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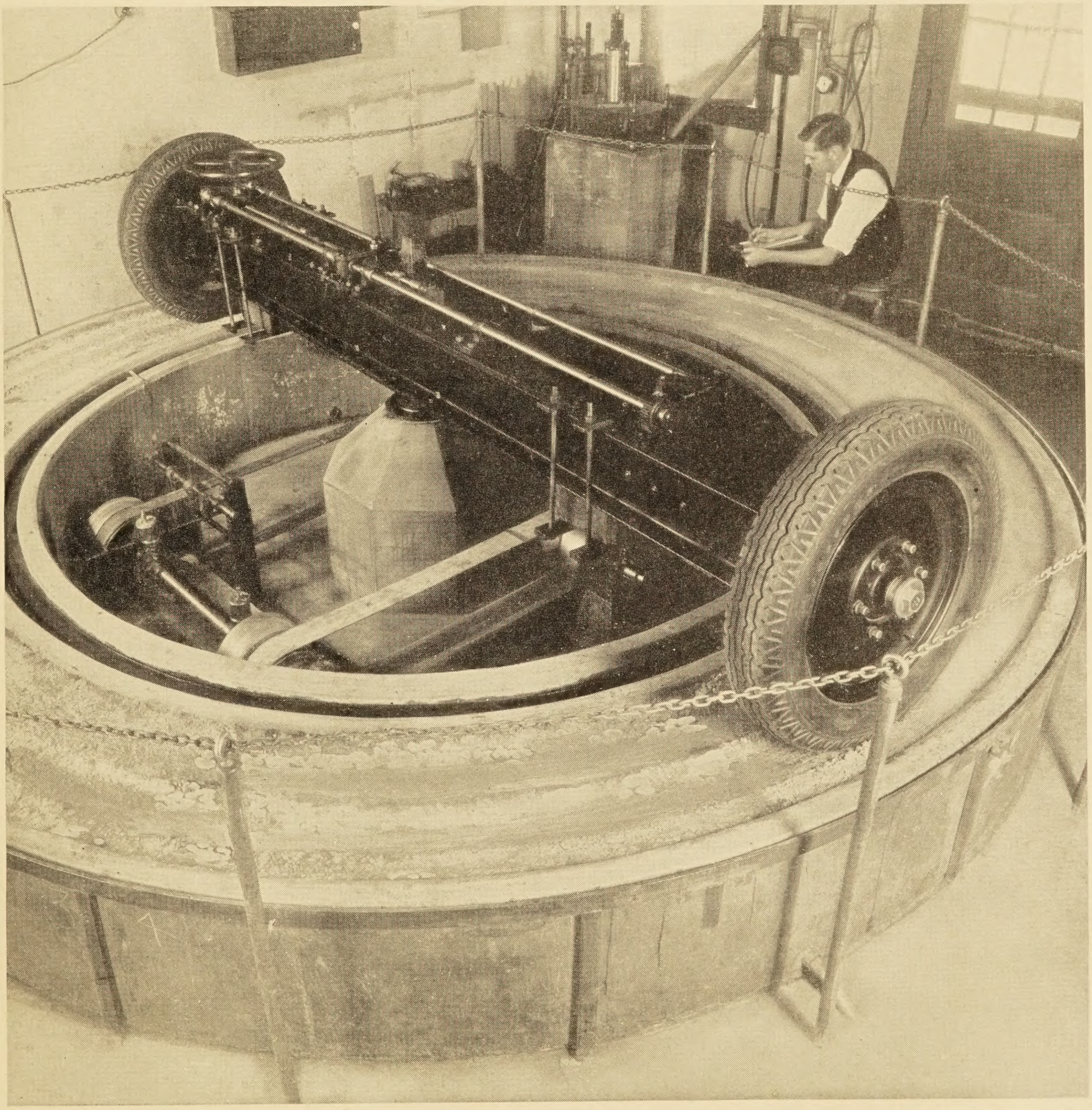

CIRCULAR TRACK USED IN TESTING ROAD MATERIALS

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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.

In This Issue

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A STUDY OF SAND-CLAY 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

LONG EXPERIENCE with low-cost roads in the southern States has demonstrated that certain natural mixtures of sand and clay, with varying amounts of coarser materials, serve excellently as road surfaces for relatively light traffic. As traffic on such roads increases, the necessity for applying some type of treatment to provide abrasion resistance becomes increasingly apparent. Experience in the use of surface treatments has indicated that roads highly stable before treatment may become unstable bases when covered with a waterproof surfacing. This was clearly shown by a survey of surface treatments on sand-clay, topsoil, and gravel bases in North Carolina conducted by the Bureau of Public Roads in cooperation with representatives of the tar industry in 1932.[1]

In order to study further the properties of materials suitable for low-cost bases for bituminous wearing courses, the indoor circular track shown on the cover page was utilized to investigate the behavior of 11 different sand-clay base-course materials under controlled traffic and moisture conditions. The investigations were conducted in the Bureau's laboratories at the Arlington Experiment Farm, Arlington, Va.

This circular track, which has been fully described in PUBLIC ROADS, vol. 14, no. 11, page 219, and vol. 17, no. 4, page 69, consists of a circular concrete trough 12 inches deep, 18 inches wide, and 12 feet in diameter at the center line. Facilities are provided for introducing water into the trough through the bottom of the inner wall and for maintaining any desired water elevation in the trough, as shown in figure 1.

Traffic is applied by two automobile wheels, equipped with low-pressure tires and mounted on the ends of a centrally pivoted steel beam. The tires used in these tests were of the balloon type, size 6.00–20, inflated to 35 pounds per square inch. The normal load on each wheel consisted of one-half the weight of the beam and wheel assembly, or 800 pounds. This was increased to 1,000 pounds during the later stages of the tests by hanging lead weights on the beam.

[1] Tar Surface Treatment of Low Cost Roads. PUBLIC ROADS, vol. 14, no. 1, March 1933.

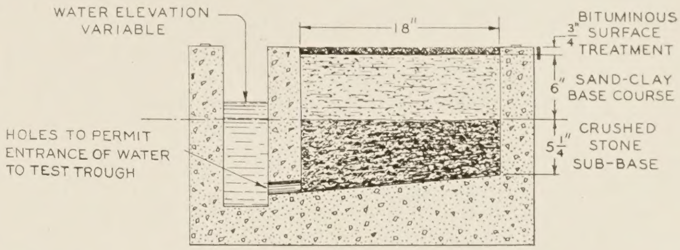


FIGURE 1.—CROSS SECTION OF TRACK SHOWING ARRANGEMENT OF MATERIALS FOR TESTING SAND-CLAY BASE COURSES.

Distributed traffic, which was used for compacting and in the early stages of the tests, was obtained by gradually shifting the axis of rotation of the beam, causing the wheels to pursue an alternately expanding and contracting spiral course covering the entire track area. Concentrated traffic, which was used in the later stages of the tests, 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 test sections.

The 11 sand-clay materials used in the tests were made up of Potomac River sand, pulverized silica, and a red clay soil of local origin having a liquid limit of 90 and a plasticity index of 78. These materials were combined in various proportions to produce mixtures having the gradings and plasticity indexes shown in table 1. Five of these mixtures, referred to in table 1 as series 1, were designed to have essentially the same grading and to have plasticity indexes varying from 0 to 18. The grading selected was similar to that of materials used extensively in actual sand-clay base construction. The other six mixtures, referred to in table 1 as series 2, were designed to have a plasticity index of approximately 5 and various gradings.

The proportions of sand to clay required to produce the desired plasticity indexes were determined by soil tests on preliminary mixtures. For this purpose, use was made of the following procedure which has been

TABLE 1.—*Gradings and soil constants of sand-clay 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 No. 10 sieve	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Passing No. 20 sieve	91.4	90.5	91.7	90.0	90.6	90.2	87.7	89.1	93.8	82.9	67.3
Passing No. 40 sieve	70.6	65.9	67.7	67.1	67.3	69.2	64.3	64.5	80.2	50.6	47.5
Passing No. 100 sieve	35.8	31.0	31.6	33.4	33.7	51.2	37.7	26.0	53.0	25.0	33.8
Passing No. 200 sieve	25.1	25.5	26.9	28.6	29.1	44.7	31.0	20.1	40.8	20.2	29.3
Passing 0.005 mm	10.5	14.5	18.0	20.5	21.5	16.0	13.0	13.0	16.0	12.0	12.0
Passing 0.001 mm	6.0	9.5	12.0	14.0	15.0	10.0	8.0	9.0	10.0	8.0	8.0
Dust ratio [1]	36	39	40	43	43	65	48	31	51	40	62
Tests on material passing No. 40 sieve:											
Liquid limit	18	20	25	28	33	21	19	21	21	20	20
Plasticity index	0	5	9	13	18	4	4	5	6	6	4

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

found advantageous in proportioning available materials for use in actual construction.

PROPORTIONS OF MATERIALS ESTABLISHED BY PRELIMINARY TESTS

Plasticity tests were made on a number of mixtures which contained different amounts of the clay soil. The liquid limits and plasticity indexes were plotted against the clay soil contents of the mixtures, as shown in figure 2. The proportions of clay soil necessary to give the mixtures the desired plasticity indexes were then determined from the lower curve. The upper curve showed the corresponding liquid limit. In series 1, where a constant grading was desired, various proportions of fine clay soil and pulverized silica, both passing the No. 200 sieve, were used to vary the plasticity index without materially altering the total amount passing the No. 200 sieve.

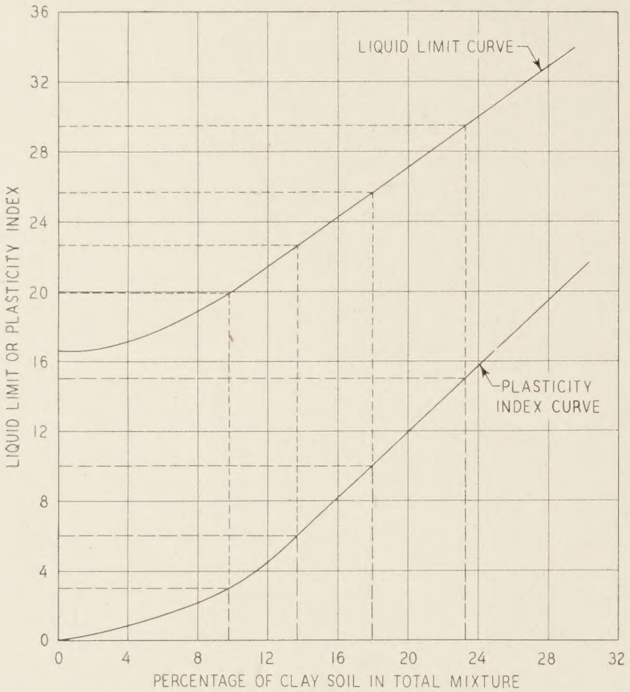


Figure 2.—Effect of Clay Soil Content on the Liquid Limit and Plasticity Index of Sand-Clay Mixtures.

The gradings and soil constants given in table 1 are those of the final mixtures as prepared for the circular track tests.

All five of the mixtures in series 1 were tested simultaneously in the track, each mixture comprising a test section 18 inches wide and approximately 7.5 feet long. In the second track containing the six mixtures of series 2, the length of each test section was approximately 6.3 feet, or one-sixth of the track circumference. The depth of each test section was approximately 6 inches after compaction.

In constructing track 1, the materials for the five test sections were combined with water to attain approximately 1 percent more than their optimum moisture contents as determined by the Proctor compaction test,[2] while the six materials in track 2 had slightly less than their Proctor optimum moisture contents. This difference was introduced for two reasons: First, because the loss of water during mixing under laboratory conditions proved to be less than was anticipated; and second, because the track tests of series 1 and concurrent molding tests utilizing direct compression of 3,000 pounds per square inch indicated that higher densities could be produced than had been obtained by the Proctor method. It was also found that maximum density was obtained both by direct compression and in the track at lower moisture contents than by the Proctor method.

[2] Fundamental Principles of Soil Compaction, by R. R. Proctor, Engineering News-Record, vol. 111, nos. 9, 10, 12, and 13.

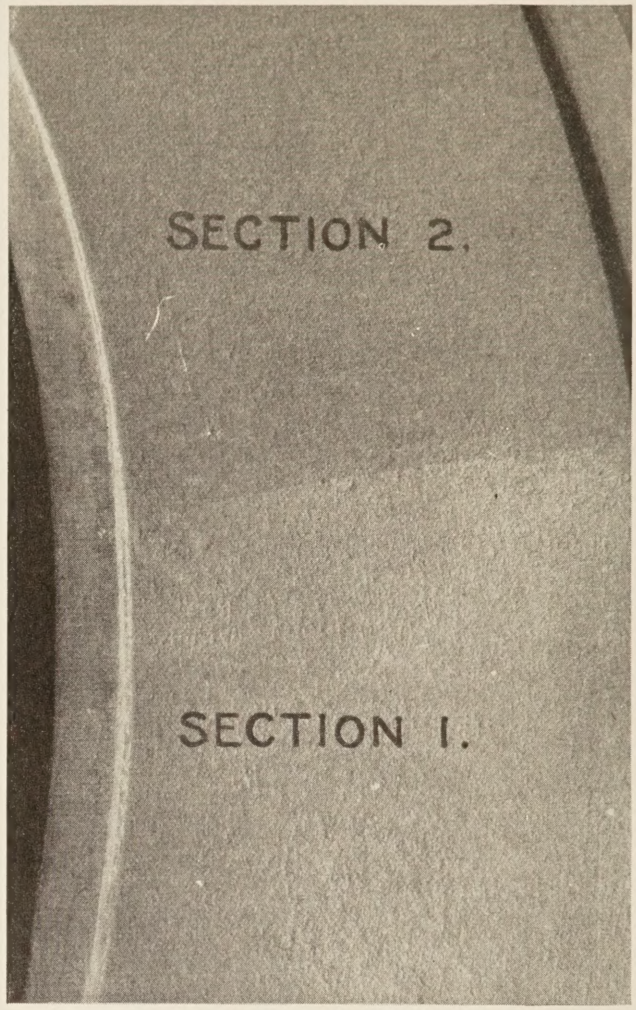


Figure 3.—Typical Appearance of Sand-Clay Base Sections in Circular Track After Compacting and Trimming Preparatory to Applying Surface Treatment.

In actual road construction, the slight excess of moisture resulting from the use of the Proctor optimum moisture content is advantageous in insuring that sufficient moisture for compaction will remain after the necessarily longer period of mixing.

The optimum moisture contents as determined by the Proctor tests and the moisture contents of the track sections after the first day's compacting traffic are given in table 2.

Aside from the differences noted in moisture contents, the procedure for preparing the materials for the track tests, constructing the test sections, and surface treating them, was the same for both tracks and was as follows:

1. The moistened sand-clay mixtures were first thoroughly mixed to distribute the water uniformly and

TABLE 2.—*Optimum moisture contents and actual moisture contents of track sections after 1 day of compaction by traffic*

Test section	Proctor optimum moisture content	Moisture in sections after 1 day of compaction
Series 1	*Percent*	*Percent*
Section 1	8.6	8.3
2	9.3	10.0
3	9.5	10.7
4	10.3	11.0
5	10.6	11.7
Series 2		
Section 1	10.0	8.3
2	9.8	7.3
3	10.2	8.7
4	9.9	8.3
5	10.0	8.6
6	9.4	6.9

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 further subsidence was noted, 8,000 wheel-trips being applied to the sections in series 1 and 32,000 wheel-trips to the sections in series 2.

3. The sections were trimmed smooth as shown in figure 3 and allowed to dry for several days.

4. A prime coat consisting of 0.3 gallon per square yard of light tar was applied and allowed to cure.

5. A surface treatment consisting of an application of 0.4 gallon of hot application bituminous material and a cover of 50 pounds per square yard of ¾-inch maximum size stone was constructed.

6. The treatment was consolidated by additional distributed traffic until the surface was well sealed and showed no movement.

Series 1.—The schedule of traffic application and changes in water elevation, with notations on the behavior of the five test sections of series 1, are given in table 3. Table 4 shows the densities and the moisture and void contents of the track sections at successive stages during compaction and at the conclusion of the traffic test.

The data in table 4 were obtained from tests on core samples taken with a 2-inch tubular core cutter. All holes from which samples were taken were filled immediately with excess material from the original mixtures, thoroughly tamped into place by hand. The factors of density and moisture content were not investigated during the test period from 36,000 wheel-trips to 266,000 wheel-trips because it was believed that sampling during this time would cause premature failures in the vicinity of the places where samples were taken. The data in table 4 represent the average conditions in the top half of the base-course sections. In series 1, the bottom half of each section was less dense than the top half and contained more water late in the compaction period and at the conclusion of the test.

ADMITTING WATER TO BASE CAUSED EARLY FAILURE OF SOME SECTIONS

Figure 4 shows the rate of consolidation and the rate at which vertical displacement occurred in the track sur-

TABLE 3.—*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, P. I.=0	Section 2, P. I.=5	Section 3, P. I.=9	Section 4, P. I.=13	Section 5, P. I.=18	Remarks
	Wheel-trips	*Inches*						
Compacting base	0–8,000	[1]	Good	Unstable	Slightly unstable	Unstable	Good	
Compacting base and surface treatment.	[2] 8,000–36,000	[1]	do	Slightly unstable	do	Slightly unstable	do	
Testing with distributed traffic.[3]	36,000–120,000	0	do	do	Good	do	Failed	Air trapped above water under impervious surface seal.
Testing with concentrated traffic.	120,000–160,000	0	do	Good	do	do		Do.
Do	160,000–176,000	0	do	do	do	Unstable, failed		Trapped air released allowing water to rise.
Do	176,000–248,000	+2	do	do	Some movement			
Do	248,000–266,000	+3	do	do	Some cracking			Tests discontinued.

[1] No water in sub-base.
[2] Water admitted to sub-base at 36,000 wheel-trips.
[3] Wheel loads increased from 800 pounds to 1,000 pounds at 107,000 wheel-trips.

TABLE 4.—*Water content and density of circular track sections from tests on core samples [1] in series 1*

Number of wheel-trips when core samples taken	Condition of sub-base	Section 1: Water content based on dry weight	Section 1: Composition by volume: Water	Aggregate	Air voids	Section 2: Water content based on dry weight	Section 2: Composition by volume: Water	Aggregate	Air voids	Section 3: Water content based on dry weight	Section 3: Composition by volume: Water	Aggregate	Air voids	Section 4: Water content based on dry weight	Section 4: Composition by volume: Water	Aggregate	Air voids	Section 5: Water content based on dry weight	Section 5: Composition by volume: Water	Aggregate	Air voids
		Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
2,200	Dry	8.3	15.8	70.8	13.4	10.0	19.8	73.7	6.5	10.7	21.4	74.6	4.0	11.0	21.7	73.5	4.8	11.7	23.1	73.6	3.3
8,000	do	5.8	11.0	70.8	18.2	8.3	17.4	78.0	4.6	9.6	19.8	76.8	3.4	9.5	19.5	76.5	4.0	10.5	21.2	75.3	3.5
20,000	do	4.2	8.0	70.9	21.1	6.9	15.0	81.1	3.9	7.8	16.5	79.1	4.4	8.6	18.2	79.0	2.8	9.3	19.2	77.1	3.7
36,000	Water admitted	------	------	------	------	6.7	14.7	81.6	3.7	7.7	16.5	79.7	3.8	9.0	19.2	79.6	1.2	------	------	------	------
[2] 266,000	Wet	7.6	15.3	74.8	9.9	6.7	14.5	80.8	4.7	8.0	17.1	79.5	3.4	8.9	18.6	78.1	3.3	10.3	21.0	76.2	2.8

[1] All core samples were taken from the top half of the base course.
[2] All traffic up to 120,000 wheel trips distributed; traffic from 120,000 to 266,000 wheel trips concentrated.

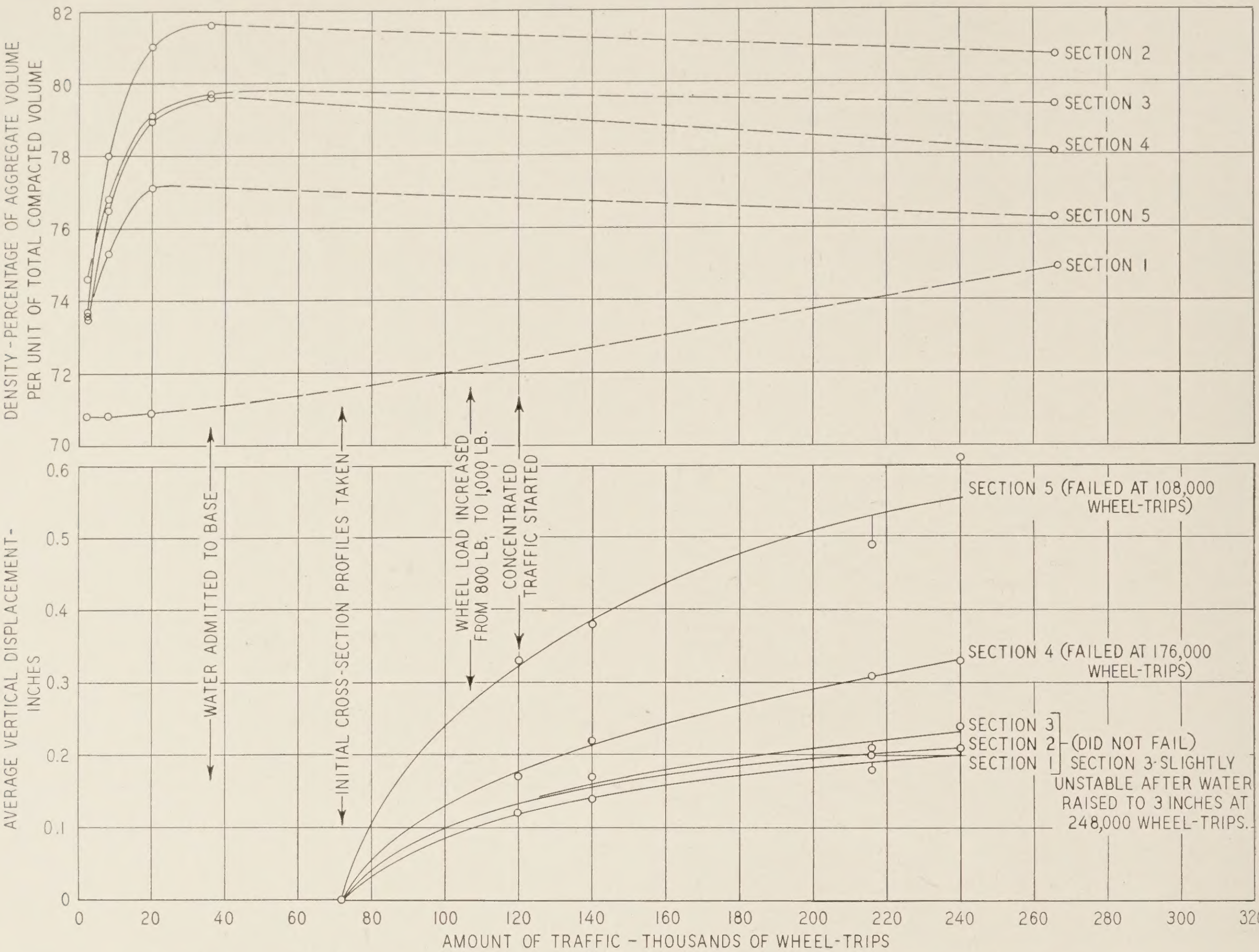


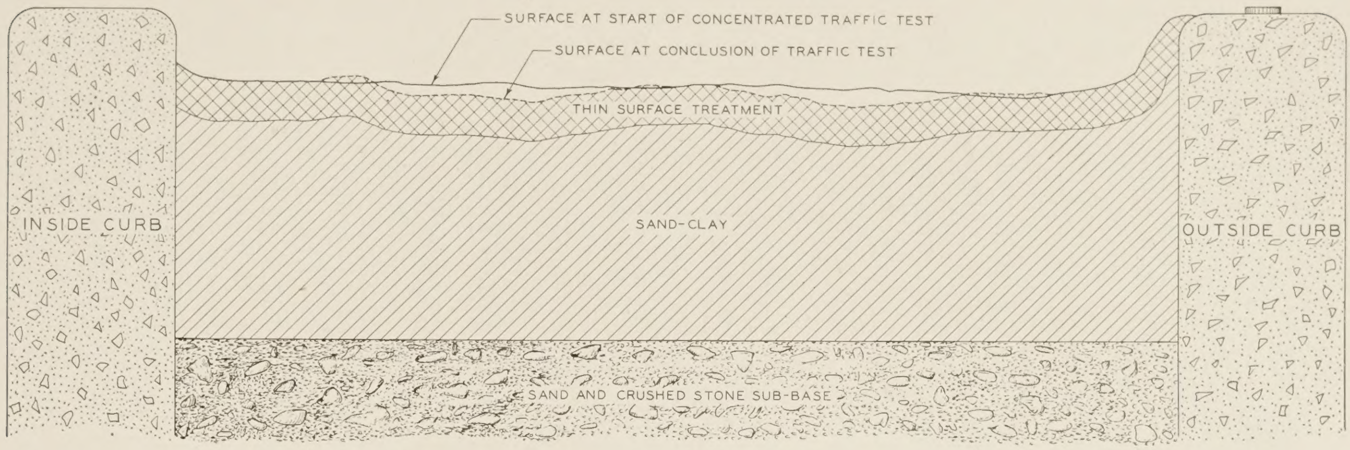
FIGURE 4.—RATES OF CONSOLIDATION AND SURFACE DISPLACEMENT UNDER TRAFFIC, SERIES 1.

face under traffic. The upper set of curves, showing rate of consolidation, was plotted from the data in table 4. The lower set, showing rate of surface displacement, was plotted from data obtained by means of a recording transverse profilometer with which changes in shape and elevation of the surface caused by displacement of material within the base course were measured. Typical examples of the records obtained with the transverse profilometer are shown in figure 5. Two stations were established on each test section for taking periodic readings with the profilometer and each point on the displacement curves in figure 4 represents the average of the displacement obtained at the two stations.

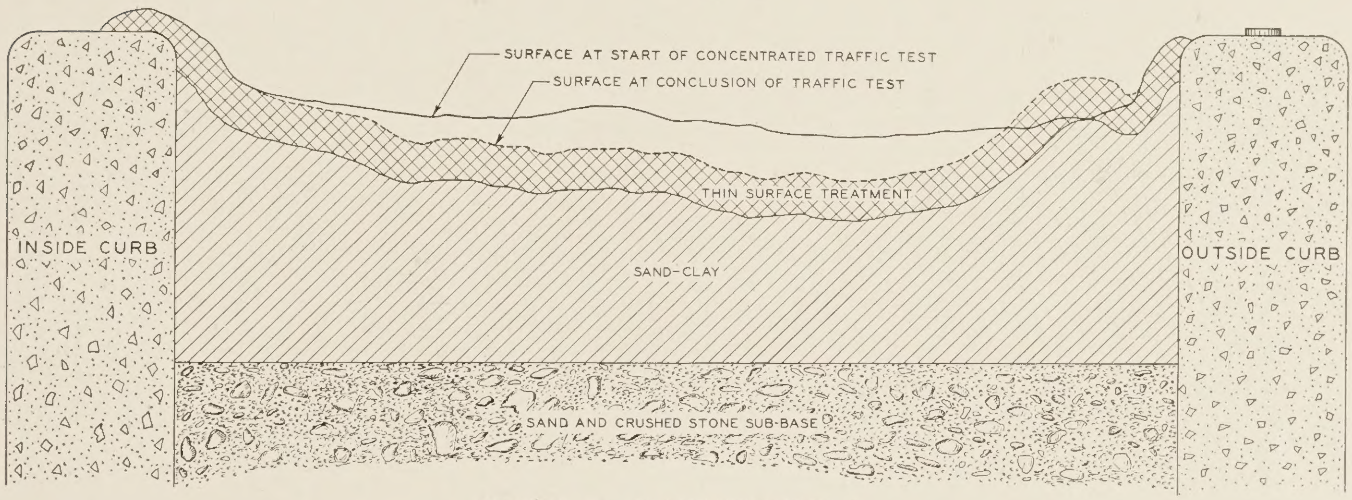
The comparative quality of the various materials used in both series 1 and series 2 was judged primarily by the amount of vertical displacement produced in the test sections by the test traffic and the rate at which this displacement occurred. It was observed that, regardless of the time required to produce it, an average vertical displacement of about 0.25 inch was sufficient to cause marked damage to the bituminous wearing course. Such damage consisted of cracking, curling, and separation of the surface treatment from the base course. It was also observed that the rate of displacement was relatively high early in the test period (see figure 4) and became progressively less as traffic was continued. That there was a wide difference in the quality of the materials in both tracks was shown by the fact that the satisfactory materials withstood from 90,000 to 140,000 more wheel-trips than the unsatisfactory materials without showing appreciable indications of failure.

In series 1, section 4 failed at approximately 176,000 wheel-trips, or 140,000 wheel-trips after water was admitted to the sub-base. Previous to that time or at approximately 108,000 wheel-trips, section 5 had failed. Sections 1 and 2 continued to withstand traffic without any indication of failure to the end of the test at 266,000 wheel-trips. Section 3 remained in good condition, except for minor cracking, up to 266,000 wheel-trips but could be seen to move slightly under each wheel passage after the water level was raised to 3 inches above the bottom of the base course at 248,000 wheel-trips. The material in section 3 was therefore classed as doubtful or border-line.

The behavior of the test sections in series 1 is well illustrated by the appearance of sections 1, 4, and 5 during the traffic test, as shown in figure 6. Sections 4 and 5 had definitely failed as indicated by excessive movement, rutting and corrugation, and the breaking and scaling of the surface treatment. Although section 1 looked slightly better at this stage than sections 2 and 3, the difference was so slight that the left illustration in figure 6 may be considered typical of the appearance of sections 1, 2, and 3 at this time and also at the conclusion of the test at 266,000 wheel-trips.



SECTION 1 – PLASTICITY INDEX 0



SECTION 5 – PLASTICITY INDEX 18

FIGURE 5.—TYPICAL CROSS SECTIONS OF TEST TRACK SHOWING COMPARATIVE EFFECTS OF CONCENTRATED TRAFFIC.

Series 2.—The schedule of traffic application and changes in water elevation with notations on behavior of the test sections in series 2 are given in table 5. Table 6 shows the densities and the moisture and void contents in the top half of the track sections during the compaction period and at the conclusion of the test. Data for sections 1, 2, and 4 are given up to the time of failure. These sections failed completely almost immediately after the water was admitted to the sub-base and had to be replaced with more stable material before the test on the three remaining sections could be continued.

Figure 7 shows the rates of consolidation and vertical displacement for the sections in series 2. The upper group of curves, showing rate of consolidation, was plotted from table 6 and the lower group showing rate of displacement was plotted from the cross section profile data obtained in the same way as that described in connection with 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	0–32, 000	(1)	Good	Good	Unstable	Good	Unstable	Slightly unstable.
Compacting base and surface treatment.	[2] 32, 000–60, 000	(1)	do	do	Good	do	Good	Good.
Testing with distributed traffic [3]	60, 000–120, 000	+½	Failed at 61,000	Failed at 63,000	do	Failed at 61,000	do	Do.
Testing with concentrated traffic	120, 000–200, 000	+½			do		do	Slightly unstable.

[1] No water in sub-base.
[2] Water admitted to sub-base at 60,000 wheel-trips.
[3] Wheel loads increased from 800 pounds to 1,000 pounds at 82,000 wheel-trips.

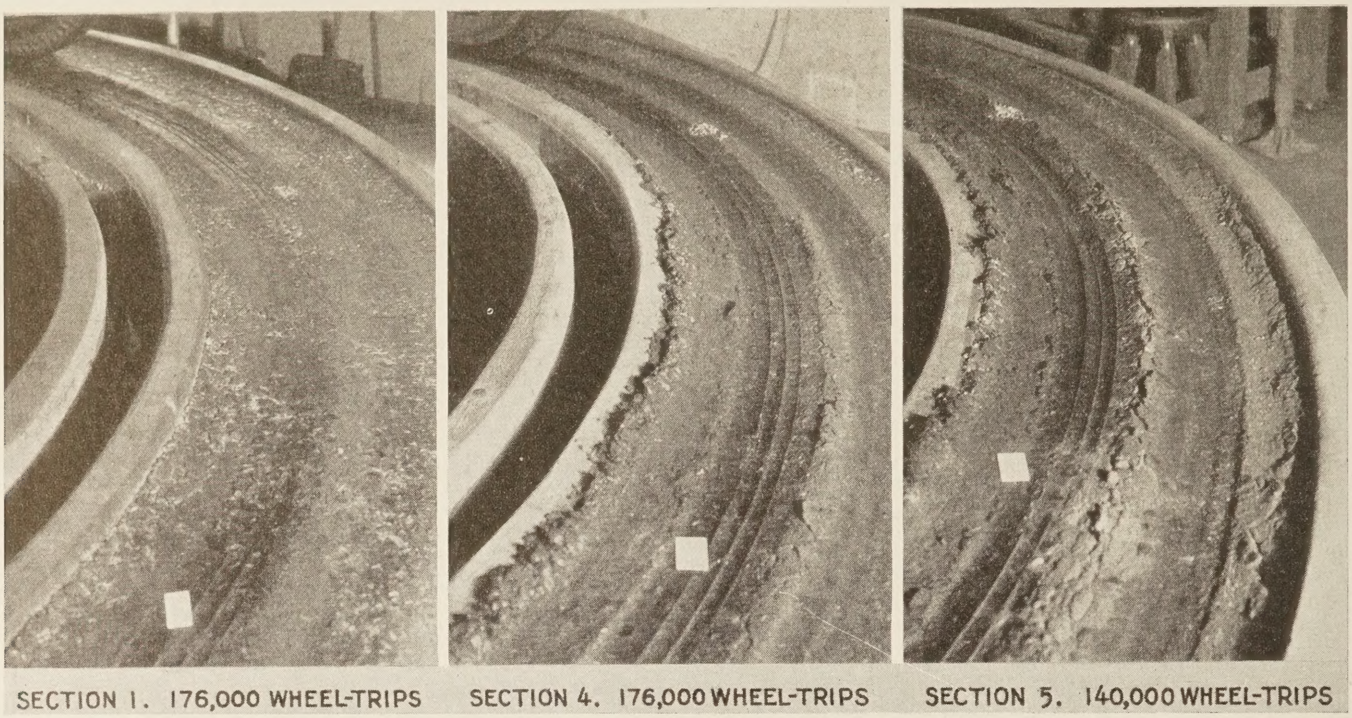


Figure 6.—Appearances of Three Sections of Track 1 After Various Amounts of Test Traffic. Sections 4 and 5 Had Failed.

Table 6.—*Water content and density of circular track sections from tests on core samples [1] in series 2*

Number of wheel-trips when core samples taken	Condition of sub-base	Section 1				Section 2				Section 3				Section 4				Section 5				Section 6			
		Water content based on dry weight	Composition by volume			Water content based on dry weight	Composition by volume			Water content based on dry weight	Oomposition by volume			Water content based on dry weight	Composition by volume			Water content based on dry weight	Composition by volume			Water content based on dry weight	Composition by volume		
			Water	Aggregate	Air voids		Water	Aggregate	Air voids		Water	Aggregate	Air voids		Water	Aggregate	Air voids		Water	Aggregate	Air voids		Water	Aggregate	Air voids
		Per-cent	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*	*Per-cent*
2,700	Dry	8.3	15.5	70.1	14.4	7.3	14.3	73.3	12.4	8.7	17.4	74.7	7.9	8.3	16.0	72.0	12.0	8.6	17.6	76.4	6.0	6.9	14.0	76.2	9.8
24,000	do	7.5	14.3	71.3	14.4	6.6	12.9	73.3	13.8	7.9	16.9	79.7	3.4	7.5	14.5	72.6	12.9	7.5	15.7	78.4	5.9	5.8	12.4	80.0	7.6
60,000	Water admitted																								
61,000	Wet	10.5	20.9	74.7	4.4									10.4	20.8	74.9	4.3								
63,000	do					9.6	19.5	76.1	4.4																
[2] 200,000	do									7.4	15.8	79.6	4.6					5.9	12.8	81.2	6.0	5.5	11.8	80.6	7.6

[1] Core samples taken from top half of the base course.
[2] All traffic up to 120,000 wheel-trips distributed; traffic from 120,000 to 200,000 wheel-trips concentrated.

As in series 1, the displacement curves clearly distinguished the satisfactory materials from those that failed. Sections 1 and 4 withstood only 1,000 wheel-trips of distributed traffic after the water was admitted to the sub-base, and section 2 withstood only 3,000 wheel-trips. The average vertical displacements measured after failure were 0.87 inch for section 1, 0.67 inch for section 4, and 0.64 inch for section 2. Sections 3 and 5 remained in excellent condition throughout the test and showed final displacements of only 0.16 inch and 0.06 inch, respectively. Section 6 was generally stable throughout the test and showed a final displacement of only 0.17 inch, but at about 120,000 wheel-trips it developed sufficient movement under traffic to cause some cracking and edge raveling. Later in the test this condition disappeared but the temporary weakness shown was held to justify the classification of the material in this section as borderline.

STABILITY DURING COMPACTION NOT NECESSARILY AN INDICATION OF QUALITY

In considering the behavior of the 11 materials used in this investigation, particularly during the compaction period, section 1 of series 1 must be placed in a class entirely separate from the rest. It was the only non-plastic material in the group. Its compaction and stability relationships were decidedly different from those of the 10 plastic materials.

From figures 4 and 7 it is evident that section 1 of series 1 was the least compactible of any of the 11 materials. Its gain in density during the compaction period, after the initial determination at 2,200 wheel-trips, was so slight as to be of no significance whatever; and at the end of the test its density, while appreciably higher than at 2,200 wheel-trips, was less than the final density of any other section except section 1 of series 2. At no time, however, either during the compaction or test periods, did it show any evidence of instability.

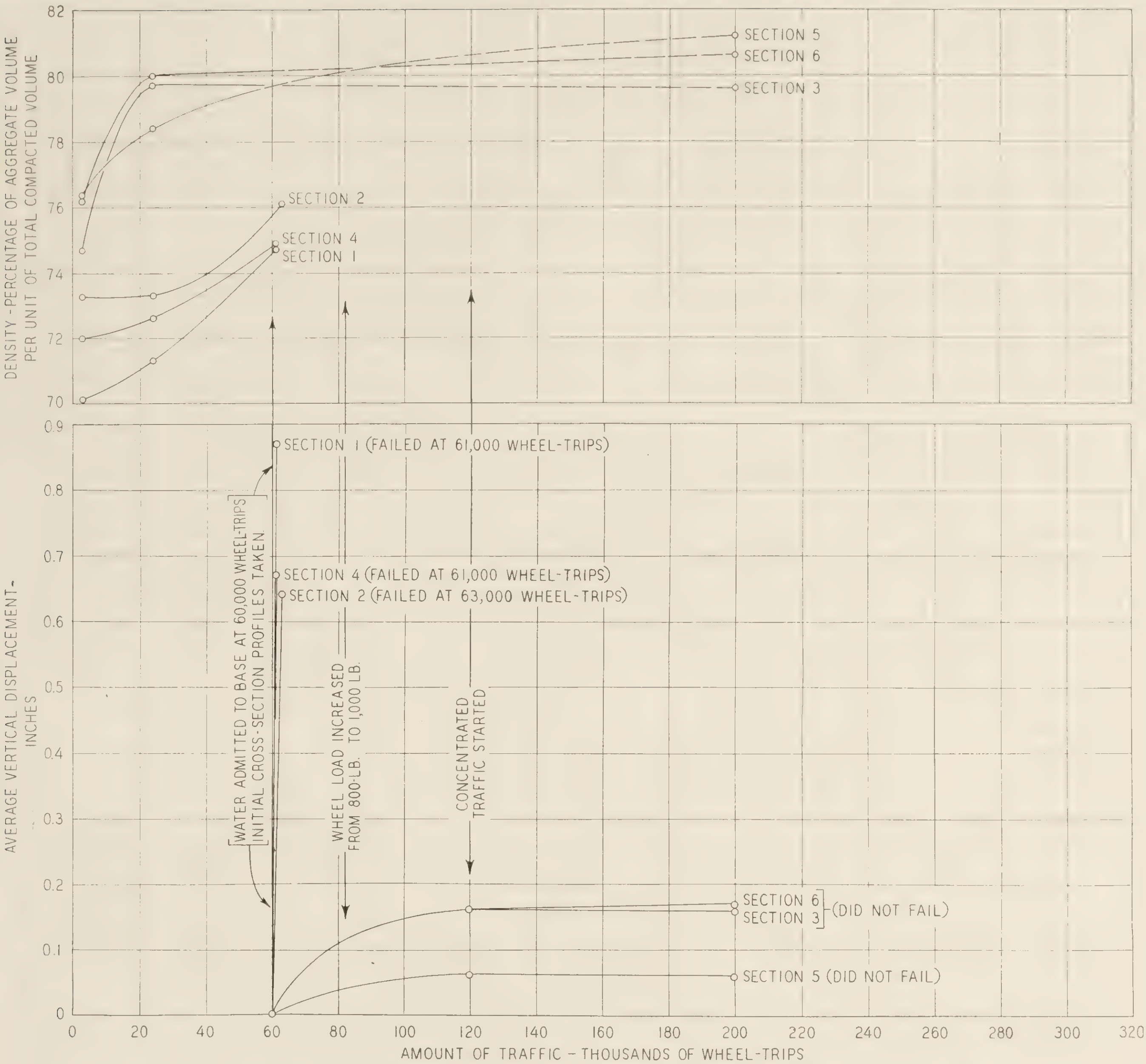


Figure 7.—Rates of Consolidation and Surface Displacement Under Traffic, Series 2.

Of the 10 plastic materials, the ones that gave satisfactory service in the traffic test were, without exception, definitely unstable during the compaction period. These were sections 2 and 3 of series 1, and sections 3, 5, and 6 of series 2. All these materials compacted quite rapidly and, with the exception of section 5 of series 2, reached either their highest or essentially their highest density under less than 40,000 wheel-trips of distributed traffic. They were so unstable during compaction that it was difficult to prevent them from working out over the curbs under traffic and it was only after a considerable drying period with continued traffic and periodic reshaping that they set up and stopped shoving and corrugating.

Section 4 of series 1 was unique in that it was unstable throughout both the compaction and test periods. The other four plastic materials, section 5 of series 1, and sections 1, 2, and 4 of series 2, all of which failed because of instability during the traffic test, set up early in the compaction period and required little or no reshaping Their tendency to set up under early traffic prevented effective compaction and their densities as shown in figures 4 and 7 were lower than those of any of the satisfactory materials, except the non-plastic section 1 of series 1 previously discussed.

The lack of early stability of the materials that later proved satisfactory during the traffic test, even after prolonged exposure to capillary moisture, is emphasized for two reasons: First, because such behavior in field construction might be misconstrued and lead to the use of too much clay in order to facilitate compaction; and second, because it so clearly illustrates the importance of thorough compaction in conjunction with drying.

Section 5 of series 2, which is shown in figure 8, during early compaction on the left, and after 200,000 wheel-trips (conclusion of the traffic test) on the right, is typical of the materials that showed early lack of

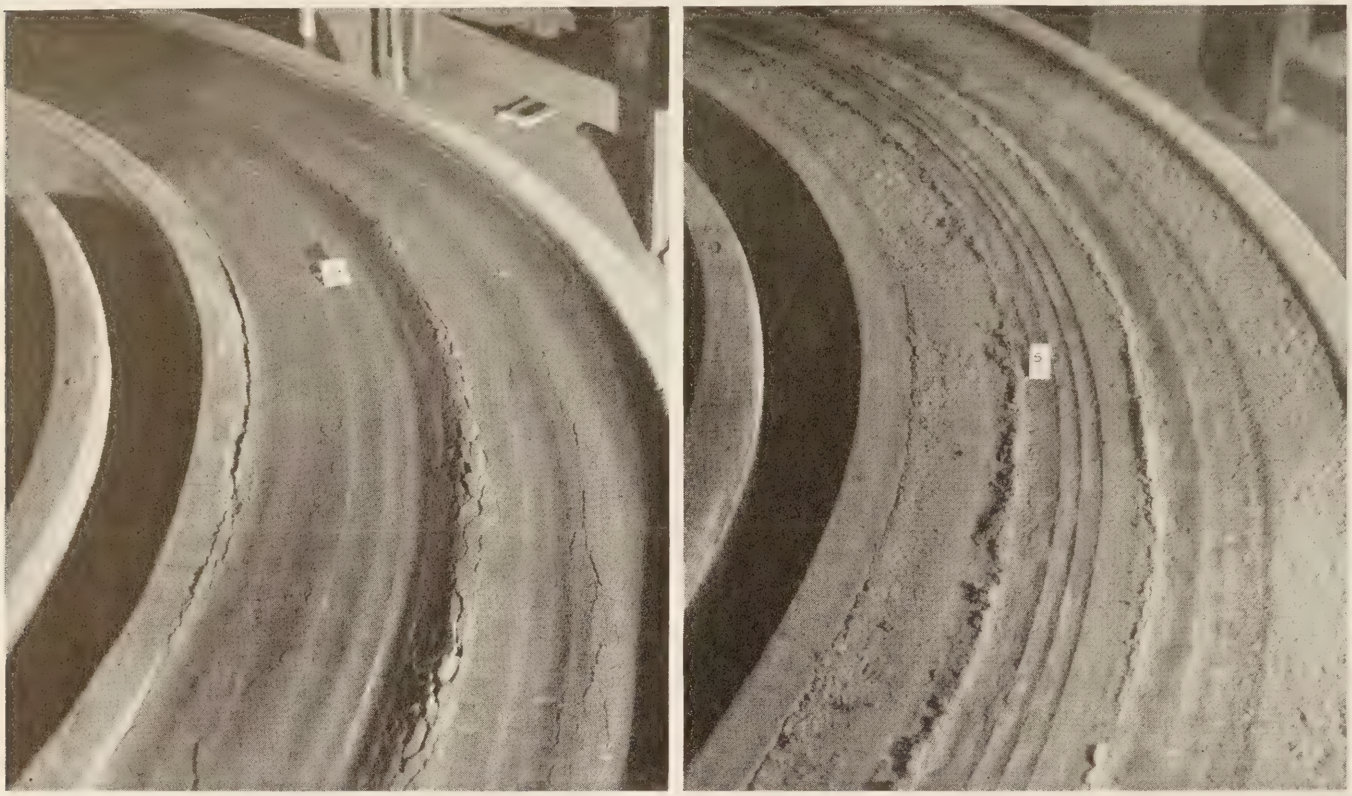


Figure 8.—Section 5 of Series 2 During Compaction at the Left and After Surface Treating and Applying Traffic to a Total of 200,000 Wheel-Trips at the Right. This Section Was Unstable During Early Compaction but Gave Excellent Service After Consolidation Was Completed.

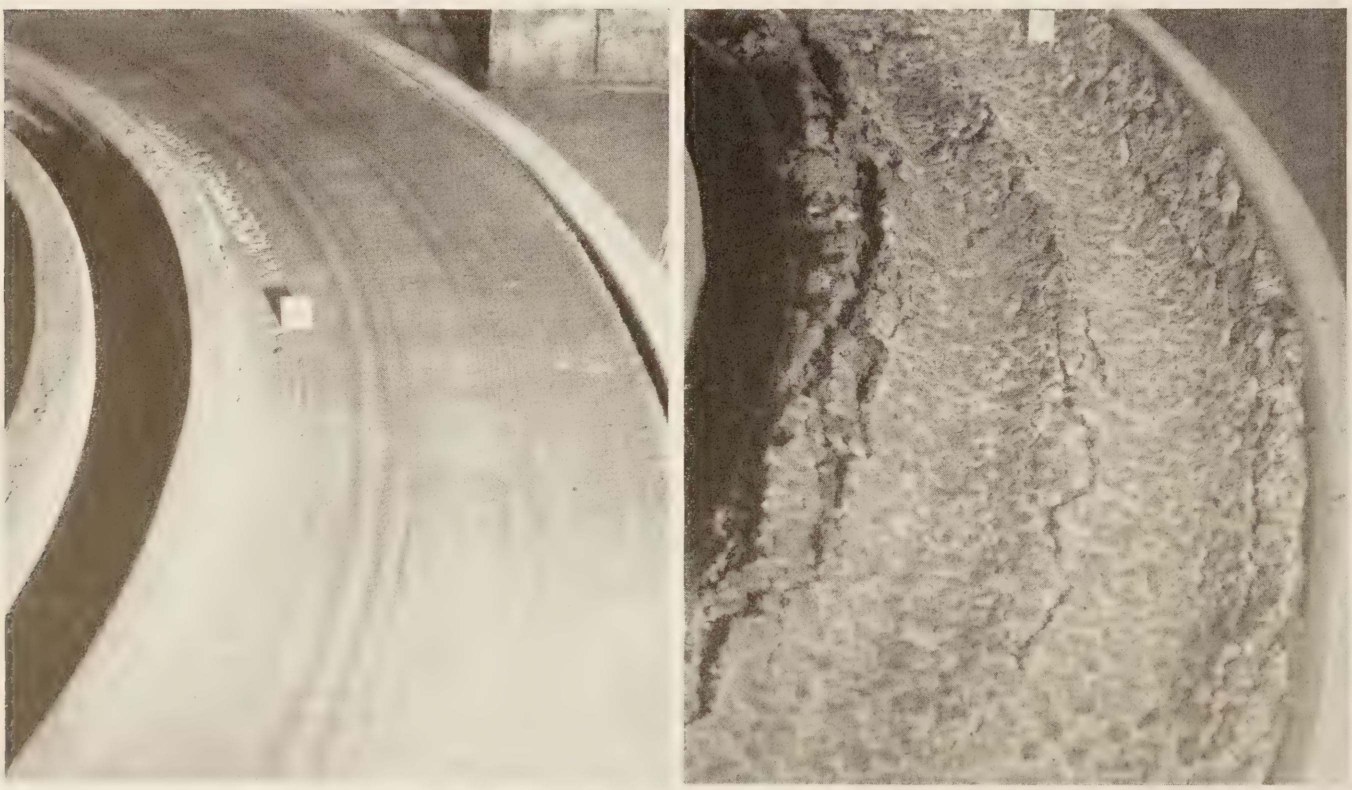


Figure 9.—Section 1 of Series 2. This Section Set Up Early During the Compaction Period, but Softened and Failed Because of Instability at 61,000 Wheel-Trips or Only 1,000 Wheel-Trips After Water Was Admitted to the Sub-base. Left Picture Taken at 20,000 Wheel-Trips or Early in Compaction Period; Right Picture Shows Surface Treated Base Section at Time of Failure.

stability and later developed high stability and resistance to the softening action of water. Figure 9 shows comparable views of section 1 of series 2, the behavior of which was typical of the sections that set up early in the compaction period but became unstable to the extent of failure under test traffic after water was admitted to the sub-base. The photograph at the left shows the section during early compaction and the one at the right shows the same section after it failed at 61,000 wheel-trips, or only 1,000 wheel-trips after water was admitted to the sub-base.

TRACK DENSITIES OF SATISFACTORY SECTIONS GENERALLY CLOSE TO MAXIMUM VIBRATED DENSITIES

Compaction tests made on the oven-dried materials from both tracks, using a vibratory method of compaction, gave the densities shown in table 7. The final densities of the track sections from tables 4 and 6 are repeated here for convenience of comparison. The densities obtained in the standard Proctor tests are also shown.

TABLE 7.—*Densities*[1] *of track materials after compaction by different methods*

Identification	Compacted by vibration	Top half of track at end of test	Proctor compaction
Series 1	*Percent*	*Percent*	*Percent*
Section 1	80.6	74.8	74.2
2	80.8	80.8	76.2
3	79.6	79.5	76.2
4	78.5	78.1	75.1
5	76.3	76.2	74.5
Series 2			
Section 1	79.6	74.7	75.0
2	81.9	76.1	76.5
3	80.5	79.6	75.3
4	78.8	74.9	75.9
5	81.4	81.2	76.2
6	82.3	80.6	77.8

[1] Percentage of aggregate volume per unit of total compacted volume.

It is believed that the densities obtained by vibrating the dry materials represent closely their maximum obtainable densities. Except for section 1 of series 1 which, because of its harshness, was stable even at the lowest density recorded, the materials that failed to compact in the track to essentially their maximum vibrated densities failed to withstand the traffic test. These were the materials in sections 1, 2, and 4 of series 2, and their lack of compactibility under traffic is attributed to their high content of material passing both the No. 40 and No. 200 sieves.

It is unlikely that the compactibility of materials 4 and 5 of series 1 played any part in their failure. They were very close to their maximum vibrated densities when failure occurred but both had sufficiently high plasticity indexes to cause them to soften and become plastic when subjected to wetting, even when fully compacted.

Except for materials 1, 2, and 4 of series 2, the densities obtained in the standard Proctor test were less than those of the track sections. However, the use of the standard Proctor test for predetermining probable actual densities of traffic- or roller-compacted, granular, road-building materials is of minor importance. It may, if desired, be modified to produce densities more nearly in agreement with those obtained on the traffic-compacted track sections by increasing either the number or the force of the tamping blows, but its value lies primarily in the ease and convenience with which it can be used to determine the necessary amount of water to obtain compaction of soil in highway construction.

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Supplementary tests made in the laboratory on small samples of the materials used in the track sections were:

1. The compaction tests with the Proctor apparatus, already discussed.
2. The vibratory compaction tests by which the maximum practical density of the aggregates was investigated.
3. Hubbard-Field stability tests on core samples from the track sections after the traffic test.
4. Hubbard-Field stability tests on molded specimens of each material over a range of moisture contents.
5. Hubbard-Field stability tests on molded specimens with constant water content molded under various loads to determine the effect of density on the stability values.

The Hubbard-Field stability tests were made in an effort to develop a method of predicting, by means of simple laboratory tests on small samples, the probable behavior of sand-clay, base-course materials under unfavorable service conditions. In making the test[3] a cylindrical specimen 2 inches in diameter and approximately 1 inch high is placed in a 2-inch cylinder, in the bottom of which is a plate with a 1¾-inch orifice. The specimen is forced through the orifice and the maximum load registered on the testing machine while forcing the specimen through the orifice is recorded as the stability. The test specimens may be either sections cut from core samples and tested at field density or they may be molded from loose material by the same method as that used by the Bureau in molding samples of sheet asphalt for the stability test. This method consists of placing the required amount of loose material in a 2-inch cylinder and compressing it under a load of 3,000 pounds per square inch with plungers in both ends of the cylinder so that compaction will be the same on both faces of the specimen.

EFFECT OF MOISTURE CONTENT ON STABILITY OF SPECIMENS STUDIED

The results of the Hubbard-Field stability tests on cores from all the test sections and the corresponding moisture contents, densities, and void contents, are shown in table 8. Sections 2 and 3 of series 1 and sections 3, 5, and 6 of series 2, showed high stabilities which agreed well with their service behavior. Sections 1, 2, and 4 of series 2 showed very low stabilities, and this was likewise consistent with their service behavior. The Hubbard-Field stabilities of sections 1, 4, and 5, of series 1, however, were highly inconsistent with their service behavior. The stability test values for section 1 were only about one-third those for section 2, while the service behavior of the two sections was equally good. Section 4 showed Hubbard-Field stabilities appreciably higher than those of section 1 and the values for section 5 were only slightly less than those for section 1. Yet both sections 4 and 5 failed in the traffic test.

Because of these inconsistencies it was concluded that the Hubbard-Field stability at any single water content or density could not be relied upon to furnish information as to the probable service behavior of a sand-clay, base-course material. The results of tests next to be described did, however, show that when stability tests were made on specimens covering a range

[3] Method described in Research Series No. 1, Oct. 15, 1935, published by The Asphalt Institute.

TABLE 8.—*Water content, density, and Hubbard-Field stability of cored specimens from the track sections in series 1 and 2*

SERIES 1

Section No.	Plasticity index	Depth	Water content based on dry weight	Composition by volume: Water	Composition by volume: Aggregate	Composition by volume: Air voids	Hubbard-Field stability	Remarks
			Percent	*Percent*	*Percent*	*Percent*	*Pounds*	
1	0	Top inch	7.5	15.2	75.4	9.4	(1)	Tested at 266,000 trips (end of traffic test).
		Second inch	7.6	15.3	74.8	9.9	1,160	
		Third inch	7.6	15.1	74.1	10.8	1,060	
2	5	Top inch	6.0	13.1	81.6	5.3	(2)	Do.
		Second inch	6.7	14.6	81.0	4.4	3,500	
		Third inch	7.4	15.8	79.8	4.4	2,820	
3	9	Top inch	7.4	15.9	79.9	4.2	3,050	Do.
		Second inch	8.0	17.1	79.9	3.0	2,570	
		Third inch	8.7	18.4	78.7	2.9	1,940	
4	13	Top inch	8.1	17.2	79.2	3.6	1,770	Do.
		Second inch	8.7	18.3	78.3	3.4	1,510	
		Third inch	9.8	20.2	76.9	2.9	1,070	
5	18	Top inch	9.2	19.0	77.3	3.7	1,100	Do.
		Second inch	10.2	21.0	76.7	2.3	740	
		Third inch	11.6	23.2	74.6	2.2	440	

SERIES 2

Section No.	Plasticity index	Depth	Water content based on dry weight	Composition by volume: Water	Composition by volume: Aggregate	Composition by volume: Air voids	Hubbard-Field stability	Remarks
1	4	Top inch	10.4	20.7	74.6	4.7	670	Tested at 61,000 trips (time of failure).
		Second inch	11.2	22.2	74.5	3.3	470	
		Third inch	9.9	19.8	74.9	5.3	730	
2	4	Top inch	9.5	19.3	75.9	4.8	930	Tested at 63,000 trips (time of failure).
		Second inch	10.1	20.6	76.2	3.2	640	
		Third inch	9.3	18.9	76.2	4.9	880	
3	5	Top inch	6.6	14.3	80.7	5.0	2,500	Tested at 200,000 trips (end of traffic test).
		Second inch	7.6	16.2	79.7	4.1	2,300	
		Third inch	8.0	16.8	78.5	4.7	2,050	
4	6	Top inch	9.9	20.0	75.5	4.5	660	Tested at 61,000 trips (time of failure).
		Second inch	10.8	21.6	74.9	3.5	410	
		Third inch	10.5	20.8	74.3	4.9	400	
5	6	Top inch	5.8	12.4	79.7	7.9	3,500	Tested at 200,000 trips (end of traffic test).
		Second inch	5.9	12.9	81.6	5.5	3,630	
		Third inch	5.9	13.0	82.4	4.6	3,700	
6	4	Top inch	5.2	11.3	81.2	7.5	3,210	Do.
		Second inch	5.5	11.9	80.8	7.3	3,120	
		Third inch	5.7	12.2	79.9	7.9	3,050	

[1] Top inch contained some tar prime so that comparable stability was not obtained.
[2] Top inch too brittle to permit making stability test.

of moisture contents the rate at which the Hubbard-Field stability decreased with increasing water contents, after rising to a maximum value, bore a strikingly definite relationship to the behavior of the materials in the track test.

A series of stability tests was made on specimens of the material from each track section molded at moisture contents ranging from well below that necessary to produce maximum density to as high a content as the soil would retain during molding. The specimens in series 1 were tested for density immediately after molding. They were then tested for stability and immediately weighed and oven-dried to determine their exact moisture contents. The results of these tests are shown in table 9 and figure 10.

Table 10 and figure 11 show the results of similar tests on the materials in series 2, the only difference being that, for series 2, the density and stability values were obtained from separate sets of specimens prepared and tested on different days. No special attempt was made to use identical moisture contents in these two sets of specimens.

As shown in figures 10 and 11, the satisfactory materials (sections 1, 2, and 3 of series 1 and sections 3, 5, and 6 of series 2) exhibited relatively small changes in Hubbard-Field stability within the range of moisture contents where stability was decreasing while density was increasing, whereas, the unsatisfactory materials (sections 4 and 5 of series 1 and sections 1, 2, and 4 of series 2) showed relatively large changes within the same range. All except material 1 of series 1 lost stability very rapidly after the maximum density was

TABLE 9.—*Relation of density and stability to water content in molded specimens of series 1* [1]

Section number	Plasticity index	Water content based on dry weight	Composition by volume: Aggregate solids	Composition by volume: Aggregate voids	Composition by volume: Air voids	Hubbard-Field stability
		Percent	*Percent*	*Percent*	*Percent*	*Pounds*
1	0	1.4	------	------	------	1,460
		1.8	------	------	------	1,550
		3.4	------	------	------	1,420
		4.3	74.3	25.7	17.1	1,420
		5.4	74.7	25.3	14.5	1,480
		5.5	------	------	------	1,490
		6.2	74.9	25.1	12.6	1,570
		7.1	74.7	25.3	11.1	1,440
		8.2	75.2	24.8	8.2	1,360
		9.4	75.3	24.7	5.7	1,490
2	5	4.1	78.8	21.2	12.5	3,760
		4.7	79.4	20.6	10.6	3,650
		5.2	79.8	20.2	9.1	3,640
		7.3	81.3	18.7	2.8	3,360
		7.6	81.4	18.6	2.0	3,025
		9.6	78.4	21.6	1.4	980
3	9	4.6	79.4	20.6	10.8	3,790
		6.0	80.9	19.1	6.1	3,630
		7.1	81.4	18.6	3.1	3,510
		7.3	81.4	18.6	2.7	3,390
		8.2	80.9	19.1	1.3	2,250
		9.4	79.0	21.0	1.1	1,160
4	13	3.9	79.6	20.4	12.1	4,630
		4.7	81.3	18.7	8.5	4,310
		6.0	82.3	17.7	4.5	3,330
		6.8	82.4	17.6	2.6	2,790
		8.0	81.3	18.7	1.3	1,760
		8.7	80.0	20.0	1.3	1,310
5	18	4.2	79.9	20.1	11.1	4,520
		4.5	80.7	19.3	9.6	4,340
		6.1	81.6	18.4	5.1	3,060
		7.2	82.2	17.8	2.0	2,180
		7.6	82.2	17.8	1.1	1,770
		8.4	80.9	19.1	.9	1,350
		10.6	76.6	23.4	1.7	580

[1] All specimens were molded at the water contents shown and tested immediately, using the Hubbard-Field apparatus for molding and testing.

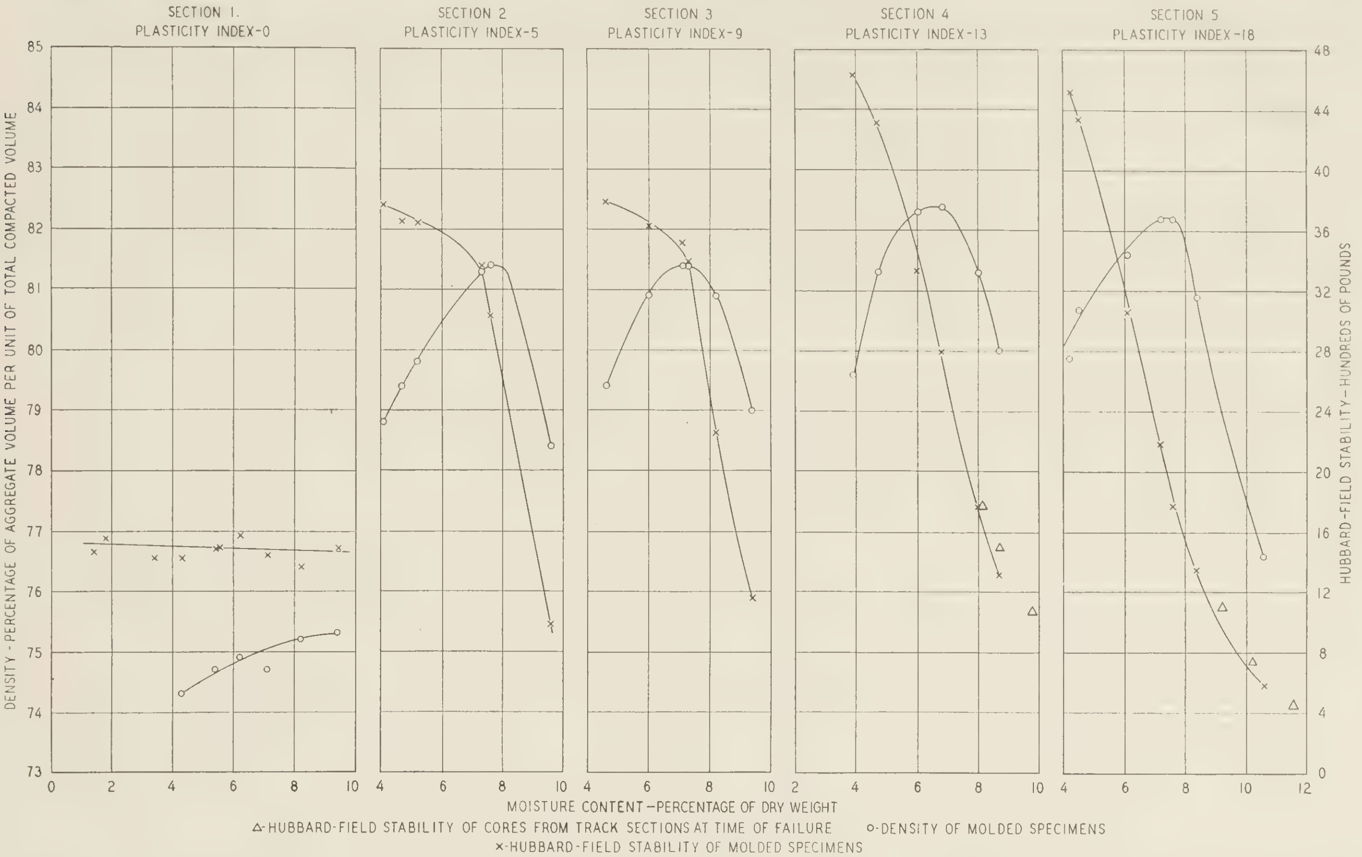

FIGURE 10.—RELATION OF DENSITY AND STABILITY TO MOISTURE CONTENT, SERIES 1. CURVES PLOTTED FROM DATA IN TABLES 8 AND 9.

TABLE 10.—*Relation of density and stability to water content in molded specimens of series 2* [1]

Section number	Plasticity index	Water content based on dry weight	Composition by volume: Aggregate solids	Composition by volume: Aggregate voids	Composition by volume: Air voids	Hubbard-Field stability
		Percent	*Percent*	*Percent*	*Percent*	*Pounds*
1	4	0.5	--------	--------	--------	1,550
		2.6	74.2	25.8	20.7	--------
		3.0	--------	--------	--------	2,620
		5.1	75.3	24.7	14.5	--------
		6.2	--------	--------	--------	3,300
		6.8	76.8	23.2	9.3	--------
		8.1	--------	--------	--------	3,600
		8.4	77.8	22.2	4.8	--------
		8.7	--------	--------	--------	3,010
		9.0	78.3	21.7	2.9	2,310
		9.1	78.4	21.6	2.6	--------
2	4	.6	--------	--------	--------	1,730
		2.8	75.8	24.2	18.5	--------
		4.2	76.3	23.7	15.1	--------
		5.8	76.8	23.2	11.3	--------
		6.3	--------	--------	--------	2,900
		7.0	--------	--------	--------	2,990
		7.9	--------	--------	--------	2,880
		8.0	78.6	21.4	4.6	2,270
		8.8	--------	--------	--------	1,170
		10.0	79.0	21.0	.0	--------
3	5	.6	--------	--------	--------	1,780
		3.2	74.5	25.5	19.1	--------
		5.1	76.2	23.8	13.4	--------
		6.3	76.7	23.3	10.4	2,200
		7.4	--------	--------	--------	2,230
		8.3	78.2	21.8	4.4	--------
		8.6	--------	--------	--------	2,170
		9.0	79.0	21.0	2.0	2,000
		9.5	79.1	20.9	.8	--------
		9.9	--------	--------	--------	1,600
4	6	1.6	--------	--------	--------	2,000
		2.9	74.8	25.2	19.4	--------
		4.9	75.6	24.4	14.5	--------
		5.8	--------	--------	--------	3,070
		6.5	77.4	22.6	9.2	--------
		7.2	--------	--------	--------	3,530
		7.5	--------	--------	--------	3,380
		8.1	79.3	20.7	3.5	2,990
		8.5	79.5	20.5	2.5	--------
		8.7	--------	--------	--------	1,790
		9.3	--------	--------	--------	830
5	6	1.6	--------	--------	--------	1,240
		2.9	74.4	25.6	19.8	1,700
		4.5	75.0	25.0	16.0	--------
		6.1	75.4	24.6	12.3	1,910
		7.9	--------	--------	--------	1,860
		8.0	--------	--------	--------	1,900
		8.2	77.6	22.4	5.4	--------
		8.9	78.5	21.5	2.8	--------
		9.0	78.8	21.2	2.2	--------
		9.2	--------	--------	--------	1,690
		10.5	--------	--------	--------	920
6	4	.5	--------	--------	--------	1,280
		2.4	76.6	23.4	18.5	2,410
		3.3	--------	--------	--------	2,570
		4.0	77.4	22.6	14.3	--------
		5.0	--------	--------	--------	2,800
		6.0	78.2	21.8	9.3	2,760
		6.4	--------	--------	--------	2,710
		7.5	--------	--------	--------	2,520
		8.0	80.2	19.8	2.7	--------
		8.4	80.5	19.5	1.4	2,120
		8.8	80.0	20.0	1.2	1,990

[1] Specimens for density and those for stability were made and tested at different times. The water contents shown were determined immediately after testing. No particular effort was made to have identical water contents in the density and stability specimens.

passed. The highest stabilities shown in figure 10 for series 1 represent only the maxima for the data obtained and are not necessarily the maxima that might have been obtained by including tests on still drier mixtures. It is obvious, however, that extension of these curves to the left through maximum obtainable stability would only serve to accentuate the difference between the good and bad materials.

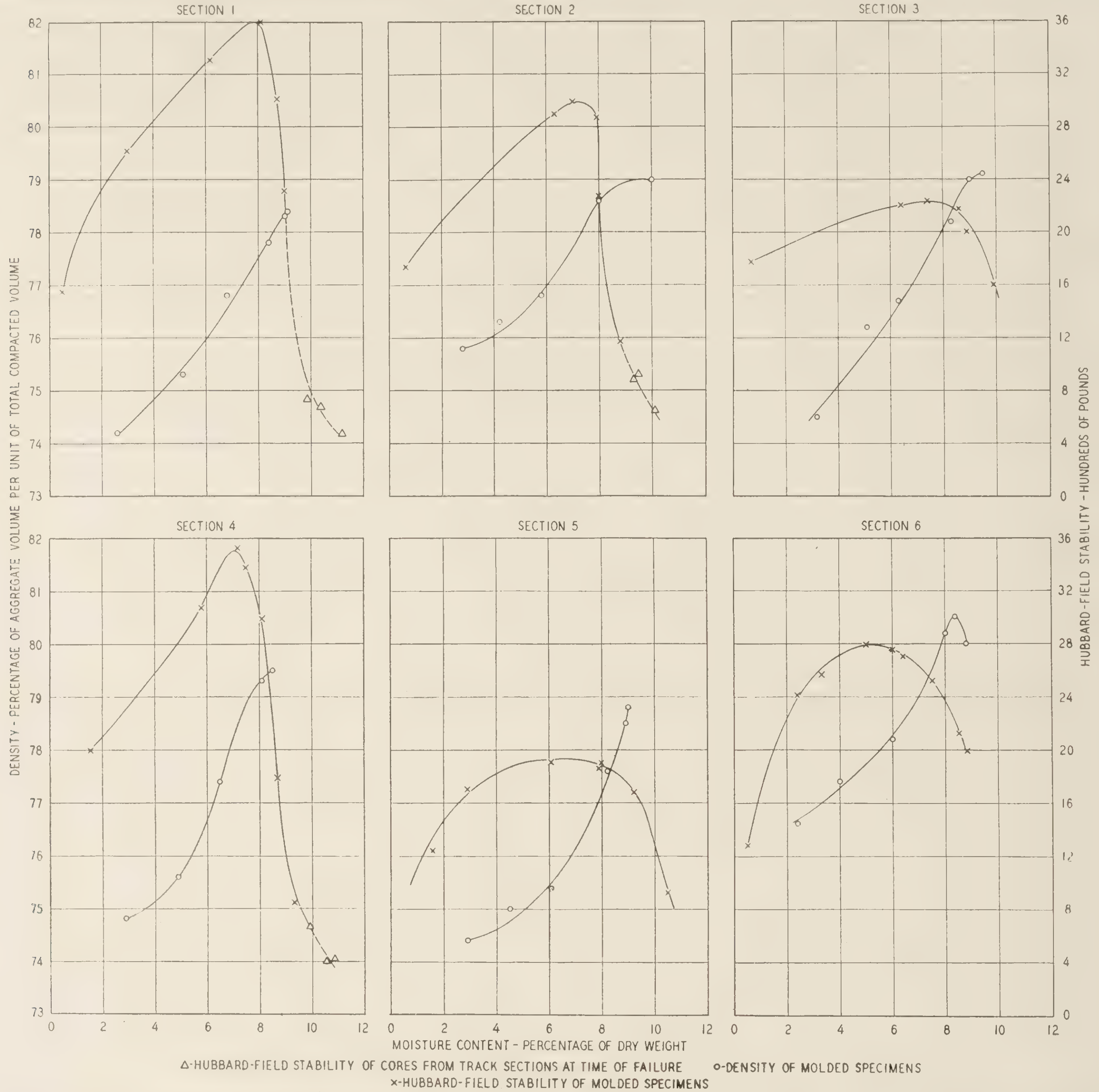

Figure 11.—Relation of Density and Stability to Moisture Content, Series 2. Curves Plotted From Data in Tables 8 and 10.

It is important to keep in mind that the changes in density and stability occurred because of changes in water content and that, in the range between maximum or essentially maximum stability and maximum density, without exception the stability was falling while the density was increasing. Tests to be described later proved that when the moisture content was held constant and density was increased by more intensive compaction, the Hubbard-Field stability invariably increased. Therefore, the losses in stability that occurred between maximum stability and maximum density (figures 10 and 11) were caused entirely by the increases in moisture content working counter to the stabilizing effect of increasing density.

The maximum stabilities and the stabilities at maximum density, with the corresponding water contents and the mean changes in stability per percent of moisture change, are shown in table 11. These rates of change, without exception, reflect the behavior of the materials in the track tests. For the satisfactory materials, the rate of stability change was low, ranging from the negligible amount of 6 pounds per percent of moisture change for material 1 of series 1 to 229 pounds for material 3 of series 1. The unsatisfactory materials exhibited much higher rates of change, the lowest for this group being 634 pounds for material 4 of series 1, and the highest being 1,800 pounds for material 1 of series 2.

Although tests on additional materials of known service behavior should be made to establish more closely the dividing rate of stability change between good and bad materials, the described method of test appears to be fundamentally sound as a means of predicting in the laboratory the probable service behavior of fine-grained base-course or subgrade material where moisture conditions may be unfavorable.

INCREASED CONSOLIDATION PRODUCED INCREASED STABILITY FOR A GIVEN WATER CONTENT

The results of the tests indicating the relations between Hubbard-Field stability and degree of compaction for representative soil materials at constant water content are shown in figure 12. The materials used were from track 1, but the curves are typical of those that would be obtained with soils in general. Each curve in this figure represents a series of tests at a selected water content which is constant for that series. Compaction procedure was varied in each series to produce a range of densities, and a corresponding range of stabilities resulted. On all four curves, points 1, 2, 3, and 4 represent test results on samples that were molded without pretamping at pressures of 1,000, 2,000, 3,000, and 4,000 pounds per square inch, respectively. Points 5, 6, 7, and 8 represent data on samples that were pretamped and then compressed at pressures of 3,000, 4,000, 5,000, and 6,000 pounds per square inch, respectively.

The specimens containing 4 percent of water showed progressive gains in both density and stability, corresponding to greater compaction, throughout the series. Those containing 8 percent of water also showed gains in density and stability throughout the series, but these gains were much smaller for the load increments between points 5 and 8 than for the lower loads, indicating that the maximum possible density was being approached. In the series having 10 percent of water, gains were observed up to point 6, although the gain in density between points 5 and 6 was so small as to indicate that little or no further increases could be obtained without squeezing out some of the water. In fact a slight amount of water was squeezed out of specimens 3, 4, 5, and 6 during molding, reducing their actual water contents to 9.4, 9.4, 9.4, and 9.3 percent, respectively. With a water content of 9.3 percent by weight or 19.7 percent by volume and a solid mineral volume content of 79.1 percent, only 1.2 percent of the compacted volume of specimen 6 consisted of air-filled voids.

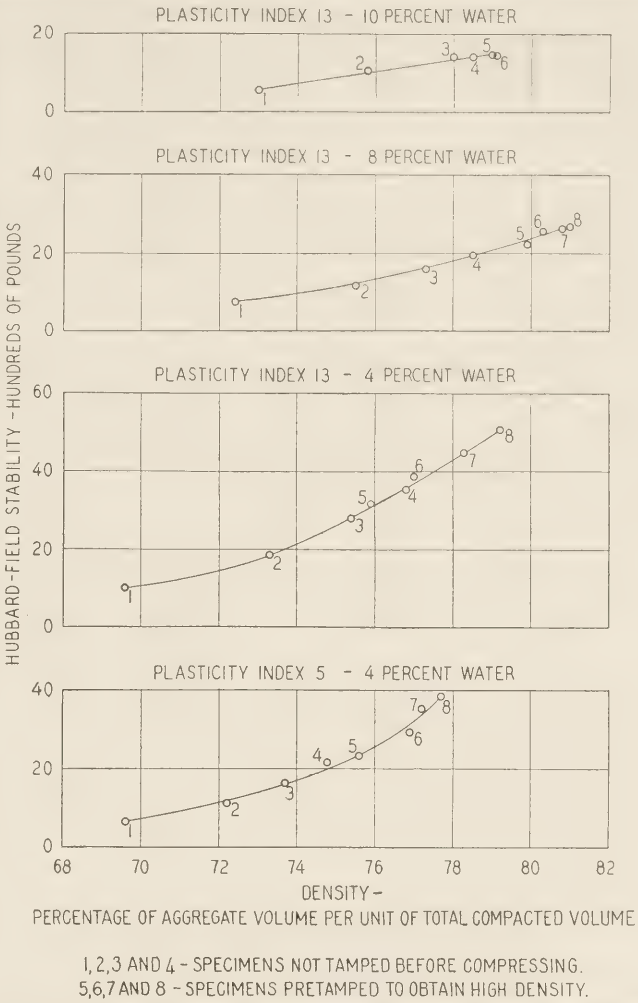

FIGURE 12.—RELATIONS BETWEEN HUBBARD-FIELD STABILITY AND DENSITY FOR SAND-CLAY MIXTURES.

TABLE 11.—*Change in Hubbard-Field stability accompanying change in moisture content in molded specimens*

SERIES 1

Section number	Hubbard-Field stability: Maximum	Hubbard-Field stability: At maximum density	Hubbard-Field stability: Difference	Water content: At maximum stability	Water content: At maximum density	Water content: Difference	Change in stability for a 1-per cent change in water content	Behavior of test section under traffic
	Pounds	*Pounds*	*Pounds*	*Percent*	*Percent*	*Percent*	*Pounds*	
1	[1] 1,500	[1] 1,450	50	1.6	9.4	7.8	6	Remained in excellent condition throughout both compaction and test periods.
2	3,760	3,025	735	4.1	7.6	3.5	210	Unstable during compaction but excellent throughout test period.
3	3,790	3,195	595	4.6	7.2	2.6	229	Fairly stable during compaction. Generally satisfactory throughout test period.
4	4,630	2,790	1,840	3.9	6.8	2.9	634	Unstable during compaction; failed under test traffic.
5	4,520	1,975	2,545	4.2	7.4	3.2	795	Stable during compaction but failed under test traffic.

SERIES 2

Section number	Hubbard-Field stability: Maximum	Hubbard-Field stability: At maximum density	Hubbard-Field stability: Difference	Water content: At maximum stability	Water content: At maximum density	Water content: Difference	Change in stability for a 1-per cent change in water content	Behavior of test section under traffic
1	3,600	[1] 1,800	1,800	8.1	9.1	1.0	1,800	Stable during compaction but failed under test traffic.
2	2,990	[1] 450	2,540	7.0	10.0	3.0	847	Do.
3	2,230	[1] 1,850	380	7.4	9.5	2.1	181	Unstable during compaction but excellent throughout test period.
4	3,530	[1] 2,100	1,430	7.2	8.5	1.3	1,100	Stable during compaction but failed under test traffic.
5	1,910	[1] 1,750	160	6.1	9.0	2.9	55	Unstable during compaction but excellent throughout test period.
6	2,800	[1] 2,200	600	5.0	8.4	3.4	176	Unstable during compaction but generally satisfactory during test period.

[1] Interpolated from stability curves, figures 10 and 11.

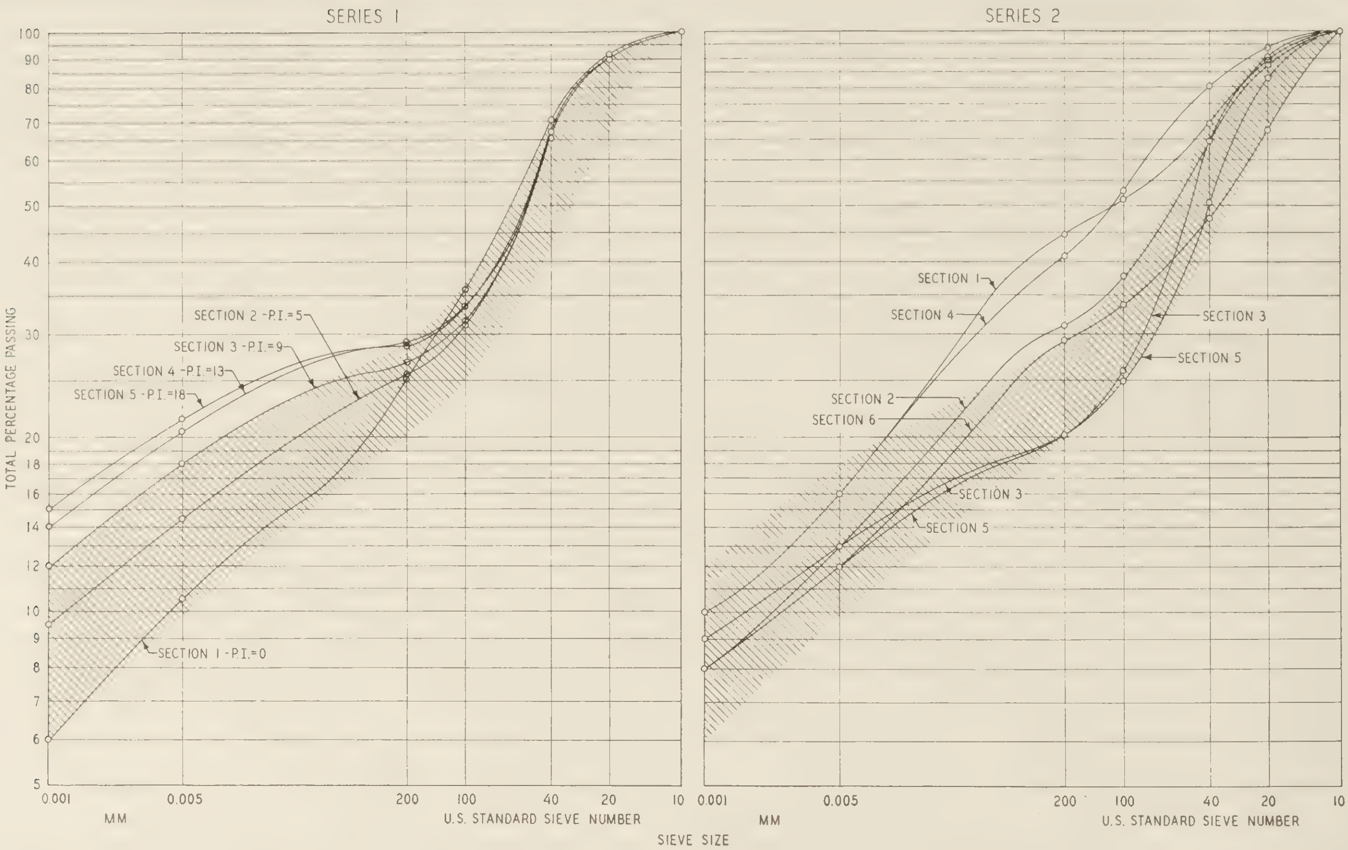

Figure 13.—Gradings of Materials in Series 1 and 2. Shaded Areas Indicate Zone Within Which All the Satisfactory Materials Are Included.

Although, without exception, increased consolidation produced increased stability for a given water content, indicating the importance of compaction, figure 12 shows that when the maximum density was reached on a very wet mixture, the stability was still far lower than that of a fully compacted mixture having less water. It is very difficult to consolidate dry or only slightly damp materials to their maximum density by any practical field method, while the presence of an appreciable amount of moisture definitely aids consolidation. The practical solution of the problem of consolidation during construction has been found to consist of adding the amount of water that tests indicate will produce the maximum compaction for the type of equipment employed. Where the compaction is to be obtained gradually as by the action of traffic, a quantity of water somewhat in excess of the Proctor optimum moisture content is desirable since under the combined drying and compacting action the maximum density is attained.

SUMMARY

The circular track tests resulted in a quite definite classification of the 11 materials studied as to their ability to withstand the disruptive action of traffic under unfavorable moisture conditions. Moisture conditions as severe as those set up for these tests are usually avoided in good highway building practice by providing surface drainage and, where needed, subsoil drainage. However, many instances occur, especially in connection with partial or low-cost improvement of soil roads where ideal drainage cannot be provided but where improvement of the plastic properties and grading characteristics of the soil are feasible.

The tests indicated that, to guard against unsatisfactory behavior under unfavorable moisture conditions, it is necessary to control both the plastic properties and the grading of the soil. The importance of thorough compaction, even of ideal base-course materials, prior to the application of a surfacing course cannot be over-emphasized.

In series 1, where the plasticity was varied by varying the ratio of silt to red clay, the gradings were essentially constant for the five materials for all grain sizes above the No. 200 sieve but varied with the plasticity index below the No. 200 sieve, as shown at the left in figure 13. A rather wide variation was obtained in the gradings in series 2 throughout the range of particle sizes as shown at the right in figure 13. In this series the variation in plasticity index was held to a minimum, the lowest being 4 and the highest 6.

The failure of sections 4 and 5 of series 1 was caused primarily by their susceptibility to softening and becoming plastic when wet, which is a distinguishing characteristic of materials having high plasticity indexes. Section 3, having a plasticity index of 9, was classified as a border-line material, although its behavior considering the severity of the test, was quite good. The other two materials, having plasticity indexes of 5 and 0, respectively, gave excellent service.

The failure of sections 1, 2, and 4 of series 2 can be attributed to no other factor but grading. The shaded areas in figure 13 were drawn to include the gradings of all the satisfactory materials and are partially bounded by the grading curves of the two border-line materials, section 3 of series 1, and section 6 of

(Continued on page 189)

REFLECTOR BUTTONS INSTALLED ON MICHIGAN HIGHWAY

By MURRAY D. VAN WAGONER, State Highway Commissioner, Michigan State Highway Department

HIGHWAY engineers have long sought means of eliminating or counteracting the effect of darkness in augmenting ordinary traffic hazards. The recent installation of special highway markers on U. S. Highway No. 16 between Lansing and Detroit, Mich., was made in an effort to counterbalance certain of the handicaps which darkness imposes on the safe use of the highway.

That darkness is a prime factor in causing highway accidents has long been recognized. Studies of traffic volumes and accident occurrence on Michigan highways have shown that there is close correlation between the two. However, when nighttime accident and traffic figures were isolated and examined, additional facts were revealed. Sixty percent of all fatalities on trunkline highways were found to occur during the hours of darkness when these highways carried only 20 to 30 percent of their total 24-hour traffic.

The reasons for this divergence are not hard to find, though they are somewhat difficult to define. In both daylight and darkness the motorist has very similar problems of physical arrangement and movement to contend with. Although the number of vehicles with which he must share the roadway decreases, the dimensions, alinement, and surroundings of the roadway itself do not change when daylight fades. The motorist's perception of the roadway, however, is radically limited and sometimes distorted at night.

In the daytime the normal driver's vision extends far ahead and to a considerable distance on either side of the road. Some time before he traverses a section of road the driver can clearly see inherent hazards such as those involved in: (1) Traffic and pedestrians on the road; (2) road alinement and grades; (3) width and condition of road surface and shoulders; (4) roadside developments as they divert attention or obscure vision of the road ahead; and (5) roads and driveways, from which traffic can enter, cross, or leave the highway.

Although exposure to accidents is greater during the daytime concentrations of traffic, the driver can perceive the number, kind, speed, and direction of all vehicles well in advance of passing them. By being forewarned of these elements of his constantly changing driving problem, because the elements are all clearly and coincidentally visible, he ordinarily will have plenty of time to act properly to prevent serious mishaps.

Darkness blanks out practically all of these elements from the driver's sight, but not from his memory. His view of the road is limited to the short section ahead illuminated by his headlights. The rest of the picture is made up of what he knows about roads in general and what he can mentally visualize of the highway ahead through his interpretation of the lights of other cars, the dim outlines of surrounding objects, and glimpses of signs, signals, and lane markings.

At night the driver attempts to discern the location of the road beyond the range of his headlights by watching telephone lines, fences, and lights in houses, but he is conscious of the vagueness of his perceptions, particularly those involving perspective. As a result

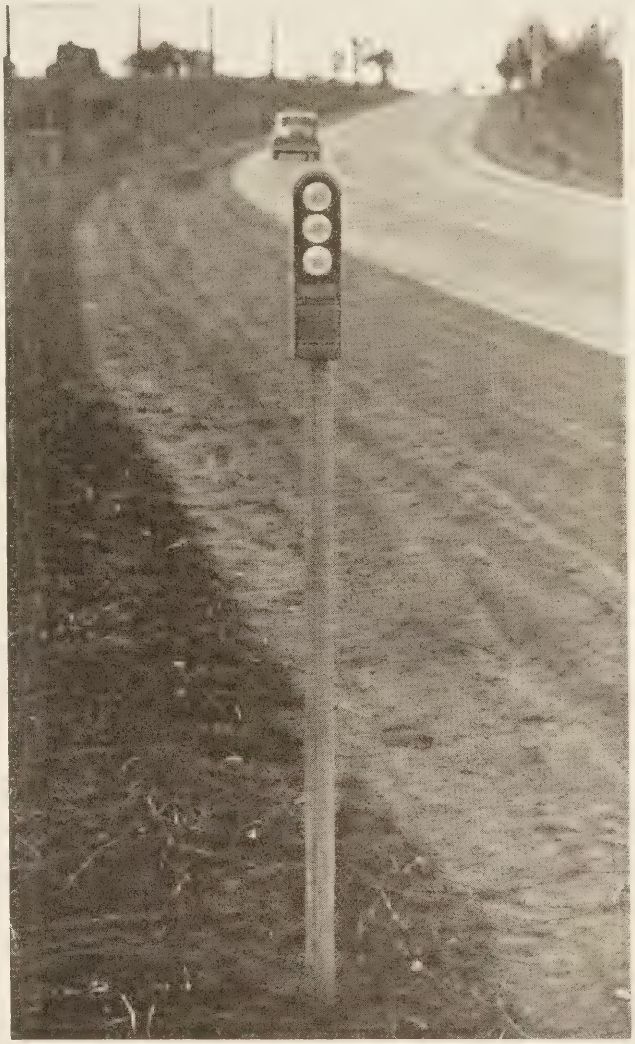

FIGURE 1.—A REFLECTOR UNIT AS INSTALLED ON U S 16 BETWEEN LANSING AND DETROIT, MICHIGAN.

he often becomes tense and over-wary, and constantly strives to rid himself of the feeling that he is driving into a darkened tunnel or that the road ends or turns abruptly just beyond his range of sight. When facing the glare of approaching headlights, he becomes uncertain of his car's position on the roadway and instinctively draws away from the pavement's right edge, often to the extent of encroaching on the lane of opposing traffic.

The obvious way to eliminate the dangers that darkness adds to highway hazards is to duplicate, as far as practicable, daytime visibility through the use of artificial light. Attempts to solve this problem with lights on the car itself have thus far proven but partially successful. Floodlight illumination of the highway is entirely feasible from an engineering angle, and such installations have been made on a few heavily traveled highways and bridges. However, the costs of construct-

Figure 2.—Reflectors Along the Highway Illuminated by the Headlights of Automobiles at Dusk.

ing and operating highway lighting installations have been so great as to prevent the lighting of any considerable mileage of highways.

The dangers of night driving have, in the past, been somewhat mitigated by the erection of signs, the letters of which are traced by reflector buttons, to warn the driver of hazards such as narrow bridges, sharp curves, etc. The use of such buttons to make warning signs visible at night has been standard practice in Michigan and other States for many years.

Recognition of the fact that something more than the warning of specific dangers was needed to protect night traffic led to the erection of special markers to outline the highway. These markers represent the nearest approach to highway illumination without the use of expensive direct lighting that has yet been attained. Although the markers give somewhat the impression of lights spaced at regular intervals along the road, they do not illuminate the road surface to any important extent. Their principal function is to delineate the highway clearly for a distance ahead of the car considerably greater than that illuminated by the headlights.

Each marker consists of 3 reflector disks set in a vertical line in a metal holder mounted on a metal post. (See fig. 1.) The disks are 1⅝ inches in diameter, and are spaced 1⅞ inches between centers in the holder. The disks, molded from a crystal-clear synthetic resin that is nonshatterable, are lenses with many facets each having high reflecting properties.

Considerable study was given to the proper arrangement of the markers to obtain the greatest safe-driving benefits for motorists. It was found that the proper mounting height was 3 feet above the road surface. Uniformity in longitudinal spacing and offset distance from the pavement edge were essential for accurate delineation of the roadway. Consequently, the units were spaced at intervals of 100 feet, and were offset 8 feet from the pavement edge in rural areas and 4 feet in urban areas where the road surface was bordered by a curb. On rural sections having extra-wide shoulders, the offset distance was increased to 10 feet.

At places where, by using this regular spacing, units would have been placed in side roads or driveways, the unit was either moved not more than 10 feet or was omitted. Units were omitted at places where they would have interfered with traffic entering or leaving business places having broad entrance driveways. No contraction of the offset distance was permitted because of the existence of any hazard on the road shoulder between the pavement and the line of markers.

The holders, which were mounted so that the plane of the reflectors was at an angle of 90° with the centerline of the road, are of two types: Monodirectional and bidirectional. In the monodirectional holder the 3 disks are backed by metal, so that they are visible from one side only. They were used on sections where the opposing lines of traffic were separated by a central dividing strip. In the bidirectional unit each disk is backed by another disk visible from the other side. These units were used on sections carrying 2-way traffic.

The holders are stamped from 12-gage sheet metal, and are coated with yellow enamel. They can be installed on the posts without tools, and can be removed by releasing a spring lock at the bottom. The posts consist of 1¼- by 1¼- by $\frac{9}{64}$-inch galvanized standard angle sections 6 feet long.

The location of each unit was marked by a stake placed by a survey party. Posts were driven by 40-pound pipe slip hammers developed especially for this work. A special wrench was developed for holding the posts while they were being driven.

Reflectors were installed on 61.6 miles of the highway between Lansing and Detroit, Mich. On 4.8 miles of this distance the roadways for the opposing streams of traffic were separated by a central dividing strip, and two lines of monodirectional markers were set along each roadway on this section.

The markers were installed by the Michigan State

Highway Department. The total cost of the work was \$23,000, which is at the rate of \$346.63 per mile, or \$3.34 per unit. The cost of the labor involved in locating and installing each marker was \$0.44.

Because of rigid adherence to the specified spacing and offset distance, the installation provides a simple but effective aid in night driving (figs. 2 and 3). As drivers become oriented to driving by the markers it is expected that many of the previous hazards to night driving will be eliminated.

The principal benefits derived from installing the markers are enumerated as follows:

1. The outline of the roadway is clearly visible for a considerable distance ahead, often as much as a mile where alinement and grades are favorable.
2. The presence of the light points along both sides of the roadway helps to minimize the blinding effect of undimmed headlights on approaching vehicles. It is believed that the installation may even encourage the practice of driving with dimmed lights.
3. Hazards that are present between the vehicle and the markers are revealed by the blanking out of one or more markers. Vehicles parked on the road shoulder or pavement, as well as moving vehicles without rear lights, and even pedestrians, are detected in this manner.
4. Gaps that occur because markers were omitted at road intersections and entrances to business places make the existence of these potential danger spots conspicuous.

Figure 3.—Reflectors Along the Highway Illuminated by the Headlights of Automobiles After Dark.

5. Because the disks show as points of light that are fixed and uniformly spaced, they enable the driver to obtain a much more accurate sense of perspective and judgment of distance than do the lights of other cars.
6. A final advantage of strictly adhering to a uniform spacing, offset, and mounting height in this installation, and in not using the units as danger markers, is that a way is left open for possible future development along logical lines. The present installation merely outlines the roadway by reflecting white light. It is possible to use units that will reflect colored light to indicate specific details of the roadway.

(Continued from page 186)

series 2. Parts of the grading curves for all five definitely unsatisfactory materials fall outside the shaded area on the left or fine side. The plasticities of the materials in series 1 are in reality related to grading because the material that causes plasticity is distinguished primarily by its extreme fineness and, as indicated in figure 13, the plasticities of these five materials are functions of the amount of very fine aggregate included in them.

Unsatisfactory or doubtful behavior of the materials in the tests is thus clearly associated with a swing of the grading curve to the left or fine side. Apparently, such a swing adversely affects the stability of the material whether it occurs immediately below the No. 10 sieve or wholly below the No. 200 sieve.

The data indicate that satisfactory control of stability would be obtained by maintaining limits of fineness essentially as shown by the upper border of the shaded areas in figure 13. The setting up of a maximum plasticity index requirement of 6, corresponding to the highest value used in series 2 and only 1 point higher than that of the most plastic material giving wholly satisfactory service in series 1, also appears to be a necessary safeguard in specifying sand-clay materials for base-course construction.

Experience and tests have shown that poorly graded materials are less stable than well graded ones and, in order to insure a reasonable gradation from coarse to fine, minimum limits for each size fraction are believed to be essential.

Both laboratory and field experience with bituminous surfacing mixtures and soil-bound bases have repeatedly demonstrated that when the aggregate content larger than the No. 10 sieve does not exceed about 35 percent, the material retains essentially the characteristics of the mortar or fraction finer than No. 10. Although the tests described in this report were made on materials all of which passed the No. 10 sieve, the results are believed applicable to materials containing some coarser material as long as the amount of such material does not exceed 35 percent.

The results of this investigation, as well as those of a later one on coarse-graded materials, 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 made the basis for the specification given in table 12.

Table 12.—*Recommended specification for sand-clay type base course materials*

	Minimum	Maximum
Percentage of material passing No. 10 sieve	65	100
Percentage of material finer than No. 10 sieve passing:		
No. 20 sieve	55	90
No. 40 sieve	35	70
No. 200 sieve	8	25
Percentage of material finer than No. 40 sieve passing No. 200 sieve (dust ratio)		50
Liquid limit (material finer than No. 40 sieve)		25
Plasticity index (material finer than No. 40 sieve)		6

CONCLUSIONS

The results of the tests justify the following conclusions:

1. Control of grading, particularly by limiting the amount of material passing the No. 40 and No. 200 sieves, is essential to insure satisfactory stability.
2. Control of grading in the sizes smaller than the No. 200 sieve is aided by establishing and maintaining a maximum limit for the plasticity index.
3. The tests demonstrated the importance of the plasticity index as a quality control. They also indicated that a plasticity index very much in excess of 6

cannot be permitted without danger of impairing the serviceability of the materials under unfavorable moisture conditions.

4. The use of an excessive amount of fine mineral dust, even though it may be relatively free of colloidal particles and therefore not productive of high plasticity, seems to retard or prevent effective compaction. This was evidenced in the tests of series 2, in which the materials having high dust contents and low plasticity indexes failed to compact well under traffic and failed soon after the water was introduced in the traffic test.

5. Thorough compaction is essential if satisfactory stability is to be obtained, particularly with base-course materials having plastic properties. Hubbard-Field stability tests on specimens having the same moisture content but different degrees of compaction showed that stability increased consistently as the amount of consolidation increased. Sections 1, 2, and 4 of series 2 failed to consolidate under traffic to densities comparable to their maximum practical densities as determined by vibratory compaction tests, and this characteristic of noncompactibility is believed to have contributed largely to their failure in the service test.

6. The noncompactibility of the unsatisfactory sections in series 2 was caused by their high dust content and is quite different in its results from the noncompactibility of section 1, series 1, which because of its harshness never became particularly dense but showed good serviceability.

7. Early difficulties encountered in compacting soils having acceptable gradings and plasticity indexes need not be taken as an indication of poor quality since in these tests such soils, without exception, gave satisfactory service when compaction in conjunction with drying was continued until essentially maximum practical density was obtained.

8. Compaction of the base courses having plastic properties should be completed to essentially maximum practical density before surfacing courses are applied because movements in the base course that are common during compaction of good materials will cause damage to prematurely constructed surfacing courses.

9. In the Hubbard-Field stability tests increases in moisture content were invariably accompanied by decreases in stability except at the very low water contents where the materials could not be properly compacted for the stability test. The rate of stability change per percent of change in moisture content, over the range of moisture contents where stability was falling while density was increasing, appears to be highly significant as a means of predicting service behavior. (See figures 10 and 11, and table 11.)

PUBLICATIONS ON BRIDGE FLOOR DESIGN AVAILABLE

Two publications that present solutions to problems of bridge floor design are now available. The solutions are applicable to various arrangements of slab and rigid or flexible supports which in the past have been susceptible of only the roughest rule-of-thumb design.

These publications, both University of Illinois Bulletins, are: No. 303, "Solutions for Certain Rectangular Slabs Continuous Over Flexible Supports," and No. 304, "A Distribution Procedure for the Analysis of Slabs Continuous Over Flexible Beams." The bulletins are the result of a cooperative investigation by the Engineering Experiment Station of the University of Illinois, the United States Bureau of Public Roads, and the Illinois Division of Highways.

The Bureau of Public Roads has a limited number of these bulletins for free distribution. Requests should be addressed to the Bureau of Public Roads, Department of Agriculture, Washington, D. C.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF OCTOBER 31, 1938

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	$2,474,871	$1,143,635	68.3	$8,577,587	$4,277,875	361.6	$2,027,554	$1,012,685	59.3	$2,317,246
Arizona	1,529,586	1,211,663	87.7	868,732	624,897	30.1	402,971	225,311	14.5	1,239,784
Arkansas	826,321	821,912	65.3	2,394,901	2,391,298	151.0	916,805	915,930	57.0	1,098,294
California	4,516,963	2,403,557	97.1	9,134,644	4,853,656	166.6	2,245,024	1,201,220	55.2	1,380,789
Colorado	1,884,301	1,026,742	68.0	3,189,623	1,769,547	103.2	620,310	220,710	18.2	1,707,876
Connecticut				1,373,618	672,300	14.7	244,930	120,990	2.0	1,525,005
Delaware	318,655	159,140	4.0	799,097	395,550	19.5	199,690	99,845	.1	1,186,556
Florida	518,900	259,450	16.9	3,787,138	1,893,569	79.7	468,000	234,000	.3	2,981,695
Georgia	3,345,960	1,661,822	170.2	5,646,414	2,823,207	266.4	676,280	338,140	42.7	5,483,199
Idaho	1,516,494	901,846	157.1	1,613,753	962,216	71.0	88,216	52,796	5.4	1,096,155
Illinois	4,454,334	2,243,843	147.3	10,688,711	5,343,049	222.1	1,917,690	958,845	60.8	2,284,235
Indiana	2,340,690	1,170,345	70.4	5,008,669	2,504,335	102.5	1,803,024	893,466	36.7	2,181,226
Iowa	3,488,075	1,616,819	115.9	7,394,006	3,339,683	238.3	1,153,768	542,300	39.8	152,947
Kansas	2,771,755	1,384,791	406.7	4,964,881	2,438,750	420.4	3,346,652	1,672,326	153.9	2,797,831
Kentucky	2,714,554	1,344,535	100.3	4,751,912	2,375,956	136.2	1,434,094	717,047	43.8	2,073,578
Louisiana	814,659	407,227	28.9	11,742,342	2,265,858	39.6	1,761,038	775,592	28.6	2,272,042
Maine	2,209,580	1,100,369	51.2	1,651,161	825,580	34.6	569,260	284,629	10.0	84,849
Maryland	520,000	260,000	8.6	2,097,767	1,047,121	34.9	1,270,781	611,730	19.2	1,670,916
Massachusetts	1,562,989	781,493	7.1	2,639,271	1,319,632	15.1	1,175,626	585,355	12.6	2,119,705
Michigan	5,269,093	2,541,884	107.5	5,259,583	2,627,237	140.2	1,049,490	521,390	20.1	1,480,614
Minnesota	1,649,382	769,274	67.0	7,675,990	3,815,033	387.7	1,443,896	721,043	75.7	1,837,387
Mississippi	1,641,308	779,312	63.0	8,421,212	3,144,207	379.6	2,676,160	1,077,870	101.6	2,024,656
Missouri	2,945,444	1,459,173	98.0	3,249,950	1,591,110	75.0	4,532,557	2,030,444	184.4	3,024,930
Montana	1,763,004	991,238	82.1	495,573	278,176	12.5	252,912	142,262	8.4	4,361,587
Nebraska	2,273,484	1,136,634	268.5	6,206,076	3,093,437	460.9	3,332,814	1,032,854	202.7	1,830,681
Nevada	1,210,686	1,048,938	163.1	791,087	679,651	21.9	217,950	189,001	5.0	1,176,650
New Hampshire	817,799	405,206	21.1	501,386	249,733	3.8	93,802	46,900	1.4	1,141,725
New Jersey	242,315	121,100	1.9	2,786,016	1,390,128	19.8	370,230	184,490	3.3	2,505,753
New Mexico	1,642,148	1,001,044	187.5	1,805,471	1,201,322	100.6	841,369	513,146	47.0	340,604
New York	5,628,509	2,757,475	123.6	13,839,330	6,846,742	223.8	4,717,370	2,174,495	73.2	1,494,619
North Carolina	4,610,957	2,186,400	180.3	5,554,799	2,771,532	322.1	1,446,660	702,530	88.3	1,784,819
North Dakota	2,039,728	1,928,527	173.6	1,739,781	1,530,474	138.0	83,622	44,787	7.4	3,568,606
Ohio	4,282,540	2,114,689	53.1	9,460,472	4,709,270	87.0	2,811,060	1,395,163	36.7	5,700,481
Oklahoma	3,580,717	1,890,953	150.4	4,464,673	2,323,247	138.2	1,172,660	622,764	38.9	2,601,051
Oregon	2,222,570	1,296,270	89.2	1,251,206	755,997	22.6	636,823	388,490	85.1	1,786,618
Pennsylvania	5,560,911	2,759,048	95.2	8,246,282	4,115,964	96.6	2,771,445	1,369,252	26.5	2,691,996
Rhode Island	559,790	279,895	7.7	978,112	489,056	12.2	73,900	36,950	.6	1,028,924
South Carolina	3,244,245	1,431,448	191.0	2,975,292	1,325,376	95.0	1,722,221	788,500	49.0	1,232,875
South Dakota	1,176,622	666,065	143.0	4,466,176	2,469,840	399.0	1,235,890	682,900	131.2	2,540,329
Tennessee	2,586,220	1,293,110	94.1	4,036,354	2,018,177	93.0	1,267,040	633,020	30.6	4,046,640
Texas	8,408,382	4,170,715	552.8	11,055,979	5,323,067	531.5	3,527,337	1,696,440	159.8	5,544,952
Utah	777,266	541,930	104.2	1,915,150	1,364,145	62.2	296,595	208,615	13.7	703,283
Vermont	1,010,022	459,608	28.4	881,090	415,883	23.4	309,470	154,545	5.4	158,292
Virginia	3,792,051	1,894,195	126.0	4,306,764	2,151,841	145.1	1,158,260	576,780	29.9	563,911
Washington	2,730,290	1,435,225	59.0	2,852,944	1,495,259	42.2	934,709	491,500	11.0	686,362
West Virginia	796,020	572,573	31.0	2,078,993	1,272,718	57.2	840,440	414,565	19.8	2,026,161
Wisconsin	3,204,269	1,554,071	96.5	7,171,759	3,350,450	210.5	472,173	217,400	19.0	1,458,413
Wyoming	1,845,727	1,136,088	213.8	844,002	515,981	92.8	343,970	212,540	51.0	517,704
District of Columbia	513,335	256,538	9.9	1,126,605	554,362	17.2	28,450	11,865	.3	1,202,411
Hawaii				1,128,739	562,390	20.5	63,720	31,660	4.2	640,325
Puerto Rico										
TOTALS	115,833,521	60,777,815	5,253.5	215,891,073	106,555,884	6,939.6	63,046,708	30,777,078	2,191.3	97,357,287

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF OCTOBER 31, 1938

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	$ 73,700	$ 36,850	10.7	$ 488,300	$ 243,850	27.1	$ 344,805	$ 170,550	11.8	$ 601,722
Arizona	281,361	187,459	18.6	111,297	70,491	6.8	86,796	51,357	10.7	347,175
Arkansas				19,426	12,863		195,706	195,541	21.5	655,704
California	484,307	279,472	45.9	1,231,089	668,519	51.6	772,772	392,914	40.1	506,321
Colorado	439,466	243,558	31.6	581,370	322,425	30.1	156,140	65,010	7.3	290,575
Connecticut				79,344	39,512	1.3	38,340	19,135	.3	259,631
Delaware				59,680	29,340	10.9	44,024	22,012	6.0	195,023
Florida				20,122	10,061		293,782	146,850	4.9	517,941
Georgia	197,440	96,612	23.8	488,118	244,059	61.2	292,280	146,140	44.0	771,911
Idaho	240,507	110,501	40.2	268,483	117,491	17.1				246,379
Illinois	775,700	387,850	69.7	1,569,392	730,696	107.6	764,900	373,950	47.1	534,419
Indiana	370,100	144,250	42.9	818,500	391,450	77.9	798,948	384,152	68.9	322,586
Iowa										1,298,449
Kansas	43,670	21,835	.5	95,410	47,705	13.3	236,972	118,485	42.7	1,130,946
Kentucky	486,158	142,751	64.0	829,660	233,659	55.6	868,143	255,398	77.6	182,618
Louisiana				267,530	119,085	21.9	801,332	335,820	67.1	269,231
Maine	240,400	120,200	15.8	378,226	183,996	19.9	18,700	9,350	1.2	15,027
Maryland				45,164	15,787	5.8	237,800	95,400	12.7	301,289
Massachusetts				78,010	39,005	1.4	237,740	118,080	4.9	489,315
Michigan	41,961	19,481	7.1	796,744	398,372	61.2	290,250	145,125	10.3	972,561
Minnesota	217,978	101,203	30.8	484,284	237,735	53.3	262,924	131,462	10.4	909,180
Mississippi				299,000	149,500	23.8				739,427
Missouri	255,371	125,865	35.2	197,480	98,350	14.4	573,710	253,365	103.2	588,092
Montana				13,983	7,865					1,027,170
Nebraska	299,900	149,950	47.4	406,738	203,369	67.2	516,788	248,693	90.0	394,745
Nevada	344,302	295,660	50.8	102,833	86,034	24.7	48,020	41,642	3.5	71,016
New Hampshire	97,426	48,277	1.8	151,155	75,181	4.3	81,281	39,806	2.4	83,611
New Jersey				274,120	135,215	5.0	48,780	17,500	.1	520,058
New Mexico	404,779	246,873	17.6	333,076	183,006	23.4	470,128	234,453	26.7	142,440
New York	1,673,234	833,319	118.0	2,107,000	1,053,500	122.1	452,800	213,900	25.8	261,776
North Carolina	302,920	151,460	36.3	980,204	486,630	84.7	208,720	96,490	18.0	347,275
North Dakota	33,860	18,135	5.5	119,670	64,092	29.3	115,886	62,065	8.5	642,960
Ohio	156,560	78,280	3.8	27,840	13,920		164,600	81,750	10.2	1,657,691
Oklahoma	12,200	6,491		396,738	209,979	34.3	449,030	215,734	29.9	752,711
Oregon	376,674	228,080	53.5	80,975	49,452	2.5	55,431	24,900	7.6	452,701
Pennsylvania	1,108,373	534,838	84.9	1,485,615	732,319	80.6	1,110,998	517,049	58.8	332,478
Rhode Island	66,840	33,420	3.5	121,188	60,594	4.0	131,140	65,570	1.7	28,307
South Carolina	155,841	69,232	16.6	811,345	345,230	93.4	300,420	112,900	28.2	151,318
South Dakota				11,300	6,250					816,436
Tennessee	78,900	39,450	2.2	631,306	242,753	34.3	159,480	79,740	6.0	697,547
Texas	1,080,001	505,260	177.8	2,053,183	897,470	220.0	1,000,701	434,555	124.6	1,287,484
Utah	375,496	205,870	38.8	226,449	120,125	15.9	75,170	37,360	8.7	187,015
Vermont	192,650	84,150	11.6	138,306	69,153	6.2	43,300	20,500	.5	35,793
Virginia	333,947	164,758	43.4	824,230	362,175	54.3	99,190	49,595	8.5	315,471
Washington	400,253	210,000	43.0	257,753	135,578	22.1	413,793	202,600	26.0	182,378
West Virginia	124,300	62,150	9.5	119,750	59,875	11.9	108,950	54,475	6.2	373,174
Wisconsin	141,919	62,885	4.6	989,096	481,460	47.8	109,051	54,000	2.1	613,883
Wyoming	389,390	240,600	53.9	302,002	186,599	12.6	187,238	115,691	14.8	42,303
District of Columbia										
Hawaii				56,250	28,125	2.4				218,750
Puerto Rico				272,250	135,650	15.2				111,225
TOTALS	12,297,884	6,287,025	1,261.3	22,500,984	10,836,050	1,680.4	13,666,959	6,451,064	1,101.5	23,893,238

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.

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.

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.

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 OCTOBER 31, 1938

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	$220,910	$220,910	5		4	$544,948	$543,924	6			$428,505	$427,600	5		3	$793,085
Arizona						9,452	9,452									620,761
Arkansas	46,565	45,628	3			423,519	423,299	7			124,319	124,039	4			1,165,803
California	366,760	366,760		2		1,266,665	1,266,090	7	2		196,046	196,046	2			1,871,313
Colorado	30,465	27,839	1		1	61,728	61,728	1			38,842	35,082			16	1,163,663
Connecticut						18,930	12,665									831,825
Delaware	11,500	11,500			3						65,770	65,770			20	416,480
Florida						196,216	196,216	2			207,100	207,100	1			1,002,481
Georgia						45,786	45,786	1			338,250	338,250	5		1	2,033,351
Idaho	87,825	87,652	1			177,264	177,264	3			173,299	159,270	4			398,684
Illinois	186,500	186,500	1		1	1,544,675	1,544,675	5	2		896,230	896,230	9		60	2,577,238
Indiana	408,486	322,500	2	3		462,000	435,100	3	2		689,780	689,780	1	1	89	966,769
Iowa	640,258	606,297	5		11	570,770	538,800	7	1	6	36,420	33,900		2	4	1,388,564
Kansas	456,074	456,074	8		4	383,515	383,515	5			675,801	675,801	8		7	1,059,118
Kentucky	145,000	145,000	1			51,100	51,100	1			584,830	584,830	7	2	31	1,033,115
Louisiana						225,285	196,478	3			248,872	248,090	9			1,132,503
Maine	48,590	48,590	2			327,315	327,315	3	2		69,830	69,830	1		1	245,468
Maryland						64,586	64,586	1								962,247
Massachusetts						275,196	275,000	1	2		235,125	233,194		3		1,405,094
Michigan	844,383	837,972	7	1	7	689,516	689,516	7	1	38	81,290	81,290		1		1,676,407
Minnesota	40,218	40,218		1	1	770,180	770,180	3	5		5,589	5,589			1	1,629,147
Mississippi						406,900	406,900	5			308,900	308,900	2			868,351
Missouri	128,120	128,120	2	1		223,970	223,970	2			453,520	453,520	2	1		2,219,643
Montana	248,112	243,230	3			556,992	556,992	5								524,249
Nebraska	150,374	150,374	4			390,393	390,393	10			556,270	556,270	8		33	667,373
Nevada	97,187	97,187		2	2						157,121	157,121	1	1	3	204,333
New Hampshire	61,732	61,425		1		93,487	93,428	5			4,590	4,590			2	334,307
New Jersey						210,005	204,779	2	1		130,800	130,800		1	2	1,634,678
New Mexico	109,669	109,669	4			196,671	196,671	3	1							545,151
New York	141,400	141,400		1	2	2,447,252	2,438,751	9	8		148,600	148,150		1		4,041,628
North Carolina	73,550	73,550		1		771,710	739,010	6	4		300,730	300,730	2	1	11	1,331,104
North Dakota						606,278	606,278	2	2							972,943
Ohio						95,850	95,850	2			640,700	640,700	7			3,492,618
Oklahoma						17,343	17,343				129,105	128,305			34	2,135,861
Oregon	308,391	307,742	1	2		58,634	58,634									608,955
Pennsylvania	138,043	122,837	1			161,073	161,073	1			143,528	138,621	1			5,310,004
Rhode Island						335,019	335,019	1	2							158,731
South Carolina						169,559	114,593	3	2		215,610	215,610	10		38	1,158,899
South Dakota	91,230	91,230	1		6	250,628	250,628	2	2	3	90,870	90,870	4		2	887,372
Tennessee						14,381	14,381				105,010	105,010		2	5	1,772,642
Texas	34,527	33,861	2			1,219,529	1,218,452	12	5		301,040	254,605	3			3,836,198
Utah	97,174	97,174	2		1	47,359	47,359	2			5,319	5,319			3	423,211
Vermont	202,882	197,882	6	1		29,964	29,964		1	1	31,640	31,640			11	220,950
Virginia	245,670	245,670	12	1		241,018	241,018	3	1		386,338	386,338	4	1	8	923,004
Washington	149,718	147,618	1	2	2	613,456	613,456	7	2		87,926	87,926		1	8	534,800
West Virginia	120,100	119,080	1			315,009	315,009	1			40,070	40,070	2		6	849,939
Wisconsin	162,493	162,493	2			1,246,886	1,160,026	12								1,150,990
Wyoming	30,890	30,890		1	1	135,630	135,630		1		15,960	15,960			6	494,250
District of Columbia											34,262	34,262		1		291,169
Hawaii						33,230	33,230	1			152,570	152,570	2	1		307,950
Puerto Rico						214,569	213,370	4								516,930
TOTALS	6,124,796	5,964,872	78	20	46	19,211,441	18,924,896	166	49	48	9,536,377	9,459,578	104	20	405	62,791,349