resisting moments to develop during warping it is not possible to calculate the stresses in the tie bars and even if it were practicable to do so it would not be desirable, in a joint offering high restraint to warping, to introduce sufficient steel to take the warping stresses since this would invite failure in other portions of the slab. The ideal longitudinal joint that acts wholly as a hinge has not yet been developed but by proper attention to the details of design it is possible to effect some reduction in the warping stresses that are caused by restraint in the joint.

In longitudinal joints that contain bonded tie bars the use of a design that does not permit the development of large resisting moments is desirable not only to reduce transverse warping stresses in the pavement as a whole, but also to reduce compressive stresses in the concrete at the joint and to prevent the tie bars from being overstressed in tension.

If restraint to warping is to be reduced it is necessary to prevent the abutting faces of the joint from being brought into close contact during warping, particularly at the top and bottom of the joint. In the butt joints of figure 22–D and E this may be accomplished by the introduction of a compressible layer of filler material between the slab edges.

The use of filler material throughout the depth of the joint would not be practicable in the dummy joint of figure 22–C. In this joint the resistance to downward warping is reduced by the groove in the top of the slab and it would appear that the most practical way to reduce the resistance to upward warping would be to form a similar groove in the bottom of the slab.

In the tongue-and-groove joints of figure 22–A and B the use of a compressible filler for the full depth of joint would be undesirable since it would reduce the ability of the joint to transfer load and to reduce edge stresses. However, strips of filler fastened to the vertical portions of the steel partition plates should be quite effective in reducing joint restraint without greatly reducing joint efficiency.

Even under the most favorable conditions it does not appear probable that restraint to warping will be completely eliminated in any of the types of longitudinal joints now in use and this should be taken into account in determining the length of tie bars. When warping takes place in a pavement it causes rotation of the joint faces, and when the rotation is sufficient to bring the faces into tight contact it develops compression in the concrete and causes the slab edges to separate at the plane of the steel. The tensile stress developed in the steel for a given separation of the joint faces is entirely dependent on the length of steel that is free to elongate.

EFFICIENCY OF JOINTS DISCUSSED

When a tie bar is in bond a very small rotational movement in the joint may create a very high initial stress in the steel. This may be expected to result in a necking down of the steel until it is ruptured or until the bond is destroyed over a suficient length to permit the bar to elongate the required amount without rupture. It has been observed in pavements that this destruction of the bond actually takes place for a distance of several inches on each side of the joint. As a result Friberg [7] has suggested that the midsection of tie bars, for a distance of several inches on each side of the joint, be coated with bitumen definitely to break the bond and also to furnish protection against corrosion.

Even if no definite provision is made for breaking the bond in the midsection of the bar it appears very probable that the bond will be destroyed over some unknown length by high stresses produced by warping. Therefore it appears desirable to make some arbitrary increase in the theoretical length of tie bars as computed by equation 27. An additional depth of embedment of at least 6 inches on each side of the joint or an increase of not less than 1 foot in the total length of the bar, is suggested.

Efficiency of joints.—The efficiency of any joint device used for edge strengthening is dependent on the degree to which it reduces the edge stresses that would otherwise be developed. In the past it has frequently been assumed that the relation between observed maximum deflections of adjacent slab ends under load could be taken as a measure of joint efficiency and that when these deflections were equal the joint was 100 percent efficient.

The Arlington tests (*18*) have shown that this assumption is incorrect. It was found, when a load was applied on one side of a joint, that the maximum deflections of the two edges might be identical but that the maximum stress in the loaded edge might be more than twice as great as that in the unloaded edge. As a result, the efficiencies of the joints involved in the Arlington tests were determined by a more logical method of analysis.

This analysis is based on the conception that if the joint fulfills its function perfectly, that is, with an efficiency of 100 percent, the stresses at the joint will not be greater than if the continuity of the slab were not broken. The efficiency of a given joint may then be expressed by the equation

$$J=100\left(\frac{\sigma_e-\sigma_j}{\sigma_e-\sigma_i}\right) \qquad (28)$$

in which

J=joint efficiency in percent;

σ_e σ_j, and σ_i are the critical stresses due to the application of a given load at the free edge, the joint edge, and the interior, respectively, of a slab of given uniform thickness.

This equation indicates a joint efficiency of zero when the critical stress at the joint equals the critical edge stress and an efficiency of 100 percent when the joint stress equals the interior stress.

Design of dowels.—The first theoretical analysis of the required spacing of dowel bars was that of Westergaard (*44*). This analysis enables one to compute the effect of dowel spacing on the critical stress in the edge of a joint, when the load is applied midway between two dowels, on the assumption that only the four dowels nearest the load are sufficiently active to require consideration and on the further assumption that the dowels are sufficiently stiff to cause the two joint edges to deflect exactly the same amount at all points. On the basis of his analysis Westergaard concluded that a dowel spacing of 3 feet is too great to result in any significant reduction in the critical edge stress and that, if the dowels are to be effective for the purpose, the spacing should not exceed about 2 feet.

A more detailed study of dowel spacing, on the basis of the Westergaard analysis, is included in the report of the Arlington tests (*18*). This study indicated that if rigid dowels are to effect the same stress reduction that would be effected by slab continuity, the spacing must be considerably less than 2 feet.

In considering these indications it should be remembered that they are based on the assumption that the dowels are rigid. Therefore they cannot apply to the small round dowels commonly used except as they may indicate general trends. Also it may be noted that, while increasing the stiffness of dowels will increase their efficiency, it will at the same time increase restraint to longitudinal warping. Dowels that are too stiff may cause more distress in the pavement slab than would result from their complete omission.

The analysis and tests by Friberg (*45, 46*), which have become available only in recent months, make it possible for the first time to design dowelled joints on a

[7] Bengt F. Friberg, Research Engineer, Laclede Steel Co., St. Louis, Mo.

rational basis. The analysis shows that a maximum joint efficiency can be obtained with round steel dowels of reasonable size only by using much smaller spacings than those indicated by the Westergaard analysis.

DOWEL LENGTH OF 2 FEET FOUND EXCESSIVE

The analysis and tests by Friberg show that:

1. The lowest joint efficiency occurs when the load is between two dowels.
2. If the dowels are to have their greatest effectiveness in slabs of normal thickness the dowel spacing should not exceed about 12 inches.
3. The efficiency of the dowel decreases as the width of the joint is increased and increases as the diameter of the dowel is increased. For example, Friberg has shown that for a dowel directly under a load the percentage of load transfer of a 1-inch dowel across a joint in a 7-inch slab is 29 percent for a ½-inch joint and 25 percent for a 1-inch joint; and that for a ½-inch joint the load transfer of a ¾-inch dowel is 22 percent as compared with 29 percent for a 1-inch dowel.

On the assumption that the effectiveness of the dowel is such that it will result in a stress relief of 25 percent it is of interest to compute the efficiency of a dowelled joint in a 7-inch slab. For the 8,000-pound wheel on dual high-pressure tires that has been used in previous stress computations, the same assumed characteristics of the concrete and a value of $k=100$, the interior load stress in a 7-inch slab is 290 pounds per square inch and the edge stress at a transverse joint (equation 15) is 490 pounds per square inch. By means of equation 28 it is found that the joint efficiency equals $100\left(\frac{490-0.75\times490}{490-290}\right)$, or 61 percent.

4. The length of effective embedment of the dowel in the concrete of each slab need not be greater than 5 inches for ¾-inch dowels and not greater than 7 inches for 1-inch dowels. Thus it is indicated that the dowel length of 2 feet, that has been customary, is excessive. It is important to note that when these short lengths of embedment are used the length of dowel cap and the width of joint opening should be considered in determining the required length of dowel.
5. Initial failure at dowels occurs by spalling of the concrete at the face of the joint under loads that may be as much as 50 percent less than the ultimate load sustained by the joint. This initial failure greatly reduces, if it does not completely destroy, the effectiveness of the dowels for stress relief.

Required efficiency of joints and load transfer devices.—Theoretically, even with very stiff dowels, the maximum amount of load transfer at a joint can never equal exactly 50 percent of the load applied on one side of the joint, on account of the eccentricity of the point of load application with respect to the joint. The unavoidable, and also desirable, flexibility of the joint device further reduces the possibility of ever obtaining at a joint a stress reduction of 50 percent. However, such a reduction is not necessarily required in order to obtain a joint efficiency of 100 percent nor is a joint efficiency of 100 percent always required in order to limit joint stresses to safe values.

In the preceding example it has been shown that, for the conditions assumed, a stress reduction of 25 percent results in a joint efficiency of 61 percent. In this example the interior and edge stresses are, respectively, 290 and 490 pounds per square inch. If it be assumed that a safe unit stress is 350 pounds per square inch, then the required joint efficiency equals $100\left(\frac{490-350}{490-290}\right)$, or 70 percent. This joint efficiency would require a stress reduction of $100\times\frac{140}{490}$, or about 29 percent.

The preceding computations of joint efficiency have involved only stresses due to load. In the following examples the combined stresses due to load and temperature warping will be considered. It will be assumed that the slab is 10 feet wide and 10 feet long, that $k=100$, and that the load, the temperature differential, and the properties of the concrete are the same as in preceding stress calculations.

JOINT EFFICIENCY OF 100 PERCENT NOT REQUIRED FOR SAFE STRESSES

In a thickened-edge slab having an interior thickness of 7 inches the load stresses at the interior and at the joint edge (equation 9) are, respectively, 290 and 420 pounds per square inch. The interior and edge warping stresses are, respectively, 90 and 70 pounds per square inch. The combined stresses are then 380 pounds per square inch at the interior and 490 pounds per square inch at the edge. The joint efficiency will be computed on the assumption that the joint device used results in a stress reduction at the joint of 25 percent. No joint device can be expected to reduce the transverse warping stresses and therefore the stress reduction applies only to load stress. Reducing by 25 percent the load stress of 420 pounds per square inch and adding to this the warping stress of 70 pounds per square inch gives a value of the combined stress, σ_j, equal to 385 pounds per square inch. The joint efficiency then equals $100\left(\frac{490-385}{490-380}\right)$, or about 95 percent.

It has been shown in table 15 that if the slab length is 10 feet the combined stresses at the edge and interior of a 10–6.8–10-inch thickened-edge slab are well balanced and are limited to approximately 425 pounds per square inch. With $k=100$ the combined interior stress in this slab is 390 pounds per square inch and the combined stress at the edge of a free transverse joint (table 16) is 520 pounds per square inch. If it is desired to limit the combined edge stress to 425 pounds per square inch, the required joint efficiency is $100\left(\frac{520-425}{520-390}\right)$, or 73 percent. The load stress at the joint edge is 440 pounds per square inch and therefore the reduction in load stress equals $100\times\frac{95}{440}$, or about 22 percent. On the other hand, if it were desired to have a joint of 100 percent efficiency it would be necessary to reduce the edge stress from 520 pounds per square inch to 390 pounds per square inch. In this case the required reduction in load stress, or transfer of load, equals $100\times\frac{130}{440}$, or about 30 percent.

Thus it is seen that a load transfer, or stress reduction of 50 percent is not necessarily required in order to obtain a joint efficiency of 100 percent and that a joint efficiency of 100 percent is not necessarily required in order to limit to safe values the stresses in the joint edge.

Tests of joint efficiency.—In connection with the Arlington tests (*18*) a great many tests were made on the types of joints included in the investigation to determine their effectiveness in reducing edge stresses due to load. The results are summarized in tables 21 and 22, the reported efficiencies having been computed by equation 28.

With respect to the longitudinal joints it may be noted that the measured efficiencies of the two tongue-and-groove joints containing bonded tie bars were relatively high even though the tie bars were only one-half inch in diameter, and were spaced 5 feet apart. It may also be noted that the omission of tie bars from a tongue-and-groove joint reduced its efficiency by about one-third.

TABLE 21.—*Observed efficiency of longitudinal joints (average values for tests at a number of points)* [1]

Type of joint	Designation in fig. 22	Spacing of tie bars [2]	Diameter of bars	Joint efficiency
		Inches	*Inches*	*Percent*
Triangular tongue	A	60	½	75
Rectangular tongue	B	60	½	78
Do		None		50
Butt	D	24	¾	52
Do	D	36	¾	42
Do	D	48	¾	51
Do	D	60	¾	47
Dummy	C	60	½	44
Do		None		39

[1] Data from table 11, PUBLIC ROADS, October 1936.
[2] All tie bars in bond.

TABLE 22.—*Observed efficiency of transverse joints (average values for a number of tests)* [1]

Type of joint	Designation in fig. 22	Spacing of dowels[2]	Joint opening	Joint efficiency				
				Winter	Summer	Average (various seasons)	Over dowels	Between dowels
		Inches	*Inches*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Dowel	F	36	½				46	8
Do	F	27	½				31	6
Do	F	27	¾				16	20
Do	F	18	½				28	8
Do	F	18	¾				40	28
Dummy	I	18		71	66			
Do		None		4	41			
Dowel plate [3]	H		½			59		
Do	H		¾			66		

[1] Data from table 10, PUBLIC ROADS, October 1936.
[2] All dowels ¾-inch diameter—not in bond.
[3] Dowel plates 4 inches by ¼ inch.

The longitudinal butt joints, which were all in slabs of the same thickness, had much lower average efficiencies than the tongue-and-groove joints in spite of the fact that the tie bars were of larger size and in general were more closely spaced. In the butt joints there is no consistent relation between average joint efficiency and tie-bar spacing. This is contrary to what would be expected and may be at least partially explained by the fact that the figures given are average values from tests in which the loads were applied at a great many different points. It was found in testing these butt joints that there was a rather consistent relation between joint efficiency and the distance from the center of the load to the center of the nearest tie bar. The average observed efficiencies for a load directly over a tie bar and at distances of 18 and 30 inches from it were about 70, 45, and 35 percent, respectively (fig. 35, PUBLIC ROADS, Oct. 1936). This would indicate that tie-bar spacing has an influence on the efficiency of longitudinal butt joints in spite of the lack of evidence in the average values given in table 21.

TESTS INDICATE DOWEL SPACINGS FORMERLY USED ARE EXCESSIVE

The average efficiency of the longitudinal dummy joint with tie bars was of about the same order of magnitude as that of the butt joints and the omission of tie bars reduced the average efficiency by only 5 percent. Both results may seem somewhat surprising, the first because it is so low and the second because it is so high, but here again average values are being considered. In testing these longitudinal dummy joints it was found that for loads at certain positions the indicated efficiency was very high while at other positions it was practically zero. It was also noted frequently that the joint was efficient for a load on one side of it and inefficient when the load was placed directly opposite on the other side of the joint. It seems evident that the measured efficiency of a dummy joint is largely dependent on the form of the fracture, particularly the direction of its slope, directly under the load.

The thickened-edge longitudinal joint shown in figure 22–E was not investigated in the Arlington tests but no tests are necessary to establish its efficiency. This is entirely dependent on the proper proportioning of the edge section in the manner that has already been discussed.

The transverse doweled expansion joints were tested at points directly over the dowels and midway between them, as indicated in table 22. In general the average efficiency was very low for a load between the dowels and, with one exception, was considerably greater for a load directly over a dowel. This investigation was planned in 1930 when the knowledge of the action of joint devices was considerably less than at present. The tests themselves, now supplemented by the analysis by Friberg, have shown that the program was quite inadequate for a thorough investigation of the efficiency of doweled joints. It is rather definitely indicated that the dowel spacings were too great for effective dowel action and analysis of the data is complicated by the fact that the joints were installed in slabs of different thickness. Therefore the results obtained should not be considered as indicative of the best performance of doweled expansion joints that can be expected.

The transverse dummy contraction joints were tested both in summer and winter and the joint with dowels had a high efficiency in both seasons of the year. The joint without dowels had a fair efficiency during the summer when the slabs were in an expanded condition and the width of the crack was small, but the efficiency was negligible in the winter when contraction had taken place and the width of crack was as great as 0.03 inch. Therefore, it appears that even in slabs as short as these (20 feet) the interlocking of the fractured faces in a transverse dummy joint cannot be depended upon to provide adequate load transfer when the slabs are in a contracted condition.

The two dowel-plate expansion joints that were tested had efficiencies comparable with the efficiency of the dummy contraction joint with dowels. The figures indicate that a dowel plate of the size investigated is an effective means for bridging the openings in expansion joints but more information is needed regarding the required depth of embedment of the dowel plate in the slab and the required thickness of plate.

The butt contraction joint shown in figure 22—J was not investigated in the Arlington test but its performance should be expected to be much the same as that of the doweled expansion joints, with probably a somewhat greater efficiency on account of the smaller width of joint opening.

For the thickened-end transverse expansion joint shown in figure 22–G the efficiency observed in the Arlington tests was low since the edge thickness was inadequate. When the edge section is properly designed the edge stress is the same as the interior stress and no edge strengthening or load transfer is required.

In the past the thickened-end type of transverse joint has been criticised on the ground that it offers additional resistance to contraction, with the result that a transverse crack is likely to develop near the junction of the end section with the interior of the slab. No action of this kind has been observed in the Arlington tests. The slabs with thickened ends have expanded and contracted as freely as any of the other slabs tested and no transverse cracks have developed in them in a period of more than 8 years. There is nothing in the results of these tests to indicate that edge thickening cannot be applied to transverse expansion joints with as much success as to the longitudinal edges of the slab.

Very little information of a definite character is available concerning the reported unsatisfactory performance of thickened-end transverse joints. The only reference that has been found is in a 1932 report of a committee of the American Road Builders' Association (*47*). This report merely states that experience with the thickened-end joint in three States has not been entirely satisfactory; that transverse cracking usually develops near the joint, with subsequent buckling of the slab ends due to expansion and with the further result, in some cases, of complete breakage under the action of traffic.

In contrast to this is the experience of Kent County, Mich. Mr. Otto S. Hess[8] is authority for the following report of that experience.

EXPERIENCE SHOWS THICKENED-END SLABS SATISFACTORY

Since 1926 practically all of the concrete pavements built by the Kent County Road Commission have been constructed with thickened-end transverse expansion joints spaced 50 feet apart and with no intermediate contraction joints. The 50-foot slabs are reinforced with wire fabric or bar mats. The expansion joints are ¾ inch wide and a premolded joint filler is used. The ends of adjacent slabs are not connected in any manner.

With this design, transverse cracking has been almost eliminated. Not a single transverse crack has been observed in the vicinity of the joints where the end-thickening begins. The contention that contraction in a thickened-end slab will cause the ends to ride up on the subgrade and create roughness at joints has not been supported since no difficulty has developed because of vertical movement of the slab ends. The experience of Kent County indicates that if the strength required in joint edges is obtained by thickening the slab ends it is not necessary to connect the slabs with dowels or other devices in order to maintain smooth joints.

The Arlington tests were quite inadequate from the standpoint of a comprehensive study of joint action since the variables included in the program were not of sufficient number or of sufficient range. However, the results obtained, when viewed in the light of the Friberg analysis and the discussion of the required efficiency of joints, indicate that if proper attention is given to the design of both the slab and the joint a number of the types of joints in common use can be expected to effect the required stress reduction.

[8] Engineer-Manager, Kent County Road Commission, Grand Rapids, Mich.

Effect of joints on corner stresses.—An assumption similar to that used in deriving equation 28, which gives a measure of the efficiency of a joint in reducing edge stress, might be used in developing a measure of the efficiency of a joint in reducing corner stress. For example, it might be assumed that with a joint of 100 percent efficiency the corner stress should be no greater than the stress in the edge of the slab at some distance from the corner. However, it is not necessary to do this and, in some cases, such an assumption would result in an indicated efficiency in excess of 100 percent in joints having no provision whatever for stress reduction.

In a slab of uniform thickness, corner load stresses computed by equation 11 exceed edge load stresses computed by equation 15, but only by relatively small amounts. In the case of combined stresses in slabs 15 to 30 feet long and ranging in depth from 7 to 10 inches, figures 15 and 17 show that the edge stresses are always greater than the corner stresses. In 10-foot slabs of these depths the combined corner stresses exceed the combined edge stresses by 50 to 80 pounds per square inch when $k=100$, but when $k=300$ the edge and corner stresses are practically the same. Therefore it appears that in a slab of adequate design there is no great need for stress reduction at the joint corners and that any reduction effected by the joint device will be in the nature of a factor of safety.

In the Arlington tests the difference between the stress at a free corner and that at a joint corner was determined and this stress reduction was expressed as a percentage of the stress at the free corner (table 12, PUBLIC ROADS, October 1936). It was found that the transverse joints (table 22) were about equally effective in reducing corner stress and that the average reduction was about 40 percent. Of the longitudinal joints that could be tested, the butt joint with tie bars spaced 24 inches apart and the dummy joint with tie bars resulted in an average reduction in corner stress of about 50 percent and the dummy joint without tie bars reduced the corner stress by about 40 percent. Thus all the joints tested were quite effective in reducing corner stress although some of them were quite ineffective in reducing edge stress.

CONCLUSIONS

The discussion that has been presented leads inevitably to certain conclusions which, if accepted, require a rather drastic revision in some of the accepted ideas concerning the structural design of concrete pavements. These conclusions are open to attack principally on the ground that practical experience in certain localities or under certain conditions does not always support them. This is recognized but it is believed that, for the country as a whole, they are supported by observations of the behavior of pavements in service. The exceptions may be due to a number of causes, an important one being that many concrete pavements are not subjected to loads of the magnitude and frequency for which presumably they were designed.

In other engineering structures, such as bridges and buildings, the absence of failure is not necessarily an evidence of adequate design since structures do not always fail even when dangerously overstressed. The same is true of concrete pavements. It is recognized, of course, that it would be unreasonable to be as conservative in the design of pavements as in the design of bridges but it should also be recognized that the factor of safety in many pavement designs in current use is negligible.

On the basis of the information presented, concrete pavements may be designed with reasonable assurance that they will be free from structural defects over a long period of time. A lowering of the indicated requirements of design may result in structural failures of varying degrees of importance. The extent to which the possibility of such failures can be tolerated is a matter to be decided on the basis of engineering judgment.

The more important conclusions that are indicated are as follows:

1. The critical load stresses developed in a concrete pavement are primarily dependent on single wheel loads and not on axle loads, axle spacing or the gross weight of vehicle.

2. Impact forces considerably in excess of static wheel loads should be used in the design of pavements. The impact factor (ratio of total impact reaction to static wheel load) is less for balloon tires than for high-pressure tires and decreases as the wheel load increases.

3. The stresses in a concrete pavement are approximately the same for an 8,000-pound wheel load on dual high-pressure tires and for a 9,000-pound wheel load on dual balloon tires.

4. The stress analyses of Westergaard, with the modifications suggested by the Arlington tests, are suitable for use in the design of concrete pavement slabs and form the only adequate basis for such design.

5. Since the physical characteristics of the subgrade and of the concrete can never be foretold with certainty it is desirable to be conservative in the selection of values representing these various characteristics for use in design.

6. Warping stresses due to differentials of temperature within the slab may be of the same order of magnitude as the stresses due to heavy wheel loads and therefore require consideration in pavement design.

7. Reasonable assurance of the absence of transverse cracking in concrete pavements can be obtained only by the use of short slabs having lengths not greater than 10 to 15 feet.

8. Transverse cracks in thickened-edge pavements without reinforcement create a weakened condition in the interior of the slab which may be serious. The introduction of properly designed steel reinforcement in long slabs will not completely eliminate transverse cracking but it will reduce or eliminate the detrimental effect of the cracks which may develop.

9. The edges of transverse joints in thickened-edge slabs require strengthening because the central portion of the joint has the same thickness as the interior of the slab but is subjected to the higher stresses that are associated with edge loading.

10. When the pavement is designed for the combined stresses due to load and temperature it is safe practice to use an allowable unit stress in excess of 50 percent of the 28-day flexural strength of the concrete.

11. When the pavement is designed for maximum legal wheel loads and in such manner that the combined stresses due to load and temperature are limited to safe values and are reasonably well balanced, the thickened-edge section has no great advantage over the section of uniform thickness from the standpoint of over-all cost per mile.

12. Transverse joints are required in concrete pavements to relieve warping stresses due to temperature and also to provide for longitudinal expansion and contraction. Longitudinal joints are required to prevent the longitudinal cracking that usually develops otherwise.

13. If proper attention is given to the design of both the slab and joint, the required edge strengthening at joints in thickened-edge slabs can be obtained with a number of the types of load-transfer devices in common use.

14. The thickened-end transverse expansion joint is indicated, both by tests and experience, to be a highly effective method of providing the edge strengthening that is required at transverse joints in thickened-edge slabs.

15. Longitudinal joints of the tongue-and-groove type appear to be considerably more effective than other types in common use in providing the strengthening that is required in the edges of the longitudinal joints of thickened-edge slabs.

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DISPOSITION OF STATE MOTOR-FUEL TAX RECEIPTS, 1938

[Compiled for calendar year from reports of State authorities]

State	Net total receipts of calendar year	Adjustments due to undistributed funds, etc.[1]	Net total funds distributed[2]	Expenses of collection and administration	For other administrative purposes[3]	For State highway purposes: Construction, maintenance, and administration[4]	For State highway purposes: State highway police	For State highway purposes: Service of State highway obligations: State highway bonds and notes	For State highway purposes: Service of State highway obligations: State assumed local obligations[5]	For State highway purposes: Service of State highway obligations: Total	Total for State highway purposes	For local roads and streets[6]: For work on county and local roads[4]	For local roads and streets[6]: For work on city streets[7]	For local roads and streets[6]: Service of local highway obligations	For local roads and streets[6]: Total	For other highway purposes (park and forest roads, etc.)	For nonhighway purposes: To general funds[8]: Inspection fees, dealers' licenses, etc.	For nonhighway purposes: To general funds[8]: Motor-fuel tax	For nonhighway purposes: For relief of unemployment or destitution	For nonhighway purposes: For education	For nonhighway purposes: For other specific purposes[9]	For nonhighway purposes: Total
	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars
Alabama	13,579	−549	13,030	81	58	3,499		3,231		3,231	6,730	6,161			6,161							
Arizona	4,243		4,243	56		2,790	119				2,909	*1,273			1,273						5	5
Arkansas	10,092		10,092	310	3	2,438	12	3,985	2,182	6,167	8,617	1,042		101	1,143		19					19
California	47,117	33	47,150	167		27,670					27,670	*12,573	3,757		16,330				2,983			2,983
Colorado	7,465	18	7,483	79	24	3,966	265	1,154		1,154	5,385	1,995			1,995							
Connecticut	9,242		9,242	32		7,565			1,645	1,645	9,210											
Delaware	2,073		2,073	11		[4] 1,305	165	109	334	443	1,913	([4])	8		8	138					3	3
Florida	23,232	31	23,263	27	95	10,229	29		9,288	9,288	19,546						309	1,629		1,628	29	3,595
Georgia	19,633	−2	19,631	590		10,043			2,402	2,402	12,445	3,341			3,341					3,255		3,255
Idaho	4,085	1	4,086	15		4,057	14				4,071											
Illinois	36,888	12	36,900	555	242	9,518					9,518	8,952	6,021	2,195	17,168		164		6,895	2,358		9,417
Indiana	22,770	−3,382	19,388	89	127	9,317					9,317	6,708	1,868		8,576		384	895				1,279
Iowa	13,234	−25	13,209	85		3,976			3,304	3,304	7,280	*5,844			5,844							
Kansas	10,168	64	10,232	325	117	6,312	117	228	681	909	7,338	2,452			2,452							
Kentucky	12,531	2	12,533	45		10,577	243				10,820	1,668			1,668							
Louisiana	16,627	−99	16,528	37	77			9,319		9,319	9,319								4,777	1,159	1,159	7,095
Maine	5,558		5,558	17		3,370	170	1,493		1,493	5,033	508			508							
Maryland	9,929		9,929	34		4,357		1,519		1,519	5,876	1,034	2,476	509	4,019							
Massachusetts	20,194	−5	20,189	50		6,807	279	[10] 2,552		2,552	9,638	7,957		[10] 367	8,324	735			[10] 1,442			1,442
Michigan	27,683	−123	27,560	309		15,714		4,982		4,982	20,696	*6,550			6,550		5					5
Minnesota	19,570	1	19,571	59	108	12,749	171				12,920	*6,460			6,460		24					24
Mississippi	10,181	40	10,221	110	18	2,696	21	2,916		2,916	5,633	*4,190			1,190	270						
Missouri	11,636		11,636	64	58	4,414	340	6,373	123	6,496	11,250		197		197		67					67
Montana	4,452	120	4,572	21		3,573		972		972	4,545										6	6
Nebraska	11,139		11,139	114	30	6,501					6,501	*3,030	364		3,394				1,100			1,100
Nevada	1,202		1,202	3		1,164	28		7	7	1,199											
New Hampshire	3,298	−2	3,296	([11])		2,164	36	824		824	3,024	158		[12] 114	272							
New Jersey	22,362	−143	22,219	263		3,596	187	7,482		7,482	11,265	2,695		570	3,265	173			5,415	1,190	648	7,253
New Mexico	4,090	−49	4,041	90	2	2,201		1,748		1,748	3,949											
New York [13]	66,195	598	66,793	104		10,491	609	4,655		4,655	15,755	*8,778	1,659		10,437	2,334		38,163				38,163
North Carolina	24,308	−132	24,176	([14])	108	[4] 15,777	324	6,688	321	7,009	23,110	([4])					885	7			66	958
North Dakota	2,318	96	2,414	12	64	1,206		344		344	1,550	775			775			13				13
Ohio	[15] 45,982	236	46,218	388		19,033					19,033	8,576	6,363		14,939					11,858		11,858
Oklahoma	13,910	−56	13,854	416		10,070					10,070	*3,364			3,364		4					4
Oregon	9,838	−30	9,808	38		4,489	259	3,290		3,290	8,038	*1,628	17		1,645	87						
Pennsylvania	52,001	−87	51,914	349		25,923	3,011	2,559		2,559	31,493	*6,503			6,503	112			12,958		499	13,457
Rhode Island [13]	3,495	−50	3,445	22		1,104	106	156		156	1,366					16		2,041				2,041
South Carolina	11,462		11,462	[16] 39	[16] 12	4,160		1,360	3,831	5,191	9,351	*1,870			1,870		190					190
South Dakota	4,102		4,102	43	46	3,445					3,445	440			440	33	18				77	95
Tennessee	19,231	857	20,088	199	44	2,411		7,038	1,931	8,969	11,380	5,377			5,377		1,000				2,088	3,088
Texas	42,747	−163	42,584	693		20,370			10,475	10,475	30,845	580			580					10,466		10,466
Utah	3,478	−75	3,403	17	1	3,268	117				3,385											
Vermont	2,530	30	2,560	3		1,388		301		301	1,689	847			847	5					16	16
Virginia	16,621	−47	16,574	([14])	21	[4] 15,793		353		353	16,146	[4] 289	78		367						37	37
Washington	15,431		15,431	26		6,274		[10] 121	51	172	6,446	*6,173	1,673	[10] 98	7,944				[10] 1,015			1,015
West Virginia	9,397	−1	9,396	13		[4] 4,728		4,655		4,655	9,383	([4])										
Wisconsin	19,447	432	19,879	72	194	9,882			2,411	2,411	12,293	4,226	693		4,919	104		2,297				2,297
Wyoming	2,478	−1	2,477	25		1,670	51	112		112	1,833	619			619							
District of Columbia	2,520	−1	2,519	([17])									2,508		2,508		11					11
Total	771,764	−2,451	769,313	6,097	1,452	344,050	6,673	80,519	38,986	119,505	470,228	134,636	27,682	3,954	166,272	4,007	3,080	45,045	36,585	31,914	4,633	121,257

[1] Amounts distributed during the calendar year often differ from actual collections because of undistributed funds and lag between accounts of collecting and expending agencies.

[2] In many States the proceeds of highway user taxes are placed in a common fund from which a distribution is made. The amounts so distributed have been prorated in proportion to the receipts not otherwise dedicated. See tables, pp. 128 and 129.

[3] Where reported separately from collection expenses, funds allotted for motor-fuel inspection, administration of Motor Vehicle Department, and regulation of motor vehicles are shown in this column.

[4] The following allotments for construction and maintenance of county roads under State control are included in State highway purposes: Delaware, $295,000; North Carolina, $7,985,000; Virginia, $7,500,000; West Virginia, $1,882,000.

[5] Reimbursement to local units of government for amounts spent on roads now on State system.

[6] In States indicated by star (*) law provides that these funds may also be used for service of local highway obligations. Amounts so used not reported separately. In Colorado funds may be used on both State and local roads.

[7] This column shows specific allotments for city streets. Where reported separately, funds allotted for urban extensions of State highway system are included in allotments for State highway purposes.

[8] To State general funds, except in Wisconsin where amounts went to towns, cities, and villages in lieu of personal-property taxes formerly imposed on motor vehicles. Allocation, to local general funds may have been used in part for highways, but such amounts not reported

[9] For the following purposes: Arizona, irrigation engineering expenses; Delaware, C. C. C. ditching, Florida, aviation; Louisiana, harbor improvement; Montana, labor furnished counties; New Jersey, service of institutional construction bonds, $514,000, and Department of Commerce and Navigation, $134,000; North Carolina, State Probation Commission; Pennsylvania, aircraft landing fields, $455,000, and cooperative work, other departments, $44,000; South Dakota, payment on real-estate bonds; Tennessee, debt service on nonhighway bonds, $2,081,000, and aviation, $7,000; Vermont, debt service on nonhighway portion of flood-relief bonds; Virginia, aviation.

[10] Includes debt service charges on emergency relief bond issues prorated in proportion to use of proceeds for State highway, local road, and nonhighway purposes.

[11] Paid out of motor-vehicle revenue, $3,000. See following table.

[12] Service of highway relief bonds, a State obligation incurred for improvement of local roads.

[13] Appropriations for highway purposes out of State general fund have been credited against payments of motor-fuel tax and motor-vehicle registration fees to the general fund and prorated in proportion to net receipts from highway-user taxes not otherwise dedicated.

[14] Included in cost of collecting motor-vehicle revenue. See following table.

[15] Tax of $633,000 on non-motor-vehicle fuels not included.

[16] Estimated from fiscal year appropriations.

[17] Paid out of general revenue. Amount not reported.

DISPOSITION OF STATE MOTOR-VEHICLE RECEIPTS, 1938

[Compiled for calendar year from reports of State authorities]

State	Net total receipts of calendar year	Adjustments due to undistributed funds, etc.[1]	Net total funds distributed[2]	Expenses of collection and administration[3]	For other administrative purposes[4]	For State highway purposes: Construction, maintenance, and administration[5]	State highway police	Service of State highway obligations: State highway bonds and notes	Service of State highway obligations: State-assumed local obligations[6]	Service of State highway obligations: Total	Total for State highway purposes	For local roads and streets: For work on county and local roads[5]	For work on city streets[8]	Service of local highway obligations	Total	For other highway purposes (park and forest roads, etc.)	For nonhighway purposes: To general funds[9]	For relief of unemployment or destitution	For education	For other specific purposes[10]	Total
	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars
Alabama	4,314		4,314	446		1,612	402	1,131		1,131	3,145						723				723
Arizona	1,076	-1	1,075	241		797	31				831										
Arkansas	2,908		2,908	67		765	108	1,252	716	1,968	2,841										
California	23,930	923	24,853	3,786	253	3,701	3,000	3,876		3,876	10,580	*3,626			3,626		6,608				6,608
Colorado	2,544	53	2,597	114		823	55	239		239	1,117	1,036			1,036						
Connecticut	6,611	59	6,670	996		1,959	325		426	426	2,710	2,964			2,964						
Delaware	1,216	-67	1,149	97		[5] 666	85	56	170	226	977	([5])	1		1	70				1	1
Florida	6,432		6,432	479	210														5,743		5,743
Georgia	1,974	-3	1,971	305		1,103	279		264	264	1,646	20			20						
Idaho	2,380	25	2,405	69		247	103				350	*1,986			1,986						
Illinois	21,591	602	22,193	803	383	7,977	1,377	9,671	24	9,695	19,049	1,903			1,903		55				55
Indiana	9,635	-566	9,069	985		3,846	707				4,553	2,375	654		3,029		502				502
Iowa	11,797	8	11,805	1,000		5,902			4,903	4,903	10,805										
Kansas	3,823	227	4,050	351		2,385	45	86	257	343	2,773	926			926						
Kentucky	4,599	-10	4,589	456	26	1,895	55				1,950	896			896		1,261				1,261
Louisiana	4,892	287	5,179	161	50	3,294	334	1,340		1,340	4,968										
Maine	3,582	50	3,632	147		2,411	120	936		936	3,167	318			318						
Maryland	5,069		5,069	340	56	1,969	118	1,086		1,086	3,473		776		776			124			421
Massachusetts	6,759	12	6,771	1,604	37	1,734	71	[11] 650		650	2,455	2,027		[11] 94	2,121	187		[11] 367			367
Michigan	20,856		20,856	1,868			378				378	*18,366			18,366		214				214
Minnesota	9,377	21	9,398	437		1,657	149	1,706	2,331	1,037	8,813						118				118
Mississippi	4,001	-77	3,924	177		326					326	*3,421			3,421						
Missouri	9,439	18	9,457	698		3,377	261	4,877	94	4,971	8,609		150		150						
Montana	1,546	-30	1,516	104			228				228	1,146	38		1,184						
Nebraska	2,442	63	2,505	196	11	658	111				769	1,529			1,529						
Nevada	265	2	267	28		150	4	85		85	239										
New Hampshire	2,711	36	2,747	138	3	2,247	77				2,324	161		[12] 118	282						
New Jersey	20,204	-128	20,076	1,754		7,289	379				7,668	5,465		1,155	6,620	351		3,683			3,683
New Mexico	1,643	242	1,885	140	122	769	81				850	209	5		214		559				559
New York [13]	47,124	323	47,447	2,441	166	13,783	800	6,117		6,117	20,700	*9,812	4,565		14,377	3,066	6,697				6,697
North Carolina	7,211	236	7,447	446		[5] 4,780	98	2,026	97	2,123	7,001	([5])									
North Dakota	1,523	18	1,541	152	10	566	16	161		161	743	636			636						
Ohio	27,204	-961	26,243	2,634		6,220	681				6,901	*11,036	5,264		16,300					408	408
Oklahoma	5,779	1	5,780	795	90	1,344	410				1,754	*2,468	673		3,141						
Oregon	2,922	8	2,930	385		1,170	67	857		857	2,094	*424	4		428	23					
Pennsylvania	34,513	-1,647	32,866	1,799		25,086	2,914	2,476		2,476	30,476					108				483	483
Rhode Island [13]	2,778	115	2,893	269		817	81	120		120	1,048					12	1,564				1,564
South Carolina	1,633		1,633	139		504	361	165	464	629	1,494										
South Dakota	1,983	21	2,004	89	19	312	8				320	1,254			1,254	3	319				319
Tennessee	4,173	-13	4,160	284		3,352	367				3,719	157			157						
Texas	20,263	3	20,266	967	419	6,206	679				6,885	*11,995			11,995						
Utah	1,097	467	1,564	126	11			547		547	547	560	240		800		80				80
Vermont	2,365	25	2,390	53		1,211	107	262		262	1,580	739			739	5				13	13
Virginia	6,134	22	6,156	551		4,904	387	258		258	5,549		56		56						
Washington	3,262	47	3,309	366		2,530	401				2,931	12			12						
West Virginia	5,498	8	5,506	208		[5] 2,322	36	2,940		2,940	5,298	([5])									
Wisconsin	13,001	-15	12,986	930	33	6,057			1,478	1,478	7,535	2,590	425		3,015	63	1,410				1,410
Wyoming	601	-4	597	24		395	12	166		166	573										
District of Columbia	2,145		2,145	110	107								508		508		1,420				1,420
Total	388,825	400	389,225	31,088	2,006	143,851	16,611	43,086	11,224	54,310	214,772	90,060	13,362	1,367	104,789	3,888	21,560	4,474	5,743	905	32,682

[1] Amounts distributed during the calendar year often differ from actual collections because of undistributed funds and lag between accounts of collecting and expending agencies.

[2] In many States the proceeds of highway user taxes are placed in a common fund from which a distribution is made. The amounts so distributed have been prorated in proportion to the receipts not otherwise dedicated. See tables on pp. 127 and 129.

[3] Collection expenses in many States include service charges deducted by county and local collectors.

[4] Where reported separately from collection expenses, funds allotted for collection of motor-fuel tax, payments to auto-theft fund, and miscellaneous expenses of motor-vehicle regulation are shown in this column.

[5] The following allotments for construction and maintenance of county roads under State control are included in State highway purposes: Delaware, $151,000; North Carolina, $2,419,000; West Virginia, $525,000.

[6] Reimbursement to local units of government for amounts spent on roads now on State system.

[7] In States indicated by asterisk (*) law provides that these funds may also be used for service of local highway obligations. Amounts so used not reported separately. In Colorado funds may be used on both State and local roads.

[8] This column shows specific allotments for city streets. Where reported separately, funds allotted for urban extensions of State highway system are included in allotmentsf or State highway purposes.

[9] To State general funds except in the following States: Alabama, county and municipal general funds; California, general funds of counties and cities, $3,933,000; New Mexico, county general funds, $349,000; Wisconsin, towns, cities, and villages in lieu of personal property taxes formerly imposed on motor vehicles, $1,407,000. Allocations to local general funds may have been used in part for highways, but such amounts not reported.

[10] For the following purposes: Delaware, C. C. C. ditching; Ohio, hospitalization of indigents injured in motor-vehicle accidents; Pennsylvania, aircraft landing fields, $440,000, and cooperative work other departments, $43,000; Vermont, debt service on nonhighway portion of flood-relief bonds.

[11] Includes debt service on emergency relief bond issues prorated in proportion to use of proceeds for State highway, local road, and nonhighway purposes.

[12] Service of highway relief bonds, a State obligation incurred for improvement of local roads.

[13] Appropriations for highway purposes out of State general fund have been credited against payments of motor-fuel tax and motor-vehicle registration fees to the general funds and prorated in proportion to net receipts from highway user taxes not otherwise dedicated.

DISPOSITION OF STATE MOTOR-CARRIER TAX RECEIPTS, 1938

[Compiled for calendar year from reports of State authorities]

State	Net total receipts of calendar year	Adjustments due to undistributed funds, etc.[1]	Net total funds distributed[2]	Expenses of collection and administration	For State highway purposes: Construction, maintenance, and administration[3]	For State highway purposes: State highway police	Service of State highway obligations: State highway bonds and notes	Service of State highway obligations: State-assumed local obligations[4]	Service of State highway obligations: Total	Total for State highway purposes	For local roads and streets[5]: For work on county and local roads[3]	For local roads and streets: For work on city streets[6]	For local roads and streets: Service of local highway obligations	For local roads and streets: Total	For other highway purposes (park and forest roads, etc.)	For nonhighway purposes: To general funds	For nonhighway purposes: For education	For nonhighway purposes: Total
	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars
Alabama	201	42	243	51	189					189								
Arizona	166		166	35	126	5				131								
Arkansas	1		1		1					1								
California	2,735	234	2,969	399	119					119	90			90		2,361		2,361
Colorado	591	−15	579	107	190	13	56		56	259	213			213				
Connecticut	253	−39	214		99			22	22	121	93			93				
Delaware	(7)																	
Florida	275	−10	265	55								9	195	204			6	6
Georgia	74	15	89	59	30					30								
Idaho	80	−2	78	31	17	30				47								
Illinois	(7)																	
Indiana	767	121	888	132	319					319	308	87		395		42		42
Iowa	537		537	158							*379			379				
Kansas	1,167	10	1,177	251	571	11	20	62	82	664	222			222				
Kentucky	330	−1	329	50	237	5				212	37			37				
Louisiana	11		11	11														
Maine	19	−3	16	16														
Maryland	(7)																	
Massachusetts	99	−1	98	60												38		38
Michigan	427	98	525	242	283					283								
Minnesota	40		40	39	1					1								
Mississippi	129		129	16							* 113			113				
Missouri	492	−396	96	96														
Montana	42	−9	33	33														
Nebraska	47		47	47														
Nevada	193		193	12	176	4		1	1	181								
New Hampshire	3		3	3														
New Jersey	74	7	81		81					81								
New Mexico	177	10	187	50	137					137								
New York	(7)																	
North Carolina	253		253	9	[3] 166	4	71	3	74	244	(3)							
North Dakota	17		17	17														
Ohio	469	142	611	107	276					276	228			228				
Oklahoma	1,464	11	1,175	127	709					709	639			639				
Oregon	1,069	−15	1,051	198	383	14	281		281	708	* 139	2		141	7			
Pennsylvania	13		13		13					13								
Rhode Island	10		10	10														
South Carolina	230	72	302	53	222					222		27		27				
South Dakota	177	−19	158	40	114					114					4			
Tennessee	398	−3	395	86	211					211	21			21		77		77
Texas	108	−1	107	100	7					7								
Utah	9	7	16	5	11					11								
Vermont	(7)																	
Virginia	253	−1	252	38	168		9		9	177		2		2		35		35
Washington	189		189	189														
West Virginia	79		79		30		49		49	79								
Wisconsin	2,014	−11	2,003	433												1,570		1,570
Wyoming	220		220	33	181	6				187								
District of Columbia	216		216													216		216
Total	16,421	243	16,664	3,411	5,367	122	486	88	574	6,063	2,182	127	195	2,504	11	4,339	6	4,345

[1] Amounts distributed during the calendar year often differ from actual collections because of undistributed funds and lag between accounts of collecting and expending agencies.
[2] In many States the proceeds of highway user taxes are placed in a common fund from which a distribution is made. The amounts so distributed have been prorated in proportion to the receipts not otherwise dedicated. See tables pp. 127 to 128.
[3] Approximately $84,000 allotted for use on county roads under State control in North Carolina included in State highway purposes.
[4] Reimbursement to local units of government for amounts spent on roads now on State system.
[5] In States indicated by star (*) law provides that these funds may also be used for service of local highway obligations. Amounts so used not reported separately. In Colorado funds may be used on both State and local roads.
[6] This column shows specific allotments for city streets. Where reported separately, funds allotted for urban extensions of State highway system are included in allotments for State highway purposes.
[7] No special taxes on motor carriers reported.
[8] Ton-mile and passenger-mile taxes paid by motor carriers in lieu of registration fees included in motor vehicle receipts. Table on p. 128.

DISPOSITION OF RECEIPTS FROM STATE IMPOSTS ON HIGHWAY USERS, 1938

[Compiled for calendar year from reports of State authorities]

State	Net total receipts of calendar year [1]	Adjustments due to undistributed funds, etc. [2]	Net total funds distributed	Expenses of collection and administration [3]	For State highway purposes: Construction, maintenance, and administration [4]	For State highway purposes: State highway police	For State highway purposes: Service of State highway obligations: State highway bonds and notes	For State highway purposes: Service of State highway obligations: State-assumed local obligations [5]	For State highway purposes: Service of State highway obligations: Total	Total for State highway purposes	For local roads and streets [6]: For work on county and local roads [4]	For local roads and streets: For work on city streets [7]	For local roads and streets: Service of local highway obligations	For local roads and streets: Total	For other highway purposes (park and forest roads, etc.)	For nonhighway purposes: To general funds [8]: Motor-fuel inspection fees, dealers' licenses, etc.	For nonhighway purposes: To general funds: All other highway user imposts	For nonhighway purposes: For relief of unemployment or destitution	For nonhighway purposes: For education	For nonhighway purposes: For other specific purposes [9]	For nonhighway purposes: Total
	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars
Alabama	18,094	507	17,587	639	5,300	402	4,362		1,362	10,064	6,161			6,161			723				723
Arizona	5,485	−1	5,484	335	3,713	158				3,871	*1,273			1,273						5	5
Arkansas	13,001		13,001	380	3,204	120	5,237	2,898	8,135	11,459	1,042		101	1,143		19					19
California	73,782	1,190	74,972	4,605	31,493	3,000	3,876		3,876	38,369	*16,289	3,757		20,046			8,969	2,983			11,952
Colorado	10,603	56	10,659	654	4,979	333	1,449		1,449	6,761	3,241			3,244							
Connecticut	16,106	20	16,126	1,028	9,623	325		2,093	2,093	12,041	3,057			3,057							
Delaware	3,289	−67	3,222	108	[4] 1,971	250	165	504	669	2,890	([4])	12		12	208					4	4
Florida	29,939	21	29,960	866	10,229	29		9,288	9,288	19,546		9	195	204		309	1,629		7,377	29	9,344
Georgia	21,681	10	21,691	954	11,176	279		2,666	2,666	14,121	3,361			3,361					3,255		3,255
Idaho	6,545	24	6,569	115	4,321	147				4,468	*1,986			1,986							
Illinois	58,479	614	59,093	1,983	17,495	1,377	9,671	24	9,695	28,567	10,855	6,021	2,195	19,071		164	55	6,895	2,358		9,472
Indiana	33,172	3,827	29,345	1,333	13,482	707				14,189	9,391	2,609		12,000		384	1,439				1,823
Iowa	25,568	−17	25,551	1,243	9,878			8,207	8,207	18,085	*6,223			6,223							
Kansas	15,158	301	15,459	1,084	9,268	173	334	1,000	1,334	10,775	3,600			3,600							
Kentucky	17,460	−9	17,451	577	12,709	303				13,012	2,601			2,601			1,261				1,261
Louisiana	21,530	188	21,718	336	3,294	334	10,659		10,659	14,287								4,777	1,159	1,159	7,095
Maine	9,159	47	9,206	180	5,481	290	2,429		2,429	8,200	826			826							
Maryland	14,998		14,998	430	6,326	418	2,605		2,605	9,349	1,034	3,252	509	4,795				421			424
Massachusetts	27,052	6	27,058	1,751	8,541	350	[10] 3,202		3,202	12,093	9,984		[10] 461	10,445	922		38	[10] 1,809			1,847
Michigan	48,966	−25	48,941	2,419	15,997	378	4,982		4,982	21,357	*24,916			24,916		5	244				249
Minnesota	28,987	22	29,009	643	17,407	320	1,706	2,331	4,037	21,764	*6,460			6,460		24	118				142
Mississippi	14,311	−37	14,274	321	3,022	21	2,916		2,916	5,959	*7,724			7,724	270						
Missouri	21,567	−378	21,189	916	7,791	601	11,250	217	11,467	19,859		347		347		67					67
Montana	6,040	81	6,121	158	3,573	228	972		972	4,773	1,146	38		1,184						6	6
Nebraska	13,628	63	13,691	398	7,159	111				7,270	*4,559	364		4,923				1,100			1,100
Nevada	1,660	2	1,662	43	1,490	36	85	8	93	1,619											
New Hampshire	6,012	34	6,046	144	4,411	113	824		824	5,348	322		[11] 232	554							
New Jersey	42,640	−264	43,376	2,017	10,966	566	7,482		7,482	19,014	8,160		1,725	9,885	524			9,098	1,190	648	10,936
New Mexico	5,910	203	6,113	404	3,107	81	1,748		1,748	4,936	209	5		214			559				559
New York [12]	113,319	921	114,240	2,711	24,274	1,409	10,772		10,772	36,455	*18,590	6,224		24,814	5,400		44,860				44,860
North Carolina	31,772	104	31,876	563	[4] 20,723	426	8,785	421	9,206	30,355	([4])					885	7			66	958
North Dakota	3,858	114	3,972	255	1,772	16	505		505	2,293	1,411			1,411			13				13
Ohio	73,655	−583	73,072	3,129	25,529	681				26,210	*19,840	11,627		31,467					11,858	408	12,266
Oklahoma	21,153	−44	21,109	1,428	12,123	410				12,533	*6,471	673		7,144		4					4
Oregon	13,829	−37	13,792	621	6,042	370	4,428		4,428	10,840	*2,191	23		2,211	117						
Pennsylvania	86,527	−1,734	84,793	2,148	51,022	5,925	5,035		5,035	61,982	*6,503			6,503	220			12,958		982	13,940
Rhode Island [12]	6,283	65	6,348	301	1,951	187	276		276	2,414					28		3,605				3,605
South Carolina	13,325	72	13,397	243	4,886	361	1,525	4,295	5,820	11,067	*1,870	27		1,897		190					190
South Dakota	6,562	2	6,564	237	4,171	8				4,179	1,694			1,694	40	18	319			77	414
Tennessee	23,802	841	24,643	613	5,974	367	7,038	1,931	8,969	15,310	5,555			5,555		1,000	77			2,088	3,165
Texas	63,118	−161	62,957	2,179	26,583	679		10,475	10,475	37,737	*12,575			12,575					10,466		10,466
Utah	4,584	399	4,983	160	3,279	117	547		547	3,943	560	240		800			80				80
Vermont	4,895	55	4,950	56	2,599	107	563		563	3,269	1,586			1,586	10					29	29
Virginia	23,008	−26	22,982	613	[4] 20,865	387	620		620	21,872	[4] 289	136		425			35			37	72
Washington	18,882	47	18,929	581	8,804	401	[10] 121	51	172	9,377	*6,185	1,673	[10] 98	7,956				[10] 1,015			1,015
West Virginia	14,974	7	14,981	221	[4] 7,080	36	7,644		7,644	14,760	([4])										
Wisconsin	34,462	406	34,868	1,662	15,939			3,889	3,889	19,828	6,816	1,118		7,934	167		5,277				5,277
Wyoming	3,299	−5	3,294	82	2,246	69	278		278	2,593	619			619							
District of Columbia	4,881	−1	4,880	217								3,016		3,016		11	1,636				1,647
Total	1,177,010	−1,808	1,175,202	44,084	493,268	23,406	124,091	50,298	174,389	691,063	227,178	41,171	5,516	273,865	7,906	3,080	70,944	41,059	37,663	5,538	158,284

[1] Includes receipts from motor-fuel taxes, motor-vehicle fees and fines, and special imposts on motor vehicles operated for hire (motor-carrier taxes). See tables on pp. 127 to 129 which give distribution of receipts separately.

[2] Amounts distributed during the calendar year often differ from actual collections because of undistributed funds and lag between accounts of collecting and expending agencies.

[3] Includes expenses of collection and administration of motor-fuel tax, motor-vehicle fees, and motor-carrier taxes, and miscellaneous expenses of motor-vehicle regulation.

[4] The following allotments for construction and maintenance of county roads under State control are included in State highway purposes: Delaware, $446,000; North Carolina, $10,488,000; Virginia, $7,500,000; West Virginia, $2,407,000.

[5] Reimbursement to local units of government for amounts spent on roads now on State system.

[6] In States indicated by star (*) law provides that these funds may also be used for service of local highway obligations. Amounts so used not reported separately. In Colorado funds may be used on both State and local roads.

[7] This column shows specific allotments for city streets. Where reported separately, funds allotted for urban extensions of State highway system are included in allotments for State highway purposes.

...ifornia, general funds of counties and cities, $3,933,000; New Mexico, county general funds, $349,000; Wisconsin, towns, cities, and villages in lieu of personal property tax formerly imposed on motor vehicles, $3,704,000. Allocations to local general funds may have been used in part for highways, but such amounts not reported.

[9] For the following purposes: Arizona, irrigation engineering expenses; Delaware, Civilian Conservation Corps ditching; Florida, aviation; Louisiana, harbor improvement; Montana, labor furnished counties; New Jersey, service of institutional construction bonds, $514,000, and Department of Commerce and Navigation, $134,000; North Carolina, State Probation Commission; Ohio, hospitalization of indigents injured in motor-vehicle accidents; Pennsylvania, aircraft landing fields, $895,000, and cooperative work other departments, $87,000; South Dakota, payment on real estate bonds; Tennessee, debt service on nonhighway bonds, $2,081,000, and aviation, $7,000; Vermont, debt service on nonhighway portion of flood relief bonds; Virginia, aviation.

[10] Includes debt service charges on emergency relief bond issues prorated in proportion to use of proceeds for State highway, local road, and nonhighway purposes.

[11] Service of highway relief bonds, a State obligation incurred for improvement of local roads.

[12] Appropriations for highway purposes out of State general funds have been credited against payments of motor-fuel tax and motor-vehicle fees to the general fund and prorated in proportion to net receipts from high-

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF JULY 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 315,780	$ 157,890	7.8	$ 8,626,458	$ 4,298,788	325.0	$ 300,490	$ 148,840	4.6	$ 3,139,131
Arizona				1,374,864	974,827	62.6	810,226	507,403	31.3	1,443,719
Arkansas	440,705	440,320	31.3	2,830,677	2,827,167	188.8	196,261	194,176	4.3	1,739,214
California	1,084,411	584,194	12.5	4,591,890	2,536,694	52.1	777,908	390,071	25.0	4,138,966
Colorado	444,860	246,057	9.1	3,746,322	2,087,712	82.8	181,648	100,818	8.3	2,168,233
Connecticut				1,610,878	798,814	18.1	655,381	326,776	4.9	1,240,344
Delaware				1,076,616	520,270	32.6	899,862	449,931	5.9	1,008,742
Florida	121,000	60,500	1.4	2,863,020	1,431,510	49.2	1,213,842	606,696	23.3	2,870,545
Georgia	843,180	421,590	50.5	5,709,912	2,854,956	288.4	2,795,862	1,397,931	169.9	5,578,139
Idaho	79,917	47,830	5.0	1,984,872	1,198,289	50.3	589,438	335,651	75.3	1,315,743
Illinois	888,326	441,870	24.3	8,687,392	4,343,048	181.9	2,979,466	1,499,138	62.9	3,168,887
Indiana	981,766	490,783	16.3	5,513,924	2,704,662	118.4	1,638,266	819,058	42.3	2,356,202
Iowa				5,182,204	2,287,233	185.7	1,111,098	516,975	64.3	1,356,234
Kansas	382,175	191,087	21.9	3,330,532	1,657,561	146.4	4,024,231	2,011,236	232.6	3,996,345
Kentucky	153,876	76,938	8.4	4,162,856	2,079,872	84.0	1,202,551	601,275	65.9	2,923,328
Louisiana				12,119,911	3,154,050	53.3	1,288,985	607,838	37.3	2,606,854
Maine	180,830	90,414	2.3	1,620,494	810,247	37.8	1,000,566	500,283	23.2	346,446
Maryland	30,000	15,000	.3	3,034,521	1,504,611	50.2	1,136,000	560,505	16.2	1,812,219
Massachusetts				3,348,765	1,671,693	25.1	1,737,109	865,037	12.1	2,480,941
Michigan	567,420	283,710	19.6	4,444,881	2,219,993	128.5	1,679,800	741,700	35.0	3,044,809
Minnesota	152,180	76,090	12.2	6,781,774	3,372,040	375.9	1,828,298	912,294	137.5	3,697,382
Mississippi	344,700	88,820	17.4	7,522,098	2,732,821	308.6	2,335,920	1,032,934	79.5	2,160,470
Missouri				5,490,632	2,724,704	204.3	2,164,833	1,010,228	66.3	4,621,710
Montana	175,536	98,737	6.7	3,565,472	2,017,571	188.5	33,284	18,876		4,476,608
Nebraska	88,656	44,328	6.1	5,470,556	2,734,909	462.3	3,111,076	1,555,537	322.0	2,659,155
Nevada	641,752	556,259	26.4	350,219	301,188	18.9	259,656	223,231	10.1	1,395,617
New Hampshire				967,275	476,119	20.8	632,282	313,281	23.0	931,103
New Jersey	489,290	244,645	6.0	3,017,436	1,507,168	23.9	730,970	315,485	1.5	2,253,487
New Mexico	7,493	4,569		1,964,676	1,202,567	99.2	239,261	149,320	35.8	1,462,380
New York	467,350	233,675	8.9	13,183,320	6,333,352	222.3	3,313,250	1,438,140	45.5	2,726,494
North Carolina	165,600	82,800	9.2	6,462,153	3,225,477	386.7	1,409,850	653,930	70.3	2,098,375
North Dakota	78,830	42,219	14.6	177,260	94,870	11.5	3,289,179	1,762,926	323.6	3,431,003
Ohio	174,940	87,470	.8	10,199,046	5,028,634	112.4	1,123,660	561,830	16.3	7,314,136
Oklahoma	22,269	10,037		1,955,411	1,036,437	29.7	2,992,370	1,591,844	134.9	3,682,030
Oregon	98,106	59,730	7.4	2,897,043	1,755,292	120.6	121,321	72,690	2.3	2,191,803
Pennsylvania	656,450	328,225	8.5	9,768,921	4,698,808	93.9	2,635,456	1,316,599	30.0	4,651,364
Rhode Island	38,190	19,095	1.1	719,556	359,471	8.7	436,736	218,000	4.7	1,045,633
South Carolina	99,300	45,000	1.7	2,762,734	1,232,487	84.6	192,700	88,000	23.9	2,407,197
South Dakota	432,300	239,000	51.3	4,060,749	2,245,380	379.9	1,450,540	839,000	131.4	3,430,595
Tennessee	25,320	12,660	.5	4,282,770	2,141,385	120.6	321,220	160,610	7.8	4,473,151
Texas	1,859,403	905,730	101.9	11,225,771	5,557,713	524.8	1,168,907	573,515	92.4	6,995,777
Utah	497,290	353,550	17.2	1,895,370	1,370,460	87.8	150,195	109,230	12.1	937,159
Vermont	124,223	57,881	.7	583,432	291,622	17.1	163,610	81,615	5.1	638,311
Virginia	306,256	152,583	8.2	2,484,063	1,239,362	69.9	1,731,176	809,445	36.9	1,108,183
Washington	143,966	75,800	1.6	3,092,282	1,549,358	30.4	1,341,311	640,500	21.7	996,836
West Virginia	140,327	73,650	3.9	2,271,985	1,153,230	54.0	1,512,172	755,741	42.0	1,938,953
Wisconsin	1,202,417	590,300	49.0	6,981,670	3,427,310	191.4	1,838,860	895,190	91.5	1,776,822
Wyoming	137,430	84,930	20.6	1,433,215	888,065	118.0	441,810	278,875	51.0	936,642
District of Columbia				136,024	68,012	.8	267,900	133,950	1.7	285,538
Hawaii	9,596	2,580		1,122,170	538,595	17.3	571,307	281,993	9.8	1,058,510
Puerto Rico	204,550	101,475	3.5	1,428,248	709,855	29.9	224,982	111,005	4.1	452,460
TOTALS	15,297,966	8,220,021	596.1	210,114,320	103,976,259	6,575.9	64,253,082	32,087,152	2,779.3	126,969,925

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF JULY 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 186,105	$ 91,750	13.7	$ 873,345	$ 349,000	19.4	$ 61,700	$ 30,850	1.2	$ 753,656
Arizona				266,190	175,012	29.3	15,912	11,475		353,769
Arkansas				460,228	456,775	59.0	220,976	220,857	29.0	342,257
California	89,549	50,796	11.5	1,053,147	542,386	36.8				761,745
Colorado	103,990	58,120	6.2	589,183	305,367	19.5	138,092	77,829	2.9	144,901
Connecticut				172,794	72,417	2.9				286,249
Delaware	35,480	17,740	7.8	45,360	22,680	9.7	73,930	36,965	7.8	231,250
Florida				903,022	446,794	32.1	86,600	43,300	5.6	374,950
Georgia	116,720	58,360	18.5	335,989	167,994	36.7	145,180	72,590	20.3	1,084,114
Idaho				251,098	142,939	11.2	242,537	88,430	29.2	204,329
Illinois	267,000	133,500	16.0	1,234,632	563,316	67.9	424,000	212,000	30.0	750,529
Indiana				1,065,070	532,535	89.5	247,860	104,198	13.5	651,525
Iowa	6,407	2,950	7.1				109,588	51,154	37.6	1,625,703
Kansas				47,588	23,794	11.7	461,942	230,971	39.0	1,325,016
Kentucky	47,153	11,285	5.0	1,131,615	316,760	71.1	839,690	291,418	72.0	229,113
Louisiana	160,157	67,560	15.1	551,474	240,620	42.6	271,384	125,120	23.2	398,713
Maine	126,024	63,012	6.0	254,556	127,548	15.8	133,060	64,294	7.3	3,807
Maryland	25,000	12,500	4.3	177,670	87,835	11.8	186,000	63,355	11.7	371,991
Massachusetts				344,984	171,164	7.6	372,470	184,000	7.5	434,504
Michigan	136,800	68,400	3.3	1,168,304	567,952	79.1	234,200	117,100	27.3	983,402
Minnesota	46,322	23,161	6.2	793,472	394,692	64.1	97,614	48,807	14.7	1,191,845
Mississippi	176,500	88,250	6.8	330,762	165,381	27.0	576,700	272,565	50.7	624,670
Missouri	97,140	48,570	12.5	702,684	339,648	77.8	608,418	262,925	80.0	616,015
Montana				730,363	414,191	63.2	174,315	98,870	12.8	818,967
Nebraska	76,688	38,344	17.6	666,778	324,274	126.2	542,425	260,219	83.9	393,396
Nevada	92,183	79,909	8.3	28,021	24,275	7.2	51,737	44,685	9.5	192,987
New Hampshire				62,951	30,804	2.3				189,160
New Jersey				397,240	195,820	13.1	98,920	49,460	6.7	546,678
New Mexico				450,024	271,508	28.1	61,133	38,153	12.7	214,884
New York	302,200	151,100	18.7	1,609,300	803,350	87.8	701,100	264,500	12.5	708,952
North Carolina	47,440	23,720	3.6	1,204,044	602,000	113.1	102,590	51,295	10.4	352,495
North Dakota	80,460	43,092	8.1	34,570	18,514	.1	42,770	22,907	8.2	875,949
Ohio				672,030	342,790	35.2	377,800	188,900	17.4	1,763,662
Oklahoma	73,190	38,943	.8	9,796	5,213		703,615	351,194	37.5	932,095
Oregon	80,550	48,520	7.7	630,555	375,107	64.6	59,356	35,620	3.3	270,110
Pennsylvania	457,286	222,047	28.8	1,664,239	820,934	87.5	724,700	358,050	25.9	453,095
Rhode Island				99,335	49,644	2.2	72,008	36,004	.2	98,167
South Carolina	69,340	19,400	8.1	514,567	219,669	48.8	169,800	66,200	12.4	280,651
South Dakota				12,340	6,790					1,051,260
Tennessee	166,160	59,380	9.4	558,498	263,779	22.6				863,178
Texas	380,390	184,736	53.8	1,880,405	894,891	170.5	325,509	154,835	40.5	1,080,139
Utah	22,390	10,155	2.2	165,595	96,708	25.0	57,245	31,000	12.2	194,199
Vermont	25,258	12,203	.8	101,290	50,645	3.7	65,800	32,900	2.6	80,772
Virginia	251,400	123,937	27.9	296,334	140,798	27.9	305,076	130,573	22.3	278,412
Washington	68,078	35,800	3.8	644,006	337,896	42.2	110,651	57,000	11.2	237,306
West Virginia				153,296	76,648	8.3				515,848
Wisconsin	114,273	56,970	17.1	843,931	421,267	14.9	180,651	74,028	4.9	696,482
Wyoming	296,610	183,260	18.4	109,970	67,950	3.9	343,427	211,171	33.6	55,604
District of Columbia				3,192	1,096		54,500	27,250	.6	44,779
Hawaii	22,900	11,450	1.3	170,080	85,040	4.6	113,020	56,510	2.5	167,000
Puerto Rico				178,505	86,825	10.4	55,188	27,140	2.1	82,069
TOTALS	4,247,143	2,138,920	376.4	26,644,422	13,241,035	1,836.0	11,041,189	5,278,667	896.4	27,182,349

PUBLICATIONS of the PUBLIC ROADS ADMINISTRATION

(Formerly the *BUREAU OF PUBLIC ROADS*)

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Agency and as the Agency does not sell publications, please send no remittance to the Federal Works Agency.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1938. 10 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 25 cents.

No. 191MP. . Roadside Improvement. 10 cents.

No. 272MP. . Construction of Private Driveways. 10 cents.

No. 279MP. . Bibliography on Highway Lighting. 5 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

Federal Legislation and Rules and Regulations Relating to Highway Construction. 15 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. 60 cents.

Highways of History. 25 cents.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.

No. 265T. . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP. . Bibliography on Highway Safety.

House Document No. 272 . . . Toll Roads and Free Roads.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration (formerly the *Bureau of Public Roads*), classified according to subject and including the more important articles in Public Roads, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF JULY 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR		NUMBER			UNDER CONSTRUCTION		NUMBER			APPROVED FOR CONSTRUCTION		NUMBER			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 392,350	$ 391,400	2	1		$ 856,112	$ 847,584	13			$ 38,400	$ 37,200	1	1	1	$ 824,073
Arizona						469,516	443,841	5								281,092
Arkansas						189,891	189,891	3			494,487	485,634	3		4	806,022
California	139,517	139,517	1			1,632,128	1,631,033	10								1,296,732
Colorado	76,815	76,815	1		4	422,168	422,168	3		2	160,488	135,936	1		19	817,198
Connecticut						172,722	161,008		1							850,557
Delaware						9,150	9,150	2			2,320	2,320			1	513,891
Florida						503,994	503,994	3			130,037	129,202	1		1	1,032,656
Georgia						427,280	427,280	7			137,020	137,020	1	2	10	2,301,720
Idaho						314,492	282,961	4								452,544
Illinois	336,240	336,240	5		13	2,494,305	2,437,305	13	3	36	603,406	533,837	2		21	2,364,764
Indiana	208,408	208,408	2		8	639,775	639,775	1	1	61	491,055	491,055	2	1	14	963,192
Iowa	14,045	13,200	1			316,682	278,506	6			608,481	570,200	8		105	1,240,038
Kansas	172,163	172,163	2			762,449	762,449	9			473,030	473,030	5		8	1,075,447
Kentucky						591,947	591,947	9	4		829,350	778,903	7	1		550,353
Louisiana						586,952	586,158	5			648,384	617,362	14			592,427
Maine	100,000	100,000	2			360,545	360,545	2	3		90,800	90,800	1			220,802
Maryland						75,197	75,197	1			262,200	165,407	1		15	986,891
Massachusetts						520,631	519,367	4	2		14,320	14,320	1			1,713,382
Michigan	64,000	64,000	1	1		758,526	758,526	5	1		438,600	438,600	3		22	1,758,873
Minnesota						1,309,497	1,292,066	7	6	11	95,060	95,060	1		1	1,533,677
Mississippi						606,714	606,714	8			37,300	37,300	1			894,187
Missouri	66,000	66,000	1			1,235,432	1,231,642	7	1		335,060	335,060	1	2		1,677,258
Montana	351,251	351,251	3			546,443	546,443	7								327,257
Nebraska						931,327	931,327	24			490,208	490,208	4		41	517,888
Nevada	109,707	109,707		1		58,621	58,621	1			30,558	30,558			11	112,509
New Hampshire						102,389	101,921	7		1	100,837	100,801	2			318,763
New Jersey						493,541	493,541	1	2	1	255,740	255,740	1	1		1,426,875
New Mexico						75,081	75,081	2			2,861	2,861			1	675,857
New York	178,300	177,650		2		2,072,492	2,066,962	6	8		1,013,448	817,697	5	4		3,933,389
North Carolina	105,010	105,010	1		5	1,096,000	1,060,900	6	5	4	386,635	386,635	5		37	972,965
North Dakota	60,590	60,590	1	1		858,712	810,310	11			75,960	75,960	1			395,838
Ohio						1,254,902	1,218,140	13	1		639,680	590,490	3	1	2	3,254,391
Oklahoma	169,000	135,000	2			129,080	129,080			42	217,400	217,400	3		8	2,191,397
Oregon						169,719	164,635	1			135,740	135,740	2			314,891
Pennsylvania						1,967,294	1,755,395	3	3		570,652	362,200	3		2	4,545,633
Rhode Island	103,716	103,716		2		335,075	335,075	1	1							152,459
South Carolina	15,630	14,428		1		607,566	553,050	8	1		226,079	226,079	2	2	35	879,664
South Dakota	3,670	3,670			1	301,010	301,010	3	2	7	29,460	29,460			9	1,117,940
Tennessee	2,700	2,700			1	648,909	648,909	2	2		239,060	239,060	1	2	4	1,320,130
Texas						2,788,465	2,757,002	24	3		576,459	545,080	4		21	2,091,283
Utah						77,578	77,578	2		13	305,879	305,879	1		116	213,367
Vermont	18,879	14,256			2	5,260	5,070			2	10,920	10,920			3	319,709
Virginia						608,509	519,509	8	1	14	240,588	234,588	1	2	6	912,659
Washington	7,502	7,502			3	312,536	311,126	3	1		75,439	75,439		1	9	502,865
West Virginia						370,941	355,181	7			18,800	18,800	1		1	964,852
Wisconsin	58,506	58,500	1			1,449,259	1,404,405	14	1	1	157,191	157,183		1	10	1,147,209
Wyoming						125,553	125,553	1	1	1	13,460	13,460			6	515,939
District of Columbia						292,412	258,868	1								119,318
Hawaii	50,320	50,320	1			132,850	132,850	3	1							359,450
Puerto Rico						394,352	392,150	9								426,676
TOTALS	2,804,319	2,762,043	27	9	37	33,461,981	32,648,799	295	55	196	11,702,852	10,890,484	93	21	544	54,778,949

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D. M. BEACH, *Editor*

Volume 20, No. 7 September 1939

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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ESSENTIAL FEATURES OF TRIAXIAL SHEAR TESTS[1]

BY THE DIVISION OF TESTS, U. S. PUBLIC ROADS ADMINISTRATION

Reported by C. A. HOGENTOGLER, Senior Highway Engineer and E. S. BARBER, Junior Highway Engineer

IN THE DESIGN of retaining walls, three types of earth pressure may be considered.

Without movement of the earth, pressures against the walls, figure 1—A and 1—B, become the "earth pressures at rest" which depend upon the coefficient K, expressed by the relation

$$K=\frac{l}{v}$$

In which

l=lateral pressure,
v=vertical pressure.

However, soil must deform to fail. The pressures it produces at maximum deformation without failure are termed active or passive, depending on the directions of the applied forces responsible.

Wedges (*1, 2*)[2] assumed in the design of retaining walls (fig. 1) have lower boundaries, D—D, on which the soil slips when it shears. Weight of the earth in figure 1—A produces the active earth pressure which forces walls outward and causes D—D to incline at an angle a with the horizontal and b with the vertical. Forcing walls backward as in figure 1—B, produces the passive earth pressure which causes D—D to incline at an angle b with the horizontal and a with the vertical.

The diagram of supporting value of soil under a strip load, considered in a formula published in PUBLIC ROADS (*3*), is shown in figure 1—C.

Beneath each half of the load, which acts like an embankment breaking in the middle, is a triangular diagram of active earth pressure similar to the one shown in figure 1—A. It is assumed that wedges of earth beneath the surface adjacent to the loaded area and subjected to passive earth pressure function like retaining walls to prevent failure of the wedges subjected to the active earth pressure. Therefore, diagrams of passive earth pressure similar to the one shown in figure 1—B are used to complete the diagram of the supporting value of the soil (fig. 1—C). The angle a and its complement b are utilized also in theories suggested for the determination of stresses in embankments (*4*), soil foundations for rigid loads (*5*), and flexible type pavements (*6, 7*).

The coefficient of earth pressure at rest, K, (*8*) depends upon the soil's elasticity rather than its resistance to shear. Active and passive earth pressures in contrast depend upon the soil's cohesion c, and its angle of internal friction ϕ.

EARTH PRESSURES STUDIED EXTENSIVELY

Tests to determine relations of the laterial to applied vertical pressures on soil and their use to furnish design data have become accepted practice.

In 1900 J. A. Jamieson (*9*) a Canadian engineer, utilized manometers as shown in figure 2 to measure both lateral and vertical pressures of grain in model bins. About the same time, E. P. Goodrich, investigating pressures against retaining walls, utilized the apparatus shown in figure 3, and his findings published in 1904 (*10*) are substantiated by later work in this country (*11, 12*) and quite recently by extensive investigations in Germany (*13*).

On January 18, 1933, F. N. Hveem filed an application for letters patent on a stabilometer, figures 4 and 5, to test various sorts of reasonably stiff plastic ma-

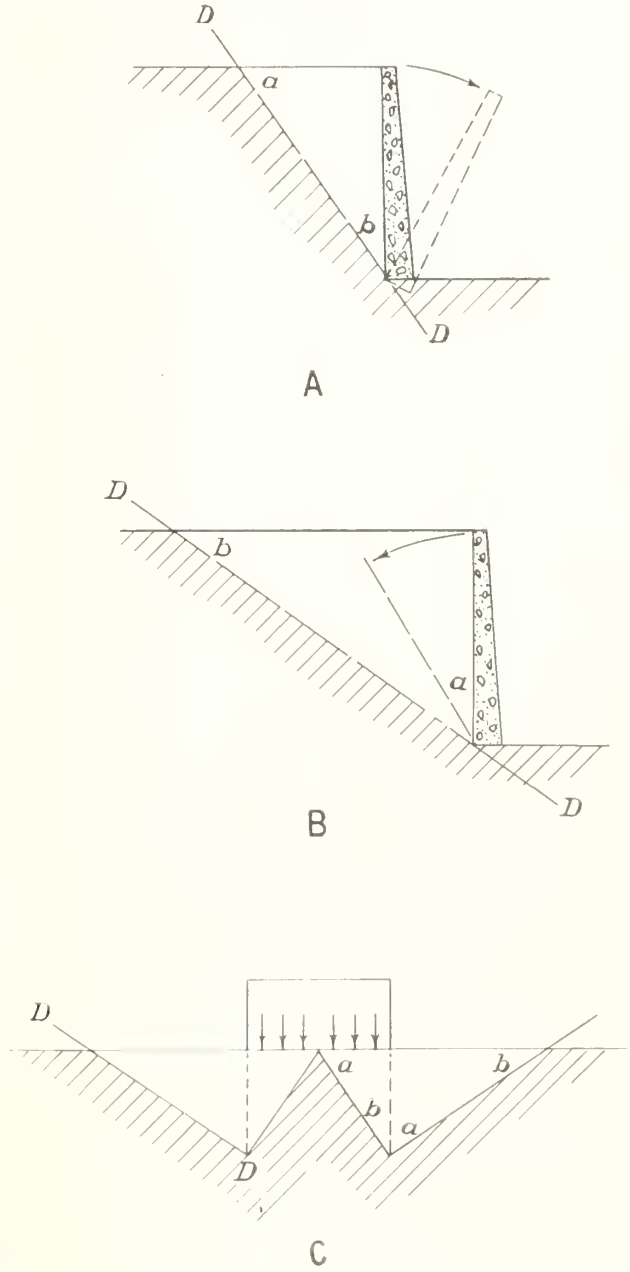


FIGURE 1.—SURFACES OF SLIP ILLUSTRATED.

[1] Paper presented at the annual meeting of the American Society for Testing Materials, Atlantic City, N. J., June 28, 1939.
[2] Italic figures in parenthesis refer to bibliography, p. 153.

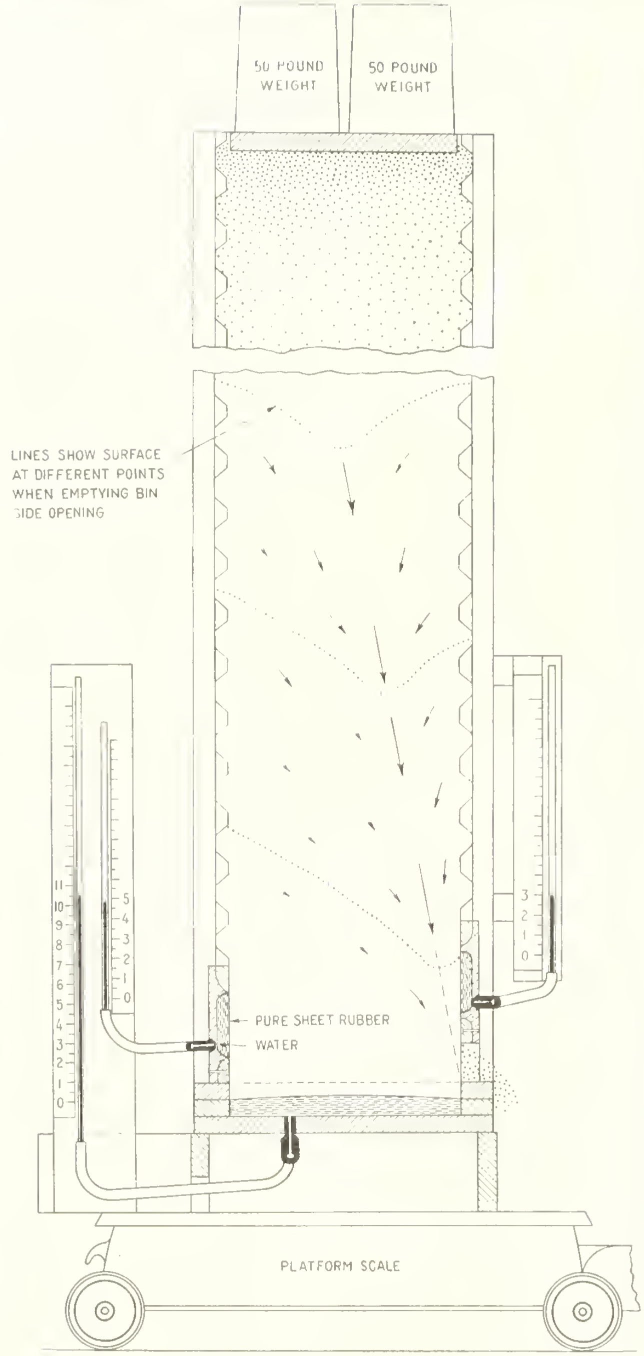


FIGURE 2.—MANOMETER USED BY JAMIESON.

terials, such as clay, soil (to determine bearing values), etc. The apparatus had essential features as follows:

1. Flexible cylinder arranged concentrically within a cylindrical shell, a pressure chamber being formed between the two.

2. Specimens in the flexible cylinder loaded axially and means to measure accompanying changes in the chamber pressures.

3. Means to measure deformations of the specimens in the direction of load and perpendicular to it. The patent [3] was granted April 23, 1935 (*14*).

In Hveem's apparatus the flexible rubber cylinder is attached at both ends to the pressure chamber, which in turn is of metal and filled with a liquid.

[3] U. S. Patent Office No. 1998722.

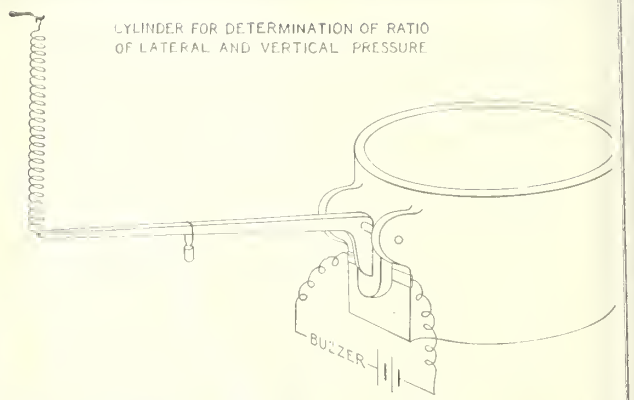


FIGURE 3.—APPARATUS DEVISED BY GOODRICH FOR DETERMINING RATIO OF LATERAL TO VERTICAL PRESSURES.

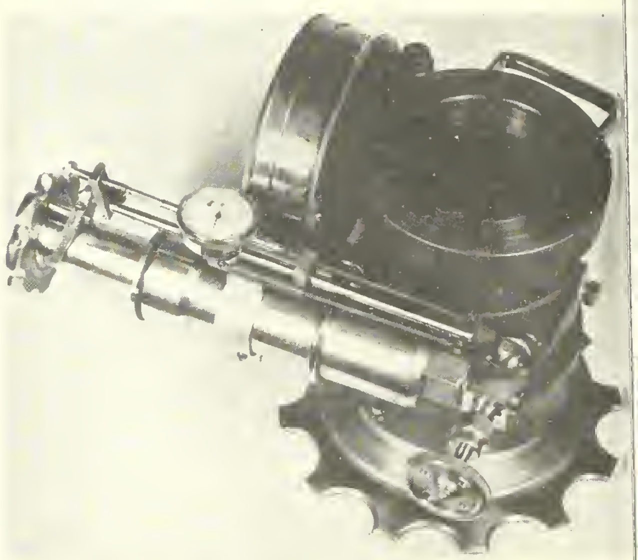


FIGURE 4.—STABILOMETER DEVELOPED BY HVEEM.

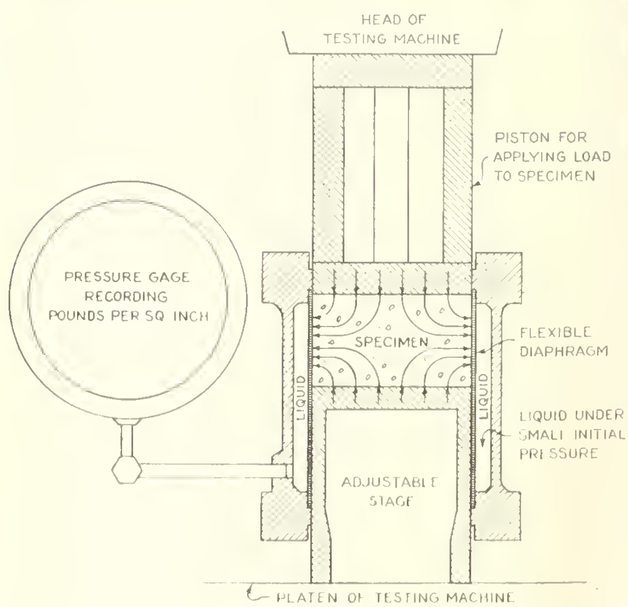


FIGURE 5.—DIAGRAM OF STABILOMETER DEVELOPED BY HVEEM.

In 1934, Leo Jürgenson (*15*) described apparatus in which the rubber was fixed at but one end to the chamber, and which utilized compressed air to maintai

FIGURE 6.—RUBBER SLEEVE AND CLAMPS USED TO ENCASE SAMPLES FOR STABILOMETER TESTS.

constant pressure in the chamber. In the same year Keverling Buisman of Delft, Netherlands, (*16*) suggested the use of transparent material for the outer shell.

Since then modifications of these basic conceptions have been reported by: Delft Laboratories, 1936 (*17*); W. S. Housel, 1936 (*18*); Seibert and Palmer, 1938 (*19*); John D. Watson, 1938 (*20*); Corps of Engineers, United States Army, 1939 (*21*); and the Public Roads Administration, Levi Muir, the Shell Oil Co., and the Bureau of Reclamation in 1939 (*22*).

Purposes of the tests, types of soil investigated, and laboratory facilities necessitated procedures and equipment which varied widely in some respects and yet had enough in common to suggest use of simplified apparatus with interchangeable parts to satisfy all the requirements. Methods employed include a "closed" system which prevents volume change of samples, and an "open" system which permits their swell or consolidation during test. An impervious encasement which prevents entrance or escape of air and water encloses samples in the closed system, and placing them between porous stones provides for the entrance or egress of air and water in the open system.

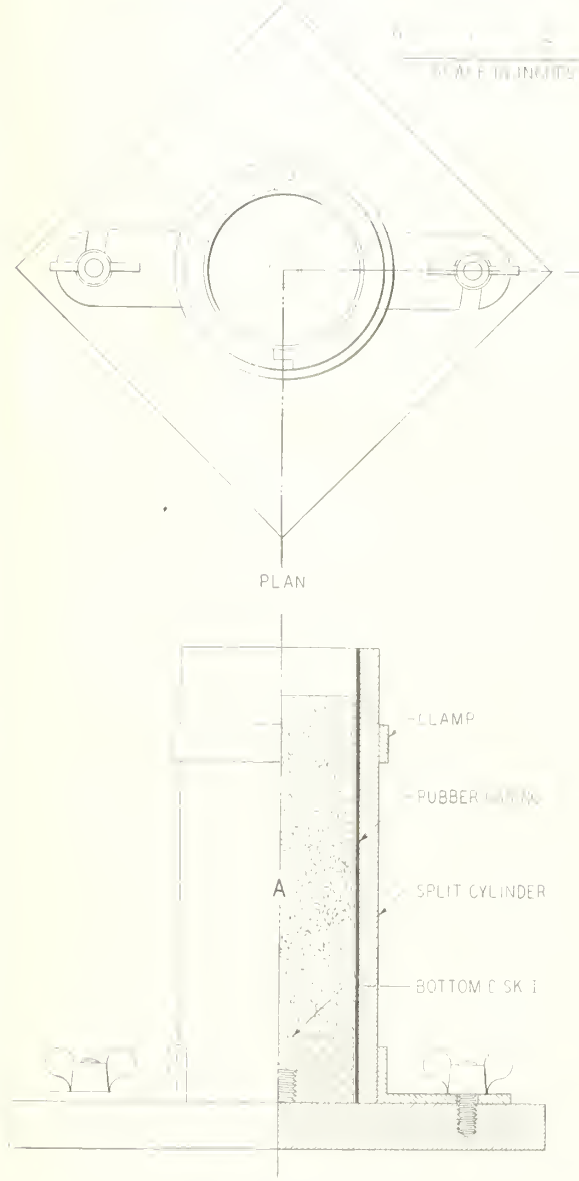


FIGURE 7.—MOLD USED FOR COMPACTING SAMPLES FOR STABILOMETER TESTS.

PRESSURE CHAMBER SUGGESTED FOR USE IN PREPARING SAMPLES

For stabilometer tests, cylindrical samples are encased in rubber sleeves clamped about bakelite disks (fig. 6) which, with or without porous stones, are placed at the ends of the samples. Samples of stabilized soil and embankment materials may be compacted in the apparatus shown in figure 7.

For tests using the closed system, compacted samples are placed in the rubber jackets with impervious disks at the bottom ends, and, after removal from the split cylinder mold, impervious disks are also placed at their upper ends. The clamps are adjusted and threaded studs screwed into the bottom disks as shown in figure 8—B. This assembly can also be used for testing undisturbed samples at their natural moisture contents.

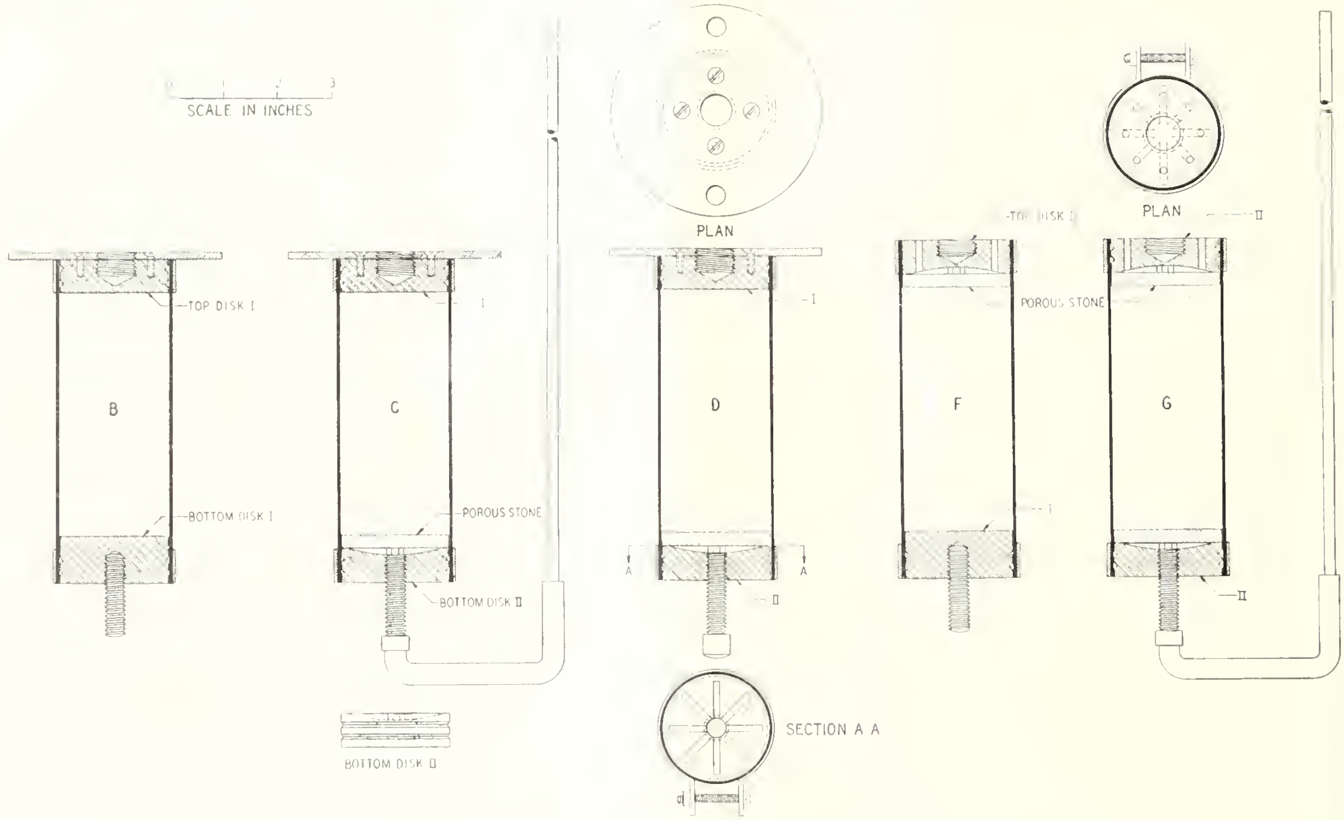


FIGURE 8.—ASSEMBLIES SHOWING POROUS STONES AND DISKS PLACED AT ENDS OF SAMPLES.

Studs afford means to fasten sample assemblies in the stabilometers, and threaded recesses in the top disks afford means for attachment to plungers of the stabilometers.

Determinations of the critical density of sands may be desired also. The critical density has been defined by Arthur Casagrande (*23*) as that density at which a soil can undergo deformation or actual flow without volume change, (see also (*24*)). For this purpose assembly C is suggested.

Assembly G (fig. 8) is suggested for use in determining permeabilities and capillarities of samples by application of water through the porous stone and tube in the bottom disk, which furnishes a connection with the burette. At times, tests on samples at the pore pressures of the pressure chamber may be required. Assembly F is suggested for this purpose.

The apparatus shown in figure 9 is usable in the pretesting of samples for consolidation and swell (see also (*15*, *25*)). Rise of water in the burette, assembly C, discloses the speed and amount of consolidation of samples at the applied air pressures; and drop of water in the burette indicates their swell. Metal guides attached to the top disks are to prevent tilting of samples during such tests.

At the conclusion of the preliminary tests, all spaces in porous stones, disks and tubes at the bottoms of assemblies C and G are filled with water. Disconnecting the burettes and capping the tubes and, for assembly G, replacing the perforated top disk with an impervious one, completes the change to assembly D (fig. 8).

Figure 10 is a diagram of a pressure chamber and sample assembly which is usable in the closed system of test. A nut on the tube fastens the assembly D to the chamber. A similar nut on the threaded stud, assembly B, serves a similar purpose.

Attaching one end only of sample assemblies to the pressure chamber distinguishes the stabilometer, figure 10, as the free rubber type used by Jürgenson, Delft Laboratories, Harvard University, Corps of Engineers, United States Army, and the Bureau of Reclamation.

Harvard University and the Corps of Engineers suggest glycerine as a liquid satisfactory for use in the cylinder. To prevent leakage, Harvard University utilized the stuffing box (fig. 11) and the Corps of Engineers, the bronze bushing (fig. 10).

Relative to experience at Harvard University, John D. Watson (*20*) states:

> It is absolutely essential that friction between this piston and the head be reduced to a negligible amount. At the same time it must be possible to maintain the hydrostatic pressure in the compression chamber constant while a test is in progress. A highly viscous fluid in the compression chamber would be far better than air because air under pressure is very difficult to confine without leakage. Glycerine was chosen because in addition to a high viscosity it is soluble in water and easy to wash off and clean up, and it does not attack rubber. The piston rod is jacketed with graphite steam packing but the packing gland is screwed down so little that the piston rod will fall slowly under its own weight.

Relative to the use of the bronze bushing, a report by the Corps of Engineers (*21*) states:

> The hemp packing box has been eliminated and a bronze bushing substituted in its place. Experience has shown that friction is eliminated thereby and that leakage of glycerine even at high hydrostatic pressures is negligible.

Relative to the closed system, a report (*22*) on the Bureau of Reclamation's apparatus states:

> The specimens are encased in thin-wall rubber tubes which clamp to metal end plates, thus keeping the water which completely fills the pressure cylinder from wetting them.

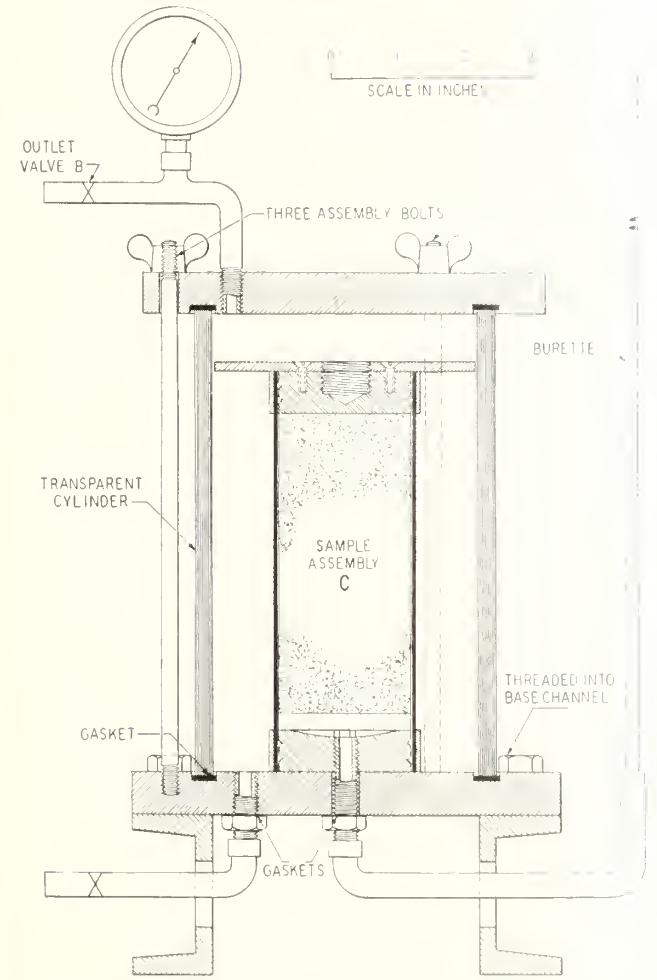


FIGURE 9.—PRESSURE CHAMBER FOR USE IN CONSOLIDATING SAMPLES.

COEFFICIENT *K* DETERMINED USING THE FIXED RUBBER TYPE OF STABILOMETER

It is convenient to arrange the stabilometer in loading devices so that upward movement of the plunger is prevented while pressures within the chamber are increased to those selected for use in the tests. At particular lateral pressures thus provided, samples are compressed to failure by vertical pressures applied through the plunger.

Figure 12 illustrates the failure of a cylindrical sample. As the cylinder shortens it bulges first (fig. 12–B) and then fails along the surfaces of slip (fig. 12–C and 12–D) which incline to the horizontal at the angle a shown in the diagrams, figure 1.

Tests on samples comprised of differently colored modeling clays disclosed the deformations, figure 13, undergone by the layers which had uniform thicknesses prior to test.

Reduction of the vertical pressure, accompanied by increase of lateral pressures, facilitates removal of samples from the chamber and container by reducing their diameters.

Analyses of test data by means of Mohr's circles of stress has been described in PUBLIC ROADS (*26*). Common tangents which disclose the values of c and ϕ are drawn to arcs constructed from a knowledge of the vertical pressures, v, and the lateral pressures, l, on the sample at failure.

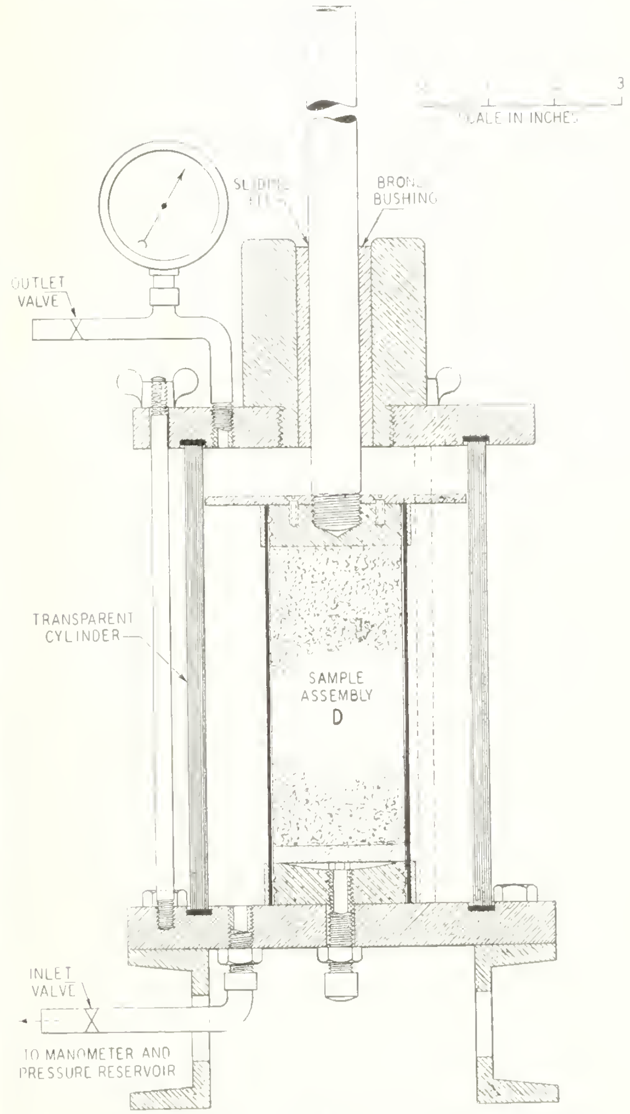


FIGURE 10.—STABILOMETER OF THE PLUNGER TYPE.

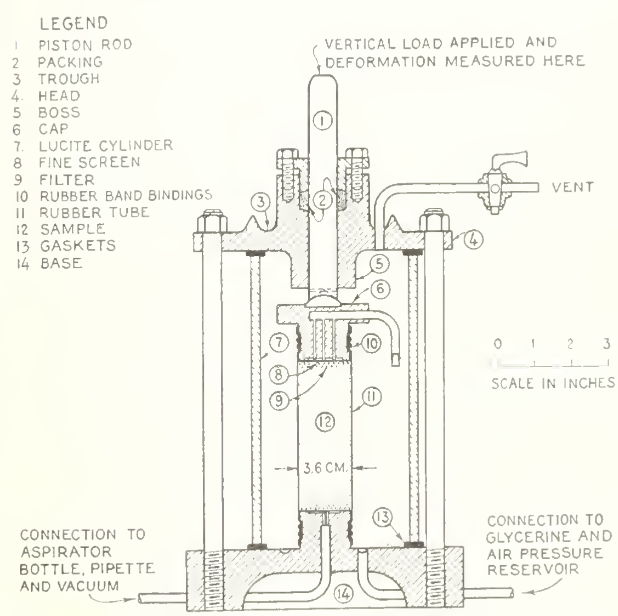


FIGURE 11.—THE TRIAXIAL COMPRESSION CHAMBER.

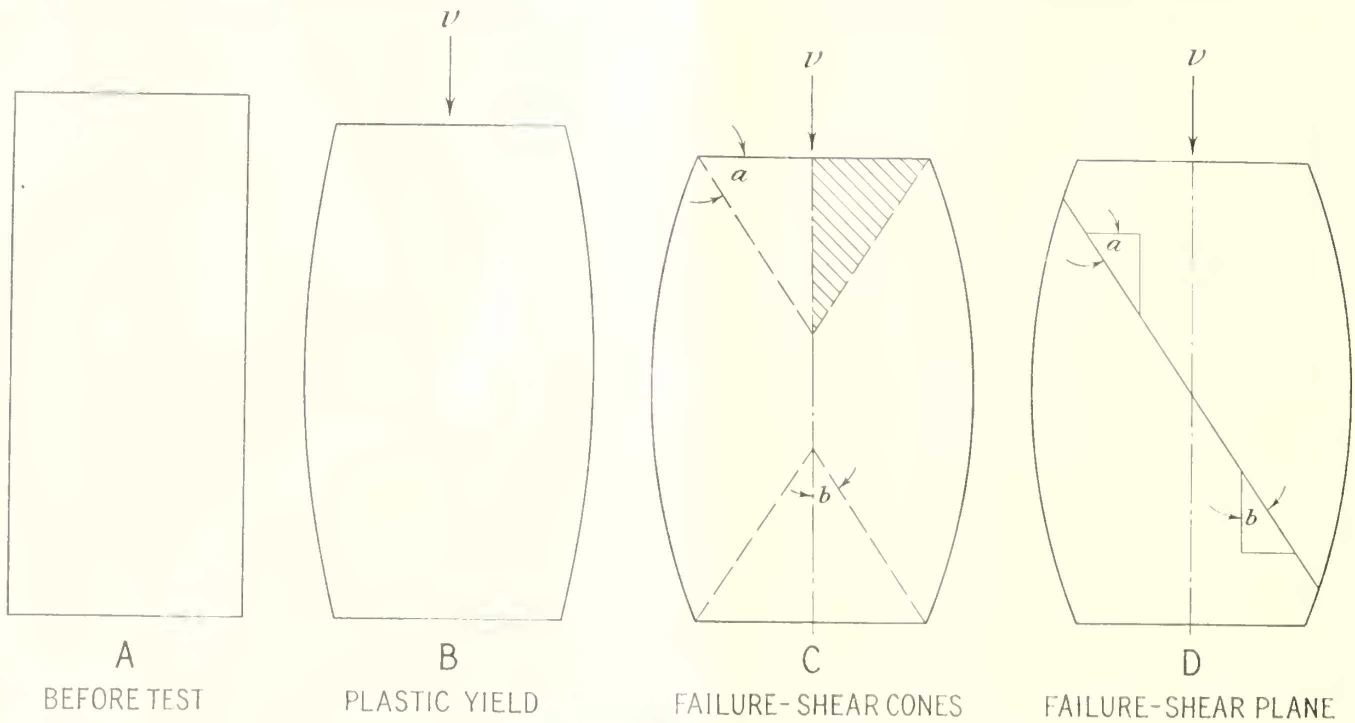


FIGURE 12.—DEFORMATION OF THE SAMPLE DURING TESTING.

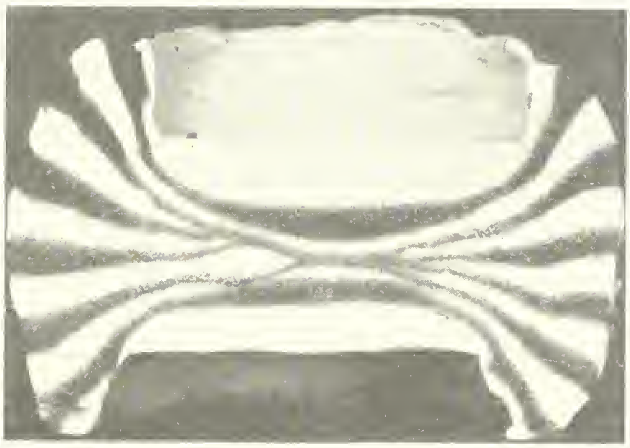


FIGURE 13.—DEFORMATION OF A CLAY SAMPLE. THE LIGHT AND DARK BANDS WERE OF EQUAL AND UNIFORM THICKNESSES BEFORE TESTING.

To illustrate, let the stress-strain relations, figure 14, represent data furnished by two tests. The cylinder tested at a lateral pressure, l, of 100 pounds per square foot, failed at a vertical pressure, v, of 1,046 pounds per square foot. The cylinder tested at l equals 500 pounds per square foot failed at v equals 1,900 pounds per square foot.

Figure 15 shows the graphical analysis. The full line at the top shows the relation between shear stress and normal pressure at failure of the cylinders. The straight broken lines show similar relations for strains less than the ultimate.

The arcs have centers on the abscissa at a distance of $\frac{v+l}{2}$ from the origin, and radii to the same scale of $\frac{v-l}{2}$. This places the center of the smaller full line circular arc at the point corresponding to $\frac{1{,}046+100}{2}$ pounds per square foot, and makes its radius equivalent to $\frac{1{,}046-100}{2}$ pounds per square foot.

As the next step, the relations of c and ϕ to deformations of the samples may be shown as previously described (*8*, *27*).

Use of sample assembly C with the special manometer, figure 16, permits the determination of pore pressures within samples during test. The special manometer has been discussed elsewhere (*25*, *28*).

In the determination of coefficients of earth pressure at rest, lateral deformation of samples is confined to a minimum. For this purpose, the stabilometer, figure 17, is suggested. By the use of sample assembly G and at the discretion of the operator, water may be applied directly to the sample's top and by the connection through the lower disk, to its bottom.

The rubber sleeve of the sample assembly attached at both ends to the pressure chamber, distinguishes the stabilometer, figure 17, as the fixed rubber type which has been used by Hveem, Buisman, Housel, Seibert and Palmer, Muir, the Shell Oil Company and the Delft Laboratories (*22*). Figure 18 shows stabilometers of the free and fixed rubber types.

In making the test for K, the chamber is completely filled with water and both outlet and inlet valves are closed to prevent escape of the water during test. The vertical pressure is then applied through the plunger, and the gradually increasing lateral pressures are read from the gage.

Relations of K to moisture content of a soil are obtained from samples compacted at or consolidated to different moisture contents and tested at pressures within the range for which information is desired.

The stabilometer, figure 17, with confinement of liquid in the pressure chamber, typifies also the cell apparatus, figure 19, used at the present time to test the soft undersoils for which Holland is noted. Thirty-eight of the devices were in use at the Delft Laboratories

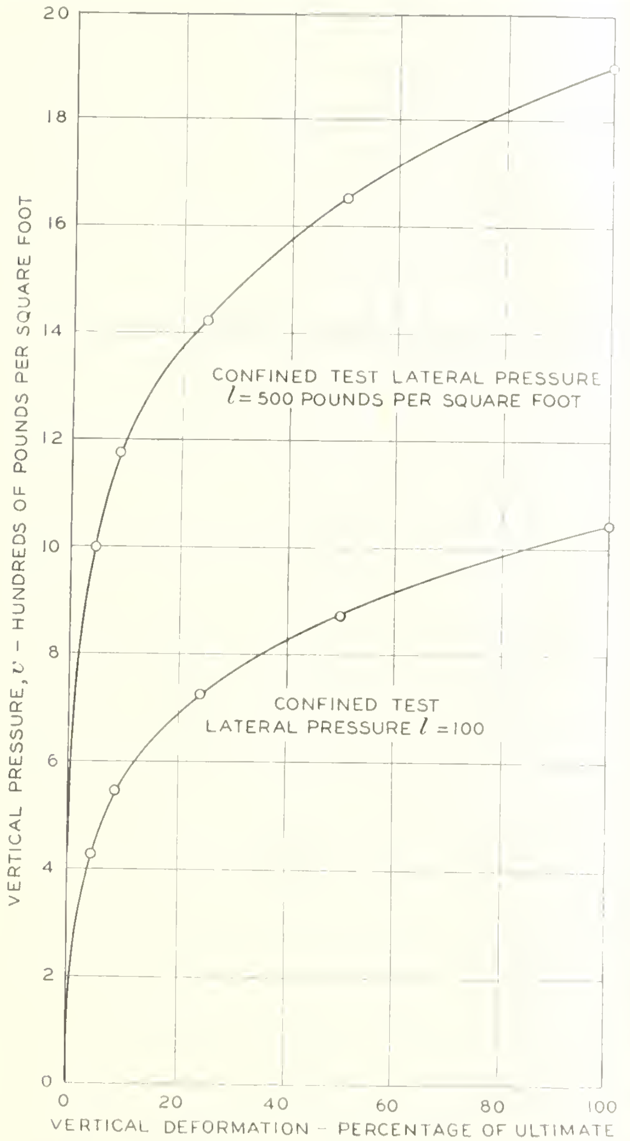


FIGURE 14.—STRESS-STRAIN RELATIONS FOR TWO SAMPLES.

in the summer of 1938, and 25 more had been prepared for shipment to the University of Ghent, Belgium.

The testing procedure, described elsewhere (*22*) provides for escape of the liquid, in small amounts at a time, from the chamber. This in turn causes increasing shear resistance to be developed as the soil deforms. Its unique feature is the testing of but one sample to obtain values of c and ϕ of an undisturbed soil at its natural moisture content. For shear tests of the same soil at lower moisture contents, samples are first consolidated in the stabilometers.

VARIOUS FEATURES OF APPARATUS DISCUSSED

The long period of time required for this makes it advisable to preconsolidate the samples in the separate chamber, figure 9.

The impervious top disk of the sample, assembly C, would then be replaced by the porous stone and perforated disk, assembly G, and the rubber sleeve slipped over and clamped about the metal extension as shown in figure 17.

The selection of the type of stabilometer depends primarily upon the size of the samples to be tested,

FIGURE 15. GRAPHICAL ANALYSIS OF STRESSES IN CYLINDER

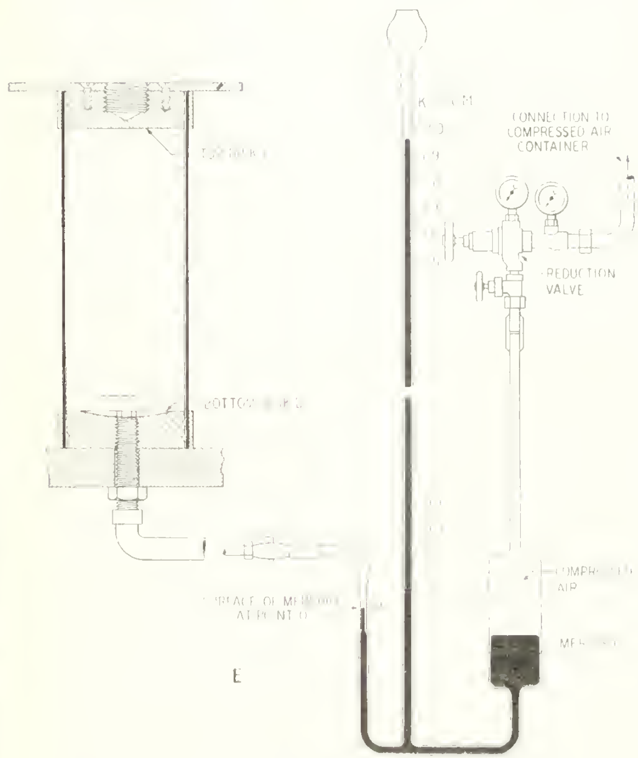


FIGURE 16. STABILOMETER ASSEMBLY WITH SPECIAL MANOMETER.

pressures to be used during test, laboratory facilities, and personal preferences as to the use of air or liquid in the pressure chamber.

Sample dimensions.—To insure that planes of rupture intersect the sides of samples, their heights should be at least twice their diameters. A diameter of 2 inches is satisfactory for soil which passes the No. 10 sieve.

Samples with larger sized particles require larger diameters. H. N. Hveem has found a diameter of 4 inches satisfactory for certain types of bituminous road surfacings; and the Bureau of Reclamation apparatus is suitable for testing samples up to 6 inches in diameter by 16 inches long.

Chamber walls.—Apparatus of the size illustrated in figures 9, 10, 17, 20, and 21, provides for the testing of samples 2 inches in diameter by 4½ inches high. For tests of such samples the use of transparent tubing for the outer shell of the pressure chamber is recommended, since among other things it provides desirable visual inspection of samples during test.

Use of glass for this purpose, figure 22, was proposed

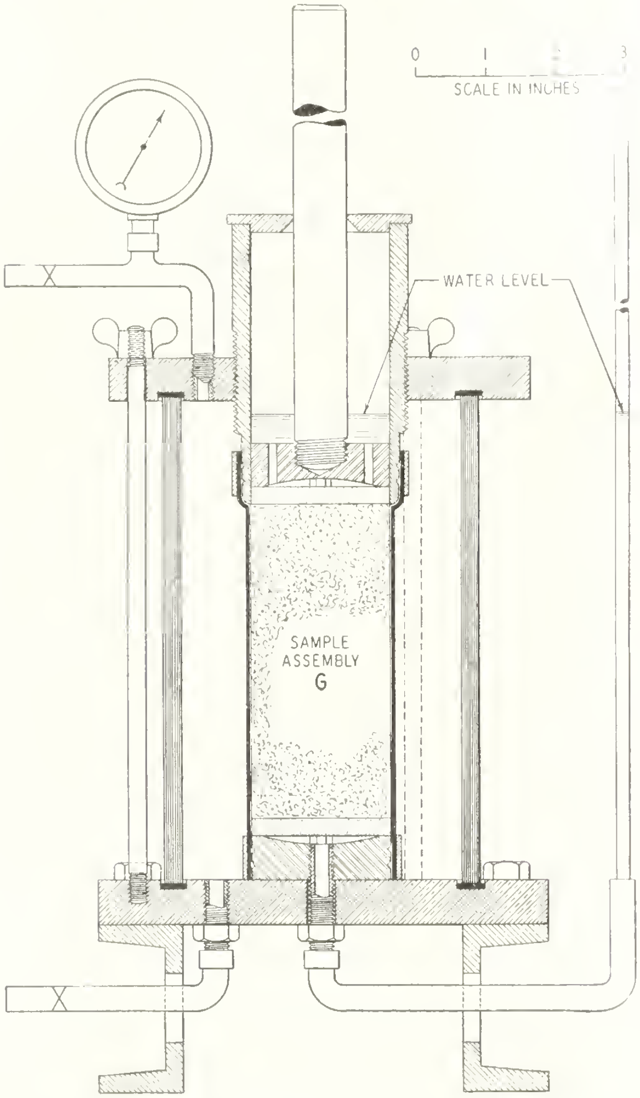


FIGURE 17.—STABILOMETER ASSEMBLY OF THE FIXED RUBBER TYPE.

by Buisman in 1934 and it is still used in European laboratories. The transparent plastics used in this country are recommended as more suitable. Relative to experience with them at Harvard University the Watson report (*20*) states:

> This pressure chamber was designed for and has been successfully used under an internal pressure up to 10 kg. cm.$^{-2}$ Plate BII-1 (fig. 11) shows that it consists of a "Lucite" cylinder enclosed with rubber gaskets between a cast-brass head and base.

For large samples and for the high pressures commonly used to test semirigid pavement surfacing materials, outer cylinders consisting of metal are used. The elaborate apparatus constructed by the Bureau of Reclamation is shown in figure 23. Relative to the latter's apparatus, their report (*22*) states:

> The loading equipment will develop and measure an axial load up to a maximum of 7,500 pounds and deform specimens as much as 4 inches.

Application of load.—Some laboratories use testing machines for applying load to the samples, and measuring their vertical deformations. Others make use of yokes, levers, or threaded plungers to apply the loads, and micrometer dials to measure the deformations.

Figure 24 shows the testing machine used at Harvard

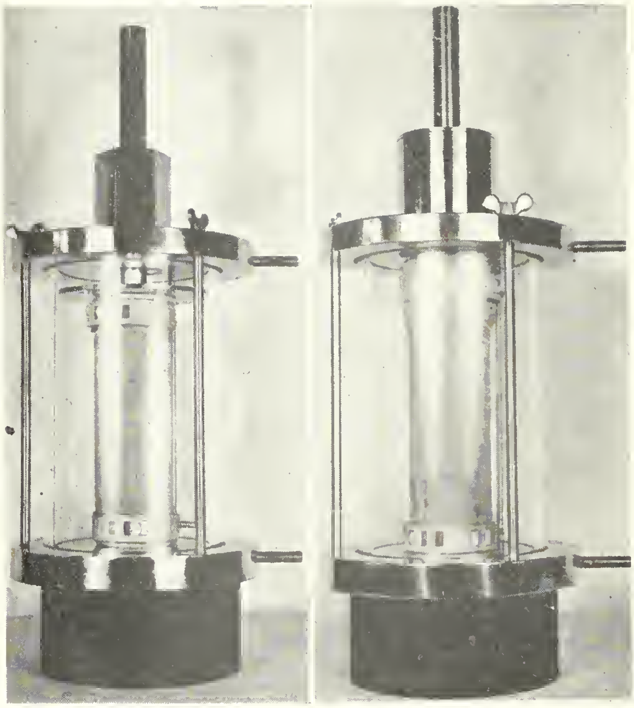


FIGURE 18.—LEFT, FREE, AND RIGHT, FIXED RUBBER TYPES OF STABILOMETER.

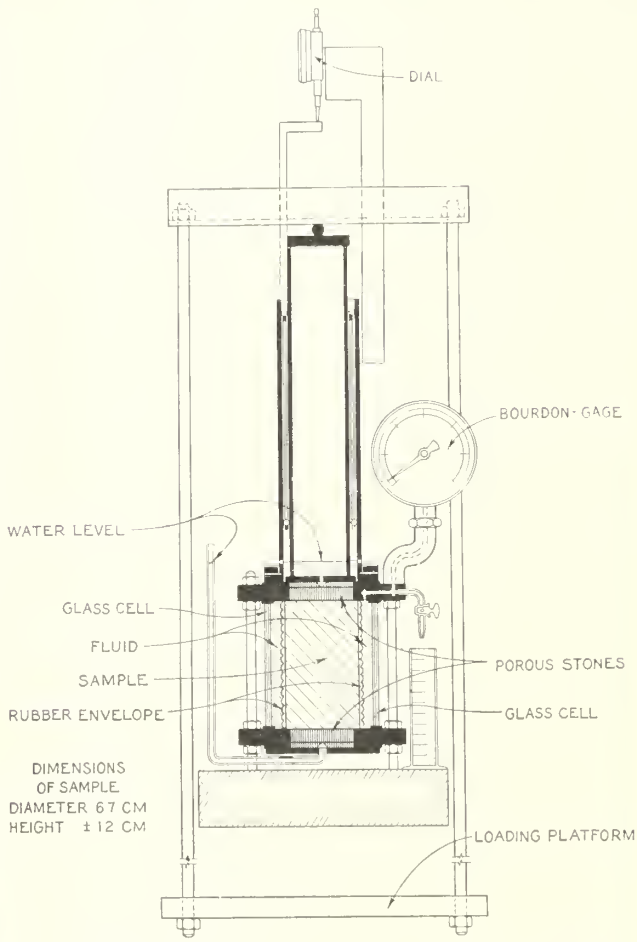


FIGURE 19.—CELL APPARATUS DEVELOPED BY THE DELFT LABORATORIES.

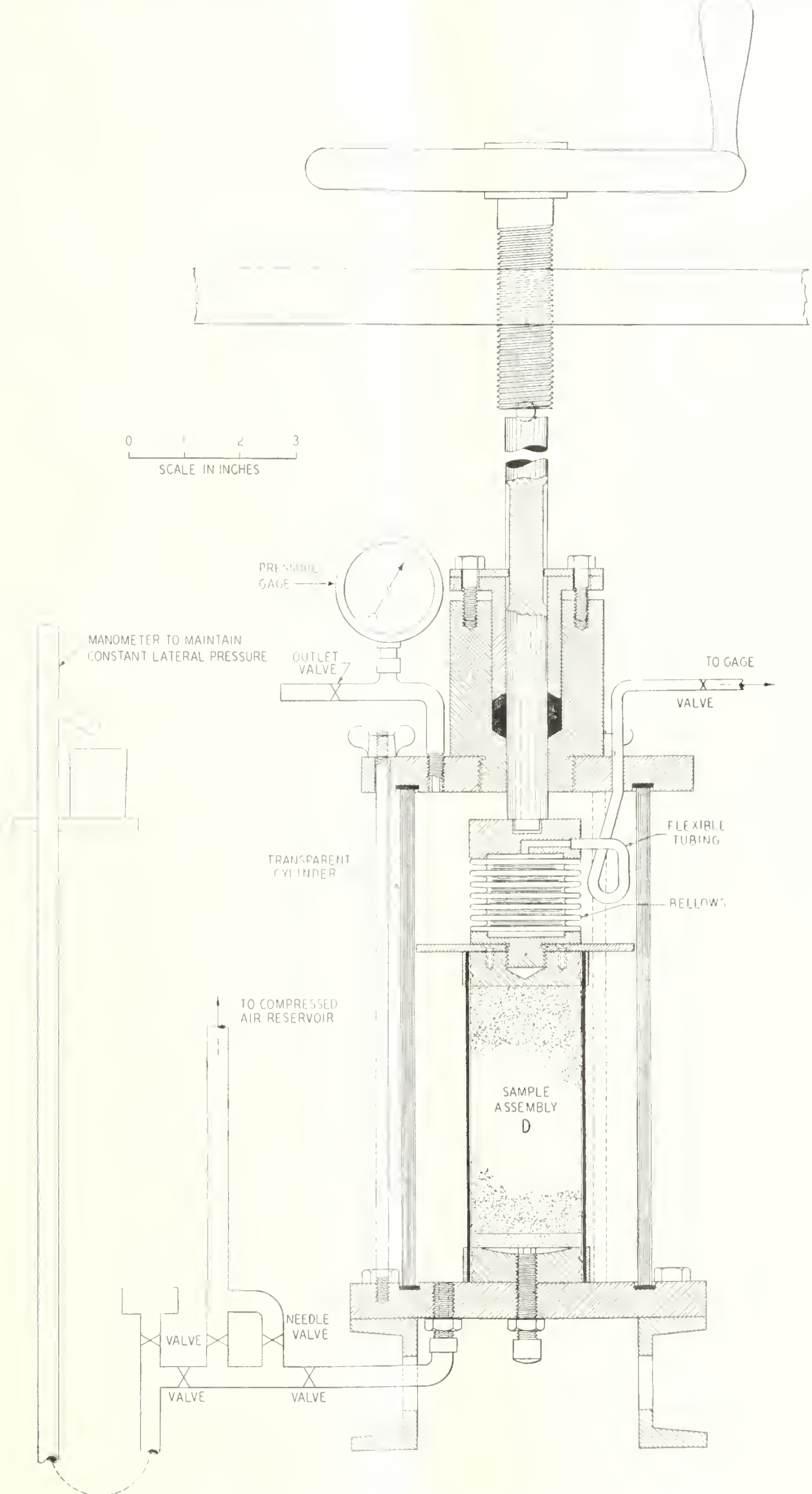


FIGURE 20.—STABILOMETER ASSEMBLY OF THE BELLOWS TYPE.

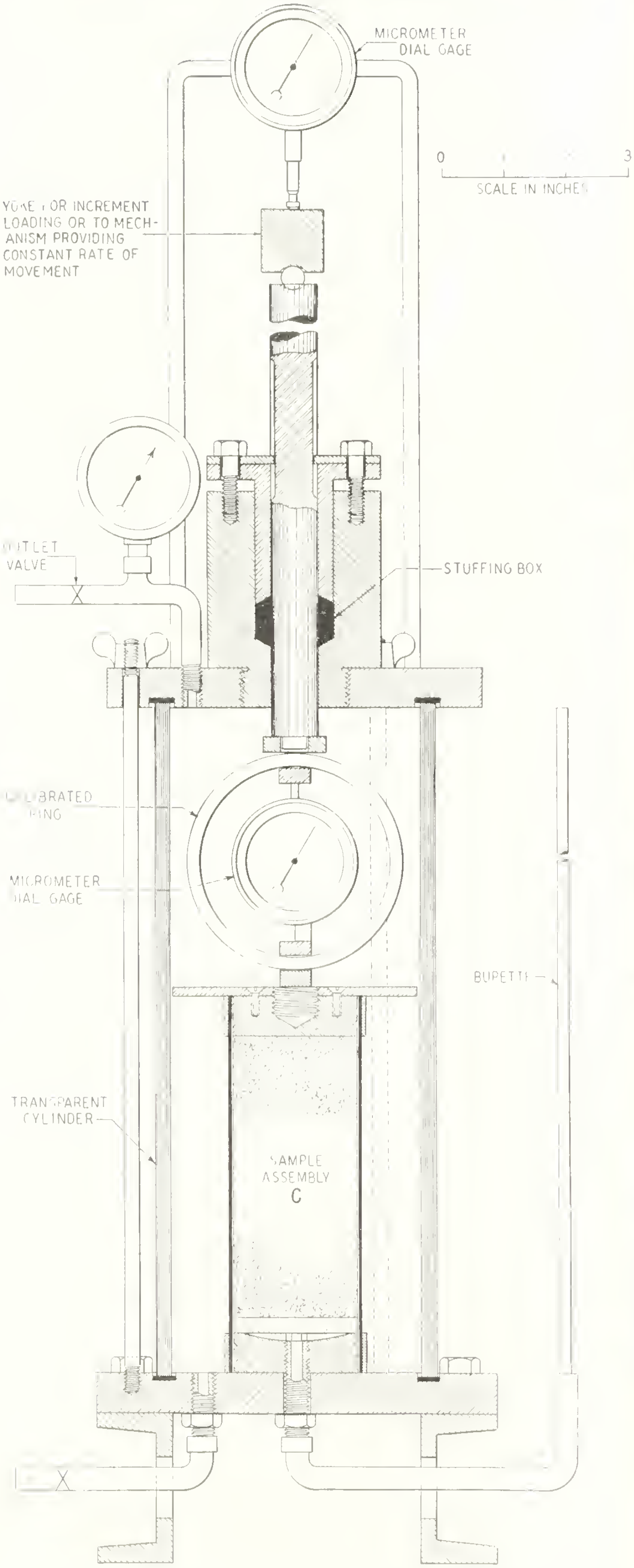


FIGURE 21.—STABILOMETER ASSEMBLY OF THE RING TYPE.

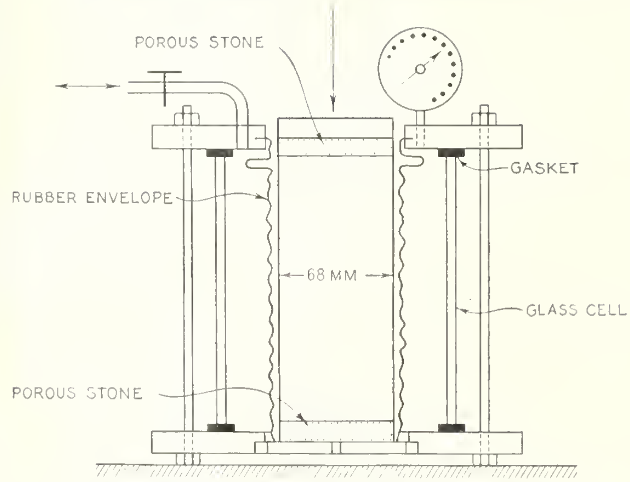


FIGURE 22.—CELL APPARATUS WITH GLASS CYLINDER USED BY BUISMAN.

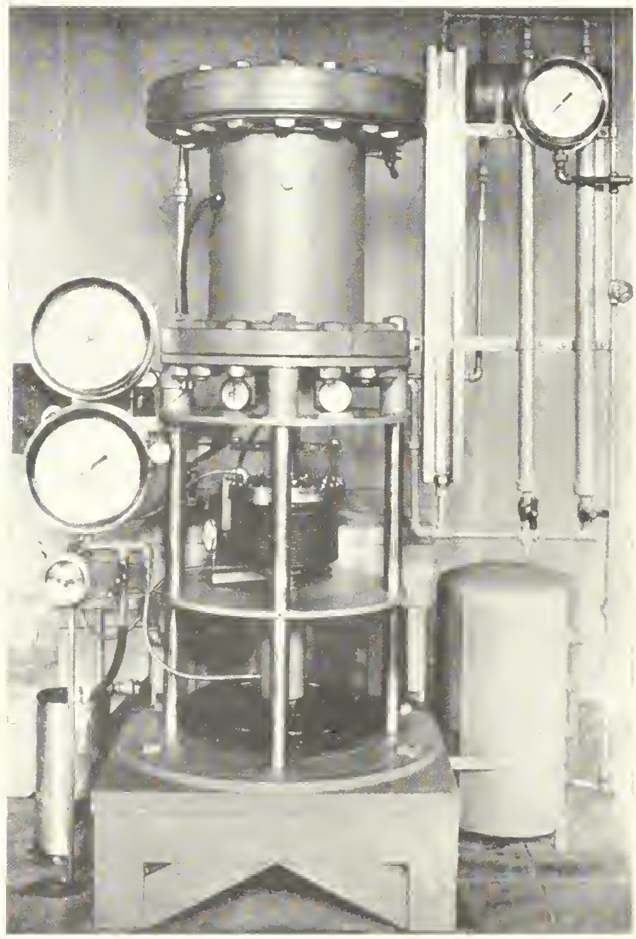


FIGURE 23.—STABILOMETER USED BY THE BUREAU OF RECLAMATION.

University. The method using a threaded plunger is employed by Jürgenson, Hennes, the Bureau of Reclamation, and, in tests of unconfined cylinders, as shown in figure 25, by Burmister.

All methods are considered satisfactory. However, methods causing a constant rate of strain facilitate the determination of deformations indicative of ultimate failure and are therefore preferred.

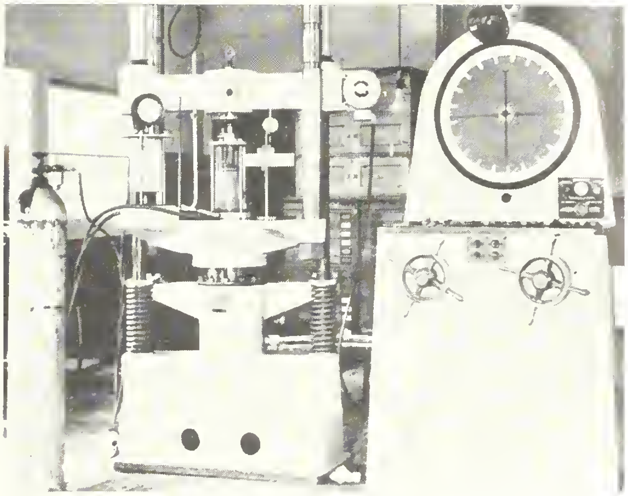


FIGURE 24.—APPARATUS USED IN MAKING TRIAXIAL SHEAR TEST AT HARVARD UNIVERSITY.

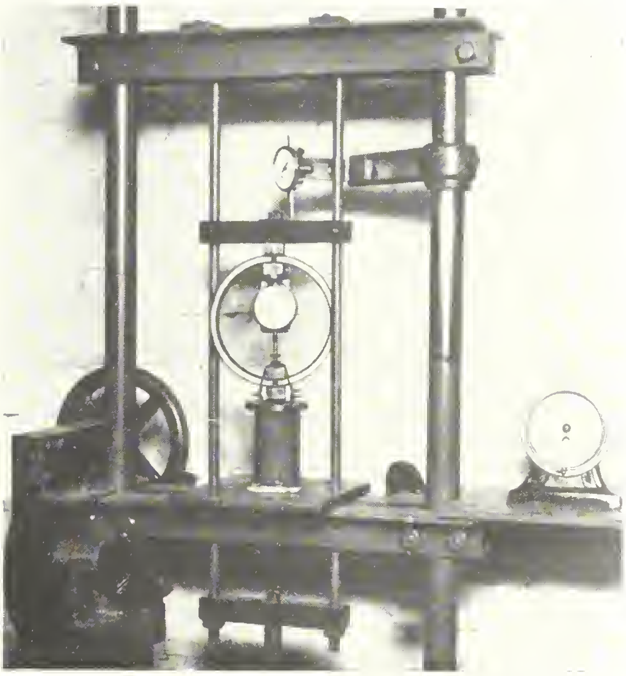


FIGURE 25.—LOAD MEASUREMENT BY CALIBRATED RING.

Pressure chambers.—Except for their tops, the stabilometers, figures 10 and 17, and the consolidation chamber, figure 9, are alike, and all are required for routine stabilometer tests. However, for use in making only occasional tests the one base and cylinder with the three different tops should prove adequate. If the use of glycerine within the chamber or the possibility of leakage from it is considered objectionable, the apparatus, figures 20 and 21, can be substituted for the free rubber type, figure 10.

To make the chambers airtight, packing must be compressed enough to prevent frictionless movement of the plunger. Therefore, means within the chambers to measure vertical pressures applied to the samples is required.

For this purpose use of a sylphon bellows (fig. 20), or a calibrated ring (fig. 21), is suggested. Jürgenson (*15*) placed a bellows inside the chamber, and the Bureau of Reclamation places the bellows on the outside. The calibrated ring has been used in direct shear tests at

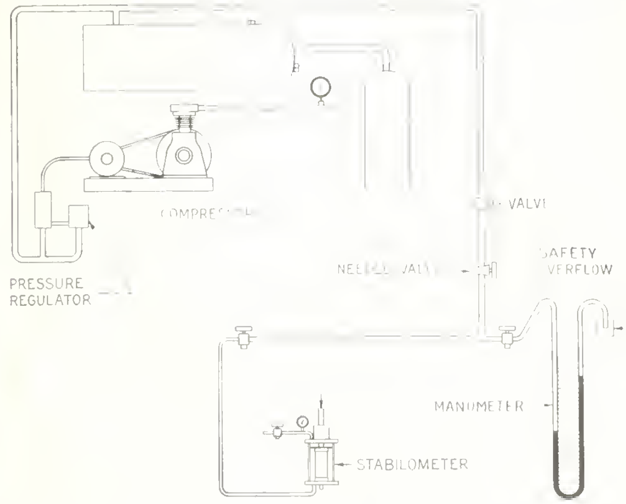


FIGURE 26.—STABILOMETER, MANOMETER, AND COMPRESSED AIR SYSTEM.

Massachusetts Institute of Technology (*29*) and by Burmister (*22*) in unconfined cylinder tests (see fig. 25).

Figure 20 illustrates the use of a manometer for controlling lateral pressures and the threaded plunger method of applying load. Manometers used in Jamieson's early experiments have been employed also in the Delft Laboratories, and to supplement pressure gages in the control of low lateral pressures by the Public Roads Administration. Since provision is made for measuring applied vertical pressures, the threaded plunger is usable to obtain a constant rate of strain. Figure 21 illustrates also an arrangement for measuring vertical movements of the plunger when the loads are applied through yokes.

Chamber pressures.—The air supply system used by the Public Roads Administration, figure 26, provides for pressures up to 125 pounds per square inch and a reservoir of 2-cubic feet capacity. The Bureau of Reclamation's apparatus provides lateral pressures to a maximum of 200 pounds per square inch. Constant pressures are maintained by means of a pressure control device which automatically starts and stops the compressor. For maintaining constant lateral pressures up to at least 10 pounds per square inch, the manometer shown in figure 20 is a valuable supplement to the automatic pressure-control device. For larger lateral pressures, the controlled pressure air reservoir is used.

CLOSED SYSTEM SUGGESTED FOR THE DETERMINATION OF c AND ϕ

Data furnished by direct shear tests illustrate advantages of the closed as compared with the open system of test. Relations of s to n, figure 27, were obtained from data furnished by open system tests, and published elsewhere (*30*). Samples placed between porous stones and consolidated to equilibrium at the moisture contents indicated were sheared at several normal pressures up to and including the consolidation pressure.

To illustrate deficiencies of the data, figure 27, let it be assumed that an embankment which on completion will produce a pressure of 6,000 pounds per square foot, is to be constructed on the soil, at a natural moisture content of 77 percent. At this moisture content and for pressures up to n equals 2,000 pounds per square foot, c equals 1,140 pounds per square foot and ϕ equals 4°. Consolidation by the embankment pressure of 6,000 pounds per square foot can be expected ultimately to

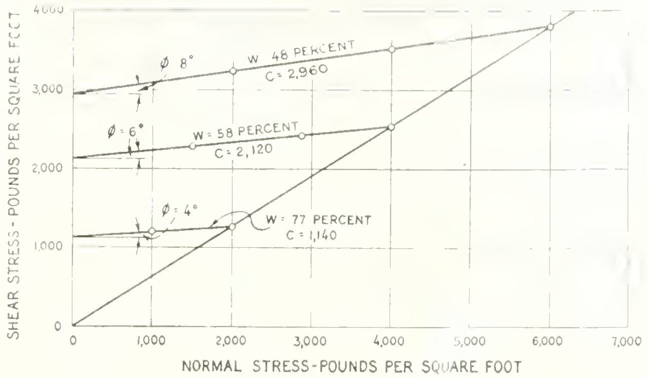


Figure 27.—Shear Strengths of Samples at Different Moisture Contents, Using the Open System.

reduce the soil's moisture content to 48 percent; and at this moisture content it has for normal pressures up to 6,000 pounds per square foot, values of c equals 2,960 pounds per square foot and ϕ equals 8°.

Depending on the relative speeds of embankment construction and consolidation of the undersoil, excessive pore pressures may be produced which make failure of the undersoil imminent. In such cases it has been considered advisable (*31*) to make use of standpipes inserted in the undersoil so that the speed of its consolidation can be observed.

Information required for the intelligent use of such standpipes necessitates extension of the data, figure 27, to include relations of s to n up to normal pressures of 6,000 pounds per square foot for the samples which contain both 77 and 58 percent moisture.

To obtain the supplementary data, shear tests must be made on samples at normal pressures greater than the consolidation pressures and for this purpose the open system as used in direct shear tests is impractical, because of the rapid speed at which the relatively thin samples used in such tests consolidate.

Therefore, the closed system which furnishes the complete data, figure 28, is deemed more suitable.

To obtain the information given in figure 28, samples compacted at the moisture contents shown were placed between metal plates to simulate the closed system and sheared.

Determination of the pressures at which the relations of s to n change, as shown in figure 28, is especially important since they indicate the upper limit of stresses that can be applied without causing the angle ϕ of the soil at a particular moisture content to become reduced.

Thus, the soil, figure 28, at a moisture content of 30 percent has c equals 460 pounds per square foot and ϕ equals 7.4° for normal pressures up to the limit of n equals 1,230 pounds per square foot. At normal pressures greater than n equals 1,230 pounds per square foot, the shear stress became constant at 620 pounds per square foot.

Change of the soil's character with increase of its ratio of free water to film moisture has long been recognized. As discussed elsewhere (*8*) this ratio may be increased in two ways as follows:

1. By increasing the moisture content of the soil at constant pressure.
2. By increasing the pressure on the soil at constant moisture content.

It has been explained in PUBLIC ROADS (*32*) that increasing the moisture content of semirigid soils at constant pressure increases the ratio of free or lubricating water to the more viscous film moisture, until at moisture contents above the plastic limit the ratio becomes large enough to give soils the properties of plastic instead of semirigid materials. In the same publication, data from tests of unconfined cylinders, which are triaxial shear tests with the lateral pressure l equals 0, showed that at moisture contents above the "critical," which for plastic soils is the plastic limit, the samples exhibited little or no additional strength with increasing deformations above the resistance they had at the plastic limit.

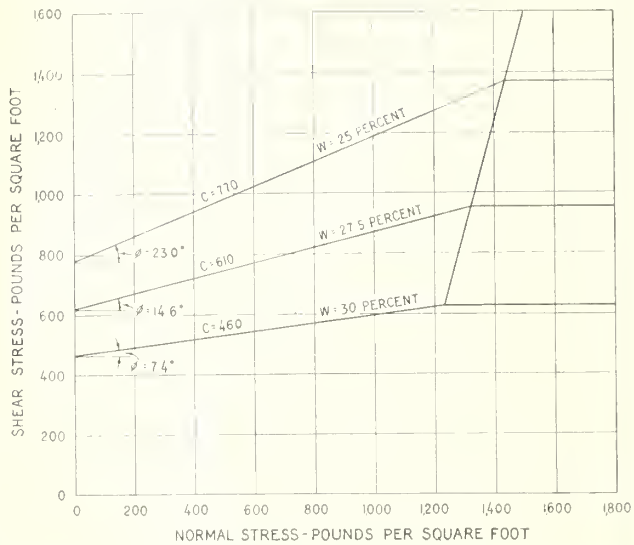


Figure 28.—Shear Strengths of Samples at Different Moisture Contents, Using the Closed System.

Increasing the pressure (*33*) reduces thicknesses of adsorbed films and therefore, in soil maintained at constant moisture content, causes the ratio of lubricating to film moisture to be increased as effectively as raising the moisture content of soil at constant pressure.

The effect of pressure increase to reduce the lower limit of moisture contents of soil in the plastic state has been reported (*34*).

Therefore, the breaks in the relations of s to n, as shown, are explained on the basis of film phenomena, and for the particular pressures at which they occurred, the respective moisture contents are considered as the critical moisture contents.

What relation, if any, these critical moisture contents bear to pressures required to consolidate the soil has not been disclosed by investigations of the consolidation characteristics of this soil made to determine if any such relation exists.

From the complete data, figure 28, one obtains relations of moisture content to c and ϕ, the stresses at which the soil becomes plastic, and the pore pressure. From such relations and by means beyond the scope of this report, usable safe values of c and ϕ can be selected with respect to the speed of consolidation as indicated by the elevations of water in the standpipes (*35*) inserted in soft undersoils.

SUMMARY

The foregoing presents essential features of what seem to be the most promising methods of making stabilometer tests. It is recognized that compressed air as well as liquid may be used to determine the coefficient, K, and also that the open system may be

(Continued on p. 153)

SAFETY PROMOTION ACTIVITIES OF THE STATE HIGHWAY COMMISSION OF WISCONSIN

By WILLIAM F. STEUBER, Assistant Safety Director, State Highway Commission of Wisconsin

THE SAFETY DEPARTMENT of the State Highway Commission operates on a budget of $50,000 per year. At first thought such a sum may seem ample to eliminate highway accidents altogether. Actually, to conduct a year's safety activity in Wisconsin expenditures must be made very carefully to carry on with $50,000 all the activities that seem desirable.

Wisconsin's population is 2,926,000 persons or 730,000 families. There is only 1.7 cents per person or 6.8 cents per family to spend on safety education. A letter to each family twice a year, urging safe use of the highways, would consume the entire fund in postage and stationery without being an effective approach to the traffic problem.

To use $50,000 effectively in highway safety promotion is a real task. First of all, the expenditures must be pyramidal in effect. That is, a single expenditure must reach one group, then another, and still another, carrying a message to each group. The effect of funds used to support the school safety patrols is a good example. In Wisconsin, belts and badges for school safety patrols are supplied free to schools by the State Highway Commission. Each outfit, one belt and one badge, represents an investment of 37½ cents. Each belt and badge identifies one boy as a safety patrolman. Before he assumes his duties and goes about his patrol tasks he learns the elements of pedestrian safety in traffic. His duty is to protect his classmates in traffic—they are the second group reached.

Teachers in the schools are also reminded of traffic dangers as they guide the safety patrols in their work, thus the teachers make up the third group. The school children tell about the school patrol to smaller children, thereby reaching a fourth group. The school patrol is discussed at home, reaching the fifth group, the parents. Motorists passing the school see the safety patrols at work—a sixth and very large group is reached. Pedestrians who walk past the school and see the patrol boys on duty make up a seventh group.

Thus, for an expenditure of 37½ cents, at least seven different groups of people are reached. But the pyramidal effect does not end here. At the close of the school year the boy turns in his belt and badge and the next school year another boy uses them, a new class is protected, new parents are brought face to face with a safety activity, and a new pyramid is started, all financed by the original expenditure of 37½ cents.

Compilation and use of accident statistics produces a pyramidal effect since they are used by speakers, in newspapers, and over the radio. Creation of county safety councils to conduct local safety programs outlined by the Department carries on the idea of pyramiding. So does the use of motion pictures—films can be projected time after time with low upkeep cost. Driver testing equipment requires little expenditure after the initial cost is paid, and is used by group after group. Each proposed activity of the Safety Department is judged on its pyramidal value, its ability to reach more and more people cheaply and effectively.

The highway safety program in Wisconsin is planned for an attack on the traffic problem at sources of trouble. The State is concerned with the education of the driver, the pedestrian, and the school child. To make the driver and the pedestrian and the school child realize the extent and seriousness of the accident problem, to teach them all that accidents are the result of human failings, to get them to conduct themselves properly in any occasion to avoid accidents, all are functions of the Safety Department of the State Highway Commission.

ACCIDENT STATISTICS USED IN PLANNING SAFETY WORK

Accident records and statistics.—In Wisconsin, traffic accidents that result in a human fatality or injury, or in property damage in excess of $50 must, by law, be reported to the State Highway Commission within 48 hours. The compilation and analysis of the data in these accident reports is a main function of the Safety Department. Statistical studies are made to determine what accident-producing factors deserve the most attention in safety promotional work. Individual reports are strictly confidential, but the statistics of accidents are made public. Much material for newspaper articles, lectures, and radio talks is derived from these statistics. Many special statistical studies are made available to legislative committees, to other State departments, to localities and communities, and to groups and individuals whose special research may require an analysis apart from the regular tabulations. In several instances, detailed studies have been made for persons desiring to use the material in the preparation of theses, and in preparing technical papers or articles for professional publications.

Facts and trends, as shown by these statistical studies, are used by the Safety Department to identify the places where intense safety activity is necessary and to determine the type of safety activity most urgently needed. Statistics are often considered dull by the general public, but they are a necessary foundation in carrying on a comprehensive highway safety program.

County safety councils.—The basic organizations for highway safety promotion in Wisconsin are the county safety councils. Each of the 71 counties of the State has its own safety council that meets regularly and studies the safety needs of the county. The council is organized by and functions under the Safety Department of the State Highway Commission. It consists of a general chairman, a general secretary, and chairmen of committees of engineering, enforcement, education, and publicity. Its members are civic-minded persons who work without pay for the betterment of their community. Each council accepts as its duty a study of community safety needs and suggests to governing bodies solutions of local traffic difficulties. It helps to provide a better understanding between the public, the traffic enforcement officials, and the courts. It counsels the local populace repeatedly in proper behavior of both driver and pedestrian to prevent local traffic accidents.

Through the county safety councils safety programs are arranged in schools, at luncheon clubs, at civic and service meetings, in P. T. A. groups, and in fraternal, industrial, and religious groups. Safety exhibits, meetings, parades, and campaigns are planned and executed by these councils. They maintain speaker bureaus and spot maps; they prepare safety addresses and supply safety news releases to the local papers; and they compile statistics on the traffic accidents occurring in their county. Typical membership in the county council includes county judges, traffic officials, school superintendents and principals, county highway commissioners, representatives of fraternal, service and business clubs, industrial leaders, and professional men as well as those who have no special qualifications other than a wholesome, live interest in the welfare of their community.

Guiding and advising all the county councils is the Safety Department. To the county councils it sends regular letters outlining suggested activities, and field contact men who advise the local officials and learn their ideas to the end that each community benefits by the experience and suggestions of all the others. County councils receive every aid the Safety Department has at its disposal—statistics; supplies of literature for free distribution; special speakers from the Department; motion picture programs on safety with operator, machine, and films furnished; aid in preparing and releasing safety publicity; aid to schools in conducting suggested study courses; and supplies for school safety patrols.

To focus special attention on highway safety in each community of the State, no method better than the county safety councils has been found.

Public contacts.—The personnel of the Safety Department consists of a safety director, an assistant director, a supervisor of publicity, an office manager, a statistician, three district field representatives, a stenographer, three clerks, a publicity assistant, and a part-time student who serves as an additional clerk.

The director, the assistant director, the publicity supervisor, and the three district field representatives maintain close contact with the public in safety promotional work. Each of these six men is a competent public speaker with a background of traffic and safety research and experience. Each of the six meets with the county councils at regular intervals to give advice and to correlate their activities for greatest efficiency. Assistance is given in forming councils and in keeping them active and informed. Assistance by these men is given the county councils when special programs of motion pictures are desired in schools, at clubs, at P. T. A. meetings, or public safety meetings arranged by the councils themselves.

SAFETY PROGRAMS GIVEN TO ANY GROUP REQUESTING THEM

When a council wishes to schedule a program on highway safety with a speaker and motion pictures, arrangements are made to supply them. County superintendents of schools are contacted and through them arrangements are made to present safety programs in the schools. A 45-minute program consisting of a 15-minute safety talk followed by 30 minutes of safety movies is enthusiastically received by students from elementary grades through senior high schools. Of course any of several different speeches and motion pictures can be presented so that the program is in keeping with the particular problems of the audience. A 45-minute program can be given in four schools a day, two in the forenoon and two in the afternoon.

When a Safety Department man comes into a county to conduct these programs, the local council usually keeps him busy. It is not unusual for a safety lecturer to speak at four school meetings a day, plus a luncheon club address at noon and a P. T. A. meeting at night. With such a number of meetings in a single day, it is imperative that the public contact men have a variety of facts at the tips of their tongues, and an ability to blend those facts into an interesting talk. Of prime importance is the ability to sense immediately the interests of the audience and to address it in terms and manner so that the message is vital to the group's own traffic problems.

Public relations is an important phase of the highway safety program in Wisconsin. Any group in any part of the State may address a penny post card to the Safety Department requesting a program, and that program will be provided at no cost to the group. P. T. A. groups, service clubs, luncheon clubs, chambers of commerce, fraternal organizations, 4-H groups, boy and girl scouts, schools, traffic enforcement bureaus, and industrial plants have availed themselves of this service.

In 1938 public relations men of the Safety Department attended 1,183 meetings. Of these meetings, 515 were contacts with Safety Councils, and 668 were highway safety contacts with other groups. A total audience of 162,542 persons was reached with direct messages of highway safety. The county safety councils by themselves held 3,223 safety meetings and reached an additional 229,106 persons.

The Safety Department realizes that regardless of the size of the audiences, all automobile drivers and pedestrians in the State cannot be reached directly. In each address the plea is made for all listeners to carry the appeal for street and highway safety to their families, neighbors, friends, and co-workers. How extensively this is actually done depends in large part upon the quality of the program presented and the competence of the speaker to present his ideas in a manner that generates an urge to carry the message further. With a reduction of 10.5 percent in all traffic accidents in the State and a fatality reduction of 23 percent in 1938 as compared to 1937, it is felt that the accident-prevention work of the county and State organizations has been effective and a worthwhile investment.

Driver testing equipment.—Owing to the importance of agriculture in Wisconsin, county fairs and the State fair are prominent occasions in the State. When plans for fairs are being made, the county safety councils appeal to the Safety Department for aid in promoting highway safety by means of a dignified yet striking display. To comply with these requests, three sets of driver testing equipment have been assembled.

The driver testing equipment has been designed to bring a concrete representation of the problems of automobile driving to an individual without taking him onto the highway. Fundamentally it is similar to the testing equipment used by automobile associations, insurance companies, and others. Briefly, each person who takes the test is subjected to eye examinations, to a glare test, to a distance judgment test, to a coordination test which measures how well the body responds to what the eyes see, and to a test of knowledge of Wisconsin traffic laws. Each of these tests is explained in its relation to actual traffic on the highway.

School Children Are Educated in Traffic Safety by Means of School Patrols, Lectures, Movies, and Instruction in Safe Practices for Bicyclists and Pedestrians.

A score sheet is kept for each individual. At the close of the test the operator in charge analyzes each person's score. If defects are found the individual is told what they are and how to compensate for them in the interests of safety.

Operation of the driver testing equipment is the most elaborate and spectacular safety activity of the Safety Department. In 1938 the driver testing equipment was used in 52 Wisconsin localities and tests were given to 10,428 persons. It is felt that the tests benefit both the persons actually reached and the thousands who also learn a few new traffic facts as they watch their friends go through the lines. Further benefits are derived through newspaper articles based on the results of the tests in each locality. Individual test results are kept confidential, but publicity is given to the scores obtained in each community as well as to facts disclosed by the tests. For example, the tests showed that one man out of every twelve tested was color blind to the extent that traffic lights may be confusing.

The driver testing equipment is valuable in safety promotion because it creates an urge for persons to try it; it is curiosity provoking to onlookers; it creates safety publicity material; and it reveals typical characteristics of drivers.

School contacts.—It is the belief of the Safety Department that every effort to promote safety education in the schools will have a beneficial effect on the traffic picture of the future. Because the achievement of traffic safety requires continued efforts over a long period of time rather than a quick flash of brilliance, the logical place to build for the future is in the schools. The drivers and pedestrians of tomorrow are the school children of today. The Safety Department therefore feels that its most effective work can be accomplished through promoting safety education in the schools of Wisconsin.

School safety patrols have become quite generally accepted throughout the Nation as an effective and necessary safeguard to protect school children from traffic. In Wisconsin, the school safety patrols are directly sponsored by the Safety Department, and the belts and badges for safety patrols are given to the schools by the Highway Commission. The badges bear the name of the Commission. Each school requesting safety patrol supplies gets more than just the belts and badges. Detailed directions for the establishment of the patrol are included as well as descriptions of the exact duties of the patrol members.

In six months of State sponsorship, Wisconsin schools have been supplied with over 4,500 belts and a like number of badges. In many of the counties, belts and badges are first turned over to the county traffic officer. He visits the school, gives a lecture on the duties of school patrols, gives a talk to the student body on cooperation with the patrol for safety, pins the badges on the members, and presents to each member an official certification card bearing a pledge of office and a list of ten patrol duties. These cards are also supplied by the Highway Commission, and are signed by a representative of the Safety Department and countersigned by the enrolling officer and the school principal. The entire ceremony gives an air of official standing to the school patrol, and goes far toward making each member fully cognizant of his duty and the other school children more respectful of each patrolman's authority.

LITERATURE ON HIGHWAY SAFETY WIDELY DISTRIBUTED

Many excellent safety pamphlets are available for free distribution by the insurance companies, automobile companies, and automobile associations. The Safety Department receives large supplies of this literature. It has prepared a bibliography appraising the value of much of this material, and distributes it in quantity to any school in the State on request. This literature is excellent reference material for teachers and students. Lesson sheets and posters are also distributed throughout the State by the Safety Department.

Members of the Safety Department are frequently asked to address State, district, and county conventions of school boards and teachers. During these contacts, many teachers have asked aid in preparing courses of study in highway safety. Individual or group assistance is always given.

In keeping with modern trends in education, an increasing emphasis has been placed on motion pictures as an aid in teaching highway safety. The Safety Department has a library of 56 reels of 16-millimeter films on street and highway safety. Both sound and silent films are available on a free loan basis to any school equipped with suitable projectors. Schools without projectors have equal opportunity to receive motion-picture programs, for the Safety Department has five portable projectors which handle either sound or silent films. To avail itself of a program, any school without a projector contacts either its county safety council or the Safety Department directly, and one of the public relations men of the Department brings the program to that school and to as many others in the county as he can reach in the time he has available in that area.

The Safety Department in 1938 sponsored a contest for the best courses in safety study arranged by school officials for use within their schools. Awards of silver shields mounted on wall plaques were made for the best course for city schools, the best course for rural schools, and for the best course offered in vocational schools. Awards were made by the safety director of the State Highway Commission at the annual convention of the Wisconsin Education Association.

Courses in safety education in three State normal schools were offered for credit in 1938. These courses were organized by the schools with the assistance of the Safety Department. During 1939 several more normal schools are offering such courses. Providing advance training to those who will teach is of extreme importance to the future of safety education.

From the accident reports coming in to the Safety Department an unusual and effective safety textbook has been compiled. One hundred different typical highway accidents have been selected for inclusion. Names of characters and locations have been changed to fictitious ones, but the circumstances of each accident and the street or highway lay-out have been retained exactly as in the accident report. Each of these 100 accident cases has been repeated on a page of the textbook, complete with a diagram of the accident, an explanation of how it happened, and a summary of the injuries and damages. Concluding each case are questions of this type: Why did the accident happen? Who was to blame? How could this accident have been avoided?

A section of the book preceding the case studies is devoted to a summary of Wisconsin traffic laws. Following the case studies are 150 questions on traffic laws and safety practices. None of the questions is answered in the book, but a set of answers, prepared by a committee of Wisconsin traffic judges, is available to teachers. This textbook is entitled "Traffic Accidents—Their Causes and Their Prevention." It is supplied in quantity to high schools, normal schools, vocational schools, and individuals, free of charge. The popularity of the book is evidenced by the demand for it. In 1938, 30,000 copies were supplied to Wisconsin schools. A 1939 edition of 35,000 copies will be almost entirely used up in filling orders already on hand.

The Safety Department is devoting much of its time, energies, and money to safety education in the school systems. A triple purpose is served when school children are educated in highway safety—greater safety will be assured for the future, immediate results are obtained in greater safety for the youth of today, and much of the material presented to children is repeated at home for the adults and parents to think about.

Enforcement aids.—One of the most necessary aids to a comprehensive highway safety program is the work of the traffic officer. The Safety Department has no direct authority over the county traffic officers or the city traffic departments of the State, yet cooperation and help from these men has been outstanding in quality. Because efficient enforcement tends to reduce the number of accidents, the Safety Department has exerted its influence to effect employment of traffic officers in counties that have previously been without them. In many cases where officers have been employed, the Safety Department has conducted civil-service examinations so that choice of men employed was made entirely upon ability and experience.

The help of the Safety Department has not ended with aid in employing a traffic officer. Present-day enforcement is not confined to arresting violators and investigating accidents. The modern traffic officer is wholeheartedly engaged in safety education as well as in enforcement. In this work of education, the traffic officer visits schools, addresses clubs and safety meetings, frequently prepares articles for the press, and often participates in radio programs. In all of these contacts the services of the Safety Department are at his disposal—accident statistics, posters, literature, school safety patrol belts and badges, motion picture films, and assistance in preparing speeches and news releases.

TRAFFIC SCHOOL OF BENEFIT TO ENFORCEMENT OFFICIALS

Competent traffic enforcement is a highly exacting task calling for excellent qualities of personal efficiency and judgment. Heretofore the officer who had those qualities did his work in a manner meeting with the approval of the motorist and the community; the officer who lacked them was seriously handicapped and means were lacking for adequate training. The traffic officers themselves were the first to realize that regular training schools for traffic officers would help raise the general

A Driver Clinic, Consisting of Various Tests to Determine a Person's Fitness to Drive A Car, Is Popular at Fairs Throughout the State, and Brings the Message of Highway Safety to Thousands. Lower Right, Provision of Safety Lanes for Free Brake Inspection Is One of the Activities of the County Safety Councils.

level of officer efficiency. When the idea of conducting district traffic schools for officers was discussed with them, they endorsed the plan with enthusiasm. So did the county officials under whom the traffic officers work, as well as city officers, chiefs of police, mayors, and traffic justices and judges.

A traffic school has been organized and operates as follows: The State is divided into nine districts corresponding to the nine division areas of the State Highway Commission. Monthly schools are held in each of these divisions. All persons directly interested in traffic enforcement are invited to attend, from village marshals to county judges. One subject is covered at each meeting, in lecture by competent authorities and in supervised discussion by those in attendance. Typical subjects covered in these meetings are: Accident investigation—obtaining evidence, use of photography and measurements, how to keep records efficiently, first-aid studies; public relations—appearance of officer, approach to a traffic violator, how to get cooperation from witnesses, conduct on and off the job, public speaking hints; court procedure—filing and presenting complaints, the officer's relation to the court, the rules of admissible evidence; and selective enforcement—use of spot maps as reference to accidents, patrolling high-accident areas, checking physical hazards, checking vehicles.

Wisconsin traffic accident statistics show that 51 percent of the accidents are caused by violations of traffic laws. With the traffic officers working to familiarize the citizens with the traffic laws, and with improvement in enforcement methods continuing, even further reductions in traffic accidents and fatalities should result.

Publicity.—Highway safety cannot advance unless the citizenry is aroused to a realization of how acute the problem actually is. Concrete suggestions for improved driving habits and for better pedestrian behavior must get to the general public. The Safety Department receives excellent cooperation from the newspapers, both dailies and weeklies. Practically every Wisconsin newspaper contains at least one good news story each issue on some phase of the traffic problem.

However, the newspapers themselves rarely originate stories on accident prevention. The news value of highway safety information is not apparent to most reporters; or if it is, the reporters often do not have the technical background to offer concrete suggestions of solution.

Of necessity highway traffic news stories with an accident-reduction theme must originate from a source that has accident facts and highway information readily available. Such a news source must be in constant contact with those who are actively engaged in highway safety work. The publicity section of the Safety Department is therefore a vital part of the organization.

Publicity is directed through several channels. The Department publishes "Safety News," a monthly magazine containing items on the activities of the county safety councils, suggested plans of activity for accident reduction, and presenting the latest State and local accident statistics and analyses. This magazine is distributed free within the State to the county safety councils, school authorities, county highway departments, traffic officials, and city authorities. It is also sent outside the State to safety workers in the departments of other States and to Federal departments, including the Library of Congress. Its circulation is now 5,500 copies and is increasing rapidly.

News releases are prepared daily and are mailed to every newspaper within the State. Articles of a general safety nature and special releases dealing with particular local problems are prepared. Use of this material has been almost universal.

The State Highway Commission subscribes to a clipping service. By tabulating the clippings of its stories as they come in, the Commission has an accurate picture of the extent its releases are used by the newspapers. Very often a story sent out as a news release is published in the form of an editorial—infallible evidence that the material submitted was of vital interest to the community.

Newspapers place a high value on printed pictures. The Safety Department releases photographs of unusual accidents or of outstanding safety activities at least once a week. For convenience, these photographs are submitted to the papers as mats so they can be printed at no great expense.

Publicity on traffic safety has other outlets. For distribution at county safety council exhibits and displays at fairs, conventions, expositions, and general meetings, the Safety Department has prepared many types of handout literature. One is "An invitation to drive home safely—we want you with us at our next meeting." Another is a brochure in color reviewing the types of highway signs—what each one means and where they are located. Another is a card illustrating approved hand signals for turning and stopping. Each publication is short and to the point, designed to carry one message since persons visiting a fair or exposition will not spend time reading a lengthy article.

RADIO PROGRAMS ON HIGHWAY SAFETY GIVEN

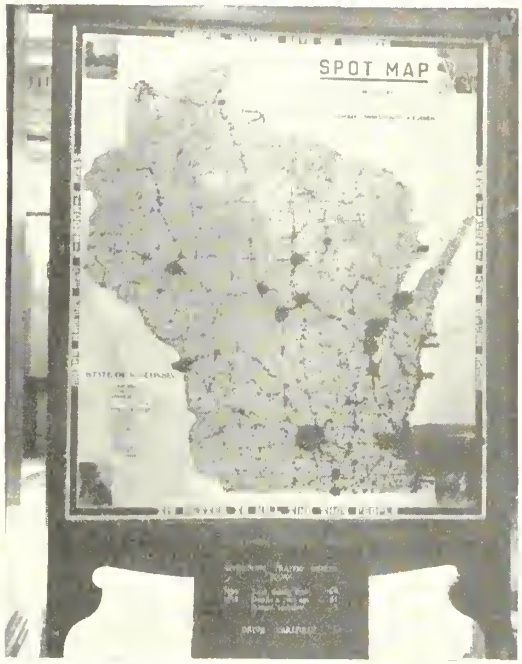
No modern approach to the public is complete without extensive use of the radio. In Wisconsin the radio stations have cooperated generously in furthering the State Highway Commission's safety activities. Each week end six 1-minute radio scripts on highway safety are prepared and sent to each of the 19 radio stations in the State. The scripts are written as traffic warnings, each one stressing some pertinent danger. A survey of the use of these warnings by the radio stations reveals that they are broadcast at those times of day when radio audiences are at the maximum. The 19 Wisconsin radio stations are so located throughout the State that every community is reached by one or more stations.

Longer radio programs, addresses running up to 15 minutes in length, are prepared at intervals and are submitted to the county councils that have access to radio stations. Local speakers present the addresses over their local stations.

Radio broadcasts are of great value in reaching citizens who do not or cannot attend safety meetings. Since radio stations receive their operating licenses with the stipulation that they offer their services for the public good, no radio station charges for the time devoted to safety promotion work.

That press and radio are effective in carrying safety messages to the motorists is revealed by experience over holiday periods such as Memorial Day, the Fourth of July, and the Labor Day week end. In the year 1937 accidents showed a marked increase over 1936, yet on those three holidays in 1937 there were less than half the fatalities there had been in 1936. The only explanation for the decrease on these days of heavy traffic was that the intense publicity campaign in press and radio and by traffic police had been heeded, resulting in greater motoring care over the holidays. Again in 1938 on those same holidays there was a further reduction over the 1937 record—a percentage of reduction greater than the general reduction for the whole year. Of course the campaign in press and radio and by traffic police was repeated and enlarged for the 1938 holidays.

Motion pictures.—The State Highway Commission of Wisconsin believes strongly in the value of motion pictures as an effective aid in safety education. The elements of traffic—drivers, vehicles, pedestrians, streets, highways—all are ideal material for motion pictures. More concrete suggestions for traffic improvement can be packed into an 11-minute film than can be described in 2 hours by a speaker.



Safety Contests, Spot Maps of Traffic Accidents, and Displays of Accident Statistics Are A Few of the Means Used by the County Safety Councils and the Safety Department in Furthering Street and Highway Safety.

The Safety Department has five portable motion picture sound projectors in constant use and a film library of 56 reels of motion pictures. Each of the three district field supervisors has a projector and the other two are used by the men working out of the main office. In schools or at adult meetings where these men appear, the programs they present begin with a short address on pertinent facts on accidents and suggestions for their prevention. Motion pictures make up the remainder of the program. Films not in use by Department representatives are available for free loan to any group in the State requesting them. Schools, service and fraternal clubs, safety councils, industrial plants, and others have borrowed these films. In 1938 alone the films were shown to a total audience of more than 400,000 persons.

The Department's film library is made up of sound and silent reels on a loan basis from automobile companies, insurance companies, and automobile clubs as well as many films it has purchased outright from the producers. The Department does not distribute films that are obviously advertising in nature, but it does not object to the mention of a commercial concern as sponsor to the picture if the film is primarily one of good safety practices presented in a friendly manner.

MOTION PICTURE FILMS ON HIGHWAY SAFETY PRODUCED BY THE COMMISSION

Supplementing the films available by purchase or loan from outside sources, the State Highway Commission produces motion pictures of its own. No attempt is made to film subjects already covered by available films. The Commission's motion picture productions deal with subjects especially vital to its program of accident prevention. The motion pictures produced by the Safety Department had a 1938 circulation of 412 bookings, reaching a total audience of 71,000 persons. One film, "Wisconsin School Safety Patrols," shows how school children must avoid traffic when no protection is provided. Then in contrast it shows how safety is provided by an efficient school safety patrol. Pictures of the operations of many Wisconsin school patrols are also shown. This film is excellent to take into a community to show what can be done by establishing a patrol.

More than one-third of the persons killed in traffic accidents in Wisconsin are pedestrians. To plead for improved pedestrian habits the Department has produced the motion picture, "We Who Walk." A picture of pedestrians, this film shows how pedestrians walk into trouble. The pictures was filmed in Wisconsin and shows pedestrians jaywalking, roaming behind parked cars, crossing the street in midblock, and loitering in the street. The pedestrians themselves show by their careless actions why they are so frequently hit by automobiles. The film closes by showing correct pedestrian behavior under all conditions in city and country.

"Safety News" is another film produced by the State Highway Commission. It is a news-reel type of production showing the results of traffic accidents throughout the State. Safety activities to prevent traffic accidents by various county safety councils are shown in story form—a parade in one county, a unique driver testing device in another, traffic control in a third, and a brake testing lane in a fourth. The film closes with a plea for comprehensive safety activity in all communities.

"Driving Hazards" shows in pictures the usual and some unusual conditions each Wisconsin motor-vehicle operator must encounter in the course of his driving.

"Watch the Road Signs" is an all-color film showing the history of highway signs and the meaning of sign types now in use. The picture opens showing Indians marking their trails. The horse-and-buggy days follow, showing travelers asking their way. Early crossroad signs are shown; signs which were often inadequate to keep the early motorist from taking the wrong road. Then in contrast the modern highway with its comprehensive sign system is shown. The film continues with pictorial explanations of each type of sign, clearly showing how each type differs in purpose and in appearance from the others. It closes with an appeal to drivers to be guided by the highway signs.

The program of motion picture production by the Safety Department calls for four pictures per year. Contemplated productions for the future are on the subjects of bicycling, traffic enforcement, and a newsreel of unusual safety activities in 1939.

The Safety-Department uses only 16-millimeter films, as that size has become the standard for nontheatrical motion pictures. The Department has its own motion picture camera complete with supplementary lenses, film magazines, and titling and editing equipment. Because of the increasing use of motion pictures in schools, in industry, and at public gatherings, motion pictures on highway safety will in the future have more outlets and reach more people. A recent survey of safety aids to teachers, made by the National Education Association, revealed that "more good films on safety" was recorded as a need by more than 50 percent of the teachers. The Safety Department is attempting to do its share in supplying that need.

Conclusion.—In 1938 there were 23 percent fewer fatal highway accidents in Wisconsin than in 1937. This represents a saving of 203 lives. Injuries in traffic accidents were reduced 8.1 percent while all accidents, including those involving only property damage, were reduced 10.5 percent.

How much do traffic accidents in Wisconsin cost? If a human life is valued at $10,000, if an average injury cost of $500 is taken, and if $150 property damage is assumed for each reportable accident, then the 1937 traffic accident cost was $14,773,500. This amounts to $5.05 for every person in the State. Figured on the basis of cost per motor vehicle (1937 registration was 871,592 vehicles) accidents in 1937 cost $16.95 per vehicle—a cost greater than the average Wisconsin motor-vehicle license fee.

On the same basis of valuation, accidents in 1938 cost Wisconsin $12,220,350. The reduction of traffic accidents and fatalities in 1938, figured on the above scale, saved $2,553,150. The Safety Department operates on a budget of $50,000 per year. . Thus for every State dollar spent in highway safety promotional work, a saving of $51.06 in reduced accidents was accomplished in 1938. On the basis of population, the expenditure of 1.7 cents per person in 1938 saved each citizen 87 cents, reducing his annual highway-accident cost from $5.05 to $4.18. The reduction in costs of traffic accidents to each motor vehicle (1938 registration was 857,794 vehicles) was $2.70, bringing the annual traffic-accident cost from $16.95 down to $14.25 per motor vehicle.

The problem of bringing a definite highway safety program to each citizen on a budget of 1 cent and 7 mills per person has been no easy task. To reach almost 3,000,000 persons on a total appropriation of $50,000 calls for a careful appraisal of each activity to see that it reaches the greatest number of persons in the most direct manner to convince them that careful driving is good common sense. The greatest return in safety for the least expenditure has been, and will continue to be, the objective of the Safety Department.

INDEX TO PUBLIC ROADS, VOLUME 19, NOW AVAILABLE

The index to volume 19 of PUBLIC ROADS is now available. In addition to the index a chronological list of articles and a list of authors are given. The index will be sent free to subscribers to PUBLIC ROADS requesting it. Requests should be addressed to the Public Roads Administration, Federal Works Agency, Washington, D. C.

Indexes to volumes 6 to 18, inclusive, are also available and will be sent to PUBLIC ROADS subscribers upon request. Indexes to volumes 1 to 5, inclusive, have never been prepared, and it is not expected that these volumes will ever be indexed.

(Continued from p. 144)

utilized to evaluate c and ϕ. Samples used in stabilometer tests have heights of possibly 10 times the thicknesses of samples tested in direct shear and therefore, according to the theory of consolidation, consolidate only one-hundredth as rapidly and in consequence have considerably less error due to change of moisture content during test in the open system. Such tests made in the Delft Laboratories by what Professor Huizinga terms the quick method (*22*) are considered satisfactory by him for relatively impermeable soils.

It was the consensus of opinion at the Eighteenth Annual Meeting of the Highway Research Board (*36*) that:

> The triaxial compression or stabilometer device is the most useful shearing method and despite all obstacles it is proposed to obtain and use complete stress-deformation diagrams in connection with highway problems.

Among the advantages provided by this method of test may be listed the following:

1. Samples have the shape common to usual compaction, permeability, and sampling devices.
2. Properties of samples as a whole instead of only a fraction thereof can be determined.
3. Samples of embankment materials may be tested as compacted and after they have been tested for permeability and capillarity.
4. Samples of road materials may be tested as prepared and after their subjection to saturation, freezing and thawing, and the like.
5. Samples of foundation soils may be tested in their natural undisturbed state and at several other moisture contents to the end that complete relations of moisture content to c, ϕ, and pore pressure are provided.
6. Samples may be tested at pressures similar to those that soils and roads are expected to resist under service conditions.
7. Uniform pressures are applied on the surfaces of samples.
8. All stresses on the sample are measured and may be varied or kept constant as desired.
9. Both the vertical and horizontal deformations can be controlled and are measurable.
10. The data are usable in theories of design.

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HISTORIC HIGHWAYS ILLUSTRATED

Highways of History, a pictorial story of the improvement of transportation in the United States, has recently been published by the Public Roads Administration.

The 35 pictures the publication contains begin with the introduction of horses on this continent by Hernando De Soto in 1539, and trace chronologically the modes of transportation used in the United States up to the present time with special emphasis on highway transportation. Beside each picture is a brief description of the historical significance of the scene. The pictures are photographic reproductions of dioramas created by the Public Roads Administration and now exhibited at the Golden Gate International Exposition at San Francisco.

The pamphlet has been prepared particularly for the use of teachers in elementary schools and for school libraries. A limited free supply is being distributed by the Public Roads Administration, Federal Works Agency, Washington, D. C. Copies are also available by purchase from the Superintendent of Documents, Washington, D. C., at 25 cents each.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF AUGUST 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 480,680	$ 239,590	19.0	$ 8,581,998	$ 4,266,593	318.5	$ 755,190	$ 375,290	23.1	$ 2,863,176
Arizona				2,123,069	1,430,377	92.0	588,373	368,409	32.9	1,138,637
Arkansas	1,136,548	1,136,037	82.1	2,134,834	2,131,450	138.0	196,261	194,176	4.3	1,741,330
California	3,567,800	1,963,610	39.2	3,058,086	1,657,708	45.4	501,758	265,296	19.8	3,768,993
Colorado	513,050	280,133	9.1	3,800,018	2,122,309	91.0	437,295	246,422	10.1	1,956,104
Connecticut	100,040	50,020	.8	1,978,350	981,636	20.6	159,879	79,939	1.7	1,254,367
Delaware	402,580	200,650	5.7	1,047,253	506,228	28.5	526,644	263,322	4.2	1,018,460
Florida	121,000	60,500	1.4	3,065,361	1,532,681	54.7	1,322,279	660,915	25.3	2,700,462
Georgia	1,596,460	798,230	83.5	6,801,233	3,400,616	358.0	967,651	483,826	67.8	5,589,273
Idaho	561,344	335,701	34.4	2,000,692	1,213,790	87.4	241,539	146,927	13.8	1,210,755
Illinois	1,590,449	787,840	29.2	8,333,951	4,166,328	187.6	3,559,554	1,777,788	69.3	2,745,990
Indiana	1,831,866	915,933	27.8	4,792,414	2,371,607	114.7	2,294,400	1,147,075	44.7	1,859,537
Iowa	586,507	274,500	52.2	5,305,338	2,345,233	177.5	1,283,453	602,175	49.9	940,783
Kansas	401,945	200,972	22.3	3,447,407	1,715,998	150.8	3,524,443	1,761,342	204.6	4,199,353
Kentucky	1,177,522	588,761	25.2	3,414,631	1,705,760	91.2	945,943	472,971	41.9	2,929,70[illegible]
Louisiana				12,259,936	3,175,601	53.3	1,288,985	607,838	37.3	2,589,[illegible]
Maine	717,258	358,628	10.9	2,006,286	1,003,1[illegible]3	52.4	178,230	89,115	4.1	296,[illegible]
Maryland	549,500	273,000	9.9	2,696,021	1,334,611	44.6	973,000	481,505	12.2	1,803,410
Massachusetts	1,314,520	657,260	.7	2,034,245	1,014,433	24.4	1,738,647	865,806	12.1	2,480,172
Michigan	919,948	459,010	25.0	4,421,885	2,208,303	129.4	1,911,850	807,850	50.7	2,815,049
Minnesota	939,988	469,994	60.0	6,203,672	3,083,163	335.4	2,873,929	1,431,311	134.8	3,073,784
Mississippi	631,000	231,470	35.1	8,294,088	3,007,345	337.5	1,276,200	619,450	33.0	2,156,780
Missouri	745,320	372,660	28.8	4,881,356	2,428,226	183.5	2,614,591	1,121,128	69.3	4,342,531
Montana	451,429	254,234	26.6	3,300,956	1,868,498	168.6	28,060	15,913		4,502,018
Nebraska	173,101	86,551	24.2	6,209,395	3,104,328	502.4	2,411,628	1,205,814	280.1	2,611,816
Nevada	742,246	642,664	29.0	608,842	523,560	32.2	335	288		1,309,783
New Hampshire	35,028	17,492	3.5	1,089,750	537,371	24.1	528,418	260,319	17.2	911,394
New Jersey	538,290	269,145	6.0	3,552,816	1,774,858	25.4	739,442	369,721	6.5	1,907,061
New Mexico	127,251	78,717	35.0	1,943,029	1,189,651	97.8	246,886	154,081	14.0	1,404,297
New York	1,113,780	556,890	19.6	14,636,010	7,033,557	242.6	2,481,490	1,085,345	43.5	2,059,318
North Carolina	927,960	463,980	52.8	6,414,123	3,201,482	375.2	1,555,830	770,175	62.1	1,630,657
North Dakota	78,830	42,219	14.6	521,709	279,480	46.1	3,053,450	1,636,589	297.5	3,372,730
Ohio	506,200	253,100	4.6	10,341,726	5,102,474	114.8	2,349,840	1,109,920	21.4	6,529,162
Oklahoma	197,269	103,154	4.9	3,203,482	1,700,365	75.4	1,875,669	997,704	99.5	3,520,579
Oregon	380,270	231,860	21.5	2,589,728	1,567,887	106.5	873,095	514,730	44.6	1,764,978
Pennsylvania	1,078,604	539,302	15.4	10,408,627	5,018,661	99.5	2,514,555	1,253,601	32.7	4,224,357
Rhode Island	129,150	64,575	2.6	689,376	344,381	7.6	380,506	189,885	4.3	1,043,358
South Carolina	537,210	241,000	20.7	2,324,824	1,036,486	65.6	232,000	98,000	24.7	2,397,197
South Dakota	830,555	459,214	102.1	4,014,859	2,243,310	374.6	1,235,190	696,300	92.6	3,357,105
Tennessee	223,860	111,930	4.0	4,155,644	2,077,822	117.1	414,640	207,320	9.0	4,392,963
Texas	3,836,494	1,886,900	249.6	10,206,312	5,051,553	462.2	1,403,864	683,185	47.1	6,511,955
Utah	882,760	632,080	36.7	1,613,260	1,168,085	80.0	210,510	147,590	13.5	822,644
Vermont	459,811	225,631	11.1	280,548	140,034	7.0	147,400	73,700	4.8	630,064
Virginia	966,130	482,520	28.6	2,925,287	1,420,890	75.0	835,657	412,114	24.5	1,000,657
Washington	1,125,541	587,959	6.8	2,428,432	1,266,640	32.3	840,626	323,758	9.2	1,079,838
West Virginia	534,427	303,875	19.4	2,400,755	1,217,614	53.6	1,170,603	584,957	38.0	1,846,232
Wisconsin	1,690,625	834,523	74.6	7,685,324	3,784,610	233.9	987,154	446,255	35.3	1,631,793
Wyoming	364,641	225,241	34.2	1,197,223	747,754	104.3	703,200	435,219	87.3	780,297
District of Columbia				130,624	65,312	.8	264,800	132,400	1.7	269,788
Hawaii	133,706	64,635	1.0	994,580	475,460	16.3	574,217	282,968	9.8	1,056,640
Puerto Rico	302,230	150,315	6.4	1,555,549	772,020	31.2	96,128	47,440	1.2	405,020
TOTALS	38,252,723	20,464,205	1,457.8	210,104,914	103,517,282	6,688.5	58,315,287	29,005,227	2,289.0	118,468,320

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF AUGUST 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 186,105	$ 91,750	13.7	$ 873,345	$ 349,000	19.4	$ 90,100	$ 45,050	9.7	$ 739,456
Arizona	55,191	40,524	11.0	225,057	161,795	22.4	21,128	15,238		322,699
Arkansas	286,997	284,639	35.3	251,392	250,221	35.8	161,841	161,790	19.2	321,892
California	110,917	63,199	17.1	1,031,194	529,642	31.2				762,115
Colorado	140,928	78,065	7.8	661,208	346,829	20.8	95,856	46,892	4.8	114,772
Connecticut				172,794	72,417	2.9				286,249
Delaware	80,840	40,420	17.5	73,930	36,965	7.8				231,250
Florida	106,317	52,800	3.4	883,305	437,294	34.2				374,950
Georgia	160,720	80,360	20.1	326,120	163,060	39.1	133,260	66,630	19.2	1,073,008
Idaho	101,455	60,357	1.5	368,562	174,486	40.3				202,031
Illinois	457,113	228,216	17.0	1,337,200	614,600	89.0	255,100	127,550	20.6	690,385
Indiana	232,000	116,000	18.5	895,170	446,381	71.4	130,926	65,463	9.7	660,414
Iowa	11,587	5,328	11.8	94,399	44,241	15.4	480,447	225,635	86.5	1,404,603
Kansas				47,588	23,794	11.7	583,633	296,596	41.7	1,259,390
Kentucky	117,736	41,185	9.7	1,061,032	286,860	66.4	856,726	296,918	72.0	223,613
Louisiana	202,622	98,805	19.6	515,473	232,522	38.2	363,402	160,120	30.3	341,124
Maine	211,924	105,962	11.8	249,556	123,692	15.2	48,500	24,250	2.3	6,575
Maryland	57,600	28,800	7.8	143,670	71,835	8.2	205,000	72,055	12.6	362,991
Massachusetts				344,984	171,164	7.6	372,470	184,000	7.5	434,504
Michigan	223,900	111,950	7.1	1,197,904	582,752	97.3	337,500	168,750	17.3	873,402
Minnesota	184,910	92,318	10.6	704,453	350,319	66.3	238,593	119,296	32.2	1,096,571
Mississippi	176,500	88,250	6.8	500,662	224,646	36.3	406,800	213,300	41.4	624,670
Missouri	166,164	82,230	25.8	761,040	372,906	92.3	629,715	260,918	71.0	551,104
Montana	111,913	63,475	10.8	702,330	398,292	58.3	61,495	32,683	5.9	837,578
Nebraska	301,935	141,625	63.9	846,530	416,708	138.8	158,745	79,372	31.3	378,528
Nevada	109,499	94,925	15.0	117,798	101,366	26.5	26,563	23,035	1.6	122,530
New Hampshire				62,951	30,804	2.4	43,023	20,639	.9	168,522
New Jersey				429,020	211,770	15.0	136,820	68,410	9.4	511,778
New Mexico	27,411	17,107	1.8	422,613	254,401	26.3	44,885	28,013	12.7	225,025
New York	366,400	183,200	18.9	1,965,760	971,930	99.6	833,600	285,572	11.3	488,791
North Carolina	281,780	140,890	20.0	1,094,224	547,090	104.9	60,550	29,760	7.4	329,105
North Dakota	115,030	61,606	8.3				107,790	57,757	10.7	841,099
Ohio	94,160	47,080	6.1	757,570	384,250	39.3	296,000	148,000	9.7	1,716,022
Oklahoma	73,190	38,943	.8	219,796	107,065	9.8	513,715	273,338	28.5	908,099
Oregon	200,005	120,440	28.4	532,452	301,842	47.2	79,829	15,820	12.6	291,255
Pennsylvania	1,168,909	572,778	70.9	1,248,548	618,169	54.7	598,162	294,781	23.9	370,483
Rhode Island	41,487	20,720	.7	57,848	28,924	1.4	72,008	36,004	.2	98,167
South Carolina	219,307	87,390	21.3	364,600	151,679	35.6	330,400	142,044	22.2	204,807
South Dakota				12,340	6,790		5,070	2,790	4.1	1,048,470
Tennessee	343,180	147,890	18.0	390,836	179,948	14.0				858,499
Texas	1,034,825	481,034	132.1	1,387,460	677,595	110.3	190,339	94,165	36.1	1,071,473
Utah	108,785	60,906	14.7	112,815	61,957	20.8	17,800	12,000	4.0	197,199
Vermont	91,158	45,153	4.0	102,682	51,341	3.1	123,680	23,642	4.7	56,385
Virginia	335,770	160,389	34.0	326,214	161,471	31.5	285,600	133,335	19.4	220,142
Washington	386,938	202,678	19.8	383,574	201,118	29.1	103,829	53,400	11.2	210,806
West Virginia	108,950	54,475	6.2	49,215	24,607	2.1				513,414
Wisconsin	195,748	97,693	18.4	866,455	432,527	15.9	279,545	138,268	8.5	580,770
Wyoming	406,669	251,210	22.3				343,427	211,171	33.6	55,604
District of Columbia				14,592	6,796	.1	38,500	19,250	.5	47,079
Hawaii	91,030	45,515	3.7	214,970	107,485	4.6				167,000
Puerto Rico				178,504	86,825	10.4	101,148	49,445	4.6	60,233
TOTALS	9,486,605	4,928,280	814.0	25,581,735	12,590,171	1,771.9	10,263,520	4,823,145	813.0	25,536,631

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Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

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Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

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STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF AUGUST 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 474,150	$ 473,200	3	1		$ 784,912	$ 783,384	13			$ 28,108	$ 27,900		1	1	$ 815,773
Arizona						469,516	443,841	5			69,362	69,362	1			211,730
Arkansas	104,053	104,053	2			85,838	85,838	1			719,056	710,203	5		8	581,453
California	288,422	288,422	3			1,494,155	1,493,060	8								1,303,375
Colorado	294,383	294,383	3		4	313,042	292,252	2		5	52,046	48,284			16	817,198
Connecticut						172,722	161,008		1							850,557
Delaware						9,150	9,150	2			2,320	2,320			1	513,891
Florida						503,994	503,994	3			130,037	129,201	1		1	1,032,656
Georgia	27,440	27,440	1			399,840	399,840	6			144,673	144,673	2	2	10	2,294,067
Idaho						314,492	282,961	4								466,886
Illinois	1,473,775	1,472,915	7	2	13	1,889,760	1,766,372	13	1	37	381,405	363,755	1	1	23	2,083,108
Indiana	208,408	208,408	2		8	787,675	787,675	2	1	61	480,309	479,309	1	2	21	827,038
Iowa	264,726	249,000	5			196,228	165,906	7		2	533,515	499,500	3		123	1,188,268
Kansas	245,805	245,805	3			695,011	695,011	8		3	466,826	466,826	5		5	1,075,447
Kentucky	100,576	100,576	1	1		675,746	627,797	10	3		671,980	671,980	6			522,281
Louisiana						594,134	594,116	5			670,863	617,362	14			584,469
Maine	259,690	259,690	2	2		291,655	291,655	3	1							220,802
Maryland						291,197	194,405	2		2	46,200	46,200			13	986,891
Massachusetts	104,330	104,220		1		418,236	417,082	4	1		14,320	14,320	1			1,711,447
Michigan	162,900	162,900	1	1		710,976	710,976	6	1		547,920	547,920	2		39	1,603,746
Minnesota	41,045	41,045	1	1		1,359,812	1,342,381	7	5	11	48,099	48,099	1		2	1,489,278
Mississippi						606,714	606,714	8			37,300	37,300	1			894,187
Missouri	65,589	64,284	1			1,197,536	1,197,536	7	1		474,961	435,060	1	4		1,613,080
Montana	439,450	439,450	4			458,244	458,244	6			147,055	50,805	2			276,452
Nebraska	28,481	28,481	1			1,124,384	1,124,384	24	1		149,269	149,269			41	637,289
Nevada	119,640	119,640		1		50,017	50,017		1		36,218	36,218			13	105,520
New Hampshire	11,649	11,649	3		1	150,935	150,431	5			42,980	42,980	1			316,424
New Jersey						629,721	629,721	2	2	1	119,560	119,560		1		1,426,875
New Mexico						75,081	75,081	2			2,572	2,572			1	682,071
New York	440,550	439,450		3		2,029,732	1,975,462	8	8		1,068,844	901,543	5	3		3,679,243
North Carolina	315,050	315,050	3	1	13	1,065,270	1,030,170	6	4	6	231,650	231,650	4		27	955,265
North Dakota	60,590	60,590	1	1		858,712	810,310	11			75,960	75,960	1			395,838
Ohio	63,730	63,730	2			1,545,013	1,508,251	13	1		438,780	389,590	2	1	2	3,101,450
Oklahoma	193,200	159,200	2		7	104,880	104,880			35	307,750	295,850	4	1	8	2,112,947
Oregon	21,689	21,689				148,072	146,777	1			135,740	135,740	2			311,060
Pennsylvania						2,161,799	1,949,900	5	3		620,652	412,200	2	1	2	4,301,128
Rhode Island	103,716	103,716		2		335,075	335,075	1	1							152,459
South Carolina	15,630	14,428		1		633,716	579,200	8	2		225,479	225,479	2	2	35	854,114
South Dakota	8,190	8,190			2	307,520	307,520	3	2	9	83,750	83,750	1	1	6	1,052,620
Tennessee	2,700	2,700			1	654,506	654,506	2	2		239,060	239,060	1	2	4	1,314,533
Texas	196,135	195,800	2	1		2,703,230	2,672,102	23	2		707,549	630,930	5		26	1,894,533
Utah	51,810	51,810	2		6	61,408	61,408	1		23	297,280	297,280			110	186,570
Vermont	18,878	14,256			2	17,642	17,452			5						318,248
Virginia	80,557	80,557	1		11	620,110	527,210	8	2	5	135,880	135,880		1	4	923,109
Washington	33,602	33,602			4	293,142	291,732	3	1	2	66,494	66,494		1	6	505,318
West Virginia	40,817	40,817				334,034	318,274	7			20,600	20,600	1		2	959,142
Wisconsin	195,093	194,417	2		1	1,315,849	1,271,665	13	1	1	154,014	154,006		1	9	1,147,209
Wyoming	27,424	27,424	1		3	111,589	111,589		1	4						516,862
District of Columbia						292,412	258,868	1								119,318
Hawaii	50,320	50,320	1			132,850	132,850	3	1							359,450
Puerto Rico						394,352	392,150	9								426,676
TOTALS	6,634,193	6,573,307	60	19	76	32,871,636	31,798,183	291	50	212	10,826,436	10,056,990	78	25	559	52,719,351

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

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PUBLIC ROADS ►►► *A Journal of Highway Research*

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PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 20, No. 8 October 1939

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CERTIFICATE: By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

DESIGN OF A FILL SUPPORTED BY CLAY UNDERLAID BY ROCK

AN APPLICATION OF SOIL MECHANICS IN SOLVING A HIGHWAY FILL PROBLEM

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by L. A. PALMER, Associate Chemist

THIS REPORT is a continuation of the theoretical considerations contained in two previous publications.[1] [2] Its purpose is to present in usable form the analytical methods based on the assumption of conditions of plane strain [2] and to extend these analyses to include the problem of determining the supporting power of a clay stratum supporting a symmetrical earth fill when the clay stratum is underlaid by rock.

As shown in one of the previous publications [2] a problem involving plane strain conditions is one involving two dimensions. The load is distributed over an area that is quite long as compared to its width and the analytical procedure is applied to a vertical cross section of unit thickness in the direction of the longitudinal axis of the load. This is taken as the Y direction. It is considered that there is no displacement of material in this direction and that whatever soil movements occur are in the Z direction, which is toward the center of the earth, and in the X or horizontal direction, that is, perpendicular to both the Y and Z directions.

The analytical procedures used in the theoretical solution of the present problem involve two theories, that of elasticity and that of plastic equilibrium, and four principal assumptions are involved. The first three are common to both theories. The fourth is made only when the theory of plastic equilibrium is applied. These are:

1. The strength of the clay stratum depends essentially on its cohesion. The strength due to the element of friction is comparatively small and may be neglected. Hence, whenever and wherever the unit shearing stress becomes equal to the unit cohesion, c, the soil becomes plastic and undergoes plastic flow; that is, the soil fails.

2. The adhesion of the clay to the rock surface is "perfect." No slippage occurs at this surface although there may be lateral movement in the clay at points very near the rock surface.

3. The soil deformations considered in this paper are those that occur at an assumed constant volume. It seems reasonable to assume that the deformations caused by lateral yield in the X direction occur during a period of time that is brief in comparison with the time required for an appreciable degree of consolidation of the stressed clay stratum. When deformations occur at constant volume, Poisson's ratio is taken as $\frac{1}{2}$ (the approximate value).

4. In applying the method of plastic equilibrium it is considered that the fill acts like an absolutely rigid body in its production of stresses in the clay stratum when the soil is in the plastic state. Thus the fill above and

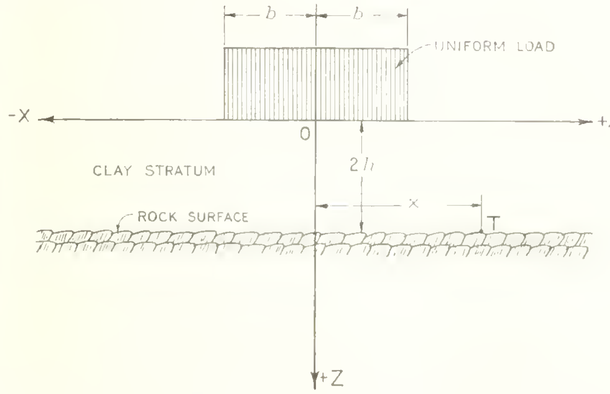

FIGURE 1.—UNIFORM LOAD ON A LONG STRIP SUPPORTED BY CLAY UNDERLAID BY SOLID ROCK.

the solid rock boundary below the clay constitute a "nutcracker."

Probably the fourth assumption is the least valid of the four.

Since it is assumed that there is no displacement either in the fill or in the supporting soil in the direction of the longitudinal (Y) axis of the fill, the problem is one of plane strain. One vertical cross section perpendicular to the Y axis is the same as any other insofar as stresses and deformations are concerned, assuming, of course, that both the fill material and the supporting clay are, in themselves, homogeneous. Since the rock is supposedly rigid, it follows that there is no vertical displacement of soil at this boundary.

STRESSES IN THE CLAY COMPUTED FROM THEORY OF ELASTICITY

Carothers [3] has shown that for a uniform load p per unit area on a long strip of width $2b$ (see fig. 1) at the surface, the shearing stress, s_{xz}, at the rock surface is

$$s_{xz}=\frac{p}{2}\left[\operatorname{sech}\frac{\pi}{2}\frac{x-b}{2h}-\operatorname{sech}\frac{\pi}{2}\frac{x+b}{2h}\right] \quad (1)$$

where $2h$ is the thickness of the intervening clay layer.

This expression for s_{xz} for uniform strip loading and other expressions for stresses for other types of surface loading (see for example equation 12) are developed from the theory of elasticity. When these expressions are used it is considered that the clay mass has not been stressed to its ultimate supporting power and is therefore not reduced to a plastic condition throughout.

In the following discussion equations 2, 3, 4, and 8 are those frequently seen in texts on the theory of elasticity.[4]

[1] Principles of Soil Mechanics Involved in Fill Construction, L. A. Palmer and E. S. Barber, Proceedings Highway Research Board, Annual Meeting 1937.
[2] Principles of Soil Mechanics Involved in the Design of Retaining Walls and Bridge Abutments, L. A. Palmer, PUBLIC ROADS, vol. 19, No. 10, December 1938.

[3] Test Loads on Foundations as Affected by Scale of Tested Area, S. D. Carothers, Proceedings International Mathematical Congress, Toronto, 1924, pp. 527-549.
[4] See, for example, pp. 8-20, inclusive, of Theory of Elasticity, by S. Timoshenko. McGraw-Hill Book Co., 1st. ed., 1934.

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The fundamental strain relations are

$$\epsilon_x=\frac{1}{E}\left[p_x-\mu(p_y+p_z)\right] \quad (2)$$

$$\epsilon_y=\frac{1}{E}\left[p_y-\mu(p_x+p_z)\right] \quad (3)$$

$$\epsilon_z=\frac{1}{E}\left[p_z-\mu(p_x+p_y)\right] \quad (4)$$

where ϵ_x, ϵ_y, and ϵ_z are the strains and p_x, p_y, and p_z are the normal stresses in the X, Y, and Z directions, respectively; E is Young's modulus; and μ is Poisson's ratio.

Since $\epsilon_y=\epsilon_z=0$ at the rock surface and since $\mu=\frac{1}{2}$, equation 3 becomes

$$p_y=\frac{p_x+p_z}{2} \quad (5)$$

and equation 4 becomes

$$p_z=\frac{p_x+p_y}{2} \quad (6)$$

By substituting for p_y in equation 6 from equation 5,

$$p_z=p_x \quad (7)$$

which is true at the boundary of rock and clay.

The maximum shearing stress, $s_{max.}$, at any point of the undersoil is

$$s_{max.}=\left[\frac{p_z-p_x}{2}^2+s_{xz}^2\right]^{1/2} \quad (8)$$

which (since $p_z=p_x$ at the rock surface) becomes

$$s_{max.}=s_{xz} \quad (9)$$

at all points along the rock surface. Hence at the rock boundary equation 1 becomes

$$s_{max.}=\frac{p}{2}\left[\operatorname{sech}\frac{\pi}{2}\frac{x-b}{2h}-\operatorname{sech}\frac{\pi}{2}\frac{x+b}{2h}\right] \quad (10)$$

which is the expression for the shearing stress at any point T of the rock surface (see fig. 1). For a triangular loading, $dp'=\frac{p}{b}dB$ (see fig. 2), where B is any variable horizontal distance from the OZ axis to the slope. By differentiating s with respect to p in equation 10 and substituting $\frac{p}{b}dB$ for dp', there is then obtained

$$ds_{max.}=\frac{p}{2b}\left[\operatorname{sech}\frac{\pi}{2}\frac{x-B}{2h}-\operatorname{sech}\frac{\pi}{2}\frac{x+B}{2h}\right]dB \quad (11)$$

This is the shearing stress at T due to the shaded horizontal element of figure 2. Integration between the limits, 0 and b, yields for all such elements

$$s_{max.}=\frac{4h}{b}\frac{p}{\pi}\left[2\arctan e^{\frac{\pi}{2}\frac{x}{2h}}-\arctan e^{\frac{\pi}{2}\frac{x+b}{2h}}-\arctan e^{\frac{\pi}{2}\frac{x-b}{2h}}\right] \quad (12)$$

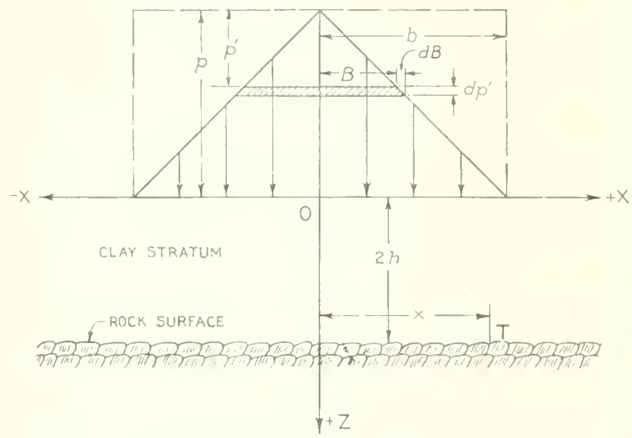

FIGURE 2.—TRIANGULAR LOAD ON A LONG STRIP SUPPORTED BY CLAY UNDERLAID BY ROCK.

This is Jürgenson's [5] formula for the shearing stress at a point T of the rock surface when the loading is triangular. (See fig. 2.) The use of equations 10 and 12 is not dependent on the relative magnitudes of h and b.

From equation 12, the greatest value of $s_{max.}$, denoted by s_g, depends on the ratio of b to h. For example, if the depth to the rock surface, $2h$, is $\frac{1}{2}b$, then $s_{max.}=s_g=0.318p$ at the point $x=0.625b$. If the clay has no friction, the plastic condition for these relative dimensions begins to be developed at the point $x=0.625b$ at the rock surface when

$$s_{max.}=s_g=c=0.318p$$

or when p (see fig. 2)$=3.14c$ where c is the unit cohesion.

Similarly, for $2h=\frac{1}{4}b$, $s_{max.}=s_g$ at $x=0.67b$ and the plastic zone begins when

$$s_{max.}=s_g=c=0.22p$$

or when p (see fig. 2)$=4.55c$.

For any fixed ratio, $b:h$, ordinate values of $s_{max.}$ may be plotted against x as abscissa, using equation 12. The value of x, where $s_{max.}=s_g=$the greatest shearing stress, is the maximum ordinate of the curve thus obtained.

HENCKY'S METHOD OF PLASTIC EQUILIBRIUM IS FUNDAMENTAL

The application of the method of plastic equilibrium to this problem involving the boundary conditions illustrated in figures 1, 2, 3, 4, and 5 is limited to the condition that the distance, $2h$, must not exceed the distance, $b/2$ where $2h$ is the thickness of the clay layer and b is half the base width of the loaded surface area.

A thin layer of soil between two rigid plates whose surfaces in contact with the soil are rough and which are of great length and of width $2b$ (see fig. 3) is considered. The soil is supposed to have cohesion and a zero or very small value for its effective angle of internal friction. The method of Hencky [6] will now be shown

[5] The application of Theories of Elasticity and Plasticity to Foundation Problems, by Leo Jürgenson, Journal of the Boston Society of Civil Engineers, vol. 21, No. 3, 1934.

[6] Über Statisch bestimmte Fälle des Gleichgewichtes in plastischen Körpen, H. Hencky, Zeitschrift für ang Mathematik und Mechanik, 1924, vol. 3, p. 291, p. 401. See also Plasticity, Chapter 33, A. Nadai, 1931. McGraw-Hill Book Co.

as originally devised and applied by Prandtl[7] to the problem illustrated by figure 3, the plastic flow of soil from between two rigid plates. Certain equations for stresses will be derived in this application. Then these expressions for the stresses will be used in the solution of the problem of the fill, *ABCD*, figure 4, supported by a clay stratum underlaid by rock. First of all it is assumed (figs. 3 and 4) that h is either equal to or less than $b/4$. In no case in the following development may h be considered as greater than $b/4$. The solution follows.

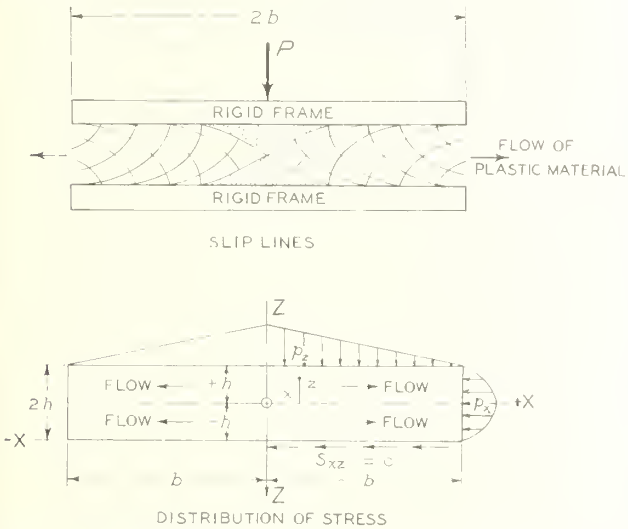

FIGURE 3.—CONDITIONS AT FAILURE IN A PLASTIC MATERIAL PRESSED BETWEEN TWO ROUGH PARALLEL PLATES.

When the material pressed between the plates by a load P (see fig. 3) becomes a plastic mass, flow occurs with a constant maximum shear expressed by the equation,

$$s_{\max.}=\sqrt{\left[\frac{p_z-p_x}{2}\right]^2+s_{xz}^2}=\text{the unit cohesion } c \text{ or}$$

$$p_z-p_x=\pm 2\sqrt{c^2-s_{xz}^2}\text{------}(13)$$

for according to theory, $s_{max.}$=constant=c under these conditions. There are two other equations of equilibrium, namely,

$$\frac{\partial p_x}{\partial x}+\frac{\partial s_{xz}}{\partial z}=0\text{------}(14)$$

and

$$\frac{\partial p_z}{\partial z}+\frac{\partial s_{xz}}{\partial x}=0\text{------}(15)$$

The stresses p_x, p_z, and s_{xz} may be determined from equations 13, 14, and 15. Differentiating 15 with respect to x and 14 with respect to z and subtracting, there is obtained

$$\frac{\partial^2}{\partial x\partial z}(p_z-p_x)=\frac{\partial^2 s_{xz}}{\partial z^2}-\frac{\partial^2 s_{xz}}{\partial x^2}\text{------}(16)$$

substituting equation 13 in equation 16,

$$\pm 2\frac{\partial^2}{\partial x\partial z}\sqrt{c^2-s_{xz}^2}=\frac{\partial^2 s_{xz}}{\partial z^2}-\frac{\partial^2 s_{xz}}{\partial x^2}\text{------}(17)$$

[7] L. Prandtl, Zeitschrift für ang., Mathematik und Mechanik, vol. 6, 1923.

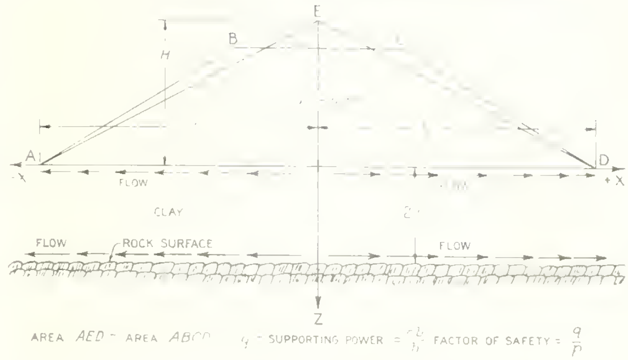

FIGURE 4.—SUPPORTING POWER OF CLAY LAYER UNDERLAID BY ROCK, METHOD OF HENCKY.

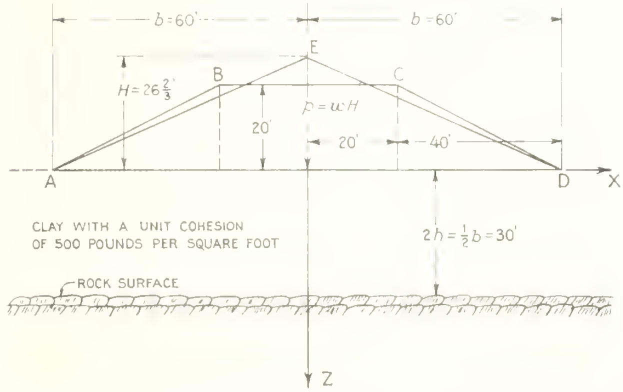

FIGURE 5.—PROBLEM OF THE SUPPORTING POWER OF A CLAY STRATUM SANDWICHED BETWEEN A FILL, *ABCD*, AND SOLID ROCK.

Equation 17 is now solved by assuming that s_{xz} depends on z alone and not on x. When this is true equation 17 reduces to

$$\frac{\partial^2 s_{xz}}{\partial z^2}=0\text{------}(18)$$

which is readily integrable, and there is obtained

$$s_{xz}=K_1+K_2z\text{------}(19)$$

The shearing stress s_{xz} cannot anywhere exceed c, the unit cohesion. If K_1 be taken as zero, there are two straight lines (the upper and lower boundaries, fig. 3), the equations of which are $z=+h$ and $z=-h$ along which the shearing stress s_{xz} becomes $s_{\max.}=c$ since by equation 9, $s_{\max.}=s_{xz}$ at the rock (rigid) surface. In the present case there are two rigid surfaces, at $z=\pm h$, which form natural limits for the plastic mass. The sign of K_2 in equation 19 depends on whether $s_{xz}=+c$ or $s_{xz}=-c$ for $z=h$. If for $z=+h$, $s_{xz}=+c$, then for $K_1=0$, equation 19 becomes

$$s_{xz}=s_{\max.}=+c=K_2h$$

or

$$K_2=+\frac{c}{h}$$

and therefore for any value of z between $+h$ and $-h$,

$$s_{xz}=+\frac{cz}{h}\text{------}(20)$$

by substitution in equation 19.

Now, from equation 14,

$$\frac{\partial p_x}{\partial x}=-\frac{\partial s_{xz}}{\partial z}=-\frac{\partial}{\partial z}\left[+\frac{cz}{h}\right]=-\frac{c}{h}\text{------} \quad (21)$$

and from equation 15,

$$\frac{\partial p_z}{\partial z}=-\frac{\partial s_{xz}}{\partial x}=-\frac{\partial}{\partial x}\left[+\frac{cz}{h}\right]=0\text{------} \quad (22)$$

By integration, equations 21 and 22 yield

$$p_x=-\frac{cx}{h}+f_1(z)\text{--------------} \quad (23)$$

and

$$p_z=f_2(x)\text{--------------} \quad (24)$$

respectively, where f_1 (z) is a function of z alone and f_2 (x) is a function of x alone. Both f_1 (z) and f_2 (x) must be so determined that equation 13,

$$p_z-p_x=\pm 2\sqrt{c^2-s_{xz}^2}\text{----------} \quad (13)$$

will be satisfied. Equation 13 is called the "condition of plasticity." Substituting the values for p_z and p_x as given in equations 23 and 24 and for s_{xz} from equation 20 in equation 13 there results,

$$f_2(x)+\frac{cx}{h}-f_1(z)=\pm 2c\sqrt{1-z^2/h^2}\text{------} \quad (25)$$

Putting $x=0$ in equation 25. Then

$$f_1(z)=K\mp 2c\sqrt{1-z^2/h^2},\text{ where } K=f_2(0).$$

Putting $z=0$ in equation 25. Then

$$f_2(x)=K-\frac{cx}{h},\text{ where } K=f_1(0)\pm 2c.$$

It may be easily shown that $f_1(0)\pm 2c=f_2(0)$. Hence the symbol K may denote either value.

By substitution in equations 23 and 24 there results,

$$p_x=K-\frac{cx}{h}\mp 2c\sqrt{1-z^2/h^2}\text{--------} \quad (26)$$

and

$$p_z=K-\frac{cx}{h}\text{-------------} \quad (27)$$

where K is a constant.

Equations 26 and 27, together with equation 20, completely determine the stresses at any point in the plastic mass when K is known. With reference to figure 3, when $z=+h$ and $x=b$, $p_z=0$ so that by substitution in equation 27

$$0=K-\frac{cb}{h}.$$

or

$$K=+\frac{cb}{h}.$$

Therefore

$$p_x=\frac{c(b-x)}{h}\mp 2c\sqrt{1-z^2/h^2}\text{--------} \quad (28)$$

$$p_z=\frac{c(b-x)}{h}\text{-------------} \quad (29)$$

and

$$s_{xz}=+\frac{cz}{h}\text{----------------} \quad (20)$$

At the boundaries, $z=+h$ and $z=-h$,

$$p_z=p_x=\frac{c(b-x)}{h}\text{ and } s_{xz}=s_{\max.}=\pm c.$$

HENCKY'S METHOD APPLICABLE IN FILL DESIGN

From equation 29 it is seen that p_z is a maximum when $x=0$, and diminishes as x increases (b is positive on the right and negative on the left of OZ). The loading on the surfaces of plastic clay is therefore triangular as shown in figure 3, although the load applied to the rigid frames is uniform.

The problem illustrated in figure 4, a fill, $ABCD$, supported by a clay stratum underlaid by rock, is considered next. The computation of the supporting power, q, of the soil layer, figure 4, is based on the assumption that the structure, $ABCD$, is absolutely rigid. This assumption is equivalent to saying that the soil layer is between two rigid frames, the fill above and the rock below. But in order to use equations 28, 29, and 20, derived for soil between two plates, there must be made another simplifying assumption for the problem illustrated in figure 4, which is that the resistance to flow offered by the soil in the clay layer to the left of A and to the right of D (figure 4) is small enough (relatively) to be neglected.

With all these simplifying assumptions, equations 28, 29, and 20 apply in computing the supporting power, q, of the soil layer, figure 4. Since the structure, $ABCD$, is rigid, then according to equation 29 the distribution of vertical pressure, p_z, at the upper boundary (figure 4) is triangular. The same vertical stress distribution at this boundary would be realized in fact if the load diagram, $ABCD$, becomes triangular, AED, the area of $ABCD$ and that of AED being identical since the total load of the fill cross section (1 foot thick in the direction perpendicular to the plane of fig. 4) is the same.

The total vertical force, P, on a strip of unit width ($y=1$, fig. 4) on the plane boundary, $z=h$, is

$$P=2\int_0^b p_z dx=2\int_0^b \frac{c(b-x)dx}{h}$$

or

$$P=\frac{cb^2}{h}\text{---------------} \quad (30)$$

But $P=pb$ from figure 4, where p is the maximum surface load per unit area and hence

$$P=\frac{cb^2}{h}=pb$$

or

$$p=\frac{cb}{h} \qquad (31)$$

The factor of safety against overloading of the clay stratum is q/p, q being the supporting power. At the instant of failure, $q=p=\frac{cb}{h}$.

A comparison of values obtained by the elastic theory on the one hand and the theory of plasticity on the other is now considered. It has already been shown that for $2h=\frac{1}{2}b$, the plastic zone starts to appear when the magnitude of p is such that $p=3.14c$. From equation 31 plastic flow of the entire soil mass below the fill begins when

$$q=p=\frac{cb}{h}=\frac{cb}{\frac{b}{4}}=4c$$

when

$$2h=\frac{1}{2}b \text{ or } h=\frac{b}{4}.$$

Hence for a comparison:

1. By the elastic theory, a plastic zone is started when $p=3.14\ c$.
2. By the theory of plastic equilibrium the ultimate bearing capacity q of the supporting soil is $q=4\ c$.

Thus for $2\ h=\frac{b}{2}$ the development of a plastic zone or region in the supporting soil mass begins when p is $\frac{3.14}{4}\times100$ or 78.5 percent of the ultimate supporting power. Similarly when $2\ h=\frac{b}{4}$, the plastic zone is started when the value of p is $\frac{4.55}{8}\times100$ or 57 percent of the ultimate bearing capacity or supporting power.

APPLICATION OF THEORY ILLUSTRATED

Suppose that it is required to know the factor of safety with respect to the supporting power of the soil below the fill, $ABCD$, figure 5, when the following conditions obtain:

1. $b=4\ h=60$ feet.
2. The fill, $ABCD$, is symmetrical with a 2 : 1 slope.
3. The height of the fill is 20 feet and the top width BC is 40 feet.
4. The unit weight w of fill material is 100 pounds per cubic foot.
5. The supporting soil is essentially clay. Its cohesion is 500 pounds per square foot and its angle of internal friction is too small to consider. It is then assumed that all of the supporting power is due to cohesion.

The area of the trapezoid, $ABCD$, is $\frac{BC+AD}{2}\times$height$=\frac{40+120}{2}\times20=1{,}600$ square feet. The area of triangle AED is also 1,600 square feet and its height H is $\frac{1600}{60}=26.67$ feet. Then p is equal to $wH=100\times26.67=2{,}667$ pounds per square foot. q is equal to $4\ c=4\times500$ or 2,000 pounds per square foot. The factor of safety against failure of the undersoil is then

$$F=q/p=\frac{2000}{2667}=3/4.$$

Therefore the supporting soil will fail under the fill of the proposed dimensions. For the undersoil to be safe, the height H of the triangle AED must be reduced since $p=wH$ must be reduced. If the width of the roadway (BC, fig. 5) remains 40 feet and the height of the fill, $ABCD$, is reduced to 12 feet, the area of $ABCD$ is then $\frac{40+120}{2}\times12=960$ square feet and the height of the equivalent triangle is $\frac{960}{60}=16$ feet. The value p is then 1,600 pounds per square foot and

$$F=q/p=\frac{2000}{1600}=1\tfrac{1}{4}.$$

It has been shown[1] that for a cohesive soil (with no angle of internal friction) extending downward to a great depth the bearing capacity, q, for the soil supporting a symmetrical fill, as computed by two different methods, is as follows:

Method	*Value of q in terms of unit cohesion*
Terzaghi	$q=4c$ (assuming fill is nonrigid).
Prandtl	$q=5.14c$ (assuming fill is rigid).

In the foregoing example, if the rock boundary were removed and the clay extended far below it, the value of q according to Prandtl would be computed as being $5.14\times500=2{,}570$ pounds per square foot which is larger than the value, 2,000 pounds per square foot, as found in the example. On the other hand with different relative values of b and h and the same fill as that considered in the example, the supporting power q of the clay stratum could be much greater than 2,000 pounds per square foot. Thus for h equal to $\frac{b}{8}$, $q=\frac{cb}{h}=\frac{8cb}{b}=8c=4{,}000$ pounds per square foot, a value that is much greater than that obtaining when the rock layer is nonexistent. If this condition had existed in the preceding example, the factor of safety (all other conditions being the same) would have been

$$F=\frac{q}{p}=\frac{4000}{2667}=1.5.$$

This is in accord with common sense and experience. It is obviously more difficult to "squeeze out" a thin layer of soil from between two rough steel blocks than it is to cause a much thicker layer of the same soil to flow out laterally. There is always the practical consideration that as the clay layer becomes increasingly thin, it is less a major item of cost to excavate and place the fill directly on the solid rock.

SUMMARY

Subsequent to construction a new fill tends to consolidate the supporting clay. Prior to the realization of any appreciable degree of consolidation, the fill load is carried for the most part by water in the supporting clay mass. Thus initially the superimposed fill load theoretically causes no contact pressure between solid particles and therefore no frictional force is developed by the neutral hydrostatic pressure in the supporting clay. It is during this early period following con-

[1] Principles of Soil Mechanics Involved in Fill Construction, L. A. Palmer and E. S. Barber. Proceedings Highway Research Board, Annual Meeting 1937.

struction (or possibly during construction) that failure of the supporting soil is most likely to occur. Hence it is entirely on the side of safety to consider only the cohesion in computing the supporting power.

For the case of a supporting layer of cohesive soil underlaid by rock, the author has found no expressions for shearing stresses other than those published by Carothers. Biot [8] has derived quite complicated expressions for the vertical stress p_z for the case of axially symmetric stress distribution and for the case of a line load. For $2h$=infinity his derived expressions reduce to those of Boussinesq and Mitchell. The formulas derived by Carothers do not similarly reduce, but this fact in itself indicates nothing insofar as validity is concerned.

There is no flaw in the analytical derivations of the formulas for supporting power as developed by Hencky and Prandtl and extended by Jürgenson. The limitations are inherent in the assumptions. Obviously the less rigid the fill the more untenable is the assumption of rigidity.

A solution called the "Method of Haines" has been indicated by Hough [9] for the case of a nonrigid structure.

The cases of partially rigid structures are beyond the borderline of present theoretical knowledge existing in published form and there is therefore opportunity for progress beyond this frontier.

Jürgenson [10] has recently suggested that if the fill is nonrigid, the bearing capacity, q, should be taken as $\frac{1}{2}\left(\frac{cb}{h}\right)$ which is half its value when the fill is rigid. This suggested value is only for the case when $2h$ is less than $b/2$.

The method of Haines referred to by Hough requires a more complete presentation and description than has been published to enable the student of theoretical soil mechanics to evaluate properly its utility. The fact that this method follows Jürgenson's boundary case up to $2h=0.3b$ is interesting and adds a degree of confidence in the use of Jürgenson's formula

$$q=\frac{cb}{h}$$

for relatively thin supporting soil strata.

It is the opinion of the author that it is useless to assume a surface of failure in the supporting soil stratum in this problem. The conditions are too variable to warrant this procedure. A surface of failure is not assumed in the method of Hencky as extended and applied by Prandtl and Jürgenson. The slip lines shown in figure 3 are determinable from equations 20, 26, and 27 and are families of cycloids.

In the absence of rock, q, the supporting power is taken with reference to the weight of a column of fill material of height equal to that of the fill and of 1 square foot cross-sectional area. For this case there are obtained by three different analytical methods the following values for q in terms of the unit cohesion c (ϕ being small enough to be neglected):

By the method of Terzaghi, $q=4$ [illegible].
By the method of Prandtl, $q=$ [illegible] $+2$ [illegible].
By the method of Krey, $q=$ [illegible].

These values are all for a factor of safety of [illegible].

For the case of a rigid rock boundary below the supporting clay, the formula of Jürgenson is

$$q=\frac{cb}{h}=p$$

for a factor of safety of one, where p is the weight of a column of fill material of height equal to that of the equivalent triangle. (See fig. 3.) For $2h$ equal to or less than $b/2$, q is equal to or greater than $4c$, according to this formula. For values of $2h$ greater than $b/2$, Jürgenson's formula gives such increasingly small values for q as to be obviously in error.

The question arises as to the best procedure to follow when $2h$ is greater than $b/2$. Pending the time that a more general and satisfactory solution of this problem is obtained, the following procedures are believed to be warranted and their use is suggested.

1. For depths to rock less than one-fourth of the base width of the fill, the supporting power, q, is computed directly from Jürgenson's formula if the fill has a rigidity and strength such that it resists the shearing stress, $s_{zx}=c$, at its base.
2. For depths to rock greater than one-fourth and less than three-fourths of the base width of the fill, the value of q is considered as constant and equal to $4c$ regardless of the rigidity of the fill. In this case also q is considered as equal to p, the weight of a column of fill material of height equal to that of the equivalent triangle (*AED*, fig. 3).
3. When the depth to rock exceeds three-quarters of the base width of the fill, the analytical procedures are the same as those followed when the depth of the supporting clay is infinite. If the fill is rigid, the method of Prandtl [11] is applied. If the fill is nonrigid, the method of Terzaghi yields an appropriate value for q.
4. For an absolutely nonrigid fill and for b greater than $4h$ (fig. 2), the supporting power, q, may be computed from the formula.

$$q=\frac{1}{2}\left(\frac{cb}{h}\right).$$

In this case the ultimate supporting power of the undersoil is taken as the value of p in equation 12 when $s_{max.}$ becomes equal to c at any point x. (See fig. 2.) For depths to rock less than one-quarter of the base width of the fill, this value of p is about one-half that which is computed from the formula,

$$q=\frac{cb}{h}$$

assuming the fill is rigid.

5. All intermediate conditions, when the fill can neither resist a shearing stress, $s_{zx}=c$, nor is it nonrigid, are reserved for future study.
6. It should be possible to increase the ultimate supporting power of the undersoil by increasing the rigidity of the fill either by selection of material, methods of compacting, by special reinforcement such as the use of fascines, or by all of these means.
7. Spreading a thin blanket of gravel or sand over the undersoil and building the fill thereon would tend to hasten the process of consolidation of the soft layer of supporting soil with a consequent increase in its supporting power. The granular material in this case acts as a drainage course, providing a direct outlet for water in the voids that is under pressure transmitted by the fill load.

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[8] Effect of Certain Discontinuities on the Pressure Distribution [illegible] M. A. Biot. Publications from the Graduate School of Engineering [illegible] University, No. [illegible]

[9] Stability of Embankment Foundations, by B. K. Hough, Jr. [illegible] Society of Civil Engineers [illegible]

[10] On the Stability of Foundations and Embankments [illegible] No. G-8, vol. 2, Proceedings International Conference on [illegible] Foundation Engineering, 1936.

[11] Principles of Soil Mechanics Involved in Fill Construction, by L. A. Palmer and E. S. Barber, Proceedings Highway Research Board, Annual Meeting, 1937.

SIGNIFICANT TRENDS IN MOTOR-VEHICLE REGISTRATIONS AND RECEIPTS

BY THE DIVISION OF CONTROL, PUBLIC ROADS ADMINISTRATION

Reported by ROBERT H. PADDOCK, Associate Highway Engineer-Economist

MOTOR-VEHICLE registrations in the United States in 1938 numbered 219,540 fewer than in the preceding year. This amounted to a decline of 0.7 percent from 1937 registrations and marked the fourth time in the history of the automotive industry that the total registrations for one year were less than those for the preceding year.

The history of motor-vehicle registrations in this country has generally been one of continual growth; an increase each year over the preceding one has come to be expected. The course of registrations since 1914 is shown graphically in figure 1. The decreases in 1931, 1932, and 1933 resulted from the economic depression which started in 1929, and the recession of 1937 undoubtedly accounts for most of the registration decrease in 1938 from 1937. It will be interesting in succeeding years to observe the registration trends and to compare motor-vehicle registrations of the next decade with those of the nine-year period ending with 1938.

Passenger-car and bus registrations of 25,261,649 and truck registrations of 4,224,031 made up the reported 1938 total of 29,485,680 vehicles. It should be noted that in spite of marked improvements in registration practice in all States during the past decade, the available data are not entirely comparable among States. Passenger-car registrations in some States include vehicles that elsewhere would be registered as trucks. Busses are registered with passenger cars in some States, and with trucks in other States, and in many cases are not readily separable. However, it is believed that these inconsistencies in registration practice are not great enough in total to affect the general observations and conclusions which can be drawn from the available data.

The percentage of decrease recorded in 1938 for passenger-car registrations was slightly greater than that for trucks. This condition was also characteristic of motor-vehicle registrations in the early part of the decade. In 1930 an increase in truck registrations more than compensated for a decrease in passenger-car registrations, causing a slight net increase in total motor-vehicle registrations for that year over 1929.

PERCENTAGE INCREASE IN TRUCK REGISTRATIONS EXCEEDS THAT FOR PASSENGER CARS

Table 1 shows the respective annual changes and the differences in the annual rates of change during the past 18 years in passenger-car and truck registrations. Since 1921 truck registrations have increased faster or have decreased more slowly as compared with the preceding year's registrations for every year but 2 than have the corresponding passenger-car registrations. These 2 years were 1923 and 1932. In the former year, the greatest single year's percentage increase in passenger-car registrations since 1920 occurred. This was an increase of 23.8 percent while truck registrations recorded an increase of 19.2 percent. This lag in truck registration growth was more than compensated for by the 1924 registrations when passenger cars recorded a substantial increase of 14.7 percent while truck registrations were 32.8 percent higher than those of the preceding year.

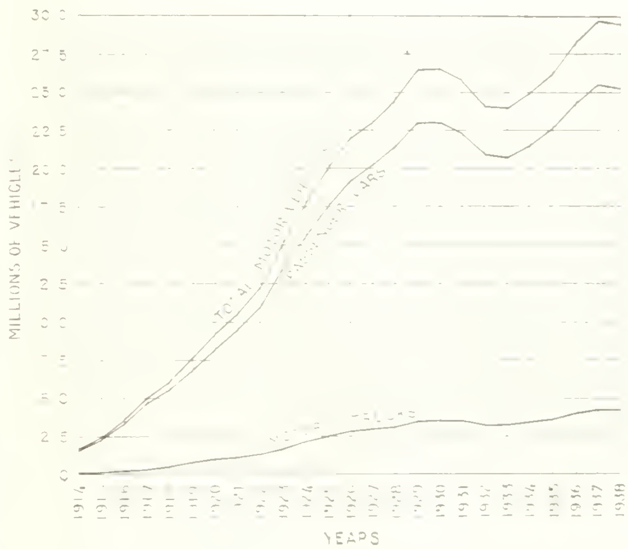

FIGURE 1.—MOTOR-VEHICLE REGISTRATIONS IN THE UNITED STATES, 1914–38.

Again, in 1932, the drop in truck registrations was 6.8 percent compared to 6.5 percent for passenger cars. But in 1931 passenger car registrations had dropped 3.1 percent in contrast to a 0.6 percent drop for trucks and in 1933 passenger-car registrations showed a drop of 1.2 percent compared to a very small increase for trucks.

TABLE 1.—*Comparison of variation in registration of passenger cars and trucks, 1921 to 1938* [1]

Year	Increase or decrease in registration from previous year				Increase in registration over 1921			
	Passenger cars		Trucks		Passenger cars		Trucks	
	Number	Per cent	Number	Per cent	Number	Per cent	Number	Per cent
1922	1,523,170	16.3	251,[illegible]	[illegible]	1,523,170	16.3	251,910	23.0
1923	2,594,22[illegible]	23.8	[illegible]	19.2	4,1[illegible]	44.[illegible]	[illegible] 245	46.5
1924	1,976,282	14.7	[illegible]	32.8	6,[illegible]3,678	65.[illegible]	1,036,704	94.6
1925	2,335,[illegible]	13.2	[illegible]	14.4	8,[illegible]449	86.8	1,344,5[illegible]	122.6
1926	1,74[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1[illegible]5.4	1,667,[illegible]	152.1
1927	982,[illegible]52	[illegible]	[illegible]	[illegible].4	[illegible]	[illegible]	1,[illegible]	165.8
1928	1,159,[illegible]	5.[illegible]	[illegible]	6.[illegible]	12,[illegible]	128.2	2,[illegible]675	184.0
1929	1,742,464	8.2	[illegible]	[illegible]	13,[illegible]	146.8	2,283,530	208.3
1930	−62,327	−.3	[illegible]	[illegible]	[illegible]	146.2	2,382,695	218.[illegible]
1931	−711,239	−3.1	[illegible]	−.6	12,[illegible]152	[illegible]	2,3[illegible]756	216.2
1932	−1,462,2[illegible]	−6.5	−2[illegible]	−6.8	11,[illegible]	[illegible]	2,132,[illegible]	194.6
1933	−242,2[illegible]	−1.2	[illegible]	[illegible]	[illegible]	[illegible]	2,134,344	194.7
1934	888,844	4.3	[illegible]	7.8	12,[illegible]437	12[illegible].9	2,322,130	211.9
1935	1,[illegible]	4.[illegible]	[illegible]	[illegible]	[illegible]449	[illegible]	2,55[illegible]	232.7
1936	[illegible]594,7[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2,[illegible]	263.7
1937	1,271,7[illegible]3	5.[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3,1[illegible]	[illegible]
1938	−188,275	−.7	−[illegible]	−.7	15,[illegible]678	[illegible]	3,127,[illegible]	285.[illegible]

[1] Busses included with passenger cars. [2] Less than 0.1 percent.

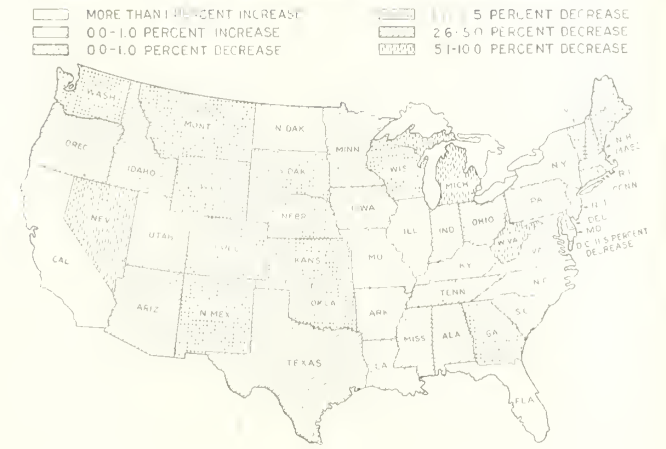

FIGURE 2.—CLASSIFICATION OF STATES ACCORDING TO PERCENTAGE OF CHANGE IN TOTAL MOTOR-VEHICLE REGISTRATION IN 1938 OVER 1937.

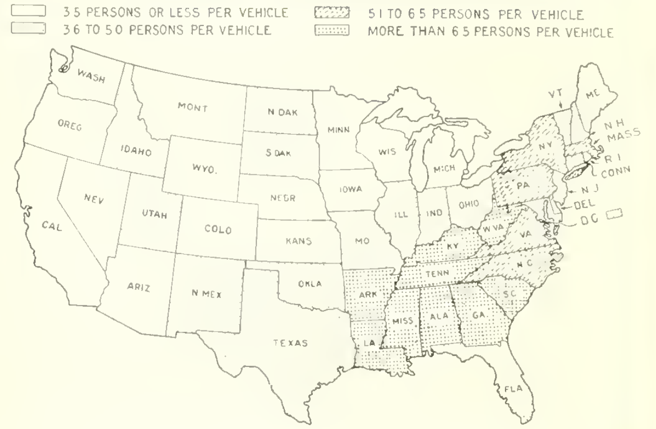

FIGURE 3.—CLASSIFICATION OF STATES ACCORDING TO NUMBER OF PERSONS PER REGISTERED VEHICLE IN 1938.

The percentage of increase for trucks from 1922 to 1938 was almost 1.7 times as great as the corresponding increase for passenger cars. Whereas trucks comprised approximately 10.5 percent of the total motor-vehicle registration in 1921, in 1938 they were 14.3 percent of the total registration. Important features of future motor-vehicle regulation will be dependent upon the changes that may occur in those relationships. It can be seen from table 1 that the rates of change in truck registrations have been different from those for passenger-car registrations except in 1938. Though an approximately stable relation in the national economy between cars and trucks may now have been reached, it is probable that apparent changes in these relationships will be observed in the future without the occurrence of any real changes. Such apparent though not real changes may occur if more nearly correct classification and registration practices are adopted by those States where passenger-car registrations, for example, now contain a considerable number of vehicles that should properly be designated as trucks.

The Administration's statistical tables, State Motor-Vehicle Registrations and Receipts, 1938, appearing in the June 1939, issue of PUBLIC ROADS showed that 33 States [1] reported decreases in total 1938 registrations from their respective 1937 registrations. The greatest numerical decrease was in Michigan with a reported decrease of 96,276 vehicles, which accounted for 29 percent of the change in the 33 States reporting such losses. The Michigan condition was exaggerated by reflection of the conditions in the automobile market in the rest of the country.

The large decrease in the District of Columbia registration, where the largest percentage decrease was recorded, is believed to have been occasioned largely by the revision in registration fees in 1938 when the previous $1 fee was abandoned for higher rates. This change undoubtedly caused the retirement of some vehicles that might have been registered at the lower rate. The change also probably resulted in the proper registration of vehicles from other States in their own States where formerly they had escaped the higher rates in their own States by registering in the District of Columbia or had been registered both in their own States and in the District of Columbia.

Large decreases were also reported in Indiana, West Virginia, and Wisconsin. Other States showing decreases of more than 10,000 vehicles were Alabama, Kansas, Mississippi, Oklahoma, and Washington. Only four States—California, Illinois, New York and North Carolina—reported increases of more than 10,000 in their registrations.

[1] The District of Columbia is classed as a State in this report.

The percentage changes by States in total vehicle registrations are shown in figure 2. It is significant that there is no uniform pattern among the States except in the Rocky Mountain area. States showing increases are scattered throughout the country.

SUBSTANTIAL DECLINE NOTED IN PERSONS PER REGISTERED VEHICLE

The characteristics noted for all motor vehicles were generally true for passenger cars and trucks separately, though only 28 States showed decreases in truck registrations. Arizona, Iowa, Kansas, Massachusetts, Montana, Nebraska, New Hampshire, Ohio, Tennessee, Texas, West Virginia, and Wyoming all reported increases in truck registrations though the total number of vehicles registered in each of those States decreased. However, in Florida, Louisiana, Missouri, New Jersey, New York, Utah, and Virginia where there were net increases in total motor vehicles registered there were actual decreases in the number of trucks registered.

These differences among the States suggest that with the exception of Michigan and the District of Columbia, which apparently reflect certain peculiar conditions, the causes of the changes in registration in other States must be sought in a variety of governmental, economic, and social factors. For example, the decreases in total registrations in some States, accompanied by increases in truck registrations, may actually be caused by changes in local registration practices rather than by changes in the classes of vehicles in operation. Again, decreases in car registrations as contrasted to increases in truck registrations in such States as Kansas, Nebraska, and Texas may be caused by farmers who, for reasons of economy, refrain from registering automobiles still owned, and use their trucks for both business and pleasure driving.

Since it is impossible to draw sound general conclusions from the data for a single year or even for a few years, it is desirable to identify certain basic State and national trends in motor-vehicle ownership. One approach to this is a determination of the distribution, by States, of motor vehicles among the entire population. These data are presented in figure 3 which shows graphically the number of persons per registered motor

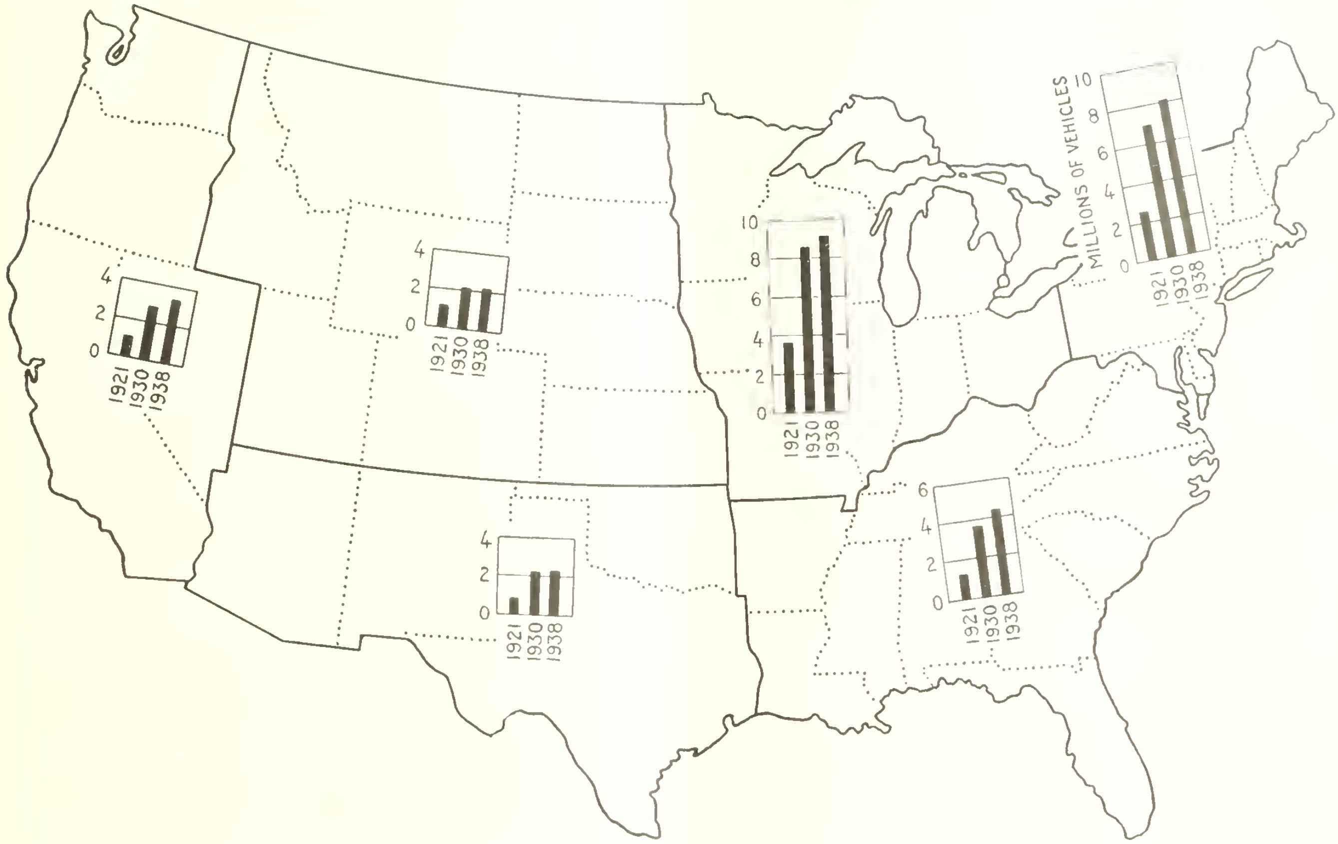

FIGURE 4.—TOTAL MOTOR-VEHICLE REGISTRATIONS, IN MILLIONS OF VEHICLES, BY REGIONS IN 1921, 1930, AND 1938.

vehicle in the several States in 1938. This figure indicates certain quite definite patterns of motor-vehicle ownership throughout the country with relatively the fewest vehicles in the Southeast and the most in the Far West.

In order to study these characteristics in greater detail and to determine what regional characteristics there may be the country was divided into six areas. These differ somewhat from the geographical areas used by the United States Bureau of the Census since adherence to those areas would not bring out clearly the significant differences throughout the country. The areas are similar to those selected by the National Resources Committee in their report Problems of a Changing Population. One change from the grouping used in that study has been made—West Virginia has been grouped with the Southeastern States instead of with those of the Northeast.

The States included in the several areas are shown in figure 4 which also gives the number of motor-vehicle registrations in the several areas in 1921, 1930, and 1938. This graph indicates the greater proportional registration growth in the Southeastern States between 1921 and 1938, and particularly between 1921 and 1930, in comparison with the increases in other areas. Table 2 shows this growth strikingly also by expressing the data as persons per registered vehicle at the beginning, middle, and end of the period studied. Thus, while the change in the Southeast constituted a 63-percent decrease from 1921 to 1930 in the number of persons per vehicle, the corresponding decrease in the Northwest was only 48 percent, and in the Far West 50 percent.

The year-by-year change in persons per vehicle in the several regions is shown in figure 5 which illustrates the rapid drop for all areas until 1929, followed by the rise during the depression years and the subsequent drop again for all regions since 1933. The computations for this figure are based on the annual midyear population estimates, by States, made by the United States Bureau of the Census. Computations for 1938 are based on the latest available population estimates—those for 1937.

TABLE 2.—*Persons per registered motor vehicle, by regions*

Region	Persons per motor vehicle in—		
	1921	1930	1938
Northeast	12.1	5.2	4.8
Southeast	20.1	7.4	6.9
Southwest	10.2	4.3	4.1
Middle States	8.1	3.9	3.8
Northwest	6.6	3.4	3.4
Far West	6.0	3.0	2.6
United States	10.4	4.6	4.4

SOUTHEAST REGION HAS GREATEST NUMBER OF PERSONS PER VEHICLE

It is evident that though since 1921 there has been a relatively greater increase in the number of vehicles in relation to the population in the Southeast than in any other region, it still is considerably higher than the country as a whole in persons per vehicle. Judged by this criterion alone, the Southeast may be thought of as the region where potentially the greatest percentage increase in vehicles may occur in the future.

It is significant that all of the 11 States having over 6 persons per vehicle were in the Southeast region. In Florida, the only other State in this region, the number of persons per vehicle in 1938 was lower than

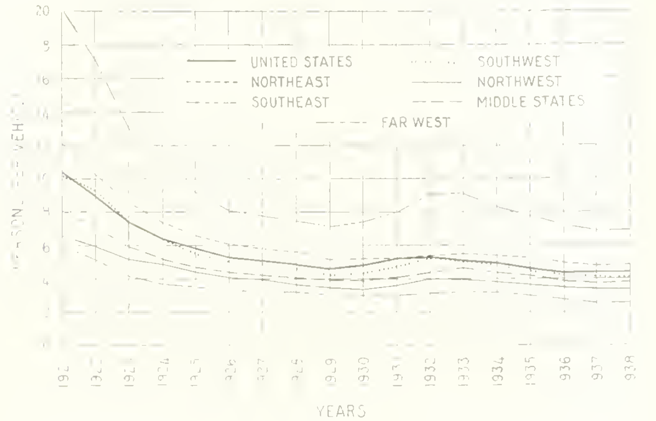

FIGURE 5.—NUMBER OF PERSONS PER REGISTERED MOTOR-VEHICLE BY REGIONS, 1921–38.

the average for the country. The lowest States in this region were Florida with 3.9, Virginia with 6.1, and Louisiana, North Carolina, and South Carolina each with 6.5 persons per registered motor vehicle. The nearest approaches to these figures in any other States were Massachusetts with 5.2, Pennsylvania with 5.1, New York with 5.0, and Missouri and Oklahoma each with 4.8 persons per registered motor vehicle. These conditions for Oklahoma and Missouri may be explained on the basis of the economic similarity of large areas and of large sections of the population in those States to adjacent Southern States. The high degree of urbanization of Massachusetts, New York, and Pennsylvania with an accompanying decrease in the economic utility of a car for large portions of the population and the presence of large economically depressed coal-mining regions in Pennsylvania provide at least partial explanations of the figures for those States.

Comparison of the State motor-vehicle-registration data for the years 1929, 1930, and 1931 reveals that the peaks of registration during that period were reached at different times in different States. With the exceptions of Montana, North Dakota, and Oklahoma, no western State reached its peak in 1929. On the other hand, of the 10 States which had their greatest registration for the period in 1931, 4 were in the West.

In a study of trends in motor-vehicle registration, however, it is more significant that in 11 States registrations in 1938 were less than in the peak year of the 1929–31 period and that of these, only Massachusetts and the District of Columbia have had in at least 1 year since 1931 a total registration which exceeded the peak year of the 1929–31 period. Table 3 shows the States where such conditions existed for passenger cars, for trucks, and for all motor vehicles. Though the increases in car ownership since 1934 have been considerable it is significant that in almost one-fifth of the States, representing 10.6 percent of the registrations in 1938, motor-vehicle registrations had not yet regained the peak reached during the 1929–31 period.

Whether recovery in registrations is only delayed in those nine States, or whether the 1929–31 peak will remain an all-time high or will remain unequaled for several years in at least some of those States is dependent on many national economic and demographic factors. Six of the nine States recorded their greatest registrations since the 1929–31 period in 1937, but the post-depression high was reached in Nebraska and South Dakota in 1936 while the registration in North Dakota was greater in 1938 than in 1937.

TABLE 3.—*States in which registrations since 1929–31 have not reached those of the peak year of that period*

Passenger cars	Trucks	All motor vehicles
Arkansas.	Delaware.	Arkansas.
Iowa.	Michigan.	Iowa.
Kansas.	New Jersey.	Kansas.
Massachusetts.	New York.	Mississippi.
Mississippi.	Ohio.	Nebraska.
Nebraska.	Rhode Island.	North Dakota.
North Dakota.		Oklahoma.
Oklahoma.		South Dakota.
South Dakota.		Vermont.
Vermont.		

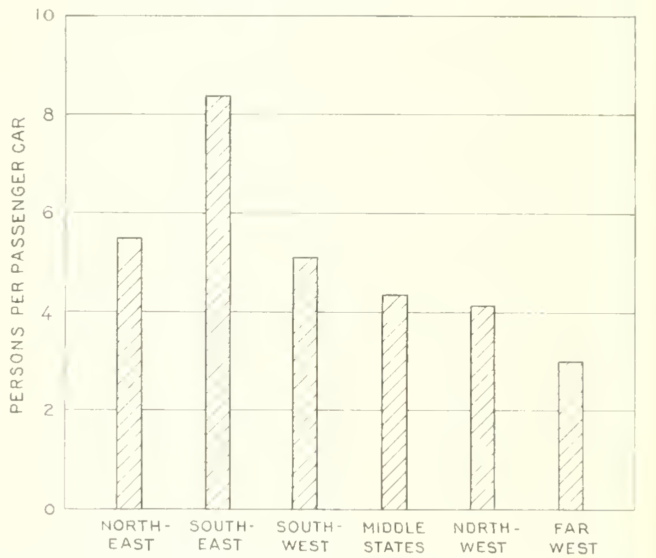

FIGURE 6.—NUMBER OF PERSONS PER REGISTERED PASSENGER CAR BY REGIONS, IN 1938.

SOUTHWEST REGION HAS SMALLEST RATIO OF PASSENGER CARS TO TRUCKS

Some further indication of regional characteristics may be brought out by a comparison of the ratio of passenger-car to truck registrations in the several regions. Table 4 shows the results of that analysis by regions for 1921, 1930, and 1938. The comparison in table 2 of persons per registered motor vehicle only does not present a complete picture of vehicle ownership characteristics by regions. One reason for this is that the relative ownership and use of trucks varies considerably in different parts of the country, particularly among the agricultural population. In some areas trucks serve both for the usual hauling purposes and also for transportation of persons. In other areas, the use of trucks is restricted more to the hauling function. Figure 6 shows for 1938 the persons per registered passenger car in the several regions. This chart indicates a general similarity between passenger-car and total motor-ve-

TABLE 4.—*Ratio of passenger cars to trucks by regions*

Region	Registration years		
	1921	1930	1938
Northeast	6.5	6.2	6.6
Southeast	7.8	6.4	4.8
Southwest	12.5	6.2	4.0
Middle States	8.7	7.0	7.1
Northwest	12.1	6.3	4.7
Far West	11.8	7.7	6.6
United States	8.5	6.6	6.0

hicle registrations by regions, with the Southeast showing the highest number of persons per passenger car and the Far West the lowest number.

Table 4 shows, however, that there is a considerable difference between the ratio of passenger cars to trucks in the Middle States and in the Southwest. The observed characteristic of the Middle States is probably due in large part to the relatively high ownership of passenger cars in connection with the automotive industry in Michigan and adjacent States. In contrast, the low ratio in the Southwest probably indicates the more general use of trucks for purposes for which passenger cars are used in other areas. Conditions in the Southeast and Northwest are also apparently somewhat similar in this respect to those in the Southwest.

It is particularly surprising to note the condition in the Northeast. It is the only region where the ratio of passenger cars to trucks was higher in 1938 than in both 1930 and 1921. No explanation of this condition is immediately apparent though registration practices may have had considerable effect.

In addition to the 29,485,680 privately owned passenger cars and trucks registered in 1938, there were also in operation 109,761 Federal motor vehicles and 257,469 State, county, and municipal motor vehicles. These figures, shown in table 5, represent a 4.7 percent increase in Federal vehicles and an 11.3 percent increase in other publicly owned vehicles in 1938 over 1937. This tabulation also illustrates strikingly the inadequacies of present registration practice in the several States. In some instances publicly owned vehicles are included with those privately owned; in others no record is kept of such vehicles at all; and in still others there is no segregation between Federal vehicles and those owned by the States, counties and municipalities.

TABLE 5.—*Publicly owned vehicles in the United States in 1938* [1]

State	Federal [2]								State, county, and municipal [3]								
	Motor vehicles					Trailers and semi-trailers	Motor-cycles	Total vehicles	Motor vehicles						Trailers and semi-trailers	Motor-cycles	Total vehicles
	Passenger motor vehicles			Motor trucks, tractor trucks, etc.	Total motor vehicles				Passenger motor vehicles			Motor trucks, tractor trucks, etc.	Type not reported	Total motor vehicles			
	Automobiles	Motor busses	Total						Automobiles	Motor busses	Total						
Alabama	440	13	453	1,539	1,992	55	4	2,051					3,755	3,755		143	3,898
Arizona	496	73	569	1,805	2,374	97	4	2,475	527	207	734	1,320		2,054	181	23	2,258
Arkansas	270	8	278	1,789	2,067	25	1	2,093	1,264	1,042	2,306	887		3,193	13	31	3,237
California	1,121	71	1,192	6,347	7,539	276	75	7,890					24,502	24,502	1,646	1,157	27,305
Colorado	367	23	390	1,957	2,347	23	7	2,377									(4)
Connecticut	64	1	65	590	655	6	1	662	1,450		1,450	2,441		3,891	87	294	4,272
Delaware	15		15	297	312	4		316					896	896	25	62	983
Florida	349	10	359	1,428	1,787	25	15	1,827	1,130	707	1,837	3,235		5,072	317	168	5,557
Georgia	582	39	621	2,044	2,665	45	33	2,743	1,033		1,033	2,986		4,019	63	138	4,220
Idaho	134	3	137	1,404	1,541	80		1,621	341	144	485	1,063		1,548	89	11	1,648
Illinois	507	9	516	2,801	3,317	98	19	3,434	2,573		2,573	6,919		9,492	323	654	10,469
Indiana	193	1	194	1,437	1,631	86	15	1,732	1,784		1,784	4,385		6,169		195	[5] 6,364
Iowa	187	1	188	1,224	1,412	22	6	1,440	1,309		1,309	4,745		6,054	357	66	[5] 6,477
Kansas	230	2	232	1,272	1,504	61	9	1,574									(4)
Kentucky	277	3	280	1,291	1,571	11	109	1,691	1,011		1,011	3,314		4,325			[5] 4,325
Louisiana	413	16	429	1,398	1,827	27	13	1,867	1,971	32	2,003	2,722		4,725	330	63	[5] 5,168
Maine	119	1	120	432	552	10	5	567	630	87	717	1,370		2,087	164	31	2,282
Maryland	379	21	400	1,966	2,366	61	21	2,448									(4)
Massachusetts	428	20	448	2,269	2,717	31	11	2,759					5,700	5,700			5,700
Michigan	304	5	309	2,233	2,542	81	17	2,640									(4)
Minnesota	395	3	398	2,111	2,509	56	10	2,575					4,790	4,790			4,790
Mississippi	174	18	192	1,283	1,475	57	1	1,533									(4)
Missouri	317	15	332	1,753	2,085	27	6	2,118	571		571	1,606		2,177		9	2,186
Montana	389	3	392	1,698	2,090	27		2,117					2,201	2,201			[5] 2,201
Nebraska	243	3	246	953	1,199	14	8	1,221	539	52	591	1,871		2,462		46	2,508
Nevada	141	5	146	550	696	21	2	719	147	25	172	467		639	48	8	695
New Hampshire	21		21	634	655	19	1	675									(4)
New Jersey	266	5	271	2,492	2,763	26	14	2,803	4,180		4,180	6,297		10,477		545	11,022
New Mexico	457	12	469	1,709	2,178	67		2,245	571		571	340		911		41	952
New York	919	36	955	4,674	5,629	49	81	5,759	6,462	1,577	8,039	18,044		26,083	881	1,036	[5] 28,000
North Carolina	380	13	393	1,788	2,181	42	3	2,226		4,850	4,850		6,821	11,671			11,671
North Dakota	154	19	173	615	788	12		800					699	699			699
Ohio	390	10	400	2,076	2,476	85	6	2,567	3,234	7,125	10,359	8,409		18,768	1,125	576	20,469
Oklahoma	507	20	527	1,908	2,435	42	14	2,491					6,719	6,719			6,719
Oregon	337	8	345	2,476	2,821	33	4	2,858	1,672		1,672	2,399		4,071			4,071
Pennsylvania	482	5	487	3,496	3,983	123	24	4,130	5,601	138	5,739	12,061		17,800	463	1,226	19,489
Rhode Island	43	12	55	405	460	16	1	477	492		492	917		1,409	23	105	1,537
South Carolina	245	17	262	1,308	1,570	14	5	1,589					4,468	4,468		146	4,614
South Dakota	208	13	221	1,020	1,241	17	2	1,260	227	122	349	783		1,132	152	12	1,296
Tennessee	369		369	1,799	2,168	45	2	2,215					7,105	7,105			7,105
Texas	1,193	33	1,226	4,142	5,368	148	75	5,591	2,350	3,989	6,339	9,566		15,905	1,193	378	17,476
Utah	265	3	268	1,490	1,758	56	10	1,824	332	228	560	818		1,378	64	46	1,488
Vermont	126	2	128	520	648	14		662									(4)
Virginia	408	42	450	2,781	3,231	217	52	3,500	2,685		2,685	2,785		5,470	165	183	[5] 5,818
Washington	603	5	608	2,607	3,215	77	27	3,319	1,644	1,621	3,265	3,685		6,950	336	164	7,450
West Virginia	148	2	150	1,038	1,188	6		1,194	1,760		1,760	3,349		5,109	148	70	5,327
Wisconsin	267	3	270	2,030	2,300	39	5	2,344	1,232	344	1,576	6,970		8,546	204	360	9,110
Wyoming	205	4	209	1,091	1,300	26	9	1,335	297		297	384		681	57		738
District of Columbia	354	7	361	901	1,262	20	62	1,344	[6] 1,265		1,265	1,101		2,366	106	94	2,566
At large [7]	1,090	9	1,099	4,272	5,371	45	10	5,426									
Total	17,971	647	18,618	91,143	109,761	2,564	799	113,124	50,284	22,290	72,574	117,239	67,653	257,469	8,610	8,081	274,160

[1] Because the 2 parts of this table were obtained from different sources, and the State, county, and municipal figures contain some duplication of Federal vehicles, totals of all publicly owned vehicles are not given. Data given in this table are included in condensed form in table State Motor-Vehicle Registrations, 1938.

[2] This information was obtained by the Procurement Division, Department of the Treasury, by means of a circular letter addressed to all departments and independent offices.

[3] This information, compiled from reports of State authorities, is incomplete in many cases. Some States give State-owned vehicles only; others exclude from registration certain classes, such as fire apparatus and police vehicles.

[4] Not reported. Included with private and commercial registrations in table State Motor-Vehicle Registrations, 1938.

[5] Includes unknown number of Federal vehicles.

[6] Includes 405 automobiles of the diplomatic corps.

[7] Includes 2,314 War Department vehicles operated in military reservations, arsenals, etc., but not distributed to State of domicile.

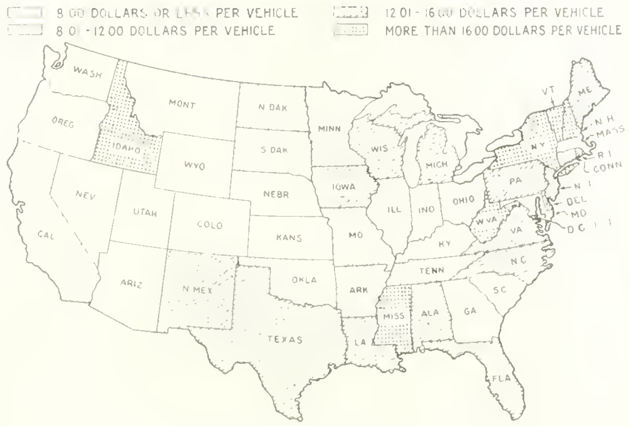

FIGURE 7.—CLASSIFICATION OF STATES ACCORDING TO AVERAGE MOTOR-VEHICLE REGISTRATION FEES IN 1938.

Consequently, the data of table 5 serve only as an indication of the extent of public vehicle ownership and should not be considered a definitive tabulation of publicly owned vehicles in the United States in 1938.

STATES RANKED ACCORDING TO REGISTRATIONS AND FEES PAID

The Administration's statistical table State Motor-vehicle Receipts, 1938, published in the June 1939, issue of PUBLIC ROADS, revealed a slight decrease in total collections from those reported for 1937. Receipts of registration fees rose from $328,285,000 in 1937 to $330,866,000 in 1938, an increase of 0.8 percent; but reductions in other receipts, including those from operators' and chauffeurs' permits, certificates of title, and transfer or reregistration fees, caused the total receipts to fall from $399,613,000 in 1937 to $388,825,000 in 1938, a decrease of 2.7 percent.

While it has been observed that there are rather general regional patterns of motor-vehicle ownership in the several States, such patterns are not so marked in the case of motor-vehicle receipts. Figure 7 shows the grouping of States by various average registration fees paid and indicates that a general pattern comparable with that of figure 3 is not apparent. In general, the lowest average fees are charged in the Western States but the Eastern and Southern States of Georgia, Kentucky, Massachusetts, and South Carolina are in the lowest group and Georgia collects the lowest average fee of any State. These data are presented in more detail in table 6 for passenger cars and trucks as well as for all motor vehicles. It will be seen that average passenger-car fees range from $2.74 in Georgia to $18.12 in Vermont, that average truck fees range from $6.56 in Georgia to $63.48 in Vermont and that average fees for all motor vehicles range from $3.39 to $22.81 in the same States.

The figures for Vermont are not truly representative because the lighter trucks are included in the passenger-car registrations, thus raising the average of those fees in comparison with other States. This illustrates another of the weaknesses of existing registration data when comparisons such as these are desired.

Table 7 shows the ranking of the States in 1938 in registrations, in gross receipts from motor-vehicle license fees, in average motor-vehicle receipts per vehicle, revenue from the motor-fuel tax, average motor-fuel tax receipts per vehicle, and average motor-vehicle and motor-fuel tax receipts per vehicle. It will be observed that there is apparently little correlation between the ranking of the States according to number of vehicles registered and according to motor-vehicle registration receipts. This is to be expected, of course, because of the wide disparity in registration fees charged in the several States.

TABLE 6.—*Average registration fees per vehicle in 1938*

State	Passenger vehicles [1]	Motor trucks	Average for all motor vehicles
Alabama			$12.78
Arizona	$3.76	$17.87	6.28
Arkansas	9.43	17.24	11.32
California	7.68	13.61	8.38
Colorado	5.65	8.70	6.16
Connecticut	7.63	20.20	9.65
Delaware	11.33	27.28	13.95
Florida	11.46	25.28	13.75
Georgia	2.74	6.56	3.39
Idaho	14.93	22.43	16.46
Illinois	8.95	27.37	11.19
Indiana	7.43	13.27	8.23
Iowa	12.94	30.99	15.12
Kansas	5.34	9.44	6.03
Kentucky	4.98	19.25	7.17
Louisiana	11.80	16.51	12.92
Maine	12.32	19.38	13.85
Maryland	8.72	11.20	9.06
Massachusetts	3.71	13.22	4.88
Michigan	9.93	37.37	12.64
Minnesota	9.77	16.75	10.75
Mississippi			18.34
Missouri	9.86	10.00	9.88
Montana	6.54	7.70	6.82
Nebraska	3.63	13.73	5.24
Nevada	5.02	11.96	6.38
New Hampshire			18.19
New Jersey	11.28	30.33	13.80
New Mexico	10.66	20.41	12.91
New York	14.02	36.17	16.80
North Carolina	8.92	36.24	12.79
North Dakota	7.60	10.01	8.06
Ohio	8.66	42.24	11.96
Oklahoma	5.99	18.66	8.22
Oregon	5.12	17.97	7.27
Pennsylvania	11.06	34.88	14.02
Rhode Island	11.57	25.19	13.13
South Carolina	2.97	14.13	4.57
South Dakota	8.93	7.86	8.76
Tennessee			9.80
Texas	9.76	21.14	12.09
Utah	4.76	20.58	7.25
Vermont	18.12	63.48	22.81
Virginia	10.94	20.08	12.34
Washington	3.16	13.20	4.75
West Virginia	15.08	23.22	16.41
Wisconsin	13.32	22.00	14.73
Wyoming	5.70	10.92	6.83
District of Columbia			8.18
Average for United States	[2] 9.25	[2] 22.66	11.22

[1] Includes automobiles and busses. In some States busses are registered with motor trucks. In Alabama, Mississippi, New Hampshire, Tennessee, and the District of Columbia, no classification of registration fees by types was available.
[2] Excluding those States for which no segregation of fees was available.

It will be noted that the average receipts from motor-fuel taxes vary much less than do receipts from motor-vehicle registration fees. The maximum is the $54.92 average for Florida where the State tax is 7 cents per gallon and a large amount of gasoline is used by non-residents. The latter fact, particularly, causes certain of the State figures—based on registrations—to be inflated when compared with data for other States. The lowest collections per vehicle were in Missouri, North Dakota, and the District of Columbia. The first and last of these can be explained by the 2-cent gas tax in effect, while in North Dakota the refund procedure followed acts to reduce the average tax collected per vehicle. California, Iowa, Kansas, and Michigan, all with motor-fuel tax rates of 3 cents per gallon, also received less than $20 in motor-fuel taxes per vehicle. The remaining five States with 3-cent tax rates all collected less than $24 per vehicle in motor-fuel taxes and of these, only two—Massachusetts and New Jersey—collected more than $21 per vehicle from such taxes.

TABLE 7.—*Total motor vehicles registered, State registration fees, motor-fuel taxes paid, and averages per vehicle, in 1938* [1]

State	Number of registered private and commercial passenger cars, busses, and trucks	Rank of State	Total receipts from State motor-vehicle registration and other fees	Rank of State	Average State motor-vehicle receipts per vehicle	Rank of State	Revenue from State motor-fuel tax	Rank of State	Average State motor-fuel tax receipts per vehicle	Rank of State	Average State motor-vehicle and motor-fuel tax receipts per vehicle	Rank of State
			1,000 dollars				*1,000 dollars*					
Alabama	301, 990	30	4, 314	24	$14. 28	19	13, 579	21	$14. 97	8	$59. 25	4
Arizona	128, 791	41	1, 076	46	8. 35	38	4, 243	37	32. 94	13	41. 29	22
Arkansas	220, 391	33	2, 908	31	13. 19	23	10. 092	29	45. 79	5	58. 98	5
California	2, 510, 867	2	23, 930	4	9. 53	34	47. 117	3	18. 77	44	28. 30	45
Colorado	332, 774	28	2, 544	34	7. 64	41	7, 465	34	22. 43	38	30. 07	44
Connecticut	440, 335	20	6, 611	16	15. 01	15	9, 242	33	20. 99	40	36. 00	32
Delaware	64, 078	47	1, 216	44	18. 98	5	2, 073	47	32. 35	14	51. 33	11
Florida	423, 021	22	6, 432	17	15. 20	14	23. 232	9	54. 92	1	70. 12	1
Georgia	432, 360	21	1, 974	39	4. 56	48	19, 633	13	45. 41	6	49. 97	12
Idaho	137, 851	40	2, 380	36	17. 27	10	4. 085	40	29. 63	18	46. 90	15
Illinois	1, 780, 865	5	21, 591	5	12. 12	26	36, 888	6	20. 71	41	32. 83	42
Indiana	922, 788	9	9, 635	11	10. 44	33	22, 770	10	24. 68	32	35. 12	36
Iowa	740, 021	14	11, 797	10	15. 94	12	13, 234	22	17. 88	45	33. 82	39
Kansas	573, 985	15	3, 823	27	6. 66	44	10, 168	28	17. 71	46	24. 37	47
Kentucky	414, 207	23	4, 599	23	11. 10	29	12, 531	23	30. 25	17	41. 35	21
Louisiana	326, 199	29	4, 892	22	15. 00	16	16. 627	17	50. 97	2	65. 97	2
Maine	196, 690	35	3, 582	28	18. 21	8	5, 558	35	28. 26	21	46. 47	16
Maryland	395, 347	26	5, 069	21	12. 82	25	9, 929	30	25. 11	31	37. 93	28
Massachusetts	843, 789	10	6, 759	15	8. 01	40	20, 194	12	23. 93	34	31. 94	43
Michigan	1, 408, 835	7	20, 856	6	14. 80	17	27, 683	7	19. 65	43	34. 45	38
Minnesota	821, 241	13	9, 377	13	11. 42	27	19, 570	14	23. 83	35	35. 25	35
Mississippi	215, 195	34	4, 001	26	18. 59	6	10, 181	27	47. 31	4	65. 90	3
Missouri	837, 118	12	9, 439	12	11. 28	28	11, 636	24	13. 90	47	25. 18	46
Montana	171, 326	38	1. 546	42	9. 02	35	4, 452	36	25. 99	28	35. 01	37
Nebraska	407, 330	24	2, 442	35	5. 99	46	11, 139	26	27. 35	25	33. 34	41
Nevada	38, 424	48	265	48	6. 90	43	1, 202	48	31. 28	15	38. 18	26
New Hampshire	124, 379	43	2, 711	33	21. 80	2	3, 298	43	26. 51	26	48. 31	14
New Jersey	1, 000, 684	8	20. 204	8	20. 19	3	22, 362	11	22. 35	39	42. 54	20
New Mexico	116, 537	44	1, 643	40	14. 10	20	4, 090	39	35. 09	11	49. 19	13
New York	2, 584, 123	1	47, 124	1	18. 23	7	66, 195	1	25. 62	30	43. 85	18
North Carolina	537, 242	16	7, 211	14	13. 42	22	24, 308	8	45. 25	7	58. 67	7
North Dakota	174, 256	37	1. 523	43	8. 74	36	2, 318	46	13. 30	48	22. 04	48
Ohio	1, 870. 249	4	27, 204	3	14. 54	18	45, 982	4	24. 59	33	39. 13	24
Oklahoma	535, 399	17	5, 779	19	10. 79	31	13, 910	20	25. 98	29	36. 77	30
Oregon	357, 321	27	2, 922	30	8. 18	39	9, 838	31	27. 53	23	35. 71	34
Pennsylvania	1, 976, 466	3	34, 513	2	17. 46	9	52, 001	2	26. 31	27	43. 77	19
Rhode Island	168, 888	39	2, 778	32	16. 45	11	3. 495	41	20. 69	42	37. 14	29
South Carolina	287, 913	31	1, 633	41	5. 67	47	11, 462	25	39. 81	9	45. 48	17
South Dakota	180, 632	36	1, 983	38	10. 98	30	4, 102	38	22. 71	37	33. 69	40
Tennessee	398, 624	25	4, 173	25	10. 47	32	19, 231	16	48. 24	3	58. 71	6
Texas	1, 548, 343	6	20, 263	7	13. 09	24	42, 747	5	27. 61	22	40. 70	23
Utah	127, 004	42	1, 097	45	8. 64	37	3, 478	42	27. 38	24	36. 02	31
Vermont	87, 402	45	2, 365	37	27. 06	1	2, 530	44	28. 95	20	56. 01	8
Virginia	441, 462	19	6, 134	18	13. 89	21	16, 621	18	37. 65	10	51. 54	10
Washington	523, 328	18	3, 262	29	6. 23	45	15, 431	19	29. 49	19	35. 72	33
West Virginia	275, 691	32	5, 498	20	19. 94	4	9, 397	32	34. 09	12	54. 03	9
Wisconsin	840, 291	11	13, 001	9	15. 47	13	19, 447	15	23. 14	36	38. 61	25
Wyoming	80, 765	46	601	47	7. 44	42	2, 478	45	30. 68	16	38. 12	27
District of Columbia	162, 863		2, 145		13. 17		2, 520		15. 47		28. 64	
1938 totals	29, 485, 680		388, 825		13. 19		771, 764		26. 17		39. 36	
1937 totals	29, 705, 220		399, 613		13. 45		761, 998		25. 65		39. 10	
Increase or decrease	−219,540 (−0.7 percent)		−10,788 (−2.7 percent on total)				9,766 (1.3 percent on total)				−0.1 percent on both totals	

[1] This tabulation is based on tables, State Motor-Fuel Tax Receipts, State Motor-Vehicle Registrations, and State Motor-Vehicle Receipts, 1938.

The figures in table 8 indicate that although motor-vehicle receipts in 1938 were well above those collected in 1930, the peak year of the 1929–31 period, receipts dropped much more rapidly after 1930 and again in 1938, than did passenger-car or truck registrations. In contrast, the percentage of increase in receipts in 1937 was much greater than the percentage of increase in registration of passenger cars or trucks.

WESTERN STATES HAVE LOWEST REGISTRATION FEES

It has been noted that motor-vehicle registrations in 1938 in 11 States were less than during the peak year of the 1929–31 period. In the case of motor-vehicle receipts this condition is even more pronounced, for 25 States in 1938 collected less from motor-vehicle imposts than they did in the peak year of the 1929–31 period. This included 3 of the 4 States in the Far West, Oklahoma and Texas in the Southwest, all but Colorado, Idaho, and Utah in the Northwest, 6 of the 12 States in the Southeast, only Connecticut, Massachusetts, and Vermont in the Northeast, and all but Illinois, Indiana and Ohio in the Middle States.

Many of these decreases are due to changes in basic registration rates since 1929 and a shift from registration fees to increased motor-fuel taxation as a source of funds for the support of highways. While the trend is not so pronounced today, there is some indication that for the present the general movement for lower registration fees is over, even though legislatures in several States during recent sessions considered various bills embodying downward revisions of registration fees for passenger cars. Since average registration fees in the different regions vary by almost 100 percent, it is reasonable to expect continued agitation for revision in the fees charged.

Table 9 shows that the average registration fees range from $7.11 in the Northwest States to $13.46 in the Northeast States. This regional comparison bears out the indications of figure 7 that the lowest average fees generally were collected in the Western States.

TABLE 8.—*Comparison of changes in registrations and motor-vehicle receipts, 1921–38*

Year	Increase or decrease in motor-vehicle receipts from previous year		Increase or decrease in registration of—	
	Amount	Percent	Passenger cars [1]	Trucks
	1,000 dollars		*Percent*	*Percent*
1922	29,569	24.1	16.3	23.0
1923	36,923	24.3	23.8	19.2
1924	36,521	19.3	14.7	32.8
1925	35,127	15.6	13.2	14.4
1926	27,663	10.6	9.9	13.2
1927	12,779	4.4	5.1	5.4
1928	21,569	7.2	5.7	6.9
1929	25,214	7.8	8.2	8.5
1930	7,861	2.3	−.3	3.1
1931	−11,367	−3.2	−3.1	−.6
1932	−20,064	−5.8	−6.5	−6.8
1933	−22,959	−7.1	−1.2	(2)
1934	5,945	2.0	4.3	5.8
1935	15,714	5.1	4.9	6.7
1936	36,809	11.4	7.1	9.3
1937	39,830	11.1	5.3	6.7
1938	−10,788	−2.7	−.7	−.7

[1] Includes busses.
[2] Less than 0.1 percent.

TABLE 9.—*Average motor-vehicle registration fees by regions, 1938*

Region	Average registration fee
Northeast	$13.46
Southeast	10.96
Southwest	10.92
Middle States	11.75
Northwest	7.11
Far West	7.69
United States	11.22

Table 10 gives the average registration fees and other motor-vehicle imposts collected in the several regions in 1921, 1930, and 1938. Differences in classification make it difficult to compare these regions satisfactorily for different years on any other basis than that of total motor-vehicle imposts collected. In many States, records were so maintained in 1921 that segregation of fees by types of vehicles as well as by miscellaneous types of fees could not be obtained. Unfortunately, for desirable comparisons which might be made, this is still true for many States.

The comparison in table 10 of average motor-vehicle imposts by regions in 1921, 1930, and 1938, indicates no pronounced trend in the average amount of such imposts collected since 1921. In all regions except the Far West, the average amounts collected in 1938 were above the average amounts collected in 1921, the greatest increase being in the Southwest, amounting to 35 percent. Much of this increase is due not to changes in registration fee schedules but to additional charges levied on motor-vehicle owners since 1921. For example, the licensing of operators and chauffeurs and the collection of fees therefor is much more widespread today than in 1921. Other charges such as fines and penalties and certificates of title and transfer fees, individually small but providing considerable sums of revenue, are included in the total of motor-vehicle imposts.

TABLE 10.—*Average registration fees and other motor-vehicle imposts per registered vehicle, by regions, in 1921, 1930, and 1938*

Region	Average fee in—		
	1921	1930	1938
Northeast	$13.82	$16.80	$16.79
Southeast	12.18	15.24	12.58
Southwest	9.13	10.66	12.35
Middle States	11.37	12.42	13.33
Northwest	8.13	10.17	8.21
Far West	12.48	9.84	8.86
United States	11.71	13.40	13.19

Analyses of motor-vehicle data will be materially aided when more uniform methods and classifications are adopted by the several States. At present, busses are sometimes included with passenger-car registrations, sometimes with trucks, sometimes shown separately; and the segregation of such registrations at the end of the registration year is usually not economically practicable. Similar conditions exist with reference to certain types of trucks registered with passenger cars and with reference to certain types of commercially operated passenger cars registered with trucks.

There has been marked improvement in registration practice in recent years as far as the segregation of vehicles by types is concerned but much improvement is still possible in the segregation of registration fees by types of vehicles. Table 6 indicates that in five States no segregation is possible. Moreover, the reported segregations are believed to be of doubtful accuracy in other States. However, analysis of the existing data, unsatisfactory as they are in certain respects, makes possible the general observations and conclusions noted in this discussion and suggests that further study of social, economic, and demographic factors in the United States will reveal other important relationships to motor-vehicle statistics.

HIGHWAY RESEARCH BOARD WILL MEET IN DECEMBER

The Nineteenth Annual Meeting of the Highway Research Board of the National Research Council will be held in Washington, D. C., Tuesday to Friday, December 5–8, 1939. Reports on highway research investigations will be presented, and the formal meetings of the Board will be supplemented with open meetings for informal discussion of pertinent topics. A program of reports will be announced by the Board about November 1.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF SEPTEMBER 30, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 491,450	$ 479,159	4	1		$ 767,612	$ 766,084	12			$ 32,908	$ 32,700		1	2	$ 826,692
Arizona						518,061	515,813	6								209,120
Arkansas	189,891	189,891	3			68,938	68,938	1			639,990	633,485	4		8	577,263
California	398,219	398,219	4			1,384,358	1,383,263	7			137,560	137,560	2		1	1,165,815
Colorado	309,307	309,305	3		11	306,992	306,992	2			48,514	44,754			14	791,132
Connecticut						172,722	161,008		1							850,557
Delaware						9,150	9,150	2			2,320	2,320			1	513,891
Florida						643,232	642,396	4			7,800	7,800			2	1,016,395
Georgia	56,530	56,530	3			370,750	370,750	4			315,346	315,346	5	3	20	2,123,394
Idaho						314,492	282,961	4								466,886
Illinois	1,580,055	1,579,195	8	2	29	2,073,480	1,950,092	13	2	21	131,050	105,955			37	2,051,037
Indiana	316,903	316,903	2		45	680,850	680,850	2	1	25	513,919	513,919	1	2	32	790,758
Iowa	313,878	295,800	7			191,842	161,006	7		10	686,446	642,800	1		168	1,003,068
Kansas	404,329	404,329	6		4	943,832	943,832	9		1	237,831	237,831	3		3	897,466
Kentucky	149,212	149,212	3	2		626,864	626,864	9	2		630,607	630,607	4		4	515,950
Louisiana	122,838	122,830	2			490,999	490,989	5			651,161	597,659	12			584,469
Maine	264,771	264,771	2	2		271,066	271,066	3	1							236,310
Maryland	24,510	15,402				275,795	179,002	2		2	46,200	46,200			13	986,891
Massachusetts	265,259	264,538	1	2		257,307	256,764	3			14,320	14,320	1			1,711,447
Michigan	194,426	194,426	2	1		977,150	977,150	6	1		310,220	310,220	2		39	1,543,746
Minnesota	163,676	163,646	1	3	6	1,222,692	1,202,541	7	3	6	192,473	191,433	3		5	1,364,578
Mississippi						611,373	611,373	8			37,300	37,300	1			889,528
Missouri	65,589	64,284	1			1,197,536	1,197,536	7	1		775,239	710,060	3	5		1,338,080
Montana	439,450	439,450	4			624,130	509,049	8								284,589
Nebraska	28,481	28,481	1			1,124,383	1,124,383	24	1		149,270	149,270			41	637,289
Nevada	122,064	122,064		1	1	72,234	72,234		1	7	11,577	11,577			5	113,530
New Hampshire	15,305	15,305	3		1	147,279	146,776	5			51,837	51,837	2			307,567
New Jersey						737,456	737,456	2	3	1						1,439,345
New Mexico	59,805	59,805	2			15,276	15,276				2,572	2,572			1	682,071
New York	809,800	807,450	1	4		2,414,022	2,376,472	11	9		542,933	400,813	1	2		3,419,903
North Carolina	506,240	506,240	4	1	16	1,027,390	992,290	9	4	6	186,930	186,930		1	43	846,675
North Dakota	105,450	105,450	3	1		813,489	770,087	9			75,960	75,960	1			391,201
Ohio	241,760	241,760	4			1,365,783	1,329,021	10	2		639,920	590,730	3	1	2	2,901,510
Oklahoma	267,055	266,955	2		32	183,725	183,725	3		10	220,700	207,900	2	1	8	2,014,297
Oregon	21,689	21,689				148,072	146,777	1			135,740	135,740	2			311,060
Pennsylvania						2,161,799	1,949,900	5	3		702,857	494,405	3	1	2	4,218,923
Rhode Island	327,613	327,613	1	2		111,178	111,178		1							152,459
South Carolina	98,810	65,428	2	1		586,856	564,520	6	4		262,279	262,279	2	2	35	780,994
South Dakota	67,040	67,040		1	7	332,420	332,420	4	2	10	47,550	47,550	1			1,005,070
Tennessee	73,600	73,600		1	1	632,746	632,746	2	2	2	179,620	179,620	1	1	2	1,326,544
Texas	726,135	725,800	6	1		2,283,130	2,252,002	20	2		572,319	503,830	4		26	1,911,733
Utah	82,650	82,650	2		20	125,198	125,198	1		45	213,310	213,310			79	175,910
Vermont	32,489	27,676			6	4,032	4,032			1	118,940	118,940		1		199,307
Virginia	195,562	195,562	2		11	531,513	438,613	7	2	5	109,256	109,256		1	4	923,325
Washington	35,507	35,506			5	295,956	294,546	3	1	3	60,782	60,782		1	4	514,672
West Virginia	40,817	40,817				334,034	313,274	7			24,200	24,200	1		3	958,494
Wisconsin	335,448	334,617	5		1	1,189,040	1,145,011	10	1	4	341,541	336,847	1	2	9	950,823
Wyoming	40,626	40,470	1		7	98,543	98,543		1							516,862
District of Columbia						292,412	258,868	1			74,400	74,400				44,918
Hawaii	50,320	50,320	1			132,850	132,850	3	1							359,450
Puerto Rico	49,040	48,840	1			345,312	343,310	8								426,676
TOTALS	10,083,599	9,999,028	97	26	203	32,512,351	31,461,977	282	52	159	10,135,697	9,451,017	66	25	613	50,269,670

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF SEPTEMBER 30, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 1,419,026	$ 704,435	80.8	$ 8,239,554	$ 4,097,698	279.8	$ 520,090	$ 259,040	10.2	$ 2,683,476
Arizona	393,055	262,718	12.6	1,879,258	1,322,913	90.0	709,475	447,524	36.4	904,668
Arkansas	1,766,060	1,765,115	133.0	1,686,701	1,658,251	89.6	85,631	83,150	2.7	1,696,477
California	4,105,120	2,241,010	53.9	2,654,076	1,448,733	37.9	1,558,518	812,071	35.1	3,154,770
Colorado	1,013,767	554,704	25.1	3,608,403	2,016,061	81.7	464,920	234,769	17.1	1,799,439
Connecticut	357,558	176,334	5.1	1,880,711	935,261	17.9	176,447	87,811	2.0	1,166,556
Delaware	456,831	225,935	12.0	1,311,331	640,107	26.2	208,316	104,158	.3	1,018,460
Florida	121,000	60,500	1.4	3,893,470	1,946,511	67.0	896,393	448,196	13.8	2,500,284
Georgia	2,296,600	1,148,300	123.3	6,375,253	3,187,626	332.0	1,461,411	730,706	70.5	5,238,060
Idaho	1,337,479	803,955	56.5	1,378,889	839,260	70.2	335,689	176,791	42.7	1,092,894
Illinois	2,027,248	1,006,240	41.9	9,273,775	4,635,841	190.7	2,257,930	1,127,375	59.8	2,782,680
Indiana	2,028,266	1,014,133	35.3	5,654,834	2,802,817	130.1	1,363,774	681,762	22.1	1,800,458
Iowa	1,097,129	512,074	68.5	5,803,838	2,581,483	202.5	284,982	133,625	9.7	935,894
Kansas	1,326,445	655,517	67.0	3,438,215	1,718,228	174.5	2,631,828	1,315,914	137.9	4,182,783
Kentucky	1,324,019	662,010	42.8	3,529,810	1,763,349	95.8	894,180	447,090	23.1	2,828,763
Louisiana				12,212,226	3,151,747	53.3	1,289,631	624,124	37.3	2,596,998
Maine	1,151,290	575,286	21.2	1,687,170	843,585	45.8	23,360	11,680	.5	322,718
Maryland	1,155,690	548,711	19.0	2,644,573	1,307,905	39.4	487,000	239,500	8.3	1,796,410
Massachusetts	2,529,677	1,263,104	18.6	820,626	409,358	6.5	1,737,109	865,037	12.1	2,482,079
Michigan	1,605,321	773,982	37.9	4,687,281	2,341,001	142.7	1,114,810	409,880	24.4	2,780,254
Minnesota	1,710,526	845,601	103.9	6,331,663	3,142,809	357.5	2,048,113	1,022,966	87.5	3,046,876
Mississippi	631,000	231,470	35.1	8,649,588	3,184,995	348.9	933,100	448,050	21.6	2,151,301
Missouri	1,124,969	561,802	43.2	4,832,386	2,397,484	182.1	2,883,263	1,167,400	77.8	4,144,443
Montana	657,051	370,857	42.6	3,095,327	1,751,875	152.6	669,089	379,505	37.4	4,142,038
Nebraska	272,839	136,420	31.5	6,752,521	3,375,612	583.4	1,646,619	823,310	191.8	2,675,130
Nevada	927,829	802,281	43.1	709,242	609,648	30.0	409,491	352,209	17.0	729,568
New Hampshire	98,756	48,285	4.0	1,257,038	617,621	29.9	427,033	211,357	14.6	849,313
New Jersey	538,290	269,145	6.0	4,028,848	2,012,874	29.8	459,840	229,920	2.2	1,808,846
New Mexico	891,754	546,888	76.6	1,226,520	751,434	58.5	259,102	161,705	31.1	1,372,235
New York	3,361,580	1,653,997	64.5	13,440,309	6,588,408	229.3	2,521,750	1,082,785	31.7	1,427,095
North Carolina	1,728,670	862,255	103.5	7,000,203	3,492,617	380.4	395,780	193,490	21.1	1,525,401
North Dakota	87,260	46,736	17.1	1,325,469	710,291	96.4	2,241,260	1,201,261	244.7	3,372,887
Ohio	1,149,720	574,860	13.2	10,001,476	4,932,024	110.1	2,246,760	1,101,687	22.5	6,386,295
Oklahoma	708,559	375,209	5.9	2,895,525	1,536,496	103.8	2,230,520	1,186,465	89.9	3,244,489
Oregon	807,310	490,870	34.5	2,770,347	1,674,927	131.8	1,220,045	566,648	19.4	1,352,271
Pennsylvania	2,789,564	1,378,205	39.9	9,656,285	4,659,067	87.3	2,269,893	1,124,738	32.2	3,946,942
Rhode Island	300,790	150,275	4.2	517,736	258,681	6.0	647,521	323,235	5.9	910,008
South Carolina	1,145,840	516,200	52.2	1,716,194	761,287	34.0	232,000	98,000	24.8	2,397,197
South Dakota	1,570,025	861,560	134.8	3,749,899	2,095,600	360.2	1,075,150	608,160	113.6	3,178,828
Tennessee	618,580	309,290	15.5	4,023,864	2,011,932	110.3	458,698	229,349	9.8	4,246,863
Texas	5,757,839	2,838,675	352.3	8,607,749	4,260,263	368.5	2,037,596	1,005,300	97.7	6,038,518
Utah	1,075,365	774,250	46.9	1,474,670	1,065,935	81.3	128,925	92,740	2.0	838,437
Vermont	543,111	267,281	11.9	210,948	104,734	6.6	164,000	82,000	4.9	615,414
Virginia	1,125,870	560,870	32.9	2,919,747	1,419,640	73.6	681,457	335,014	21.6	1,000,657
Washington	1,410,072	734,800	17.8	2,297,579	1,199,257	28.8	1,450,676	602,058	10.1	722,895
West Virginia	568,977	321,150	19.7	2,753,255	1,393,865	62.7	856,777	424,083	29.3	1,819,165
Wisconsin	2,342,651	1,146,023	101.1	7,346,868	3,626,860	218.9	606,750	294,765	23.8	1,637,610
Wyoming	716,787	444,071	79.0	917,797	566,162	76.0	941,766	594,331	104.6	583,947
District of Columbia				198,624	99,312	1.2	257,500	128,750	1.9	259,438
Hawaii	139,685	66,970	1.0	1,005,840	480,750	16.4	568,447	280,803	9.8	1,051,180
Puerto Rico	302,230	150,315	6.4	1,567,819	778,155	31.2	163,352	80,795	1.8	365,530
TOTALS	62,615,060	33,290,674	2,396.2	205,941,330	101,208,350	6,561.1	51,633,767	25,673,082	1,970.1	111,604,938

PUBLICATIONS of the PUBLIC ROADS ADMINISTRATION

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No. 265T. . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

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House Document No. 272 . . . Toll Roads and Free Roads.

Indexes to PUBLIC ROADS, Volumes 6–19, inclusive.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration (formerly the *Bureau of Public Roads*), classified according to subject and including the more important articles in Public Roads, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF SEPTEMBER 30, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 186,105	$ 91,750	13.7	$ 885,145	$ 354,900	22.9	$ 78,300	$ 39,150	6.3	$ 739,456
Arizona	56,191	40,524	11.0	241,691	173,791	22.4				325,941
Arkansas	328,755	324,677	39.7	266,508	261,812	37.6	153,862	153,756	17.6	276,875
California	151,277	85,419	17.1	990,834	507,423	31.2	219,075	117,065	4.9	645,261
Colorado	211,957	108,270	10.1	694,169	367,583	23.3	108,036	43,368	.2	67,572
Connecticut				172,794	72,417	2.9	108,041	37,810	.2	248,439
Delaware	80,840	40,420	17.5	71,661	35,830	7.8				232,384
Florida	123,817	61,550	3.4	865,805	428,544	34.2	7,358	3,679		371,271
Georgia	168,617	83,378	21.0	317,740	158,870	38.3	165,599	82,800	22.3	1,058,626
Idaho	127,733	76,396	4.9	310,653	166,378	36.9	138,206	70,687	10.1	123,412
Illinois	589,113	294,216	18.5	1,247,200	569,600	88.7	431,700	213,795	30.0	586,506
Indiana	300,200	150,100	25.8	826,970	412,281	64.1	130,926	65,463	9.7	660,414
Iowa	24,095	11,069	22.9	296,129	139,410	31.1	738,581	346,825	107.6	1,182,503
Kansas	7,806	3,903	6.0	159,712	79,856	38.6	411,025	210,292	9.3	1,286,782
Kentucky	199,808	66,485	31.2	1,142,618	311,560	60.3	696,256	246,918	56.6	223,613
Louisiana	322,110	154,055	28.4	430,351	194,455	32.8	356,361	154,721	29.6	329,341
Maine	282,703	141,280	16.2	202,820	100,324	11.8	19,700	9,850	1.2	9,067
Maryland	197,291	94,987	14.5	11,296	5,648	1.5	263,000	84,555	14.1	350,491
Massachusetts	101,519	50,435	2.4	243,465	120,729	5.2	341,556	169,241	7.5	449,264
Michigan	275,490	132,202	10.0	1,266,090	630,545	110.4	342,108	171,054	26.4	803,053
Minnesota	284,968	142,347	23.8	701,916	349,051	66.1	232,118	116,059	41.1	1,051,048
Mississippi	176,500	88,250	6.8	636,062	292,246	45.2	271,400	145,700	32.5	624,670
Missouri	215,534	105,775	41.2	782,154	381,177	85.8	553,987	225,340	62.0	554,866
Montana	111,913	63,475	10.8	702,330	398,292	58.3	61,970	35,149	6.9	835,112
Nebraska	445,634	212,802	84.4	802,179	394,532	141.5				380,221
Nevada	160,777	139,268	25.0	70,067	60,261	18.1	57,534	28,767	9.6	142,327
New Hampshire	61,156	29,708	2.4	2,192	1,096		111,620	53,835	3.1	135,325
New Jersey	87,010	43,300	2.9	393,530	194,230	16.8	94,300	47,150	4.7	507,278
New Mexico	159,661	97,765	9.8	339,901	208,610	32.4	370,828	152,394	26.9	70,687
New York	692,736	341,609	35.4	2,128,960	997,052	93.8	687,380	269,240	6.5	321,249
North Carolina	470,594	235,275	37.0	965,730	482,865	94.4	6,030	2,500	1.0	326,205
North Dakota	115,030	61,606	8.3				148,770	79,717	10.9	819,207
Ohio	94,160	47,080	6.1	870,960	440,945	43.9	236,000	118,000	7.1	1,689,327
Oklahoma	73,190	38,943	.8	217,196	115,568	11.8	501,355	266,763	32.8	906,171
Oregon	243,134	146,330	30.8	554,456	282,952	57.3	35,927	16,820	3.0	291,366
Pennsylvania	1,578,647	777,647	96.7	1,297,656	642,723	46.7	454,542	227,271	15.4	221,917
Rhode Island	93,827	46,890	2.2	81,236	40,618	.2	36,060	18,030	.4	78,277
South Carolina	504,587	204,690	48.8	79,320	34,379	8.2	330,400	142,044	22.2	204,807
South Dakota				16,170	8,890	4.1				1,049,160
Tennessee	472,560	197,110	24.3	261,456	130,728	7.7				858,499
Texas	1,448,151	686,551	172.9	1,104,202	536,220	91.9	83,340	41,550	17.3	1,055,553
Utah	123,645	71,116	22.7	97,955	51,747	12.8	17,800	12,000	4.0	197,199
Vermont	91,158	45,153	4.0	222,662	78,983	7.8				52,328
Virginia	472,654	228,831	47.2	268,130	132,429	23.8	229,030	105,050	14.6	209,027
Washington	387,157	201,942	19.8	383,574	201,118	29.1	103,829	53,400	11.2	211,542
West Virginia	145,150	72,575	8.4	13,015	6,507		165,676	82,838	8.6	430,576
Wisconsin	403,848	201,709	22.4	690,232	344,457	13.3	324,158	157,423	7.9	545,669
Wyoming	402,460	248,619	22.3	231,836	146,343	14.1	111,591	64,828	19.5	58,196
District of Columbia				14,592	6,796	.1	109,600	54,800	1.1	11,529
Hawaii	90,660	45,330	3.8	205,590	102,795	4.6	179,480	89,705	6.4	82,170
Puerto Rico				224,465	109,130	12.8	55,188	27,140	2.1	60,233
TOTALS	13,341,928	6,832,812	1,135.3	25,003,375	12,264,696	1,744.6	10,279,603	4,854,542	732.4	23,952,012

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

FEDERAL WORKS AGENCY
PUBLIC ROADS ADMINISTRATION

VOL. 20, NO. 9 NOVEMBER 1939

A SECTION OF U S 91 IN UTAH

PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

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PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 20, No. 9 November 1939

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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CERTIFICATE: By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

STUDIES OF WATER-RETENTIVE CHEMICALS AS ADMIXTURES WITH NONPLASTIC ROAD-BUILDING MATERIALS

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by E. A. WILLIS, Associate Highway Engineer, and C. A. CARPENTER, Associate Civil Engineer

DURING the past several years the Public Roads Administration has conducted laboratory and field studies of various types of base-course materials and the factors that influence their behavior in service. The results of two of the laboratory investigations have been published in recent issues of PUBLIC ROADS.[1]

Observation of the behavior of soil road surfaces and the performance of the same materials following the application of bituminous surfaces has suggested the need for laboratory study of this type of construction. Such observations have already established the following facts:

1. Mixtures of granular aggregate and clay binder that form highly stable road surfaces may become unstable as bases when covered with a waterproof surfacing.
2. Nonplastic granular materials, having gradings within definitely established limits, provide stable base courses for relatively thin bituminous surface treatments.
3. These same nonplastic materials when subjected to traffic prior to surface treatment may be loose and dusty in dry weather and the loss of surface metal may be excessive.
4. Moisture films serve to bind such nonplastic aggregates into a coherent road surface.
5. Certain chemicals used either as admixtures or surface applications aid materially in maintaining these moisture films under suitable climatic conditions.

This report describes investigations using the outdoor circular track, shown in figure 1, to determine the effect of the water-retentive chemicals, calcium chloride and sodium chloride, on nonplastic granular mixtures under controlled traffic and moisture conditions both before and after the application of a thin bituminous surface treatment.

The circular track used in these investigations was, with the exception of tire equipment, a duplicate of the indoor track used in the studies previously reported.[1] The test wheels for the outdoor setup were equipped with high-pressure tires, size 30×5, requiring an inflation pressure of 80 pounds per square inch instead of the size 6.00–20 low-pressure tires that were used with the indoor equipment. The load was, as in the indoor track tests, 800 pounds on each wheel. This was increased to 1,000 pounds near the end of some of the tests.

Distributed traffic which was used for compacting and testing the unsurfaced mixtures was obtained by gradually shifting the rotating beam longitudinally with respect to its axis of rotation, causing the wheels to pursue alternately expanding and contracting spiral courses covering the entire track area. Concentrated traffic, which was used after the surface treatment had been constructed, was obtained by locking the sliding pivot of the beam in such a position that the wheels pursued two concentric circular courses whose center lines were about 2½ inches on either side of the center line of the track.

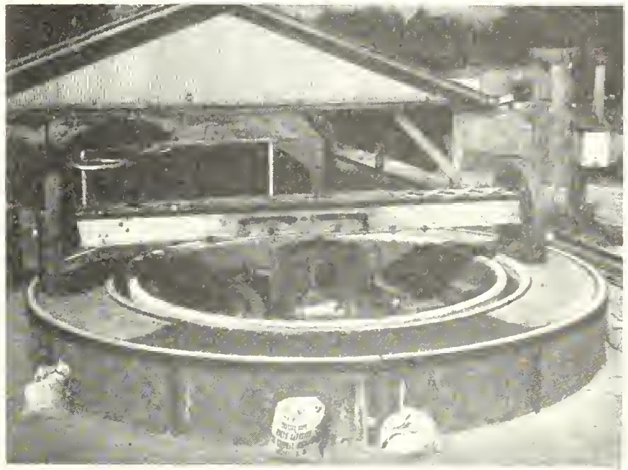

FIGURE 1.—THE OUTDOOR CIRCULAR TRACK USED IN TESTING ROAD-BUILDING MATERIALS. IN THE BACKGROUND IS THE MOVABLE SHED USED TO COVER THE TRACK AT NIGHT AND DURING RAINY WEATHER.

This investigation involved the construction and testing of 20 track sections. Each section was 18 inches wide, 6 inches deep, and approximately 7.5 feet long. Five sections comprised a test track and were tested as a group. Thus four tracks were required to test the 20 sections.

VARIOUS AGGREGATES AND ADMIXTURES USED IN TEST SECTIONS

The gradings and soil constants of the aggregates used in the 20 test sections are given in table 1. The materials comprising the 15 test sections of tracks 1, 2, and 3 were prepared by combining Potomac River gravel, Potomac River sand, pulverized silica, and a local clay soil having a liquid limit of 41 and a plasticity index of 18.

Crusher-run limestone, blast-furnace slag, and granite were used in the construction of the five sections tested in track 4.

Tracks 1, 2, and 3, except for minor differences in grading incident to slight variations in the stock materials, had identical composition. In section 1 of each of the three tracks the material passing the No. 200 sieve was primarily the clay soil while in all other sections the fines consisted primarily of the inert pulverized silica. Sections 1 and 2 of tracks 1, 2, and 3, had approximately the same amounts of material passing the No. 200 sieve. Sections 3, 4, and 5, differed from sections 1 and 2 and from each other primarily in the amount of mineral dust present.

[1] A study of Sand-Clay Materials for Base-Course Construction, by C. A. Carpenter and E. A. Willis. PUBLIC ROADS, November 1938. A study of Sand-Clay-Gravel Materials for Base-Course Construction, by C. A. Carpenter and E. A. Willis. PUBLIC ROADS, March 1939.

TABLE 1.—*Gradings and soil constants of materials used in study of water-retentive chemicals*

	Track No. 1, section—					Track No. 2, section—					Track No. 3, section—					Track No. 4, section—				
	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5
Grading:	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Passing 1-inch sieve	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
Passing ¾-inch sieve	98	98	95	96	97	98	96	97	97	98	96	92	97	93	97	100	100	100	100	100
Passing No. 4 sieve	75	80	66	69	63	76	73	67	62	58	79	67	56	61	59	98	94	65	98	95
Passing No. 10 sieve	62	69	57	59	52	65	64	56	50	46	66	59	48	50	46	55	63	35	64	56
Passing No. 40 sieve	40	46	37	35	31	40	43	35	30	26	45	41	33	30	29	25	43	19	41	37
Passing No. 200 sieve	23	24	18	12	9	23	26	19	12	7	25	22	16	12	9	12	16	5	16	14
Passing 0.005 mm	7	6	5	4	4	8	8	6	5	5	11	5	4	3	3	3	3	1	3	2
Dust ratio[1]	58	52	49	34	29	58	60	54	40	27	56	54	48	40	31	48	37	26	39	38
Tests on material passing No. 40 sieve:																				
Liquid limit	17	17	18	16	18	18	17	16	15	16	17	14	14	13	10	14	15	27	25	25
Plasticity index	2	0	0	0	0	3	2	2	0	0	2	0	0	0	0	2	0	0	0	0

[1] $\text{Dust ratio}=100\left[\frac{\text{percentage passing No. 200 sieve}}{\text{percentage passing No. 40 sieve}}\right].$

In track 4, section 1 consisted of limestone, section 2 of granite, section 3 of blast-furnace slag, section 4, 90 percent by weight of granite and 10 percent slag, and section 5, 90 percent by weight of granite and 10 percent limestone.

Calcium chloride was used as an admixture in track 1 and sodium chloride in track 2. Track 3 was tested without a chemical admixture. Track 4 was tested first without chemical treatment and then with a surface application of calcium chloride.

In constructing the test sections of tracks 1, 2, and 3, sufficient water including that used to dissolve the chemicals was added to the aggregates to bring the mortar portion to its optimum moisture content as previously determined by the Proctor test (A. A. S. H. O. Standard Compaction Test No. T99–38) with a slight excess for wetting the coarse aggregate.

No Proctor compaction tests were made on the crusher-run materials used in track 4. Just enough water was combined with the mixtures used in this track to cause them to hold a cast when squeezed in the hand. Vibratory compaction[2] tests were made on these materials subsequent to the construction of the sections.

The moisture contents of all sections immediately after being placed in the track and the optimum moisture contents for the mortars of the materials used in tracks 1, 2, and 3, are shown in table 2.

The procedures for preparing the materials for the track tests, constructing the test sections, and surface-treating them were as follows:

1. Sufficient materials were prepared for only one track at a time. The aggregates were proportioned by weight from the stock materials to give the desired gradings and were thoroughly mixed before any water was added.

2. Water was added and mixing continued to distribute the moisture.

3. In tracks 1 and 2, the chemical admixture, in the amount of 2 pounds per square yard, was added as a solution along with the water.

4. The moistened mixtures were then placed in the trough of the track in two approximately equal layers, each layer being compacted with the traffic of pneumatic-tired wheels uniformly distributed over the surface.

5. Compaction was continued on the top layer until no further subsidence was noted and all sections were in suitable condition for testing. This required 18,200 wheel-trips, 64,000 wheel-trips, 60,000 wheel-trips, and 82,600 wheel-trips for tracks 1, 2, 3, and 4, respectively.

6. Testing of the materials without a bituminous surface treatment then proceeded.

7. After this phase of the testing had been completed, the sections were reshaped and trimmed smooth.

8. A prime consisting of 0.3 gallon per square yard of light tar was applied and allowed to cure.

9. A surface treatment consisting of 0.4 gallon of hot application bituminous material and a cover of 50 pounds per square yard of stone of ¾-inch maximum size was constructed.

10. The treatment was consolidated by additional distributed traffic until the surface was well sealed and showed no movement.

[2] A New Vibratory Machine for Determining the Compactibility of Aggregates, by J. T. Pauls and J. F. Goode, PUBLIC ROADS, May 1939.

TABLE 2.—*Moisture contents immediately after construction and optimum moisture contents on the fraction of material passing the No. 10 sieve*

Track No.	Section No.	Moisture content of sections after placing[1]	Optimum moisture content of material passing No. 10 sieve[2]
		Percent	*Percent*
1	1	8.6	9.8
	2	6.9	9.8
	3	7.0	8.6
	4	6.2	9.1
	5	6.9	9.0
2	1	7.1	10.0
	2	6.4	9.5
	3	6.6	9.5
	4	5.4	8.9
	5	4.3	8.6
3	1	6.9	10.0
	2	6.2	10.3
	3	5.3	9.7
	4	4.8	9.8
	5	4.3	9.1
4	1	6.7	
	2	10.0	
	3	8.0	
	4	9.6	
	5	11.2	

[1] Based on the dry weight of the total aggregate.
[2] Based on the dry weight of the portion of the aggregate passing the No. 10 sieve.

WEATHER CONDITIONS VARIED CONSIDERABLY DURING TEST

The outdoor track was used in these investigations because it was desired to subject the materials treated with water-retentive chemicals to the influence of changes in temperature and humidity similar to those encountered on roads in service. A recording thermometer and hygrometer was installed near the track to determine these factors. A movable sheet metal roof, shown in figure 1, was used to cover the track at night and on rainy days so that the amount of water placed on the surface of each section could be accurately controlled.

The tests described in this report were conducted at different times of the year. A brief summary of the temperature and humidity data collected by means of the recording instrument, previously mentioned, during the tests on the four tracks is presented in table 3.

The behavior of the materials under test was judged on the basis of the appearance of the sections at various stages of the tests supplemented by measurements of vertical displacement of the surface. The measurements were made with the transverse and longitudinal profilometers which have been described in the previous reports.

TABLE 3.—*Summary of weather data*

	Track No. 1	Track No. 2	Track No. 3	Track No. 4
Date constructed	7-15-36	10-19-36	4-12-37	10-8-37
End of test	10-12-36	4- 3-37	6-11-37	4-2-38
Average daily maximum temperature ° F.	83.3	51.0	75.2	52.1
Average daily minimum temperature ° F.	62.1	32.0	51.7	31.9
Maximum recorded temperature ° F.	101	81	93	86
Minimum recorded temperature ° F.	42	16	32	16
Greatest change in 24 hours:				
From ° F.	101	69	93	74
To ° F.	67	31	42	29
Average daily maximum relative humidity percent	88.4	81.0	84.0	82.8
Average daily minimum relative humidity percent	35.8	31.0	26.0	39.1
Maximum recorded relative humidity percent	94	93	92	93
Minimum recorded relative humidity percent	14	9	9	6
Greatest change in 24 hours:				
From percent	90	90	92	92
To do	14	10	9	8

The resistance to raveling of the various materials when tested without the protective surface treatment was judged primarily by visual observation. No close correlation could be obtained between vertical displacement and the time raveling started because the concrete curbs prevented much of the loosened material from being thrown off the surface. During the portion of the test period in which water was sprinkled on the surface, increasing rates of vertical displacement were observed in some instances even though during this stage the surface was generally well bonded and in good condition.

An average vertical displacement of about 0.25 inch, measured after the sections had been surface treated and subjected to the action of concentrated traffic, was observed to be sufficient to cause noticeable damage to the bituminous surface. This is in agreement with conclusions reached in previous investigations using the same apparatus. Numerically, the amount of rutting measured with the longitudinal profilometer agreed in general with the amount of vertical displacement measured with the transverse profilometer.

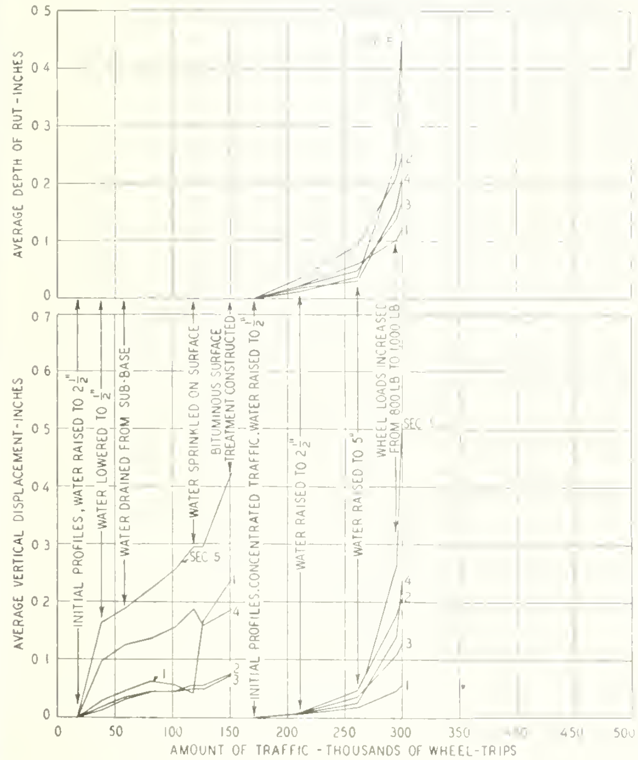

FIGURE 2.—SURFACE DISPLACEMENTS OF SECTIONS OF TRACK 1 AT VARIOUS STAGES OF THE TEST.

Changes in behavior of the various sections under altered test conditions are clearly shown by abrupt changes in the slopes of the displacement curves in figure 2 for track 1 and in subsequent figures for tracks 2, 3, and 4.

Track 1: Calcium chloride admixture.—The schedule of traffic applications and changes in water elevation with notations on the behavior of the five test sections of track 1 are given in table 4.

Figure 2 shows the combined effect of consolidation and loss of surface material as measured by the transverse profilometer for the period up to 151,200 wheel-trips during which time the sections were being tested under distributed traffic, without bituminous surfaces. It also shows, for the period from 171,200 wheel-trips to the end of the test, the displacements of the sections as measured with both profilometers while testing under concentrated traffic, with bituminous surfaces.

TABLE 4.—*Schedule of operations and behavior of test sections in track 1 with calcium chloride*

Operation	Traffic	Water level above top of sub-base	Behavior: Sec. 1	Sec. 2	Sec. 3	Sec. 4	Sec. 5
	Wheel-trips	*Inches*					
Placing and compacting	0 to 18,200	[1] 0	Unstable	Good	Good	Good	Good.
Testing with distributed traffic	18,200 to 38,200	2½	Slightly unstable	do	do	do	Slight raveling.
Do	38,200 to 58,200	½	Good	do	do	do	Raveled.
Do	58,200 to 118,200	[1] 0	do	do	do	Slight raveling	Do.
Sprinkling and testing with distributed traffic.	118,200 to 151,200	[1] 0	Slightly unstable	do	do	Good during sprinkling, some raveling later.	Good during sprinkling, raveled later.
Compacting bituminous surface treatment.	151,200 to 171,200	[1] 0	Good	do	do	Good	Good.
Testing with concentrated traffic	171,200 to 211,200	½	do	do	do	do	Do.
Do	211,200 to 261,200	2½	do	do	do	do	Do.
Do	[2] 261,200 to 298,500	5	do	do	do	Slightly unstable	Unstable.

[1] No water in sub-base.
[2] Wheel loads increased from 800 to 1,000 pounds at 295,000 wheel-trips.

Loosening of the surface metal under distributed traffic was first noted at about 35,000 wheel-trips in section 5, which was the section having the lowest percentage of No. 200 material. At this time the water was $2\frac{1}{2}$ inches above the bottom of the test layer. Traffic was continued and the water level lowered (see table 4) until the base was finally drained. Raveling progressed in section 5 until, at 118,200 wheel-trips, the surface was quite loose and open as shown in figure 3. A similar action in lesser degree was noted in section 4.

FIGURE 3. TYPICAL SECTIONS OF TRACK 1 AT 118,200 WHEEL-TRIPS, JUST BEFORE THE FIRST SPRINKLING. LEFT, SECTION 2, WHICH IS ALSO REPRESENTATIVE OF SECTIONS 1 AND 3; RIGHT, SECTION 5, WHICH IS ALSO REPRESENTATIVE OF SECTION 4.

Sections 2 and 3 remained in good condition throughout this portion of the test. Section 1 failed to compact well during the initial compaction period (0 to 18,200 wheel-trips) but began to set up soon after water was admitted to the sub-base and exhibited no signs of excessive raveling from about 38,000 wheel-trips to 118,200 wheel-trips, when the track was first sprinkled. Figure 3 shows section 2 at 118,200 wheel-trips. Sections 1 and 3 were in a similar condition at this time. Some exposed aggregate was evident, particularly along the curb lines where abrasion was most severe, but in general the surfaces were dense and well bonded.

LEACHING TESTS ON TRACK 1 STARTED AT 118,200 WHEEL-TRIPS

Water was applied to the surface of the test sections in track 1 during the traffic test period from 118,200 to 129,600 wheel-trips in the following manner:

1. Temporary dikes of plastic clay were placed at the ends of each section.
2. Water was sprinkled on the surface in increments equivalent to one-fourth inch of rainfall distributed over the area of each section.
3. The water was allowed to soak into the respective sections and to percolate through the test course, into the sub-base, and out the drains at the bottom.
4. After each of the first six applications of water had disappeared from the surface the dikes were removed and about 2,000 trips of test traffic applied.

Nine applications of water or the equivalent of $2\frac{1}{4}$ inches of rainfall were allowed to percolate down through the test course and six increments of traffic, 11,400 wheel-trips in all, were applied, bringing the total traffic to 129,600 wheel-trips.

The first application of water disappeared from the surface of section 5 in about 2 hours, and about 24 hours were required for the water to disappear completely from section 1. The time required for the water to enter the mixtures became progressively greater with each increment of water until toward the end of this phase of the test, 24 hours was required for section 5 to transmit a $\frac{1}{4}$-inch application of water.

Samples were taken from each section near the centerline just before the first application of water (118,200 wheel-trips) and again after the final application had leached through all sections. These samples were obtained by boring through the entire thickness of the test layer with a $1\frac{1}{2}$-inch soil auger. Care was taken to save all the material from the test holes, which were made as nearly uniform in cross section throughout their depth as possible. The moisture content of each section as well as the calcium chloride content recovered from that portion of each boring passing the No. 10 sieve are shown in table 5 for the times indicated above as well as at the beginning and end of the test.

TABLE 5.—*Moisture contents and calcium chloride contents in track 1 at several stages of the test*

Section No.	Number of wheel-trips	Stage of test	Moisture content based on dry weight	Calcium chloride content of portion passing No. 10 sieve
			Percent	*Percent*
1	2,700	Start	8.6	0.22
	118,200	Before sprinkling	1.7	1.11
	129,600	After sprinkling	4.5	.32
	298,500	After testing with bituminous surface	5.6	.17
2	2,700	Start	6.9	.19
	118,200	Before sprinkling	1.3	.33
	129,600	After sprinkling	3.7	.08
	298,500	After testing with bituminous surface	5.3	.05
3	2,700	Start	7.0	.22
	118,200	Before sprinkling	1.4	.20
	129,600	After sprinkling	4.1	.11
	298,500	After testing with bituminous surface	4.6	.03
4	2,700	Start	6.2	.26
	118,200	Before sprinkling	1.4	.06
	129,600	After sprinkling	3.3	.05
	298,500	After testing with bituminous surface	5.7	0
5	2,700	Start	6.9	.27
	118,200	Before sprinkling	1.3	.06
	129,600	After sprinkling	3.2	.12
	298,500	After testing with bituminous surface	5.9	0

Tests on the mortar portion of the five mixtures just before laying showed calcium chloride contents of 0.19 to 0.27 percent of the dry weight of the fraction passing the No. 10 sieve. After 118,200 wheel-trips, the samples showed calcium chloride contents in the mortar portion of 1.11 percent for section 1, and 0.33 percent for section 2. The percentages of calcium chloride in the other sections at this time were less than at the start of the test, being 0.20 percent for section 3, and 0.06 percent for both sections 4 and 5.

Sections 1 and 2, which showed marked increases in chloride content along the center line, were denser and had higher dust contents than sections 3, 4, and 5. As will be shown later even greater increases were observed in sections 1 and 2 of track 2 in which sodium chloride was used as an admixture. There was nothing disclosed by the tests to explain these increases.

The effect of leaching on the chloride content is clearly shown in table 5. All sections except section 5 showed a decrease in the amount of the soluble salt present. Further decreases in chloride content were revealed by analyses made at the end of the test period. The retention of the admixture was greatest in section 1 which contained the clay-soil and decreased as the amount of material passing the No. 200 sieve decreased.

After the final application of water on the surfaces of the test sections, distributed traffic was continued to 151,200 wheel-trips with no water in the sub-base. During this period section 1, which had showed signs of surface rutting when saturated from the top, became stable again although the accumulated average vertical displacement had reached 0.24 inch before the surface treatment was applied. Sections 2 and 3 showed little movement and were not affected by the water applied to the surface. Sections 4 and 5 appeared to be benefited temporarily by the surface applications of water. Their surfaces became smooth and well bonded under the action of traffic. This improvement, although of very short duration, is shown by the temporary change in slope of their vertical displacement curves (fig. 2). As traffic was continued under drying conditions the previous tendency of these two sections to ravel reappeared. Figure 4 illustrates the appearance of typical sections of track 1 at 151,200 wheel-trips, or just before the bituminous surfaces were applied. The view of section 2 is representative of the condition of sections 1, 2, and 3. That of section 5 is representative of the condition of sections 4 and 5, and shows the decidedly loose and open-surface texture of these two sections.

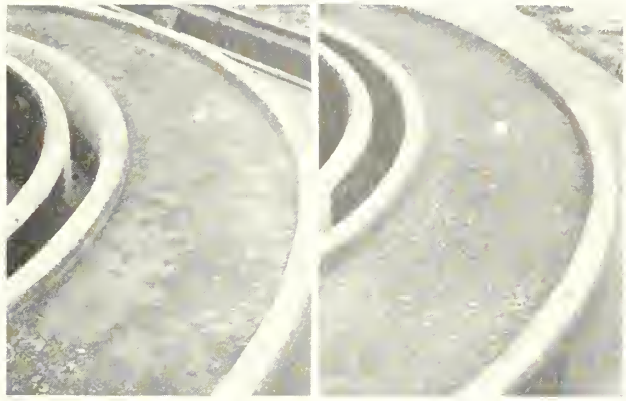

FIGURE 4.—TYPICAL SECTIONS OF TRACK 1 AT 151,200 WHEEL-TRIPS, JUST BEFORE CONSTRUCTION OF THE BITUMINOUS SURFACE. LEFT, SECTION 2, WHICH IS ALSO REPRESENTATIVE OF SECTIONS 1 AND 3; RIGHT, SECTION 5, WHICH IS ALSO REPRESENTATIVE OF SECTION 4.

TRAFFIC TESTS CONTINUED AFTER BITUMINOUS SURFACE APPLIED

As shown in figure 2, new initial or zero displacement readings were taken after the application and compaction of the bituminous surface and the record from that time on or from 171,200 wheel-trips to the end of the test indicates the behavior of the chemically treated materials when acting solely as base courses.

The materials in all sections of track 1 gave good service and showed little movement as base courses even under the very severe test conditions imposed by maintaining the water elevation at 2½ inches. At 261,200 wheel-trips, or 90,000 wheel-trips after the start of concentrated traffic and 60,000 wheel-trips after the water had been raised to the 2½-inch level, the average vertical displacement of the surface on all the sections was less than 0.05 inch and the maximum amount of rutting was 0.09 inch. It was not until the water had been raised to the 5-inch level, or to within 1 inch of the bituminous surfacing, that pronounced base movement was observed. Under this extreme condition and with increased wheel loads, section 5 had definitely failed at the end of the test, 298,500 wheel-trips. Section 4 exhibited considerable movement and the surface treatment between the wheel courses was cracked. The wheel tracks were visible on sections 1, 2, and 3, but there was little distortion of the surface treatment. The condition of the track at the end of the test is shown in figure 5. The final condition of sections 2 and 3 was similar to that of section 1.

Track 2: Sodium chloride admixture.—This track consisted of five mixtures similar to those tested in track 1.

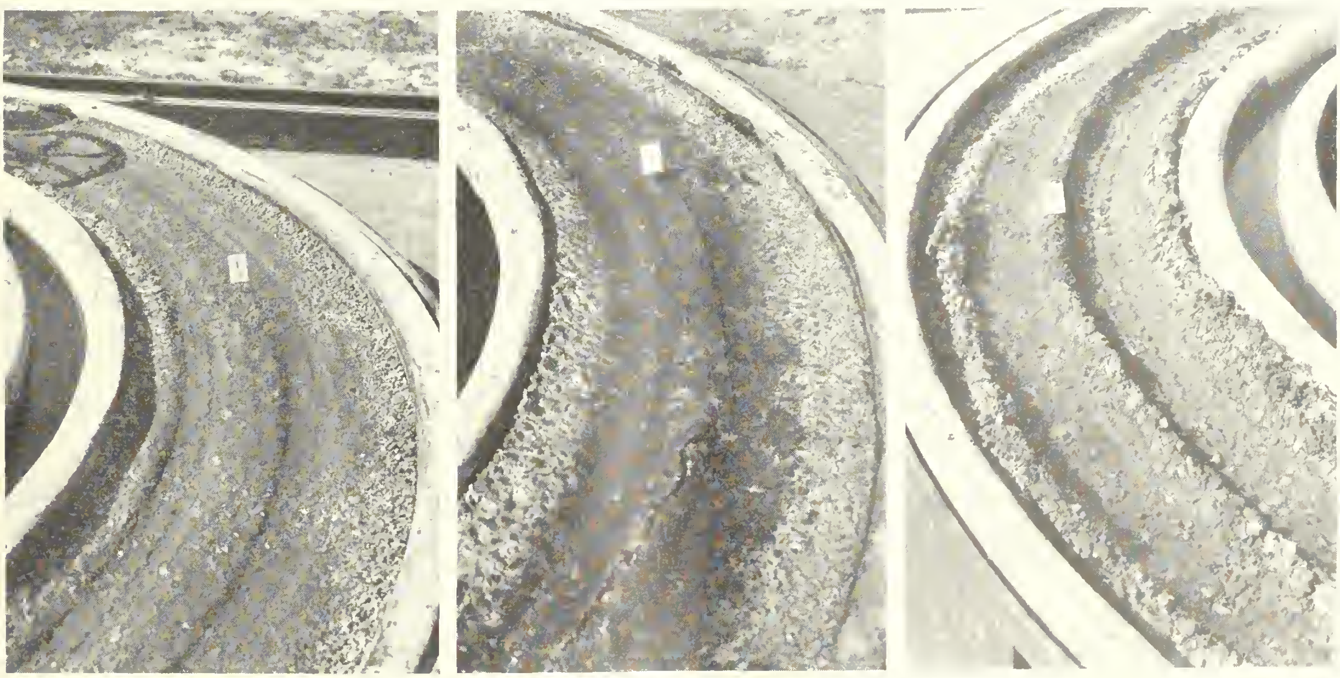

FIGURE 5.—SECTIONS OF TRACK 1 AT THE CONCLUSION OF THE TEST. LEFT, SECTION 1, WHICH IS ALSO REPRESENTATIVE OF SECTIONS 2 AND 3; MIDDLE, SECTION 4; RIGHT, SECTION 5.

The test schedule together with notations on the behavior of the five test sections are given in table 6. Figure 6 shows the results of the displacement measurements.

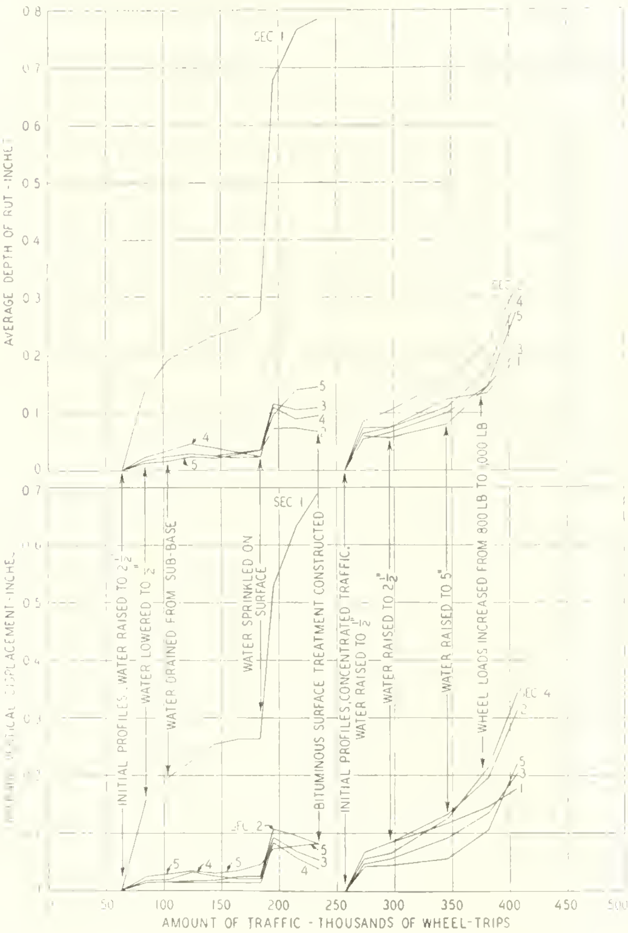

FIGURE 6.—SURFACE DISPLACEMENTS OF SECTIONS OF TRACK 2 AT VARIOUS STAGES OF THE TEST.

Raveling of the surface under distributed traffic was first noted at about 160,000 wheel-trips in section 5 and progressed gradually to 184,000 wheel-trips, when sprinkling was started. At this time sections 2, 3, and 4 had also started to ravel to some extent along the curb line. The condition of section 5 is illustrated in figure 7.

FIGURE 7.—SECTION 5 OF TRACK 2 AT 184,000 WHEEL-TRIPS, JUST BEFORE THE FIRST SPRINKLING. SOME RAVELING HAD DEVELOPED, PARTICULARLY ALONG THE EDGES.

The average vertical displacement of sections 2, 3, 4, and 5 was less than 0.05 inch and the amount of rutting was correspondingly low. Section 1 of track 2 failed to compact readily as was the case with the corresponding section in track 1. In track 2, this section finally became stable at about 84,000 wheel-trips although the rate of average vertical displacement continued to be much higher than in the other sections up to about 150,000 wheel-trips. Thereafter little additional movement was noted until water was applied to the surface.

Sprinkling was started at 184,000 wheel-trips and continued in a manner similar to that described for track 1. The water passed through the salt treated sections slowly. The first application was made on a Saturday and had all disappeared by the following Monday. The second application required about 24 hours to disappear from section 5 and between 32 and 48 hours to disappear from the other sections. Four days after the last application there was still some water remaining on sections 1 and 2 in the low spots.

The moisture content of each section as well as the sodium chloride content determined on that portion of

TABLE 6.—*Schedule of operations and behavior of test sections in track 2 with sodium chloride*

Operation	Traffic	Water level above top of sub-base	Behavior				
			Sec. 1	Sec. 2	Sec. 3	Sec. 4	Sec. 5
	Wheel-trips	*Inches*					
Placing and compacting	0 to 64,000	[1] 0	Unstable	Good	Good	Good	Good.
Testing with distributed traffic	64,000 to 84,000	2½	Slightly unstable	do	do	do	Do.
Do.	84,000 to 104,000	½	Slight pitting	do	do	do	Do.
Do.	104,000 to 184,000	[1] 0	Good	Slight raveling	Slight raveling	Slight raveling	Raveled.
Sprinkling and testing with distributed traffic.	184,000 to 234,300	[1] 0	Slightly unstable	Good	Good	Good	Good during sprinkling but raveled later.
Compacting bituminous surface treatment.	234,300 to 257,000	[1] 0	Good	do	do	do	Good.
Testing with concentrated traffic	257,000 to 297,000	½	do	do	do	do	Do.
Do.	297,000 to 347,000	2½	do	do	do	do	Do.
Do.	[2] 347,000 to 407,000	5	do	Slightly unstable	Slightly unstable	Slightly unstable	Slightly unstable.

[1] No water in sub-base.
[2] Wheel loads increased from 800 to 1,000 pounds at 375,000 wheel-trips.

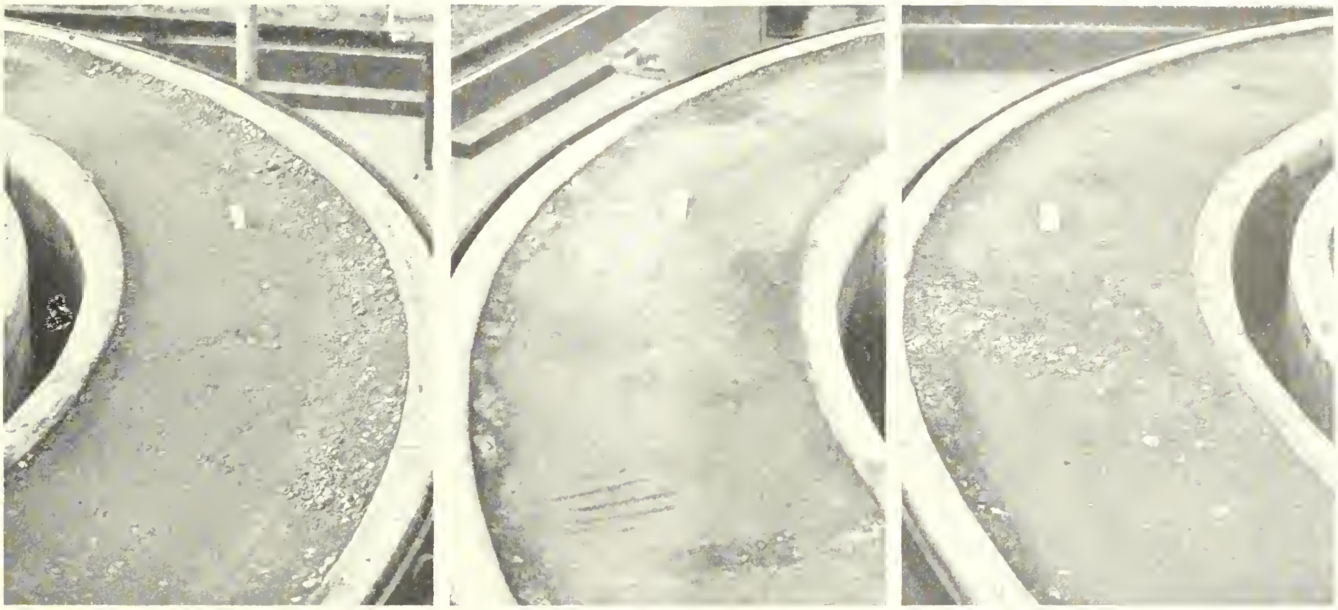

FIGURE 8.—SECTIONS OF TRACK 2 AT 234,300 WHEEL-TRIPS, JUST BEFORE CONSTRUCTION OF THE BITUMINOUS SURFACE. LEFT, SECTION 1; MIDDLE, SECTION 3, WHICH IS ALSO REPRESENTATIVE OF SECTIONS 2 AND 4; RIGHT SECTION 5.

the material passing the No. 10 sieve is shown in table 7 for various time during the testing period. The leaching effect is clearly illustrated in this table, being most pronounced in the sections with the lowest dust contents.

The sodium chloride contents of samples taken from sections 1, 2, and 5, were much greater at 184,000 wheel-trips than at the start of the track test. Section 3 showed a slight increase and section 4 a slight decrease

TABLE 7.—*Moisture contents and sodium chloride contents in track 2 at several stages of the test*

Section No.	Number of wheel-trips	Stage of test	Moisture content based on dry weight	Sodium chloride content of portion passing No. 10 sieve
			Percent	*Percent*
1	1,600	Start	7.1	0.24
	184,000	Before sprinkling	3.6	1.29
	196,000	After sprinkling	5.3	.21
	407,000	After testing with bituminous surface.	5.1	.16
2	1,600	Start	6.4	.31
	184,000	Before sprinkling	3.5	1.49
	196,000	After sprinkling	4.6	.11
	407,000	After testing with bituminous surface.	5.5	.07
3	1,600	Start	6.6	.23
	184,000	Before sprinkling	2.7	.35
	196,000	After sprinkling	3.9	.06
	407,000	After testing with bituminous surface.	4.3	.03
4	1,600	Start	5.4	.27
	184,000	Before sprinkling	2.6	.19
	196,000	After sprinkling	4.4	.17
	407,000	After testing with bituminous surface.	4.7	.03
5	1,600	Start	4.3	.18
	184,000	Before sprinkling	2.3	.43
	196,000	After sprinkling	3.3	.04
	407,000	After testing with bituminous surface.	5.1	.02

Distributed traffic was continued after the final application of water on the surface up to 234,300 wheel-trips. All sections showed a marked increase in the rate of vertical displacement after the application of water. Section 1 softened on the surface but did not become unstable throughout its entire depth. The excessive displacements measured on section 1 (see fig. 6) may be explained by the fact that the softened surface crust picked up under the wheels and was either deposited on other sections or thrown off the track.

The photograph of section 1, figure 8, taken at 234,300 wheel-trips, shows this condition. It can be seen that the surface is definitely lower than that of the adjoining section shown in the background although there are no indications of rutting.

Sections 2, 3, and 4 showed an increase in vertical displacement during the sprinkling operations but bonded firmly under distributed traffic and actually became smoother as the test progressed up to 234,300 wheel-trips or the end of this phase of the test as illustrated by the view of section 3 in figure 8.

Section 5 continued to show increasing amounts of vertical displacement both during and after the sprinkling operation and while this section was not loose during the time water was being applied, evidence of raveling was noted as drying started soon after the last application. This section is also shown in figure 8.

ALL MIXTURES IN TRACK 2 PROVED SATISFACTORY AS BASE COURSES

A bituminous surface treatment was applied to track 2 at 234,300 wheel-trips. All the mixtures proved satisfactory as base courses when treated with sodium chloride as they did in track 1 when treated with calcium chloride. Again it was necessary to raise the water table to the 5-inch level and increase the wheel loads to 1,000 pounds before definite indications of failure could be produced. The average vertical displacements and rutting (see fig. 6) varied from 0.04 to 0.09 inch for all sections between the time concentrated traffic was started at 257,000 wheel-trips and the time the second set of profiles was taken at 274,000 wheel-trips. Most of this displacement resulted from incomplete initial compaction of the surface treatment which was constructed in cold weather. Even with this displacement, which cannot be attributed to movement in the base, neither the average vertical displacements nor

FIGURE 9.—SECTIONS OF TRACK 2 AT THE CONCLUSION OF THE TEST. LEFT, SECTION 1; MIDDLE, SECTION 3, WHICH IS ALSO REPRESENTATIVE OF SECTIONS 2 AND 4; RIGHT, SECTION 5.

the average depth of ruts exceeded 0.25 inch for any of the sections until near the end of the test.

When the test was concluded at 407,000 wheel-trips, section 1 was in fairly good condition except for the superficial rutting caused by poor compaction of the surface treatment (fig. 9), and showed the least amount of displacement. Profilometer measurements indicated the greatest amounts of movement to have occurred in sections 2 and 4. The appearance of these two sections at the end of test was very similar to that of section 3, shown in figure 9. The surface treatment on all three of these sections had cracked between the wheel courses. Section 5 was showing signs of failure at the end of the test although the total vertical displacement was not as great as for some of the other sections. The surface treatment was breaking and the section was becoming rough generally as shown in figure 9.

Track 3: Without chemical admixture.—Five mixtures similar in composition to those placed in tracks 1 and 2 were tested in track 3 without the admixture of a water-retentive chemical.

The schedule of testing operations and observations on the behavior of the five sections of track 3 are given in table 8. Figure 10 shows the average vertical displacement and the amount of rutting.

In general, the behavior of the five materials without chemical admixture was conspicuously different from that of the corresponding sections of tracks 1 and 2 prior to the application of the surface treatment. Section 1 failed to compact well, as did the same section in the two previous tracks, showing considerable movement throughout the 60,000 wheel-trips of compacting traffic. It differed widely from the others, however, during the initial flooding of the sub-base from 60,000 to 100,000 wheel-trips. (See table 8.) The surface became dry and dusty, indicating that evaporation was proceeding at a faster rate than the water could be brought up through the material by capillarity. No such behavior was observed in tracks 1 and 2 where water-retentive chemicals were used as admixtures.

Raveling in section 1 began shortly after 80,000 wheel-trips when the water was dropped to one-half inch above the bottom of the test course. Shortly before the sub-base was drained at 100,000 wheel-trips, sections 2 and 3 also started to ravel in the order named. The surfaces of all three sections were dry at this time in contrast to the surfaces of sections 4 and 5 which appeared damp and well bonded.

SPRINKLING AIDED IN SURFACE MAINTENANCE OF GRANULAR MIXTURES

Upon the complete withdrawal of water from the sub-base, sections 4 and 5 also started to ravel. The condition of representative sections at 160,000 wheel-trips just prior to sprinkling is illustrated by figure 11. Section 1 is representative of the condition of both sections 1 and 2. Section 3 shown at the bottom of figure 11 was intermediate and sections 4 and 5 were in slightly better condition than section 3.

TABLE 8.—*Schedule of operations and behavior of test sections in track 3 without chemical admixtures*

Operation	Traffic	Water level above top of sub-base	Behavior				
			Sec. 1	Sec. 2	Sec. 3	Sec. 4	Sec. 5
	Wheel-trips	*Inches*					
Placing and compacting	0 to 60,000	[1] 0	Unstable	Slightly unstable	Good	Good	Good.
Testing with distributed traffic	60,000 to 80,000	2½	Dusty	Good	do	do	Do.
Do	80,000 to 100,000	½	Raveled	Raveled	Slight raveling	do	Do.
Do	100,000 to 160,000	[2] 0	do.[3]	do.[3]	Raveled [3]	Raveled [3]	Raveled.[3]
Sprinkling and testing with distributed traffic.	160,000 to 180,500	[2] 0	Good	Good	Good	Good	Good.
Compacting bituminous surface treatment	180,500 to 200,500	[2] 0	do	do	do	do	Do.
Testing with concentrated traffic	200,500 to 240,000	½	do	do	do	do	Do.
Do	240,000 to 260,000	2½	do	do	Slightly unstable	do	Do.
Do	[4] 260,000 to 300,000	5	do	do	Unstable	Unstable	Slightly unstable.

[1] No water in sub-base. Water admitted to sub-base at 10,000 wheel-trips for 400 wheel-trips, then drained.
[2] No water in sub-base.
[3] Raveling was progressive from secs. 1 to 5.
[4] Wheel loads increased from 800 to 1,000 pounds, at 290,000 wheel-trips

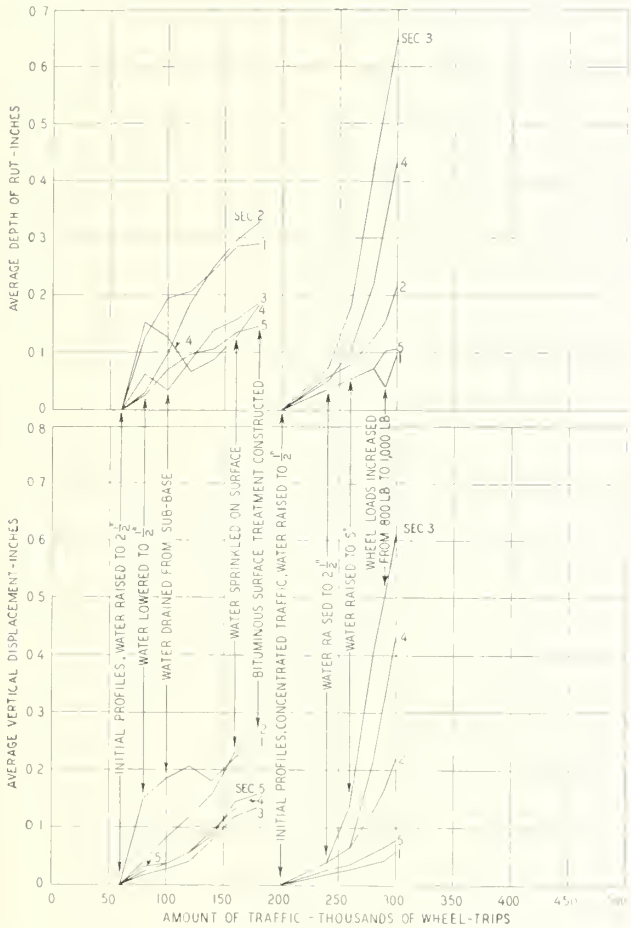

FIGURE 10.—SURFACE DISPLACEMENTS OF SECTIONS OF TRACK 3 AT VARIOUS STAGES OF THE TEST.

Sprinkling was started at 160,000 wheel-trips and continued in a manner similar to that described for tracks 1 and 2. The sections transmitted the water much more readily than did the corresponding sections treated with water-retentive chemicals.

All sections in track 3 were benefited by the application of water to the surface. Although the vertical displacements continued to increase (fig. 10) the surfaces became firm and the aggregates were well bonded under the action of traffic. Figure 12 shows the condition of sections 1 and 3 just prior to the construction of the bituminous surface at 180,500 wheel-trips. Comparison of the sections at this time with their condition as shown in figure 11 clearly illustrates the beneficial effect of the surface water.

A bituminous surface treatment was applied to track 3 at 180,500 wheel-trips. All five materials proved satisfactory as base courses without chemicals. The average vertical displacements and amounts of rutting (see fig. 10) indicated that detrimental movements were not produced until the water had been raised to the 5-inch level and the wheel loads increased to 1,000 pounds.

Sections 1 and 5 exhibited the least amount of movement when tested as base courses. They remained in excellent condition throughout this phase as illustrated in figure 13.

Section 2 moved more than sections 1 and 5 but was still in good condition at the end of the test. Some cracking of the surface treatment between the wheel courses was observed. The condition of these three sections was similar and is illustrated by the view of section 5, figure 13. Sections 3 and 4 showed sufficient rutting at the end of the test to indicate failure. However, this condition was produced only after unreasonably severe test conditions had been imposed. Section 3 in figure 13 is representative of the condition of both sections 3 and 4 at the conclusion of the test.

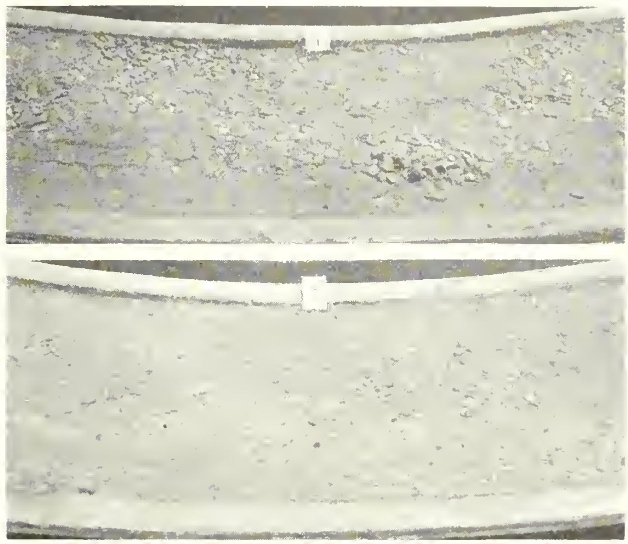

FIGURE 11.—SECTIONS OF TRACK 3 AT 160,000 WHEEL-TRIPS, JUST BEFORE THE FIRST SPRINKLING. UPPER, SECTION 1, WHICH IS ALSO REPRESENTATIVE OF SECTION 2; LOWER, SECTION 3. SECTIONS 4 AND 5 WERE IN SLIGHTLY BETTER CONDITION THAN SECTION 3.

FIGURE 12.—SECTIONS OF TRACK 3 AT 180,500 WHEEL-TRIPS, SOON AFTER SPRINKLING WAS DISCONTINUED. UPPER, SECTION 1, WHICH IS ALSO REPRESENTATIVE OF SECTION 2; LOWER, SECTION 3, WHICH IS ALSO REPRESENTATIVE OF SECTIONS 4 AND 5.

Track 4: Crusher-run materials.—The five sections of track 4 were constructed of three types of crusher-run materials. Sections 1, 2, and 3 consisted of limestone, granite and slag materials, respectively, as obtained from commercial sources. Section 4 was a

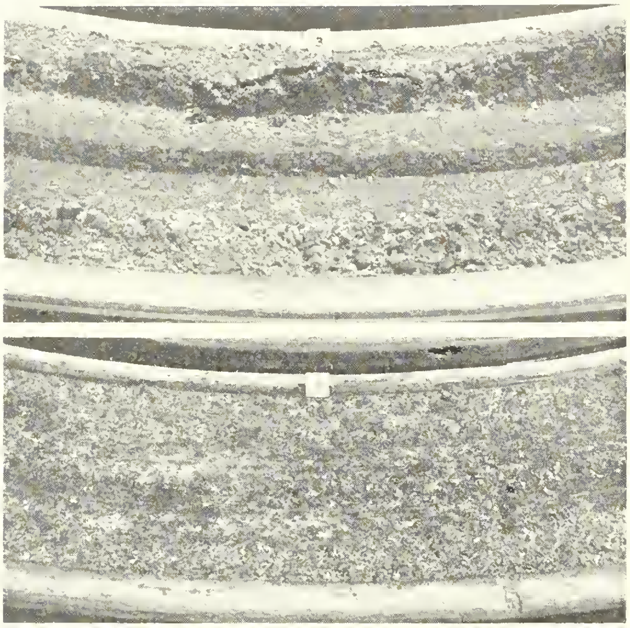

FIGURE 13.—SECTIONS OF TRACK 3 AT THE CONCLUSION OF THE TEST. UPPER, SECTION 3, WHICH IS ALSO REPRESENTATIVE OF SECTION 4; LOWER, SECTION 5, WHICH IS ALSO REPRESENTATIVE OF SECTIONS 1 AND 2.

mixture of 90 percent granite and 10 percent slag, and section 5 was a mixture of 90 percent granite and 10 percent limestone. The sections were constructed by dampening and compacting the materials without chemical admixtures.

After the initial compaction period (82,600 wheel-trips) the test was carried out in three distinct steps as shown in table 9.

1. The water level was raised to 2½ inches and distributed test traffic was applied from 82,600 to 182,600 wheel-trips while the water was gradually lowered and finally drained out of the sub-base. Distributed traffic was then continued to 242,600 wheel-trips.

2. The water was again raised to 2½ inches, and a surface application of calcium chloride at the rate of 1½ pounds per square yard was made. Testing with distributed traffic was then resumed while the water was again lowered and finally drained out at 308,800 wheel-trips. Distributed traffic was then continued to 366,000 wheel-trips.

3. A bituminous surface was constructed and concentrated traffic was applied while the water level was

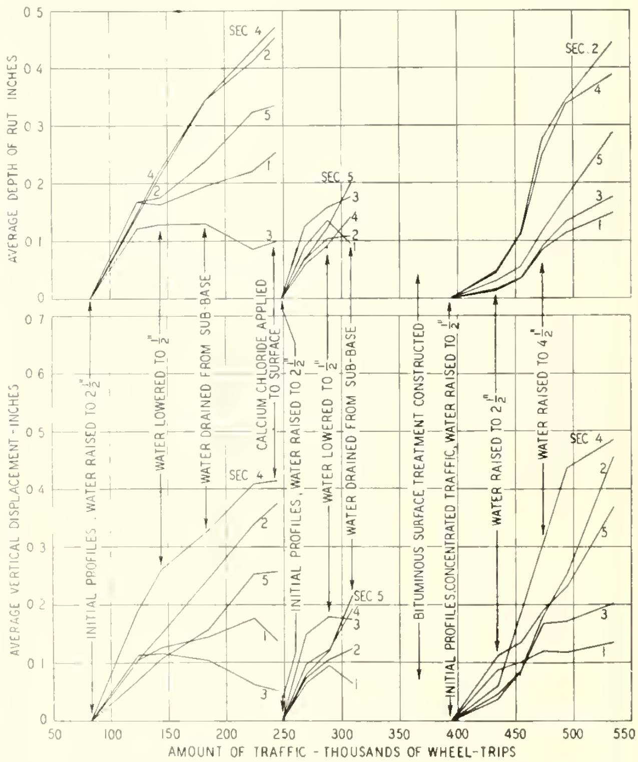

FIGURE 14.—SURFACE DISPLACEMENTS OF SECTIONS OF TRACK 4 AT VARIOUS STAGES OF THE TEST.

gradually increased to a maximum of 4½ inches at 474,300 wheel-trips. An additional 60,000 wheel-trips of concentrated traffic was applied with the water remaining at the 4½-inch elevation.

Sections 1 and 3 compacted well and showed no signs of raveling until the water had been completely withdrawn from the sub-base at 182,600 wheel-trips. Sections 2, 4, and 5 on the other hand did not bond or set up well. The surfaces of these sections became loose and dusty even with the water 2½ inches above the bottom of the test course.

Figure 14 shows the amounts of rutting and the average vertical displacements as measured by the profilometers. Both instruments indicated the greatest amount of movement up to 242,600 wheel-trips in sections 2 and 4 and the least movement in section 3. Section 1, figure 15, is representative of both sections 1 and 3. Slight raveling along the curbs was observed as well as

TABLE 9.—*Schedule of operations and behavior of test sections in track 4*

Operation	Traffic	Water level above top of sub-base	Behavior				
			Sec. 1	Sec. 2	Sec. 3	Sec. 4	Sec. 5
	Wheel-trips	*Inches*					
Placing and compacting	0 to 82,600	[1] 0	Good	Slightly unstable	Good	Slightly unstable	Unstable.
Testing with distributed traffic	82,600 to 142,600	2½	do	Raveled	do	Raveled	Raveled.
Do	142,600 to 182,600	½	do	do	do	do	Do.
Do	182,600 to 242,600	[1] 0	Slight raveling	do	Slight raveling	do	Do.
Applying calcium chloride and compacting treated surface.[2]	242,600 to 248,800	[1] 0	Good	Good	Good	Slightly unstable	Slightly unstable.
Testing with distributed traffic	248,800 to 288,800	2½	do	Slightly unstable	do	do	Unstable.
Do	[3] 288,800 to 308,800	½	do	Unstable	do	Unstable	Do.
Do	308,800 to 366,000	[1] 0	do	Slightly unstable	do	Slightly unstable	Do.
Compacting bituminous surface treatment.	366,000 to 394,300	[1] 0	do	Good	do	Good	Good.
Testing with concentrated traffic	394,300 to 434,300	½	do	do	do	do	Do.
Do	434,300 to 474,300	2½	do	Unstable	do	Unstable	Slightly unstable.
Do	474,300 to 534,300	4½	do	do	do	do	Unstable

[1] No water in sub-base.
[2] Sections scarified, sprinkled, compacted lightly, and treated with a surface application of 1½ pounds of calcium chloride per square yard.
[3] Section 5 scarified at 292,200 wheel-trips. Secs. 2, 4, and 5 scarified at 308,800 wheel-trips.

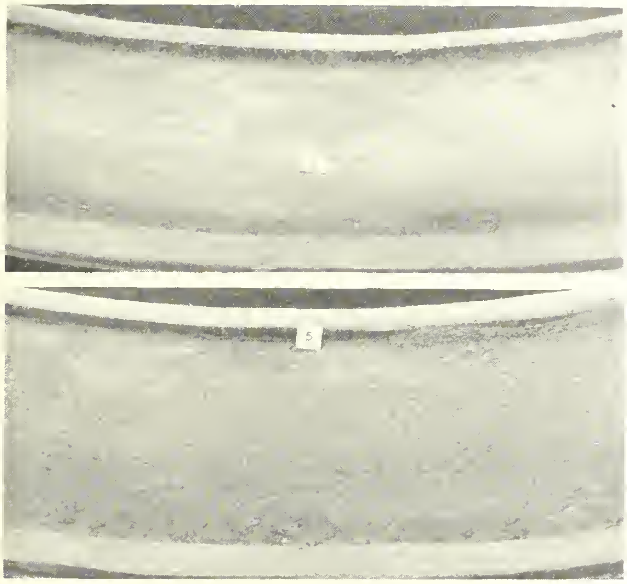

FIGURE 15.—SECTIONS OF TRACK 4 AT 242,600 WHEEL-TRIPS, JUST BEFORE APPLICATION OF CALCIUM CHLORIDE. UPPER, SECTION 1, WHICH IS ALSO REPRESENTATIVE OF SECTION 3; LOWER, SECTION 5, WHICH IS ALSO REPRESENTATIVE OF SECTIONS 2 AND 4.

some wear on the surface. The appearance of section 5, also shown in figure 15, is typical of sections 2, 4, and 5, at 242,600 wheel-trips. The surfaces were loose and unbonded and were wearing badly.

At 242,600 wheel-trips, the sections were scarified lightly and sprinkled. The water level was raised to 2½ inches and calcium chloride was applied uniformly to the surface. Traffic was started on the following day after all calcium chloride had disappeared from the surface.

No dusting or raveling was observed on any of the sections throughout the test period from the time calcium chloride was applied until the bituminous surface treatment was constructed.

The limestone and slag in sections 1 and 3, respectively, remained in good condition during this phase of the test as illustrated in figure 16. The other sections, which were constructed with granite as the predominating constituent, exhibited a marked movement of the surface. This was distinct from the raveling noted earlier in the tests and consisted of shoving and displacement in the direction of traffic. This is well illustrated in figure 16, which shows section 5. The condition described became so bad that it was necessary to scarify and reshape section 5 at 292,200 wheel-trips and sections 2, 4, and 5 at 308,800 wheel-trips.

At 366,000 wheel-trips, the sections were reshaped and compacted and the bituminous surface treatment was applied. Water was brought in contact with the base course and testing with concentrated traffic started at 394,300 wheel-trips.

Sections 1 and 3 remained in good condition throughout the test period. At the end of the test sections 2, 4, and 5, had definitely failed. The displacements for these latter sections were in excess of 0.25 inch and all three sections showed considerable movement under individual wheel-trips. As shown in figure 14 the displacement curves for these three materials rose continuously throughout the test. The displacement curves for sections 1 and 3 on the other hand flattened

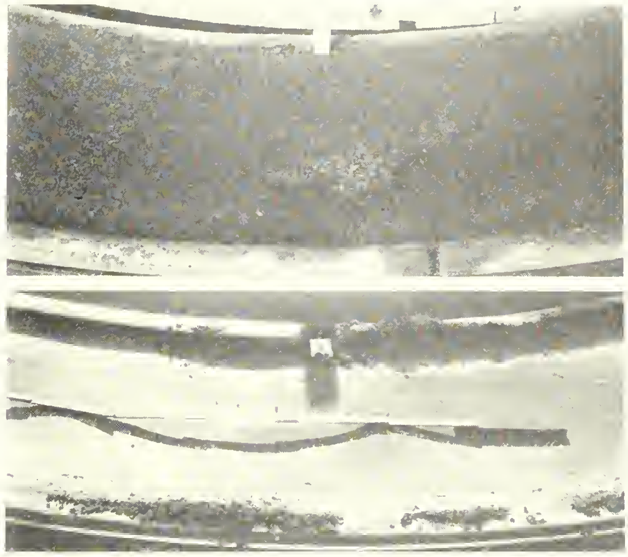

FIGURE 16.—SECTIONS OF TRACK 4 AT 366,000 WHEEL-TRIPS, JUST BEFORE CONSTRUCTION OF THE BITUMINOUS SURFACE. UPPER, SECTION 3, WHICH IS ALSO REPRESENTATIVE OF SECTION 1; LOWER, SECTION 5. SECTIONS 2 AND 4 WERE IN SOMEWHAT BETTER CONDITION THAN SECTION 5.

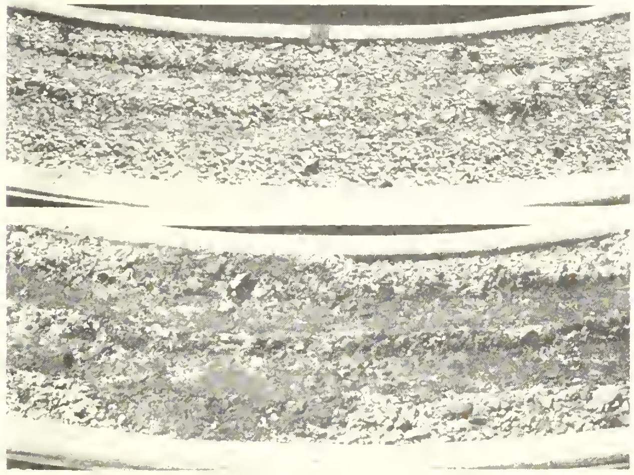

FIGURE 17.—SECTIONS OF TRACK 4 AT THE CONCLUSION OF THE TEST. UPPER, SECTION 1, WHICH IS ALSO REPRESENTATIVE OF SECTION 3; LOWER, SECTION 4, WHICH IS ALSO REPRESENTATIVE OF SECTIONS 2 AND 5.

even under the extremely severe test conditions and never exceeded 0.2 inch. While sections 2, 4, and 5, gave evidence of fairly satisfactory service with the water elevation at one-half inch they appeared definitely inferior to sections 1 and 3 even at this stage of the test.

Figure 17 illustrates the condition of representative sections of the track at the conclusion of the test.

SUMMARY

The test behavior of all the sections in tracks 1, 2, and 3, is correlated in table 10.

Performance as surfaces.—The grading curves for the 5 materials tested in tracks 1, 2, and 3 are shown in figure 18. The shaded band in this figure is drawn to include the A. A. S. H. O. specification requirements for coarse-graded, aggregate-type surfacing materials. These specifications stipulate that the fraction passing the No. 40 sieve shall have a liquid limit not greater than 35 and a plasticity index not less than 4 nor more

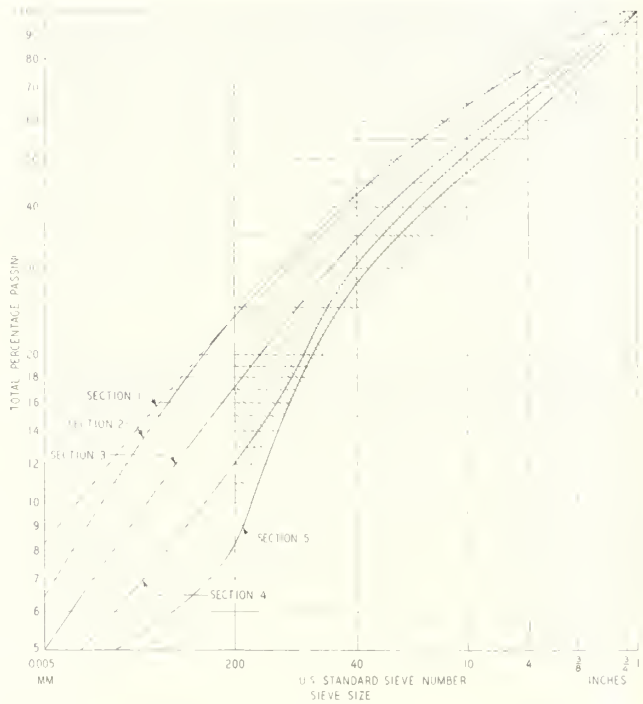

Figure 18.—Gradings of Materials in Tracks 1, 2, and 3. Shaded Area Indicates Zone Within Which Are Included the Specification Requirements of the A. A. S. H. O. for Type "B" Material for Stabilized Surface Course. Each Grading Curve Represents the Average Grading of the 3 Sections Having the Same Number Designation.

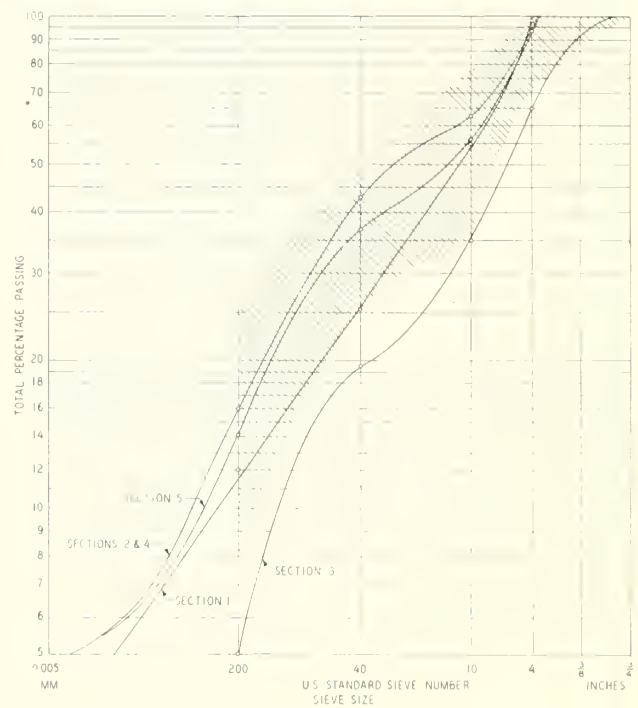

Figure 19.—Gradings of Materials in Track 4. Shaded Area Indicates Zone Within Which Are Included the Specification Requirements of the A. A. S. H. O. for Type "C" Material for Stabilized Surface Course.

than 9. The maximum plasticity index of any of the mixtures tested was 3 so that while all the mixtures except section 5 conform to the specifications in grading none of them has a plasticity index high enough to meet the specification requirements.

The tests with distributed traffic prior to surface treatment on track 3 without chemical admixture showed that these materials all raveled badly unless they were kept damp by capillary moisture from the ground water table or by water sprinkled on the surface. With decreasing ground water elevation, sections 1 and 2 with the greatest amount of material passing the No. 200 sieve raveled first. Further lowering of the ground water level produced raveling successively in sections 3, 4, and 5, which had dust ratios respectively of 48, 40, and 31. (See table 1.)

Table 10.—*Correlation of test behavior of the sections in tracks 1, 2, and 3*

Track No.	Admixture	Sec. No.	Behavior under traffic: Without bituminous surface: Compacting without water in sub-base	Without bituminous surface: Water level $2\frac{1}{2}$ inches	Without bituminous surface: Water level $\frac{1}{2}$ inch	Without bituminous surface: No water in sub-base just before sprinkling	Without bituminous surface: After sprinkling and draining	With bituminous surface: Water level $\frac{1}{2}$ inch	With bituminous surface: Water level $2\frac{1}{2}$ inches	With bituminous surface: Water level 5 inches
1	Calcium chloride	1	Unstable	Slightly unstable	Good	Good	Slightly unstable.	Good	Good	Good
2	Sodium chloride	1	do	do	Slight pitting	do	do	do	do	Do.
3	None	1	do	Dusty	Raveled	Raveled	Good [1]	do	do	Do
1	Calcium chloride	2	Good	Good	Good	Good	do	do	do	Do.
2	Sodium chloride	2	do	do	do	Slight raveling	do	do	do	Slightly unstable.
3	None	2	Slightly unstable.	do	Raveled	Raveled	do.[1]	do	do	Good.
1	Calcium chloride	3	Good	do	Good	Good	do	do	do	Do.
2	Sodium chloride	3	do	do	do	Slight raveling	do	do	do	Slightly unstable.
3	None	3	do	do	Slight raveling	Raveled	do.[1]	do	Slightly unstable.	Unstable.
1	Calcium chloride	4	do	do	Good	Slight raveling	Slight raveling	do	Good	Slightly unstable.
2	Sodium chloride	4	do	do	do	do	Good	do	do	Do.
3	None	4	do	do	do	Raveled	do.[1]	do	do	Unstable.
1	Calcium chloride	5	do	Slight raveling	Raveled	do	Raveled	do	do	Do.
2	Sodium chloride	5	do	Good	Good	do	Slight raveling	do	do	Slightly unstable.
3	None	5	do	do	do	do	Good [1]	do	do	Do.

[1] On track 3 traffic was discontinued 20,000 wheel-trips after sprinkling while the sections were still in good condition. Tests prior to sprinkling had indicated that 60,000 wheel-trips with water withdrawn from the sub-base would produce raveling in all sections.

The grading curves for the 5 materials tested in track 4 are shown in figure 19. The shaded band represents the A. A. S. H. O. specification limits for crusher-run surfacing materials. The slag tested in section 3 is coarser than provided for by the specifications. All other materials conform to the specification requirements. Sections 1 and 3, consisting of limestone and slag materials, were satisfactory throughout the tests and were definitely superior to sections 2, 4, and 5 which consisted of granite or largely of granite. The limestone and slag were naturally cementitious and bonded well in the test, whereas the pure granite which was used in section 2 failed to bond and was unstable under traffic. Admixtures of limestone or slag in the amount of 10 percent failed to improve to any appreciable extent the behavior of the crusher-run granite used in this investigation.

Performance as base courses.—The five materials tested in tracks 1, 2, and 3 gave good service as base courses except under the most severe testing conditions. The materials in sections 1, 2, and 3 were finer than the A. A. S. H. O. specification for base courses. The materials in sections 4 and 5, while conforming essentially to the specification, approached its fine limit. Previous investigations had shown that concentrated traffic, with the ground water elevation one-half inch above the bottom of the base course, provides a condition which is sufficiently severe to identify the definitely unsatisfactory materials. In these tests traffic was continued with increased wheel loading after the water had been raised to 5 inches above the bottom of the base course before evidences of failure were produced in tracks 1, 2, and 3.

At the conclusion of these very severe tests the following sections in tracks 1, 2, and 3 were in comparatively poor condition:

Track 1—sections 4 and 5.
Track 2—sections 2, 3, 4, and 5.
Track 3—sections 3 and 4.

In general, mixtures which had from 20 to 25 percent of material passing the No. 200 sieve proved more stable than those having lower dust contents. However, previous investigations [1] have shown that if the fines were plastic this amount of fine material would be detrimental.

[1] See footnote 1, p. 173.

The limestone and slag sections in track 4 gave good service as base courses under all conditions of the test. The granite sections exhibited increasing amounts of movement under traffic with the water one-half inch above the bottom of the base and failed under the severe conditions imposed toward the conclusion of the test.

Densities measured at the conclusion of the test on each track are shown in table 11. Densities obtained in the Proctor or A. A. S. H. O. standard compaction test are also shown in this table. The compaction tests were run on the soil mortar, or that fraction of the material passing the No. 10 sieve. The values shown in table 11 for tracks 1, 2, and 3, are corrected for the material retained on the No. 10 sieve.

With few exceptions, the densities measured in the track were less than the maximum densities computed from the Proctor compaction test. Section 1, which failed to compact readily early in the test in all three tracks, ultimately reached the highest density. Sections 4 and 5 which set up well initially, had densities considerably lower than the other sections in all tracks.

The densities attained in the track by the five crusher-run materials as compared with densities obtained in the vibratory compaction test (see table 12) gave no indication as to their suitability. Their behavior depended on other characteristics.

Effect of chemical treatments. The effect of the chemical admixtures on the compactibility of the graded materials is shown by the behavior of the test sections during the initial compaction period. Track 1 which contained calcium chloride reached a condition considered suitable for starting the test at somewhat less than one-third the wheel-trips required to produce a similar condition in tracks 2 and 3.

Testing with distributed traffic prior to the construction of the bituminous surface treatment produced less raveling in sections 1, 2, and 3 in both tracks 1 and 2 in which a chemical admixture was used than in the corresponding sections of track 3 which contained no chemical. Section 4 of the chemically treated tracks

TABLE 11.—*Moisture content and density of laboratory compacted aggregates and of circular track sections at conclusion of traffic test*

Track No.	Admixture	Sec. No.	Compacted by Proctor method [1]: Water content based on dry weight	Compacted by Proctor method: Composition by volume: Water	Compacted by Proctor method: Composition by volume: Aggregate	Compacted by Proctor method: Composition by volume: Air voids	Samples cut from track at end of test: Water content based on dry weight	Samples cut from track at end of test: Composition by volume: Water	Samples cut from track at end of test: Composition by volume: Aggregate	Samples cut from track at end of test: Composition by volume: Air voids
			Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1	Calcium chloride	1	6.1	13.9	86.1	0	5.6	12.8	86.0	1.2
		2	6.8	15.1	84.0	.9	5.3	11.8	84.3	3.9
		3	4.9	11.3	87.4	1.3	4.6	10.5	85.8	3.7
		4	5.4	12.4	86.6	1.0	5.7	12.4	82.0	5.6
		5	4.7	10.8	86.5	2.7	5.9	12.8	81.7	5.5
2	Sodium chloride	1	6.5	14.7	85.3	0	5.1	11.8	87.3	.9
		2	6.1	13.7	84.9	1.4	5.5	12.4	84.9	2.7
		3	5.3	12.2	87.0	.8	4.3	9.8	86.3	3.9
		4	4.5	10.5	88.4	1.1	4.7	10.3	82.8	6.9
		5	4.0	9.4	88.1	2.2	5.1	10.9	80.6	8.5
3	None	1	6.6	14.8	84.9	.3	5.3	12.0	85.5	2.5
		2	6.1	13.9	86.0	.1	5.2	11.6	84.2	4.2
		3	4.7	11.1	88.8	.1	4.8	10.5	82.7	6.8
		4	4.9	11.5	88.5	0	4.9	10.7	82.6	6.7
		5	4.2	9.7	87.6	2.7	5.2	11.1	80.6	8.3
4	Calcium chloride [2]	1					5.4	11.6	79.4	9.0
		2					8.1	16.9	79.1	4.0
		3					9.2	19.7	79.7	.6
		4					6.7	14.3	80.9	4.8
		5					7.1	15.3	81.3	3.4

[1] Compaction test made on portion passing No. 10 sieve and moisture contents and densities calculated for total mixture containing the coarse fraction.
[2] Surface application.

was only slightly better and section 5 no better than the corresponding sections of track 3. Sections 1 and 2 had the highest and section 5 the lowest dust contents.

In track 1, sections 4 and 5, which displayed the greatest amount of raveling, had calcium chloride contents of 0.06 percent when sampled at 118,200 wheel-trips or just before sprinkling. At the corresponding period of test on track 2, 184,000 wheel-trips, the sodium chloride content of section 4 was 0.19 percent and of section 5 was 0.43 percent. (See table 7.)

TABLE 12.—*Densities of crusher-run materials in track compared to densities obtained by vibration*

Sec. No.	Density in track	Density obtained by vibration
	Percent	*Percent*
1	79.4	84.0
2	79.1	79.2
3	79.7	77.9
4	80.9	79.4
5	81.3	80.1

The appearance just before sprinkling of section 5 in the two tracks containing admixtures is shown in figures 3 and 7, respectively. At the corresponding stage of the test, the condition of section 5 in track 3, which contained no admixture, was very similar to that of section 5 in track 1.

While water applied to the surface benefited all sections of track 3, it made section 1 of both the calcium chloride and sodium chloride treated tracks less stable. This loss of stability did not however, extend deeply into the course but was confined to the top inch.

The surface sprinkling failed to improve except temporarily the surface condition of the remaining sections of track 1, but had no detrimental effect on their stability. Aside from its detrimental effect on the surface of section 1, the sprinkling caused an improvement of considerable duration in track 2, which contained the sodium chloride (figs. 7 and 8). A shorter period of drying and less traffic were required to cause raveling to start again in both tracks after leaching than before.

In section 5 of track 1, the amount of raveling caused by only 25,000 wheel-trips subsequent to the surface application of water was decidedly greater than that produced by the 60,000 wheel-trips immediately preceding the sprinkling (figs. 3 and 4). Similarly, in section 5 of track 2, the 40,000 wheel-trips applied after sprinkling and prior to the construction of the bituminous surface treatment had a more detrimental effect than the 80,000 wheel-trips immediately preceding the first application of surface water (figs. 7 and 8).

The chloride content of all sections was reduced by the leaching action of the water sprinkled on the surface as indicated in tables 5 and 7. The calcium chloride content of the sections of track 1 varied from 0.05 percent for section 4 to 0.32 percent for section 1 after the leaching test. In track 2 the sodium chloride content varied from 0.04 percent for section 5 to 0.21 percent for section 1 after leaching.

Determinations at the conclusion of the track tests showed that, with the exception of section 3, the densities of corresponding sections in tracks 1, 2, and 3 were quite similar. In general the sections containing chemicals were slightly denser than the corresponding untreated sections and the densities were roughly proportional to the amount of material passing the No. 200 sieve. The greatest difference was in section 3. In tracks 1 and 2 the final densities of this section were 85.8 and 86.3 percent, respectively, as compared to 82.7 percent where no admixture was used.

CONCLUSIONS

The following conclusions appear to be justified, for the sections considered as surface courses:

1. Nonplastic granular mixtures (tracks 1, 2, and 3) which have the grading requirements of the A. A. S. H. O. specifications for surfacing materials but lower plasticity indexes should give excellent service without chemical admixture when kept damp by capillary moisture or by water sprinkled on the surface. In permanently wet areas, therefore, it appears desirable to waive the minimum plasticity index requirement of 4 as required by the A. A. S. H. O. specification for surface courses, provided the nonplastic materials so admitted have dust ratios of 40 percent or less.

2. It was indicated that in dry locations and without chemical treatment the materials used in tracks 1, 2, and 3 would be subject to raveling and dusting if used as surfaces.

3. Crusher-run limestone and slag were satisfactory as surfacing courses under wet conditions but became dusty under dry conditions. The particular granite used in this investigation was not satisfactory as surfacing because it failed to bond or set up and because it shoved badly when wet.

4. Chemical treatments proved beneficial in the construction of bases for bituminous surfaces. The admixture of calcium chloride expedited compaction. Both calcium chloride and sodium chloride reduced raveling while the base courses were carrying traffic prior to construction of the bituminous wearing course. These results were obtained under conditions of high relative humidity.

5. The presence of 15 to 25 percent of material passing the No. 200 sieve is necessary to prevent the loss of a large part of the water-retentive chemicals when water falls on the surface and percolates through the mixture.

6. A surface application of calcium chloride was effective in reducing dusting and preventing raveling on all five sections in track 4. However, the moisture held near the surface of sections 2, 4, and 5 by the calcium chloride promoted the formation of corrugations to a detrimental extent.

For the sections considered as base courses, the following conclusions appear to be justified:

7. All materials tested in tracks 1, 2, and 3 both with and without chemical admixtures, gave excellent service as base courses except under moisture conditions much more severe than could reasonably be expected in service. It is believed therefore that existing surfaces which meet the A. A. S. H. O. surface course specifications for grading but which are nonplastic in character may be surface treated without altering their composition.

8. The limestone and slag sections of track 4 gave excellent results when tested as bases for bituminous surfacing under all conditions of moisture. Sections 2, 4, and 5, in which the crusher-run granite was the predominating constituent, were inferior to sections 1 and 3 but gave satisfactory service except under unreasonably severe test conditions.

9. Considerable latitude in grading requirements can be permitted when materials such as crusher-run limestone or slag are used for base courses. The natural cementing properties of these materials assist greatly in the formation of stable bases even when the grading is definitely coarser than would be allowed by the present A. A. S. H. O. specifications.

10. Materials that gave trouble during the early compaction period ultimately attained the highest density of any of the sections and gave satisfactory service. This confirms the conclusion reached in previous investigations that early difficulties encountered in compacting materials having acceptable gradings and plasticity indexes need not be taken as an indication of poor quality.

11. Because of its greater density and stability a well-graded sand-clay-gravel material having a low plasticity index is to be preferred to absolutely nonplastic material of comparable grading for base-course construction.

12. The tests indicate that properties other than those revealed by the mechanical analysis and plasticity tests influence the behavior of crushed stone or slag aggregates.

13. It is indicated that the crushed granite with the nonplastic binder used in these tests is not wholly satisfactory either as a surface or as a base. Since satisfactory roads have been built using granite from other sources a more comprehensive investigation of this class of material seems desirable.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF OCTOBER 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 1,740,986	$ 864,785	100.6	$ 7,917,594	$ 3,892,318	259.9	$ 520,090	$ 259,040	10.2	$ 2,724,845
Arizona	972,915	676,732	38.5	1,850,334	1,273,112	93.9	364,101	241,364	20.9	746,615
Arkansas	4,346,672	3,364,023	185.1	831,620	804,785	37.6	1,360,957	719,635	61.1	339,956
California	4,335,690	2,369,920	59.8	3,081,581	1,656,863	56.7	1,568,019	822,299	34.6	2,807,576
Colorado	1,665,527	918,250	41.4	3,311,197	1,844,183	77.8	366,428	206,519	7.8	1,636,253
Connecticut	357,558	176,334	5.1	1,964,755	977,627	19.9				1,237,956
Delaware	516,499	252,872	12.0	1,278,959	626,818	26.4	181,020	90,510	.1	1,019,316
Florida	121,000	59,928	1.4	4,284,214	2,141,882	72.9	505,649	252,825	7.9	2,500,855
Georgia	2,713,300	1,356,650	150.8	6,296,468	3,148,234	346.7	1,459,842	729,921	43.2	5,071,027
Idaho	1,658,103	999,180	82.6	1,155,908	703,715	64.6	274,979	154,602	27.0	1,055,403
Illinois	4,083,413	2,032,669	106.9	8,650,009	4,323,762	167.9	1,272,438	634,825	24.4	2,527,322
Indiana	2,115,702	1,057,851	38.6	6,069,452	3,010,126	135.3	732,436	366,143	13.7	1,865,082
Iowa	2,353,005	1,097,074	108.8	4,769,192	2,099,858	170.4	193,010	91,190	8.0	874,954
Kansas	2,375,055	1,179,822	130.8	2,398,403	1,198,322	115.4	2,915,692	1,457,846	148.1	4,043,296
Kentucky	1,428,269	714,135	48.7	3,668,742	1,832,815	94.4	915,801	457,900	23.1	2,696,361
Louisiana	318,148	156,000	10.8	12,132,849	3,115,133	49.7	1,518,979	742,323	45.1	2,361,588
Maine	1,813,070	906,176	42.1	1,031,510	515,755	24.8	90,670	45,335	2.7	287,017
Maryland	1,249,690	595,711	21.0	2,564,573	1,267,905	37.4	473,000	232,500	8.3	1,816,744
Massachusetts	3,107,045	1,550,834	25.1	713,333	355,469	4.5	1,258,822	627,124	7.5	2,486,225
Michigan	2,471,624	1,206,013	65.5	4,213,089	2,104,400	131.3	682,200	246,200	8.2	2,748,504
Minnesota	3,021,247	1,503,732	198.1	5,395,173	2,675,117	285.6	1,611,402	804,611	70.1	3,081,111
Mississippi	658,400	238,370	35.3	9,163,058	3,448,335	362.5	491,900	227,650	15.4	2,101,461
Missouri	1,452,003	725,302	55.9	5,105,064	2,513,032	180.7	2,742,221	1,138,414	93.0	3,889,219
Montana	1,670,146	944,760	111.4	2,159,501	1,221,799	86.3	1,770,551	1,004,254	113.9	3,485,875
Nebraska	1,159,765	574,441	76.1	6,126,753	3,062,728	571.0	2,045,743	877,990	202.8	2,509,368
Nevada	974,123	836,466	48.7	617,017	529,657	24.7	603,296	518,547	28.5	625,668
New Hampshire	539,946	264,083	18.1	949,318	467,479	21.3	228,372	113,783	9.1	881,230
New Jersey	400,110	191,531	3.6	4,409,098	2,202,999	34.4	180,660	90,330	.1	1,835,924
New Mexico	1,290,907	795,998	108.3	980,780	598,068	38.6	496,131	309,635	38.6	1,128,560
New York	4,660,080	2,298,697	89.5	13,967,219	6,751,448	226.7	1,470,990	602,595	16.1	1,099,572
North Carolina	2,794,090	1,394,965	172.7	6,105,823	3,045,372	322.7	653,610	311,720	38.1	1,335,891
North Dakota	134,060	71,820	28.5	1,297,335	695,199	84.9	2,241,260	1,201,261	244.7	3,363,394
Ohio	2,702,668	1,351,334	32.5	9,116,948	4,489,760	98.0	3,140,160	1,450,580	27.9	5,709,230
Oklahoma	1,223,129	649,003	33.8	2,448,975	1,298,625	99.9	2,633,570	1,382,197	85.4	3,012,834
Oregon	1,687,139	1,005,727	89.1	2,432,864	1,455,960	85.2	1,276,854	600,150	36.4	1,039,348
Pennsylvania	4,601,129	2,283,475	63.0	8,677,516	4,168,217	76.6	3,025,417	1,500,078	37.3	3,147,258
Rhode Island	477,910	238,835	6.9	340,616	170,121	3.2	726,151	362,550	6.5	870,693
South Carolina	1,418,740	639,800	64.1	1,442,574	637,686	22.2	785,140	349,200	60.4	2,145,997
South Dakota	1,823,240	1,007,274	177.8	3,794,559	2,117,860	359.2	1,271,900	716,340	163.1	2,902,674
Tennessee	2,390,148	1,126,512	42.7	3,500,068	1,750,034	86.4	990,674	495,337	18.4	3,425,551
Texas	7,566,986	3,713,589	446.0	8,064,001	4,016,253	322.7	2,011,982	966,505	118.8	5,459,832
Utah	1,802,564	1,290,958	80.9	807,965	593,700	54.2	397,045	200,850	10.0	685,854
Vermont	708,655	347,793	17.8	209,404	104,512	5.5	631,844	315,900	20.9	301,224
Virginia	1,583,530	789,668	52.0	2,664,668	1,283,615	65.4	699,304	344,453	15.2	925,789
Washington	1,443,003	750,367	19.3	3,235,330	1,566,045	29.5	997,102	459,564	18.6	487,088
West Virginia	748,677	411,000	25.8	2,700,515	1,367,495	66.7	723,900	357,645	19.1	1,829,157
Wisconsin	3,845,993	1,887,062	140.3	6,096,440	2,998,880	188.6	393,586	185,145	14.6	1,637,742
Wyoming	1,204,864	752,152	118.0	689,717	422,277	75.3	713,328	450,147	66.3	563,935
District of Columbia				341,624	170,812	2.1	106,700	53,350	1.0	263,338
Hawaii	139,841	66,868	1.0	993,980	480,750	16.4	579,027	286,093	10.1	1,055,906
Puerto Rico	647,050	322,600	13.8	1,282,253	635,060	24.9	83,225	40,975	.6	376,160
TOTALS	94,513,414	50,009,141	3,646.6	193,299,938	93,836,977	5,908.9	52,377,179	26,095,950	2,102.9	102,123,778

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF OCTOBER 31, 1939

STATE	Completed During Current Fiscal Year: Estimated Total Cost	Completed: Federal Aid	Completed: Miles	Under Construction: Estimated Total Cost	Under Construction: Federal Aid	Under Construction: Miles	Approved for Construction: Estimated Total Cost	Approved: Federal Aid	Approved: Miles	Balance of Funds Available for Programmed Projects
Alabama	$ 197,905	$ 97,650	17.2	$ 942,462	$ 383,350	25.6	$ 30,200	$ 15,100	6.8	$ 729,156
Arizona	150,487	108,529	16.5	147,395	105,786	16.9				325,941
Arkansas	765,480	612,922	65.7	183,579	178,910	22.7	73,469	73,414	6.5	151,875
California	662,750	357,967	34.9	499,961	245,274	14.0	191,095	102,335	4.3	651,077
Colorado	518,727	265,180	19.5	478,290	269,039	14.0	36,596	20,626	3.1	31,948
Connecticut				172,794	72,417	2.9	108,041	37,810	.2	251,419
Delaware	80,840	40,420	17.5	71,661	35,831	7.8				232,384
Florida	194,117	96,700	7.3	795,505	393,394	30.3	7,358	3,679		371,271
Georgia	226,952	111,740	28.0	269,282	134,641	31.3	129,939	64,970	18.1	1,072,324
Idaho	214,675	129,195	22.9	319,386	163,267	25.8	130,982	70,634	9.3	73,778
Illinois	841,207	417,975	31.3	1,179,200	529,608	87.8	356,700	177,050	21.0	551,933
Indiana	489,000	244,500	41.7	714,596	356,094	54.3	112,122	56,050	9.4	631,614
Iowa	24,095	11,069	22.9	546,140	257,335	52.5	868,031	406,700	148.3	1,004,703
Kansas	56,428	28,214	26.1	183,802	91,901	20.5	340,325	170,161	7.3	1,291,203
Kentucky	445,912	143,463	44.0	1,057,612	299,582	54.3	521,864	181,918	49.8	223,613
Louisiana	471,353	215,553	41.5	348,970	166,888	27.0	358,993	166,352	29.0	283,778
Maine	324,403	162,130	18.6	161,120	79,474	9.4	19,700	9,850	1.2	9,072
Maryland	204,891	98,787	16.0	80,696	23,048	2.5	186,000	63,355	11.7	350,491
Massachusetts	101,519	50,435	2.4	507,052	251,158	11.1	49,200	24,525	1.6	463,550
Michigan	430,989	209,952	44.7	1,367,782	681,391	95.3	170,000	85,000	10.3	760,511
Minnesota	513,256	254,584	35.3	588,696	294,348	73.8	97,458	48,729	23.4	1,060,844
Mississippi	176,500	88,250	6.8	785,262	396,346	60.7	67,500	33,750	17.0	632,520
Missouri	645,117	311,688	96.0	720,758	342,330	67.3	138,612	52,437	25.8	561,273
Montana	468,607	265,799	48.6	381,003	216,037	23.0	59,718	33,872	6.6	816,329
Nebraska	464,642	222,306	88.3	823,370	404,510	144.7	139,699	69,850	25.4	319,657
Nevada	160,893	136,525	25.0	70,067	60,261	18.1				145,070
New Hampshire	61,156	29,708	2.4	83,280	39,815	3.1				150,441
New Jersey	255,320	125,610	7.1	295,020	146,820	17.3	24,500	12,250		509,585
New Mexico	466,270	286,858	42.1	27,020	15,690		361,210	146,391	26.9	80,519
New York	918,636	454,559	45.9	2,393,105	1,149,608	86.7	273,200	95,750	4.1	229,233
North Carolina	711,294	355,625	57.7	725,030	362,515	73.7	35,240	12,500	2.3	316,205
North Dakota	115,030	61,606	8.3	37,500	20,100		111,270	59,617	10.9	819,207
Ohio	230,820	115,410	15.2	900,170	455,550	39.9	802,000	401,000	27.8	1,323,392
Oklahoma	81,838	38,943	.8	336,696	179,153	16.8	470,655	250,428	36.9	858,921
Oregon	551,886	310,902	59.9	271,962	126,380	31.2	14,696	8,820		292,800
Pennsylvania	1,905,307	940,977	109.3	1,096,322	542,056	38.5	370,074	185,037	13.3	211,879
Rhode Island	93,827	46,890	2.2	81,236	40,618	.2	36,060	18,030	.4	78,277
South Carolina	562,159	228,890	56.9	22,620	10,179		303,500	126,600	22.2	220,251
South Dakota	3,830	2,100	4.0	23,396	12,878					1,043,072
Tennessee	732,508	313,954	27.1	226,796	113,398	5.3				759,186
Texas	1,735,044	856,416	198.2	887,272	428,005	83.9	313,260	135,950	40.9	899,503
Utah	190,570	110,765	30.2	48,830	24,098	9.2	115,770	44,000	5.2	153,199
Vermont	126,051	62,026	4.5	187,872	61,587	7.3				52,851
Virginia	525,334	255,171	57.3	232,980	114,854	14.4	310,590	136,228	17.9	172,598
Washington	473,395	247,383	26.0	290,290	151,918	22.9	103,829	53,400	11.2	215,301
West Virginia	145,150	72,575	8.3	13,015	6,507		203,901	101,951	10.8	411,464
Wisconsin	523,087	260,059	27.1	591,300	295,162	9.3	471,585	221,418	7.3	473,606
Wyoming	464,992	288,144	26.0	159,218	100,513	10.3	112,112	65,156	19.6	64,172
District of Columbia				101,892	50,446	1.1	15,800	7,900	.2	14,779
Hawaii	90,660	45,330	3.7	205,590	102,795	4.6	179,480	89,705	6.4	82,170
Puerto Rico				224,464	109,130	12.8	55,188	27,140	2.1	60,233
TOTALS	19,794,909	10,191,425	1,638.9	22,859,317	11,091,995	1,482.1	8,877,522	4,167,438	702.5	22,490,178

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF OCTOBER 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR: Estimated Total Cost	Federal Aid	NUMBER: Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	UNDER CONSTRUCTION: Estimated Total Cost	Federal Aid	NUMBER: Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	APPROVED FOR CONSTRUCTION: Estimated Total Cost	Federal Aid	NUMBER: Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
Alabama	$ 515,350	$ 503,059	5	1		$ 743,712	$ 742,184	11			$ 43,508	$ 43,300	1	1	2	$ 816,092
Arizona						518,061	515,813	6								209,120
Arkansas	189,891	189,891	3			669,853	663,348	3			23,968	23,968	2		8	593,191
California	605,661	605,611	5			1,137,816	1,136,771	6		1	137,560	137,560	2		1	1,243,433
Colorado	309,307	309,305	3		11	306,992	306,992	2			48,514	44,754			14	791,132
Connecticut						172,722	161,008		1		21,333	21,333			7	829,223
Delaware						9,150	9,150	2			2,320	2,320			1	513,891
Florida						624,747	620,249	4			11,800	11,800			3	1,034,542
Georgia	56,530	56,530	3			405,396	405,396	4	1		446,616	446,616	6	3	21	1,957,478
Idaho	191,612	160,443	3			122,880	122,518	1								466,886
Illinois	1,846,535	1,845,675	11	3	43	1,828,339	1,689,967	10	1	21	184,242	173,570	3		24	1,979,000
Indiana	739,703	739,703	3	1	45	398,555	398,555	1	1	33	423,614	423,614	1	1	52	740,558
Iowa	340,647	321,200	10		2	279,344	242,456	4		40	755,748	707,700	1		180	831,318
Kansas	697,393	697,393	8		4	655,704	655,704	7		1	354,569	351,643	5		7	778,718
Kentucky	380,015	380,015	6	4		396,061	396,061	6			771,423	771,423	5		13	375,135
Louisiana	122,838	122,830	2			824,494	770,989	7			317,665	317,659	10			584,469
Maine	327,109	327,109	3	2		208,728	208,728	2	1							236,310
Maryland	24,510	15,402				240,795	240,795	2		2	46,200	46,200			13	925,099
Massachusetts	265,259	264,538	1	2		257,307	256,764	3			14,320	14,320	1			1,711,447
Michigan	386,326	386,326	3	2	7	855,015	855,015	5		3	475,081	475,081	3		29	1,309,120
Minnesota	254,481	251,067	1	4	6	1,237,359	1,217,528	9	2	7	509,223	336,263	3		6	1,117,339
Mississippi						611,373	611,373	8			37,300	37,300	1			889,528
Missouri	65,589	64,284	1			1,266,356	1,266,356	7	2		680,534	655,256	3	4		1,324,064
Montana	614,653	614,653	6			448,927	333,846	6			80,000	80,000	1			204,589
Nebraska	381,275	380,675	13			781,157	781,157	12	1	2	269,075	269,075		1	45	511,406
Nevada	156,993	156,993		1	3	37,305	37,305		1	5	11,577	11,577			5	114,944
New Hampshire	48,623	48,202	5		1	113,962	113,879	3			45,804	45,804	2			313,600
New Jersey	7,140	7,140			1	730,316	730,316	2	3		150,090	150,090	1			1,289,255
New Mexico	59,805	59,805	2			15,276	15,276				2,572	2,572			1	692,665
New York	1,172,330	1,168,630	2	5		2,360,212	2,182,492	10	9		200,820	200,120	1	1		3,453,396
North Carolina	668,844	636,144	4	2	18	876,400	874,000	9	3	8	444,735	444,735	2	1	39	586,364
North Dakota	105,450	105,450	3	1		818,489	770,087	9			75,960	75,960	1			391,201
Ohio	308,640	293,640	6			1,449,673	1,378,721	8	3		917,090	917,090	6		4	2,473,570
Oklahoma	266,955	266,955	3		32	187,025	187,025	3		11	307,500	294,700	8	1	8	1,924,197
Oregon	40,500	39,002				266,498	265,204	3								311,060
Pennsylvania						2,344,004	2,132,105	6	4		520,652	312,200	2		2	4,218,923
Rhode Island	327,613	327,613	1	2		111,178	111,178		1							152,459
South Carolina	177,614	144,232	4	2	7	590,456	568,120	6	3		179,375	:79,375		2	26	782,723
South Dakota	72,757	72,757		1	8	336,475	336,475	4	2	9	47,550	47,550	1			995,433
Tennessee	73,600	73,600		1	1	799,806	799,806	3	3	2	6,760	6,760			2	1,332,344
Texas	1,204,696	1,173,640	12	2		2,321,258	2,256,842	18	1		51,150	51,150			26	1,913,207
Utah	111,841	111,778	2		32	146,758	146,758	1		53	161,010	161,010			58	177,522
Vermont	35,058	30,246			7	1,462	1,462				118,940	118,940		1		200,249
Virginia	207,404	207,404	2		16	515,975	423,075	7	2		129,256	129,256	1	1	4	909,331
Washington	126,622	126,621	1		8	209,590	208,180	2	1	2	208,720	208,037	1	2	2	362,668
West Virginia	64,417	64,417	2			310,434	294,674	5			24,200	24,200	1		3	959,985
Wisconsin	480,877	478,594	6		2	1,056,194	1,012,263	9	1	6	684,738	625,617	3	2	6	650,823
Wyoming	40,626	40,470	1		7	98,543	98,543		1							516,862
District of Columbia						292,412	258,868	1			74,400	74,400				47,053
Hawaii	52,950	50,320	1			132,850	132,850	3	1		6,216	6,216			1	353,234
Puerto Rico	49,040	48,840	1			345,312	343,310	8								426,676
TOTALS	14,175,079	13,968,202	148	36	261	31,468,706	30,287,537	248	49	206	10,023,728	9,478,114	78	21	613	47,522,832

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PUBLIC ROADS

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PUBLIC ROADS ADMINISTRATION

VOL. 20, NO. 10 ▽ DECEMBER 1939

STRIPE TO RESTRICT PASSING ON A 3-LANE ROAD IN MARYLAND

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Issued by the

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PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 20, No. 10 December 1939

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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CERTIFICATE: By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

MARKING AND SIGNING NO-PASSING ZONES ON TWO- AND THREE-LANE ROADS

BY THE DIVISION OF DESIGN, PUBLIC ROADS ADMINISTRATION

Reported by JOSEPH BARNETT, Senior Highway Design Engineer

THE delineation of traffic lanes on highways by pavement inserts or painted stripes has long been recognized as an important contribution to safety and driving comfort. The increase in speed of motor-vehicle travel during the past decade has emphasized the importance of the use of centerline or lane-line pavement marking with the result that nearly all States have adopted some system of marking their important highways.

These systems naturally reflected local road conditions, driver habits, and the different opinions of highway officials, so that marking and signing now encountered on the highways frequently differ radically from one State to another and sometimes from section to section in the same State. This confusion is particularly critical in the methods of pavement striping and signing used to indicate zones of short sight distance, unsafe for passing maneuvers.

The obvious solution to the problem created by zones of short-sight distance is their entire elimination, but immediate elimination is not economically feasible in many instances. Most States, therefore, have resorted to some system of pavement striping or signing, or both, to warn traffic against encroachment upon the lane of opposing traffic within the limits of these zones. The marking of "No-Passing Zones" has proved effective in encouraging safe driving, despite the widely varied systems of marking used. Highway engineers are convinced that the Nation-wide adoption of rational standards for marking no-passing zones is highly desirable.

The common desire of the State highway departments, the Public Roads Administration, and other highway organizations to encourage the development of a uniform system for pavement marking led to action through the American Association of State Highway Officials. The problem was approached through action of three existing committees of the Association. The Special Committee on Administrative Design Policies undertook the work of preparing criteria to designate which portions of the highway should be marked as no-passing zones, where the markings should be located, and what the markings should represent. The Committee on Traffic Control and Safety and the Committee on Maintenance jointly undertook the work of determining the details of a normal centerline or lane-line stripe as to color, width, continuity, etc., the changes to be made to the normal stripe to indicate no-passing zones, and the signing of no-passing zones.

SURVEY MADE OF STATE PRACTICES IN MARKING PAVEMENTS

Early in 1939 the Special Committee on Administrative Design Policies requested the Public Roads Administration to make a survey of existing practice in marking and signing pavements on rural highways. In this survey data were obtained from each State regarding the color, width, continuity, mileage, etc., of the normal pavement stripe, or stripes, on rural highways, and the variations in these details, plus information regarding shoulder signs, to indicate no-passing zones on two- and three-lane pavements. Tables 1 to 4 summarize this information. In each table the numbers of States reporting the same systems, values, or items are listed. The large variation in different details and systems now in use by the several States is evident.

TABLE 1.—*Summary of State practices regarding definition of no-passing zones* [1]

Minimum sight distance	2-lane roads	3-lane roads
300 feet	2	2
400 feet	1	1
500 feet	13	3
600 feet	2	1
700 feet	3	1
800 feet	4	3
1,000 feet	4	2
475 to 1,260 feet	1	1
1,000 to 1,200 feet	1	
1,200 to 2,400 feet	1	
Minimum sight distance bases not used	5	2
Also use minimum radius control	6	3

[1] 11 States do not mark zones on 2-lane roads, 24 States have no rural 3-lane roads, and 8 States do not mark zones on 3-lane roads.

TABLE 2.—*Summary of State practices regarding color of normal pavement stripe* [1]

Stripe color	Concrete pavement	Bituminous pavement
Black	23	4
White	13	28
Yellow	9	14
Miscellaneous or none	3	2

[1] The same color stripe is used on both concrete and bituminous pavements in 25 States. Different color stripes are used in 22 States

TABLE 3.—*Summary of State practices regarding type and width of normal pavement stripe*

Type	2-lane roads		3-lane roads [1]	
	Concrete pavement	Bituminous pavement	Concrete pavement	Bituminous pavement
Joint only, or no painted stripe	6	5	2	1
Broken stripe	10	16	4	7
Continuous stripe	29	26	18	16
Miscellaneous	3	1		
Width of stripe:				
3 inches	2			
4 inches	26		15	
5 inches	2			
6 inches	8		6	
Other	10		3	

[1] 24 States indicated no rural 3-lane roads.

Twenty-two States use broken stripes varying from stripes 10 feet long spaced 10 feet apart to stripes 100 feet long spaced 100 feet apart.

The location of the stripe on widened curves on 2-lane roads also varies considerably among States: In 27 States the stripe is placed on the physical centerline of the road; in 10 States the stripe is placed inside the physical centerline of the road; in 4 States the stripe is placed outside the physical centerline of the road; 4 States have no standard; and the remaining 3 States place the stripe either on the physical centerline or outside the centerline.

State practices regarding the marking of no-passing zones on 2-lane roads are summarized in table 4.

TABLE 4.—*Summary of State practices regarding the marking of no-passing zones on 2-lane roads*

Method	States
Signs—normal stripe	5
Special stripe—no signs	11
Special stripe—with signs	21
Not marked—normal stripe	11

Type of special stripe	States
Single continuous line	14
Double or triple continuous line	8
Added line on right of normal stripe	9
Added line on left of normal stripe	1

Signs indicate—	Signs only	Stripe and signs	Total
Beginning only (2 signs per zone)	3	10	13
Beginning and end (4 signs per zone)	1	3	4
Beginning or beginning and end, but not used at all zones	1	8	9

Twenty-two States continue the special stripe (different from the normal stripe) throughout the zone. Ten other States use an added stripe only for the first part of the zone, terminating it at the point where sight distance is no longer restricted. Of these 10, 3 States use a double stripe composed of a broken stripe and continuous stripe of the same color, and 7 States add a continuous stripe of a different color, either white or yellow. One State uses a double or triple stripe throughout but changes the color of the right stripe at the point where sight distance is no longer restricted. One State uses a continuous double stripe throughout but in some cases indicates the end of no-passing length from one direction by a diagonal arrow across the stripe. Thus of 32 States using stripe markings in no-passing zones on 2-lane pavements only 12 incorporate the feature of a unidirectional indication of the point beyond which the sight distance is no longer restricted.

Method (3-lane roads)	States [1]
Special stripe on center line—no signs	3
Special stripe on center line—with signs	6
Special stripes on lane lines—with signs	7
Zones not marked—normal stripe	8

Type of special stripe	States
Single continuous line	8
Double or triple continuous line	4
Added line on right of normal stripe	4

Signs indicate—	States
Beginning only (2 signs per zone)	6
Beginning and end (4 signs per zone)	3
Beginning or beginning and end, but not at all zones	4

[1] 24 States indicate no rural 3-lane roads.

State practices regarding the marking of no-passing zones on 3-lane roads are summarized in the previous tabulation.

Of the 16 States marking no-passing zones on three-lane highways, 9 have systems restricting all passing in the entire length of the zone by means of centerline striping, 6 have a lane-marking system permitting two-lane operation as soon as the road opens up to view and 1 permits two-lane one-way operation on the upgrade only. Thirteen States apparently use one or more signs in conjunction with the pavement stripes in the zones and 3 States use no signs.

The summary of existing practice was given serious consideration by the three committees of the Association.

MARKING SHOULD INDICATE NO-PASSING ZONES SEPARATELY FOR TRAFFIC IN EACH DIRECTION

In establishing criteria for no-passing zones and the location of stripes the Special Committee on Administrative Design Policies felt that it was of paramount importance that the marking should indicate no-passing zones separately for traffic in each direction. The sight distances ahead and to the rear on a road are generally of unequal lengths. If a stripe indicating a no-passing zone, such as a single stripe on the centerline, does not differentiate between opposing directions of travel, the usefulness of the road is seriously impaired and a disrespect for restrictive stripes may be developed. When, for example, a highway is on tangent and sight distance over the crest of a hill is inadequate for passing, the restrictive stripe should begin where the sight distance for traffic approaching the hill is less than a desirable minimum, but beyond the crest where the road ahead opens up to view the stripe should not restrict a vehicle from making a passing maneuver. A stripe, however, is required to prevent passing by vehicles in the opposite direction approaching the crest. Both objectives may be attained by the use of a system of striping which restricts passing to traffic in one direction only, such as a dashed stripe throughout the length of the road with an additional continuous stripe on the side where sight distance is limited.

A no-passing zone for traffic in one direction may overlap a no-passing zone for traffic in the opposite direction or there may be a gap between the ends of the zones. A system of striping which differentiates between traffic in opposing directions naturally will show these overlaps and gaps. A normal broken stripe with a continuous stripe alongside to indicate a no-passing zone would, for example, have a continuous stripe on both sides where the no-passing zones overlap on a two-lane road.

Passing on three-lane roads is accomplished on the middle lane. If the system of marking at no-passing zones restricts traffic in both directions from passing, the middle lane becomes ineffective. Most of the middle lane, however, can be used effectively; confusion can be avoided, hazard diminished, and utility of the road increased by a system of striping which restricts traffic in one direction to one lane but permits traffic in the opposite direction to use two lanes. There has been some question regarding the most desirable type of operation over the crests of hills on three-lane roads on which widening to four-lanes is not justified. Some engineers contend that the delay caused by slow-mov-

View Taken From Crest of Hill. Non-Directional Marking Restricts Passing Despite Adequate Sight Distance.

ing trucks going uphill should be avoided by reserving two lanes for upgrade traffic. From the standpoint of sight distance this system is hazardous in that it encourages passing when the sight distance is limited. Traffic should be confined to the right lane when sight distance is inadequate for passing just as on two-lane roads. When the road ahead opens up to view, the restriction should be terminated and passing permitted on the middle lane.

Diagonal striping should be provided in the middle lane, crossing from the inside of the left lane to the beginning of the restrictive striping on the inside of the right lane. The diagonal striping should meet the longitudinal restrictive striping at the beginning of the no-passing zone. The diagonal striping should inform drivers in one direction of the necessity of moving over to the right lane without crossing the diagonal striping but should not restrict crossing by vehicles traveling in the opposite direction.

In striping a highway to restrict passing of vehicles where sight distance is inadequate the general concept in choosing factors to determine sight distance below which passing should be restricted is totally different from that in choosing factors to determine sight distance to design a highway. If the same factors are chosen and a highway is striped to restrict passing wherever the sight distance is less than the passing minimum, and almost all drivers accept the dictum that they are required to keep to the right of a restrictive stripe throughout its length, the use of the highway is severely impaired.

Three-Lane Highway Marked To Restrict Traffic to Two Lanes Over the Crest of a Hill Where Sight Distance for Up-Hill Traffic Is Restricted. Such Non-Directional Marking Unnecessarily Restricts Down-Hill Traffic Which Has Adequate Sight Distance for Passing, Thus Encouraging Disregard for Restrictive Line and Passing as Shown.

SIGHT DISTANCE FOR STRIPING DEPENDS ON DESIGN SPEED

The desirable minimum sight distance on which to base restrictive striping for a two- or three-lane road lies between the minimum passing sight distance used in design and a sight distance of no appreciable length. The former is on the side of safety if the line is respected but restricts the use of the road. The latter is highly hazardous but permits passing at will so that even if a driver sees only enough of the road ahead to pass a vehicle practically standing still in the face of opposing traffic traveling at a very low speed he is not deterred by a restrictive stripe.

The sight distance for striping as finally recommended in the second column of table 5 is a compromise based on a passing maneuver such that the frequency of maneuvers enabling passing where sight distances are shorter is not great enough to impair seriously the usefulness of the road. Note that the sight distance for striping varies and is dependent on the design speed. The design speed is one which is greater than that used by almost all drivers on any particular road or section of road when traffic is not heavy enough to impede

Table 5.—*Relation of limits of no-passing zones to the point of intersection of vertical curves for purpose of marking pavements*

Design speed (miles per hour)	Minimum passing sight distance for marking pavements (feet)	Value of algebraic difference of grades, percent ÷ 100													
		0.04		0.06		0.08		0.10		0.12		0.14		0.16	
		A [1]	B [2]	A	B	A	B	A	B	A	B	A	B	A	B
30	500	40	330 −170	160	390 −110	200	420 −80	280	440 −60	340	460 −40	390	490 −10	420	510 10
40	600	180	420 −180	310	480 −120	410	520 −80	525	580 −20	630	620 20	740	660 60	930	720 120
50	800	320	620 −180	500	700 −100	680	790 −10	860	860 60	1,030	940 140	1,200	1,020 220	1,360	1,080 280
60	1,000	600	820 −180	930	980 −20	1,250	1,120 120	1,530	1,260 260	1,880	1,400 400	2,160	1,540 540	2,500	1,690 690
70	1,200	980	1,060 −140	1,480	1,320 120	1,970	1,540 340	2,460	1,760 560	------	------	------	------	------	------

[1] A=Length of vertical curve resulting in minimum sight distance permitted in design.

[2] *B*=Horizontal distance from the point of intersection of vertical curve to limits of no-passing zone. The upper figure in each case is the distance in feet to the beginning of the no-passing zone. The lower figure is the distance to the end of the no-passing zone. When the lower figure is a minus value the end of the zone is on the near side of the point of intersection, the length of the no-passing zone is the difference between the two figures, and there is a gap between the ends of no-passing zones in opposite directions. When the lower figure is a plus value the end of the zone is on the far side of the point of intersection, the length of the no-passing zone is the sum of the two figures, and no-passing zones in opposite directions overlap.

smooth operation. It is governed largely by the physical characteristics of the road such as sharp curvature and by the surroundings, wide-open spaces encouraging higher speeds than built-up areas. Sight distance for marking is based on height of eye and height of object above the road surface both being 4.5 feet.

No-passing zones on an existing road are evaluated by determining the design speed of the road or section of road, after which the beginning and end of each no-passing zone are located. The sight distance at these points corresponds to the minimum passing sight distance for marking. The methods of measurement are relatively simple and are not discussed in this report.

An idea of the location of no-passing zones at hill crests can be obtained from table 5 which shows their location for various changes in grade for which the lengths of vertical curve are those resulting in the minimum sight distance permitted in design.

RESTRICTIVE STRIPE NEEDED AT INTERSECTIONS AT GRADE

It is desirable that vehicles approaching an intersection on two- and three-lane roads keep to the right and that all passing maneuvers should be completed before reaching the intersection. Passing while crossing an intersection is hazardous because: (1) The passed vehicle may obstruct the view of the cross road to the right; (2) the passed vehicle may turn left in front of the passing vehicle; and (3) the driver of a passing vehicle may find it difficult to observe crossing and turning traffic at the same time that he is required to watch traffic ahead. Two- and three-lane roads, therefore, should be striped to restrict passing for some distance each side of an intersection. Once beyond an intersection, there is no further need to restrict passing if sight distance and traffic conditions permit passing. The stripe, therefore, should restrict vehicles approaching the intersection from passing and not restrict passing beyond the intersection.

When one road at an intersection is a preference road and traffic on the nonpreference road is required to stop at the intersection the use of restrictive striping on the preference road is open to serious question. There is some hazard in the possibility of a left-turning vehicle cutting in front of a passing vehicle but the hazard due to possible restriction of sight caused by the passed vehicle is nil. It appears to be inadvisable, therefore, to restrict the free movement of traffic on the preference road by the use of no-passing marking if not required otherwise.

Normally the driver of a passing vehicle should return to the right lane before reaching the beginning of a restrictive stripe. The length of restrictive stripe at the approach to an intersection is, theoretically, zero. The restrictive stripe should, however, be visible for some distance so an arbitrary length is chosen, say 100 to 200 feet. An appreciable length of restrictive stripe will also have the desirable effect of encouraging drivers, who normally return to the right lane some distance past the beginning of the stripe, to return to the right lane before reaching the intersection.

At intersections where vehicles are stopped by a traffic light, stop sign, preference road sign or, in the absence of such controls, by cross traffic, it is desirable that vehicles facing the intersection line up on the right so that traffic is free to move in both directions when permitted. The restrictive stripe encourages vehicles to keep to the right under such circumstances and its length may be determined by the probable number of vehicles which will be thus lined up, allowing about 20 feet for each passenger vehicle.

A restrictive stripe on a three-lane road approaching an intersection normally should be located on the right lane line in the same manner as at a location with short sight distance. This marking serves to line up vehicles approaching the intersection and permits passing by vehicles leaving the intersection. If vehicles are likely to be stopped, however, it may be desirable to locate the restrictive stripe on the center line of the pavement. If stopping is effected by continuous traffic light control it may be desirable to use only normal lane stripes and omit restrictive stripes altogether. Passing may be accomplished when the light is green if sight distance and traffic conditions along the road are favorable and two lanes in each direction may be used for storage when the light is red. If traffic is evenly divided in both directions, opposing traffic in the middle lane will have to free itself on the go signal. When traffic is heavy, however, it generally is unbalanced and the omission of restrictive stripes may have the desirable effect of providing two lanes for storage and movement in one direction. At important intersections three-lane roads may be widened to four lanes and a restrictive stripe placed at the centerline.

At intersections where there is a considerable volume of left-turning traffic it may be desirable to omit the restrictive stripe and mark the middle lane on the approaches to the intersection for the exclusive use of left-turning vehicles.

Traffic should be restricted from passing while crossing a railroad at grade. While there is no left-turning traffic the passed vehicle may obstruct the view of the signal and the track to the right. A restrictive stripe also has the desirable effect of lining up vehicles in the right lane when the crossing is closed so that traffic is free to move in both directions when the crossing is clear.

At places where the number of traffic lanes change, as where a two-lane road changes to a three- or four-lane road, traffic should be informed of the change by appropriate signs and marking in both directions to encourage traffic to keep in its proper lane or lanes.

The Special Committee on Administrative Design Policies has prepared a statement entitled "A Policy on Criteria for Marking and Signing No-Passing Zones on Two- and Three-Lane Roads" from which the preceding discussion is largely taken. The approved conclusions are given at the end of this report.

RESTRICTIVE STRIPES SHOULD VARY IN COLOR, TYPE, AND WIDTH

The committees which jointly undertook to establish a standard system for the details of marking and signing no-passing zones took the following basic principles into consideration:

1. The no-passing marking should be easily understandable.
2. It should be economical to place.
3. It should conflict as little as possible with existing practice in road striping.
4. The stripe itself should be the same for two- and three-lane roads, regardless of its position on the pavement.

There was general agreement that the essential element should be a conspicuous and distinctive "restrictive" or "barrier" line placed along the right side of the centerline. This line could be made distinctive and conspicuous in any one of the three details of color, type, and width, or by combinations of them. It is evident that the starting point for the distinctive line is the normal stripe, the three details of which must be known. By analysis of existing practice a normal stripe, either black or white in color, continuous or broken in type, and about 4 inches in width, appears to be the best compromise.

To be distinctive the barrier stripe should differ from the normal stripe in at least one or two, and preferably all three, of the color, type, and width details. The color difference itself appears to be the least effective distinction, but certainly desirable, and, in some instances, necessary. With regard to type difference the barrier line should invariably be a solid line. Where the normal center line is a broken line, a solid auxiliary line will stand out conspicuously, whereas reversed types would seem definitely weak. If both lines are solid, either color or width variations must be included and preferably both should be used.

It is quite generally agreed that for the normal centerline a broken line, which permits a 50 percent saving in paint, is about as effective as a continuous line. Recent developments indicate that equipment can be perfected to lay down either continuous or broken lines exactly as desired once the general demand for such mechanical performance is created.

In view of the possible variations in color and type of stripe, considerable attention should be given to the width of the barrier line. It should be at least 4 inches wide, never narrower than the normal stripe and preferably at least 6 inches wide. The barrier line should be separated from the normal stripe rather than immediately adjacent to it.

The use of shoulder signs provides warning when conditions such as snow or dirt on the pavement are such as to make the marking insufficiently visible. Equally important, they help make clear the meaning of the pavement striping. Standard signs ("no-passing" and "end no-passing zone") are preferable to more explanatory signs, but signs at every no-passing zone are an unnecessary and unwarranted expense. Their use is optional.

On two-lane roads the no-passing marking should be placed along the centerline. On three-lane roads exactly the same type of barrier stripe can be used, placed so as to prevent use of the middle lanes by any vehicle having a restricted sight distance. On three-lane roads the no-passing marking should permit two-lane operation of traffic in one direction only (and one-lane operation in the other direction) after it has passed beyond the point of restricted sight distance.

With these considerations a subcommittee representing the Committees on Maintenance and on Traffic Control and Safety prepared a report [1] from which the preceding discussion regarding the details of marking and signing are largely taken and recommended standards for marking and signing no-passing zones as given at the end of this report.

The recommendations submitted by the three committees together comprise complete standards for determining, locating, and marking no-passing zones on two- and three-lane highways. They have been recommended to the American Association of State Highway Officials for adoption, and at present are being considered by the association members. Fortunately pavement markings rarely are permanent in character so that little expense is involved in changing to any standard system, once adopted, in a comparatively short time, possibly in the course of a single year. It will, of course, be necessary to educate drivers as to the meaning of the directional system of marking, but once they understand the principles upon which it is based they may be expected to adapt themselves rather easily to any minor interstate differences that may arise.

[1] The Marking of No-Passing Zones on Highways, by E. W. James, Papers and Discussions, Convention Group Meetings, American Association of State Highway Officials, 1938.

Centerline Stripe and Directional Stripe to Restrict Traffic Approaching the Curve. The Marking Would Be More Effective Were the Centerline Stripe Broken Instead of Solid or Different in Width or Color.

Following are the approved conclusions of the committees:

A POLICY ON CRITERIA FOR MARKING AND SIGNING NO-PASSING ZONES ON TWO- AND THREE-LANE ROADS

A no-passing zone for the purpose of marking two- and three-lane pavements shall be one in which the sight distance ahead is less than 500, 600, 800, 1,000, and 1,200 feet for assumed design speeds of 30, 40, 50, 60, and 70 miles per hour, respectively.

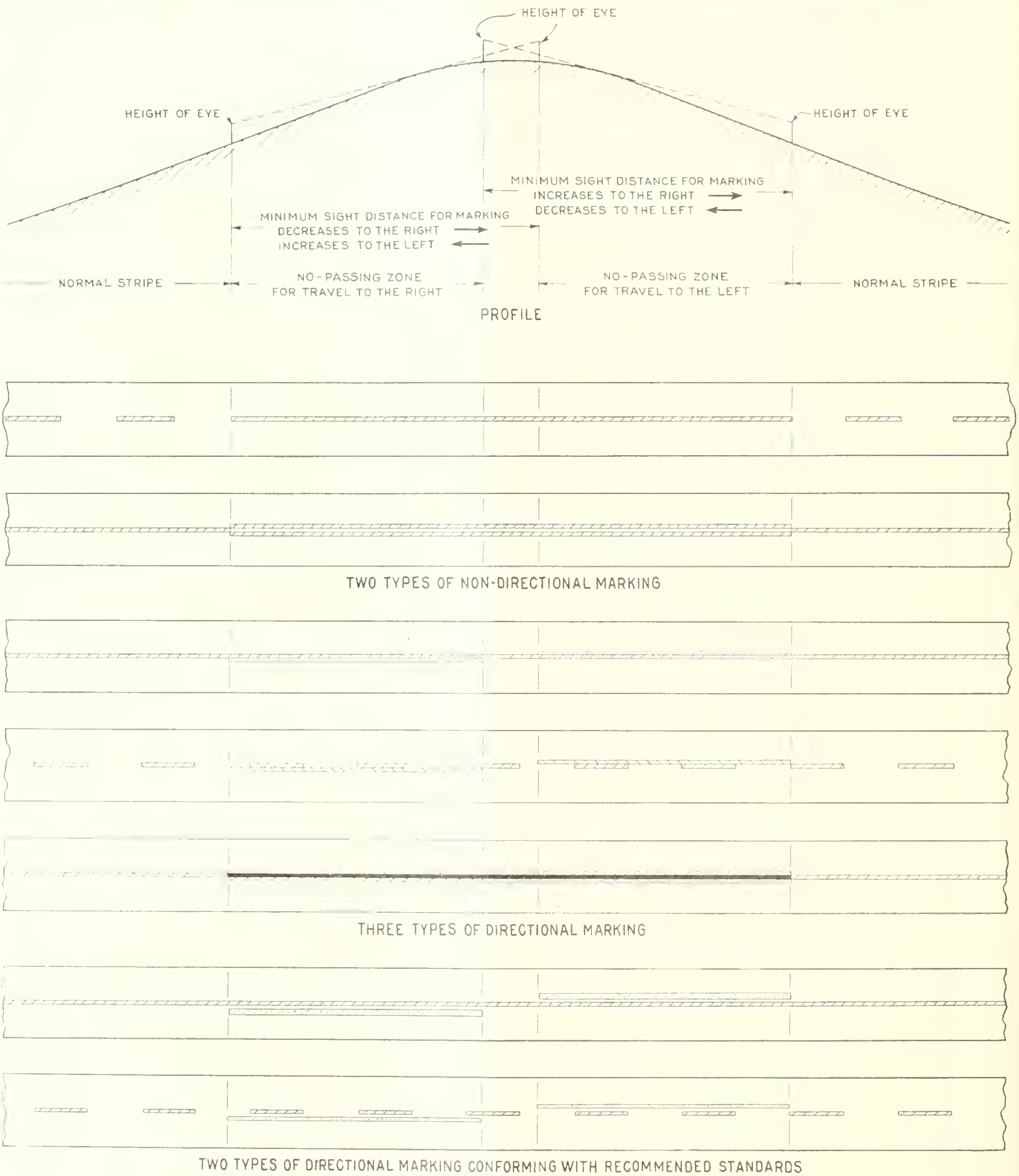
No-passing zones shall be determined and indicated separately for traffic in each direction. No-passing zones for traffic in opposite directions may overlap or there may be a gap between their ends.

Sight distances shall be measured between eye and top of vehicle, both 4.5 feet above the pavement surface.

The system of marking pavements of two- and three-lane roads shall restrict passing within the limits of no-passing zones and shall differentiate between traffic in opposing directions so that traffic in each direction will not be restricted from passing when the road opens up to view.

The system of marking pavements of three-lane roads shall restrict traffic in each direction to the right lane within the limits of a no-passing zone. Diagonal striping across the middle lane shall be provided approaching the beginning of a no-passing zone. The diagonal striping shall indicate that it must not be crossed by traffic approaching the no-passing zone but may be crossed by traffic in the opposing direction.

Intersecting roads for some distance from the intersection should be considered no-passing zones for traffic approaching the intersection. Where one road is a preference road the nonpreference road only may be considered a no-passing zone.



Types of Pavement Markings for No-Passing Zones Over Hill Crests on 2-Lane Roads.

No-passing zones at intersections may be marked in the same manner as other no-passing zones except that three-lane roads with stop control should have the restrictive stripe on the centerline instead of on the right lane line and where stop control on three-lane roads is effected by traffic lights restrictive stripes may be omitted.

The system of marking pavements for no-passing zones is primarily intended to be used for restricted vertical sight distance or a combination of restricted vertical and horizontal sight distance. The restriction in horizontal sight distance alone usually is obvious while the impairment of vertical sight distance on tangents generally is not realized by the average motorist.

(*Continued on p. 202*)

A SIMPLE ACCUMULATING-TYPE TRAFFIC COUNTER

BY THE DIVISION OF HIGHWAY TRANSPORT, PUBLIC ROADS ADMINISTRATION

Reported by O. K. NORMANN, Associate Highway Economist

A TRAFFIC COUNTER consisting of a dollar watch, a lever arm, a diaphragm and a rubber tube is the newest and simplest addition to the growing family of traffic counters. Previous articles have described two types of automatic traffic counters, one of the recording type and one of the simple accumulating or nonrecording type.[1] Each type has its own field of use in obtaining data on highway traffic volume needed in planning future highway improvements. In the highway planning surveys now being conducted by 46 States in cooperation with the Public Roads Administration, more than 500 recording-type counters and over 200 nonrecording-type counters are in use.

In general the recording counters are used to obtain long-time records at fixed locations. The most important function of these counters, aside from providing a continuous record of traffic flow past that particular point, is to furnish data to serve as a basis for establishing fundamental traffic trends and determining normal traffic patterns. These data provide a valuable basis for deriving and checking factors for expanding short counts of traffic made at numerous locations.

Simple accumulating-type traffic counters are useful in taking a relatively few counts properly distributed as to time and location. These short counts are then expanded by applying appropriate factors, thus providing a measure of the increases or decreases of the total traffic volume. It is essential that such counters be inexpensive and easily portable. Other requirements are simplicity, ruggedness, ease of installation, and ability to make an accurate count of vehicles traveling at high speeds.

The new counter consists essentially of a watch mounted over a rubber diaphragm which is connected to a rubber tube stretched across the highway. The air impulse generated by the wheels of a vehicle passing over the rubber tube is transmitted to the diaphragm, causing it to rise. This actuates a lever arm which moves the escapement arm of the watch. The front and rear wheels of a passing vehicle each cause a separate air impulse; thus each two-axle vehicle moves the escapement arm twice. In the type of inexpensive watch used, two cycles of the escapement arm are required to move the second hand of the watch forward one second.

OPERATION OF COUNTER DESCRIBED IN DETAIL

Figure 1 shows the various parts of the counter, consisting of: A metal cap, which fits over the watch and screws onto the base; a watch, which is mounted on a heavy metal base containing the rubber diaphragm; a hollow stem, which screws into the base and over which the rubber tube fits; and a section of rubber tubing. Figure 2 shows two views of the assembled counter ready for use. The stem can be screwed into the center of the base, as shown at the right in figure 2, or into the side of the base as shown at the left. A short screw, as shown at the right in figure 2, is used to plug the hole not used for the stem connection.

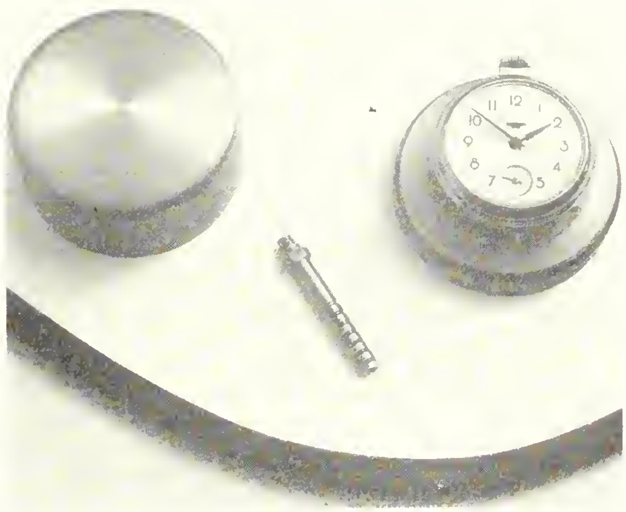


FIGURE 1.—THE SIMPLE ACCUMULATING-TYPE TRAFFIC COUNTER. UPPER LEFT, THE COVER; UPPER RIGHT, THE COUNTING UNIT MOUNTED ON METAL BASE; CENTER, THE STEM; LOWER, RUBBER TUBING OF THE KIND USED.

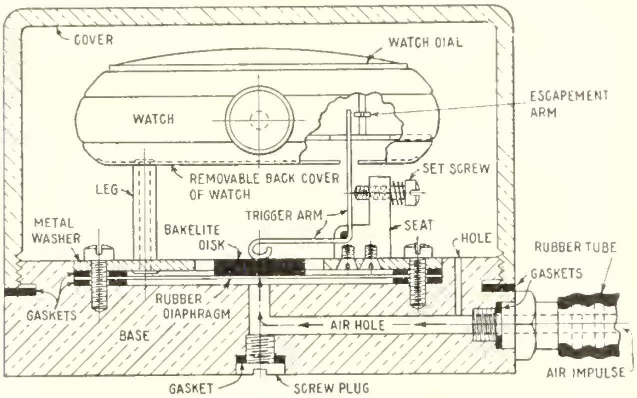
FIGURE 2.—THE COUNTER ASSEMBLED AND READY FOR USE.

Details of the counter are shown in figure 3. The watch is rigidly mounted over a metal base. The base consists of a metal disk ⅝ inch thick and 2.8 inches in diameter. A thin rubber diaphram is mounted in a circular depression in the top of the base. A metal washer is placed over the rubber diaphram. Screws hold both in place and, with the aid of rubber gaskets above and below the diaphragm, make an airtight connection. A small bakelite disk is cemented on top of the diaphragm at its center.

Two holes have been drilled through the base with a No. 13 drill, one passing vertically and one passing radially and connecting with the first hole at the center. These holes are threaded to fit the stem and the

[1] An Automatic Recorder for Counting Highway Traffic, by R. E. Craig. PUBLIC ROADS, vol. 19, no. 3, May 1938.
A Simple Portable Automatic Traffic Counter, by R. E. Craig and S. E. Reymer. PUBLIC ROADS, vol. 19, no. 11, January 1939.

screw plug. The top edge of the base is threaded to fit threads on the inside of the cap. Rubber gaskets placed at the threaded connections insure airtightness and watertightness. A vertical hole made with a No. 70 drill runs from the top of the base down to the radial hole and serves to equalize the air pressure above and below the rubber diaphragm without affecting its normal operation.



SECTIONAL ELEVATION

FIGURE 3.—DETAILS OF THE COUNTER.

On the metal washer that holds the rubber diaphragm in place is mounted a seat on which the L-shaped trigger arm pivots. This seat also holds an adjusting screw which prevents the trigger arm from moving far enough to injure the escapement arm of the watch. The trigger arm has a small loop on the bottom end, which rests on top of the bakelite disk on the diaphragm. The trigger arm extends vertically through a hole in the removable back cover of the watch and rests against the escapement arm of the watch, which is mounted above the base on three legs. Each of these legs rigidly connects the removable cover of the watch with the metal washer over the diaphragm.

The watch, which serves as the counting unit, is an inexpensive type costing less than a dollar. It has been slightly altered, as shown in figures 4 and 5, to serve as a simple counter instead of a timepiece. This has been done by cutting away a section of the back of the watch containing the rear bearing for the balance wheel and removing the balance wheel and the hairspring. A light wire spring has been inserted to act against the escapement arm of the watch. One end of the spring rests against the case of the watch and the other end is bent in a U-shape and straddles the escapement arm. The spring thus tends to push the escapement arm toward the center of the watch.

The trigger arm is adjacent to the inside edge of the escapement arm, and, when an air impulse causes the diaphragm to rise, the trigger arm pushes the escapement arm away from the center of the watch. After the impulse, the diaphragm descends to its normal position, the trigger arm pivots away from the escapement arm, and the light spring pushes the escapement arm toward the center of the watch, thus completing one cycle and permitting the second hand of the watch to move ahead. The watch must be wound for use but there is no movement of the parts except as permitted by the trigger arm.

In the type of watch used two cycles of the escapement arm are required to move the second hand ahead one second. Each two-axle vehicle is therefore counted

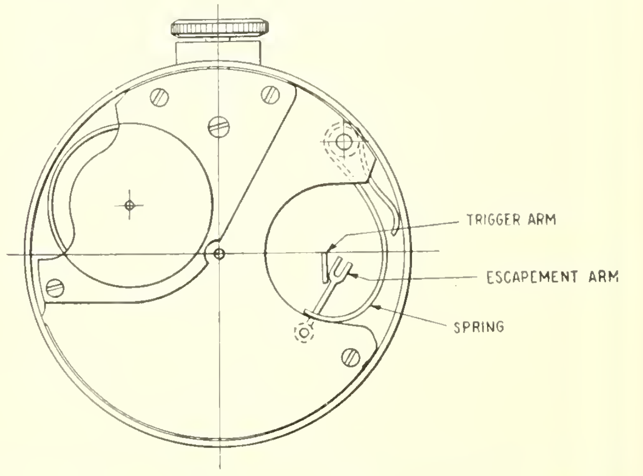


FIGURE 4.—ALTERATIONS ON THE BACK OF THE WATCH THAT CONVERT IT INTO A SIMPLE COUNTING MECHANISM.

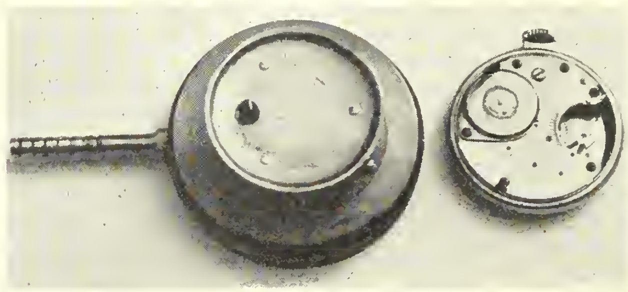


FIGURE 5.—THE COUNTER, SHOWING MOUNTING OF THE REMOVABLE REAR COVER OF THE WATCH AND THE ALTERED WATCH.

as one vehicle. Were the counter used on roads carrying appreciable numbers of three-axle vehicles, the count registered would be excessive since two three-axle vehicles would be counted as three vehicles. In such instances it would be necessary to estimate the percentage of three-axle (or four-axle) vehicles in the total traffic and adjust the count accordingly.

In making a traffic count with this counter an initial reading of the watch is made at the start of the counting period and a final reading is made at the end of the period, recording the hours, minutes, and seconds at each time. The number of seconds indicated by the difference between the two readings represents the number of two-axle vehicles passing that point during the period. Since each minute indicated on the counter represents 60 vehicles, it is obvious that counts for short periods, even on heavily traveled roads, would not move the hands forward by more than an hour or two. A complete revolution of the hour hand, 12 hours, would indicate passage of 43,200 two-axle vehicles. Since the type of watch used will run more than 24 hours on one winding, more than 86,000 vehicles could be counted before the counter would require attention. If the hour hand has made a complete revolution during a counting period, that fact will be revealed by the amount of winding necessary to rewind the watch.

COUNTER QUICKLY AND EASILY INSTALLED

The new counter has many advantages. Since no type of electrical apparatus is involved in its operation, there is no need for a connection with a power line or

(*Continued on p. 203*)

ACCELERATED SETTLEMENT OF EMBANKMENTS BY BLASTING

REPORT ON WORK IN WASHINGTON

Reported by A. W. PARSONS, Senior Engineering Aide, District 1, Public Roads Administration

BLASTING has been used as a means of accelerating fill settlement in a number of States during recent years. By both displacing and liquefying unstable materials under and adjacent to fills, blasting enables stable fill materials to be placed upon firm strata underlying peat bogs and muck beds.

Blasting was recently resorted to in effecting fill settlement on sections of Washington Primary State Highway No. 9 between Quilcene and Sequim.

Traffic is fairly heavy on this highway, especially during the summer tourist season. Glacial drift constitutes the surface material of the region traversed by the highway. The terrain is rolling, with many glacial marshes and lakes in the depressions, and the highway crosses several of these marshes in order to preserve minimum curvature in alinement.

The swamps probably originated through sedimentation of lakes and ponds with detritus from surrounding areas. Silts and clays deposited during floods covered the vegetable matter collected in the depressions, with the formation of peat as the result of incomplete oxidation of the organic matter under water. Thus the swamps have been built up gradually with alternate layers of peat and silt or clay.

First construction financed in part with Federal funds on the section of highway (Washington Federal-aid project No. 136–B) consisted of 6.53 miles of grading to a width of 28 feet. The contract for this project was awarded in April 1927, and the work was carried on during the construction seasons of 1927 and 1928. Considerable difficulty was encountered in constructing the embankments across the marshes, and it became immediately apparent that the quantities originally estimated would fall far short of actual needs because of extreme subsidence of the fills.

Conditions at that time were most critical between stations 125 and 140. There the highway crossed what appeared to have been formerly an arm of Lake Leland, now filled to a depth of 35 to 50 feet with soft blue clay and peat. Preliminary soundings had been made at a stream crossing near station 130, but subsurface explorations were not made through the entire section as the surface appeared sufficiently dry and stable to support adequately the proposed low fill.

The available funds were exhausted without completing the embankment between stations 126 and 139 under the original contract. A second contract was therefore awarded in May 1929, for construction of the remaining portion of the fill and for crushed stone surfacing of the entire project. A limited amount of blasting was done to obtain settlement of the fill through the swamp, and at the time of completion (November 1929) it was believed that the embankment had been permanently stabilized.

Further settlement occurred, however, and additional work was done by State maintenance forces from time to time, including some underfill blasting. The roadbed between stations 126 and 139 now appears to be quite stable and further blasting is not considered necessary.

Between stations 227 to 240 and stations 248 to 265, subsidence of the embankment during original construction was so slow that it was believed that the swamp was shallow and that the roadbed would soon become stabilized. During the time since the original construction was completed the fills have continued to settle slowly, resulting in impaired riding qualities of the surface and high maintenance costs. It was on these two sections, therefore, that blasting was done to settle the fills to solid bottom. Figure 1 shows the road before the new work was begun.

A contract was awarded in September 1938 for regrading 5.821 miles of the original project (stations 29 to 337) to a width of 28 feet, for reinforcing the subgrade with selected gravel, and for placing crushed stone surfacing 20 feet wide. No changes in alinement were contemplated, but several revisions in grade were made through low sections where seasonal floods had covered the existing road. Accelerated settlement of the existing fill and new embankment material to be placed between stations 227 to 240 and stations 248 to 265 were provided for in the contract, the estimated cost being $10,000.

UNDERFILL BLASTING SELECTED AS BEST METHOD OF STABILIZING FILL

Assumptions made in designing the reconstruction of the embankments through the swamp areas were based upon preliminary soundings and results of laboratory tests on the various materials encountered. Soundings indicated that the existing fill extended some 8 to 10 feet below the swamp surface, and that the depth from the swamp surface to solid bottom ranged from 15 to 28 feet. The existing fill was therefore floating on a layer of swamp muck and peat averaging about 10 feet thick.

Profiles of the solid swamp bottom, the surface of the old road, and the new road surface, are shown in figure 2. A cross section showing the old and new road surfaces and the holes for mat and underfill blasting are shown in figure 3.

Samples of the swamp material taken from borings were tested in the laboratory. Table 1 shows the results of these tests.

The swamp material was principally peat mixed with brown silty clay near the surface. Solid bottom consisted of firm blue clay. The swamp material was classified as group A–8. As shown in table 1, consolidation of the swamp material could be expected to continue for a long time if left under the fill. The cost of displacing the peat with suitable fill material was therefore considered justified.

TABLE 1.—*Results of consolidation tests on swamp material*

Laboratory tests			Probable field results	
Time	Consolidation		Time	Consolidation
Minutes	*Inches*	*Percent*	*Years*	*Inches* [2]
SAMPLE 1 [1]				
0.5	0.0162	24	0.13	2.9
1.0	.0220	33	.25	4.0
1.5	.0265	40	.39	4.8
2.0	.0300	45	.50	5.5
3.0	.0354	53	.76	6.5
5.0	.0420	63	1.26	7.7
7.0	.0463	70	1.76	8.4
10.0	.0503	76	2.52	9.2
15.0	.0544	82	3.78	9.9
20.0	.0572	86	5.04	10.4
30.0	.0604	91	7.56	11.0
45.0	.0634	96	11.34	11.6
60.0	.0663	100	15.12	12.1
SAMPLES 2A AND 2B [3] (AVERAGE)				
0.5	0.0082	14	0.29	2.3
1.0	.0114	19	.58	3.2
1.5	.0138	23	.87	3.8
2.0	.0159	27	1.17	4.4
3.0	.0195	33	1.75	5.4
5.0	.0252	43	2.92	7.0
10.0	.0348	59	5.83	9.6
15.0	.0408	69	8.75	11.3
20.0	.0448	76	11.66	12.4
30.0	.0509	86	17.49	14.0
45.0	.0558	94	26.24	15.4
60.0	.0593	100	34.98	16.4

[1] Taken at station 257+00.
[2] Per total depth of peat stratum.
[3] Taken at station 229+00.

Roadside pits at station 220 and between stations 241 and 248 were selected as sources of borrow material for the fill. Laboratory tests on samples of material from these pits showed the material to be not well suited to the intended use. The top 12 feet of material from each pit consisted of brown cloddy soil. The material below consisted of a gray shale, very hard in its original position but slaking readily in water. These soils were found to have the composition and characteristics given in table 2.

Figure 4 shows materials representative of those obtained from the upper and lower portions of the pit.

TABLE 2.—*Composition and characteristics of materials in borrow pits*

	Material in top 12 feet	Material below top 12 feet
Coarse sand .... percent..	5	3
Fine sand .... do....	13	11
Silt .... do....	53	59
Clay .... do....	29	27
Liquid limit	36	36
Plasticity index	12	15
Field moisture equivalent	25	23
Shrinkage limit	18	16
Centrifuge moisture equivalent	15	15
Classification	A–4	A–4

The characteristics of the materials listed in table 2 are indicative of a silty clay soil without coarse material, possessing moderate cohesion and having no appreciable elasticity but important capillary properties with resulting tendency to frost-heave. The pit material is hard in its original position and breaks into fairly large pieces, but its proneness to slake indicates that it would lose stability in embankments under extreme moisture conditions.

This material was not considered particularly suitable for use as fill material to displace the peat; but observations of the same material used in roadway fills indicated that it might give satisfactory results. Since suitable ledge rock or gravel was not available within reasonable hauling distance, it was decided to use this pit material.

In order to cut off capillarity and decrease the possibility of damage to the surface from frost-heave, a gravel ballast course was placed on top of the clay subgrade, the surfacing materials being placed on this base.

Various methods of stabilizing highway embankments were considered by the State before it selected the underfill blasting method as the one best suited to the conditions. The practicability of using vertical sand drains to remove water from the underlying mud was investigated and rejected because the water level of the swamp often rises above the existing road during the winter months.

SWAMP MAT BLASTED TO CREATE RELIEF DITCHES ALONGSIDE THE ROAD

It was believed preferable to include the settlement of the embankments as a force account item in the contract rather than attempt to specify too closely the procedure to be followed. The number of holes to be drilled through the existing embankment and the amount of powder required could not be accurately determined in advance. Also, by using the force account method the procedure could be changed readily as required by any unforeseen conditions encountered during construction.

The fill settlement work was begun at the north end of the swamp (station 226) and was carried forward progressively toward the south end. The first operation consisted of blasting the swamp mat on each side of the existing road from the toe of the slope out approximately 25 feet. The purpose of this blasting was to break up the top mat of clay and fibrous swamp material, and to liquefy the underlying peat and muck, thus decreasing the side support and promoting lateral displacement of the material under the fill.

Experimental blasting was done to determine what method of loading and quantity of explosives would most economically produce the desired results. Shots were placed in three rows spaced 12 feet apart, the rows being parallel to the center line of the road and the inside row being about 4 feet out from the toe of the slope. Charges were spaced 18 inches apart in the rows, and at intervals of 25 feet lateral rows connected the parallel rows. Each charge consisted of five 1¼- by 8-inch sticks of dynamite, placed end to end. The two end sticks and middle stick in each hole were 40-percent gelatin dynamite, the other two sticks being 50-percent straight nitroglycerin dynamite. The charges extended from a foot below the surface down to a depth of from 4 to 6 feet.

The mat was shot in sections from 100 to 200 feet long. Since the charges were detonated by propagation, only one blasting cap was needed in firing each section.

Results obtained by this method of blasting showed that, although the mat was well broken, most of the material was lifted vertically and fell back into its original position without creating the desired relief ditches along the sides of the fill. This was remedied by eliminating the lateral rows between the inside and center rows of holes, thus isolating the inside row and cutting off propagation to it from the outer rows.

An instantaneous electric blasting cap was then used to detonate the center and outside rows of charges, and

FIGURE 1.—EXISTING ROAD PRIOR TO RECONSTRUCTION. PICTURE TAKEN AT STATION 235, SHOWING CRACKS CAUSED BY UNSTABLE SUBGRADE.

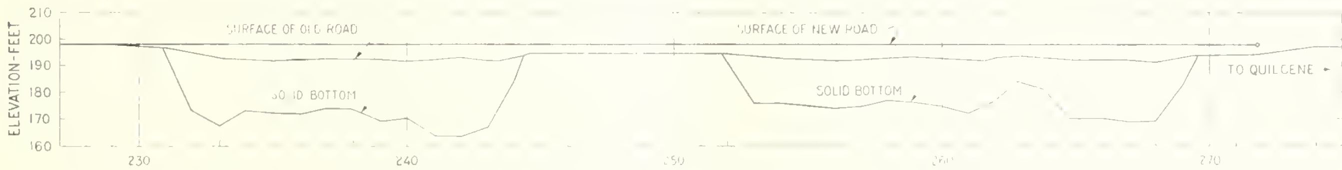


FIGURE 2.—PROFILES OF THE OLD ROAD, THE NEW ROAD, AND SOLID SWAMP BOTTOM.

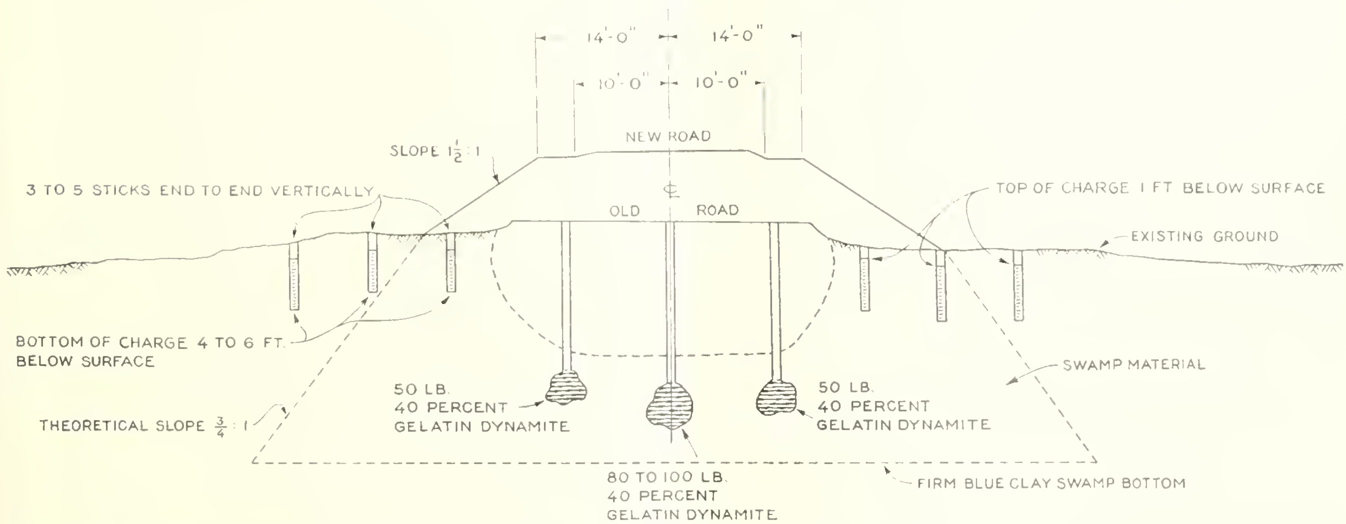


FIGURE 3.—CROSS SECTIONS OF THE OLD ROAD AND OF THE NEW ROAD, SHOWING THE TRANSVERSE POSITION AND DEPTH OF CHARGES FOR SWAMP MAT AND UNDERFILL BLASTING.

a first delay cap was used for the inside row. The two caps were connected in series and fired simultaneously. The delay of approximately 1½ seconds between blasts had the desired effect of throwing the material out and away from the roadway, leaving a ditch along each side of the embankment averaging 4 to 6 feet deep.

In the softer portions of the swamp it was found that satisfactory results could be obtained by spacing the rows of charges about 7 feet apart, and using 3 sticks of dynamite per hole with holes spaced 2 feet apart. Two sticks of 40-percent gelatin dynamite and one stick of 50-percent straight nitroglycerin dynamite per hole were then used.

Figure 5 shows the punching and loading of holes for blasting the swamp mat. Figure 6A shows a blast in

Figure 4.—Fill Material Piled on Road Prior to Underfill Blasting. A, Material From Upper 12 Feet of Borrow Pit; B, Material From Lower Part of Borrow Pit.

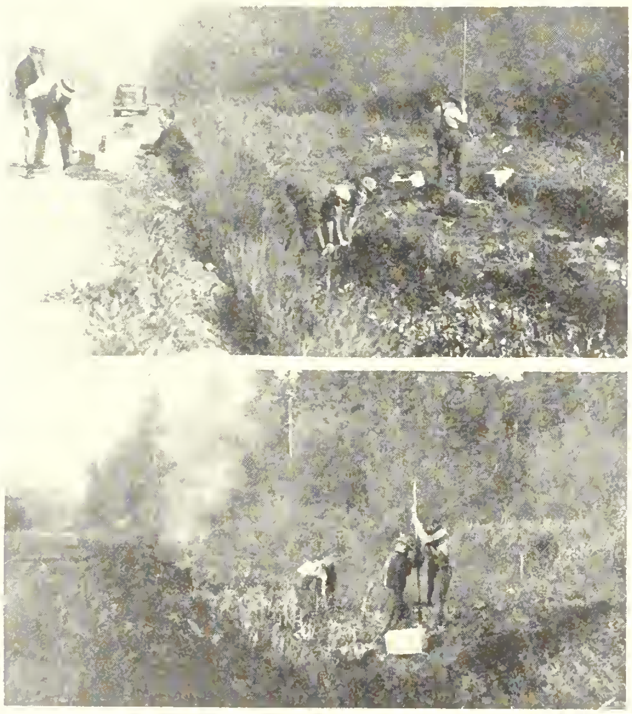


Figure 5.—Punching and Loading Holes for Swamp Mat Blasting.

which a delay cap was used to fire the inside row of charges, and figure 6B shows the relief ditch obtained using this method.

After the crew engaged in blasting the swamp mat had advanced several stations, a second crew began the underfill blasting. A trial section was loaded by driving 2-inch iron pipes under the fill from the sides and loading through the pipes. This method was abandoned after the first trial because it was too slow and difficult to place the charges under the fill effectively.

After charges had been placed under a section of road but before they were exploded, the new fill material was piled on the road, as shown in figure 4. This was done to increase the fill mass in order to direct the force of the explosion outward to the sides, as well as to gain more settlement by impact as the fill dropped into place.

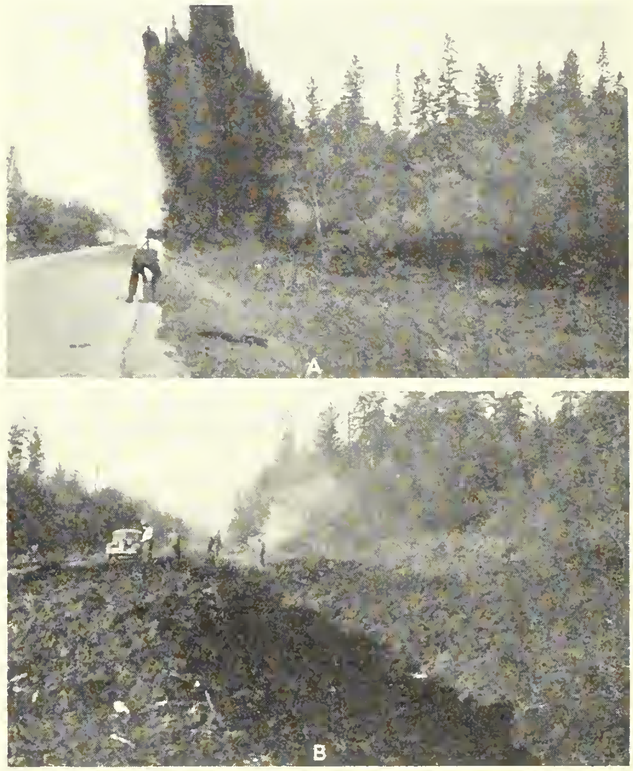


Figure 6.—A, Blast in Which a Delay Cap Was Used to Fire the Inside Row of Charges. B, Relief Ditch Obtained by Blasting the Swamp Mat.

It was found that the time consumed in driving pipes under the toe of the embankment, and in loading the holes and withdrawing the pipes did not allow the blasting crew to keep ahead of the grading operations. A well-drilling outfit was tried and abandoned as too slow. Jackhammers with special 3-inch hardpan bits were finally adopted. Water was poured into the holes during drilling and air jets were used to clean the holes.

After drilling through the compacted fill material, it was frequently possible to air-jet the holes through the underlying peat and muck to the required depth. Loading was done by inserting 2-inch iron pipes in the drill holes and tamping the dynamite into place through the pipes.

Using this equipment and procedure, two drillers in about 8 hours could complete all holes necessary for shooting a section approximately 175 feet long (about 40 holes). The holes averaged about 15 feet in depth, and the cost of drilling was approximately 3½ cents per foot. Five two-man crews could load the holes for a 175-foot section in about 6 hours.

METHODS USED PRODUCED SATISFACTORY RESULTS

Holes were spaced 24 feet apart along the center line of the existing road, and 12 feet apart in rows 11 feet on each side of the center line. Forty-percent gelatin dynamite was used for all underfill shots, 80 to 100

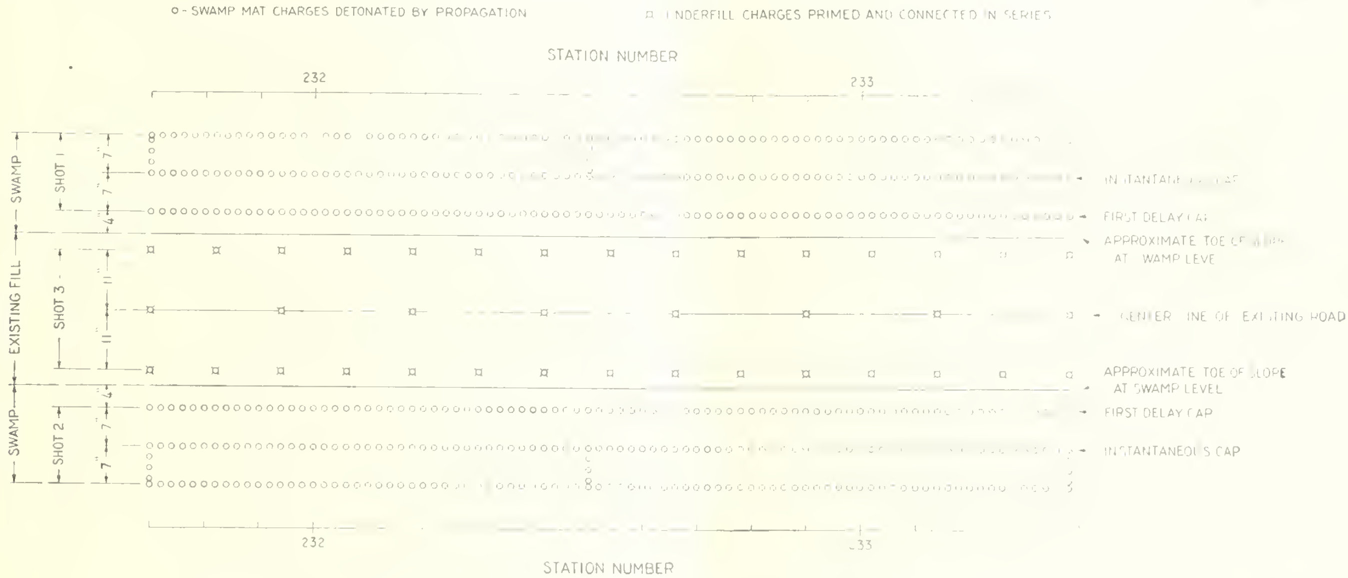


Figure 7.—Locations of Charges for Blasting the Swamp Mat and the Underfill.

pounds per hole being used in the center row and 50 pounds per hole in the outside rows. The depth of the charges varied with the depth of the swamp. The center charges were placed in the lower third of the distance from hard bottom to the bottom of the existing fill, and the side charges were placed at about the midpoint of this distance.

Sections 100 to 200 feet long were blasted at one time, the length being dependent upon the time required to place the borrow material and limited by the capacity of the electric blasting machine. Each charge was primed and caps were connected in series, leaving adequate lengths of wire between caps to prevent breakage by uneven settlement under the weight of the borrow material placed prior to shooting.

A loading diagram for the shooting of a typical section is shown in figure 7. A section of fill ready for blasting is shown in figure 8A, and after blasting in figure 8B.

After completion of the blasting operations described above, additional shooting was done along the edges of the embankment for settlement of the shoulders where such treatment appeared necessary. Holes were drilled through the embankment at the shoulder line, and charges consisting of 15 to 25 pounds of 40-percent gelatin dynamite were placed about 15 feet apart.

The average crew employed on the mat blasting consisted of one powder man and eight unskilled laborers. On the underfill blasting this crew was augmented by two jackhammer operators. The same crew worked on both mat and underfill blasting except for several days at the beginning of the work when it was necessary to use two separate crews to prevent delaying the grading operations.

The following equipment was used on the drilling and blasting work:

- One air compressor and accessory equipment.
- Two jackhammers and special hardpan bits.
- Two iron punch bars, 5 feet long.
- Several sections of ½-inch iron pipe from 10 to 20 feet long, used for air-jetting.
- Several sections of 2-inch iron pipe from 14 to 20 feet long, used as casings for loading dynamite under the fill.

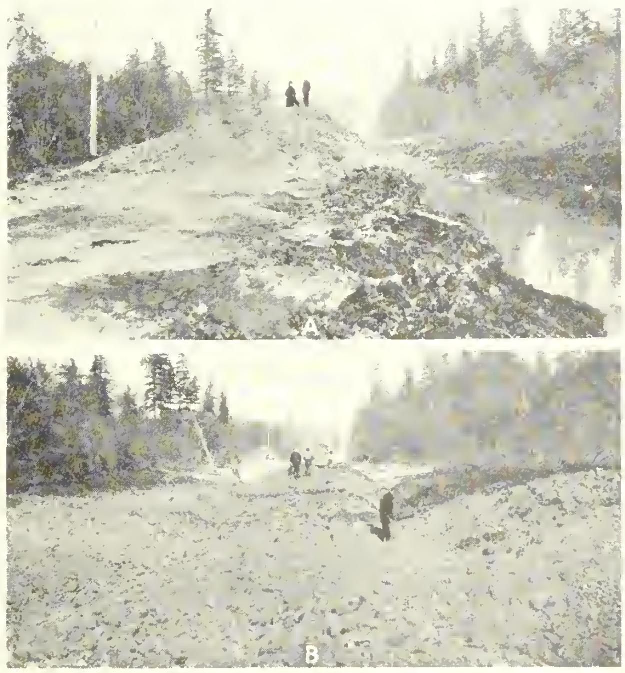


Figure 8.—A, Old Road Loaded With Borrow Material Before Underfill Blasting; B, the Same Section of Road Shown in A After Blasting. The Fill Mass Settled Approximately 15 Feet. Note the Swamp Muck Pushed Out on Each Side.

- Miscellaneous small tools such as shovels, picks, axes, pipe wrenches, etc.

Because the matted root growth and clay overlying the swamp peat averaged about 4 feet thick, the cost of breaking up the mat on this project was probably greater than would ordinarily be expected. The total area of swamp mat blasted was approximately 14,000 square yards. The cost of mat blasting was $2,353, or a unit cost of about 16.5 cents per square yard. Approximately one-half pound of dynamite was used per square yard of mat, or a total of 6,700 pounds.

The original volume of swamp material displaced by

Figure 9.—Completed Road. Picture Shows About the Same Section as in Figure 1.

the embankment was estimated to have been 70,000 cubic yards. The cost of underfill blasting was $7,046. About 11 pounds of explosives were required per linear foot of embankment, or a total of approximately 34,000 pounds. The total cost of mat and underfill blasting was thus $9,399, or 13.4 cents per cubic yard of swamp material displaced.

After completion of the blasting operations and before the fill had been completed to final grade, several heavy rainstorms occurred causing considerable rise of the water level. Between stations 232 and 235 the fill material and adjacent swamp muck softened to the extent that a major subsidence of the fill was started. There was a corresponding upheaval of the peat banks along the sides of the embankment and movement of the swamp surface appeared to extend 100 feet or more from the roadway on each side.

When inspected a few months later, settlement of the fill had stopped and it had been brought back up to grade with gravel hauled from a borrow pit at station 342, approximately 2 miles south. Although it appeared reasonable to expect some additional subsidence, because of the extremely fluid condition of the swamp material through this particular section, there has been no extensive settlement. This may be partially explained by the improvement in drainage conditions along the roadway which have allowed the swamp material to dry out and consolidate, thus increasing the support given to the fill. Placing the gravel borrow over the clay borrow slopes seemed to trap the softer material and prevent lateral flow which might have taken place had the same clay type of borrow been used in raising the fill.

No appreciable settlement has been detected on other sections, probably because the adjacent swamp muck was firmer rather than because of any superiority of fill material.

Crushed stone surfacing was placed several months after completion of the embankments. Figure 9 shows the surfaced road through the swamp ready for a bituminous treatment.

The methods used in swamp blasting have produced generally satisfactory results. The use of rock or gravel as fill material would have been desirable from the standpoints of obtaining maximum settlement during blasting operations and of insuring greater stability under the adverse moisture conditions existing on this road. However, it is questionable whether the considerable expense of hauling more suitable materials to this project would have been justified in view of the relatively large quantity of fill material involved.

(Continued from p. 194)

STANDARDS FOR MARKING AND SIGNING NO-PASSING ZONES

No-passing zones for traffic in either direction on a highway, as defined by the Special Committee on Administrative Design Policies, shall be marked by an auxiliary or barrier stripe placed to the right of the normal centerline, i. e., in the lane of traffic that it is to govern.

The barrier stripe shall be a solid yellow line. In order that the barrier line shall be distinctive, the normal centerline shall be either white or black. It may be of solid or broken type.

The barrier line shall not be narrower than the normal centerline, nor in any case less than 4 inches wide. It should preferably be at least 6 inches wide.

The barrier line shall be separated from the normal centerline by a distance equal to half the width of the centerline.

The combination no-passing stripe shall be identical as applied to both two-lane and three-lane roads.

On a two-lane road the no-passing marking shall separate the two lanes throughout the no-passing zone. On a three-lane road the combination no-passing stripe shall start from the left-hand lane-marking line and extend at an angle of not less than 20 to 1 across the center lane to meet the right-hand lane line at the beginning of the no-passing zone, and thence will extend along the lane line to the end of the zone.

The same design of no-passing stripe shall be used for all types of no-passing restrictions.

The use of signs in addition to the above specified markings to designate no-passing zones shall be governed by local legal requirements or otherwise at the option of the State, but when signs are used they shall conform to the specifications set forth in the Manual on Uniform Traffic Control Devices for Streets and Highways.

(*Continued from p. 196*)

batteries. Consequently, there are no troubles from power failures, short circuits, or the other difficulties commonly experienced in the operation of electrical apparatus. Nor is it necessary that the watch used be an accurate timepiece. Almost any type of watch will prove satisfactory. Clocks, pedometers, or specially constructed counting units might also be used instead of a watch. The watch and rubber tube are the only parts of the counter apt to wear. The most delicate parts of the watch have been removed and the rest receive a negligible amount of wear as compared to their use in a timepiece. Should the watch break down and fail to operate, it can be replaced at small cost. The same type and size of rubber tube may be used in connection with this counter as is employed in other electrical counters using a rubber tube as the detector.[1]

The entire unit is enclosed in an airtight, watertight case that is small enough to be placed or buried on the shoulder of a highway without constituting a traffic hazard. The preferable means of installation, requiring only a few minutes, consists of mounting the counter on the back of a guardrail post or special stake beyond the shoulder where it will be inconspicuous. To prevent tampering with the counter when in operation on a highway, it could be placed in a suitable box having a lock, or extensions could be built on the top and base to enable both to be padlocked to a fixed mounting such as a guardrail or pole.

The far end of the rubber tube should be sealed to prevent the entrance of moisture or dust and for the proper operation of the counting unit.

Care must be exercised in selecting the place of installation on the highway for which a traffic count is desired. Places where the traffic is apt to stop, such as near a traffic signal, or where exceptionally high speeds are common, should be avoided. The best results will be obtained if the rubber tube is placed where the surface is hard and has a smooth, uniform cross-section. On gravel or dirt roads, the installation should be made at culverts, bridges, or other places where the drainage is good and the surface smooth. Places close to intersections where vehicles are apt to pass over the tube at an angle should be avoided as each wheel is apt to cause a separate air impulse, resulting in excessive counts.

Several counts have been made to check the accuracy of the unit. As shown in table 1, the traffic count indicated by the counter was only 0.9 of 1 percent less than the traffic as revealed by a manual count taken simultaneously on a two-lane highway. Part of the error for the individual 5-minute periods was undoubtedly caused by the difficulty of reading the second hand accurately at night. For the two lanes in one direction on a four-lane divided highway where speeds were exceptionally high, the error was −3.8 percent as shown by table 2. The major portion of this error was caused by the wheels of two vehicles traveling in the same direction striking the rubber tube simultaneously creating only one air impulse for two axles with the result that only one or one and one-half counts were registered for the two vehicles depending upon whether one or both axles of the two vehicles passed over the tube at the same instant. This, however, is a fault common to the other types of automatic traffic recorders.

[1] See footnote 1, p. 195.

TABLE 1.—*Comparison of mechanical and manual counts of traffic to check accuracy of watch counter on 2-lane highway*

Time (p. m.)	Manual count	Watch counter	Error
	Vehicles	*Vehicles*	*Percent*
8:45–8:50	54	55	+1.9
8:50–8:55	52	51	−1.9
8:55–9:00	56	54	−3.6
9:00–9:05	43	42	−2.3
9:05–9:10	37	38	+2.7
9:10–9:15	49	48	−2.0
9:15–9:20	28	26	−7.1
9:20–9:25	47	49	+4.3
9:25–9:30	40	40	0
9:30–9:35	28	28	0
9:35–9:40	38	38	0
9:40–9:45	45	45	0
9:45–9:50	35	35	0
9:50–9:55	56	55	−1.8
9:55–10:00	63	61	−3.2
8:45–10:00	671	665	−0.9

TABLE 2.—*Comparison of mechanical and manual counts of traffic to check accuracy of watch counter on 4-lane divided highway—1-direction traffic only*

Time (p. m.)	Manual count	Watch counter	Error
	Vehicles	*Vehicles*	*Percent*
5:25–5:30	37	37	0
5:30–5:35	90	85	−5.6
5:35–5:40	18	18	0
5:40–5:45	34	32	−5.9
5:45–5:50	33	31	−6.1
5:50–5:55	33	32	−3.0
5:55–6:00	42	41	−2.4
5:25–6:00	287	276	−3.8

The use of one counting unit on each end of the tube with a dead space of 5 feet at the center formed by plugs in the rubber tube is recommended for obtaining accurate counts on four-lane undivided highways carrying heavy traffic volumes.

As yet only a few counters have been made in the shops of the Administration for testing and development purposes. These models are being thoroughly tested. The counter is not yet available through commercial sources, although it is expected that it will soon be made available at a low price. Application for a patent has been made in the name of the author. The application guarantees free use of the invention by or for the Federal Government.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF NOVEMBER 30, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 3,218,451	$ 1,547,890	171.1	$ 6,853,609	$ 3,406,663	199.6	$ 769,690	$ 381,640	20.9	$ 2,401,970
Arizona	1,452,892	1,031,348	71.6	1,708,823	1,151,634	81.7	385,779	238,721	25.6	516,121
Arkansas	4,879,438	3,861,773	226.0	527,180	407,619	15.3	1,275,964	667,932	53.7	291,074
California	4,409,635	2,408,420	71.2	4,130,491	2,202,913	58.4	1,327,309	695,549	40.1	2,357,057
Colorado	2,182,539	1,206,875	60.2	3,115,307	1,743,743	66.9	166,665	57,849	5.1	1,596,739
Connecticut	357,558	176,334	5.1	1,964,755	977,627	19.9				1,237,956
Delaware	625,675	297,212	23.7	1,169,783	582,478	14.7	181,020	90,510	.1	1,019,316
Florida	595,100	296,978	8.1	3,810,114	1,904,832	66.2	894,037	447,019	24.1	2,306,661
Georgia	3,143,540	1,566,737	171.6	5,919,669	2,959,835	329.4	1,929,157	964,579	60.1	4,814,681
Idaho	1,749,756	1,050,550	101.1	1,113,339	677,696	50.4	337,570	205,639	36.8	980,872
Illinois	5,855,313	2,918,619	135.5	7,173,759	3,585,637	143.1	3,666,958	1,832,085	59.4	1,298,077
Indiana	2,358,830	1,177,351	38.9	6,209,533	3,098,360	140.2	418,794	209,322	8.6	1,814,169
Iowa	3,342,987	1,559,524	166.8	3,868,035	1,677,848	117.3	206,441	96,900	8.3	829,304
Kansas	2,786,800	1,385,688	157.4	2,256,696	1,127,469	104.5	3,162,223	1,578,251	141.4	3,797,902
Kentucky	2,222,696	1,100,489	76.0	2,905,069	1,450,979	67.2	906,765	453,382	22.9	2,696,361
Louisiana	318,148	156,000	10.8	12,078,255	3,088,003	49.7	1,743,135	854,401	50.6	2,276,974
Maine	2,136,361	1,064,144	51.2	768,470	384,235	17.9	13,240	6,620	.5	299,510
Maryland	1,535,408	732,386	23.9	2,536,873	1,250,055	39.5	290,000	139,500	3.9	1,791,766
Massachusetts	3,134,614	1,564,618	25.1	647,015	322,744	4.5	1,243,094	619,260	7.6	2,519,340
Michigan	2,960,062	1,444,239	72.2	3,759,484	1,879,550	125.7	988,800	393,833	15.7	2,517,094
Minnesota	3,914,504	1,940,162	289.7	5,296,979	2,628,778	239.6	1,014,974	506,543	61.9	2,989,091
Mississippi	948,200	314,460	48.4	8,876,358	3,373,795	349.4	599,600	281,500	16.7	2,046,592
Missouri	2,057,951	1,025,912	90.4	4,672,196	2,300,928	154.2	2,901,785	1,178,505	91.9	3,817,983
Montana	2,904,884	1,643,395	183.1	2,201,558	1,247,360	95.5	1,146,906	650,524	95.6	3,116,217
Nebraska	2,026,959	1,008,037	152.0	5,387,012	2,692,858	509.8	2,020,874	865,555	209.1	2,466,508
Nevada	1,072,038	920,633	53.0	794,720	682,311	32.1	354,046	304,152	21.0	603,242
New Hampshire	672,826	330,405	22.3	788,650	387,136	17.1	86,349	42,787	3.4	966,248
New Jersey	400,110	191,531	3.6	4,583,898	2,290,399	34.7	21,340	10,670		1,828,184
New Mexico	1,724,537	1,061,409	127.5	699,730	427,787	24.3	844,449	527,020	80.7	817,582
New York	7,338,759	3,636,849	145.2	12,153,702	5,921,777	185.3	1,727,650	707,345	22.5	486,360
North Carolina	3,249,800	1,622,510	204.6	5,839,388	2,912,602	304.5	916,776	442,320	39.4	1,111,227
North Dakota	209,510	112,244	35.5	1,308,595	701,251	84.2	2,177,590	1,167,133	238.4	3,364,995
Ohio	4,339,108	2,169,554	53.5	7,648,308	3,741,420	79.1	5,556,180	2,641,370	41.7	4,448,560
Oklahoma	1,482,682	787,107	64.9	2,653,866	1,407,649	91.7	2,424,065	1,259,142	84.6	2,888,761
Oregon	1,929,375	1,150,917	100.9	2,693,391	1,548,570	86.7	1,320,017	689,370	39.3	712,328
Pennsylvania	7,120,590	3,525,680	86.3	6,821,050	3,255,169	63.5	2,595,332	1,276,222	29.7	3,068,718
Rhode Island	601,970	300,865	7.8	872,308	435,441	8.3	78,630	39,315	.6	868,267
South Carolina	1,418,740	639,800	64.1	1,646,174	729,986	43.3	642,190	284,300	55.9	2,118,597
South Dakota	3,067,499	1,696,689	287.8	2,948,430	1,650,140	324.1	1,425,600	801,010	167.4	2,622,059
Tennessee	3,017,948	1,437,042	65.8	3,030,140	1,515,070	68.2	1,836,086	918,043	29.2	2,927,279
Texas	9,112,388	4,478,606	518.7	7,094,407	3,527,453	300.7	6,136,489	2,847,438	286.7	3,318,462
Utah	1,887,504	1,353,102	93.0	972,930	679,480	51.1	130,005	93,854	1.6	646,965
Vermont	737,850	361,494	18.4	716,604	358,112	22.7	73,574	36,765	3.0	313,059
Virginia	1,814,930	905,368	59.7	2,470,351	1,186,457	57.7	1,390,074	687,578	37.4	564,334
Washington	1,968,390	1,011,949	33.3	2,726,010	1,301,339	15.6	1,611,317	681,928	24.4	268,439
West Virginia	893,827	483,575	28.5	2,579,755	1,307,114	64.2	733,160	362,275	19.0	1,812,332
Wisconsin	4,769,973	2,344,836	167.4	5,384,113	2,646,310	171.5	154,090	74,165	4.6	1,644,069
Wyoming	1,443,264	895,934	133.0	1,079,572	670,788	112.0	674,561	426,466	45.0	195,323
District of Columbia	137,000	68,500	.8	265,124	132,562	1.9	235,200	117,600	2.3	168,838
Hawaii	141,181	66,938	1.3	1,001,292	488,062	16.4	571,890	282,525	9.8	1,054,625
Puerto Rico	655,310	326,655	13.8	1,357,219	671,980	25.6				376,160
TOTALS	122,325,410	64,359,333	4,821.8	179,080,089	86,700,004	5,347.1	61,307,440	30,136,412	2,348.2	90,994,348

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF NOVEMBER 30, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 215,805	$ 106,600	22.2	$ 961,062	$ 375,050	20.6	$ 67,300	$ 33,650	6.9	$ 709,956
Arizona	198,802	143,374	21.8	99,081	70,942	11.6				325,941
Arkansas	766,428	634,097	70.9	199,605	195,349	24.3	57,560	46,863	5.0	140,811
California	937,631	481,788	40.9	383,054	209,124	9.6	158,068	84,262	3.7	586,359
Colorado	704,902	368,820	25.7	330,983	186,024	10.9				31,948
Connecticut	163,280	67,660	2.9	110,274	54,954	.2				239,855
Delaware	80,840	40,420	17.5	69,537	34,768	7.8				233,447
Florida	286,228	138,856	8.8	710,218	354,651	28.9				371,538
Georgia	229,297	111,062	28.0	269,282	134,641	31.3	355,825	177,913	43.2	904,360
Idaho	310,116	187,461	35.9	226,382	115,065	16.0	88,451	49,635	6.0	84,713
Illinois	1,040,407	517,575	48.5	1,018,000	455,000	73.8	878,550	437,975	38.4	208,727
Indiana	749,553	371,415	61.3	442,370	219,981	34.7	112,122	56,050	9.4	641,801
Iowa	88,190	41,004	44.1	805,421	409,200	113.1	637,523	300,000	104.2	929,603
Kansas	78,622	39,311	43.9	174,250	87,125	4.3	539,934	269,967	20.4	1,185,543
Kentucky	511,845	158,063	52.5	1,135,905	349,982	56.0	377,638	116,918	39.6	223,613
Louisiana	668,295	313,982	57.9	266,920	125,888	20.9	234,263	107,856	18.7	284,846
Maine	432,057	215,560	25.4	68,900	33,244	3.5	8,300	4,150	.5	7,660
Maryland	204,891	98,787	16.0	124,696	50,348	4.8	132,000	48,355	9.3	338,191
Massachusetts	313,052	155,673	7.6	344,720	170,445	7.6	190,910	94,705	4.2	368,845
Michigan	836,852	409,248	64.0	1,078,982	539,491	81.3	38,400	19,200	7.9	769,295
Minnesota	513,256	254,584	35.3	654,706	327,353	95.6	223,841	68,041	21.5	1,008,527
Mississippi	176,500	88,250	6.8	828,662	408,046	68.3	314,260	157,080	17.8	497,490
Missouri	715,897	346,306	101.0	725,253	333,413	70.4	145,614	58,037	26.9	529,972
Montana	746,984	423,684	68.8	102,625	58,143	2.8	177,124	100,464	20.8	749,737
Nebraska	589,903	285,221	119.5	692,092	338,871	114.6	189,161	94,580	29.9	297,651
Nevada	160,893	136,525	25.0	70,067	60,261	18.1	36,557	31,421	2.0	113,649
New Hampshire	61,156	29,708	2.3	83,280	39,815	3.1				150,441
New Jersey	298,990	146,755	10.2	277,250	138,625	14.2	75,270	37,635	.1	471,250
New Mexico	466,270	286,858	42.1	27,020	15,690		361,210	146,391	26.9	80,519
New York	1,448,311	715,796	77.5	2,008,535	957,323	58.5	91,100	45,550	.6	210,481
North Carolina	957,064	478,510	87.5	476,700	238,350	43.9	139,380	57,542	15.5	271,163
North Dakota	115,030	61,606	8.3	37,500	20,100		111,270	59,617	10.9	819,207
Ohio	457,120	227,250	27.2	673,870	343,710	27.9	802,000	401,000	27.8	1,323,392
Oklahoma	99,638	48,414	4.7	333,896	177,663	16.6	657,580	349,882	45.1	751,486
Oregon	589,088	333,232	62.3	234,760	104,050	28.8	45,249	27,160	7.1	274,460
Pennsylvania	1,905,307	940,977	109.3	1,320,516	654,153	46.7	237,638	118,819	8.8	169,157
Rhode Island	93,827	46,890	2.1	81,236	40,618	.2	36,060	18,030	.4	78,277
South Carolina	562,159	228,890	56.9	94,120	33,479	8.3	274,000	88,000	21.2	235,551
South Dakota	16,170	8,890	4.1	11,056	6,088					1,043,072
Tennessee	812,946	353,609	31.7	146,416	73,208	.7				759,721
Texas	1,823,247	893,485	209.7	935,698	450,765	82.6	499,600	237,885	60.2	738,198
Utah	224,185	126,765	38.5	100,365	61,098	6.2	38,880	22,000	2.1	122,199
Vermont	145,522	71,278	5.6	164,292	50,336	6.3				54,849
Virginia	632,234	306,985	65.5	126,550	63,275	6.2	310,590	136,228	17.8	172,303
Washington	585,044	304,627	43.1	277,203	145,318	16.9				218,057
West Virginia	145,150	72,575	8.3	161,665	80,832	8.6	211,788	105,894	11.5	333,195
Wisconsin	803,000	399,787	31.6	408,222	203,760	7.3	391,685	181,540	4.7	465,158
Wyoming	466,828	288,087	26.0	159,738	100,841	10.4	240,660	123,954	28.6	5,103
District of Columbia				117,692	58,346	1.2				14,779
Hawaii	89,392	44,391	3.7	206,686	103,891	4.6	164,258	82,085	6.4	89,633
Puerto Rico				224,464	109,130	12.8	55,188	27,140	2.1	60,233
TOTALS	24,518,204	12,550,691	2,010.4	20,641,777	9,967,823	1,343.0	9,706,807	4,623,474	734.1	20,816,082

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF NOVEMBER 30, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR		NUMBER			UNDER CONSTRUCTION		NUMBER			APPROVED FOR CONSTRUCTION		NUMBER			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$538,658	$526,159	5	2		$743,712	$742,184	11			$42,800	$42,800	2		4	$792,946
Arizona						518,061	515,813	4								209,120
Arkansas	189,891	184,017	3			669,853	663,348	3			24,235	24,235	2		8	607,099
California	749,582	749,532	6			1,110,078	1,109,033	7		1	423,597	261,250	1			1,003,772
Colorado	612,531	612,529	5		11	15,924	15,924			3	36,358	32,599			11	791,132
Connecticut						184,564	172,850		1	4	9,492	9,492			3	829,223
Delaware						7,839	7,839	2			2,320	2,320			1	515,203
Florida	219,800	219,800	1			404,947	400,448	3			11,800	11,800			3	1,034,542
Georgia	143,190	143,190	5			396,847	396,847	6	1		437,342	437,342	5	4	21	1,888,641
Idaho	191,612	160,443	3			122,880	122,518	1								466,886
Illinois	2,045,535	2,044,675	13	3	43	1,633,089	1,494,717	8	1	22	1,060,122	897,330	3		32	1,256,783
Indiana	752,205	752,205	3	1	52	536,953	536,953	3		26	291,001	291,001		1	56	722,271
Iowa	365,439	344,400	10		8	503,354	452,506	4		102	597,165	562,050	2		126	743,718
Kansas	934,191	934,191	11		5	429,455	429,455	4			443,949	443,948	7		10	677,069
Kentucky	487,880	483,579	7	4	1	792,807	792,807	7			279,437	279,437	2		15	366,810
Louisiana	122,838	122,830	2			824,494	770,989	7			317,665	317,659	10			584,469
Maine	331,672	329,136	3	2		206,701	206,701	2	1		924	924			1	235,386
Maryland	24,510	15,403				240,795	240,795	2		2	161,200	161,200	1		13	810,099
Massachusetts	265,259	264,538	1	2		257,307	256,764	3			14,320	14,320	1			1,711,447
Michigan	460,359	458,019	4	2	14	1,012,900	1,012,900	5		8	209,865	209,865	2		18	1,344,758
Minnesota	277,522	274,394	1	4	12	1,682,384	1,661,460	13	2	5	8,010	8,010			2	978,333
Mississippi						611,373	611,373	8			246,200	246,200	4	1		680,628
Missouri	138,789	137,484	1	1		1,463,326	1,463,326	8	1		410,364	385,086	2	4		1,324,064
Montana	850,426	850,426	9			213,154	98,073	3			80,000	80,000	1			204,589
Nebraska	472,327	471,727	14		4	717,561	717,561	11	1	7	245,919	245,919		1	38	507,106
Nevada	175,349	175,349		2	4	33,771	33,771		1	6	7,695	7,695			3	104,241
New Hampshire	48,623	48,202	5		1	159,766	159,683	5			91,061	91,061	1			222,539
New Jersey	7,140	7,140			1	730,316	730,316	2	3		150,090	150,090	1			1,289,734
New Mexico	59,805	59,805	2			15,276	15,276				64,518	64,518	1		1	630,719
New York	1,481,630	1,477,930	4	5		2,125,512	2,087,612	9	9		439,300	401,300	2	2		3,037,532
North Carolina	714,864	682,164	4	3	26	908,480	906,080	9	3	10	371,420	371,420	2		30	581,579
North Dakota	105,450	105,450	3	1		818,489	770,087	9								467,161
Ohio	447,210	432,210	7	1		1,502,813	1,431,861	8	2		710,390	710,390	5		5	2,488,560
Oklahoma	266,738	266,738	3		32	187,025	187,025	3		11	419,200	418,300	9	1	8	1,801,271
Oregon	40,500	39,002				266,498	265,203	3								311,060
Pennsylvania						2,344,004	2,132,105	6	4		373,499	373,499	6		2	4,157,624
Rhode Island	431,385	431,385	1	3		7,406	7,406									152,459
South Carolina	246,354	212,972	5	2	7	577,216	554,880	5	4		174,275	174,275		3	26	732,323
South Dakota	207,373	207,373	2	2	16	201,265	201,265	2	1	1	47,550	47,550	1			996,027
Tennessee	79,920	79,920		1	3	786,478	786,478	3	3		6,760	6,760			2	1,339,352
Texas	1,285,296	1,254,240	13	2		2,238,728	2,187,942	17	1		426,767	392,610	4		26	1,560,047
Utah	148,711	148,648	2		48	168,188	168,188	1		58	102,610	102,610			37	177,622
Vermont	34,693	29,881			7	120,402	120,402		1		104,700	81,812		1		119,216
Virginia	587,309	498,309	6	2	17	241,181	237,281	3	1	1	94,395	94,395	2		2	839,081
Washington	201,163	199,751	2		11	136,831	136,831	1	1		196,172	196,172	1	2	1	378,313
West Virginia	64,417	64,417	2			310,434	294,674	5			20,391	20,391	1		3	968,121
Wisconsin	480,877	478,594	6		2	1,065,764	1,021,834	9	1	9	659,196	604,736	3	2	3	662,639
Wyoming	137,852	137,696	1	1	7											518,179
District of Columbia						366,812	333,268	1								47,053
Hawaii	52,950	50,320	1			134,312	134,312	3	1		6,216	6,216			1	351,772
Puerto Rico	49,040	48,840	1			345,312	343,310	8								426,676
TOTALS	17,528,865	17,215,013	177	46	332	31,092,637	30,140,274	237	44	276	9,820,290	9,280,587	84	22	512	44,646,994

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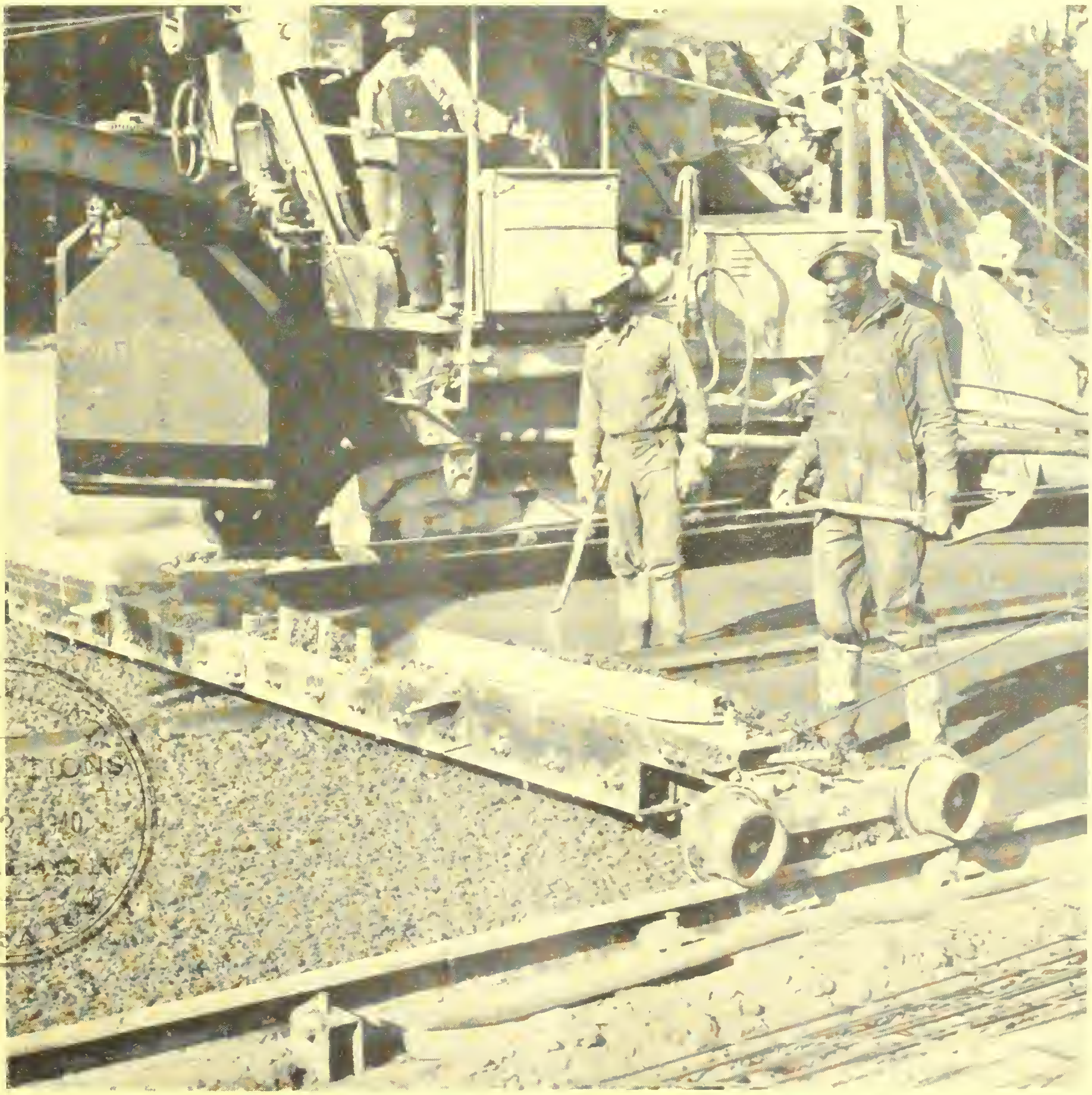

CONSTRUCTING EXPERIMENTAL CONCRETE PAVEMENT IN INDIANA

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

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PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 20, No. 11 — January 1940

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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CERTIFICATE: By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

EXPERIMENTS WITH CONTINUOUS REINFORCEMENT IN CONCRETE PAVEMENTS[1]

Reported by EARL C. SUTHERLAND, Associate Highway Engineer, Public Roads Administration, and SANFORD W. BENHAM, Research Engineer, Indiana Highway Commission.

IN THE FIELD of concrete pavement design, there is probably no subject that has provoked more discussion than that of the proper spacing for transverse joints. Today, after more than 40 years of concrete pavement construction, a wide divergence of opinion still exists as to the proper spacing to use.

The general trend for some years has been toward decreased slab lengths. Gradually the distance between joints has been reduced until at the present time most of the concrete pavement is being laid with slab lengths of 30 feet or less. Theory and experiment have indicated that for satisfactory control of the stresses that are caused by restrained temperature warping a short slab length is necessary.

A pavement designed with short slab lengths obviously contains numerous joints. Joint designs that will fulfill the requirements for flexibility, for ability to transfer load and effectively to control load stress, and for excluding water and foreign matter in a satisfactory manner, have not yet been developed.

There exists an understandable feeling that the trend toward increasing the number of joints required per mile of pavement is a mistake, although present engineering knowledge indicates this to be the most effective design. The cost, the difficulties of installation, the leakage of surface water to the subgrade (sometimes with very harmful results), the tendency for joints to be or to become rough spots producing impact—these and other criticisms are advanced as arguments against the use of short slabs and frequent joints. If it were possible to design a concrete pavement in such a manner that it would be continuous mile after mile, or even if it were possible to space the joints at intervals of 1,000 feet or 500 feet or even less, it is obvious that many of the problems that are associated with frequent joints would disappear. Although this thought has intrigued the minds of engineers for years, the solution to the problem has not been found.

Experience has shown what is to be expected in plain concrete pavements that are laid continuously or with joints placed at infrequent intervals. Contraction and warping stresses cause frequent transverse cracks and lack of provision for expansion may result in "blow-ups". While these troubles have not always been experienced to a serious degree, in general, those interested in the construction and maintenance of highways have felt that the effort to control cracking and other troubles through the use of joints was worthwhile.

The possibilities of pavement slab designs in which the frequency of constructed transverse joints is reduced through the use of continuous, bonded-steel reinforcement have never been very thoroughly explored, although a limited amount of information is available.

Concrete changes in volume when subjected to temperature changes or to changes in its moisture content. A concrete road slab is subject to severe changes in both temperature and moisture content and relatively large volume changes tend to occur. When the slab attempts to contract or to expand, it must overcome the resistance to deformation of the subgrade with which it is in contact. This is the source of the direct tensile and compressive stresses in a transverse section of the pavement. In addition to direct stress, temperature and moisture differentials down through the slab develop periodically and these create bending stresses because of the restraint to warping that exists in slabs of appreciable length. The summation of the stresses that are caused by these conditions of temperature and of moisture, either alone or in combination with the stresses that are caused by vehicle wheel loads, may exceed the tensile strength of the concrete and cause a rupture of the pavement slab.

The introduction of longitudinal steel reinforcement into a concrete pavement will not prevent the formation of transverse cracks under the stress conditions just described. However, the presence of steel will have an important effect upon the character and distribution of the cracking that occurs and upon the structural integrity of the pavement and it is this action that possibly may prove to be of advantage in reducing the number of constructed transverse joints required in concrete pavements.

REINFORCING STEEL EXPECTED TO INFLUENCE THE CHARACTER OF CRACKING

Moderate amounts of steel reinforcement in concrete under tension add relatively little to the tensile resistance of the section prior to the time that rupture of the concrete occurs. Thus, in a concrete pavement, it is to be expected that the first crack will appear at about the same place and under about the same conditions of stress in both a reinforced and an unreinforced slab, other conditions being the same. As soon as rupture of the concrete occurs, however, the concrete section that was most highly stressed is relieved immediately of all stress while any bonded steel that crosses the ruptured section suddenly assumes a much greater burden than before.

The stresses caused by subgrade resistance are carried across the rupture by the steel and transferred gradually back to the concrete through the bond between the two materials. The distance in which the full amount of stress is transferred back to the concrete is dependent directly upon the length of the section and the degree of bond between the two materials and indirectly upon the amount of steel. The result is that in long slabs reinforced with relatively large amounts of longitudinal steel, additional transverse cracks may be expected in close proximity to the first plane of rupture. The distance between the points at which the stress in the concrete section reaches a magnitude sufficient to cause rupture will depend primarily upon the amount of steel that transfers stress across the ruptured concrete section, although for a given percentage of longitudinal steel a certain slab length is necessary to develop the effect that has been described.

[1] Paper presented at the Nineteenth Annual Meeting of the Highway Research Board, December 8, 1939.

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FIGURE 1.—PRESENT APPEARANCE OF A HEAVILY REINFORCED SECTION OF THE COLUMBIA PIKE EXPERIMENTAL PAVEMENT.

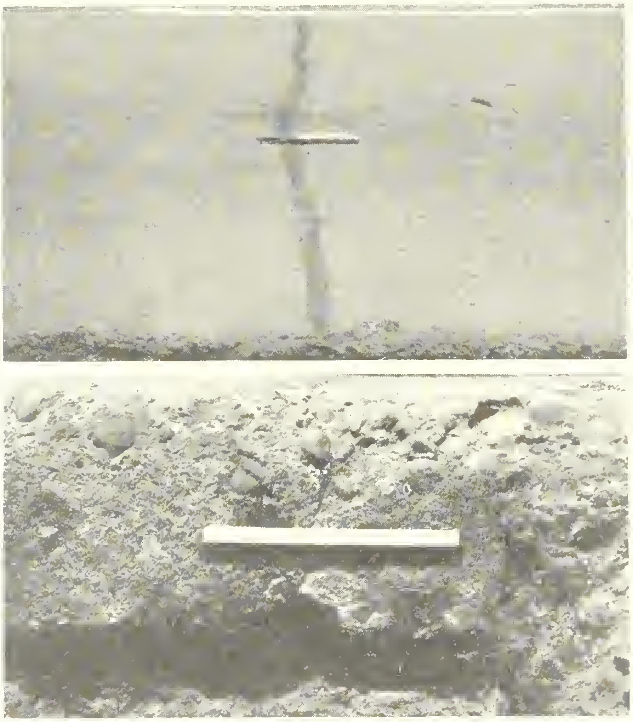

FIGURE 2.—TYPICAL CRACK IN A HEAVILY REINFORCED SECTION OF THE COLUMBIA PIKE EXPERIMENTAL PAVEMENT AFTER 18 YEARS OF SERVICE. UPPER, SURFACE OF PAVEMENT; LOWER, EDGE OF SLAB.

This action was observed and studied to a limited extent in two of the early researches of the Public Roads Administration, the significant features of which will be discussed briefly.

The earliest of these investigations is that of the Columbia Pike experimental pavement in Arlington County, Va. This pavement, 1¾ miles long, was built in 1921 and included various slab thicknesses, cross-sectional designs, and reinforcement designs. The majority of the reinforced sections are 200 feet long, and the amount of reinforcement in them was varied from a light welded fabric to a heavy bar mat. A published report describes the condition of the pavement at the time it was 2½ years old.[2] The details of design of the various sections of pavement and the characteristics of the subgrade are given in this report. The type of cracking that occurred in relation to the steel reinforcement was discussed in two subsequent papers.[3] [4]

The Columbia Pike experimental pavement is of interest at the present time chiefly because it gives some indication of the type of cracking that occurs and the performance that may be expected in pavements containing relatively large amounts of longitudinal reinforcement.

The most heavily reinforced section is 350 feet long and contains 14 three-quarter-inch round deformed bars, uniformly spaced across its 18-foot width. A large number of very fine, closely spaced, transverse cracks developed in the central part of this section soon after the pavement was laid. After 18 years of service, these cracks are still closed and very little spalling or disintegration has occurred in their vicinity. A general view of this section of pavement taken within the last few months is shown in figure 1. Figure 2 shows two views of the pavement at a typical crack. It will be observed that the general condition of this section of pavement is still good and that the structural integrity of the pavement has been maintained. Although figure 1 indicates distances of several feet between transverse cracks, actually it shows only those that have developed to an extent sufficient to catch the eye of the maintenance crew. Additional transverse cracks can be found by close examination, although they are so fine that they are difficult to see, even in a relatively close-up view such as that in figure 2.

The Columbia Pike experiment indicates that in a long concrete pavement slab the presence of relatively large amounts of longitudinal steel will greatly increase the number of transverse cracks but that these cracks will be held tightly closed (if the slab length is not too great) and further that over a long period of time no structural break-down need be anticipated. It shows, also, that the presence of longitudinal steel reinforcement increases the distance between open cracks.

REINFORCEMENT FOUND TO REDUCE CRACKING IN SHORT SLABS AND NEAR ENDS OF LONG SLABS

Several years after the Columbia Pike sections were built, a study of curing methods for concrete pavements was undertaken at the Arlington Experiment Farm, Virginia, utilizing test sections of plain concrete 200 feet long, 2 feet wide, and 6 inches thick. In connection with this study, a number of reinforced sections were included for the purpose of obtaining additional information on certain conditions that had been observed in the experimental sections on the Columbia Pike. The details of the construction and behavior of the various sections included in the curing experiments have been given in two published reports.[5] [6]

The amount of steel in the reinforced sections was varied from a very light welded fabric to two ¾-inch round deformed bars (in the 24-inch width). There were, as stated, a number of plain concrete sections and these may be used for comparison with the reinforced sections. Some of the 200-foot sections were duplicated by sections that were divided into five slabs: 20, 30, 40, 50, and 60 feet long. The stresses in the steel and in the concrete caused by the subgrade resistance alone probably would not be excessive in slabs up to 60 feet in length. In the 200-foot sections, however, the stresses caused by the subgrade resistance would exceed the ultimate tensile strength of the concrete, over a considerable part of the total length

[2] Reinforcing and the Subgrade as Factors in the Design of Concrete Pavements, by J. T. PAULS. PUBLIC ROADS, vol. 5, No. 8, October 1924.
[3] The Interrelation of Longitudinal Steel and Transverse Cracks in Concrete Roads, by A. T. Goldbeck, PUBLIC ROADS, vol. 6, No. 6, August 1925.
[4] Concrete Pavement Design, by L. W. Teller and J. T. Pauls, Proceedings American Concrete Institute, vol. 22, 1926.
[5] Tests of Concrete Curing Methods, by J. T. Pauls, PUBLIC ROADS, vol. 7, No. 10, December 1926.
[6] The Arlington Curing Experiments, by L. W. Teller and H. L. Bosley, PUBLIC ROADS, vol. 10, No. 12, February 1930.

of the sections, and would probably exceed the tensile strength of the steel in the more lightly reinforced sections.

In the 13 years since the sections were built, a number of transverse cracks have formed in each of the plain concrete sections and in the central portion of the 200-foot sections that are continuously reinforced with the larger amounts of bonded steel. There has been a negligible amount of cracking near the ends of the continuously reinforced sections and in the reinforced sections that were divided by joints into shorter slabs. This condition obtains in both the sections reinforced with large amounts and those reinforced with small amounts of steel.

In general, no appreciable amount of cracking or disintegration has occurred in the reinforced sections in which the stresses in the steel and concrete, caused by the subgrade resistance, have been held to reasonable values.

In the 200-foot sections containing a relatively large amount of steel, in which the stresses caused by subgrade resistance have exceeded the tensile strength of the concrete, but not the elastic limit of the steel, a number of very fine, closely spaced transverse cracks have developed in the central part of the sections. These cracks have remained closed and little or no spalling or disintegration has developed. A typical crack in one of the reinforced sections containing two ¾-inch deformed bars is shown in figure 3.

These experimental sections have never been subjected to loads of any kind so that the cracking and disintegration that has developed has been caused by the temperature and moisture variations and by other conditions of exposure.

The behavior of the reinforced sections in the curing experiments at Arlington tended to confirm the observations made on the Columbia Pike sections adding support to the theory that has been outlined.

These studies and others that were in progress about the same time emphasized the need for a more thorough understanding of the structural action of concrete pavements and for a number of years attention was concentrated on this problem.

Westergaard developed his important analyses of the stress conditions in pavement slabs under the action of wheel loads[7] and under the influence of temperature changes.[8] The first showed for the first time the relations between load, deflection, and stress for an elastic slab of uniform thickness on an elastic support, dealing with the three critical points, a free edge, a free corner, and an interior point. The second paper, dealing with temperature effects, presented means for calculating the stresses caused by warping restraint. Both of these contributions are of fundamental significance.

In order to study experimentally the relations analyzed by Westergaard and to supplement those analyses by studies of other than uniform cross sections, the Public Roads Administration built and subjected to intensive experiment a series of full-size pavement slabs. This general research covered a period of several years and was divided into a number of parts. Those parts of the investigation that are pertinent to the subject of this paper have been reported and cover the observed effects of temperature and moisture variations on slab behavior and the observed structural action of the several joint designs that were included in the design of the test sections.[9] The slab behavior was determined, in general, by strain measurements.

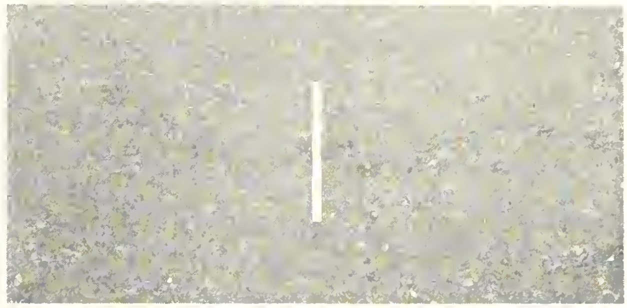

Figure 3.—Typical Fine Crack in a Heavily Reinforced Slab Constructed in the Arlington Curing Tests, 13 Years After Construction.

RESEARCHES SHOW IMPORTANCE OF CONTROLLING WARPING STRESSES

One of the important results of these researches has been development of an appreciation of the importance of the warping stresses that are present in concrete pavements at times when large temperature differentials are present, stresses that in certain regions of the slab area combine with load stresses to cause critical stress conditions. To control warping stresses, the restraint that produces them must be controlled and this can be accomplished practically by limiting the principal dimensions of the slab. The significance of these combined stresses as they affect the structural design of a pavement has been very completely discussed in a recent paper by E. F. Kelley.[10]

Recognizing on the one hand the experimental evidence that short slabs are necessary for the control of warping and consequently of combined stress and on the other hand the practical objections to the introduction of more joints in concrete pavements, the Public Roads Administration and the Indiana State Highway Commission in 1936 decided to investigate more thoroughly the possibilities of longitudinal steel reinforcement as a means for increasing the slab length of concrete pavements.

Consideration of the problem indicated that the information desired could best be obtained through the construction of special sections of reinforced pavement located on a highway in service and on this basis an extensive research project was planned. Arrangements were then made for the construction of the desired experimental reinforced sections in Indiana as part of a regular Federal Aid project. The State and the Administration cooperated in the selection of a suitable location, the adjustment of the experimental sections to the location, the construction of the sections, and in the program of measurements and observations.

The pavement was constructed on a transcontinental highway west of Indianapolis. This location was

[7] Computation of Stresses in Concrete Roads, by H. M. Westergaard, Proceedings of the Highway Research Board, pt. 1, 1925.
Stresses in Concrete Pavements Computed by Theoretical Analysis, by H. M. Westergaard, PUBLIC ROADS, vol. 7, No. 2, April 1926.
[8] Analysis of Stresses in Concrete Roads Caused by Variations of Temperature, by H. M. Westergaard, PUBLIC ROADS, vol. 8, No. 3, May 1927.

[9] The Structural Design of Concrete Pavements, by L. W. Teller and Earl C. Sutherland:
Pt. 2. Observed Effects of Variations in Temperature and Moisture on the Size, Shape, and Stress Resistance of Concrete Pavement Slabs, PUBLIC ROADS, vol. 16, No. 9, November 1935.
Pt. 4. A Study of the Structural Action of Several Types of Transverse and Longitudinal Joint Designs, PUBLIC ROADS, vol. 17, Nos. 7 and 8, September and October 1936.
[10] Application of the Results of Research to the Structural Design of Concrete Pavements, by E. F. Kelley, Proceedings of the American Concrete Institute, vol. 35, 1939, also PUBLIC ROADS, vol. 20, Nos. 5 and 6, July and August 1939.

particularly suitable because of the fairly uniform subgrade and the absence of sharp curvature and of steep grades. The maximum grade is 2.95 percent and the maximum curvature is 0°11.5′. Also the traffic on this route is heavy. The test sections were built during September and October of 1938. The remainder of this paper will be devoted to a description of the details of the planning and construction of this pavement, the schedule of observations that has been adopted, and the general behavior of the experimental sections thus far.

In selecting the range in the amounts of steel to be used in the various sections, it was necessary to consider several factors. The maximum amount of steel that can be used will probably be determined by the cost of the steel that can be added and still allow concrete to compete with other types of pavement. There is also a physical limitation to the amount of steel that can be introduced without seriously interfering with the effective placement of the concrete. In order that the steel be adequately protected from corrosion, it must be concentrated in about the middle third of the slab depth. Another factor that influences the distribution and hence the amount of longitudinal steel that can be used is the necessity for some degree of flexibility or hinge action at the transverse cracks for the relief of warping stress.

The minimum amount of longitudinal steel that should be considered is that which might be expected to produce some definite change in the character of the cracking as compared with an unreinforced pavement.

With the data at present available neither the upper nor lower limits can be selected with any degree of precision. For the present study the minimum amount of longitudinal steel was set at No. 6 wire (0.192-inch diameter) spaced at 6-inch centers (22 pounds of longitudinal steel per 100 square feet of pavement). This is equivalent to a 32-pound welded wire fabric, which is about as light as is used in pavements today.

The maximum selected was 1-inch diameter bars on 6-inch centers. This amounts to 534 pounds per 100 square feet and exceeds the amount in any known pavement installation. Between the upper and lower limits of longitudinal steel, intermediate quantities were chosen to give a fairly uniform range of reinforcement. The transverse steel used was the practical minimum thought necessary to secure a good installation of the longitudinal steel.

THREE TYPES OF STEEL USED IN REINFORCING SECTIONS

The number and length of the sections included in the pavement, together with certain details concerning the reinforcement, are given in tables 1, 2, and 3. It will be noted that the sections of pavement included in table 1 are reinforced with welded fabric; those of table 2 with billet steel (intermediate grade), and those in table 3 with rail steel. The three types of steel were included in order that any possible advantages of high elastic limit steel, for the type of construction being considered, might be revealed. The range in steel stress for each class of steel was selected to cover what was considered to be the most effective range. Attention is called to the fact that the range of maximum steel stresses is such as to permit direct comparisons of structural action to be made between sections containing different types or different percentages of longitudinal steel.

The lengths of the slabs necessary to give the desired stresses in the steel were calculated on the basis of the estimated stresses caused by subgrade resistance as the pavement expands and contracts. A coefficient of subgrade resistance of 1½ times the weight of the pavement was assumed in making these determinations.

An analysis of the stress conditions in a longitudinally reinforced section during a temperature drop can be made with reasonable certainty up to the point when the concrete fails and transverse cracks are formed. Because of the uncertainty of the distribution of localized stress in the steel in the vicinity of the cracks it is doubtful if a rigorous analysis can be made of the distribution of stress along the steel after the concrete has ruptured.

Steel stresses calculated on the assumption that the stress is due entirely to the effort of the contracting steel to drag with it along the subgrade the various segments of the fractured section are likely to be both approximate and conservative. For the present purpose the lengths of the sections that are presumably required to develop given steel stresses have been cal-

TABLE 1.—*Details of steel reinforcement in experimental reinforced concrete pavement;* [1] *cold drawn wire (welded fabric)*

Number of sections	Length of each section	Calculated maximum stress in steel	Reinforcement, size and spacing: Longitudinal	Reinforcement, size and spacing: Transverse	Weight of longitudinal steel
	Feet	*Pounds per square inch*			*Pounds per 100 square feet*
149-POUND					
6	140	25,000	No. 4-0; d=0.3938 inch; 4 inches center to center.	No. 3; 12 inches center to center.	132
6	190	35,000			
6	250	45,000			
6	310	55,000			
107-POUND					
6	90	25,000	No. 4-0; d=0.3938 inch; 6 inches center to center.	No. 3; 12 inches center to center.	91
6	130	35,000			
6	170	45,000			
6	200	55,000			
91-POUND					
6	80	25,000	No. 3-0; d=0.3625 inch; 6 inches center to center.	No. 4; 12 inches center to center.	77
6	110	35,000			
6	140	45,000			
6	170	55,000			
65-POUND					
6	60	25,000	No. 0; d=0.3065 inch; 6 inches center to center.	No. 6; 12 inches center to center.	55
6	80	35,000			
6	100	45,000			
6	120	55,000			
45-POUND					
6	30	25,000	No. 3; d=0.2437 inch; 6 inches center to center.	No. 6; 12 inches center to center.	35
6	50	35,000			
6	60	45,000			
6	80	55,000			
32-POUND					
6	20	25,000	No. 6; d=0.1920 inch; 6 inches center to center.	No. 6; 12 inches center to center.	22
6	30	35,000			
6	40	45,000			
6	50	55,000			

[1] Sections are 10 feet wide.

TABLE 2.—*Details of steel reinforcement in experimental reinforced concrete pavement;*[1] *billet steel bars (intermediate grade—deformed)*

Number of sections	Length of each section	Calculated maximum stress in steel	Reinforcement size and spacing: Longitudinal	Reinforcement size and spacing: Transverse	Weight of longitudinal steel
	Feet	*Pounds per square inch*			*Pounds per 100 square feet*
2	360	15,000	1-inch round bars; 6 inches center to center.	½-inch round bars; 24 inches center to center.	534
2	600	25,000			
2	840	35,000			
2	1,080	45,000			
4	200	15,000	¾-inch round bars; 6 inches center to center.	½-inch round bars; 24 inches center to center.	300
4	340	25,000			
4	470	35,000			
4	610	45,000			
4	90	15,000	½-inch round bars; 6 inches center to center.	½-inch round bars; 24 inches center to center.	134
4	150	25,000			
4	210	35,000			
4	270	45,000			
6	50	15,000	⅜-inch round bars; 6 inches center to center.	⅜-inch round bars; 24 inches center to center.	75
6	80	25,000			
6	120	35,000			
6	150	45,000			
6	20	15,000	¼-inch round bars; 6 inches center to center.	¼-inch round bars; 12 inches center to center.	33
6	40	25,000			
6	50	35,000			
6	60	45,000			

[1] Sections are 10 feet wide.

TABLE 3.—*Details of steel reinforcement in experimental reinforced concrete pavement,*[1] *rail steel bars (deformed)*

Number of sections	Length of each section	Calculated maximum stress in steel	Reinforcement size and spacing: Longitudinal	Reinforcement size and spacing: Transverse	Weight of longitudinal steel
	Feet	*Pounds per square inch*			*Pounds per 100 square feet*
2	600	25,000	1-inch round bars; 6 inches center to center.	½-inch round bars; 24 inches center to center.	534
2	840	35,000			
2	1,080	45,000			
2	1,320	55,000			
4	340	25,000	¾-inch round bars; 6 inches center to center.	½-inch round bars; 24 inches center to center.	300
4	470	35,000			
4	610	45,000			
4	740	55,000			
4	150	25,000	½-inch round bars; 6 inches center to center.	½-inch round bars; 24 inches center to center.	134
4	210	35,000			
4	270	45,000			
4	330	55,000			
6	80	25,000	⅜-inch round bars; 6 inches center to center.	⅜-inch round bars; 24 inches center to center.	75
6	120	35,000			
6	150	45,000			
6	180	55,000			
6	40	25,000	¼-inch round bars; 6 inches center to center.	¼-inch round bars; 12 inches center to center.	33
6	50	35,000			
6	60	45,000			
6	80	55,000			

[1] Sections are 10 feet wide.

culated in this manner, and a range of stresses selected so that the section lengths which correspond would cover a range that is believed to be sufficient to develop the data desired for each percentage of steel.

It was intended that in each group the longest section would have sufficient length to approach the elastic limit of the steel and that the shortest one would be short enough not to overstress the steel and consequently would develop no open cracks. The condition of the slabs of intermediate length should indicate what maximum length might be used for each percentage of steel with a reasonable assurance that such cracks as occurred would be held tightly closed.

For practical reasons, the standard pavement cross section used by the State of Indiana was adopted. This is a 9—7—9-inch thickened edge section 20 feet wide. The longitudinal joint used was of the deformed metal tongue-and-groove type with ⅝-inch diameter tie bars spaced at 60-inch intervals.

THREE TYPES OF JOINT USED

Because of the wide range and unusual lengths of the sections in this pavement, several different joint widths were selected. It was not possible to predict the amounts of movement that would occur at the ends of the long and the intermediate length sections because the degree of restraint to longitudinal movement of the pavement developed by the subgrade could not be accurately predicted. Three types of joints having different joint widths were designed, however, using the best information available in caring for expansion and contraction of the various section lengths.

The type I joint is of a structural steel design similar to that frequently used at the approaches of bridges. It is designed to allow a 1½-inch movement in each direction and is used at points where two sections of intermediate length are joined.

The type II joint is really two type I joints, spaced 10 feet apart. This allows an effective opening of practically twice that of the type I joints. This type is used where two of the longer sections are joined.

The type III joint is of the conventional dowel type and is used between the shorter sections.

These three types of joints are described in more detail and illustrated with photographs in a later section of this report.

Four sections, each having a total length of 500 feet, were included in which special joint designs and methods of reinforcing were employed. The details of design of these sections are as follows:

No. 1.—Submerged type weakened-plane joints placed at intervals of 10 feet. Reinforcement consisted of a 91-pound welded fabric placed continuously through the weakened-plane joints. The bond between the steel and the concrete was broken for a distance of 18 inches on each side of each weakened-plane joint by omitting the transverse steel at this point and by greasing.

No. 2.—This section was a duplicate of section No. 1, except that it was reinforced with a 45-pound welded fabric.

No. 3.—Weakened-plane joints formed by grooves at 10-foot intervals in the surface of the pavement. Reinforcement consisted of a 91-pound welded fabric, placed continuously through the weakened-plane joints. The bond was broken for a distance of 18 inches on each side of each weakened-plane joint, as in section No. 1.

No. 4.—This section was a duplicate of section No. 3, except that it was reinforced with a 45-pound welded fabric.

The amount of longitudinal steel in the 91-pound welded fabric is 77 pounds while that in the 45-pound fabric is 35 pounds. The types of weakened-plane joints used in this part of the pavement are shown in figure 4.

The object of this pavement design is to control cracking and to eliminate, as far as possible, warping stresses in the pavement. Breaking the bond between the steel and the concrete over a 36-inch length at the joints permits warping of the pavement slab to take place more freely when the joint edges of adjoining

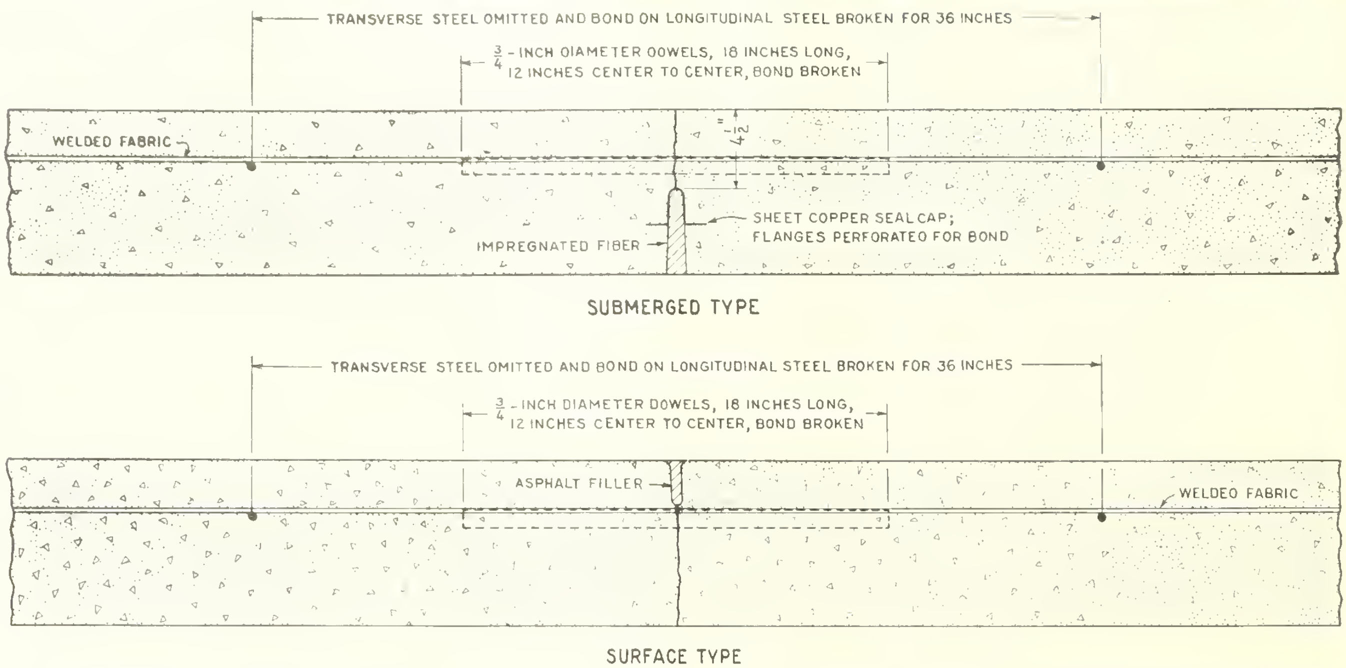

FIGURE 4.—DETAILS OF WEAKENED-PLANE JOINTS. DOWELS OMITTED IN JOINTS IN ONE-HALF THE LENGTH OF EACH SECTION.

slabs are in contact, because when warping occurs, a greater length of steel is available to provide the necessary elongation without the development of excessively high tensile stresses in the steel.

The stress conditions that may develop in steel reinforcement passing continuously through several slab units, as in this design, are:

1. That caused by subgrade resistance as the whole slab series expands or contracts about its group center.
2. That caused by the tendency of each short slab unit to expand and contract about its own center. When this occurs, a resisting or opposing force is set up between adjoining slab units causing a change in tensile stress in the steel at the point of connection.
3. That caused by resistance to bending at the joints as the edges of the pavement are warped or are deflected by the action of wheel loads. This stress is the result of the resisting moment set up between the steel and the ends of the slab in contact at the joint as deflection occurs.
4. The shearing stresses caused by loads passing over the joints.

SURFACE VIBRATION USED IN PLACING ALL SECTIONS

It is not possible to calculate accurately any of these stress conditions with the knowledge now available. The first stress condition is understood better than the others and is known to be important in long sections. It is thought that the second and third stress conditions are relieved by the breaking of the bond at the joints, as was done deliberately in that part of the experimental pavement just described. This relief is effective because only a limited opening of the joint at the point where the steel is located is necessary for its accomplishment. If the length of the steel that is available for elongation is increased, there is a corresponding reduction in the intensity of the stress in the steel because the movement at the joint remains the same.

There is no practical means of either computing or measuring the shearing stresses in the steel at weakened-plane joints of the type included in this part of the pavement. It is possible that aggregate interlock will relieve these stresses to a certain degree, but whether this will be sufficient to prevent failure of the steel is not known. For this reason, it was decided to place shear bars across the weakened-plane joints in one-half the length of each of the four sections. If it is later found that a greater number of failures occur in the steel at the joints with no shear bars than occur in those with shear bars, it will be a reasonable conclusion that shearing stresses are influencing the number of failures which occur in the steel.

It is realized that the stresses that will occur in the steel of these four 500-foot sections will probably be sufficient to cause the steel to break. The sections were designed with this in mind because it was desired to determine the number of breaks that will occur in the steel and the amount of opening that occurs at joints in pavement sections designed in this manner.

Because of the length of the pavement it was desirable to let the construction of it under two separate contracts. The two contractors used slightly different materials and methods. While this is unfortunate in a test pavement, it is doubtful if the differences are sufficient to cause serious difficulty in the interpretation of the data.

Because of possible difficulties in the consolidation of the concrete in the sections containing large amounts of steel, it was decided to apply surface vibration to all sections.

The materials and quantities used in each batch of concrete in the west project, F. A. P. 4 A–2 were as follows: Cement, 564 pounds; sand (wet), 1,336 pounds; stone (small), 1,422 pounds; stone (large) 852 pounds.

The slump ranged between 7/8-inch and 1 1/2 inches and averaged 1.3 inches.

The materials and quantities used in each batch of concrete in the east project, F. A. P. 4 B–1 were as follows: Cement, 564 pounds; sand (wet), 1,252 pounds; gravel (small), 1,441 pounds; stone (large) 884 pounds.

The slump ranged between 1 inch and 2½ inches and averaged 1.5 inches.

The Indiana specifications do not require that the amount of sand be adjusted on the basis of dry weights, but do require that the sand be allowed to stand in stock piles for 48 hours before being used. For this reason, the actual amount of sand used in a batch of concrete is not known.

Attention is called to the fact that crushed stone was used exclusively as a coarse aggregate in the west project, while in the east project the small-size coarse aggregate was gravel and the large-size aggregate was crushed stone. The large-size coarse aggregate ranged in size from ½ inch to 2¼ inches, while the small-size coarse aggregate ranged from No. 4 to 1½ inches.

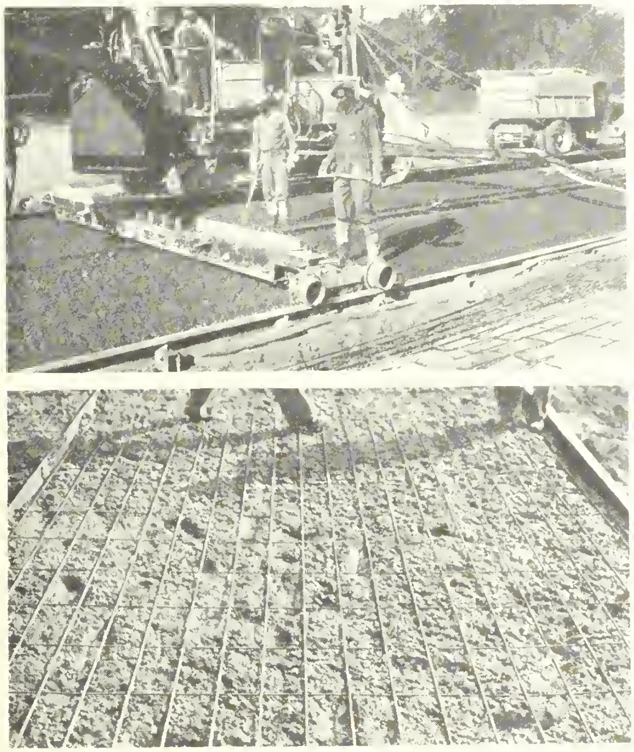

FIGURE 5.—CONSTRUCTION OF SLABS HAVING WELDED FABRIC REINFORCEMENT. UPPER, STRIKING OFF CONCRETE PREPARATORY TO PLACING WELDED FABRIC; LOWER, WELDED FABRIC IN PLACE.

The welded fabric reinforcement was placed by the strike-off method at a depth of 2½ inches from the top surface. The method of striking off the concrete preparatory to placing the steel is shown in figure 5. This figure also shows the welded fabric in place before the upper part of the concrete had been placed.

The bar mat reinforcement was erected on the subgrade and securely wired together a short time before the placing of the concrete. This type of reinforcement was placed at the mid-depth of the pavement. It was supported on welded chair assemblies attached to the transverse bars. The three views of figure 6 show different stages in the placement of the 1-inch steel. The supports were welded to every second transverse bar for the ¼- and ⅜-inch longitudinal steel and to every third transverse bar for the larger sizes. This method of supporting the steel proved to be very effective.

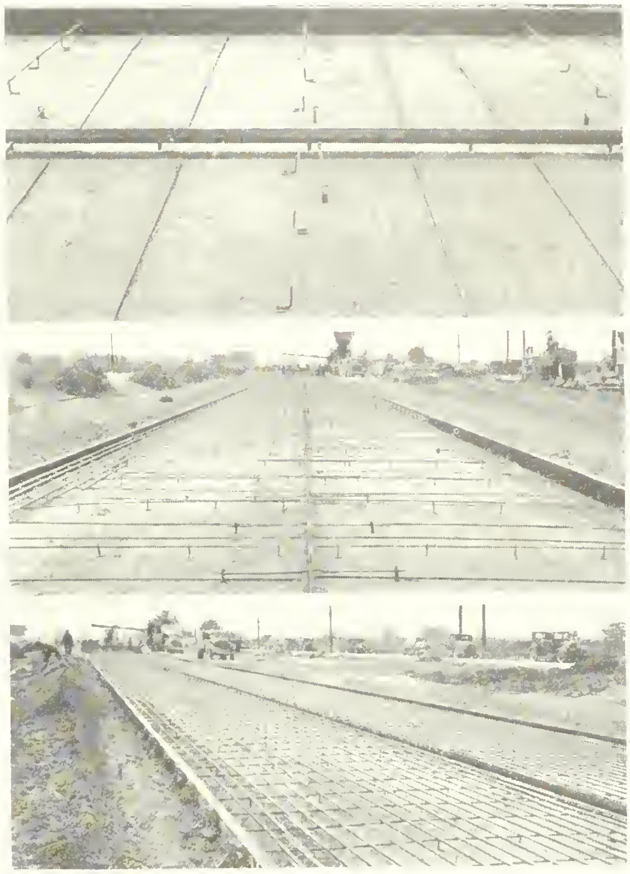

FIGURE 6.—INSTALLMENT OF HEAVY BAR REINFORCEMENT. TOP, WELDED CHAIR SUPPORTS AND TRANSVERSE BARS BEING PLACED; MIDDLE, LONGITUDINAL AND TRANSVERSE STEEL BEING PLACED; BOTTOM, HEAVY BAR MAT REINFORCEMENT IN PLACE.

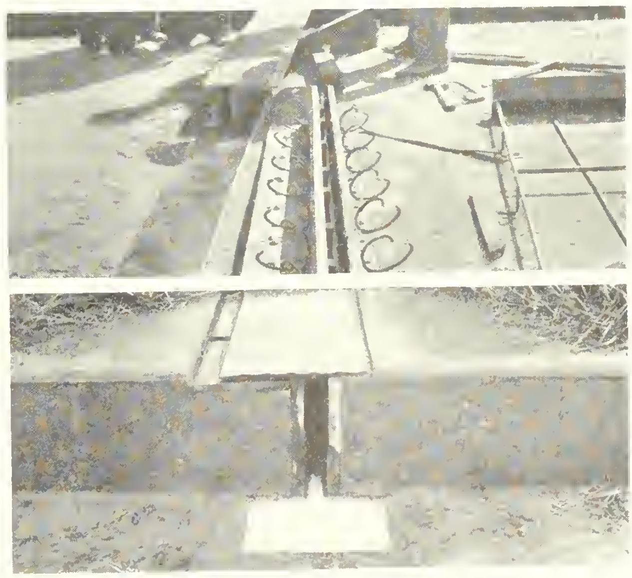

FIGURE 7.—TYPE I JOINT BEING PLACED IN POSITION ON THE SUBGRADE (UPPER), AND APPEARANCE OF JOINT AFTER CONSTRUCTION OF PAVEMENT SLABS (LOWER).

CONSTRUCTION OF JOINTS DESCRIBED IN DETAIL

The joints used between the experimental sections were described in a general way earlier in the report. The upper view in figure 7 shows a type I joint being

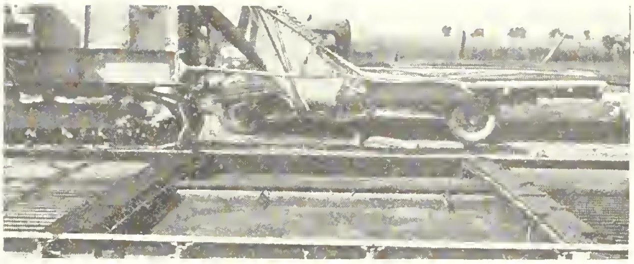

FIGURE 8.—TYPE II JOINT IN PLACE ON THE SUBGRADE PREPARATORY TO PLACING CONCRETE.

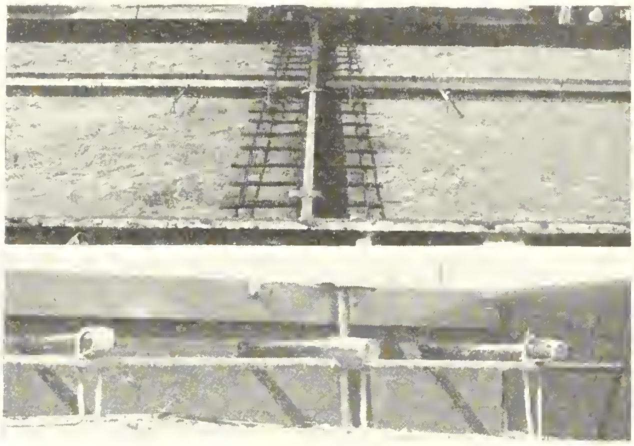

FIGURE 9.—TYPE III JOINT IN PLACE ON THE SUBGRADE (UPPER), AND METHOD OF HOLDING DOWEL BARS IN PLACE (LOWER).

placed in position on the subgrade. The section of joint in the foreground is upside down. This joint consists essentially of two 2-inch angles and a cover plate. The angles are anchored on opposite sides of the joint while the cover plate is rigidly attached to the angle on one side of the joint and held to the angle on the opposite side by a key way system which permits sliding. The different parts of the joint assembly are held together during the time the concrete is being placed by wooden boards that may be removed after the concrete has hardened. The lower view in figure 7 shows an end view of the joint after the concrete has been placed. The projecting part of the sheet metal plate shown on the subgrade is to be turned up against the edge of the slab to close the joint opening before the shoulder is completed.

A type II joint, which consists of two type I joints placed with a 10-foot slab between, is shown in figure 8.

The type III joint is a conventional doweled joint with ¾-inch plain round bars spaced 12 inches apart. A general view of this type of joint and the method of holding the dowels in place are shown in figure 9.

The submerged type of weakened-plane joints were formed by placing impregnated fiber strips on the subgrade at the points where the joints were to be formed. The height of these strips was varied so as to keep the top uniformly 4½ inches below the top surface of the pavement. A copper seal was placed at the top of the groove so that it would be unnecessary to have a watertight seal on the top surface. Figure 10 shows the submerged weakened-plane joint devices in place on the subgrade before placing the concrete. The dowel shear bars which were placed at one-half of the weakened-plane joints were held in place during concreting by tying them to the longitudinal bars of the fabric.

FIGURE 10.—DEVICES USED IN FORMING SUBMERGED WEAKENED-PLANE JOINTS IN PLACE ON THE SUBGRADE.

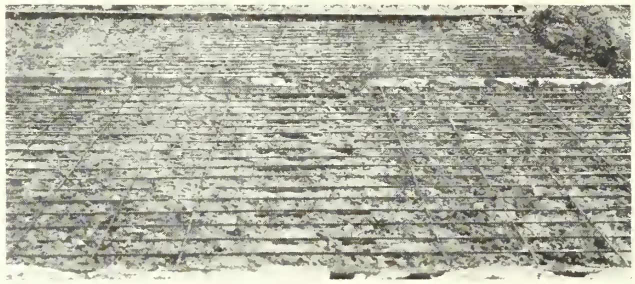

FIGURE 11.—SHEAR DOWEL BARS IN PLACE AT LOCATION WHERE WEAKENED-PLANE JOINT IS TO BE FORMED.

Figure 11 shows the bars in place at one of these joints before placing the top part of the concrete. This method of supporting dowel bars is not recommended for use at joints where longitudinal movements of appreciable magnitude are expected, because of the possibility that some misalinement may develop during the placing of the concrete. In this instance, however, with the 10-foot joint spacing and with longitudinal steel placed continuously through the joints, the longitudinal movements at any one joint should be quite small and with the close supervision given all construction operations it was thought the method of installation would be satisfactory.

Because the vibratory method of placing resulted in dense firm concrete, some difficulty was experienced in forming the grooves in the top of the pavement for the conventional type of weakened-plane joints. This difficulty was satisfactorily overcome by placing a vibrator on the T-bar used to form the groove. This device is shown in figure 12.

The concrete mixer was kept to the side of the roadway during the placing of concrete. This allowed the bar mat reinforcement to be placed a short distance in advance of the mixer. The two views in figure 13 show the placing of the concrete in one of the heavily reinforced sections.

As mentioned previously, surface vibration was used over the entire length of the pavement in order to insure good compaction of the concrete throughout the full depth of the pavement and the proper embedment of the steel. A pan-type vibrator was used on the east project (F. A. P. 4 B-1). The pan was divided in four parts, each approximately 5 feet in length, which were connected to each other by hinged joints. One-third-horsepower vibrators were mounted on each of the four units of the pan and the pan was mounted between the two screeds of the finishing machine. During operation, the pans were in contact with the pavement for a width of approximately 10 inches. The vibrator was operated during two forward passes and

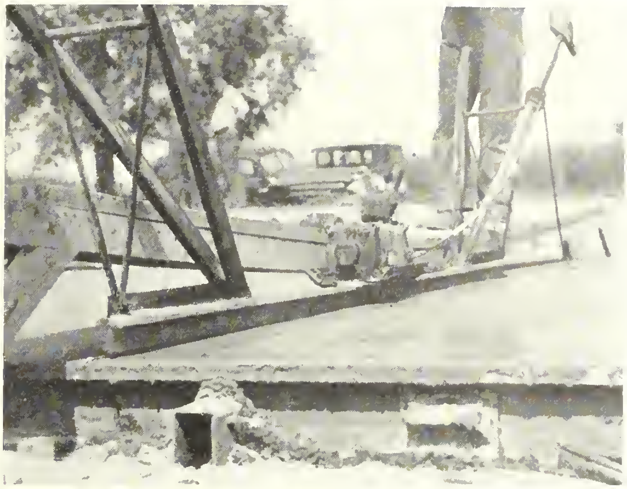

FIGURE 12.—T-BAR AND VIBRATOR USED IN FORMING GROOVES FOR WEAKENED-PLANE JOINTS.

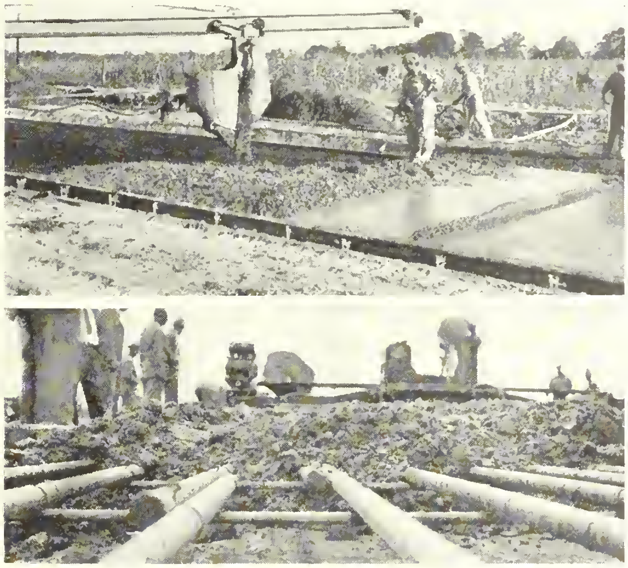

FIGURE 13.—CONCRETE BEING PLACED IN A HEAVILY REINFORCED SECTION.

one backward pass of the finishing machine. The finishing machine with the pan vibrator mounted is shown in figure 14.

On the west project (F. A. P. 4 A–2) the vibration of the concrete was applied through the front screed of the finishing machine to which three ½-horsepower vibrators were mounted. The screed was of the bull-nosed type and vibration was applied to the concrete during two or more forward passes of the finishing machine.

Special vibrators were used around the joints. Two different types of surface vibrators used around the Type III joints are shown in the two views of figure 15. An internal vibrator was used around the type I and type II joints. (See fig. 16.)

The concrete was finished in a conventional way, except that a mechanical longitudinal float was used on the west project. This machine is shown in figure 17.

After placing, the concrete was cured with wet burlap until the next morning. The burlap was then

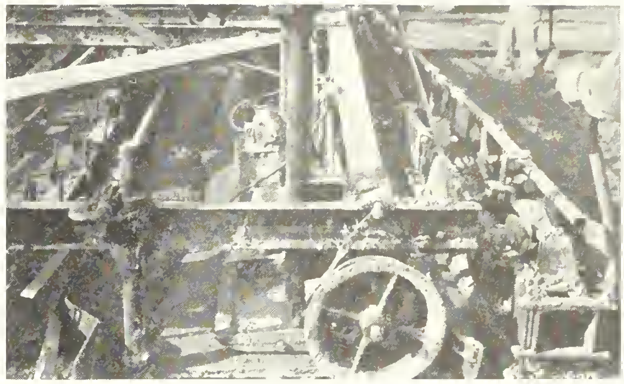

FIGURE 14.—CONCRETE FINISHING MACHINE, SHOWING PAN-TYPE VIBRATOR MOUNTED BETWEEN THE TWO SCREEDS.

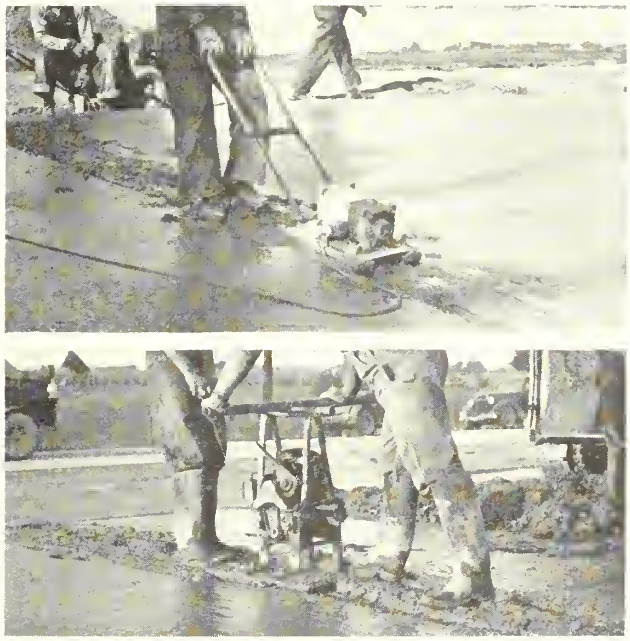

FIGURE 15.—VIBRATORS USED AROUND TYPE III JOINTS. UPPER, VIBRATOR USED ON EAST PROJECT; LOWER, VIBRATOR USED ON WEST PROJECT.

FIGURE 16.—INTERNAL VIBRATOR USED UNDER TYPES I AND II JOINTS.

removed and the concrete was covered with wet straw for seven days. Uniformly excellent weather prevailed during the construction of the entire pavement.

FIGURE 17.—LONGITUDINAL MECHANICAL FLOAT USED ON THE WEST PROJECT.

SCHEDULE OF OBSERVATIONS OUTLINED

In general, the relative value of the various sections in this experimental pavement can best be determined by a study of their behavior under traffic over a period of years. All of the sections are duplicated at least once and in most cases there are four or six sections of a given type, in order that there will be a check on the performance.

The nature of the pavement makes it necessary that very close examinations be made of the sections if the history of the performance is to be accurately recorded. Transverse cracking is frequently quite inconspicuous, yet from a research standpoint its presence or absence is significant.

The detailed examination of so many sections entails a very considerable amount of work and it is obvious that there are practical limitations to the type and number of surveys that can be made.

Furthermore, weather conditions at different seasons of the year have an important influence on the cracking and other defects that develop in concrete pavements. It is desirable, therefore, that the schedule of periodic examinations be such as to give as much information as possible regarding the seasonal effects.

With these conditions in mind, it was decided to select certain representative sections on which intensive studies would be made at different seasons of the year and to give the entire pavement a thorough general condition survey once a year. This program of course is subject to modification if developments warrant.

For the intensive study just mentioned, three parts of the pavement, each having a length of approximately 2,000 feet, were selected and the details of each are given in table 4.

It will be noted that the first part is located in the heavily reinforced pavement, the second in the pavement having a medium amount of reinforcement, and the third in the pavement reinforced with welded fabric. These different parts of the pavement were located so as to eliminate, as far as possible, all variables except the amount of reinforcement. The intensive schedule of observations which is to be made on these selected sections include the following:

1. A detailed crack survey in which a special effort is made to find all the cracks, however fine.

2. Precise level measurements made on the surface of one lane of the pavement.

3. Measurements of horizontal movements at the joints in the longitudinal direction of the pavement.

These detailed studies of the selected sections are to be made three times a year, in the fall, winter, and spring.

The annual observations made over the full length of the pavement will be made in the fall of the year and include the following:

1. A general crack survey.

2. Measurements of horizontal movements at the joints in the longitudinal direction of the pavement.

3. General observations of the condition of the pavement.

Precise level bench marks were established, at intervals, along one edge of the pavement a short time after the completion of construction. Subsequently level measurements were taken on each side of the joints and at intervals of 100 feet along the full length of one lane of the pavement. This established the normal elevation of the pavement. It is not intended to make further level measurements at regular intervals, but they will be made at such times as may appear to be desirable, either over the full length or on certain parts of the pavement.

TABLE 4.—*Parts of pavement on which intensive studies are being made* [1]

Part	Length of section	Number of sections	Length of pavement included	Longitudinal reinforcement		
				Type	Size	Spacing of bars
No.	*Feet*		*Feet*		*Inches*	*Inches*
1	600	2	600	Rail	1	6
	360	2	360	Billet	1	6
	1,080	2	1,080	Billet	1	6
2	150	2	150	Rail	½	6
	330	2	330	Rail	½	6
	90	4	180	Billet	½	6
	150	4	300	Billet	½	6
	210	4	420	Billet	½	6
	270	4	540	Billet	½	6
3	20	6	60	32-lb	Welded fabric	
	30	6	90	32-lb	Welded fabric	
	40	6	120	32-lb	Welded fabric	
	50	6	150	32-lb	Welded fabric	
	30	6	90	45-lb	Welded fabric	
	50	6	150	45-lb	Welded fabric	
	60	6	180	45-lb	Welded fabric	
	80	6	240	45-lb	Welded fabric	
	60	6	180	65-lb	Welded fabric	
	80	6	240	65-lb	Welded fabric	
	100	6	300	65-lb	Welded fabric	
	120	6	360	65-lb	Welded fabric	

[1] Sections are 10 feet wide.

Other special studies that are contemplated include:

1. Determination of the movements of the ends of the slabs between the extreme summer and winter conditions; in other words, the maximum annual change in length of the various lengths of slab.

2. Determinations of the daily movements at the ends of certain selected slabs. These measurements will be taken on days when large variations in temperature are expected.

3. Measurement of the absolute movements at the two ends, the center and at the two quarter-points of one of the longest slabs.

The device used in measuring the absolute movements of the ends of the slabs is shown in figure 18. The bench mark, at the right, on which one end of the device is resting, is a pipe that has been driven into the ground to a depth of 9 feet and is surrounded by a casing 28 inches long. There is a small hole in the cap on top of this pipe in which the point of the measuring device is placed. The point on the left end of the measuring device rests in a small hole in a metal plug placed in the concrete. The vernier attached to the point on the left makes it possible to determine the slab movements to 1/100 of an inch. It is necessary to adjust these readings to obtain the actual movements in a direction parallel to the longitudinal axis of the pavement. The bench marks are carefully covered to prevent water from entering and the shoulder material is replaced immediately after taking each reading.

(*Continued on p. 218*)

THE COST OF CURING CONCRETE PAVEMENTS WITH COTTON MATS

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by ROBERT A. MARR, Jr., Assistant Testing Engineer

IN CONNECTION with a program initiated in 1936 by the United States Department of Agriculture for the purpose of promoting a greater utilization of cotton products, the various State highway departments were invited by the Public Roads Administration (then the Bureau of Public Roads) to cooperate in a study of the possibilities of using cotton mats for curing concrete pavements.

Laboratory tests by the Administration [1] and field tests by the Texas State Highway Department [2] had already demonstrated the value of this material for curing concrete. Very limited data, however, were available as to the cost of curing with cotton mats compared to other accepted methods. It was felt that accurate data along this line could be obtained only through extensive tests on actual paving jobs located in various parts of the United States.

The State highway departments of 23 States, well distributed geographically, agreed to participate in this program. A total of 90,000 mats, valued at $355,000, was furnished these States by the Federal Government without cost. This quantity would be sufficient to cover at a single placement approximately 65 miles of 20-foot concrete pavement. The mats were made available to the States on the condition that they be used in an approved manner in concrete pavement construction. The cooperating highway departments also agreed to keep records of the life of the mats and average unit curing costs and to report these data to the Public Roads Administration.

In compliance with their agreement, 20 of the 23 participating States have filed reports. These data represent the equivalent of 668 miles of 20-foot pavement cured by this method under widely varying conditions of locale, weather, hourly wage rates, etc., and extending over several construction seasons.

The cotton mats supplied for this purpose weighed approximately 24 ounces per square yard. They were made of cotton bats quilted between cotton sheets of a type known to the trade as "osnaburg." The full-length mats were 22 feet 6 inches long by 5 feet 9 inches wide and were quilted longitudinally by rows of stitching not more than 4 inches apart. For overlap, a 6-inch flap was provided along one longitudinal edge by sewing the covers together without batting. Shorter mats varying in length from 10 feet 6 inches to 14 feet 6 inches were also provided. The mats conformed to Specification M 73–38 of the American Association of State Highway Officials.[3]

Since the mat covers were of unshrunk cloth, considerable change in dimensions was to be expected. Data reported by seven States showed an average change of 6 percent in the longer dimension and 11 percent in the shorter. Cases were reported where individual mats were too short after shrinkage to cover the pavement when laid crosswise. In determining the dimensions of future mats, proper allowance should be made for such shrinkage.

SERVICE LIFE OF 50 USES INDICATED

The specifications provided for these cotton mats to be used in a manner similar to burlap, i. e. that the saturated mats be applied to the concrete as soon as possible without marring and that they be kept wet until removed. No further curing treatment was required. A minimum curing period of 72 hours was specified though several States used a longer period.

The estimated life of an individual mat as reported by 14 States ranged from a minimum of 15 uses to a maximum of 100. The arithmetical average for all States reporting was 47 uses, while a weighted average based on the concrete pavement yardage cured was 63. Ten of the 14 States reporting life data estimated that the mats could be used at least 40 times before discarding them. It seems conservative to state that cotton mats manufactured to American Association of State Highway Officials specifications should have a life of 50 uses with ordinary care, and that this life may be increased 10 to 50 percent by extra care in handling, drying, and storage.

The cost of the mats, as delivered to certain designated points in the States, averaged approximately 48 cents per square yard of net useful coverage.[4] For an average life of 50 uses, this is equal to about 1 cent per square yard. This value checks reasonably well with the average mat cost per use in eight States for mats purchased in the open market of approximately 1¼ cents per square yard.

The cost of curing concrete pavement as revealed by reports from 19 States, excluding the material cost but including supervision, labor, and transportation, averaged 2.15 cents per square yard. These costs were, of course, greatly influenced by local factors, chiefly hourly wage rates and weather conditions. For instance, the minimum State average of 1.08 cents occurred where labor was quoted as low as 20 cents per hour, while the maximum State average of 3.11 cents corresponded to an hourly wage rate of 68 cents per hour. For individual projects the range was from slightly over ½ cent to 6 cents per square yard. On one project, heavy rains made sprinkling unnecessary about one-fourth of the time. This helped to reduce curing costs 40 percent below the average for this State.

[1] Cotton Mats for Curing Concrete. PUBLIC ROADS, vol. 14, No. 5, July 1933. Further Tests of Cotton Mats for Curing Concrete. PUBLIC ROADS, vol. 15, No. 9, Nov. 1934.

[2] Curing Concrete Pavement with Cotton Mats, by J. G. Rollins. American Highways, vol. XIV, No. 3, July 1935.

[3] Standard Specifications for Highway Materials and Methods of Sampling and Testing, published by the American Association of State Highway Officials, 1220 National Press Building, Washington, D. C.

[4] The "net useful coverage" is the actual area of pavement slab which can be covered by a mat of specified dimensions. For instance, after allowing for shrinkage, overlap, and overhanging ends, a mat having gross dimensions of 22 feet 6 inches by 6 feet 3 inches will cover a slab 20 feet by 5 feet in area. The net useful coverage of the mat is, therefore, 100 square feet although its total area is approximately 142 square feet.

FIGURE 1.—COTTON MATS IN PLACE BEING WET DOWN.

The average total curing cost, obtained by adding the average material and usage costs, is 3.15 cents per square yard (1 cent material cost plus 2.15 cents usage cost). However, as certain States omitted such items as cost of water for sprinkling, transportation of mats to and from the project, and the winter storage, it would seem safer to state that the average total cost should not exceed 3.5 cents per square yard. In nine States the cost of curing with cotton mats was compared with the cost of curing with other acceptable materials. A summary of these direct comparisons indicates that the cost of cotton mat curing is in general about the same as other commonly accepted methods used under similar conditions.

All of the State reports submitted in connection with this study confirm the preliminary laboratory tests by showing a high efficiency for cotton mat curing when judged by the following:

1. Ability to maintain a film of moisture over the surface of the concrete during the curing period.
2. Strength of cores from mat-cured slabs as compared to those cured by other standard methods.
3. Insulation of slab against temperature change.

COTTON MATS HAD GOOD ABSORPTION AND RETENTION OF MOISTURE

Although somewhat difficult to wet the first time, due to natural oils in the cotton filler, the mats will absorb from two to three times as much water as double 12-ounce burlap. Figure 1 shows mats in place being wet down. Comments from 11 States reveal that mats retain this absorbed water equally as well as earth and better than burlap. On two projects, mats receiving only the original wetting were still wet on the under side at the end of the 72-hour curing period.

Core tests (based on incomplete data) show that strengths from mat-cured slabs average approximately the same as those cured by other standard methods.

Cotton mats have excellent insulating qualities. Temperature measurements taken on two California projects during cold weather showed that the average minimum temperature under the mats was 40 percent higher than the average minimum air temperatures, of which some were within the freezing range. Even when the top fabric was frozen stiff, the under surface was still soft and moist. Two Northern States also found them very effective in preventing freezing of concrete and subgrade. Limited tests made on a few mats, blackened on top by applying a light coat of emulsified asphalt, showed the average maximum and minimum temperatures under them to be 29 percent and 5 percent higher, respectively, than those under uncolored mats.

Comments by the States reporting, based on experience, indicate that the life of cotton mats can be prolonged by proper attention to certain details, neglect of which hastens deterioration. A few such precautions are listed below.

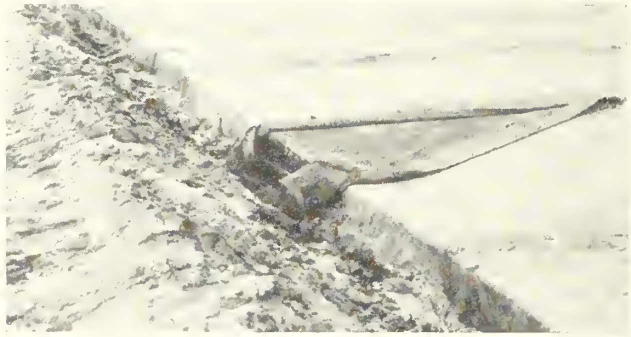

FIGURE 2.—COTTON MATS DISINTEGRATE RAPIDLY IF ALLOWED TO OVERHANG ONTO THE SUBGRADE AS SHOWN.

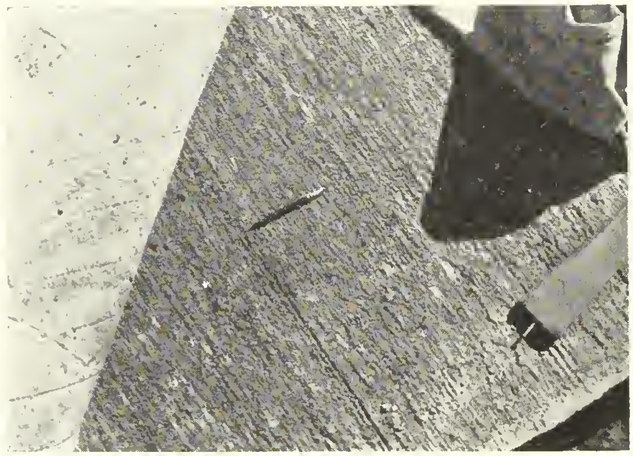

FIGURE 3.—PORTION OF CONCRETE PAVEMENT SLAB AND COTTON MATS USED IN CURING IT.

If cotton mats are rolled or folded and left damp, even for a few days, mildew sets in, destroying the covering fabric. They must be dried at the end of a job and prior to storage. Under favorable conditions, mats may be left on the slab and turned until dry; but unfavorable weather, particularly in Northern States, may require artificial drying indoors.

Mats should be so stored that they can be periodically inspected for mildew.

When saturated, full-length mats weigh approximately 100 pounds. If the center is allowed to drag on the slab, particularly in removing, this portion of the fabric soon wears out. Full-length mats may easily be handled by two men from a movable bridge. One State has developed a special type of bridge with a sloping apron from which mats can be accurately placed on very soft concrete without marking.

The original intent was for mats to be placed crosswise on the pavement with their ends overhanging the edges of the slab, but constant contact with wet earth was found to cause deterioration. (See fig. 2.) It was found preferable to bank the edges of the slab with earth and to fold the mat ends back even with these edges. Figure 3 shows edges of mats turned back, exposing the concrete surface.

There may be considerable danger from fire in the use of cotton mats, particularly on projects where traffic is maintained in an adjacent lane. Even though the under side is still moist, the top fabric can be dry enough to be ignited by lighted cigarette or cigar butts

(*Continued on p. 219*)

COST OF OPERATING RURAL-MAIL-CARRIER MOTOR VEHICLES

THE report Cost of Operating Rural-Mail-Carrier Motor Vehicles on Pavement, Gravel, and Earth, by R. A. Moyer and Robley Winfrey, has recently been published as Bulletin 143 of the Iowa Engineering Experiment Station. In this report are analyzed cost records of 293 motor vehicles as follows: 248 automobiles operated by rural mail carriers in Iowa, 43 in Indiana, and 2 in Alabama, covering 170 routes.

Operation covered the period from November, 1935 to January, 1937. The report is based upon the original cost records kept by the individual carriers. These detailed daily records covered such phases of operation as miles of travel on each type of road surface, rates of travel, weather, number of stops, load, amounts of gasoline and oil used, and expenses incurred for tires, maintenance, garage, license, taxes, insurance, depreciation, interest, and extra help.

Records submitted were analyzed to determine the average total cost of operation on pavement, untreated gravel, and earth roads for a complete year and for the four seasons. The results obtained apply directly only to cars operating under conditions similar to those encountered by rural mail carriers.

The specific results are summarized as follows:

1. The average operating cost for cars traveling almost exclusively on pavement and gravel was 3.8 cents per vehicle-mile and 7.8 cents per mile for cars traveling almost exclusively on earth.
2. Extra help in delivering the mail cost an average of 0.2 cent per vehicle-mile on pavement and gravel and 1.0 cent per mile on earth.
3. The cost of replacing cars with men on foot or horseback, when the roads were impassable to cars, averaged approximately 11 cents per mile as compared to an average cost of less than 5 cents per mile with the cars when the roads were passable.
4. The graphical solution indicated an average annual mileage of 20,000 miles for cars operated exclusively on pavement and gravel, and 4,000 miles for cars operated exclusively on earth.
5. The average rate of travel (including stops) on the route during the year was about 13 miles in an hour on pavement and gravel, and about 9 miles in an hour on earth. During the summer the approximate average rate on pavement and gravel was 14½ miles in an hour and on earth 10½ miles in an hour, while these rates were respectively 11½ and 7½ miles in an hour during the winter.
6. The cost of gasoline, oil, and maintenance increased from about 2 cents per mile for cars with life mileages under 10,000 miles to 3 cents per mile for cars with mileages of about 50,000 miles. A similar trend was indicated for these costs when the age of the car increased from 1 to 6 years, but there was no appreciable change for cars more than 6 years old.

Results by the statistical method of least squares are:

7. The average cost of gasoline, oil, tires, and maintenance for the year was 1.56 cents per vehicle-mile on pavement, 2.59 on gravel, and 3.14 on earth.
8. The average gasoline mileage obtained was 15.02 miles per gallon on pavement, 13.04 on gravel, and 13.52 on earth.
9. The oil mileage averaged 264 miles per quart on pavement, 159 on gravel, and 113 on earth.
10. During the winter season the cost of gasoline averaged 1.50 cents per mile on pavement, 1.54 on gravel, and 1.58 on earth, while during the summer these unit costs were 1.21 cents on pavement, 1.24 on gravel, and 1.13 on earth.
11. During the winter season the cost of maintenance averaged 0.28 cent per mile on pavement, 0.77 on gravel, and 1.70 on earth, while during the summer season these unit costs were 0.05 cent on pavement, 0.38 on gravel, and 0.63 on earth.

Other results may be summarized as follows:

12. The average factory list weight of the mail-carrier cars was 2,680 pounds and the empty weight was 2,950 pounds as compared to an empty weight of 3,150 pounds for the average Iowa car. The average weight of the mail carried was 135 pounds.
13. The number of boxes per mile of route averaged 4 on pavement and gravel and 3¼ on earth.
14. The total average annual cost of operating the rural-mail-delivery cars, based on an annual mileage of 15,000 miles, was $500.26 on pavement, $627.76 on gravel, and $680.26 on earth, or 3.34 cents per mile on pavement, 4.19 on gravel, and 4.54 on earth.
15. A traffic volume of 63 vehicles per day will justify an annual interest charge of 4 percent on an investment of $1,000 per mile and an increased maintenance expenditure of $40 per year per mile for improving a county trunk earth road with a gravel surface, based on the 0.35-cent-per-mile difference in motor-vehicle operating cost. If an additional charge is made to amortize this investment over a period of 10 years, a traffic volume of 128 vehicles per day will justify the change.
16. A traffic volume of 25 vehicles per day will justify the improvement from earth to gravel if travel time is evaluated as it was for the cars in this study and if the amortization of the investment is included.
17. An expenditure of 0.5 cent per vehicle-mile is justified for snow and ice removal from pavement during the three winter months when the difference in operating cost alone is considered, and 1.22 cents per vehicle-mile is justified when the time factor valued at 40 cents an hour is included.

NEW SUPPLY OF HANDBOOK ON TRANSITION CURVES FOR HIGHWAYS PRINTED

A new supply of Transition Curves for Highways has recently been printed for the Superintendent of Documents, Government Printing Office, Washington, D. C. This handbook, by Joseph Barnett of the Public Roads Administration, contains tables with which the design and location of curves with transitions involve only simple calculations. The rate at which the first supply of this handbook was sold indicates a wide interest in the subject.

Sections of the handbook discuss speed in relation to highway design, design of curves with equal transitions by use of tables, design of curves with transitions as a general case, parallel transitions, transitions for compound curves, adjusting alinement of simple curves for transitions, widening pavements on curves, and right-of-way lines in relation to transitions. All tables needed in applying the methods described are included.

The handbook, in a durable binding, is available only by purchase from the Superintendent of Documents, Government Printing Office, Washington, D. C., at 60 cents a copy. There is no free supply.

The methods described in this handbook are now being used in almost every State and in many foreign countries. Recently the Public Roads Administration approved the use of the handbook by the Argentine Bureau of Roads in preparing a similar publication in Spanish, using metric units and adapted to the Argentine method of laying out curves.

(Continued from p. 214)

FIGURE 18.—DEVICE USED FOR MEASURING MOVEMENTS AT ENDS OF SLABS, AND BENCH MARK (LOWER RIGHT).

FIGURE 19.—LEVELING ROD WITH SPECIAL ATTACHMENT.

Since the combined movements of the ends of the two slabs at a joint should be equal to the total change in width of the joints, the changes in the widths of the joints are measured as a check on the measurements of the absolute movements of the ends of the slabs. These measurements are made with a micrometer between two metal points set in the concrete.

Metal points were set in the concrete at the points where the level measurements are taken. Figure 19 shows a leveling rod resting on one of these points.

FIGURE 20.—TYPICAL CRACK IN A HEAVILY REINFORCED SECTION.

All measuring points were set slightly below the surface of the concrete so that they would not be disturbed by traffic.

CONSIDERABLE CRACKING IN LONGER SECTIONS AFTER 1 YEAR IN USE

The experimental pavement was approximately 1 year old in October 1939. Approximately 1½ miles had been under traffic for a period of about 1 year while the remaining 4½ miles had been in service for about 6 months. A large number of fine transverse cracks have occurred in the central portion of the long, heavily reinforced sections. In the long sections containing the ¾- and the 1-inch diameter longitudinal bars, the distance between the cracks is frequently less than 3 feet. There is an appreciable number of these cracks in the longer sections containing the ½-inch diameter longitudinal bars, but relatively few in the shorter sections reinforced with this amount of steel. For the sections containing the smaller amounts of reinforcement, in general the number of transverse cracks which have occurred is related more directly with the length of the sections than it is with the amount of longitudinal reinforcement. There is practically no cracking in any of the sections less than approximately 150 feet long, regardless of the amount of reinforcement.

A typical crack in one of the heavily reinforced sections is shown in figure 20. These cracks are not apparent except on close examination and are very similar to those that occurred very early in the life of the heavily reinforced sections of the Columbia Pike experimental pavement. As stated earlier in this report, the cracks in the heavily reinforced sections on Columbia Pike have remained closed and no serious spalling or disintegration has occurred in their vicinity. It seems unlikely, therefore, that these fine cracks in the heavily reinforced sections of this pavement will ever cause serious damage.

The sections in which the weakened-plane joints were placed at intervals of 10 feet are in excellent condition.

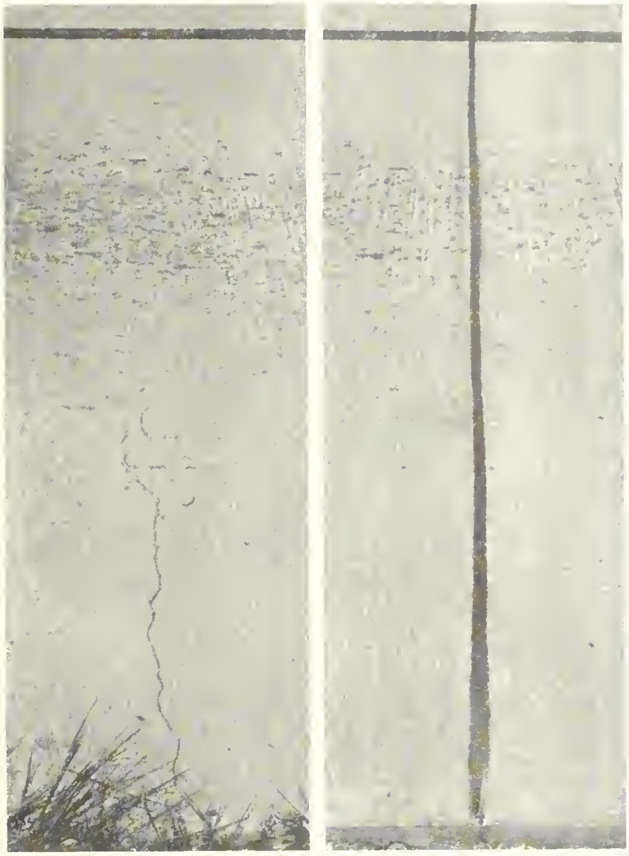

FIGURE 21.—LEFT, CRACK THAT HAS OCCURRED AT A SUBMERGED WEAKENED-PLANE JOINT; RIGHT, WEAKENED-PLANE JOINT WITH SURFACE GROOVE.

It will be recalled that these sections are 500 feet long and that the steel reinforcement is continuous through the joints. Two of the sections are reinforced with a 91-pound and two with a 45-pound welded fabric. The bond is broken between the steel and the concrete for a distance of 36 inches at each joint.

Cracks have occurred in the surface of the pavement over all except one of the submerged weakened-plane joints. The high stressing of the steel and the breaking of the bond at the joints has allowed these joints to open an appreciable amount. An irregular meandering crack typical of those over the submerged weakened-plane joints is shown in figure 21.

The weakened plane joints of the conventional type, which were formed by placing grooves in the top surface of the pavement, all appear to be in excellent condition. The appearance of a typical joint of this type is shown in figure 21. The manner in which the seal has been maintained at these joints is especially impressive. This tight seal can undoubtedly be attributed to the fact that the short slab lengths and the continuous steel through the joints have reduced to a very small amount the changes in width of the joints caused by the expansion and contraction of the pavement.

(*Continued from p. 216*)

tossed from passing cars. Constant patrolling did not prevent the destruction by fire of 55 percent of all mats in one Southwestern State. Losses by theft can be minimized by stencilling a serial number and ownership.

In conclusion, the data from 19 States indicate that the cost of curing concrete pavements with cotton mats should not exceed that of other accepted methods. The survey also corroborates the laboratory findings that such mats not only retain moisture in the concrete but also have the valuable property of controlling temperatures in the slab, thus providing a type of protection not afforded by the usual surface-sealing materials.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF DECEMBER 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 4,759,921	$ 2,284,401	188.7	$ 5,634,620	$ 2,795,383	194.8	$ 917,390	$ 457,340	25.2	$ 2,201,235
Arizona	1,690,510	1,202,718	78.3	1,765,486	1,251,066	100.5	---	---	---	484,633
Arkansas	4,932,857	3,918,101	226.3	849,062	535,804	37.4	943,598	471,403	33.1	303,090
California	4,659,289	2,536,210	77.3	4,794,259	2,547,856	82.2	1,597,920	854,300	36.1	1,725,573
Colorado	3,815,457	2,119,751	89.3	1,478,378	828,606	38.1	152,739	50,000	4.7	1,603,084
Connecticut	802,205	395,926	10.0	1,520,108	758,035	15.0	---	---	---	1,279,206
Delaware	625,675	297,212	23.7	1,169,783	582,478	14.7	181,020	90,510	.1	1,024,160
Florida	599,651	296,978	8.1	4,159,320	2,079,435	73.9	1,555,912	777,971	73.0	1,801,106
Georgia	3,670,120	1,807,761	202.8	5,219,357	2,609,679	269.5	2,647,338	1,323,669	77.3	4,571,052
Idaho	2,213,871	1,314,379	113.9	780,350	476,373	49.7	221,232	129,875	28.1	994,131
Illinois	8,604,082	4,291,979	190.2	4,932,952	2,464,487	91.1	3,315,150	1,656,575	68.8	1,242,641
Indiana	3,453,840	1,714,428	70.5	5,168,353	2,577,770	108.7	374,694	187,272	8.5	1,819,732
Iowa	3,929,715	1,826,856	190.7	3,310,255	1,460,088	96.4	601,388	284,500	27.4	592,132
Kansas	3,282,010	1,633,293	174.6	2,251,644	1,124,943	118.7	2,916,543	1,454,724	120.7	3,676,351
Kentucky	2,870,347	1,424,314	98.2	2,257,419	1,127,153	45.0	906,765	453,382	22.9	2,696,361
Louisiana	662,972	328,400	23.9	11,733,431	2,915,603	36.6	1,719,885	842,776	50.6	2,288,599
Maine	2,183,320	1,084,502	52.4	717,470	358,735	16.7	13,240	6,620	.6	304,657
Maryland	1,601,408	765,386	23.9	2,470,873	1,217,055	39.5	330,000	155,500	4.6	1,775,766
Massachusetts	3,134,614	1,564,618	25.1	647,015	322,744	4.5	1,375,898	685,030	9.7	2,453,696
Michigan	4,780,068	2,344,648	115.4	2,636,100	1,223,150	89.2	2,436,100	1,201,968	69.5	1,594,786
Minnesota	4,809,253	2,382,604	347.0	4,384,442	2,172,301	192.2	945,766	472,262	52.1	3,037,406
Mississippi	1,268,800	442,660	68.7	8,971,158	3,459,395	343.3	3,046,050	1,299,320	147.6	910,972
Missouri	3,245,097	1,619,407	139.3	3,433,978	1,674,000	105.5	3,601,713	1,437,193	98.7	3,590,981
Montana	3,084,266	1,745,139	185.9	2,411,593	1,366,493	112.7	799,839	453,668	89.3	3,102,650
Nebraska	3,948,865	1,968,342	316.7	3,960,326	1,924,162	375.9	1,922,545	872,392	237.8	2,272,526
Nevada	1,113,680	953,001	53.6	1,029,617	884,238	48.3	70,884	60,713	4.3	635,701
New Hampshire	821,381	404,233	27.3	640,094	313,308	12.1	86,349	42,787	3.4	968,020
New Jersey	532,620	257,786	3.6	4,472,728	2,234,814	34.7	1,717,280	858,640	15.4	969,544
New Mexico	1,785,561	1,098,079	134.7	1,368,261	845,017	78.3	60,211	37,577	19.2	861,430
New York	8,400,260	4,151,472	165.0	11,263,872	5,503,695	168.6	2,286,690	910,445	21.7	186,832
North Carolina	5,145,090	2,563,935	311.5	4,079,716	2,041,597	200.0	812,960	400,310	39.9	1,082,248
North Dakota	267,630	143,396	41.7	1,250,475	670,099	78.0	2,177,590	1,167,133	238.4	3,364,995
Ohio	5,945,384	2,913,332	84.3	6,065,632	3,009,442	48.3	7,919,680	3,833,755	76.6	3,246,966
Oklahoma	1,845,213	975,461	94.7	2,680,817	1,421,884	78.0	2,735,980	1,425,253	94.5	2,520,061
Oregon	2,197,800	1,313,287	102.4	3,022,433	1,641,282	96.4	835,177	501,160	40.3	645,456
Pennsylvania	9,212,237	4,569,464	106.8	4,957,406	2,323,347	45.9	2,907,642	1,431,178	32.5	2,824,608
Rhode Island	601,970	300,865	7.8	834,354	416,221	8.3	201,660	100,830	2.0	825,972
South Carolina	2,165,870	978,200	81.9	1,146,434	500,809	38.3	1,002,559	456,048	105.7	1,837,626
South Dakota	3,462,070	1,912,922	325.4	2,587,710	1,452,650	288.0	1,741,050	974,540	250.7	2,429,786
Tennessee	3,666,788	1,756,626	88.5	2,601,146	1,300,573	49.9	2,323,982	1,162,991	48.5	2,577,890
Texas	10,483,154	5,152,441	611.3	8,122,374	4,035,603	338.8	6,619,513	3,100,477	289.4	1,885,353
Utah	2,121,204	1,526,922	94.4	735,960	531,685	49.7	233,000	163,920	5.4	550,874
Vermont	737,850	361,494	18.4	717,904	358,762	22.7	73,574	36,765	3.0	312,409
Virginia	2,245,860	1,119,948	75.9	2,043,635	973,984	41.4	1,511,560	748,321	39.9	501,827
Washington	2,180,171	1,122,095	38.4	3,258,253	1,606,809	25.9	659,350	283,900	9.1	251,005
West Virginia	1,572,469	849,275	47.4	1,901,105	941,414	45.3	1,007,278	499,334	22.0	1,676,142
Wisconsin	5,217,356	2,565,431	187.9	4,978,600	2,443,880	153.0	99,163	46,805	2.6	1,653,298
Wyoming	1,494,690	922,143	141.3	1,244,964	776,622	123.7	604,868	381,837	62.1	107,909
District of Columbia	373,200	186,600	2.5	264,124	132,062	2.4	103,000	42,188	.8	126,650
Hawaii	366,881	179,428	4.6	786,412	379,777	13.1	601,757	299,199	9.8	1,033,746
Puerto Rico	661,760	329,805	13.8	1,350,769	668,830	25.6	---	---	---	376,160
TOTALS	153,246,362	79,917,654	5,906.0	160,062,527	77,261,198	4,766.5	70,915,482	34,584,356	2,723.6	80,799,034

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF DECEMBER 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 497,335	$ 242,657	27.8	$ 704,332	$ 251,393	21.8	$ 208,000	$ 103,200	11.1	$ 628,006
Arizona	234,160	168,053	25.9	63,551	45,318	7.5	102,861	61,149	16.0	266,292
Arkansas	814,879	680,644	77.1	192,460	177,987	21.3	51,765	34,812	5.8	123,677
California	1,111,026	588,216	43.5	410,544	222,524	7.9	77,458	42,762	8.6	508,031
Colorado	868,570	475,530	31.3	170,641	95,656	5.3	85,425	12,521	1.9	3,086
Connecticut	163,280	67,660	2.9	110,274	54,954	.2				233,855
Delaware	80,623	39,067	17.5	69,537	34,768	7.8				236,350
Florida	401,617	196,550	14.5	501,929	250,507	17.3	113,046	56,523	1.3	361,465
Georgia	303,338	148,082	39.4	195,242	97,621	19.9	390,372	195,186	45.0	950,010
Idaho	457,974	252,546	45.1	128,663	78,481	12.8				105,847
Illinois	1,287,704	637,468	77.1	997,700	443,550	56.7	861,050	430,525	35.5	168,904
Indiana	892,023	442,650	77.7	299,900	148,746	18.3	112,122	56,050	9.4	641,861
Iowa	274,439	130,917	107.0	1,072,286	484,900	104.0	421,565	200,100	85.4	863,890
Kansas	79,068	39,534	43.9	393,524	196,762	6.1	396,115	198,057	19.7	1,147,592
Kentucky	703,407	219,513	61.4	944,343	288,532	47.1	377,637	116,918	39.6	223,613
Louisiana	769,604	356,350	65.2	207,412	103,706	17.3	192,626	87,037	15.0	285,478
Maine	432,057	215,560	25.4	68,900	33,244	3.5	8,300	4,150	.5	7,660
Maryland	204,891	98,787	16.0	149,696	62,848	6.5	157,000	58,355	8.8	315,691
Massachusetts	373,212	185,203	9.2	284,559	140,915	5.9	190,910	94,705	4.2	368,845
Michigan	1,147,358	563,811	105.6	772,020	386,010	39.7	400,350	200,175	22.5	588,386
Minnesota	704,354	347,853	62.4	460,468	230,234	68.4	223,841	68,041	21.5	1,012,377
Mississippi	176,500	88,250	6.8	954,162	470,746	68.9	188,760	94,380	17.1	497,490
Missouri	805,975	383,820	117.4	609,607	289,288	60.7	148,360	58,237	24.9	536,383
Montana	835,992	474,167	71.6	79,149	44,829	6.6	224,054	127,084	27.1	685,948
Nebraska	787,248	381,890	168.9	542,898	264,268	77.5	171,059	85,530	23.5	285,151
Nevada	161,442	133,635	25.0	108,814	93,565	20.1	83,455	71,207	5.4	43,449
New Hampshire	61,156	29,708	2.3	83,280	39,815	3.1	39,055	17,534		132,907
New Jersey	298,990	146,755	10.2	277,250	138,625	14.2	82,880	41,440	.9	469,373
New Mexico	464,923	285,942	42.1	286,665	98,695	13.8	101,564	63,386	13.1	90,619
New York	1,708,811	846,046	90.5	1,748,035	827,073	45.6	501,500	185,550	10.6	70,635
North Carolina	994,549	497,260	94.6	468,883	237,053	39.1	147,790	73,740	19.8	238,792
North Dakota	115,030	61,606	8.3	37,500	20,100	.1	66,953	35,886	2.7	842,938
Ohio	611,800	304,590	36.6	665,089	339,320	25.6	1,399,900	699,950	49.9	951,492
Oklahoma	99,638	48,414	4.7	485,956	258,569	28.3	576,470	306,728	43.3	713,734
Oregon	613,273	347,752	67.6	210,575	89,530	23.5	78,861	47,227	14.5	254,393
Pennsylvania	2,008,795	989,867	113.8	1,209,868	598,829	42.3	237,638	118,819	8.8	175,855
Rhode Island	93,827	46,890	2.2	81,236	40,618	.2	161,211	72,824	3.4	23,483
South Carolina	562,159	228,890	56.9	100,120	36,467	8.3	331,300	114,000	26.3	208,904
South Dakota	16,550	9,100	4.1	11,056	6,088					1,042,862
Tennessee	811,584	352,928	31.7	146,416	73,208	.7	117,367	58,684	7.5	701,718
Texas	2,064,104	1,016,281	239.0	752,478	359,806	63.7	787,330	389,705	88.3	554,884
Utah	224,185	126,765	38.5	100,365	61,098	6.2	20,755	15,430	2.1	128,769
Vermont	145,522	71,278	5.6	164,292	50,336	6.3	54,115	14,000	1.8	40,849
Virginia	632,234	306,985	65.5	308,590	133,848	8.9	321,770	143,490	23.4	94,528
Washington	576,973	300,487	43.8	274,792	144,118	16.9				223,397
West Virginia	145,150	72,575	8.3	237,515	118,757	12.3	167,223	83,611	10.1	317,823
Wisconsin	898,036	446,948	34.5	321,054	160,190	4.7	379,319	179,040	4.4	464,067
Wyoming	466,828	288,087	26.0	336,904	190,329	23.5	53,943	34,052	15.5	5,517
District of Columbia	98,700	49,350	1.0	18,992	8,996	.2				14,779
Hawaii	89,393	44,391	3.7	370,944	185,976	11.0				89,633
Puerto Rico				224,465	109,130	12.8	55,188	27,140	2.1	60,233
TOTALS	28,370,286	14,477,308	2,397.1	19,414,931	9,317,926	1,140.4	10,868,263	5,178,940	798.3	19,007,521

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF DECEMBER 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 930,463	$ 917,059	9	2		$ 362,507	$ 361,884	8			$ 35,800	$ 35,800	1		5	$ 789,346
Arizona						518,061	515,813	4								209,120
Arkansas	184,063	184,017	3			669,853	663,348	3			24,235	24,235	2		8	607,099
California	1,156,964	1,156,419	8		1	1,122,970	960,073	6			58,155	58,155	1			948,940
Colorado	615,740	609,909	5		12	17,217	17,217			3	31,856	28,096			10	796,962
Connecticut						194,056	182,342		1	7	222,418	221,637	3			607,587
Delaware	7,839	7,839	2								2,320	2,320			1	516,132
Florida	417,700	417,700	2			207,047	202,548	2			11,928	11,928			3	1,034,415
Georgia	196,480	196,480	5			481,230	481,230	7	3		299,669	299,669	4	2	21	1,888,641
Idaho	204,578	172,785	3			110,538	110,176	1			129,626	100,000	2			366,886
Illinois	2,633,965	2,576,105	17	4	48	1,047,599	966,227	4		19	1,058,888	891,799	4		37	1,264,707
Indiana	756,162	756,162	3	1	54	545,876	545,876	3		31	291,979	291,979		1	54	708,413
Iowa	516,492	465,606	11		40	549,553	514,250	3		117	519,714	434,825	3		92	687,993
Kansas	933,632	933,632	11		5	479,834	479,834	5			459,902	459,901	7		10	611,296
Kentucky	592,898	588,597	8	4	1	689,484	689,484	6		1	284,875	284,875	2		17	359,677
Louisiana	122,838	122,830	2			824,494	770,989	7			317,665	317,659	10			584,469
Maine	331,672	329,136	3	2		206,701	206,701	2	1		924	924			1	235,425
Maryland	24,510	15,403				240,795	240,795	2		2	351,200	336,200	2		13	635,098
Massachusetts	400,519	399,288	2	2		122,047	122,014	2			14,320	14,320	1			1,711,447
Michigan	554,209	551,869	4	2	21	1,060,490	1,060,490	6		13	92,840	92,840	1	1	10	1,320,343
Minnesota	506,912	480,951	4	4	12	1,453,727	1,452,633	10	2	5	122,930	122,930		1	2	872,900
Mississippi	133,900	133,900	2			477,473	477,473	6			246,200	246,200	4	1		681,016
Missouri	402,019	400,714	2	1		1,150,721	1,150,721	7	1		410,364	385,086	2	4		1,373,439
Montana	850,426	850,426	9			213,154	98,073	3			81,312	80,000	2			227,848
Nebraska	625,227	624,627	19		9	726,276	726,276	6	2	13	105,241	105,241			28	486,168
Nevada	196,253	196,253		2	11	23,243	22,341		1	2						104,088
New Hampshire	100,927	100,459	7		1	107,462	107,426	3			91,061	91,061	1			222,539
New Jersey	141,570	141,570	1		1	745,976	745,976	2	3		133,371	133,371	1			1,156,364
New Mexico	59,805	59,805	2			17,848	17,848			1	61,946	61,946	1			630,719
New York	1,540,834	1,535,486	4	6		2,219,008	2,180,908	9	9		1,218,340	1,156,170	4	4		2,131,819
North Carolina	1,145,914	1,110,814	6	4	36	509,209	509,209	7	2	7	326,205	326,205	2		23	595,015
North Dakota	528,012	479,610	7	1		395,927	395,927	5			25,170	25,170			8	441,991
Ohio	453,690	438,690	7	1	2	1,502,813	1,431,861	8	2		714,510	714,510	5		3	2,477,960
Oklahoma	266,389	265,852	3		32	187,025	187,025	3		11	631,602	630,702	13	1	12	1,591,385
Oregon	40,500	39,002				269,872	268,578	3								316,052
Pennsylvania	255,686	255,686		1		2,185,368	1,973,469	7	3		276,449	276,449	5		2	4,157,624
Rhode Island	442,828	442,828	1	3		7,406	7,406									141,016
South Carolina	358,289	324,907	6	3	7	484,787	473,493	4	4		232,965	232,965	1	3	26	643,085
South Dakota	329,373	329,373	3	2	16	79,265	79,265	1	1	1	47,100	47,100	1			996,477
Tennessee	283,757	283,757		2	5	589,158	589,158	3	2							1,339,595
Texas	1,527,206	1,495,940	15	2	4	2,006,068	1,955,492	15	1	1	752,530	687,473	7		21	1,255,934
Utah	192,113	191,963	3		64	167,368	167,368			57	63,300	63,300			23	174,437
Vermont	34,676	29,864			7	225,102	202,214		2							119,233
Virginia	691,979	599,079	7	3	18	150,211	150,211	3			85,295	85,295	1		2	834,481
Washington	294,179	292,768	3		13	174,519	174,519	1	2		131,647	131,647	1	1	1	317,040
West Virginia	64,417	64,417	2			323,834	308,074	7			11,700	11,700			4	969,214
Wisconsin	883,349	879,905	9		2	858,603	812,506	7	2	9	444,636	422,403	2	1	6	652,989
Wyoming	136,598	136,441	1	1	7						85,910	85,910	1			433,605
District of Columbia						366,812	333,268	1								47,053
Hawaii	52,950	50,320	1			140,452	140,452	3	1		236,535	236,525	2		1	115,323
Puerto Rico	49,040	48,840	1			345,312	343,310	8								426,676
TOTALS	23,169,542	22,685,083	223	53	429	27,554,351	26,573,771	203	45	300	10,744,633	10,266,521	99	20	444	41,817,081

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

FEDERAL WORKS AGENCY

PUBLIC ROADS ADMINISTRATION

VOL. 20, NO. 12 ▽ FEBRUARY 1940

TRAFFIC DURING A STUDY OF VEHICLE PASSING PRACTICES

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PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

FEDERAL WORKS AGENCY

PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 20, No. 12 February 1940

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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CERTIFICATE: By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

PROGRESS IN STUDY OF MOTOR-VEHICLE PASSING PRACTICES[1]

BY THE DIVISION OF HIGHWAY TRANSPORT, PUBLIC ROADS ADMINISTRATION

Reported by O. K. NORMANN, Associate Highway Economist

KNOWLEDGE of the manner in which highways are used is a prerequisite to improving their design so that they will more adequately serve highway users. A study of passing practices of motor vehicles is part of a research program recently initiated by the Public Roads Administration to supply information on the normal driving habits of vehicle operators.

During the fall of 1938 studies of the passing practices of motor vehicles were conducted on four sections of highway in Maryland and Virginia with special equipment developed by the Public Roads Administration. A report describing in detail the methods and equipment used, and the purposes of the passing-practice studies, has been published.[2]

In cooperation with State highway officials, studies were conducted during the summer of 1939 in Massachusetts, Ohio, and Illinois, and studies are now being conducted in Texas. The program also includes studies in California and Oregon next spring. Upon completion of the field work, data will be available for normal passing practices under a wide variety of road conditions, geographically distributed to include any major differences in driving habits.

There has been so much interest manifested wherever the equipment has been in operation that, in addition to supplying most of the personnel for the field work, different State officials have felt that the information obtained will be of such immediate value to them that they have desired to supply the personnel necessary for a complete analysis of the data.

Several improvements have been made in the equipment to reduce the time required to install it on the study sections and to permit operation at night and on rainy days. The most important improvement has reduced the amount of work required in transcribing the field records. This is the major item of expense in the studies and has been reduced to less than two-thirds of its former cost.

The detailed data for 1,635 passing maneuvers recorded during 37½ hours of operation on the four study sections in Maryland and Virginia are now ready to be placed on tabulating machine cards prior to starting the comprehensive analysis. Although these 1,635 passings are but a small portion of the total number that will be obtained during this study, they will be used to illustrate the method of analysis and some of the facts with respect to passing practices and driver behavior that are being obtained.

OVER HALF OF PASSINGS ACCOMPLISHED BY MULTIPLE MANEUVERS

The first classification of the passing maneuvers was made by separating them into the single- and multiple-passing types. In the single-passing maneuvers, one vehicle passed one other vehicle, while in the multiple-passing maneuvers, two or more vehicles either passed or were passed by one or more vehicles.

Table 1 shows that 57.3 percent of the passings were accomplished by multiple maneuvers although there were only about half as many multiple maneuvers as there were single maneuvers (one vehicle passing two other vehicles accounts for two passings). These figures illustrate the importance of including in a study of passing distances and practices a study of multiple-passing maneuvers as well as single-passing maneuvers.

TABLE 1.—*Types of passing maneuvers observed (average traffic volume 375 vehicles per hour)*

Type of maneuver	Maneuvers made		Passings accomplished	
	Number	*Percent*	*Number*	*Percent*
Single	1,096	67.0	1,096	42.7
Multiple:				
1 vehicle passing 2 vehicles	181	11.1	362	14.1
2 vehicles passing 1 vehicle	161	9.8	322	12.6
1 vehicle passing 3 vehicles	63	3.9	189	7.4
2 vehicles passing 2 vehicles	42	2.6	168	6.5
3 vehicles passing 1 vehicle	30	1.8	90	3.5
1 vehicle passing 4 to 6 vehicles	31	1.9	136	5.3
2 vehicles passing 3 to 5 vehicles	13	.8	102	4.0
All other multiple passings	18	1.1	99	3.9
Total multiple	539	33.0	1,468	57.3
Grand total	1,635	100.0	2,564	100.0

The most important multiple-passing maneuvers are those in which one vehicle either passes or is passed by two vehicles. They compose 63.5 percent of the multiple maneuvers and 46.6 percent of the passings accomplished by multiple maneuvers. Three vehicles passing four other vehicles was the most complicated multiple-passing maneuver recorded.

Figure 1 shows, for various hourly volumes, the percentage of the total number of maneuvers and passings accomplished by multiple-passing maneuvers. At an hourly traffic volume of 200 vehicles, 35 percent of the total passings were accomplished by multiple maneuvers. At traffic volumes above 450 vehicles per hour, this figure exceeds 60 percent.

Figure 2 shows the average number of maneuvers and passings observed per hour on the four half-mile study sections during various hourly traffic volumes. As expected, there is a marked increase in the number of passings as the volume increases. These are the number of passings accomplished per hour and not the number of passings that would have been made had all vehicle operators that desired to pass been able to make passing maneuvers.

One vehicle usually passes another vehicle because the driver wants to travel faster than the other vehicle is moving. Within the half-mile study section it was generally possible to determine the speed that the driver of the passing vehicle desired to travel by noting his speed either before slowing down prior to making the passing maneuver or after the maneuver was completed.

Table 2 shows that in 55 percent of the passings the passed vehicle was traveling from 31 to 40 miles per hour. The speeds of the passed vehicles in nearly all of the remaining passings were almost equally distrib-

[1] Paper presented at the Nineteenth Annual Meeting of the Highway Research Board, December 6, 1939.

[2] Procedure Employed in Analyzing Passing Practices of Motor Vehicles, by E. H. Holmes, PUBLIC ROADS, vol. 19, No. 11, January 1939.

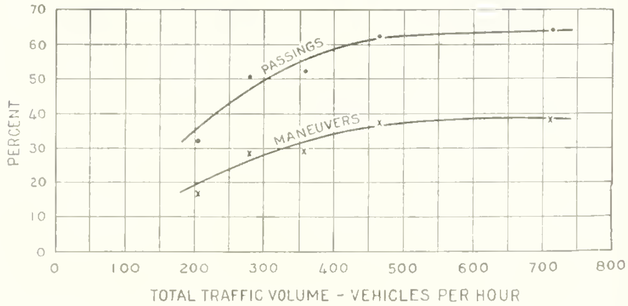

FIGURE 1.—PERCENTAGES OF TOTAL PASSING MANEUVERS AND TOTAL PASSINGS THAT WERE ACCOMPLISHED BY MULTIPLE-PASSING MANEUVERS AT VARIOUS TRAFFIC VOLUMES.

uted between the 21 through 30 and 41 through 50 mile-per-hour groups.

This table also shows that 51.4 percent of the drivers that passed desired to travel less than 11 miles per hour faster than the passed vehicle and that the desired speed of 21.2 percent was less than 6 miles per hour faster. There is a marked decrease in the average difference between the speed of the passed vehicle and the desired speed of the passing vehicle as the speed of the passed vehicle increases. Beside showing the frequency distribution of the speeds of passed vehicles, these data indicate that drivers desiring to travel at a slightly higher speed than the vehicle ahead would rather pass the preceding vehicle when the opportunity presents itself than reduce their speed slightly and stay behind.

TABLE 2.—*Single passings classified by the speed of the passed vehicle and the desired speed of the passing vehicle*

Desired speed of passing vehicle in miles per hour faster than speed of passed vehicle	Speed of passed vehicle in miles per hour: 20 and under	21-30	31-40	41-50	Over 50	Total
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
5 and under		1.9	11.2	7.8	0.3	21.2
6-10		4.0	18.8	7.1	.3	30.2
11-15	0.4	6.7	17.6	5.5	.3	30.5
16-20	.7	5.0	5.7	.8		12.2
21-30	.3	2.9	1.6	.3	.1	5.2
Over 30	.3	.2	.1	.1		.7
Total	1.7	20.7	55.0	21.6	1.0	100.0
Average difference in speed between passed and passing vehicle (miles per hour)	20.6	14.2	10.5	8.6	11.1	10.9

Of all the drivers that were able to accomplish single passing maneuvers on the study sections, table 3 shows that 84.4 percent had to slow down before they could start to pass; 53.7 percent slowed down to practically the same speed as the vehicle they were going to pass, and 16 percent slowed down to within 5 miles per hour of the speed of the vehicle they were going to pass. About one-third had to slow down and stay behind the preceding vehicle until they could see that the road was clear for a sufficient distance ahead to permit them to pass, and 50.9 percent had to slow down and wait for an oncoming vehicle to pass before they could start the passing maneuver. The other 15.6 percent were not required to slow down prior to starting the maneuver. They may have had to slow down after completing the maneuver but they started the maneuver at their normal speed.

When the drivers of the passing vehicles had completed the passing maneuvers and returned to the right-hand lane, table 4 shows that the left lane was clear for a distance of less than 500 feet in 27 percent of the passings and that there was an oncoming vehicle less than 500 feet away in 16.8 percent of the passings. The data for the passings in which the passing vehicle was not forced back into the right lane by oncoming traffic or limited sight distance may not be very useful in determining minimum passing distances, but they will show actual driving practices during unrestricted conditions. Driving practices during unrestricted as well as restricted conditions must be known when designing highways to fit the normal driving habits of vehicle operators.

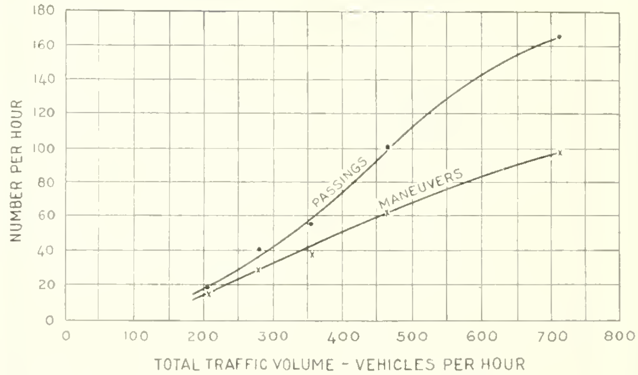

FIGURE 2.—TOTAL NUMBER OF MANEUVERS AND PASSINGS ACCOMPLISHED PER HOUR ON ½-MILE STUDY SECTIONS AT VARIOUS HOURLY TRAFFIC VOLUMES.

TABLE 3.—*Percentage of vehicles making single-passing maneuvers that were delayed before starting to pass (average traffic volume 375 vehicles per hour)*

	Delayed by insufficient sight distance	Delayed by oncoming vehicle	Total
	Percent	*Percent*	*Percent*
Slowed down to same speed as vehicle to be passed	18.0	35.7	53.7
Slowed down to within 5 miles per hour of the speed of the vehicle to be passed	6.6	9.4	16.0
Slowed down, but not to within 5 miles per hour of speed of vehicle to be passed	8.9	5.8	14.7
Total delayed in starting maneuver	33.5	50.9	84.4
Total not delayed in starting maneuver			15.6
			100.0

TABLE 4.—*Distance that drivers of passing vehicles could see that left lane was clear at the time the passing maneuver was completed (average traffic volume 375 vehicles per hour)*

Distance that left lane was clear (feet)	Sight distance limiting factor	Oncoming car in view	Total
	Percent	*Percent*	*Percent*
Less than 500	10.2	16.8	27.0
500 to 1,000	19.0	16.0	35.0
Over 1,000	30.5	7.5	38.0
Total	59.7	40.3	100.0

ILLUSTRATION OF DATA OBTAINED FOR PASSING MANEUVERS

The data obtained for each passing maneuver are illustrated by figures 3 to 9 inclusive, each representing one of seven critical positions. All speeds, distances, time intervals, and relative positions of each vehicle with respect to the other vehicles as shown in these figures were obtained from the data sheet for one passing maneuver.

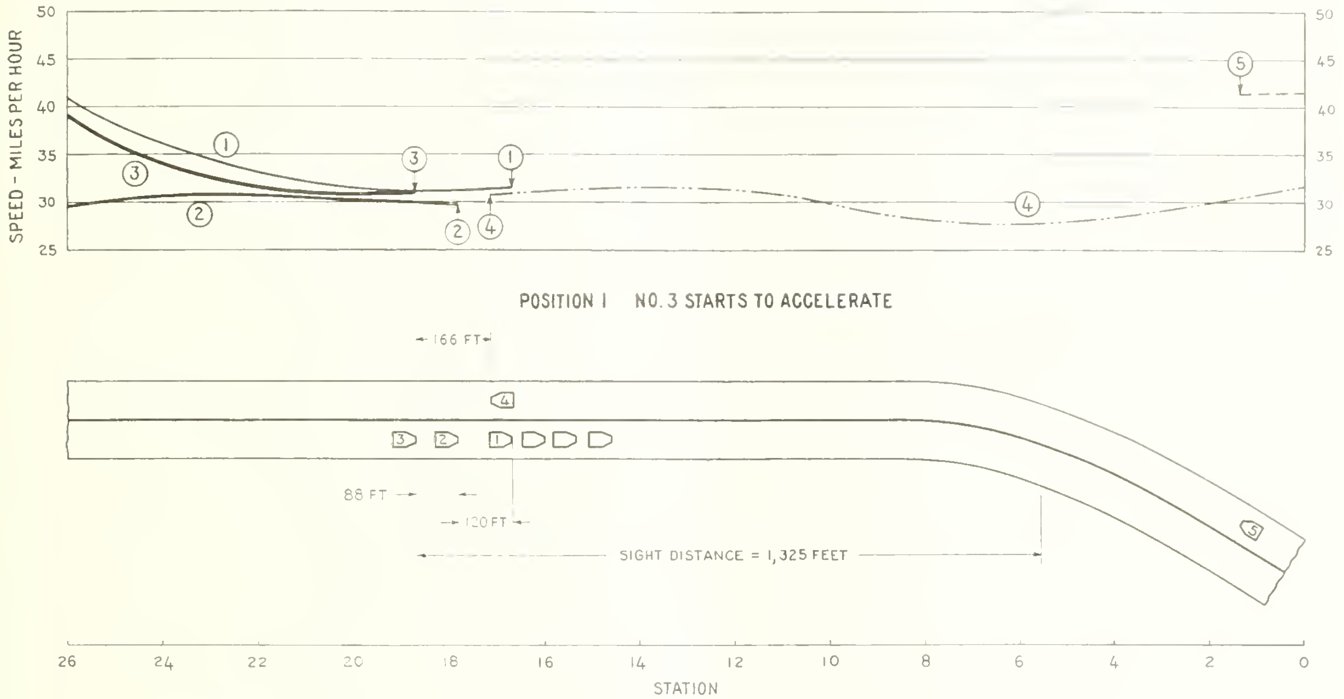

FIGURE 3.—CRITICAL POSITIONS AND SPEEDS OF VEHICLES AT START OF PASSING MANEUVER.

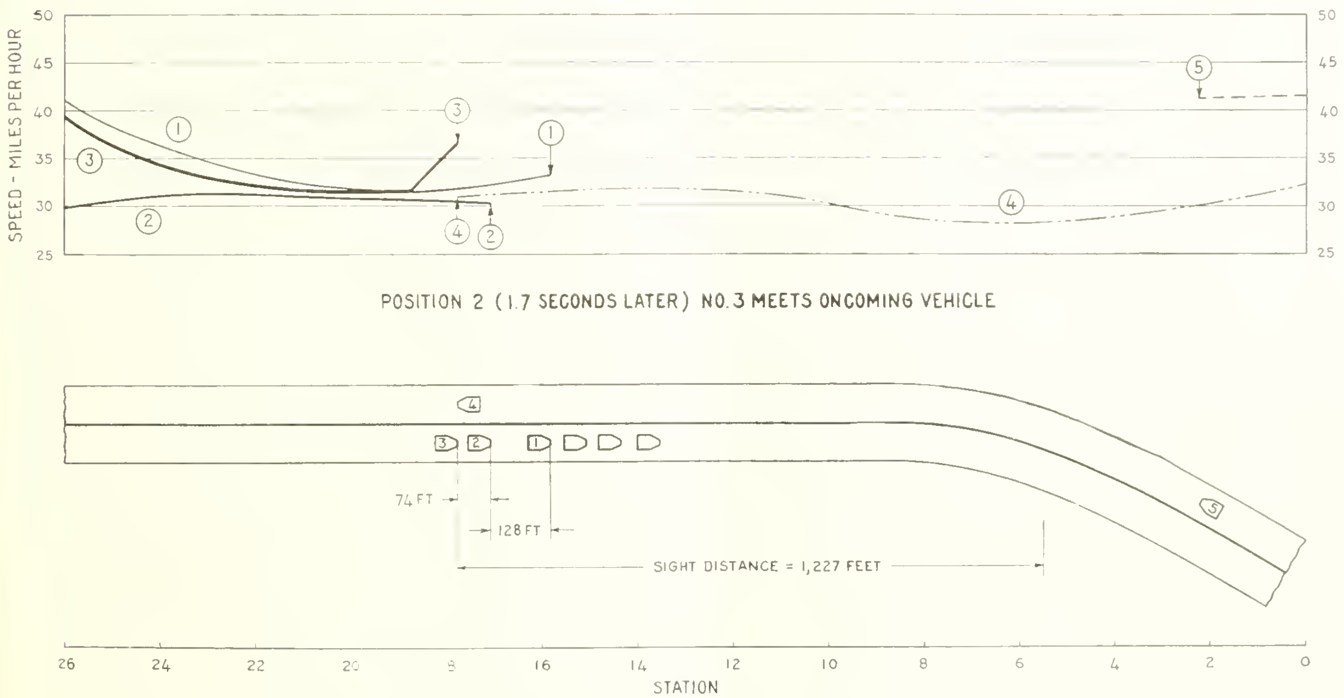

FIGURE 4.—CRITICAL POSITIONS AND SPEEDS OF VEHICLES AT SECOND STAGE OF PASSING MANEUVER.

Similar data have been compiled for the 1,635 maneuvers recorded on the study sections in Maryland and Virginia and also for 500 of the maneuvers recorded during the Massachusetts studies. It is not intended that this much detail be obtained for all the thousands of passings that will have been recorded when the scheduled field work is completed, but the factors that appear to be the most important as the analysis progresses will be taken from the field records for enough maneuvers to obtain a representative sample of each type of maneuver at a series of speeds for each available road condition.

Figure 3 shows the position of each vehicle that is likely to affect the manner in which the passing maneuver is made. At this instant, vehicle No. 3 starts to accelerate in order to pass vehicle No. 2 and possibly the four vehicles ahead of vehicle No. 2, the closest one being vehicle No. 1, a distance of 120 feet ahead of No. 2. All dimensions between vehicles represent the distances between the front of vehicles. Vehicles Nos.

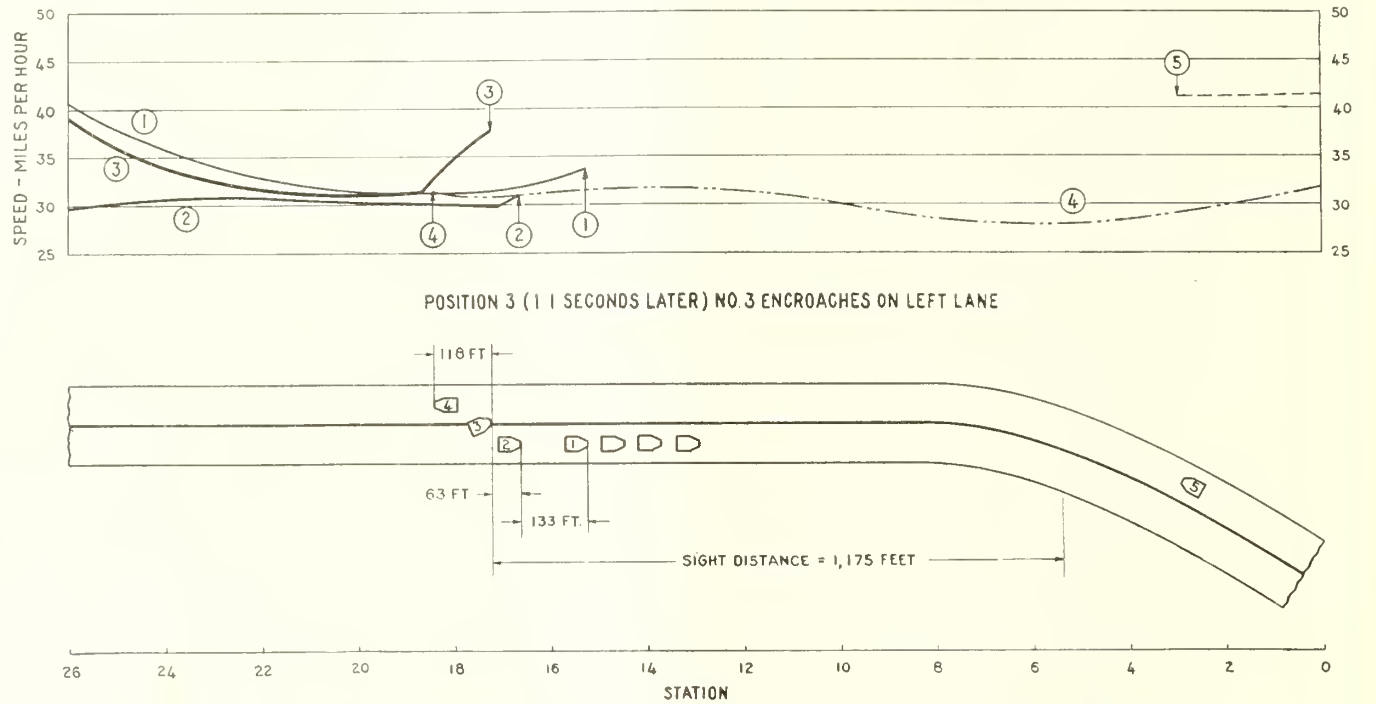

FIGURE 5.—CRITICAL POSITIONS AND SPEEDS OF VEHICLES AT THIRD STAGE OF PASSING MANEUVER.

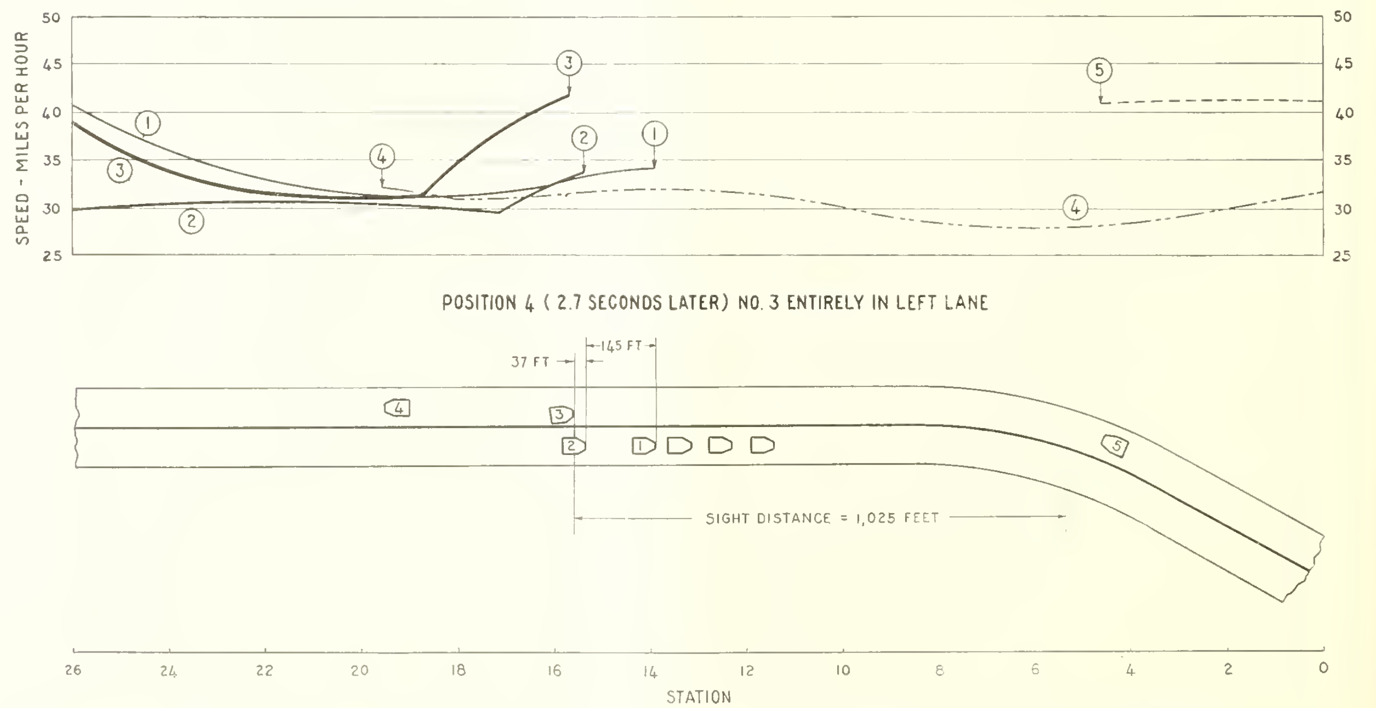

FIGURE 6.—CRITICAL POSITIONS AND SPEEDS OF VEHICLES AT FOURTH STAGE OF PASSING MANEUVER.

4 and 5 are oncoming vehicles in the opposing lane of traffic, No. 4 being the vehicle met by No. 3 before encroaching on the left lane, and No. 5 being the first oncoming vehicle met by No. 3 after completing the maneuver. The space between No. 4 and No. 5 represents the "hole" available in the opposing lane of traffic.

At the top of figure 3 is shown the speed of each of the five vehicles over the portion of the study section traversed up to this point. Vehicle No. 3 entered the section traveling about 40 miles per hour but has been required to slow down to 31 miles per hour, the approximate speed that No. 2 has maintained. (Table 3 indicated that 53.7 percent of the observed passing maneuvers were started after the passing vehicle had slowed down to the same speed as the vehicle to be passed.)

The data sheets for another passing maneuver indicate that vehicle No. 1 has just finished passing No. 2 and has also slowed down to a speed of about 31 miles per hour. Vehicle No. 4 is approaching at a speed of 31 miles per hour and No. 5 at a speed of 42 miles per

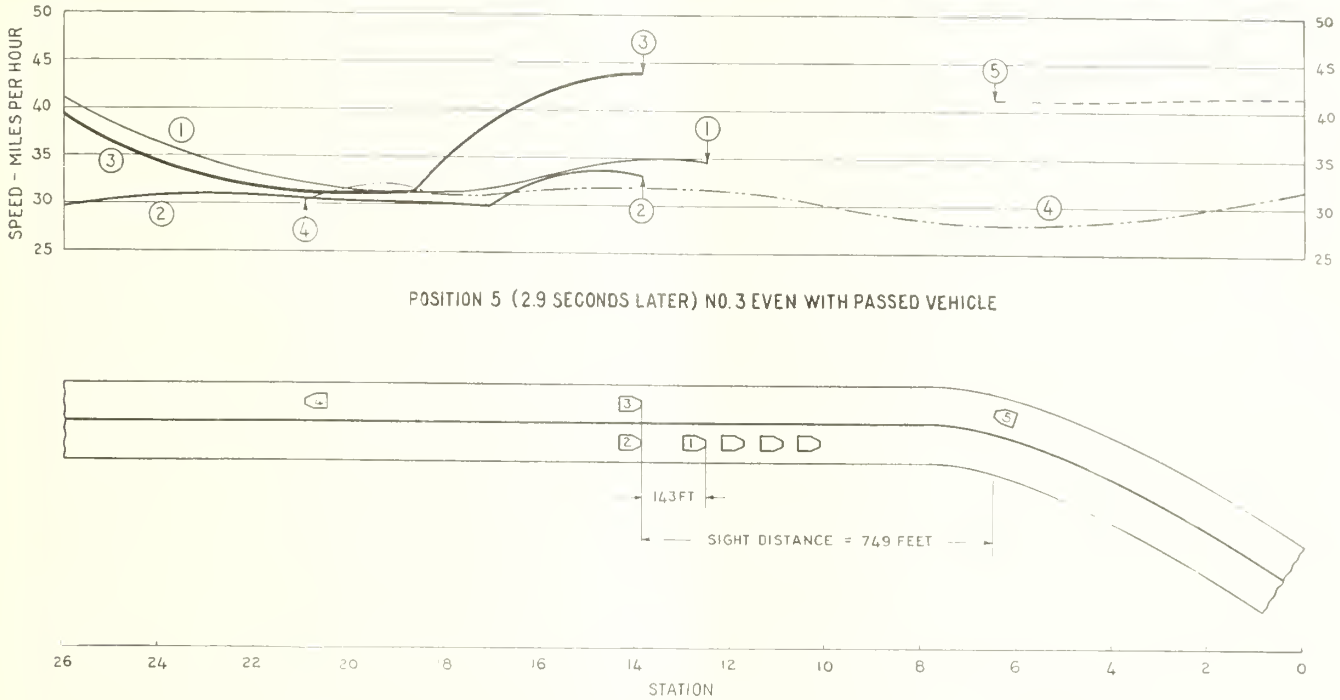

Figure 7.—Critical Positions and Speeds of Vehicles at Fifth Stage of Passing Maneuver.

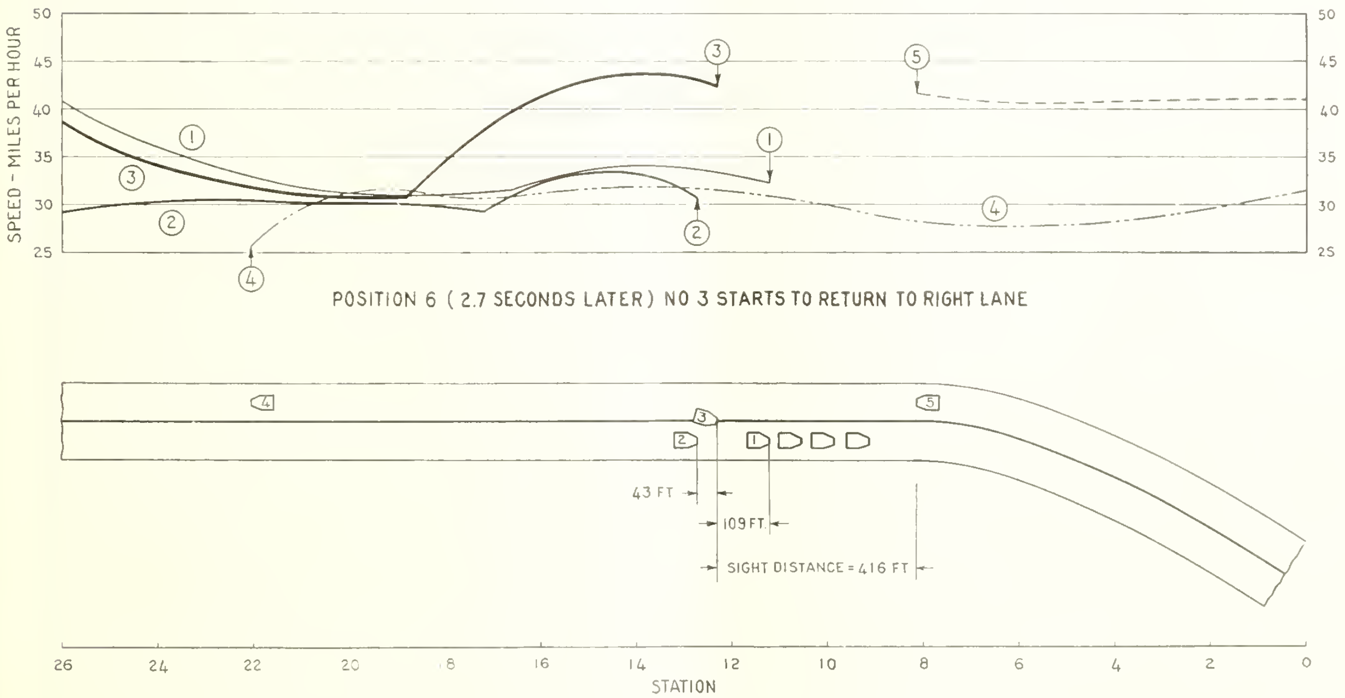

Figure 8.—Critical Positions and Speeds of Vehicles at Sixth Stage of Passing Maneuver.

hour, but No. 5 cannot be seen by vehicle No. 3. The fact that vehicle No. 3 starts to accelerate at this point indicates that the driver has already decided to attempt to pass even though No. 4 is still 166 feet away.

In the second position, occurring 1.7 seconds later and shown by figure 4, the fronts of No. 3 and No. 4 are parallel; No. 3 has accelerated to 36 miles per hour and is now 74 feet behind vehicle No. 2. It is immediately after this instant that the driver of vehicle No. 3 has his first opportunity to enter the left lane without hindrance from oncoming traffic (vehicle No. 5 still being out of sight).

In the third position, occurring 1.1 seconds later and shown by figure 5, vehicle No. 3 first encroaches on the left lane while 63 feet behind vehicle No. 2 and traveling 7 miles per hour faster than the vehicle to be passed.

In the fourth position, taking place 2.7 seconds later (fig. 6), No. 3 is entirely in the left lane for the first time and is still 37 feet behind vehicle No. 2. In the meantime vehicle No. 2, which has been traveling at a uniform speed throughout the first 900 feet of the section, starts to accelerate. The driver evidently does not like the idea of being passed or unintentionally steps on the accelerator. He cannot accelerate very

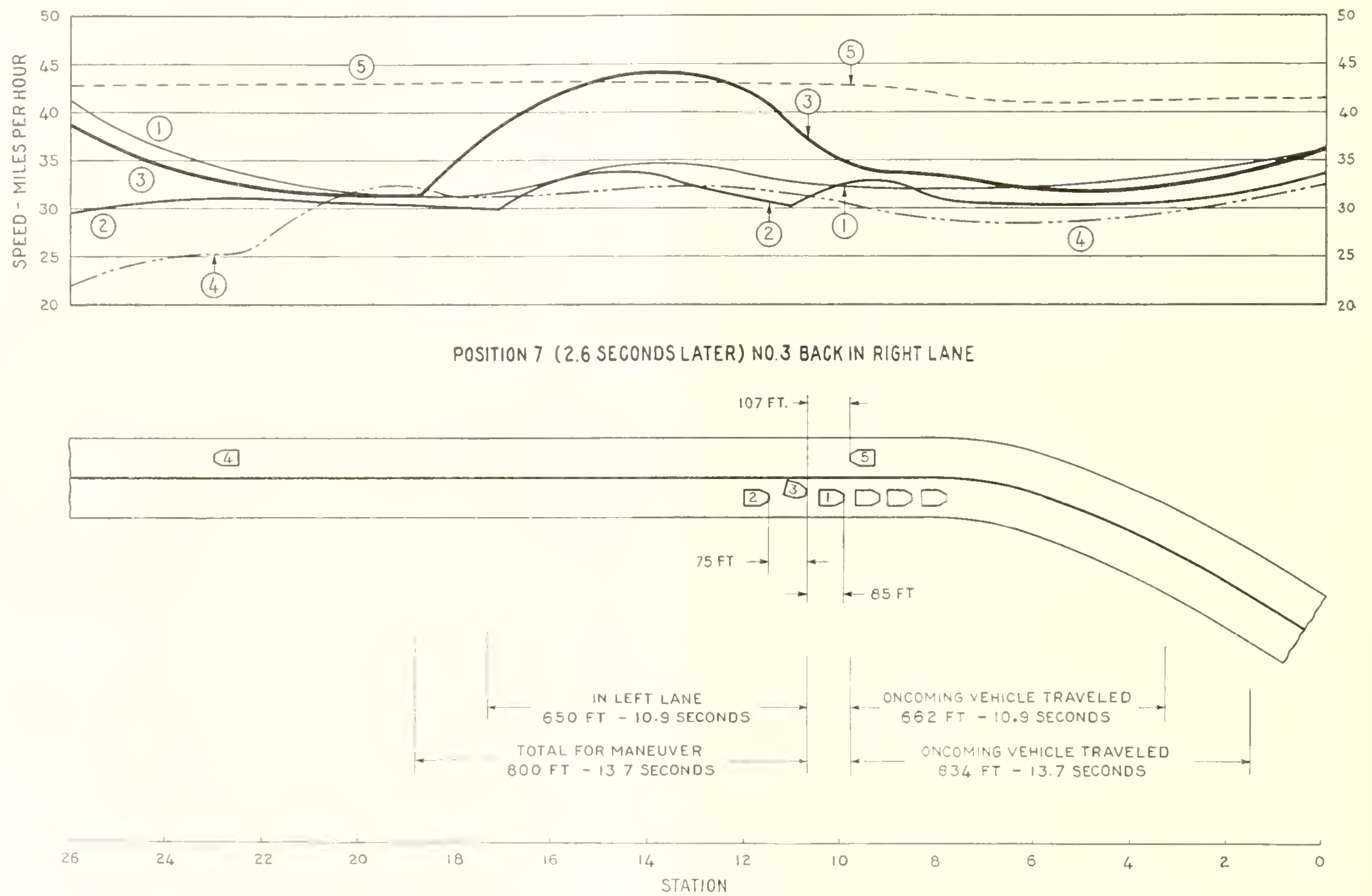

Figure 9.—Critical Positions of Vehicles After Completion of Passing Maneuver and Speeds of Vehicles Through Entire Study Section.

long without hitting No. 1 but he can reduce the space between his car and vehicle No. 1 so that No. 3 will be required to crowd his way back to the right lane. Neither No. 2 nor No. 3 can as yet see the oncoming car No. 5.

Position 5 (fig. 7) occurs 2.9 seconds after position 4, and the passing vehicle is now even with No. 2 and is no longer accelerating. Vehicle No. 2 has decelerated a little and the oncoming vehicle No. 5 is now in view.

From figure 8, representing the sixth position of the maneuver, it is seen that the driver of the passing vehicle has decided not to try to pass more than one vehicle and is now cutting back into the "hole" between No. 1 and No. 2. This occurs 2.7 seconds after No. 2 and No. 3 were parallel. Vehicle No. 3 is traveling 10 miles per hour faster than vehicle No. 1.

Figure 9, the seventh position, 2.6 seconds later, shows that the passing vehicle completed the maneuver when 107 feet from the oncoming car. After returning to the right lane, the speed of No. 3 decreased until it was below the speed of vehicle No. 1 and then increased to the same speed. All vehicles slowed down slightly going around the curve.

The acceleration and deceleration curve for the passing vehicle during the maneuver, as shown in figure 9, indicates a maximum acceleration of 2.3 miles per hour per second. This is considerably lower than any rate that would be assumed in calculating the minimum passing distance under similar conditions.

It required 13.7 seconds or 800 feet to complete the maneuver. The passing vehicle spent 10.9 seconds and traveled 650 feet in the left lane. The approaching vehicle traveled 834 feet during the maneuver and 662 feet while the passing vehicle was in the left lane. The net result of the passing maneuver is that vehicles No. 2 and No. 3 have reversed their respective positions. Providing there are sufficient passing sections and "holes" in oncoming traffic along the remaining portion of this highway, vehicle No. 3 might be able to pass two or three more of the cars of this group before getting to the next town, thereby arriving a little sooner than if no attempt were made to pass. It may be possible for No. 3 to pass all the preceding vehicles in this group at the next section of highway with a long sight distance, providing oncoming traffic does not interfere. Vehicle No. 3 could then continue at the desired speed until catching up with the next group of cars and might reach its destination somewhat sooner than by staying in line. It is evident that this section of highway with a sight distance of 1,900 feet provided practically no relief from restricted travel conditions for this particular vehicle.

MOST PASSINGS STARTED WITH SIGHT DISTANCE OF 1,000 FEET OR MORE

Such a detailed analysis of each passing necessarily takes more time than it would to obtain only such factors as the average vehicle speeds and the total passing distances, but the relative value of the results will more than compensate for the increased work.

There are so many variable factors involved in each passing that it would require almost an unlimited number of passings to obtain a representative sample of each type of passing maneuver possible by different

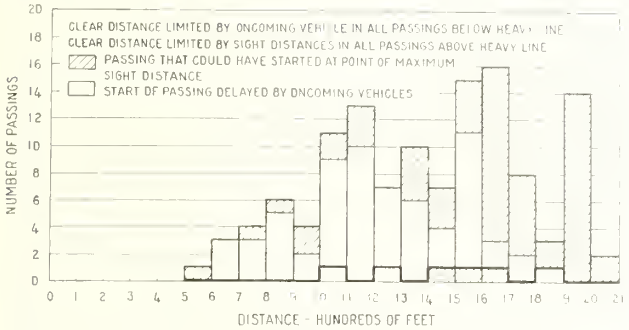

FIGURE 10.—MAXIMUM DISTANCE THAT DRIVERS OF PASSING VEHICLES COULD SEE THAT LEFT LANE WAS FREE OF ONCOMING TRAFFIC AT THE TIME THEY ENTERED THE LEFT LANE DURING PASSING MANEUVERS. (INCLUDING ONLY THOSE VEHICLES THAT SLOWED DOWN TO SAME SPEED AS VEHICLES TO BE PASSED PRIOR TO STARTING THE PASSING MANEUVERS ON STUDY SECTION.)

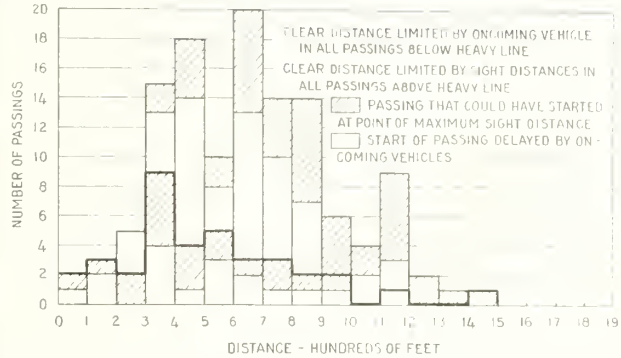

FIGURE 11.—MAXIMUM DISTANCE THAT DRIVERS OF PASSING VEHICLES COULD SEE THAT LEFT LANE WAS FREE OF ONCOMING TRAFFIC AT TIME PASSING MANEUVERS WERE COMPLETED. (INCLUDING ONLY THOSE VEHICLES THAT SLOWED DOWN TO SAME SPEED AS VEHICLES TO BE PASSED PRIOR TO STARTING THE PASSING MANEUVERS ON STUDY SECTION.)

combinations of variables. As the same variable appears in many different types of passing maneuvers, a break-down of each passing into its component parts will permit a representative sample of each variable to be obtained from a much smaller total number of passings than if the break-down is not made. The variables may then be recombined to form composite passings covering all types. It is in this respect that the method of analysis, made possible by the complete field records, will differ from other analyses of passing distances and practices that have been made.

The determination of distances involved in a passing maneuver is a relatively simple, though laborious, operation, but the determination of the effect of highway alinement and of driver psychology upon future design requirements is far more difficult. Even though sections of sufficient sight distance to complete individual passing maneuvers in safety are provided, they will not serve their purpose unless the drivers take advantage of their opportunities to pass. Figures 10 and 11 have been constructed from the data for single passings that took place at one particular study location, where vehicles traveling in either direction had maximum and minimum sight distances of 1,900 and 200 feet, respectively. To eliminate a number of variables, only the passings in which the passing vehicle started the maneuver while traveling at the same speed as the vehicle to be passed have been used.

In figure 10, the passings have been classified by the maximum distance that the driver of the passing vehicle could see that the oncoming traffic lane was clear at the time he encroached on it. Three of the drivers encroached on the left lane when this distance was between 1,800 and 1,900 feet; eight encroached when this distance was between 1,700 and 1,800 feet, etc. In 85.5 percent of the passings, the distance was over 1,000 feet and no maneuvers were started when this distance was less than 500 feet.

Sixteen vehicles encroached on the left lane before reaching the point of maximum sight distance. Fourteen were within 100 feet of this point and two were between 100 and 200 feet away. As the measured sight distance decreased to 250 feet immediately before reaching the point of maximum sight distance, the drivers of the 16 vehicles either could see farther than the measured sight distance because of unusual vehicle construction, or the drivers started the passing maneuvers knowing or hoping that the sight distance would increase while they still had a chance to return to the right lane in case an oncoming vehicle came into view. All 16 vehicles reached the point of maximum sight distance before getting entirely in the left lane.

The passings in each of these distance groups have been divided by a heavy horizontal line into two groups. The sight distance limited the distance that the driver could see a clear left lane in all the passings above the heavy horizontal line. There was an oncoming vehicle in sight when the driver encroached on the left lane in all the passings shown below the heavy line. In only six of the passings did the driver encroach on the left lane when there was an oncoming vehicle in view and not a passing was started when the driver could see an oncoming vehicle within 1,000 feet. With no oncoming vehicle in view, passings were started when the sight distance was as low as 500 feet but relatively few were started below 1,000 feet, there being a marked drop at this point.

OPPORTUNITIES TO PASS NOT UTILIZED TO MAXIMUM EXTENT

The passings are further classified into two groups as determined by whether or not the driver had to wait for an oncoming vehicle before starting the maneuver. All of the passings that are cross-hatched represent maneuvers in which the passing vehicle was immediately behind the vehicle to be passed upon reaching the point of maximum sight distance and could have started the passing maneuver immediately. The areas that are not cross-hatched represent passings that could not have been started at the point of maximum sight distance, because the driver had to wait for oncoming traffic to pass before encroaching on the left lane. Sixty-one (49.2 percent) of the 124 passings that were made could have been started immediately upon reaching the point of maximum sight distance. In the other 50.8 percent, the passing vehicle waited for an oncoming car after reaching the point of maximum sight distance before starting to encroach.

This one figure in itself contains an enormous quantity of information regarding driver behavior. The number of drivers that did not attempt to pass until after reaching a point where the sight distance was considerably less than it was where they could have started to pass is surprisingly large and offers excellent data for a study of combined perception and judgment time.

(Continued on page 237)

SAMPLE OF DATA OBTAINED IN STUDY OF MOTOR-VEHICLE PASSING PRACTICES IN ILLINOIS

By F. N. BARKER, Engineer of Highway Research, Illinois Division of Highways

A COMMON and one of the most annoying conditions that tend to reduce the traffic capacities of two-lane rural highways is the presence on these roads of vehicles moving at slow speeds. Where sight distances are restricted, where oncoming traffic is heavy, or where other conditions are such that passing maneuvers cannot be executed safely, these slow-moving vehicles restrict vehicle drivers who wish to travel at greater speeds. Under such conditions passenger vehicles move slowly because they are compelled to, not because of any mechanical limitation of the vehicle itself. Motortrucks, however, are often loaded to a weight in excess of that which the engine is able to move at a reasonable road speed, especially on the steeper grades.

During September and October 1939, the Illinois Division of Highways cooperated with the Public Roads Administration in conducting studies of truck performance on medium and heavy grades, using equipment developed by the Public Roads Administration and described in the January 1939 issue of PUBLIC ROADS. These studies were made on typical two-lane roads carrying normal traffic volumes and were conducted so that the presence of the observers and equipment had no effect on the performances of the vehicles Detectors spaced at 50-foot intervals over the test section were the only part of the equipment visible to the drivers. These detectors were small rubber tubes, connected through pneumatic switches and electrical circuits to recording instruments. Vehicles passing through the course actuated successive pens on the instruments, which recorded the time of passage of each vehicle through successive segments of the course. Travel on each of the two lanes was recorded on separate charts, so that a vehicle straddling the center line was recorded on both charts. The license numbers of commercial vehicles were recorded as they entered the course and these vehicles were stopped and weighed a considerable distance beyond the section.

Analysis of the data collected in the Illinois study is yet in a preliminary stage. Until this analysis is completed for all of the data collected in different sections of the State under various conditions of traffic and road alinement so as to provide a complete cross section of vehicle behavior on grades, and until these results are compared with the results of similar studies made in other States, no definite conclusions can be drawn or recommendations made. It is already apparent, however, that studies of this nature will ultimately provide a wealth of reliable data which will be useful in establishing design standards and in determining reasonable and desirable standards of motor-vehicle performance.

As a demonstration of the type of basic data obtained in truck performance studies, figure 1 has been developed. This is a chart of the passage of a typical group of vehicles through a course located on U. S. Route 66 near Edwardsville, Ill., between 3 : 09 : 45 p. m. and 3 : 14 : 55 p. m. (a period of 310 seconds) on October 4, 1939. This location was on a 20-foot pavement consisting of an 18-foot brick surface, with 1-foot concrete edges and a light gutter on each side. The rate of grade was 6 percent within the limits of the course. Conditions were ideal on the day of the observations—pavement dry, weather clear, and visibility excellent.

PROGRESS OF EACH VEHICLE THROUGH TEST SECTION CHARTED

For convenience, vehicles ascending the grade are numbered on figure 1 in the order of their entrance into the section. Opposing vehicles are not numbered because there were few of them and their paths through the section are easily followed. The vertical scale shows the distance the vehicle has traveled from the bottom of the grade at the times shown on the horizontal scale.

Information readily taken from the chart indicates:

1. The position of each vehicle at any time with respect to any other vehicle on the section at the same time, regardless of direction of travel.
2. The time and point on the course at which each vehicle first encroached on the opposing traffic lane, the duration of this encroachment in time and distance, and the time and point at which the vehicle returned to the proper lane.
3. The speed of each vehicle through each 50-foot section of the course (indicated by the slope of the line representing the progress of the vehicle).
4. The acceleration or deceleration of each vehicle through any portion of the course (indicated by the rate of change of the slope of the line representing the vehicle's progress).

Vehicle No. 1 was a tractor-truck semitrailer combination with a manufacturer's rated capacity of 1½ tons and a gross weight of 16,500 pounds. This combination entered the course at 20 miles an hour, but slowed to about 5½ miles an hour at 450 feet, or after 25 seconds of travel on the course. This speed gradually decreased to 4.9 miles an hour at which speed the vehicle left the test section.

Vehicle No. 2, which was a passenger car following this combination as it entered the course, crossed into the opposing traffic lane after traveling 250 feet on the course, with the apparent intention of passing vehicle No. 1. An oncoming vehicle prevented this maneuver so that vehicle No. 2 was forced to return to its proper lane. The driver of the oncoming vehicle decreased his speed appreciably until he was certain that the dirver of vehicle No. 2 had abandoned his attempt to pass, after which he accelerated and left the course at 30 miles an hour. After meeting the descending vehicle, the driver of vehicle No. 2 accelerated to a speed of 25 miles an hour, passed vehicle No. 1, and left the course at a speed of 35 miles an hour. During this passing maneuver vehicle No. 2 encroached on the opposing traffic lane for a distance of 205 feet.

Vehicle No. 3 was a tractor-truck semitrailer combination, with a manufacturer's rated capacity of 2 tons

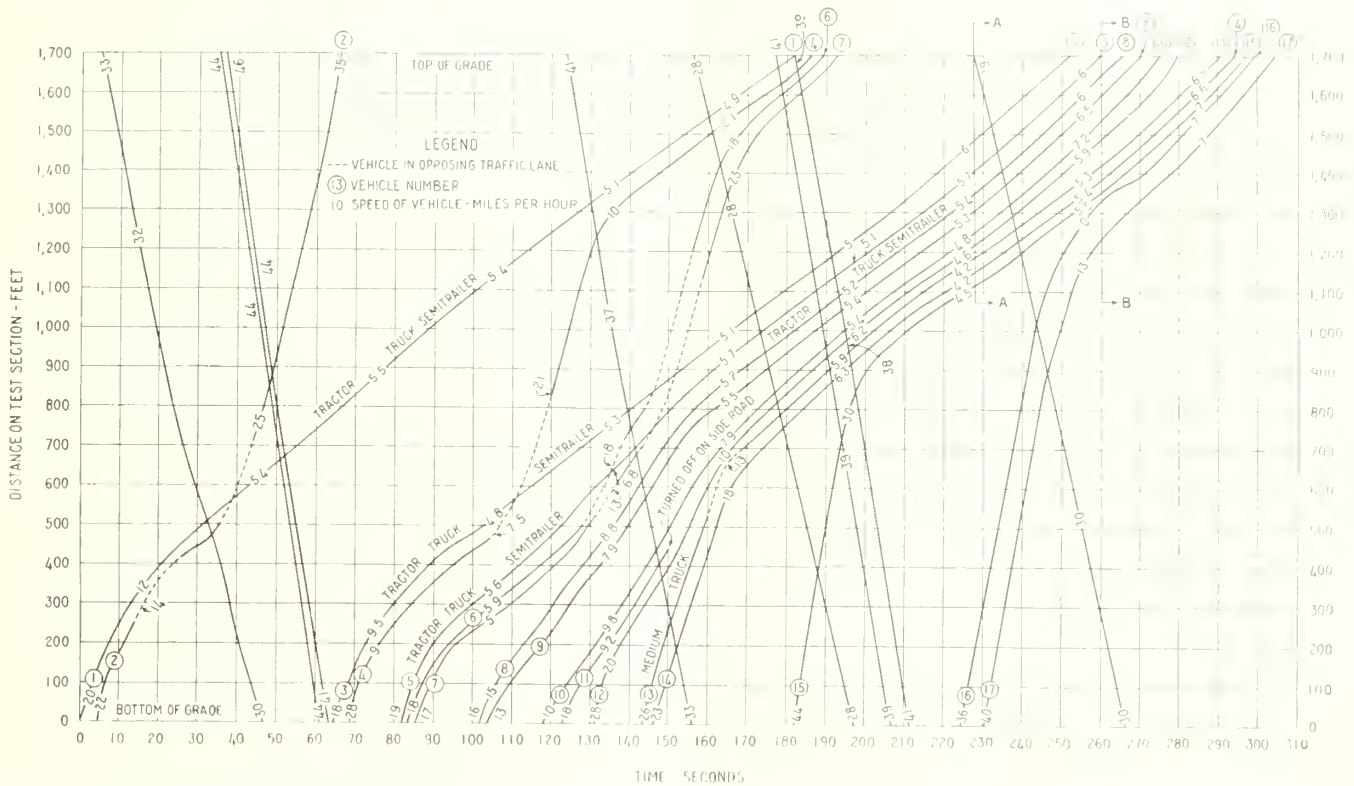

Figure 1.—Time-Distance Curves Showing Progress of Vehicles Through Test Section. Curves Sloping to Right Indicate Vehicles Ascending the Hill; Curves Sloping to Left Indicate Descending Vehicles. Ordinate Between Any Two Curves Indicates Distance in Feet Between Front Axles of Respective Vehicles. Vehicles Nos. 1, 3, 5, and 8 Are Tractor Truck Semitrailer Combinations; Vehicle No. 13 Is a Medium Truck; All Other Vehicles Are Passenger Cars.

and a gross weight of 17,000 pounds. This vehicle entered the course at 18 miles an hour, but after traveling about 450 feet its speed had decreased to 5 miles an hour, which was approximately maintained throughout the remainder of the course.

Vehicle No. 4, a passenger car, passed this combination after following it for about 475 feet on the course but was prevented by oncoming vehicles from passing vehicle No. 1. Vehicle No. 4 was forced, therefore, to follow vehicle No. 1 out of the course at a speed of about 5 miles an hour.

Vehicles Nos. 6 and 7 were successful in passing vehicles Nos. 5 and 3, vehicle No. 7 accomplishing this on the second attempt, but both were prevented by oncoming traffic from passing vehicles Nos. 1 and 4. It is apparent from the varying slope of the charted course of vehicle No. 6 that the driver considered abandoning his attempt to pass, but accelerated and completed the maneuver. It is interesting to note the small clearance between vehicle No. 6 and the oncoming vehicle as the former passed vehicle No. 5. The vertical distance between the upper end of the dotted section of his path as he passed vehicle No. 5 and the position of the oncoming vehicle at the same time indicates that vehicle No. 6 returned to its own lane with only about 100 feet of clearance from the oncoming vehicle.

The remaining ascending vehicles numbered 8 to 17, which entered the section during the period of observation covered by the chart, were all prevented by oncoming traffic from passing any of the vehicles ahead of them and were forced to follow in line behind vehicle No. 3 at its speed of about 5 miles an hour.

Vehicles Nos. 15 and 16 are the only ones recorded on this diagram entering the course at speeds which were apparently unrestricted by preceding vehicles or heavy loads. For vehicle No. 15 this speed was 44 miles an hour and for vehicle No. 16 the speed was 36 miles an hour. These are reasonable speeds for this location and it may be assumed that they are the speeds the drivers of these two vehicles desired to maintain throughout the course. As they approached the line of vehicles following vehicle No. 3, however, they were forced to decelerate to a speed approximating that of vehicle No. 3.

Pictures were taken of vehicles on the test section at points on the time axes designated "AA" and "BB" in figure 1. Figure 2 shows the vehicles at the instant designated "AA" and figure 3 the vehicles at "BB." These two photographs show a condition that is frequently encountered on sections having short sight distances, that of a group of vehicles which have accumulated behind a slow-moving vehicle and are prevented from passing by inadequate sight distances or oncoming traffic. In figure 2, vehicle No. 15 is the last car in line, while in figure 3 vehicle No. 17 is catching up with the group of vehicles ahead.

SEVERAL REMEDIES AVAILABLE TO RELIEVE SUCH CONGESTION

Two of the five commercial vehicles included in figure 1, vehicles Nos. 1 and 13, did not carry Illinois registration plates. Vehicle No. 1 was obviously loaded in excess of the weight that would allow a reasonable performance on this grade, but vehicle No. 13 carried a load well within its capacity. Examination of the charted path of this vehicle through the

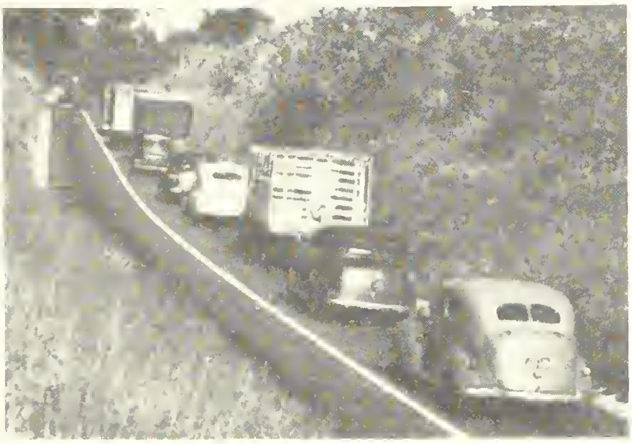

FIGURE 2.—VEHICLES CLIMBING HILL AT INSTANT DESIGNATED "AA" IN FIGURE 1.

course indicates that the driver desired to proceed up the grade at a higher speed than that which he was forced to maintain. At a point 525 feet from the start of the course vehicle No. 13 moved into the opposing traffic lane and accelerated in an attempt to pass the two vehicles ahead. Oncoming traffic prevented the passing maneuver and vehicle No. 13 was forced to remain in line.

Trucks are registered in Illinois according to graduated gross weight classifications. There is no restriction other than the higher fee which would tend to discourage registration of a vehicle of low weight carrying capacity in a high weight classification. Vehicles Nos. 3 and 5 carried consecutive Illinois license numbers, which would indicate they were probably registered by the same owner. Both carried loads within their legal classification, as they were licensed to operate in Illinois with a total gross weight of 24,000 pounds. Nevertheless, it is obvious that they were both overloaded insofar as their ability to perform satisfactorily on this grade was concerned. These two vehicles traveled several hundred feet on the section with a space between vehicles of less than 40 feet. For only a very short distance at the beginning of the section did they travel at the 300-foot spacing required by the Illinois Uniform Act Regulating Traffic on Highways.

Vehicle No. 8, although carrying a gross weight of 26,800 pounds, was also operating within its legal classification of 40,000 pounds gross weight.

From the data presented in figure 1 it may be concluded:

1. That the performances of the tractor-truck semitrailer combinations designated as vehicles Nos. 1 and 3 were not satisfactory under the loading and highway conditions under which they were operating. The same is true of vehicles Nos. 5 and 8, although this is not so strikingly apparent from an examination of the chart.

2. That the extremely slow speeds at which vehicles Nos. 1 and 3 ascended the grade resulted in inconvenience and delay to a number of vehicles which were forced to follow them through the test section.

3. That drivers of vehicles Nos. 5 and 8 violated the Illinois statute requiring that commercial vehicles traveling on rural highways maintain an interval between vehicles of not less than 300 feet.

4. That at least one passing maneuver, that when vehicle No. 6 passed No. 5, was executed under hazardous conditions.

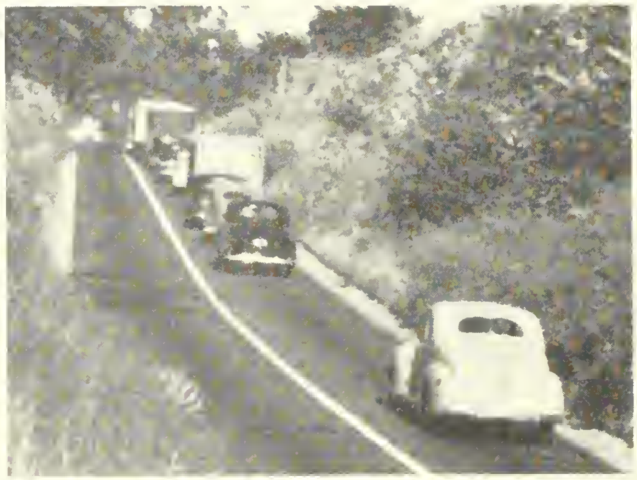

FIGURE 3.—VEHICLES CLIMBING HILL AT INSTANT DESIGNATED "BB" IN FIGURE 1.

5. That at least three of the drivers of passenger vehicles included in this group desired to travel through the course at a speed of 35 to 45 miles an hour.

There are a number of possible remedies which might be suggested to provide a less restricted movement of vehicles over this section of highway. A third lane might be constructed for the use of slow-moving vehicles; the grade might be reduced so that heavily loaded vehicles could maintain a more reasonable speed; or the loading of vehicles might be limited so that with their available power a satisfactory performance would be obtained. The same remedies might be applied to numerous locations throughout the State, but until analysis of all available data relating to the problem is completed, and these data are carefully considered by highway officials, no recommendations can be made.

PUBLICATIONS ON BRIDGE FLOOR DESIGN AVAILABLE

Three new publications that present the results of tests made to verify theoretical analyses of bridge floor designs are now available. These publications, all University of Illinois Bulletins, are No. 313, "Tests of Plaster-Model Slabs Subjected to Concentrated Loads;" No. 314, "Tests of Reinforced Concrete Slabs Subjected to Concentrated Loads;" and No. 315, "Moments in Simple Span Bridge Slabs with Stiffened Edges." The bulletins are the result of a cooperative investigation by the Engineering Experiment Station of the University of Illinois, the Public Roads Administration of the Federal Works Agency, and the Illinois Division of Highways.

The results of this investigation will have direct application to practically all modern highway bridges and to many other structural design problems and will lead to more satisfactory structures.

The Public Roads Administration has a limited number of these bulletins for free distribution. Requests should be addressed to the Public Roads Administration, Federal Works Agency, Washington, D. C.

STRESSES UNDER A LOADED CIRCULAR AREA[1]

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by L. A. PALMER, Associate Chemist

THE main purpose of this paper is to bring to the attention of engineers the fact that a precise analysis of the complete system of stresses under a uniformly loaded circular area at the surface of a semi-infinite, elastically isotropic material has been in published form for more than 10 years. Another purpose of the paper is to indicate approximate methods of checking numerical values of the stresses that have been computed by the precise methods.

Still another purpose is to indicate the possibility of using the results of the analyses as a rough guide in experimentation involved in the study of the design of flexible types of highway surfaces. In this latter connection, it is realized generally that the condition of elastic isotropy does not exist in the mass of material under an automobile tire. Hence, it would be a mistake to attempt to apply directly the analytical results, and the fact that they can serve only as a rough guide in experimentation needs to be emphasized.

Engineers are confronted with many earth problems for which no theory other than the one of elasticity has been proposed, and the treatment of earth problems apart from adequate theory is likely to lead to as many solutions as there are varieties of earth. The experimental study of such problems without reference to any theoretical basis whatsoever is an aimless procedure at best.

The problem of computing stresses at any point within a semi-infinite, elastically isotropic mass produced by a uniform load over a circular area at the plane boundary has been completely solved by A. E. H. Love [2] and S. D. Carothers.[3] The results of these two investigators are in general in good agreement and suggest a means of estimating stresses under wheel loads, since it has been shown by Teller and Buchanan [4] that the pressure distribution of a pneumatic tire on a flexible-type pavement is nearly uniform.

PROBLEM SIMPLIFIED BY MAKING REASONABLE ASSUMPTIONS

In such problems as the determination of the rate of settlement (by soil consolidation) of a foundation (fig. 1A), the vertical stresses at the upper and lower boundaries of a clay layer are computed from formulas derivable from those of Boussinesq by assuming that the entire mass of earth below the foundation is elastically isotropic. That is, the complications that may be involved, owing to the fact that the underground is comprised of alternate strata of dissimilar materials and is, therefore, not elastically isotropic, are ignored completely. Despite the questionable procedure of assuming isotropy in this case, the analysis of settlements by consolidation is one of the best established theoretical methods of soil mechanics and the present writer believes that in this instance the assumption of isotropy is as reasonable as any other assumption and is to be preferred in the interest of simplicity.

No doubt there is some refraction of the stress trajectories at the boundary of two dissimilar earth materials, that is, this boundary very likely acts as a plane of discontinuity. But any assumption as to the amount of friction between two layers of dissimilar materials such as clay and sand (fig. 1A) or as to what extent discontinuity of stresses may exist is likely to be no more valid than the assumption of isotropy in the entire earth mass below the foundation.

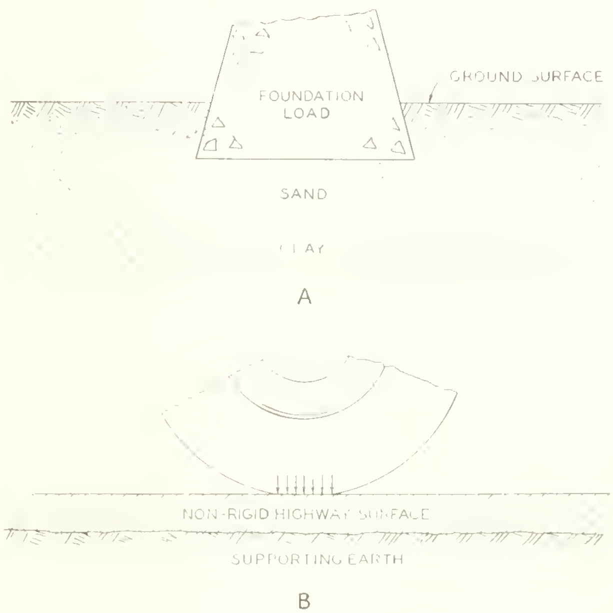

FIGURE 1.—A, FOUNDATION RESTING ON SAND UNDERLAID BY CLAY; B, WHEEL LOAD RESTING ON NONRIGID HIGHWAY SURFACE SUPPORTED BY EARTH.

For the same reason, the entire mass of materials below a wheel load (fig. 1B) may be considered as being elastically isotropic.

On the basis of such an assumption, no precise solution can be expected. Nevertheless, the theoretical development meets a definite requirement in that it shows, qualitatively at least, what happens under wheel loads and, therefore, may serve as a guide in planning methods of experimentation. All that is sought is a suggested trace of the transmission of stresses by the wheel load since rigorous results or exact formulas are out of the question.

A. C. Benkelman [5] has presented in considerable detail the present status of knowledge concerning the design of flexible pavements. It is indeed surprising to note that the conclusions reached by Carothers and Love have not been used in the technical publications discussed in Benkelman's report. However, with practically no exception, the various authors mentioned in his report have based their experimental procedures on

[1] Paper presented at the Nineteenth Annual Meeting of the Highway Research Board, December 5, 1939.

[2] The Stress Produced in a Semi-Infinite Solid by Pressure on Part of the Boundary, by A. E. H. Love, Philosophical Transactions of the Royal Society, series A, vol. 228, 1929.

[3] Test Loads on Foundations as Affected by Scale of Tested Area, by S. D. Carothers. Proceedings International Mathematical Congress, Toronto, 1924, pp. 527–549.

[4] Determination of Variation in Unit Pressure over the Contact Area of Tires, by L. W. Teller and James A. Buchanan, PUBLIC ROADS, vol. 18, No. 10, Dec. 1937.

[5] Present Knowledge of the Design of Flexible Pavements, by A. C. Benkelman, PUBLIC ROADS, vol. 18, No. 11, Jan. 1938.

theoretical considerations involving numerous simplifying assumptions which are similar in many respects to the classical theories of Love, Carothers, and Hencky.[6]

The simplest three-dimensional problem considered in the theory of elasticity is the one involving axial symmetry. Hence, the problem of a wheel load on a pavement is simplified by the assumption that the area of contact between a rubber tire and the pavement is circular, although tests indicate that the contact area is elliptic. Thus an "equivalent circular area" with uniform pressure has been used generally by investigators. The equivalent circle is one having an area equal to the elliptic area of contact and its radius is the square root of the quantity: Area of contact divided by π.

The subject of pressure distribution over a circular contact area in connection with foundation design has been discussed at length by Cummings,[7] Krynine,[8] and various others. The simplest problem is the one of uniform pressure over the contact area. In foundation problems, the actual pressure distribution depends on the relative rigidities of the structure (or loading member) and the earth mass, and in the light of present knowledge no one knows how to express these relative rigidities in quantitative terms. Another difficulty is that any formula expressing the pressure distribution over the contact area can be only approximately true when the deformations are within the elastic limits of the materials and cannot be applicable in any sense as the deformations increase more and more and become characteristic of those attending plastic yield. If an attempt is made to include all of these departures from simplicity in a theoretical development of the subject, any result would be so hopelessly complicated as to be of little practical use.

GREATEST SHEAR NEAR PERIMETER OF A UNIFORMLY LOADED CIRCULAR AREA

The outstanding conclusion of Love and Carothers was that the greatest value of the principal stress difference, S, is very close to the perimeter of the uniformly loaded circular area. This difference is twice the maximum shearing stress at any point, that is,

$$S=2\ s_{\max.} \qquad (1)$$

Table 1 is taken from Love's article and shows values of $\frac{S}{p}$, p being the unit contact pressure over the circular area, corresponding to various positions of the point Q, figure 2. The ratio, $\frac{r_1}{r_2}$, of the two radial lines, figure 2, and the magnitudes of the quantities, angle B, $\frac{z}{a}$, and $\frac{r}{a}$, a being the radius of the circle, z being the depth to the point, and B the angle between r_1 and a, determine the position of any point Q.

As the point Q moves in such a manner that the ratio, $\frac{r_1}{r_2}$, approaches zero as a limit, then according to Love

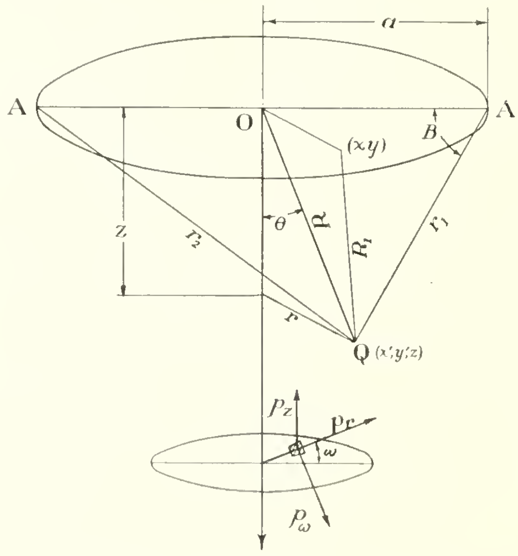

Figure 2.—Problem of the Principal Stress Difference, S, Under a Uniformly Loaded Circular Area.

the value of S depends on the angle B, or in other words, S at the point A depends on the direction of approach of point Q to point A. Love has shown that for $B=71°$ the limiting value of S as $\frac{r_1}{r_2}$ approaches zero is $0.723p$ (table 1). The $\frac{S}{p}$ values of table 1 were computed by Love for $\mu=\frac{1}{4}$ and are "partial maxima stress differences," a term that will be considered later in this report.

Table 1.—*Values of $\frac{S}{p}$ for different positions of point Q in figure 2*

[Values given by A. E. H. Love for $\mu=\frac{1}{4}$] [1]

	$\frac{r_1}{r_2}$ tends to zero	$\frac{r_1}{r_2}=\sin 5°$	$\frac{r_1}{r_2}=\sin 15°$	$\frac{r_1}{r_2}=\sin 30°$	$\frac{r_1}{r_2}=\sin 50°$	$\frac{r_1}{r_2}$ tends to unity
$\frac{S}{p}$	0.723	0.714	0.704	0.695	0.690	0.689
B	71°	67°	56°	44°	37°	32°
$\frac{r}{a}$	1	0.934	0.742	0.446	0.184	0
$\frac{z}{a}$	0	0.156	0.383	0.536	0.615	0.620

[1] The values, ¼ and 0.45 for μ, appearing in this paper are values selected by Love and Carothers, respectively, in their computations.

From table 1, the value for s_g, the greatest value of $s_{\max.}$ at any point Q in the supporting earth, is $\frac{0.723p}{2}=0.36p=\frac{S}{2}$. This is an important fact because it shows that the greatest shearing stress is at the surface where, due to lack of confinement, there is the least resistance to yield under stress. Love has given in table 1 more data than are necessary for locating the point Q, figure 2.

In texts dealing with the theory of elasticity, it is shown that when the point Q (fig. 2) is restricted to move only along the axis of symmetry, OZ, the greatest value of $s_{\max.}$ occurs at a considerable depth, z, from the surface. Thus Timoshenko[9] shows that for $\mu=0.3$, the greatest value of $s_{\max.}$ at any point on OZ is $0.33p$

[6] Über einige statische bestimmte Fälle des Gleichgewichts in Plastischen Körpen by von Heinrich Hencky, Zeitschrift für Angewandte Mathematik und Mechanik, Bd. 3, Heft 4, Aug. 1923, pp. 241–251 inc.

[7] Distribution of Stresses Under a Foundation, by A. E. Cummings, Proceedings American Society of Civil Engineers, vol. 61, No. 6, Aug. 1935.

[8] Pressures Beneath a Spread Foundation, by D. P. Krynine, Proceedings American Society of Civil Engineers, vol. 63, No. 4, pt. I, Apr. 1937.

[9] Theory of Elasticity by S. Timoshenko, Engineering Societies Monographs, first edition 1934, McGraw-Hill Book Company, Inc. (See pp. 336–337.)

and at a depth equal to $\frac{2a}{3}$. The earth at this point is confined in all directions.

Love's solution was obtained by the application of potential theory [10] and involved various elliptic integrals.

Carother's procedure is only very briefly described and therefore requires some discussion. He considers the logarithmic potential [11] of a uniform distribution of matter over a circular area of radius a, expressed by the relation,

$$\psi=\frac{p}{2\pi}\int\int\log\,(z+R_1)dx\,dy \qquad (2)$$

where dx and dy refer to the coordinates of any point x, y, on the surface and R_1 is the distance of this point to the point Q, figure 2.

By differentiating equation 2 under the integral sign,

$$\frac{\partial^2\psi}{\partial z^2}=\frac{p}{2\pi}\int\int\frac{z}{R_1^3}\,dx\,dy=\frac{pw}{2\pi} \qquad (3)$$

where w is the solid angle [12] subtended at Q by the circular area.

By use of the equations of equilibrium and the compatibility equations [13] all of the stresses at any point, Q, may be expressed in terms of w, the solid angle subtended at point Q. Thus it is found, for example, that

$$s_{rz}=z\frac{\partial}{\partial r}\frac{\partial^2\psi}{\partial z^2}=\frac{p}{2\pi}z\frac{\partial w}{\partial r} \qquad (4)$$

and

$$p_z=\frac{p}{2\pi}\left(z\frac{\partial w}{\partial z}-w\right) \qquad (5)$$

p_z being the pressure at Q that is normal to the horizontal plane and s_{rz} the shearing stress.

Similarly, p_r and p_ω (fig. 2) may be expressed as functions of the solid angle, w.

For a uniformly loaded circular area, the maximum shearing stress is obtained from the expression,

$$s^2_{max.}=\frac{(p_r-p_z)^2}{4}+s_{rz}^2 \qquad (6)$$

Love's very comprehensive tables give values for p_r, p_z, and s_{rz}, for $\mu=\frac{1}{4}$, at a great many points. Values for $\frac{S}{p}$ in addition to those given in table 1, may be computed by means of equation 6 and the other tables of Love.

The stresses at any point, Q, were computed by Carothers by expanding the various functions of w in zonal harmonics. Some of his computed values for $\frac{S}{p}$ are given in table 2. These values were computed by taking $\mu=0.45$. It is seen from this table that the greatest value of $s_{max.}$ is near the perimeter of the loaded circular area where $R=a$ and θ has values ranging from 80° to 90° (fig. 2). Love and Carothers are in good agreement as to the region of greatest shear.

Table 2.—*Values of $\frac{S}{p}$ for different positions of point Q, figure 2, according to Carothers, for $\mu=0.45$*

θ (degrees)	$\frac{S}{p}$ for $R=a$	$\frac{S}{p}$ for $R=\frac{2a}{3}$
0	0.54	0.60
30	.55	.60
45	.57	.53
60	.58	.50
75	.62	.40
80	.63	.30
85	.63	.18
90	.63	.05

MEANS GIVEN FOR CHECKING THE VALUES OF LOVE AND CAROTHERS

The method of Love, although enormously complicated, is presented in great detail and it is possible to check his numerical values, using his method. Carothers, on the other hand, gives no clue as to what particular functions he expanded in zonal harmonics in making his numerical computations. It would seem desirable then to find some means of checking the values of both Carothers and Love without using exactly the same method of either of these two investigators. This is an exceedingly difficult undertaking for the reason that Love's solution is complete, all of the stresses, p_r, p_z, and s_{rz}, being found throughout the entire region within a radial distance, a, of the loaded area. On the basis solely of his numerical values and without regard to how they were obtained, the same is true of Carothers' solution.

For the case of a uniform pressure, p, on a circular area it is easy to show [9] that for any point on the axis of symmetry and for $\mu=\frac{1}{4}$,

$$\frac{S}{p}=\frac{1}{4}+\frac{1}{4}\left[\frac{5a^2z-z^3}{(a^2+z^2)^{3/2}}\right] \qquad (7)$$

This expression may be expanded by the binomial theorem to give the infinite series,

$$\frac{S}{p}=\frac{1}{4}+\frac{5}{4}\left[\frac{z}{a}-\frac{1\times3}{2}\frac{z^3}{a^3}+\frac{1\times3\times5}{2\times4}\frac{z^5}{a^5}-\frac{1\times3\times5\times7}{2\times4\times6}\frac{z^7}{a^7}+,-\right]-\frac{1}{4}\left[\frac{z^3}{a^3}-\frac{1\times3}{2}\frac{z^5}{a^5}+\frac{1\times3\times5}{2\times4}\frac{z^7}{a^7}+,-\right] \qquad (8)$$

if a is greater than z, and

$$\frac{S}{p}=\frac{1}{4}+\frac{5}{4}\left[\frac{a^2}{z^2}-\frac{1\times3}{2}\frac{a^4}{z^4}+\frac{1\times3\times5}{2\times4}\frac{a^6}{z^6}-\frac{1\times3\times5\times7}{2\times4\times6}\frac{a^8}{z^8}+,-\right]-\frac{1}{4}\left[1-\frac{1\times3}{2}\frac{a^2}{z^2}+\frac{1\times3\times5}{2\times4}\frac{a^4}{z^4}-,+,\right] \qquad (9)$$

if z is greater than a.

From a mathematical standpoint, it is not correct to expand a function in zonal harmonics if it does not satisfy Laplace's equation. In potential theory, a Newtonian potential function, known to be harmonic (satisfies Laplace's equation) may first be computed on an axis of symmetry and thereafter it may be computed at any point not on the axis, by expanding in zonal harmonics or Legendrian polynomials.

[10] For an adequate description of potential theory, see, for example, Pt. III of Dynamics by A. G. Webster, Text, G. E. Stechert and Co., New York, second edition, 1922.
[11] See, for example, pp. 385 et seq., of Dynamics by A. G. Webster.
[12] See, for example, p. 351 of Dynamics by A. G. Webster.
[13] See, for example, top of p. 312 of Theory of Elasticity by S. Timoshenko.

[9] Theory of Elasticity, by S. Timoshenko, Engineering Societies Monographs, first edition 1934, McGraw-Hill Book Co., Inc. (See pp. 336–337.)

In general, the expression for a stress is a tensor [14] and is not therefore harmonic. However, the expression for a stress may consist of several parts, one or more of which may be harmonic, and it is legitimate to evaluate these by expanding in zonal harmonics. This, no doubt, was Carothers' procedure although he does not indicate it.

In the present consideration, the circle is not one of complete symmetry as Love has shown and in the absence of any general expression for $\frac{S}{p}$, known to be harmonic or otherwise, there is no sound mathematical basis for the following procedure which is to assume that the general expression for $\frac{S}{p}$ is harmonic and to pass from equations 7 and 8 to series of Legendrian polynomials. It is also assumed that the circle is symmetrical and this is not quite true.

If now the loaded circular area is considered as analogous to an electrically charged disk in potential theory, then one could substitute R for z in equations 8 and 9 and introduce Legendrian coefficients,[15] thereby expanding equation 7 in zonal harmonics. When this is done, equation 8 becomes

$$\frac{S}{p}=\frac{1}{4}+\frac{5}{4}\left[\frac{R}{a}P_1(\cos\theta)-\frac{1\times3}{2}\frac{R^3}{a^3}P_3(\cos\theta)+\frac{1\times3\times5}{2\times4}\frac{R^5}{a^5}P_5(\cos\theta)-\frac{1\times3\times5\times7}{2\times4\times6}\frac{R^7}{a^7}P_7(\cos\theta)\right]-\frac{1}{4}\left[\frac{R^3}{a^3}P_3(\cos\theta)-\frac{1\times3}{2}\frac{R^5}{a^5}P_5(\cos\theta)+\frac{1\times3\times5}{2\times4}\frac{R^7}{a^7}P_7(\cos\theta)\right]\text{------------}(10)$$

if all powers of $\frac{R}{a}$ beyond the seventh are neglected, equation 9 becomes

$$\frac{S}{p}=\frac{1}{4}+\frac{5}{4}\left[\frac{a^2}{R^2}P_1(\cos\theta)-\frac{1\times3}{2}\frac{a^4}{R^4}P_3(\cos\theta)+\frac{1\times3\times5}{2\times4}\frac{a^6}{R^6}P_5(\cos\theta)\right]-\frac{1}{4}\left[1-\frac{1\times3}{2}\frac{a^2}{R^2}P_1(\cos\theta)-\frac{1\times3\times5}{2\times4}\frac{a^4}{R^4}P_3(\cos\theta)+\frac{1\times3\times5\times7}{2\times4\times6}\frac{a^6}{R^6}P_5(\cos\theta)\right]\text{--}(11)$$

neglecting all powers of $\frac{R}{a}$ beyond the sixth.

EXAMPLES OF APPROXIMATE METHODS GIVEN

The terms, $P_1(\cos\theta)$, $P_3(\cos\theta)$, etc., are the Legendrian coefficients and numerical values for these coefficients, corresponding to different values of θ, figure 2, may be found in various mathematical treatises. For R less than a and $\theta=0$, equation 10 becomes equation 8 and for R greater than a and $\theta=0$, equation 11 reduces to 9.

It should be emphasized now that aside from the geometric similarity, that is, a circle and an axis of symmetry in both cases, the problem of potential at a point due to a charged disk and the problem of shearing stresses beneath a loaded circular area have nothing in common.

In table 1, for $\frac{r}{a}=0.184$, $B=37°$ and $\frac{r_1}{r_2}=\sin 50°$, it is found that $\theta=16°40'$ and $R=0.6416\ a$. The coefficients, $P_1(\cos\theta)$, $P_3(\cos\theta)$ corresponding to $\theta=16°40'$ are

$$P_1(\cos\theta)=0.9579$$
$$P_3(\cos\theta)=0.7609$$
$$P_5(\cos\theta)=0.4573$$
$$P_7(\cos\theta)=0.1207$$

Substituting these values in equation 10,

$$\frac{S}{p}=\frac{1}{4}+\frac{5}{4}(0.6141-0.3015+0.0930-0.0118)-\frac{1}{4}(0.2010-0.0744+0.0101)=0.709,$$

which compares favorably with Love's value in table 1 of 0.690.

The series in this case converges rapidly. By using Love's numerical values for p_r, p_z and s_{rz} $\left(\mu=\frac{1}{4}\right)$ and equation 6 one finds for $\frac{R}{a}=0.56$ and $\theta=45°$, that $\frac{S}{p}=0.66$. However, by equation 10 the corresponding value for $\frac{S}{p}$ is 0.75, which is 14 percent higher than Love's value. Again, for $\frac{R}{a}=0.19$ and $\theta=45°$, from Love's tables and equation 6, $\frac{S}{p}=0.42$, which is exactly the same as the value obtained from equation 10. Jürgenson[16] has computed various values for $\frac{S}{p}$, using Carothers' tables for p_r, p_z, and s_{rz} and finds for example, that for $R=2a$ and $\theta=45°$, $\frac{S}{p}=0.25$. The corresponding value found by an expression similar to equation 11 is 0.27. Love's value for $\frac{S}{p}$ for $\frac{r}{a}=0.446$ and $\frac{z}{a}=0.536$ is 0.695 (table 1), whereas equation 10 gives a value of 0.716 which is close to Love's values. For θ equal to 90° and $\frac{R}{a}$ less than 1, the value of $\frac{S}{p}$ is 0.25 by equation 10, which is Love's value in this region. Equations 10 and 11 do not fit the boundary condition for $\theta=90°$ and R greater than a.

In all other cases it may be said that for θ greater than 45° and for $\frac{R}{a}$ or $\frac{a}{R}$ greater than 2/3, the values for $\frac{S}{p}$ computed from equations 10 and 11 deviate by at least 15 percent (in many cases much more) from Love's numerical values. For all other positions of the point Q (fig. 2) the agreement is generally within 15 percent. Carothers states that for values of $\frac{R}{a}$ or $\frac{a}{R}$ approaching unity, his functions expanded in zonal harmonics were "unsatisfactory" and he gives no clue as to an alternative procedure in this case.

[14] See for example, From Determinant to Tensor, by W. F. Sheppard, Oxford at the Clarendon Press, 1923.
[15] See for example, Fourier Series and Spherical Harmonics, by W. E. Byerly, Ginn & Co., 1893.
[16] The Application of Theories of Elasticity and Plasticity to Foundation Problems, by Leo Jürgenson, Journal of the Boston Society of Civil Engineers, vol. 21, No. 3, July 1934.

In any case, equations 10 and 11 have very definite limitations in this problem although they tend to give values that are in approximate agreement with those of Love and Carothers at various points.

Functions of solid angles are for the most part of interest only to the mathematician. A practicing engineer has more confidence in numerical values if he can check them by a simple graphical method.

The following simple device is suggested by the author for this purpose. Its use is very limited. The method is as follows:

It is desired, for example, to know the value of $\frac{S}{p}$ at the point where $\theta=45°$ and $R=\frac{2a}{3}$, the value of μ being taken as 0.45 (Carothers' assumed value). In figure 3, AB is the diameter of the loaded circular area on the horizontal surface and OZ is its axis of symmetry. The radial lines AQ, QB, and $OQ=R$ are drawn. The line MN is drawn through O making an angle θ with AB. The projection of AB on MN is $A'B'$, the minor axis of an ellipse. This ellipse is in the plane that is passed through MN perpendicularly to the plane of the paper. Its major axis is equal to AB which passes through the point O and is perpendicular to MN.

The length $A'O$ is $a \cos \theta = \frac{1}{2}$ the minor axis, and $a=AO$ is $\frac{1}{2}$ the major axis. Then the area of the ellipse formed by projecting the circle in the horizontal plane on the plane MN is, $\pi a\ (a \cos \theta)=\pi a^2 \cos \theta$.

Let b=the radius of the equivalent circle, that is, the radius of the circle having an area of $\pi\ a^2 \cos \theta$. Then $\pi\ b^2=\pi\ a^2 \cos \theta$, or $b=a \sqrt{\cos \theta}$. Now the ellipse is replaced with the circle of radius$=b=a \sqrt{\cos \theta}$. A uniform pressure, p, perpendicular to the plane MN and distributed over the circle of radius $a \sqrt{\cos \theta}$ will produce approximately the same stress, $s_{max.}$, at Q as is caused by the same pressure, p, distributed over the circular area, AB, in the horizontal plane, provided θ is less than 45°. For $\theta=0$, it is observed that p_R becomes p_z.

For the general case, the value of $s_{max.}$ when the point Q is on the axis of symmetry is obtained from the expression,

$$s_{max.}=\frac{p}{2}\left[\frac{1-2\mu}{2}+(1+\mu)\ \frac{R}{(b^2+R^2)^{1/2}}-\frac{3}{2}\ \frac{R^3}{(b^2+R^2)^{3/2}}\right] \text{-- (12)}$$

where $b=a \sqrt{\cos \theta}$=radius of the equivalent circle

For $R=\frac{2}{3}a$, $\theta=45°$, $\cos \theta=\frac{1}{\sqrt{2}}$, and $\mu=0.45$,

$$\text{at point Q } s_{max.}=\frac{p}{2}\left[0.05+\frac{2\times 1.45}{3\left(\frac{1}{\sqrt{2}}+\frac{4}{9}\right)^{1/2}}-\frac{\frac{3}{2}\times\frac{8}{27}}{\left(\frac{1}{\sqrt{2}}+\frac{4}{9}\right)^{3/2}}\right]=0.296p.$$

Then $\frac{S}{p}=\frac{2s_{max.}}{p}=0.592$. Carothers obtains the value, $\frac{S}{p}=0.53$ for this point (table 2). The divergence from

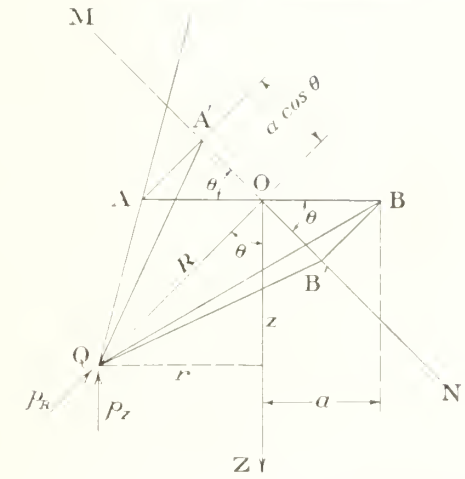

Figure 3.—The Projection of the Loaded Circular Area, AB, in the Horizontal Plane on the Plane MN is an Ellipse With Minor Axis A'B' and Major Axis AB.

Carothers' value in this case is likely indicative of the error involved in transforming the elliptic area into one that is circular.

METHOD OF KRYNINE CONSIDERED

In addition to approximate methods, such as the foregoing, there are other means of checking precise numerical values, such as those published by Love and Carothers. One such means is the photoelastic method. However, at present there has been but little progress in the analyses of three-dimensional problems of stress distribution by photoelastic devices. R. Weller [17] has suggested that it is possible to use the polarization caused by the scattering of light within a "cloudy" or opaque model in place of the usual analyzer in photoelastic investigations and that this method of polarization enables one to make analyses of three-dimensional stress systems very conveniently. An older method was to cool the model under load from an elevated temperature to room temperature and then analyze it subsequently after slicing it into plates. This is an enormously complicated procedure.

D. P. Krynine [18] has described his device, called a stereogoniometer, which offers much promise in such studies and which has the advantages of simplicity and low cost. This device makes use of the principle of projecting areas from a plane to a spherical surface rather than from one plane to another plane as was done in the preceding example. By this procedure, the stress, p_z, is found in terms of the area of the projection of the loaded surface on the sphere and the volume of the space bounded by this projected area, its projection in turn on the horizontal plane including the load, and the perpendicular lines joining the perimeters of the two projected areas. The solid angle at a point below the loaded area is equal to the projected area on the sphere divided by the square of the radius of the sphere.

It is possible to compute the solid angle w, without this device, as follows: On the axis of symmetry, the expression for the solid angle, w is

$$w=2\pi\left(1-\frac{z}{R}\right) \text{ -------------- (13)}$$

[17] A New Method for Photoelasticity in Three Dimensions, by R. Weller. Letter to the Editor, Journal of Applied Physics, vol. 10, No. 4, Apr. 1939.

[18] Stresses (especially shears) Under a Foundation by D. P. Krynine, Proc. of the Highway Research Board, vol. 18, pt. II, 1938.

where $R=(a^2+z^2)^{1/2}$. The general expression for w (see equation 3) is harmonic and it is mathematically legitimate to expand it in zonal harmonics. By so doing, one obtains the two expressions,

$$w=2\pi-2\pi\left[\frac{R}{a}P_1(\cos\theta)-\frac{1}{2}\left(\frac{R}{a}\right)^3P_3(\cos\theta)+\frac{3}{8}\left(\frac{R}{a}\right)^5P_5(\cos\theta)-\frac{15}{48}\left(\frac{R}{a}\right)^7P_7(\cos\theta)+,-,\text{etc.}\right]\text{--}(14)$$

for R less than a, and

$$w=2\pi-2\pi\left[1-\frac{1}{2}\left(\frac{a}{R}\right)^2P_1(\cos\theta)+\frac{3}{8}\left(\frac{a}{R}\right)^4P_3(\cos\theta)-\frac{15}{48}\left(\frac{a}{R}\right)^6P_5(\cos\theta)+\frac{105}{384}\left(\frac{a}{R}\right)^8P_7(\cos\theta)-,+,\text{etc.}\right]\text{-}(15)$$

for R greater than a.

For $\frac{R}{a}=\frac{2}{3}$ and $\theta=45°$, one obtains $w=1.07\pi$ from equation 14. Then substituting this value in the expression which is valid only for $\mu=\frac{1}{2}$,

$$p_z=\frac{3p}{2\pi}w\cos^2\theta\text{--------------}(16)$$

it is found that $p_z=0.803p$. The value for p_z at $R=\frac{2}{3}a$ and $\theta=45°$ is $0.858p$ when $\mu=0.45$, according to Carothers.

Similarly, for $R=2a$ and $\theta=45°$, $w=0.195\pi$ from equation 15 and on substitution in equation 16, $p_z=0.146p$. The corresponding value obtained by Carothers for $\mu=0.45$ is $0.158p$. The differential of equation 16 was proposed by D. P. Krynine.[19] The stereogoniometer is a device that performs an exact mechanical integration of the differential of equation 16, θ and w being both variable in the integration.

Equation 16 does not satisfy the boundary conditions, $\theta=0$ and $\theta=90°$. For a uniformly loaded circular area it gives results which are approximately correct for θ greater than 30° and less than 60°, R being greater than zero. On the axis of symmetry,

$$p_z=p\left[1-\frac{z^3}{(a^2+z^2)^{3/2}}\right]\text{------------}(17)$$

If θ is taken as the average angle between the vertical direction and the radial distance R drawn from any element of loaded surface to the point Q, considering all of the elements of the loaded area, the computations by equation 16 become exact. The analytical procedure of obtaining this average θ is a very difficult one. For R greater than $\frac{3}{2}a$, equation 16 together with equations 14, 15, and 17 give values for p_z that do not diverge more than 15 percent (in most cases much less) from Carothers' values when θ does not exceed 60°.

The purpose of the foregoing procedure is to indicate that in substance, at least, the mechanical method used by Krynine is correct. Its outstanding advantage is that in its use the evaluation of the stress, p_z, can be accomplished for any contour of uniformly loaded area. It is not necessary that there be axial symmetry.

[19] Shearing Stresses Under a Spread Foundation, by D. P. Krynine, Proceedings of the Eighteenth Annual Meeting of the Highway Research Board, pt. II, 1938.

LOVE'S CONCLUSIONS APPLIED TO THE CASE OF A WHEEL LOAD

The precise analytical method of obtaining p_z at a point under a uniformly loaded circular area is indicated by equation 5 which is Carothers expression for this stress.

Since

$$w=\int\int\frac{z}{R_1^3}dx\,dy,$$

Then

$$\frac{\partial w}{\partial z}=\int\int\left(\frac{1}{R_1^3}-\frac{3z^2}{R_1^5}\right)dx\,dy\text{---------------}(18)$$

and

$$z\frac{\partial w}{\partial z}=\int\int\frac{z}{R_1^3}dx\,dy-3\int\int\frac{z^2}{R_1^2}\left(\frac{z}{R_1^3}\right)dx\,dy\text{---}(19)$$

It is possible to evaluate these two integrals at any point Q. The first integral is w, known to be harmonic, and it may be computed at any desired point by means of equations 14 and 15. The second integral may be simplified by expanding $\frac{z}{R_1^3}$ in Legendrian polynomials since $\nabla^2\left(\frac{z}{R_1^3}\right)=0$. With w and $z\frac{\partial w}{\partial z}$ thus evaluated, the stress, p_z, at any point, Q, is computed from equation 5.

For a uniform pressure produced by a wheel load on the nonrigid surface, the distribution of vertical pressure, p_z, over the area of subgrade bounded by the radial line R is far from uniform as Hawthorne[20] assumed in his analysis.

Love has shown that for any given value of μ, there is a certain value of R, expressed as some multiple of a, for which $s_{max.}$ has a greatest value. Thus for $\theta=45°$, the greatest value of $s_{max.}$ on the radial line is at any point where $R=0.73a$. Similarly for $\theta=90°$, the greatest value of $s_{max.}$ is at points where $R=a$, that is, at points just under the perimeter. If the points of "partial maxima stress difference" on all of the radial lines for all values of θ are connected, the locus of such points is found to be a "basin-shaped surface of revolution about the axis of the circle." It passes through the circle and lies between two segments of spheres, which have their centers on the axis of symmetry and pass through the circle (fig. 4). These two spheres cut the axis of symmetry at depths equal to $0.620a$ and at $0.712a$ when $\mu=¼$. According to Love, it is reasonable to conclude that the foundation under a round pillar would be most likely to give way at points on such a basin-shaped surface and that it would be nearly as likely to give way at one point of this surface as at any other. The values given in table 1 are for points on the basin-shaped surface.

A practical consideration follows. If the supporting subgrade is of questionable supporting power, such as medium or soft clay, then considering all possible values of μ and the uncertainty of the effect of contact area where pavement and subgrade meet, it would be on the side of safety to have the thickness of the more resistant flexible pavement at least equal to a, the radius of the equivalent circle. This precaution would tend to confine the dangerous surface to the surfacing

[20] A Method of Designing Nonrigid Highway Surfaces, by George Edward Hawthorne, Bulletin No. 83, Engineering Experiment Station, University of Washington, Aug. 1935.

material. But it is again necessary to emphasize the fact that since the flexible pavement and the subgrade are very different materials, all such conclusions must be considered as only generally indicative and not strictly true in a quantitative sense.

The greatest values of $s_{max.}$ may be computed on the axis of symmetry from equation 12 for different values of μ. Table 3 contains such values and the points at which they are found are defined in terms of $\frac{z}{a}$. It is interesting to note from this table that the shearing stress does not vary greatly as μ varies from 0.25 to 0.50.

TABLE 3.—*Maximum values of* $\frac{s_{max.}}{p}$ *on the axis of symmetry for different values of* μ

μ	$\frac{s_{max.}}{p}$	$\frac{z}{a}$ at point of s_{max}
0.25	0.34	0.62
.30	.33	.64
.40	.31	.67
.45	.30	.69
.50	.29	.71

Benkelman [5] has rightfully emphasized our lack of information concerning conditions at the boundary plane of contact between the flexible surface and the supporting medium. It is possible that this information may be obtained by intelligent and careful experimentation, making use of adequate theory in all such experimental procedures.

[5] Present Knowledge of the Design of Flexible Pavements, by A. C. Benkelman, PUBLIC ROADS, vol. 18, No. 11, Jan. 1938.

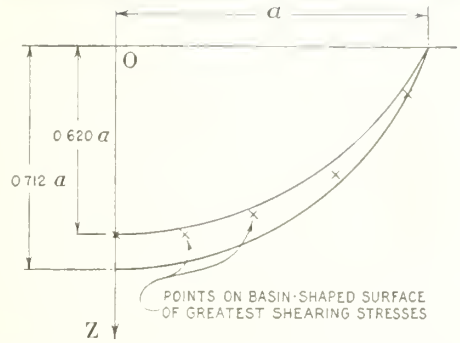

FIGURE 4.—POINTS AT WHICH THE STRESSED MATERIAL UNDER A UNIFORMLY LOADED CIRCULAR AREA WOULD BE MOST LIKELY TO FAIL.

CONCLUSIONS

On the basis of Love's complete solution of the problem of stresses under a uniformly loaded circular area, it is indicated that the greatest shearing stresses may be confined within the flexible pavement and not reach the subgrade if the thickness of the flexible pavement is no less than the radius of a circle having an area equal to the plane of contact between pneumatic tire and flexible pavement.

The conclusions reached by Carothers are in good agreement with those reached independently by Love, but the method used by Carothers requires a more complete presentation.

In the case of a uniform load on a circular area the variation in shearing stress as Poisson's ratio varies from 0.25 to 0.50 is relatively small.

(Continued from page 227)

One driver waited until the clear distance ahead decreased from 1,900 feet to less than 600 feet and then started the passing maneuver. He was fortunate, and completed the passing before an oncoming car came into view.

The horizontal scale of figure 11 shows the clear distance ahead that the driver of each passing vehicle had at the time the maneuver was completed. For all the passings shown below the heavy horizontal line, this distance represents the clearance between the vehicle that had completed the passing and the first approaching vehicle in the opposing lane of traffic. For the passings above the heavy line, no oncoming vehicles were in view, so the horizontal scale represents the sight distances when the maneuvers were completed. In this figure, as in figure 10, the passings that are cross-hatched represent maneuvers that could have started at the point of maximum sight distance. In those that are not cross-hatched, the driver had to wait for an oncoming vehicle before encroaching on the left lane.

Figure 10 showed that an oncoming vehicle was in view at the start of the passing in less than 5 percent of the maneuvers. Before the return was made to the right lane, there was an oncoming vehicle in view in about 30 percent of the maneuvers. There is a marked drop in the number of maneuvers that were completed with either a sight distance of less than 300 feet or a clearance from the oncoming vehicle of less than 300 feet. Two of the five passings that were completed with an oncoming vehicle less than 200 feet away could have been started when the maximum sight distance was available. The clearances for the vehicles involved in the two maneuvers below 100 feet were 34 and 49 feet. At the time these vehicles started to return to their right-hand lane, the distances to the oncoming vehicles were 876 and 228 feet respectively. The first vehicle could have started the maneuver when the sight distance was 1,900 feet but did not enter the left lane until the sight distance had decreased to 1,675 feet. The second vehicle had to wait for an oncoming vehicle to pass and started the maneuver with a sight distance of 625 feet.

By similar analyses of the passings that occur on different sections, it will be possible to determine the relative effectiveness of different alinements in providing for passing requirements.

Up to the present time, the analyses dealt primarily with the passings that actually occurred on the study sections. Of equal importance are the passings that the drivers wanted to make but did not attempt because they felt that the available sight distances or the "holes" in the opposing traffic were not of sufficient length to complete the passing maneuvers in safety. No attempt has been made to take this particular information from the records, but it is believed that it can be obtained. So far, it has been possible to obtain all the factors that have seemed important in a study of passing distances.

From the rather meager results that have been presented it can be seen that these studies provide, for the first time, accurate information on what actually takes place in a stream of moving traffic. They are certain to provide extremely valuable information regarding the causes of accidents even though none may actually occur during the studies.

1940 CENSUS TO PROVIDE HOUSING DATA VALUABLE TO HIGHWAY OFFICIALS

A comprehensive picture of housing and home ownership in the United States will be compiled from information to be collected by the U. S. Bureau of the Census in April when it conducts the Sixteenth Decennial Census. In response to a schedule of questions bearing on the type of structure, equipment, and ownership, data will be obtained for each of the approximately 35,000,000 dwellings throughout the country.

The data obtained will be useful to highway officials in planning street improvements, and to city planning officials in determining the need for extending transportation and communication systems, police and fire protection, schools, etc.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF JANUARY 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS *
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$6,239,830	$3,017,347	256.0	$4,624,781	$2,290,828	135.7	$1,828,891	$907,935	76.2	$3,902,598
Arizona	1,892,271	1,347,963	94.4	1,563,725	1,105,821	84.4	306,802	166,000	9.3	1,955,773
Arkansas	4,930,534	3,918,101	226.3	1,304,697	763,616	52.4	533,240	266,420	22.6	2,235,343
California	5,075,272	2,762,835	79.1	4,849,633	2,561,931	93.7	2,419,380	1,232,100	34.3	5,474,649
Colorado	3,815,457	2,119,520	89.3	1,561,772	875,606	39.9	152,739	50,000	4.7	3,636,447
Connecticut	802,205	395,926	10.0	1,520,108	758,035	15.0	1,119,652	546,326	9.4	1,460,136
Delaware	723,874	343,734	24.8	1,254,241	625,331	13.7				1,587,136
Florida	2,302,305	1,148,305	9.1	3,540,941	1,770,246	83.0	1,837,112	916,682	70.8	2,669,218
Georgia	3,897,580	1,900,267	218.7	5,071,268	2,535,634	255.0	3,156,303	1,578,152	106.3	7,196,533
Idaho	2,208,564	1,309,785	113.9	874,808	534,106	63.3	148,839	85,628	14.6	2,397,342
Illinois	8,737,645	4,353,142	195.7	4,938,202	2,467,112	85.9	3,469,550	1,734,775	76.1	5,758,549
Indiana	3,464,564	1,719,790	70.5	5,163,611	2,575,399	108.7	854,566	427,126	13.9	4,365,628
Iowa	3,929,715	1,826,856	190.7	3,599,669	1,597,988	107.9	970,207	459,300	43.4	3,194,554
Kansas	3,494,784	1,722,647	175.9	2,026,992	1,012,617	117.4	2,987,960	1,490,460	128.2	6,640,717
Kentucky	3,075,985	1,525,964	105.7	2,031,951	1,014,419	37.5	1,080,721	540,360	24.8	4,723,641
Louisiana	716,918	355,250	23.9	12,109,126	3,103,437	46.2	1,842,820	903,707	51.2	3,665,704
Maine	2,183,320	1,070,865	52.4	717,470	358,735	16.7	13,240	6,620	.6	1,321,220
Maryland	2,712,006	1,311,115	35.8	1,584,275	783,325	30.9	512,000	216,500	6.8	2,547,054
Massachusetts	3,134,614	1,564,618	25.1	731,370	364,841	5.7	1,524,431	759,083	12.1	3,918,372
Michigan	4,780,068	2,344,648	115.4	2,611,400	1,217,700	89.2	3,684,000	1,842,000	109.4	4,425,898
Minnesota	5,243,257	2,554,330	365.0	4,065,867	2,012,974	180.6	860,977	429,918	45.7	6,178,874
Mississippi	5,269,300	1,766,960	243.8	5,095,158	2,197,345	169.5	3,067,550	1,310,070	166.7	2,864,930
Missouri	3,239,051	1,615,728	139.3	3,417,706	1,680,606	105.5	3,895,737	1,540,129	112.3	6,939,732
Montana	3,281,001	1,855,632	189.8	2,118,450	1,201,259	108.8	1,261,452	715,494	107.5	5,226,496
Nebraska	4,114,859	2,045,294	327.7	3,736,128	1,812,062	364.9	2,561,899	1,192,568	305.7	4,312,607
Nevada	1,113,680	953,000	53.6	1,125,247	966,181	52.6				2,076,542
New Hampshire	821,381	404,233	27.3	726,443	356,095	15.5	59,293	29,647		1,498,999
New Jersey	829,670	405,711	7.0	4,599,878	2,298,989	36.4	1,692,720	846,360	13.8	2,299,140
New Mexico	1,986,434	1,216,018	134.7	1,047,592	660,345	78.3	785,762	490,394	33.7	2,318,901
New York	8,635,200	4,258,235	167.9	11,997,632	5,736,851	168.1	1,360,990	595,495	21.4	5,734,057
North Carolina	5,693,660	2,837,970	351.8	3,875,405	1,939,447	174.4	914,360	451,050	61.8	3,545,615
North Dakota	266,707	141,907	41.7	1,250,475	670,099	78.0	2,177,590	1,167,133	238.4	5,121,688
Ohio	5,943,447	2,912,343	84.3	7,002,692	3,477,972	57.5	9,508,921	4,694,325	91.6	6,002,222
Oklahoma	1,895,113	1,002,012	97.9	2,630,917	1,395,333	71.7	2,918,010	1,551,841	108.8	5,065,064
Oregon	2,456,341	1,470,097	105.8	3,308,178	1,786,472	96.2	623,985	364,180	39.8	2,365,373
Pennsylvania	9,665,392	4,782,885	113.7	4,812,541	2,264,071	43.6	4,183,322	2,045,285	43.8	6,910,317
Rhode Island	601,970	300,865	7.8	901,224	449,656	8.9	119,150	59,575	1.3	1,394,417
South Carolina	2,165,870	978,200	81.9	1,562,934	701,209	67.5	831,589	385,698	70.1	3,235,702
South Dakota	3,482,838	1,917,656	329.1	2,623,960	1,491,060	307.3	1,681,580	926,010	237.5	4,286,762
Tennessee	3,666,788	1,748,336	88.5	2,592,162	1,296,081	49.9	2,526,516	1,263,258	50.2	4,882,163
Texas	11,226,920	5,513,521	618.6	8,903,875	4,418,278	424.5	4,869,202	2,312,281	199.0	9,095,207
Utah	2,262,717	1,627,522	98.1	625,885	453,575	46.8	182,755	127,010	4.6	1,862,322
Vermont	736,369	352,575	18.4	711,393	355,507	22.8	129,374	64,665	4.6	857,308
Virginia	2,300,410	1,146,828	76.9	2,329,061	1,109,477	52.3	939,990	469,995	23.2	2,710,918
Washington	2,188,160	1,119,757	38.4	3,299,282	1,628,309	27.0	566,967	297,000	7.9	2,044,244
West Virginia	1,905,151	1,008,304	50.0	1,680,476	836,881	47.7	949,941	469,166	20.3	2,900,082
Wisconsin	5,226,831	2,570,449	187.9	4,978,600	2,443,880	153.0	99,163	46,805	2.6	4,419,522
Wyoming	1,494,690	918,924	141.3	1,263,852	796,581	134.8	694,202	426,821	61.1	1,489,568
District of Columbia	373,200	186,600	2.5	233,724	116,862	2.4	326,286	141,188	2.4	603,475
Hawaii	360,498	176,232	4.4	786,412	379,777	13.1	601,757	299,199	9.9	1,597,567
Puerto Rico	706,072	351,850	14.1	1,306,457	646,785	25.3	65,005	32,220	.2	904,565
TOTALS	167,270,488	86,198,652	6,321.9	156,289,646	75,896,746	4,667.1	78,898,498	38,873,954	2,900.6	183,820,891

* Includes apportionment for Fiscal Year 1941.

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF JANUARY 31, 1940

STATE	Completed During Current Fiscal Year: Estimated Total Cost	Completed: Federal Aid	Completed: Miles	Under Construction: Estimated Total Cost	Under Construction: Federal Aid	Under Construction: Miles	Approved for Construction: Estimated Total Cost	Approved: Federal Aid	Approved: Miles	Balance of Funds Available for New Projects*
Alabama	$497,335	$242,657	27.8	$742,832	$270,643	21.9	$803,200	$399,650	45.0	$625,001
Arizona	265,371	190,062	33.4	122,287	88,179	16.0	53,263	32,500	2.4	443,612
Arkansas	880,619	744,916	84.4	132,274	117,823	14.0	74,882	47,556	6.3	361,836
California	875,573	467,084	36.1	389,944	212,124	7.4	185,940	106,232	19.5	1,146,047
Colorado	872,052	471,535	31.3	225,564	103,748	5.9	30,502	4,429	1.3	278,665
Connecticut	163,280	67,660	2.9	110,274	54,954	.2	210,228	99,380	3.1	234,223
Delaware	80,623	39,067	17.5	69,537	34,768	7.8				309,475
Florida	548,017	269,750	17.4	355,529	177,307	14.4	113,046	56,523	1.3	563,503
Georgia	315,508	152,194	39.4	184,939	92,470	19.8	447,483	223,742	45.5	1,302,099
Idaho	457,974	252,546	45.1	128,663	78,481	12.8				289,799
Illinois	1,318,851	650,674	79.7	998,500	443,950	56.7	1,011,950	505,975	39.0	689,378
Indiana	892,023	442,650	77.7	299,900	148,746	18.3	229,992	114,950	18.1	946,710
Iowa	303,910	145,468	116.1	1,074,275	487,310	104.0	876,173	413,550	196.7	1,013,712
Kansas	77,730	38,849	43.9	580,564	290,282	9.4	206,496	103,248	16.5	1,537,887
Kentucky	1,096,826	325,230	71.4	552,881	182,184	37.1	561,042	181,418	54.6	432,790
Louisiana	803,271	378,742	67.2	180,205	90,103	15.3	269,567	138,197	22.8	441,101
Maine	431,716	215,038	25.4	68,900	33,244	3.5	8,300	4,150	.5	139,781
Maryland	229,641	111,157	17.7	124,696	50,348	4.8	157,000	58,355	8.8	438,989
Massachusetts	373,212	185,203	9.2	284,559	140,915	5.9	190,910	94,705	4.2	575,040
Michigan	1,281,718	630,991	112.7	818,410	409,205	42.9	710,620	341,379	32.8	808,854
Minnesota	806,551	397,568	104.6	383,659	191,829	35.0	173,348	42,795	12.5	1,430,850
Mississippi	366,500	183,250	40.7	961,822	474,626	52.1	50,530	25,260	4.9	730,903
Missouri	899,069	442,043	133.5	501,373	250,390	44.7	278,174	115,816	41.2	906,928
Montana	835,992	474,167	71.6	168,138	95,304	20.8	266,305	150,656	25.3	915,937
Nebraska	989,792	474,534	205.0	335,420	167,290	41.3	249,941	124,970	44.7	553,753
Nevada	161,442	133,635	25.0	200,928	172,790	25.5				226,136
New Hampshire	61,156	29,708	2.4	83,280	39,815	3.1	68,831	32,155	1.1	191,412
New Jersey	334,990	164,755	12.1	316,520	158,260	12.3	7,610	3,805	.8	667,730
New Mexico	485,092	298,043	42.1	266,824	86,594	13.8	101,564	63,386	13.1	331,083
New York	1,759,039	867,032	90.9	1,666,975	802,543	45.1	553,800	204,974	12.9	781,553
North Carolina	994,230	495,960	94.5	525,193	265,208	48.8	191,430	94,710	14.6	541,618
North Dakota	115,841	61,370	8.3	47,816	25,629		66,953	35,886	2.7	1,066,585
Ohio	697,800	347,590	41.8	665,090	339,320	25.6	2,167,860	1,038,287	72.6	1,107,335
Oklahoma	260,138	133,815	10.5	429,056	228,293	23.4	451,370	240,162	42.4	1,073,643
Oregon	613,273	347,752	67.6	233,493	103,177	27.6	110,460	66,300	18.6	467,534
Pennsylvania	2,087,028	1,028,639	117.1	1,217,672	602,731	41.4	298,100	149,050	10.9	736,020
Rhode Island	93,827	46,890	2.2	81,236	40,618	.2	150,483	75,215	3.4	94,217
South Carolina	562,159	228,890	56.9	141,620	54,567	12.1	264,400	95,550	22.7	408,575
South Dakota	16,243	8,940	4.1	11,056	6,088					1,284,534
Tennessee	811,584	352,928	31.7	146,416	73,208	.7	132,927	66,463	7.9	1,005,907
Texas	2,302,667	1,123,298	256.1	1,072,614	527,895	112.1	325,886	158,935	41.0	1,444,944
Utah	224,185	126,765	38.5	121,120	76,528	8.3				297,598
Vermont	143,680	69,603	5.6	164,292	50,336	6.3	105,500	47,800	3.3	81,849
Virginia	652,124	316,930	65.5	395,980	168,420	12.8	233,720	108,528	20.8	357,021
Washington	588,899	305,893	43.1	274,792	144,118	16.9	97,856	51,200	4.8	402,756
West Virginia	145,150	72,575	8.3	300,115	150,057	15.9	141,360	70,680	7.8	461,938
Wisconsin	897,970	446,696	34.5	321,054	160,190	4.7	379,319	179,040	4.4	825,785
Wyoming	470,702	286,620	26.0	329,482	195,846	23.5	53,943	34,052	15.5	189,322
District of Columbia	98,700	49,350	1.0	18,992	8,996	.2	28,500	14,250	.3	73,654
Hawaii	89,392	44,391	3.8	370,944	185,976	11.0				162,758
Puerto Rico				224,465	109,130	12.8	119,048	57,380	4.3	103,118
TOTALS	30,330,465	15,381,103	2,601.3	19,422,170	9,462,556	1,116.1	13,209,812	6,273,244	972.9	31,501,498

* Includes apportionment for Fiscal Year 1941.

U. S. GOVERNMENT PRINTING OFFICE: 1940

PUBLICATIONS of the PUBLIC ROADS ADMINISTRATION

(Formerly the *BUREAU OF PUBLIC ROADS*)

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Agency and as the Agency does not sell publications, please send no remittance to the Federal Works Agency.

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Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1938 10 cents.

Report of the Chief of the Bureau of Public Roads, 1939. 10 cents.

HOUSE DOCUMENT NO. 462

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The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. 60 cents.

Highways of History. 25 cents.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.

No. 265T. . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP. . Bibliography on Highway Safety.

House Document No. 272 . . . Toll Roads and Free Roads.

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SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

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Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration (formerly the *Bureau of Public Roads*), classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF JANUARY 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR		NUMBER			UNDER CONSTRUCTION		NUMBER			APPROVED FOR CONSTRUCTION		NUMBER			*BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 930,463	$ 917,059	9	2		$ 362,507	$ 361,884	8			$ 529,100	$ 529,100	3		5	$ 885,316
Arizona						518,061	515,813	4								400,746
Arkansas	184,062	184,017	3			669,853	663,348	3			108,542	108,542	3		9	1,032,917
California	1,156,964	1,156,419	8		1	1,122,970	960,073	6			269,114	269,114	2			1,849,410
Colorado	623,127	612,007	5		14	300,054	300,054	1		4	19,543	15,783			7	901,562
Connecticut						194,055	182,342		1	7	359,115	358,023	4			719,748
Delaware	7,839	7,839	2								96,433	96,433			16	568,269
Florida	428,094	428,094	2			196,653	192,154	2			23,917	23,917			6	1,438,713
Georgia	196,480	196,480	5			538,033	538,033	9	3	1	467,004	467,004	4	4	20	2,397,613
Idaho	204,578	172,785	3			110,538	110,176	1			254,160	223,685	3			486,062
Illinois	2,631,324	2,572,286	17	4	49	1,372,749	1,331,377	5		18	1,075,099	879,230	7		94	2,460,652
Indiana	756,020	756,020	3	1	57	556,928	556,928	3		34	295,372	295,372		2	59	1,452,673
Iowa	749,696	681,831	11		96	485,950	457,100	4		88	458,470	376,516	2		89	1,408,098
Kansas	933,613	933,613	11		5	560,421	560,421	7			450,865	450,865	6		12	1,296,522
Kentucky	596,662	592,361	8	4	5	687,789	687,789	6			321,466	321,466	4		18	855,091
Louisiana	279,995	279,978	3			674,302	620,806	6			627,885	570,153	11			789,393
Maine	331,672	329,136	3	2		206,701	206,701	2	1		924	924			1	437,590
Maryland	94,896	85,789	1		3	184,009	184,009	1		2	523,600	435,807	2	1	10	821,925
Massachusetts	400,519	399,288	2	2		122,047	122,014	2			14,320	14,320	1			2,321,561
Michigan	825,839	823,499	6	2	26	800,810	800,810	4		11	609,430	609,430	3	3	12	1,762,190
Minnesota	506,912	480,951	4	4	12	1,441,986	1,440,892	10	2	5	122,930	122,930		1	2	1,668,077
Mississippi	246,300	246,300	6			491,973	491,973	4			141,700	141,700	2	1		1,123,639
Missouri	402,019	400,714	2	1		1,337,397	1,337,397	9	1		194,902	185,086		4		2,277,234
Montana	850,426	850,426	9			213,154	98,073	3			81,237	79,925	2			624,616
Nebraska	660,200	659,600	19		19	716,695	716,695	6	2	11	158,482	158,482	1		22	930,022
Nevada	196,253	196,253		2	11	25,436	24,534		1	2						248,145
New Hampshire	100,927	100,459	7		1	104,790	104,754	3			91,061	91,061	1			371,461
New Jersey	141,570	141,570	1		1	725,086	725,086	2	3		140,550	140,550	1			1,754,825
New Mexico	75,359	75,149	2		1	187,196	187,196	2			125,614	125,614	1			631,289
New York	1,832,964	1,826,676	5	7		2,148,218	2,051,118	8	9		1,185,033	1,145,053	4	5		3,982,255
North Carolina	1,196,122	1,161,022	6	5	37	474,724	474,724	7	1	11	368,715	368,715	2	1	18	1,293,445
North Dakota	527,681	479,243	7	1		395,927	395,927	5			25,170	25,170			8	904,365
Ohio	541,190	526,190	8	1	2	1,502,813	1,431,861	8	2		627,010	627,010	4		3	3,727,509
Oklahoma	275,789	275,252	4		32	304,525	303,625	3	1	11	550,509	550,509	12		37	2,231,505
Oregon	176,240	174,742	2			134,132	132,837	1								651,272
Pennsylvania	660,961	656,054	1	2		1,876,028	1,669,036	9	2	1	110,177	110,177	2		1	5,918,697
Rhode Island	442,828	442,828	1	3		7,406	7,406									287,266
South Carolina	483,689	450,307	7	4	7	359,387	348,093	3	3		232,175	232,175	1	3	26	1,088,758
South Dakota	331,694	331,694	3	2	17	75,435	75,435	1	1		47,100	47,100	1			1,399,463
Tennessee	283,757	283,757		2	5	589,158	589,158	3	2							1,899,097
Texas	1,666,101	1,634,847	15	3	6	2,312,796	2,221,492	.20		5	300,316	284,463	2		15	2,884,462
Utah	220,570	220,416	3		77	152,298	152,298			50	48,460	48,460			17	369,488
Vermont	33,416	28,618			7	206,402	206,402		2							262,541
Virginia	691,979	599,079	7	3	18	206,811	206,811	3			27,561	27,561	1		2	1,394,206
Washington	293,029	291,618	3		13	226,450	226,450	1	3		89,111	89,111	3		1	759,141
West Virginia	175,781	175,781	3			214,770	199,010	6		1	9,400	9,400			3	1,360,732
Wisconsin	883,349	879,905	9		2	858,603	812,506	7	2	9	444,636	422,403	2	1	6	1,378,336
Wyoming	136,165	136,009	1	1	7						85,910	85,910	1			631,210
District of Columbia						366,812	333,268	1								193,303
Hawaii	162,720	160,090	2	1		30,682	30,682	2			236,535	236,525	2		1	261,573
Puerto Rico	49,040	48,840	1			345,312	343,310	8								650,883
TOTALS	25,576,874	25,062,891	240	59	531	27,696,832	26,689,881	209	42	271	11,948,653	11,400,774	100	26	520	67,444,866

* Includes apportionment for Fiscal Year 1941.

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BARNETT, JOSEPH
BECKHAM, W. K.
BENHAM, SANFORD W.
CARPENTER, C. A.
CRITZ, PAUL F.
GOODE, J. F.
HOGENTOGLER, C. A.
JACKSON, F. H.
KELLERMANN, W. F.
KELLEY, E. F.
MARR, ROBERT A., Jr.
NORMANN, O. K.
PADDOCK, ROBERT H.
PALMER, L. A.
PARSONS, A. W.
PAULS, J. T.
RODGERS, ROE P.
STEUBER, WILLIAM F.
SUTHERLAND, EARL C.
WILLIS, E. A.

ABBREVIATIONS

a.—article(s), report(s)
fn.—footnote
r.—reference(s), referred to
r. to p.—reference(s) to publication(s)

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