PUBLIC ROADS
A JOURNAL OF HIGHWAY RESEARCH
UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 15, NO. 1 MARCH 1934

THE HYDRAULIC FILL AT FOUR MILE RUN

For sale by the Superintendent of Documents, Washington, D.C. - See page 2 of cover for prices

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

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 15, No. 1 — March 1934

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

	Page
A Study of Hydraulic Fill Settlement	1
Frost Heave in Highways and its Prevention	10
Laboratory Tests of Resilient Expansion Joint Fillers	17

THE BUREAU OF PUBLIC ROADS - - - - Willard Building, Washington, D.C.
REGIONAL HEADQUARTERS - - - - - - - - - Mark Sheldon Building, San Francisco, Calif.

▾

DISTRICT OFFICES

DISTRICT No. 1. Oregon, Washington, and Montana.
Post Office Building, Portland, Oreg.

DISTRICT No. 2. California, Arizona, and Nevada.
Mark Sheldon Building, 461 Market St., San Francisco, Calif.

DISTRICT No. 3. Colorado, New Mexico, and Wyoming.
237 Custom House, Nineteenth and Stout Sts., Denver, Colo.

DISTRICT No. 4. Minnesota, North Dakota, South Dakota, and Wisconsin.
410 Hamm Building, St. Paul, Minn.

DISTRICT No. 5. Iowa, Kansas, Missouri, and Nebraska.
Saunders-Kennedy Building, Omaha, Nebr.

DISTRICT No. 6. Arkansas, Louisiana, Oklahoma, and Texas.
Old Post Office Building, Fort Worth, Tex.

DISTRICT No. 7. Illinois, Indiana, Kentucky, and Michigan.
South Chicago Post Office Building, Chicago, Ill.

DISTRICT No. 8. Alabama, Georgia, Florida, Mississippi, South Carolina, and Tennessee.
Shepherd Building, P.O. Box J, Montgomery, Ala.

DISTRICT No. 9. Connecticut, Maine, Massachusetts, New Hampshire, New Jersey, New York, Rhode Island, and Vermont.
Federal Building, Troy, N.Y.

DISTRICT No. 10. Delaware, Maryland, North Carolina, Ohio, Pennsylvania, Virginia, and West Virginia.
Willard Building, Washington, D.C.

DISTRICT No. 11. Alaska.
Room 419, Federal and Territorial Building, Juneau, Alaska.

DISTRICT No. 12. Idaho and Utah.
Federal Building, Ogden, Utah.

Because of the necessarily limited edition of this publication it is impossible to distribute it free to any person or institutions 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.

A STUDY OF HYDRAULIC FILL SETTLEMENT

Reported by HENRY AARON, Assistant Highway Engineer, Division of Tests, Bureau of Public Roads

PORTIONS of the Mount Vernon Memorial Highway traversed tidal muck flats adjacent to the Potomac River and were constructed by placing a hydraulic fill of sand and gravel dredged from the river bottom. These fills aggregate approximately 2¾ miles in length and required the movement of about 3,500,000 cubic yards of material in their construction.

The quantity of fill material placed was considerably greater than was estimated originally to be necessary. Observations were made on the representative fill at Four Mile Run, to determine the factors responsible for the use of the excess material.

CONDITIONS INDICATED DESIRABILITY OF A POROUS FILL MATERIAL

The general location of the improvement and the original soil profile on the center line of the road are shown in figure 1. The profile consists of a layer of soft muck 10 to about 30 feet thick resting on a firm sand foundation and is covered with about 3½ feet of water at high tide.

The original design for the fill is shown in figure 2. A sandy gravel was to be used to form a fill 100 feet wide at the top, at an elevation about 10 feet above mean sea level and sloping to the surface of the muck at the rate of 4 to 1.

Porous fill material was specified for several reasons: (1) It furnishes high stability throughout the fill, and the fill is not jeopardized even by total saturation of its base; and (2) when the muck underlayer is compressed by the superimposed fill water can escape freely through the porous fill and settlement of the muck takes place quickly.

Quick fill settlement was desired primarily because of the necessity for constructing the pavement on the fills as soon as possible. However, the expedited settlement of the fills has a deeper and more important significance in regard to fill stability.

As the muck layer consolidates, its resistance to lateral flow or sliding increases. The more quickly the consolidation of the under soil occurs, the more rapidly does the factor of safety against failure of the fill due to sliding of the under soil increase. Danger of fill failure due to lateral flow of the under soil diminishes rapidly after the construction of fills of porous materials has been completed. There is not the same assurance of increased stability with fills of relatively impermeable materials. The consolidation of the compressible layer proceeds only as the contained water is squeezed out of the voids. If the water is removed as fast as it is released, the compression of the muck layer will take place in a more rapid and more uniform manner than it would if the water could not escape. With the passing of time fills comprised of porous material resting on a much less stable layer may support abrupt increases of load without danger of failure of the under soil by sliding. In contrast, the safe load capacity of fills of impermeable materials under the same conditions may not increase with age in the same manner.

When a fill is constructed by the hydraulic method, large quantities of water are discharged with the fill material and the material must be such as to drain rapidly, otherwise it will require many years to become stable enough to support construction equipment. This was clearly illustrated while paving on Columbia Island on the same project. This island was made by pumping unselected material from the Potomac River during the process of channel deepening.[1] The fill had been built over 5 years before, but there were many places which would not support the weight of a man until artificial drainage was provided. No difficulties were encountered when paving over the fill described in this report, even though the pavement was placed within 2 years of the completion of the fill.

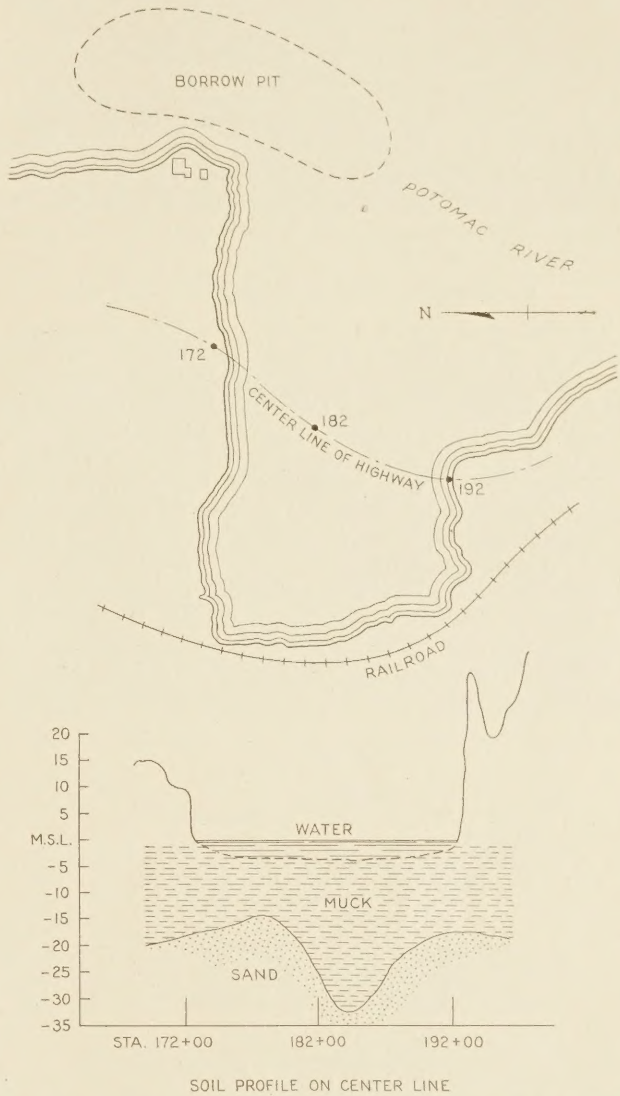

FIGURE 1.—PLAN AND SOIL PROFILE AT FOUR MILE RUN.

Borings made to locate a porous material for use in the fill disclosed a supply of sand and gravel within the range of dredging equipment at the location shown in

[1] Public Roads, vol. 13, no. 4, June 1932, p. 57.

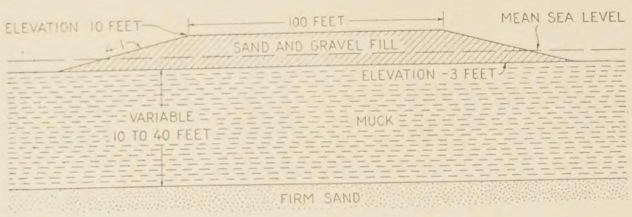

Figure 2.—Fill Section as Designed and Character of Undersoil.

figure 1. This material was graded from fine sand to gravel 6 inches in diameter or larger and had an average loose weight of 101 pounds per cubic foot and a rodded weight of 110 pounds per cubic foot. Prospecting for a suitable fill material was done with a wash boring outfit mounted on a scow.

FILL CONSTRUCTED BY HYDRAULIC METHOD

The fill was constructed in two lifts as shown in figure 3, which shows also the progress of the work. The cross hatching shows the elevation through which the fill was raised by the two lifts but is not indicative of the quantity of material required in each operation. The total amount of material equals that for the cross hatched area plus that for the space between the bottom of the cross hatching and the broken line shown below. How this additional material was divided between the two lifts is not known.

The first lift was constructed by pumping material continuously into place to an elevation of about 5 feet through a single pipe line. The second lift was constructed in three sections as regards time of placement and two as regards the position of the pipe discharging fill material. From the north end of the fill to approximately station 186 the material for the second lift was pumped through two pipe lines spaced about 50 feet apart, one pipe discharging about 25 feet to the right and the other about 25 feet to the left of the center line as shown in figure 4. Figure 5 shows a view of the fill at this stage of the construction. The two pipe lines were moved close together at the center line, as shown in lower sketch of figure 4, for the construction of the remainder of the second lift.

During the construction of the second lift with the pipe lines separated 50 feet, sliding of large portions of the fill occurred at the two locations designated in figure 3. When the slides began cracks formed around the area adjacent to the discharge pipe and widened as the pumping continued. The fill settled vertically and displacement took place laterally. With the continuation of these phenomena mounds of soft river muck were forced up to a height of as much as 8 feet above the original river bottom thus disturbing the original muck layer for distances of several hundred feet from the center line. The pumping was continued until the lateral flow stopped and the fill reached the required height. Subsequently elevations were measured at regular time intervals in order to determine the rate at which settlement occurred.

Figure 6 shows a mound produced by the slides. A close-up of one of the mounds which shows the character of the displaced river bottom material after exposure to the atmosphere is shown in figure 7. Figure 8 shows the cross sections of the fill at four different locations as indicated by the borings.

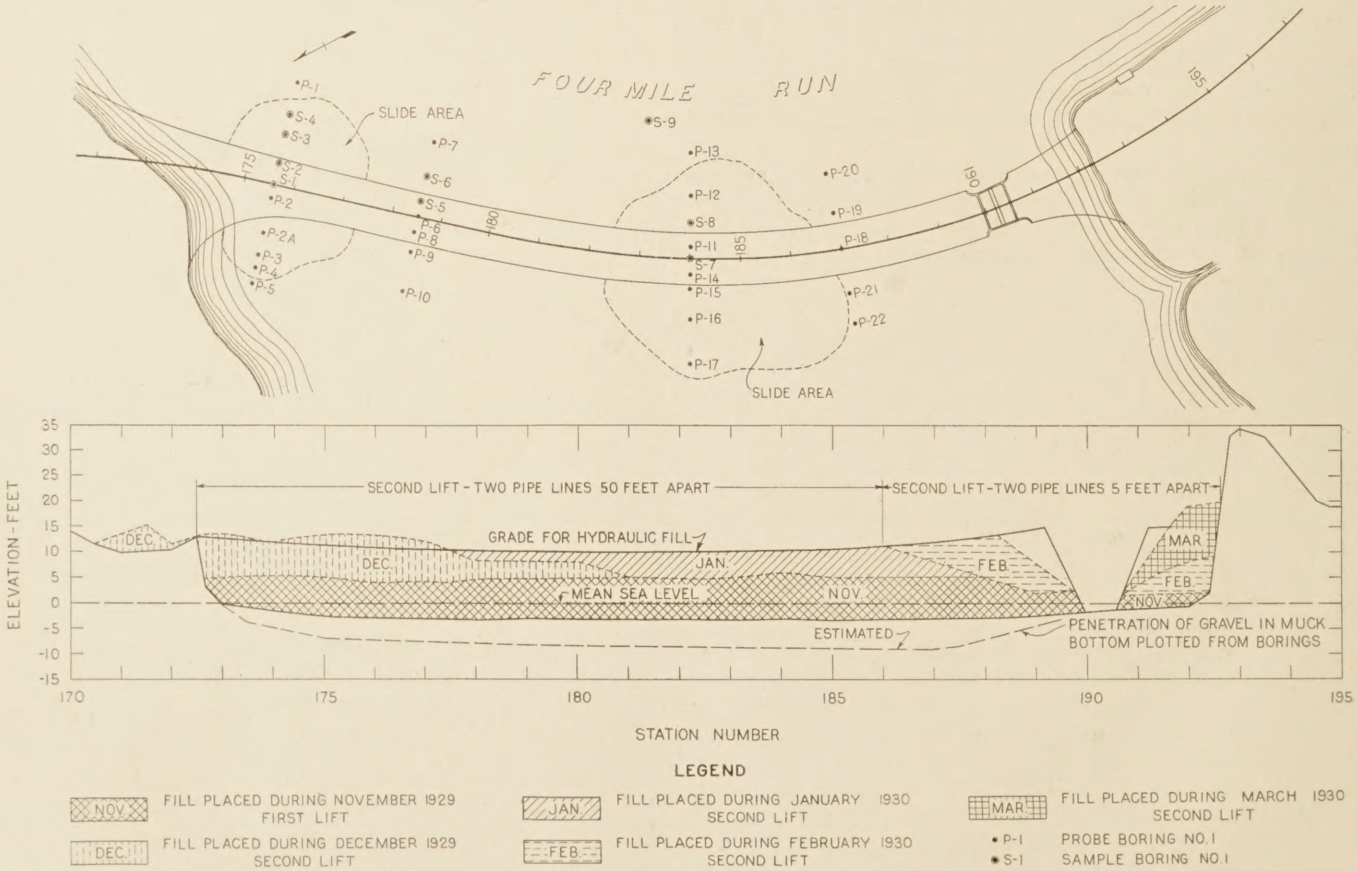

Figure 3.—Location of Borings and Progress in Constructing Hydraulic Fill at Four Mile Run, Mount Vernon Memorial Highway.

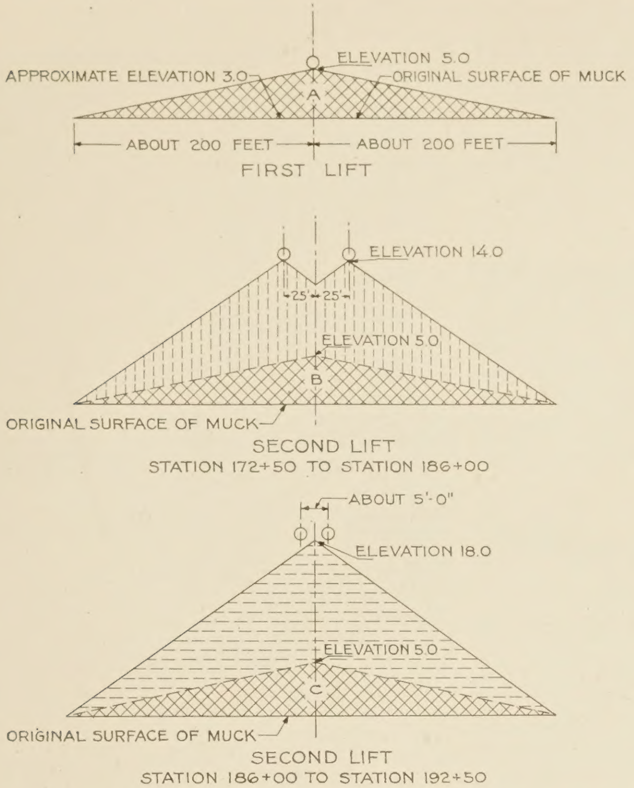

FIGURE 4.—LOCATION OF PIPE LINES AT VARIOUS STAGES OF CONSTRUCTION.

FIGURE 5.—DOUBLE PIPE LINE FOR PLACING SECOND LIFT.

METHODS OF DRILLING AND SAMPLING DESCRIBED

The location of the borings to determine the cross section of the fill and to obtain samples of the compressed muck layer are shown in figure 3. They were of two kinds, the "probe" borings designated as P–1, P–2, etc., and the "sample" borings designated as S–1, S–2, etc.

The "probe" borings were made by driving a 2-inch pipe through the fill material and soft under soil until firm foundation was reached. As the driving proceeded the material penetrated was washed up through the 2-inch pipe by water pumped through a 1-inch drill pipe. These borings indicated only the elevation of the bottom of the fill material and the bottom of the muck. A set-up of this boring equipment is illustrated in figure 9.

FIGURE 6.—LEFT SIDE OF FILL AT STATION 175+50 SHOWING AREA OF BULGED MATERIAL AND NATURE OF SLIDE. THE ROW OF STAKES IN THE FOREGROUND WAS ON THE SHOULDER LINE OF THE FILL BEFORE THE SLIDE OCCURRED.

FIGURE 7.—MATERIAL DISPLACED AND SHOVED UP BY SETTLEMENT OF FILL.

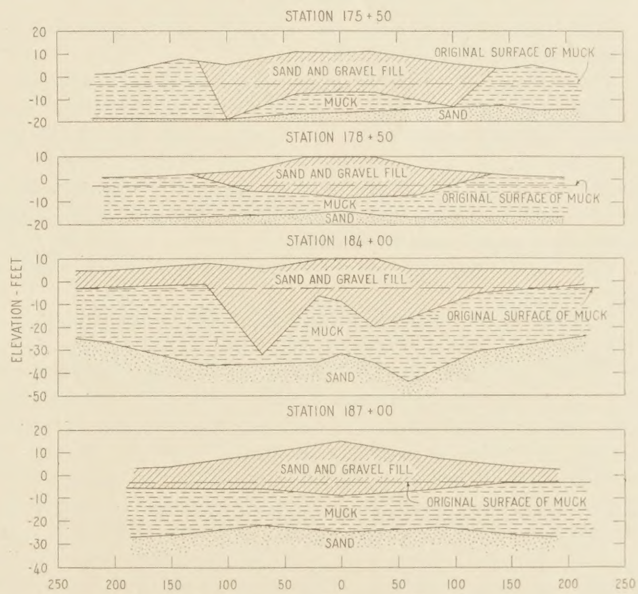

FIGURE 8.—CROSS SECTIONS OF FILL AS INDICATED BY BORINGS.

In making the "sample" borings a 4-inch pipe was used for the casing and a 2-inch pipe for drilling and washing in the same manner as for the "probe" borings. As the drilling proceeded cores of river-bottom material were obtained in an undisturbed state from different elevations in the muck layer.

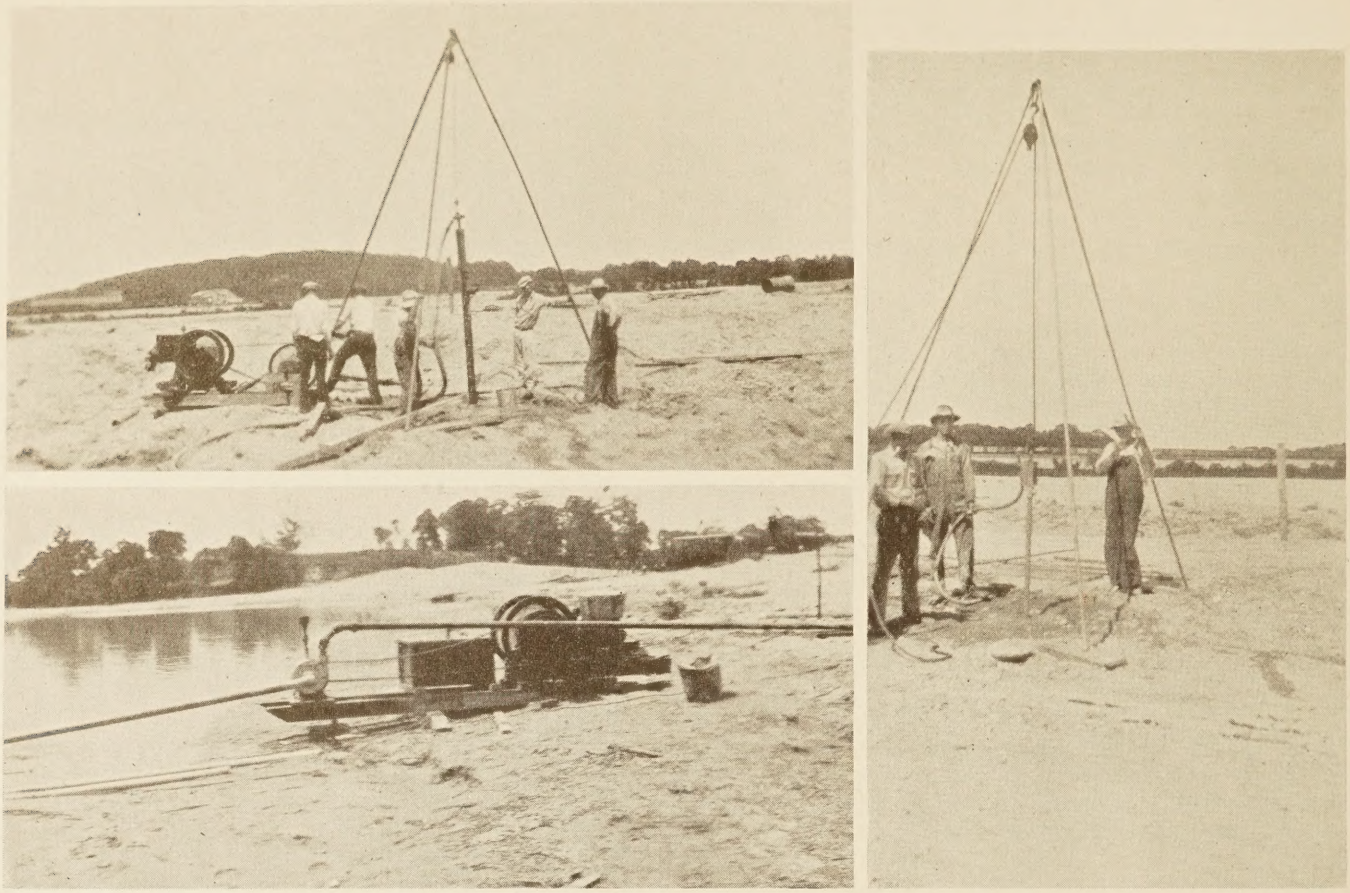

FIGURE 9.—UPPER LEFT—APPARATUS FOR TAKING UNDISTURBED CORES. LOWER LEFT AND RIGHT—APPARATUS FOR PROBE DRILLING.

The equipment used in driving and washing the casing is illustrated in figure 9.

The device for taking cores consisted of a section of 3-inch galvanized steel pipe about 15 inches long, sharpened at one end, and attached by a special coupling to a 1-inch pipe at the other. Inside the coupling was a check valve, specially designed as illustrated in figure 10 to prevent the loss of the core when the pipe was withdrawn.

The undisturbed samples were taken in the following manner:

The 4-inch casing was first thoroughly cleaned of all the material penetrated, after which the coring device was lowered through the casing and pushed down into the river-bottom material slowly and without twisting. The air and water entrapped by the soil entering the device escaped through the check valve. When the resistance to further penetration indicated that the device was filled with soil, it was turned through two revolutions and then pulled up slowly. The tube containing the soil core was next uncoupled, sealed with paraffin, and labeled. Figure 11 shows two cores sealed with paraffin and the sampling device for obtaining these cores.

At depths other than those at which the cores were taken, samples of the material washed up through the casing were obtained and placed in glass jars of 1-quart capacity.

The elevations from which both cores and samples were obtained in the "sample" borings are shown in figure 12, which shows also the character of the material penetrated as identified in the field. Sample boring S–9 (see fig. 3) was taken 264 feet from the center line at a point outside of the fill area and is intended to represent the muck in its natural uncompressed state. Even at this distance some disturbance of the muck occurred although no fill material was present.

The wash-boring samples were tested in the subgrade laboratory of the bureau at Arlington, Va. The cores were tested at the Massachusetts Institute of Technology as a part of a cooperative research on subgrade materials.[2]

ANALYSIS OF SAMPLES DISCLOSES CHARACTERISTICS OF THE RIVER-BOTTOM MATERIALS

The results of the standard laboratory tests [3] performed on the samples of soil collected in the wash boring are shown in table 1 and the results of tests [4] on the core materials are shown in table 2. Table 2 shows also the moisture content of the samples in the natural state.

The test results (table 1) show that the sand belongs to the good A–3 soil group and the mucks (tables 1 and 2) belong to the A–8 group. The wash-boring samples (table 1) had lost the finer portion of muck

[2] These tests were carried out under the supervision of Arthur Casagrande, formerly research assistant of the Bureau of Public Roads, stationed at the Massachusetts Institute of Technology. Mr. Casagrande aided in drawing up the specifications for drilling and sampling.

[3] These tests were carried out according to the Procedures for Testing Soils for the Determination of the Subgrade Soil Constants, Public Roads, vol. 12, no. 8, October 1931.

[4] The tests on the core material were performed on the materials before they had dried out. The procedures for making these tests are described in Research on the Atterberg Limits of Soils, Public Roads, vol. 13, no. 8, October 1932.

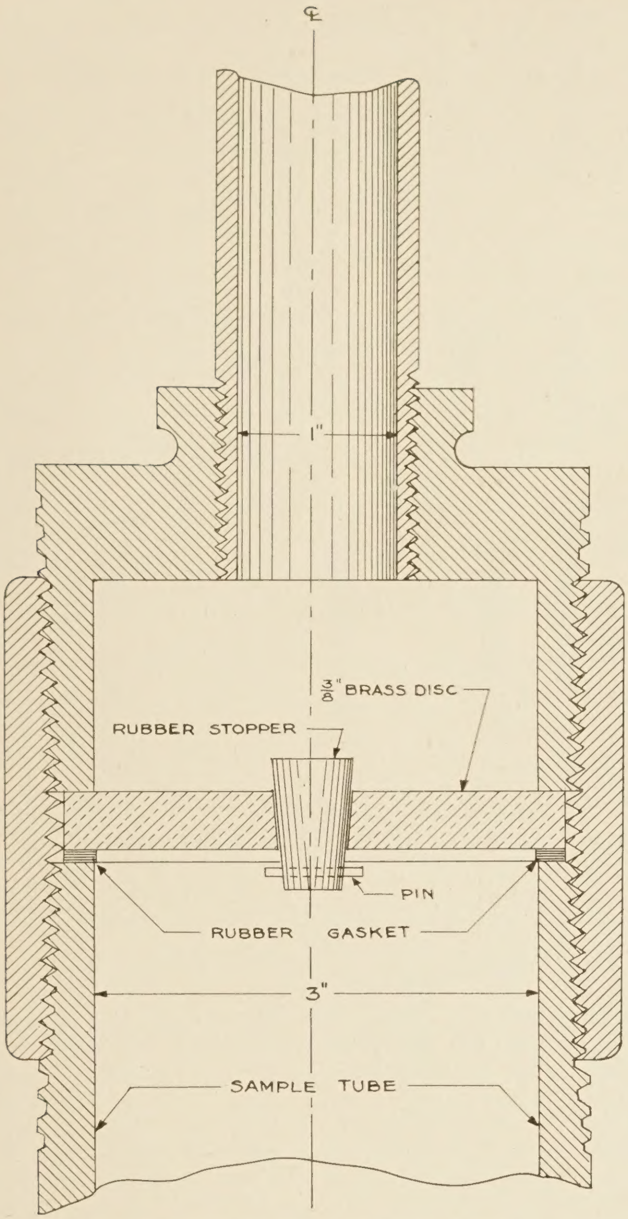

FIGURE 10.—DETAILS OF CHECK VALVE SAMPLING DEVICE.

due to the method of sampling. The samples as collected have lower liquid limits, lower plasticity indexes and higher shrinkage limits than the corresponding samples of muck (table 2).

The test results on the wash-boring samples (table 1) are more uniform than the results on core samples. There is indication that the variation in the results on cores is due to the presence of ultra fine material which was carried away in suspension in wash-boring or to some electrolytic property destroyed in taking the wash-boring samples.

Regardless of the cause of the difference, there remains the fact that the sand and muck samples of table 1 are similar to the materials used in building Columbia Island [5] which was constructed of river bottom material pumped from the Potomac River during the process of channel deepening. The wash-boring samples are representative of the material in fills

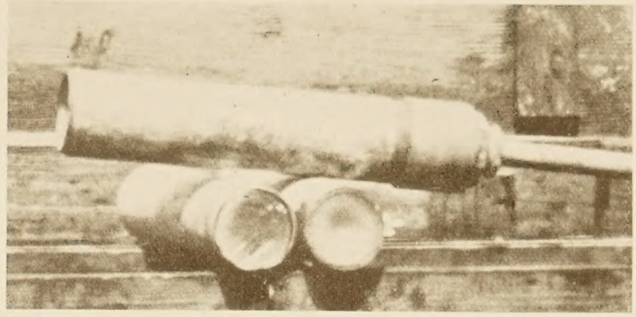

FIGURE 11.—SAMPLING DEVICE FOR OBTAINING UNDISTURBED CORES AND TWO SAMPLES SEALED WITH PARAFFIN.

constructed of muck placed by the hydraulic method and the core samples are representative of the under muck layers in place.

TABLE 1.—*Results of laboratory tests performed on samples obtained in wash boring. Sample locations are shown in fig. 12*

Boring no.	Sample no.	Mechanical analysis [1]						Physical characteristics of material passing no. 40 sieve					
		Particles larger than 2 millimeters	Particles smaller than 2 millimeters					Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Coarse sand, 2-0.25 millimeters	Fine sand, 0.25-0.05 millimeter	Silt, 0.05-0.005 millimeter	Clay, smaller than 0.005 millimeter	Colloids, smaller than 0.001 millimeter			Limit	Ratio	Centrifuge	Field
		Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*				
S-1	6	7	24	8	27	41	14	44	15	29	1.5	41	36
S-2	4	5	9	3	30	58	24	53	21	31	1.5	52	39
	5	(2)	(2)	(2)	(2)	(2)	(2)	118	0	66	.8	109	111
S-4	1	6	13	12	34	41	18	72	8	40	1.2	69	57
	2	1	66	24	3	7	3	19	0	----	----	7	21
S-5	7	6	5	11	52	32	9	64	5	40	1.3	58	54
S-7	11	1	7	4	33	56	25	53	24	33	1.5	54	42
	12	2	8	9	49	34	16	59	18	41	1.3	55	54
	13	1	9	23	40	28	14	52	17	40	1.3	46	49
	14	2	6	12	49	33	13	52	17	37	1.3	52	48
S-9	9	0	1	7	54	38	20	63	41	37	1.3	66	52
	10	0	0	4	47	49	24	62	22	37	1.3	[3] 69	54
(4)	3	0	1	5	59	35	16	55	10	37	1.3	66	51

[1] Percentages of material larger than 2 millimeters based on total sample, other percentages based on sample exclusive of this size. Percentage smaller than 0.005 millimeter includes percentage smaller than 0.001 millimeter.
[2] Organic matter.
[3] Tends to waterlog.
[4] Sample taken from surface of muck at left of station 175+50.

According to table 2 the mucks comprising the under soil have liquid limits varying from 62 to 132 and exist in place with moisture contents varying from 76 percent

TABLE 2.—*Results of laboratory tests performed on cores. Locations of cores are shown in fig. 12*

Boring no.	Core no.	Moisture content, natural state	Liquid limit	Plasticity index	Shrinkage limit
S-2	2	76	77	38	26
S-4	1	108	103	49	33
S-5	[1] 3	120	128	66	29
	[1] 4	107	131	64	34
S-6	5	125	132	69	32
S-7	[1] 8	64	89	48	27
	9	105	96	47	34
	10	116	112	61	32
S-9	6	79	62	30	29
	[1] 7	117	79	36	31

[1] These cores were very nonuniform and were not suitable for compression tests. Therefore, they are not considered in the analysis of the test data.

[5] Public Roads, vol. 13, no. 4, June 1932, p. 57.

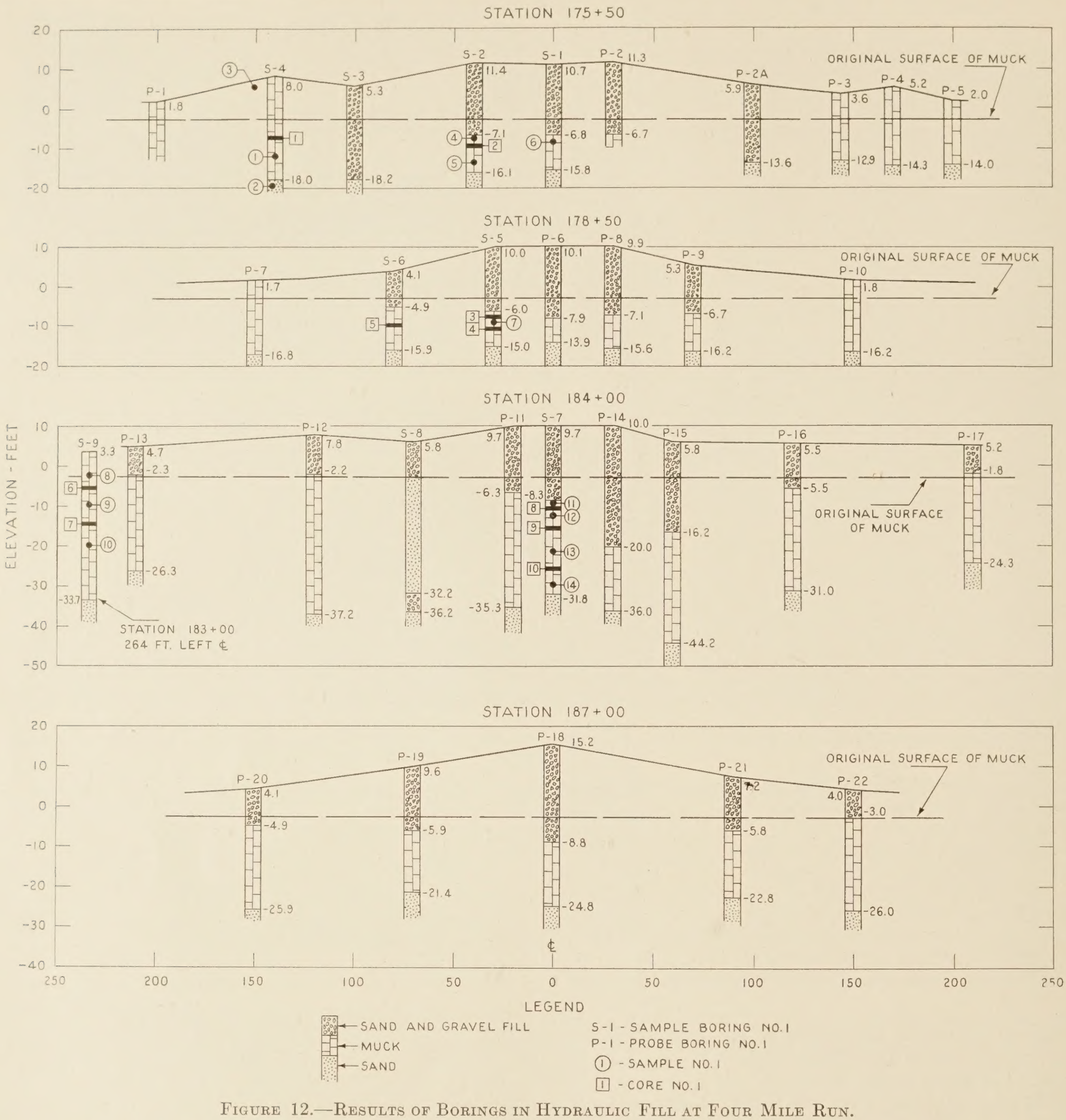

Figure 12.—Results of Borings in Hydraulic Fill at Four Mile Run.

to 125 percent of the weight of the dry soil. On the basis of relative volumes, for every part of solid soil material (with a specific gravity of 2.6 in this case) there are from 2 to 3¼ parts of water. The stable sand layer, in contrast, is not likely to contain more than 0.6 part of water by volume to each part of solid material. The significance of such differences in moisture content with respect to the relative stability of sands and mucks has been discussed previously in Public Roads.[6]

[6] Stabilization by drainage of muck and sand fill. Public Roads, vol. 13, no. 4, June 1932, pp. 57–60, inclusive.

SUBSURFACE MOVEMENTS OF FILL OCCURRING DURING CONSTRUCTION DISCLOSED BY BORINGS

The cross sections of the fill as shown by the borings plotted in figure 8 disclose the following:

1. Where no slides occurred, as at station 187, the fill material penetrated into the muck fairly uniformly throughout the cross section. The greatest penetration occurred in the center, under the greatest weight of fill, and the penetration diminished toward the sides as the fill feathered out. The contour of the lower surface of the fill at station 187 is, in a general way, similar to that of the upper surface but in a reversed

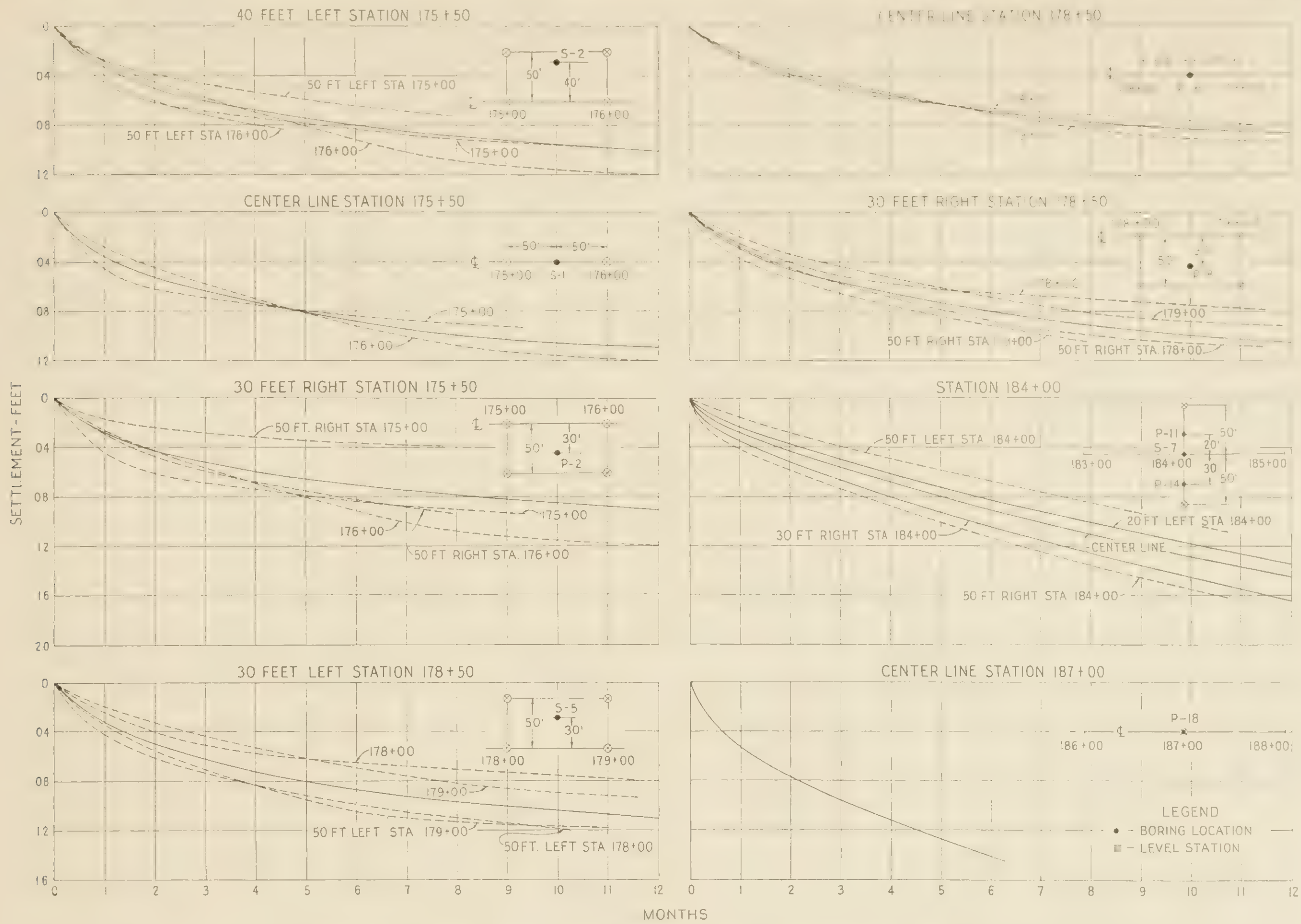

Figure 13.—Measured and Interpolated Settlements of Fill.

position. At station 178 + 50, where no abrupt slides were observed during construction, the penetration was very similar to that at station 187, but lateral flow is indicated by the position of muck surface 4 feet above its original position at the outer limits of the fill.

2. At station 175 + 50, where the muck varied from 11 to 13 feet deep, the fill penetrated to a depth of 4 feet in the portion between the shoulder lines of the roadway. On both sides, however, at a distance of about 100 feet from the center line, the fill displaced the entire depth of the muck and rested on the firm underlying sand. The displaced muck was pushed up in a mound 8 feet high at a distance of about 150 feet from the center line (fig. 6). This action trapped muck beneath the roadway. The greatest amount of upheaval occurred on the left side, where the sand layer is at a lower elevation.

3. The greatest amount of sliding and the most irregular penetration occurred at station 184 where the depth of muck ranged between 28 and 41 feet and the penetration of the fill varied from 5 feet at the center to 29 feet at 70 feet left of the center line and 17 feet at 30 feet right of the center line. Unlike the fill at station 175 + 50, at no point did the fill at station 184 penetrate the entire depth of the muck. However, at this place the layer of muck is considerably thicker than at station 175 + 50 and the surface of the sand is very irregular. To some extent the lower surface of the fill parallels the surface of the sand, especially at the right half of the section.

SETTLEMENT OF FILL DETERMINED FROM LEVEL READINGS

Settlement of the fill during the first 12 months of its existence was determined by levels, taken periodically, on stakes set in the fill on the center line and 50 feet to the right and left of the center line. The stakes were destroyed during the grading and shaping operations at the end of 12 months and no additional levels were taken until the pavement was laid about 20 months after the construction of the fill. Since that time levels have been taken periodically on the pavement. Since no readings were obtained during an interval of 8 months and there was a possibility of error in transferring elevations from the stakes to the pavement, these elevations are not discussed in this report.

Only 10 borings were located within 50 feet of the center line. The locations of the borings and of the level stations do not coincide. Level points were located at the even stations about 5 months before plans for boring were made. The borings were located at points in the fill which would give the most reliable information concerning the movements of the fill material, according to observations during construction.

Figure 13 shows graphically the settlement at drill holes and at level stations in the vicinity of the holes. The sketch with each graph shows the relative positions of the level stations. The broken lines represent the observed settlements as measured with the level while the solid lines show the settlements at the drill hole as

TABLE 3.—*Summary table of settlement, loads, muck thicknesses, and characteristics of muck layers*

Station	Location	Sample identification			Moisture content		Gravel thickness	Superimposed load [1]	Thickness of muck			Settlement at—		
		Boring no.	Core no.	Sample no.	In place	Liquid limit			Original	Time of borings	Difference	1 month	6 months	12 months
					Percent	*Percent*	*Feet*	*Kg per sq cm*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
				4		53								
175+50	40 feet left	S-2	2		69–76	77	18.5	0.8496	13.1	9.0	4.1	0.34	0.80	1.00
				5		[2] 118								
	Center	S-1		6		44	17.5	.8020	12.8	9.0	3.8	.35	.88	1.08
	30 feet right	P-2					18.0	.8308	12.1	8.4	3.7	.28	.70	.90
			3		120	128								
178+50	30 feet left	S-5		7		64	16.0	.7376	12.0	9.0	3.0	.32	.87	1.10
			4		107	131								
	Center	P-6					18.0	.8065	10.9	6.0	4.9	.23	.68	.85
	30 feet right	P-8					17.0	.7690	12.6	8.5	4.1	.29	.80	1.06
184+00	20 feet left	P-11					16.0	.7315	32.3	29.0	3.3	.23	.82	1.35
				11		53								
				12		59								
			9		105	96								
	Center	S-7		13		52	18.0	.7984	28.8	23.5	5.3	.28	.91	1.45
			10		116	112								
				14		52								
	30-feet right	P-14					30.0	1.2058	33.0	16.0	17.0	.35	1.05	1.64
187+00	Center	P-18					24.0	1.1105	21.8	16.0	5.8	.52	1.40	

[1] Computed from gravel thickness as disclosed by borings made about 6 months after completion of fill.
[2] All organic matter.

interpolated from the measured settlements at the level stations.

The level points on the center line at stations 184 and 187 coincide with the boring locations. Therefore, for station 187 the solid line represents the actual measured settlement. At station 184 the center-line stake was destroyed soon after construction of the fill and the settlement at this point was interpolated.

FILL SETTLEMENT AND LATERAL FLOW DISCUSSED

Table 3 presents the settlement at the 10 boring locations at 1, 6, and 12 months after construction of the fill, together with the fill load, thickness of muck, and information on the character of the muck soil. The data for this table were taken from tables 1 and 2 and figures 12 and 13. The superimposed loads were computed on the basis of full hydrostatic uplift effective below zero elevation.

The difference between the thickness of the muck layer prior to construction and that at the time of the borings represents the combined effect of lateral flow and 6 months compression under load. This difference varies from 3 feet to 17 feet (table 3). However, the total measured settlement for the 6-month period preceding the boring operations ranges from a minimum of 0.7 foot to a maximum of 1.4 feet. Therefore, the reduction in thickness of the muck layer was due largely to the lateral flow occurring during construction.

The variables apt to influence lateral flow are character of soil, soil profile, load, original thickness of muck layer, and construction methods. The construction procedure at station 175+50 was the same as that at station 184, and the behavior of the fill at these stations may be compared with respect to the other variables. A study of table 3 and figure 8 discloses no clear cut relationship existing between the amount of lateral flow and the above factors. However, it is true in a general way that, at station 184, where the lateral flow was of greater intensity, the natural moisture contents and liquid limits of the muck layers, the variations in the soil profile, and the original thickness of the muck layers were greater in amount than at station 175+50. On the other hand, considering station 184 by itself, these general relations do not explain the variations which have occurred.

The loads shown in table 3 have no significance as to cause of lateral flow since the variation in load is the result of lateral flow. It will be noted that the heaviest loads in any section occur at the points where the abrupt slides took place. No abrupt slides occurred at stations 178+50 and 187 where the second lift was placed in two layers with a period of time intervening between layers. This indicates that the rate of application of the load is more important in affecting fill movements than is the total load as disclosed by the borings.

The factors which are apt to influence the amount and rate of fill settlement are characteristics of the muck layer, thickness of the muck layer, and the load superimposed on the muck layer. In this case the thickness of the muck layer at the time of the borings seems the logical value to use since all the lateral flow took place during construction or continued only a brief time after construction.

From table 3 and figure 13 it will be observed that there is very little difference in the amount and rate of settlement at stations 175+50 and 178+50 although the moisture contents and liquid limits of cores no. 3 and no. 4 indicate a much more unstable soil at station 178+50 than at station 175+50 which is represented by core no. 2. It appears that a large variation in soil properties as disclosed by these tests may have little influence on settlement.

Neither the amount of superimposed load nor the thickness of the muck layer shows any relation to the amount and rate of settlement. The loads at stations 175+50 and 178+50 are generally higher than on the center line and 20 feet left of center line at station 184 but the settlement after 12 months at the latter station is greater in amount. However, at the latter station the layer of compressible muck is more than twice as thick. At all points except station 187 the settlements for the first month are practically the same in amount. Furthermore, at 6 months the settlement on the center line at station 187 is much greater than that at 30 feet right of station 184, while the thicknesses of the muck layers are equal and the loads differ only by 0.1 kilogram per square centimeter.

The foregoing discussion shows that no consistent relationships exist between the amount and rate of settlement as measured and any one of the individual influencing factors of load, thickness of muck layer and characteristics of muck layer. These variables seem to be interdependent in influencing settlement. The compression test discloses, quantitatively, the combined effect of the several variables and it was decided to investigate the results of such tests.

COMPRESSION TESTS DISCUSSED IN A SUPPLEMENTARY REPORT

Extensive original research had to be performed in connection with analyses of the compression test data as such, before it was possible to explain the behavior of the fill at Four Mile Run. It is felt that a complete description of this work should be presented in a separate report. This supplementary report shows conclusively that the differences in moisture content of the muck undersoil (table 2) and the difference in settlement characteristics (fig. 13) are not the result of haphazard accident but, instead, are entirely in accordance with well-defined physical laws which control the consolidation of muck deposits. It is demonstrated that proper analysis of the compression test data furnishes excellent means for estimating the amount and speed of settlement of loaded soil layers.

SUMMARY

The effect of the principal construction and soil variables upon the fill movements just discussed may be briefly summarized as follows:

1. No consistent relation was found to exist between the arrangement of the pipe outlets and the extent of lateral flow. It is true that no slides occurred during the placing of the entire first lift or the placing of the second lift at station 187 when the material was discharged along the center line. In contrast, sliding did occur during the placing of the second lift at stations 175 + 50 and 184 when the double pipe line arranged as in figure 4 was used. But slides occurred also during the construction of two other fills on this highway where the muck was over 40 feet deep, although a single pipe line discharging on the center line was used.

2. The slope of the fill material during construction seemed to be important. This is not surprising since theoretically it is possible to make the slope of fills so small that lateral flow of even the softest of undersoils may be prevented. In this case no appreciable lateral flow occurred during the placing of the first lift, which had very flat side slopes of 25 to 1. The second lift was placed with a much steeper slope of 10 to 1. It is possible that had the slope of 25 to 1 been used throughout the second lift the sliding which occurred in certain places would have been prevented.

It seems that the softer the muck, the deeper the muck and the more sloping the surface of the sand bed beneath, the more tendency there will be for the muck to displace laterally.

Use of gentle slopes and placement of fill in thin layers seem to furnish the best possibilities for reducing sliding. In case sliding starts, it seems that shutting down the pumps temporarily, if there is no support for extension of pipe lines for continuing the fill ahead, is the most logical procedure. Consolidation of muck under the fill already placed will then proceed and increase the resistance of the muck to further sliding while the force productive of sliding remains constant.

3. The depth of the muck and the slope of the firm sand bed beneath seemed to exert some influence. The most extensive lateral flow occurred at station 184 where the greatest depth of muck and the most irregular surface of the underlying sand were found. However, no sliding occurred at station 187 where the layer of muck was originally over 10 feet thicker than at station 175 + 50 where sliding did occur.

4. The manner in which the second lift was placed may account for the slides which occurred at stations 175 + 50 and 184 and the absence of slides at stations 178 + 50 and 187.

At stations 175 + 50 and 184 where the entire second lift was constructed in a continuous operation the entire additional fill load was imposed upon the soft undersoil before adjustment took place.

At stations 178 + 50 and 187 where the second lift was constructed in two layers, the muck had an opportunity to consolidate under the load of the first layer before the second layer was imposed.

It was observed, during the construction of another fill in a location where the mud had a minimum depth of 40 feet, that when sections of the fill slid out laterally in a manner similar to that described in this report, the movement stopped soon after the pumps were shut off. When pumping was resumed after being shut down for about 2 days, a considerable amount of fill could be placed before more sliding occurred.

42174—34——2

FROST HEAVE IN HIGHWAYS AND ITS PREVENTION

Reported by Henry Aaron, Assistant Highway Engineer, Division of Tests, Bureau of Public Roads

FROST ACTION affects road surfaces differently, depending on the type of surfacing. Rigid pavements are heaved and crack in excessive amounts but usually settle back in place after the frost leaves the ground. Successive heaves are often sufficient to break up long slabs of the thinner pavements to such an extent that replacement is required. In semirigid pavements the cracking and breakage due to one period of frost may require considerable patching, if not replacement. Low-type wearing courses, such as gravel, are often entirely lost after one season, the gravel mixing with the soft undersoil without furnishing stability. Figures 1 and 2 are examples of damage resulting from frost action.

When heave occurs abruptly there results a hazard to traffic. It is not uncommon for short sections of pavement to heave 10 or 12 inches above the adjacent surface, thus becoming an obstruction capable of causing fast-moving vehicles to leave the road. Examples of such heaves are shown in figure 3.

Rigid and semirigid pavements may adjust themselves during thaws so that traffic is carried without resorting to special construction. Frost heaved sections of gravel roads are apt to become soupy mud, impassable to any type of traffic during thaws. Figure 4 is an example of such a condition. These short impassable sections are sufficient to render the entire highway unsuitable for traffic for a period of several weeks during the spring and thus tend to isolate the towns located along the highway. Those who dare to venture forth usually require assistance in getting out of the mud before they get very far.

Therefore, aside from the large maintenance costs which arise from frost failures there is a heavy economic loss inflicted on the users of these highways.

PURPOSE AND SCOPE OF REPORT

Prevention of the harmful effects of frost action has been a major problem in constructing and maintaining roads in climates where severe freezing occurs. The heaving produced in various types of soils which form the foundation for road surfaces has been studied extensively, both in the laboratory and in the field. The results of these studies and the theories developed have been discussed in numerous published reports. Many types of remedial treatments have been installed in locations subject to frost heave by different engineers.

To correlate the various methods employed by different engineers, the Bureau of Public Roads with the cooperation of the highway departments of Minnesota, Wisconsin, and Michigan, conducted a survey during the period from 1928 to 1933 to determine for specific cases (1) method of treatment used (2) its performance under service conditions, and (3) the physical properties of the subgrade soils in the location and their arrangement in the soil profile. This report is the result of observations by the author, supplemented by information from the engineers and maintenance superintendents in the respective States. The following aided in planning and conducting the survey and furnished most of the photographs used in this report: A. Seifert and S. L. Taylor, district 4, United States Bureau of Public Roads; F. C. Lang, L. L. Allen, and C. K. Preus of the Minnesota Department of Highways; A. C. White, Mower County, Minn.; E. B. Tourtellot, Wisconsin Highway Commission; H. F. Janda, University of Wisconsin; and A. C. Benkelman, Michigan State Highway Department.

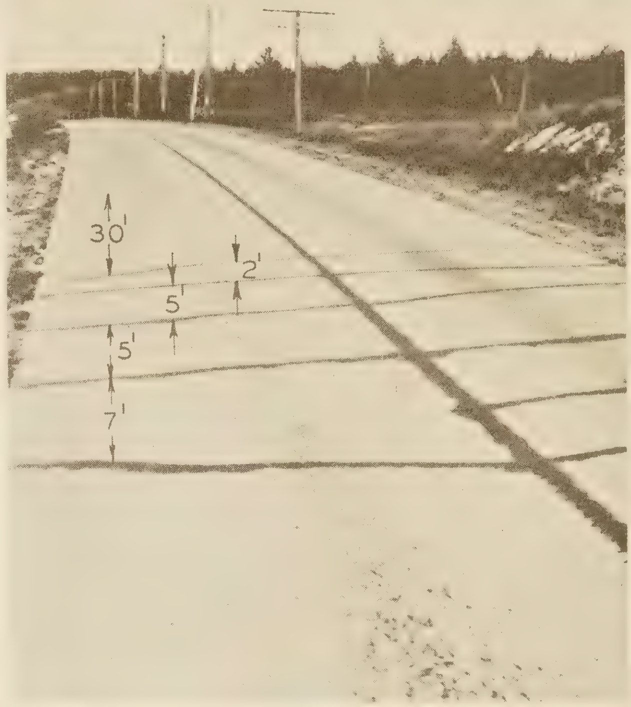

Figure 1.—Cracking in Concrete Pavement Due to Frost Heave.

No attempt is made in this report to explain the physics or mechanics of frost heaving. It is intended, primarily, as an exposition of methods used to prevent harmful frost action and the results of these methods. In order to interpret properly the results of the various methods, it is important to know the characteristics of the different subgrade soils which make up the soil profile in the location where they were used. Before considering the results of the preventive measures, it is advisable to discuss the different soil profiles found in this survey and their influence on heaving.

THE SOIL PROFILE INFLUENCES HEAVING

In many sections of the country a road surface rarely rests on a uniform subgrade for any great distance. This is due to two reasons:

(1) The grade line frequently intersects a number of layers of the soil profile depending on the depth of cuts, and the several layers, often differing in their physical characteristics, will each in turn form the subgrade of the road.

(2) An individual soil layer may possess such variable characteristics as—

A Bituminous Surface Damaged by Frost Action.

Effect of Heaving on a Gravel Road.

Heaved Gravel Road at Beginning of Thawing Period. Planks Are Necessary to Prevent Miring.

Figure 2.—Examples of Typical Heaving.

(*a*) Pockets of soil material differing greatly in properties from those of the remainder of the layer.

(*b*) Stratifications within the layer.

(*c*) Depressions in the profile of the layer boundaries which act as reservoirs for the collection of water in excessive amounts.

(*d*) Variation in the ground-water elevation due to the topography of the adjacent area.

Differential heaving of pavements productive of dangerous traffic hazards is generally due to variations in the soil profile. Figure 5 shows the different types of soil profiles in which the preventive measures described in this report were installed. Profiles *B*, *D*, *F*, *G*, *I*, *J*, and *O* were furnished by W. I. Watkins of the United States Bureau of Chemistry and Soils.

Figure 5–*A* illustrates the type of soil profile productive of the heaving shown in the top picture of figure 3. The pockets of silt and silty clay (group A–4 soil) varying in shape and depth occur within a deposit of porous sandy soils (group A–3 soil). The heaving in the silt pocket is excessive while that in the surrounding sandy soils is negligible. Considerable heave occurs

Abrupt and Hazardous Frost Heave of a Concrete Pavement.

A Series of Frost Heaves of a Gravel Surface.

Prominent Heave in a Gravel Surface.

FIGURE 3.—EXAMPLES OF TYPICAL HEAVING.

also in the pocket of silty clay. However, it is covered by an appreciable layer of sand (about 18 inches) which reduces the effect on the surface.

Figure 5–*B* shows a typical soil profile in the loessial area of Minnesota and Wisconsin. The frost heave is confined to that portion of the road resting on the unweathered or slightly weathered structureless silt. The weathered upper layers of the profile are granular in structure and apparently do not suffer detrimental frost heave.

According to the results of laboratory tests performed on representative samples of the various layers of the profile (fig. 5–*B*) the weathered silt loams and the structureless silt possess physical properties common to the group A–4 soils, while the underlying clay possesses the physical characteristics of the group A–6 soils.

The structureless loessial soil is a silt or silt loam containing a high percentage of very fine sand, has a high water-holding capacity, is unstable when wet, and

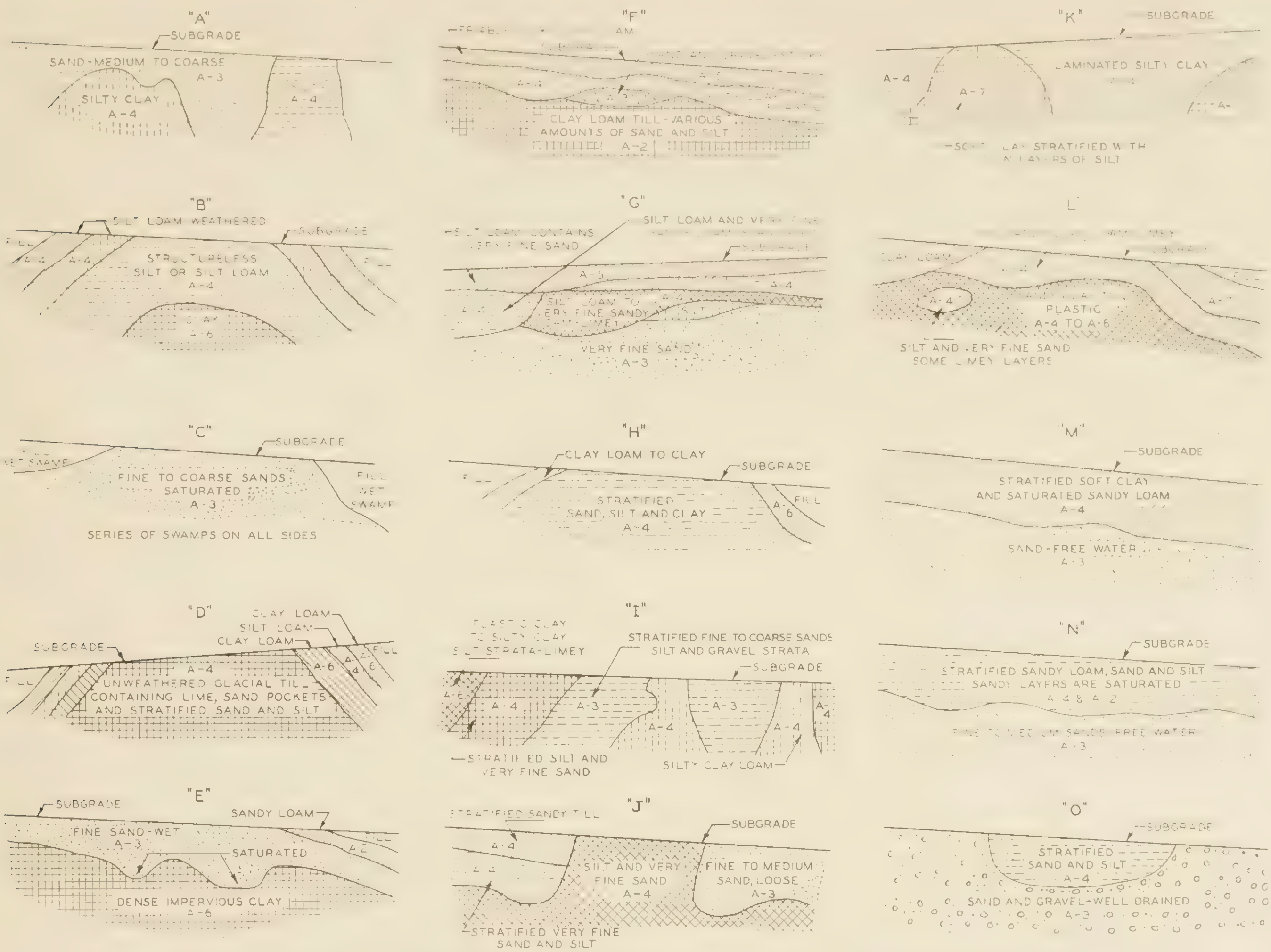

Figure 5.—Soil Profiles in Which Detrimental Frost Heave has been Observed.

Figure 4.—Result of Thaw of a Frost-Heaved Gravel

possesses capillary properties in a high degree. Resting on a relatively impervious clay which retards percolation and forms a water table at its base, a condition is created favorable to strong functioning of the high capillary properties of the structureless silt layer. The clay may be at a considerable depth below the road surface and not necessarily in as close proximity to the surface as is indicated by figure 5–*B*. The bottom picture of figure 3 is characteristic of the type of heaving produced under the above conditions.

An example of detrimental frost heave in a sandy soil is illustrated in figure 5–*C*. The sand which varies from fine to coarse in texture and possesses physical properties indicative of the group A–3 soils is maintained in a saturated condition by a water table close to the surface. This is due largely to the topographic features of the adjoining terrain. The road cuts through small knobs or ridges, interspersed with bogs located both at higher and at lower elevations than the roadway. The middle picture of figure 3 is an example of the heaving produced in such a case.

A soil profile in extremely variable glacial materials is illustrated in figure 5–*D*. The relation between frost heave and the various layers of the soil profile is similar to that described for figure 5–*B*. In this case the heaving occurs in an unweathered glacial till containing lime, sand pockets, and stratified sand and silt. Water is transmitted to the roadway by the capillarity of the glacial till and also by the stratified sand and silt. Considerable amounts of water collect in the sand pockets.

The unweathered glacial till layer is essentially a group A–4 soil. However, it includes strata and small pockets of sand which possess the physical properties of the group A–3 soil. Such variations within a soil layer are especially productive of differential frost heaving.

Figure 5–*E* is an illustration of a soil profile in which heaving occurs as a result of depressions in the surface of an impervious clay. The dense clay restricts percolation of water and maintains the overlying sand in a wet condition while the sags in the surface of the clay layer act as reservoirs for the collection of water in excessive amounts. This condition is productive of the type of heaving shown in the top picture of figure 3 and attributed also to condition 5–*A*.

Figures 5–*F* and 5–*G* are examples of soil profiles in which the heaving results as shown in the bottom picture of figure 2. The profiles consist of a surface layer of more or less unstable A–5 soil underlain by silts and very fine sands (group A–4 soils) which occur in pockets and layers of varying thicknesses adjacent to water-carrying soil layers.

Figures 5–*H*, *I*, and *J* illustrate profiles which produce the greatest and most dangerous heaving. Stratified silt and very fine sand (group A–4 soil) invariably heave in such amounts as to rupture any type of superimposed road surface. Very dangerous differential heaving is produced when soil layers subject to detrimental heave in varying amounts are arranged in pockets as in figures 5–*I* and 5–*J*.

A different type of heaving condition is shown in figure 5–*K*. The laminated silty clay possesses physical characteristics of the group A–4 soils and performs in the same manner as the A–4 materials previously described. The group A–7 soil, however, is not generally subject to heaving. In this profile it includes very thin layers of silt which assist in keeping the clay very wet and soft.

The majority of the clay loam soils similar to those shown in figure 5–*L* are subject to detrimental frost heave only when wet and poorly drained but considerable heaving has been observed in fine sandy clay loam (group A–4 soils) containing appreciable amounts of disseminated lime. Where the road surface rests on lime-free clay loam of the group A–7 soils no serious damage has resulted.

The detrimental results of lime accumulations in a soil layer have been observed also in soil profiles where a limey clay loam of the group A–7 soils was found. Two adjacent clay loam soil layers in the same road cut were found to possess similar physical properties but the one containing lime heaved excessively while the layer free of lime did not heave enough to damage the road surface.

The soil profiles illustrated in figures 5–*M* and 5–*N* produce heaving similar to that of the stratified soils previously described and need no further discussion.

Differential heaving is certain where there is the extreme variation in soils as shown in figure 5–*O*. The pocket of soil subject to frost heaving is composed of strata of water-bearing sand and strata of silt which absorb water readily and resist all attempts at drainage. The sand and gravel (A–3 soil) surrounding this pocket is well drained and does not permit the accumulation of water, and no heaving occurs in the sand and gravel.

PREVENTIVE MEASURES DESCRIBED

Various methods used by different engineers in efforts to prevent frost heaving are shown in figure 6. Considering only the locations inspected in this survey, figure 6–*A* shows a method which has been used since 1921; figures 6–*B*, *C*, and *D* since 1928; figures 6–*E*, *F*, *G*, *H*, *I* and *J* since 1929; and figures 6–*K* to 6–*P* inclusive since about 1920.

The methods of excavating heaving soil and substituting a non-heaving material are illustrated in figures 6–*A* to 6–*I* inclusive.

Figure 6–J shows excavation of the heaving soil, treating the bottom of the excavation with bituminous material to cut off capillary action and replacing the excavated material.

Drainage channels to intercept and direct water away from the roadbed are shown in figures 6–*K* to 6–*P* inclusive.

Generally no more than 2 years are required for the surface condition to reflect the effectiveness of the preventive method used. This is especially true of methods which do not serve the requirements of the location. In such cases heaving has been observed during the first winter after installation. Therefore, the results of methods which have been in service since 1929 are considered fully as significant as those which have been in service since 1920.

RESULTS OF PREVENTIVE MEASURES DISCUSSED

The performance of these preventive measures employed in connection with different types of road surfaces and the associated local soil condition is described in table 1.

The narrow center trench types *A*, *B*, and *C*, serve to eliminate only that heaving which might occur directly over the area of the excavation. The soil on the sides heaves sufficiently to crack concrete and bituminous pavements. The damage to gravel surfaces caused by the frost uplift is ironed out by maintenance after the frost leaves the ground so that there is no visible aftermath of the winter condition. These methods reduce the amount of break-up during the spring thaws and allow passage of vehicles. Compared with the results where no precautions are taken (fig. 4) the benefits of this method are considerable.

Treatment for the full width of the traveled area as illustrated by type *D* has proven advantageous. There was no noticeable difference between the riding qualities of the surface laid over this type of treatment and adjacent sections where heaving had always been absent. The entire roadway remained smooth and firm during the spring thaw.

The V-type trench (type *E*, table 1) is very popular because it can be excavated with a blade grader, which materially reduces the cost of excavation. In most cases surfaces laid over subgrades so treated carry traffic in a satisfactory manner. However, the heaving at the sides of the trench 12 feet wide in soil profiles *D* and *J* and the general heaving in the trench 26 feet wide in soil profile *I* has, in some cases, developed to the point where driving is safe only at very low speeds. The surfacing in the heaved area is badly broken.

Measures *F*, *G*, *H*, and *I* have proved satisfactory and reliable methods of preventing detrimental frost heave of concrete pavements. Except under the extreme climatic conditions of the northern parts of Michigan and Wisconsin, heaving is negligible in treatments where the excavation and backfill is not less than 2 feet. Noticeable but uniform heave has been measured in treatments 1 foot deep.

The method of excavating the heaving material for the entire width of the roadway, treating the base of the excavation with a bituminous material and replacing the excavated material (type *J*, table 1) has not been quite as successful as the full width treatments described above. Gravel surfaces on bituminous treated soils

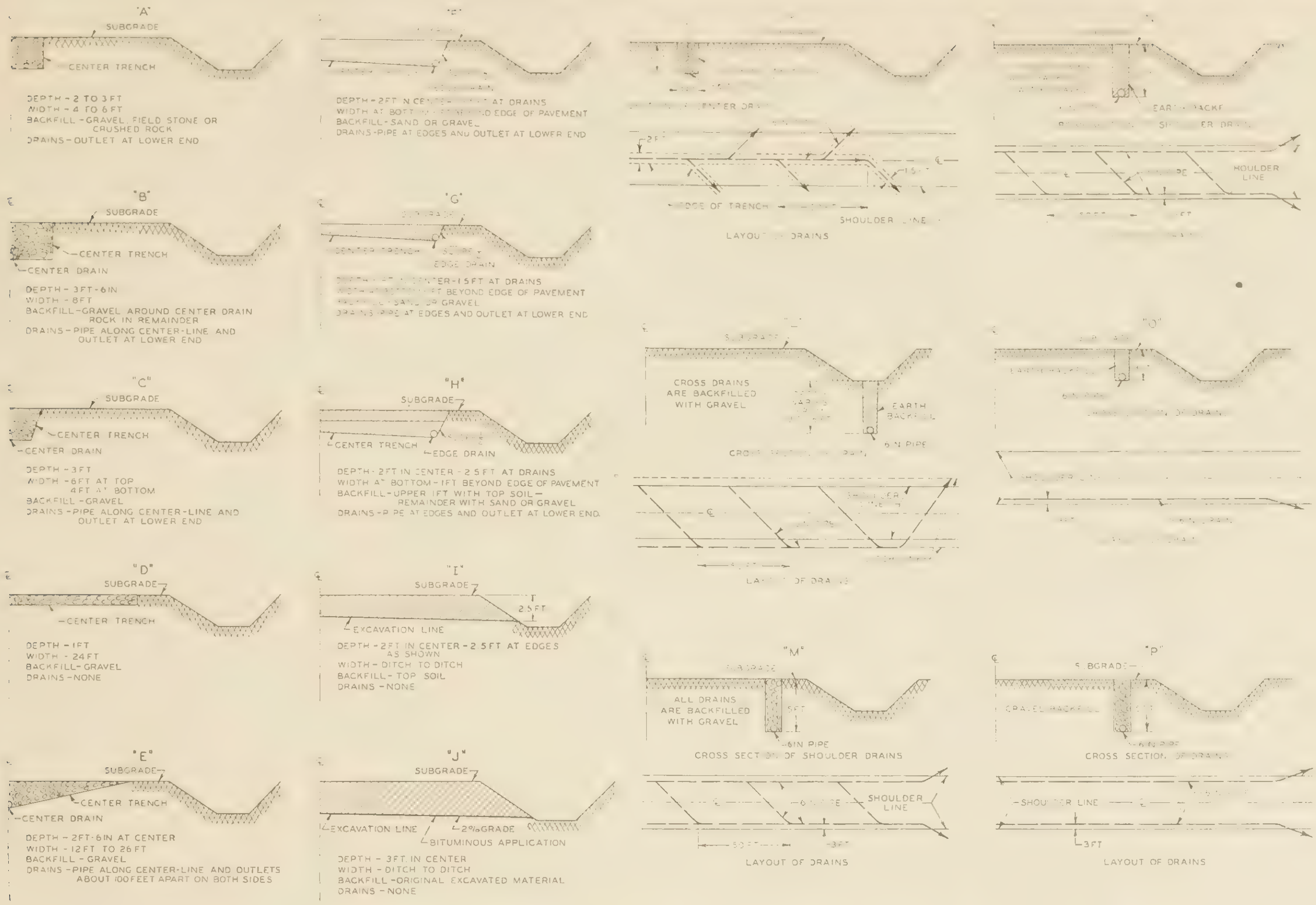

FIGURE 6.—SUBGRADE DESIGNS USED TO PREVENT FROST HEAVE.

have become uneven and rough and cut up during the spring thaw. The heaving of a concrete pavement is noticeable but fairly uniform. Cracking is more pronounced than in the locations where a selected back-fill material was substituted for the excavated frost heaving soil.

The various kinds of treatment should not end abruptly but should be tapered or feathered out. According to observations by Andrew Seifert, concrete pavements laid on gravel back-fill tapered from 10 to 25 feet cracked considerably and became noticeably rough. Fifty-foot tapers have been more satisfactory.

The result of type *K* subgrade treatment, is strikingly illustrated in figure 7 which shows how the gravel in the trench remains in place while the soil at the sides is forced up. The heaving at the sides of the narrow trenches previously described is similar in nature.

Table 1 shows that the drainage types *L* to *P*, inclusive, have been of practically no benefit in preventing heaving of soils. In most cases the drains have been placed at arbitrary depths of 3 to 6 feet without regard to the type of soil or the arrangement of the soil layers. Type *M*, when installed in water-bearing sandy soils (soil profiles *C* and *E*) in such a manner as to conform to the soil profile, served to intercept and carry away enough of the water to prevent detrimental heave. This same type, when used in the stratified silts, fine sands, and clays (soil profile *I*), was of no value whatever.

FIGURE 7.—HEAVING AT SIDES OF NARROW TRENCH FILLED WITH GRANULAR MATERIAL.

SOIL PROFILE MUST BE CONSIDERED IN DESIGN OF DRAINAGE SYSTEM

However convenient it would be to be able to establish definite and invariable spacing and depth requirements for drains, the hopelessness of thus standardizing drainage design becomes increasingly apparent as research in soils, especially for engineering purposes, progresses.

Failure to obtain the desired stability is definitely attributable in many instances to the attempt to apply arbitrary standards of spacing and depth in the placing of the drains without regard to prevailing conditions.

Table 1.—*Results of frost heave preventive measures*

Type of treatment [1]	Type of surfacing	Soil profile [2]	Results	Remarks
A	Gravel	F, D	Heaved on sides during winter; surface softens during thaw. Carries moderate amount of traffic satisfactorily.	County roads; center trench 3 by 3 feet.
	Bituminous surface treatment	I, G	Heaved excessively on sides; no serious break-up during thaw.	Center trench 3 by 3 feet.
	Concrete	H	Heaved sufficiently to cause considerable cracking	Center trench 6 by 2 feet.
B	Gravel	B	Heaved a small amount on sides; no break-up	Center trench 8 by 3.5 feet.
C	do	J, L	Heaved on sides; slight softening and rutting during thaw. Carries traffic satisfactorily.	Center trench 3 feet deep, 6 feet at top, 4 feet at bottom.
	Concrete	L	Heaved on sides removing crown; considerable cracking	Do.
D	Gravel	D	No heaving observed; smooth and firm; no break-up	Trench 24 by 1 foot.
E	Bituminous surface treatment	D, I, J	Heaved on sides in soil profiles D and J; surface broken but no rutting. Heaved badly in soil profile I; no rutting.	Treatment is 12 feet wide on soil profiles D and J and 26 feet wide on soil profile I, V-type trench.
F	Concrete	A to O, inclusive	Heave negligible, very small amount of cracking	Slightly wider than pavement.
G	do	L, M	No detrimental heaving; very small amount of cracking	Do.
H	do	D, M	Heave negligible; very small amount of cracking	Do.
I	do	B, D, L, M	Heave negligible; small amount of cracking	From ditch to ditch.
J	Gravel	D	Uneven and rough; small amount of rutting; carries traffic satisfactorily.	Do.
	Concrete	B	Slight amount of heaving; fairly uniform; small amount of cracking.	Do.
K	Bituminous surface treatment	N	Heaved on sides of trenches leaving a narrow depression along the center line and over cross trenches; reduced surface break-up.	Rough riding and dangerous at times.
L	Gravel	D	Reduced break-up in most cases so that traffic could move across, although difficult; some sections impassable.	Very unsatisfactory in this soil condition.
	Concrete	D	Considerable cracking typical of frost heave	
M	Gravel	I	Impassable during thaw	This method apparently of no value in this soil.
	Concrete	C, E	Heave negligible; small amount of cracking	Depth of drains and location of cross drains varied to conform with soil profile.
N	Gravel	D	Soft and rutted in spots during thaw; carries traffic with difficulty.	
O	do	D	Impassable during thaw	No benefit derived from this method.
P	Bituminous surface treatment	D	Heaved breaking surfacing; carried traffic without breaking through.	
	Concrete	K	Heaved and cracked considerably	One section heaved to such an extent that the pavement had to be replaced.

[1] Refers to designs in fig. 6.

[2] Refers to soil profiles in fig. 5.

There is only one practical procedure and that is to place drains at such distances apart and at such depths as the interception and removal of offending moisture requires. And this, it can readily be seen, becomes a local problem, the solution of which depends upon such factors as the source of the water to be removed, the character of the soils comprising the different layers and the arrangement of the layers in the soil profile.

To further complicate the problem, some soils very readily give up their contained water and thus may be easily stabilized by drainage; other soils exercise a high affinity for moisture and are not apt to be stabilized by drainage. The only profiles found in this survey in which drainage can be reasonably certain to prevent frost heave are shown in figures 5–*C* and 5–*E*.

Figure 8–*A* shows how drains should be placed in order to prevent frost heave in soil profiles of the type illustrated in figure 5–*E*. Frost heave occurring in drainable soils due to a permanent high water table (fig. 5–*C*) may be eliminated by the system of drains shown in figure 8–*B*. It should be borne in mind that these methods are applicable only where the soil is more or less porous and does not possess capillarity in an appreciable amount.

CONCLUSIONS

The following conclusions may be drawn from the foregoing discussion:

1. Center trenches reduced the heave of adjacent gravel road surfacing during freezing and increased stability during thaws to some extent. As a result the damage to the road surfacing which did occur could be repaired by ordinary blading operations. These benefits are not sufficient to justify recommending narrow center trenches as a standard preventive measure. However, where scarcity of gravel makes full width treatment prohibitive, center drains 3 to 6 feet wide on the less traveled roads, and not less than 8 feet wide, or a width of one traffic lane, on the more important highways, may serve as temporary expedients prior to the construction of more permanent surfaces.

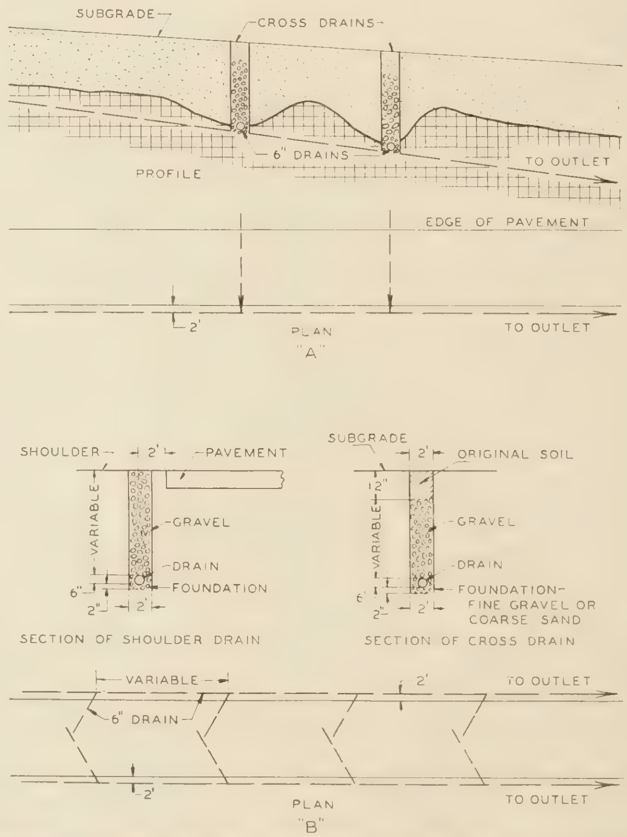

Figure 8.—Method of Preventing Frost Heave in Drainable Soils.

2. The measures adopted to prevent frost heaving of concrete road surfaces should be effective over an

(Continued on p. 25).

LABORATORY TESTS OF RESILIENT EXPANSION JOINT FILLERS

Reported by D. O. WOOLF, Associate Materials Engineer, and D. G. RUNNER, Assistant Materials Engineer, Division of Tests, United States Bureau of Public Roads

FOR MANY YEARS premolded fillers have been used in transverse expansion joints in concrete pavements to permit linear expansion of the concrete. The most widely used type has been a plastic compound consisting of bitumen and finely ground mineral matter between surfaces of felt paper. With linear expansion of the concrete, the bituminous filler is displaced and forced upward above the surface of the pavement. In some cases this extruded material is removed to preserve the smoothness of the pavement or is partly spread or worn away by traffic. In any event, subsequent contraction of the concrete results in the formation of an opening between the concrete and the joint filler since the bituminous filler, not being elastic, does not resume its original shape. Maintenance forces are expected to fill such openings with poured bituminous material so as to protect the subgrade from surface drainage. Due to the cost of this maintenance, efforts have been directed within the last few years toward the development of a resilient expansion joint which would absorb expansion of the concrete without appreciable extrusion of the filler and also expand upon subsequent contraction of the concrete, thus keeping the joint filled. A number of materials for filling expansion joints have been developed which manufacturers claim will produce these results.

TESTS COVERED ALL AVAILABLE TYPES OF JOINT FILLERS

In investigating resilient types of joint fillers, efforts were made to obtain samples representing the different materials used. Samples were obtained of sponge rubber, cane fiber, granulated cork, and compounds of asphalt and vulcanized rubber. A sample of the usual type of premolded asphaltic joint filler was also tested for comparative purposes. These fillers are described as follows:

Samples 1 and 2, brand A. Sponge-rubber fillers with nominal thicknesses of one half inch and 1 inch, respectively. The sponge rubber is placed between protective sheets of asphalt-treated felt. One edge of the filler is sealed with rubber to make a waterproof surface.

Sample 3, brand A. The usual type of preformed asphaltic filler with felt sides. Nominal thickness, 1 inch.

Samples 4 and 5, brand B. Sponge-rubber fillers with nominal thicknesses of one half inch and three quarter inch. The rubber appears to be of a much more open texture at the center than at either side of the filler. One edge of the one half inch filler and both edges of the three quarter inch filler are sealed with rubber. Both sides are protected with asphalt-coated felt.

Sample 6, brand C, is a sponge-rubber filler, one half inch thick. The rubber is of uniform texture, and is protected by felt sides. The edges are not sealed.

Sample 7, brand D, is a burlap-backed sponge-rubber filler with a nominal thickness of one half inch. The rubber is of uniform texture. The edges are not sealed.

Samples 8 and 9, brand E, are made of vegetable (cane) fiber, coated and impregnated with an asphaltic compound. No protective backing is used. The samples have nominal thicknesses of one half inch and 1 inch.

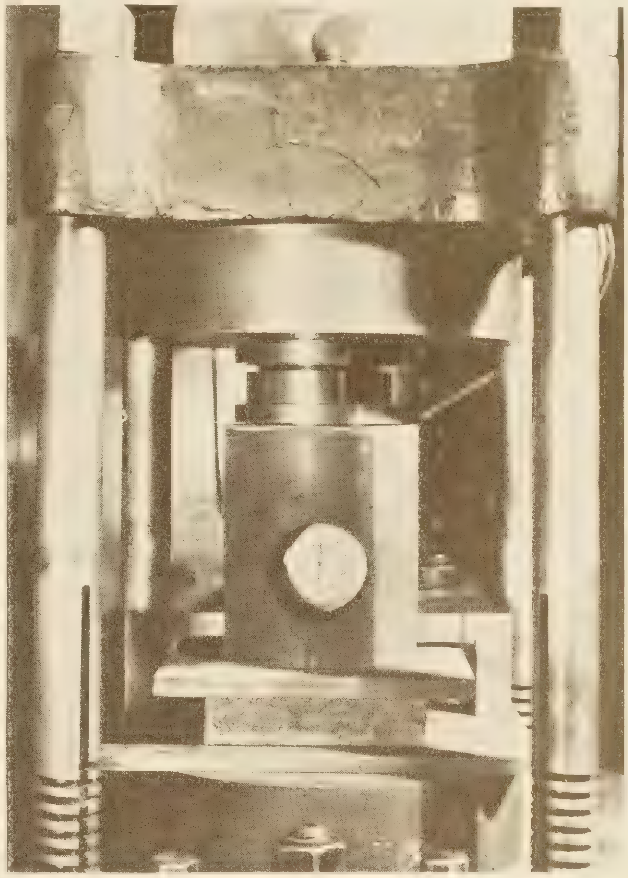

FIGURE 1.—APPARATUS FOR COMPRESSION TEST WITH EDGES FREE TO EXPAND.

Samples 10 and 11, brand F, are compounds of asphalt and fiber with particles of vulcanized rubber, some of which are three quarter inch in length. The surface of the joint is very irregular. The samples have nominal thicknesses of one half inch and 1 inch.

Sample 12, brand G, is composed of granulated cork bound with phenol formaldehyde resin. This material was submitted coated with an asphalt preparation and uncoated, but the manufacturers later advised that the coated filler was no longer used. Tests on the coated material had been started before this information was received and, since no difference was found between coated and uncoated specimens in trial determinations, the tests on coated specimens were continued. The material is of uniform texture, and has a nominal thickness of 1 inch.

Sample 13, brand H, is made of a compound asphalt and finely ground vulcanized rubber, and has a nominal thickness of one half inch.

TESTS DESIGNED TO REPRESENT SERVICE CONDITIONS

In selecting methods of testing to be used, an effort was made to duplicate as closely as possible the conditions to which joint fillers are subjected in actual service. These conditions include compression for varying lengths of time and weathering. The test methods adopted included measurements of the recovery of the filler after having been compressed a definite amount for various periods of time and, also, determinations of the effect of weathering, both natural and artificial. In addition, tests of the resiliency of the filler were made by determining the recovery following momentary compression of a definite amount.

Since the extended compression and weathering tests required considerable time, it was hoped that some correlation could be found between the results of the momentary compression tests and the extended tests. Another series of tests was made to determine the amount of extrusion of filler which might be expected in service. Some question arose regarding the amount of compression to which the filler should be subjected in these tests. Several manufacturers of fillers recommended compression to 50 percent of the original thickness, and this amount was used in all tests in which compression was applied.

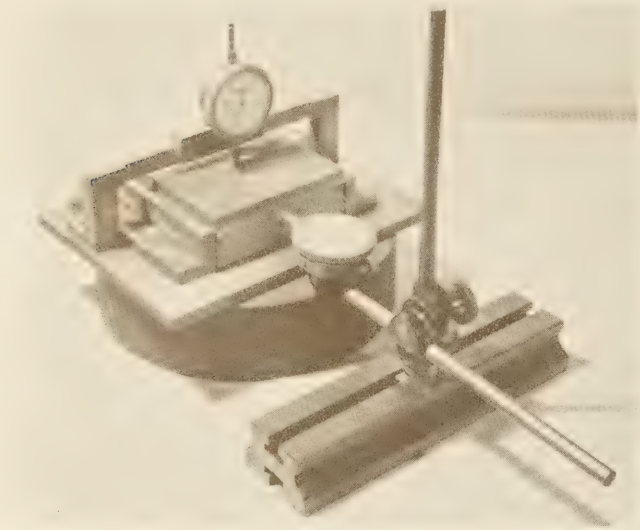

FIGURE 2.—APPARATUS USED IN EXTRUSION TEST. THREE SIDES OF THE SPECIMEN ARE RESTRAINED.

The manufacturers of rubber fillers recommended that the felt sides be removed prior to tests for resiliency. It was decided, however, that the fillers should be tested in the same condition as when in service and all tests were made on the complete filler. This practice resulted in compressing the resilient portion of the filler to a somewhat greater extent than suggested by the manufacturers since the thickness of the felt sides was included in the measurement of the original thickness. In all cases "original thickness" refers to the thickness of sample as received in the laboratory.

In the test for resiliency, the specimen was placed between two steel plates and compressed to one half its original thickness in a universal testing machine at a free head speed of 0.05 inch per minute. The movement of the specimen was measured with an Ames dial reading to 0.001 inch and mounted on a bridge above the center of the specimen as shown in figure 1. Load was transmitted to the specimen through the steel cylinder which straddled the bridge and which had a port to permit observation of the Ames dial. When the specimen had been compressed to one half its original thickness the load was quickly released, and the recovery noted at 1 minute intervals for 5 minutes. At least 1 hour later the recovery was again determined, and the procedure repeated. After three compressions the specimen was permitted to recover for 24 hours, and a final reading of recovery made.

Extended compression tests were made to measure the resistance of specimens to fatigue. Specimens were compressed to one half their original thickness, clamped between steel plates, and held in compression for periods of from 7 days to 1 year. At the expiration of each period samples were removed from compression, and the recovery in 24 hours measured. The specimens compressed for 7 and 28 days were stored in the laboratory, but the 3-, 6-, 9-, and 12-month specimens were exposed to atmospheric conditions.

Tests for durability included exposure to the weather for periods up to 1 year, exposure to 5 cycles of freezing and thawing, and exposure to 140° F. dry heat for 7 days. Following each exposure, the specimens were tested for resiliency. These tests were made to determine if natural conditions have any appreciable effect on the specimens. Specimens exposed to the weather were clamped between steel plates with only enough compression to hold the plates snug. Specimens subjected to frost action in the laboratory were saturated with water and frozen in water. Frozen specimens were thawed in water at about 80° F. One cycle was completed each 24 hours. The heat tests were made in a small oven, the source of heat being one 100-watt electric light bulb.

In the extrusion tests, the specimen was compressed to one half its original thickness between steel plates with three sides of the specimen restrained. The apparatus used is shown in figure 2. This apparatus was placed in a universal testing machine and load applied to the plate covering the specimen as in the momentary compression tests. An Ames dial reading to 0.001 inch was mounted horizontally in front of the specimen and recorded any extrusion of the material.

At the start of this investigation it was the intention to use 6- by 6-inch specimens. The load required to compress some of the fillers necessitated the use of smaller specimens since only a 40,000-pound universal testing machine was available. In some cases in the recovery tests, specimens measuring 3 by 3 inches were used. In the extrusion tests two sizes of specimens, 4¼ by 4¼ inches, and 2½ by 2½ inches, were used. With the exception of samples 10 and 11, all materials were found to be fairly uniform in structure and behavior. The variations found in samples 10 and 11 are attributed to the large particles of rubber used in the composition.

SPONGE RUBBER AND CORK FILLERS SHOW HIGH RECOVERY AFTER MOMENTARY COMPRESSION

The results shown in the accompanying figures are the average, in each case of at least three individual tests, with the exception of the data for figures 10, 13, and 14. In these figures, each point represents the average of from 1 to 6 tests. Tabulated data showing the individual results are omitted in order to conserve space.

The results of the momentary compression tests are shown in figure 3. In these tests each sample was

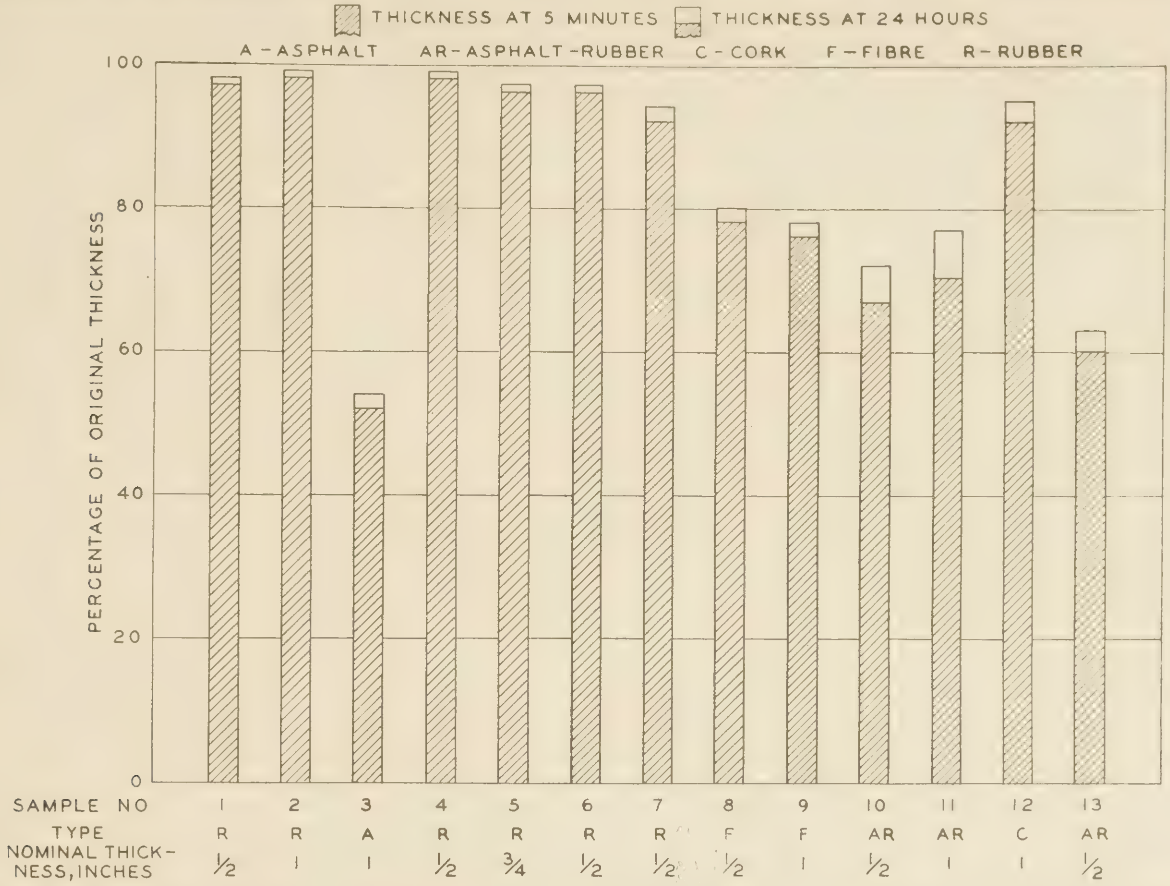

Figure 3.—Thickness of Samples After Three Applications of Momentary Compression and Recovery Periods as Indicated.

compressed three times to one half its original thickness with the load immediately released after each compression. Rest periods of 1 hour followed each of the first two compressions and one of 24 hours followed the third compression. The several sponge-rubber fillers and the cork fillers recovered to at least 94 percent of their original thickness. The fillers of cane fiber recovered at least 78 percent of the original thickness. The fillers composed of asphalt and rubber showed varying amounts of recovery ranging from 63 to 77 percent, the filler containing the finely ground rubber, no. 13, being the least efficient. The plastic asphaltic filler showed the least recovery of any specimen, as was expected.

Average curves showing the rate of recovery for 24 hours after release of pressure are given in figure 4. In most cases practically all of the recovery was obtained immediately upon release of pressure, and in only one case, that of the asphalt-rubber samples, was there any material increase in recovery between 5 minutes and 24 hours. It appears that accurate classification can be made with a recovery time of only 5 minutes.

Figure 5 shows the unit loads required to compress the different specimens to one half their original thickness. In several cases the load for the second and third compression was lower than for the one before it. This is accounted for by the relatively large set which occurred on the first compression. Due to this set, the material was not compressed to as great an extent on the second or third loading as on the first, and a smaller load was required. In some cases some change in the physical characteristics of the material occurred in the first compression, and a larger load was required thereafter to compress the material. This is illustrated by samples 1 and 5. The cork filler was apparently affected but little by the compression, since the loads required were practically constant for all three compressions.

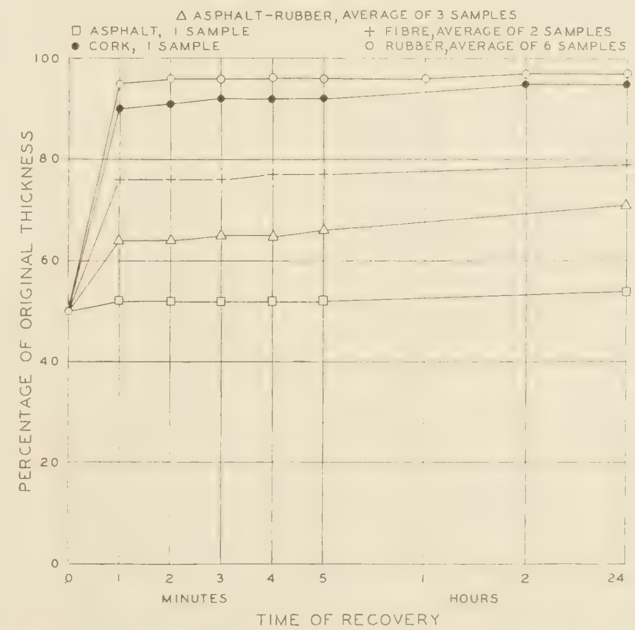

Figure 4.—Typical Time-Recovery Curves After Compression to 50 Percent of Original Thickness.

The sponge-rubber samples, nos. 1 to 7 but excluding 3, showed considerable differences in the load required

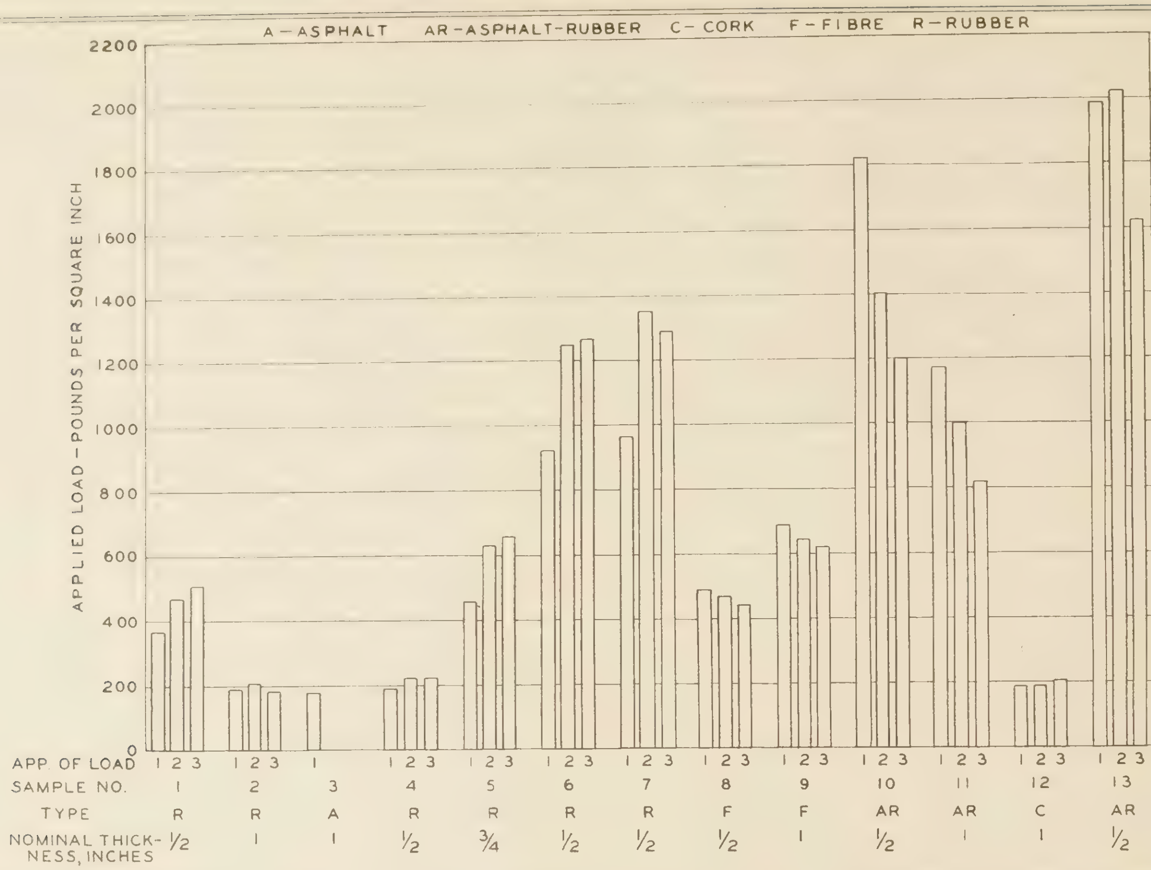

FIGURE 5.—LOAD REQUIRED TO DEFORM SPECIMEN TO ONE HALF ITS ORIGINAL THICKNESS AT EACH OF THREE APPLICATIONS OF LOAD. NO RESTRAINT AT EDGES.

to compress them. Sample 1 required about twice the load required for sample 2, although both were made by the same manufacturer. Sample 1 was only one half as thick as sample 2 and, since the felt sides were not removed from the specimens, the rubber in sample 1 was compressed to a greater extent. It was expected that a greater load would be required. On the other hand, in the cases of samples 4 and 5, the thicker material required the greater load. Samples 6 and 7, which were both approximately one half inch thick, required relatively high loads to compress them to half of their original thickness.

The load required to compress joint fillers is of interest since it is an indication of the compressive stress in the concrete pavement when the joint filler is deformed. Other things being equal, the filler requiring the least load for compression might reasonably be considered the most desirable since the concrete would be stressed the least.

A specification for resilient joint filler suggested by a State highway department requires the determination of the resiliency after 5 compressions to one half of the original thickness with 1-hour rest periods after each compression. Samples of sponge-rubber, cork, and fiber filler were tested under the proposed method and the results compared with those obtained in the three-cycle test previously discussed. Figure 6 shows that comparatively little change was found after the first application of load. It is apparent that a measure of the resiliency may be obtained from the first few cycles of compression and recovery. The five-cycle test involving a testing period at least 2 hours longer than that required by the three-cycle test does not seem to be necessary.

DIFFERENT FILLERS VARY GREATLY IN AMOUNT OF EXTRUDED MATERIAL

The capacity of the testing machine available for these tests limited the size of the sample. A number of the specifications proposed by the manufacturers of filler materials recommended the use of a 4- by 5-inch test sample for the extrusion test. It was found necessary to use smaller samples and two sizes, 4¼ and 2½ inches square, were used. Tests were made on the samples of sponge rubber and cork filler using both sizes of specimen to determine if the size had any effect on the amount of extrusion. All specimens were compressed to one half the original thickness with restraint on three edges. No appreciable difference in amount of extrusion was found between the two sizes of specimen. It was found, however, that the extrusion of the sponge-rubber filler varied directly with the thickness of the material.

The results of the extrusion tests in figure 7 show the asphaltic and asphalt-rubber fillers to have the greatest extrusion. The sponge-rubber fillers showed extrusions of from one tenth to one quarter of an inch. The cork and fiber fillers had practically no extrusion. The cork, fiber, and most of the sponge-rubber fillers had no measurable permanent extrusion, the extruded material returning to its original position upon release of pressure. Samples 6 and 7 showed some permanent extrusion but this did not exceed 0.1 inch. The asphaltic and asphalt-rubber fillers had practically no recovery of the extruded material. Figure 8 shows a sample of asphaltic filler after the test. The extrusion on the left was permitted by a slight movement of the upper plate during the test.

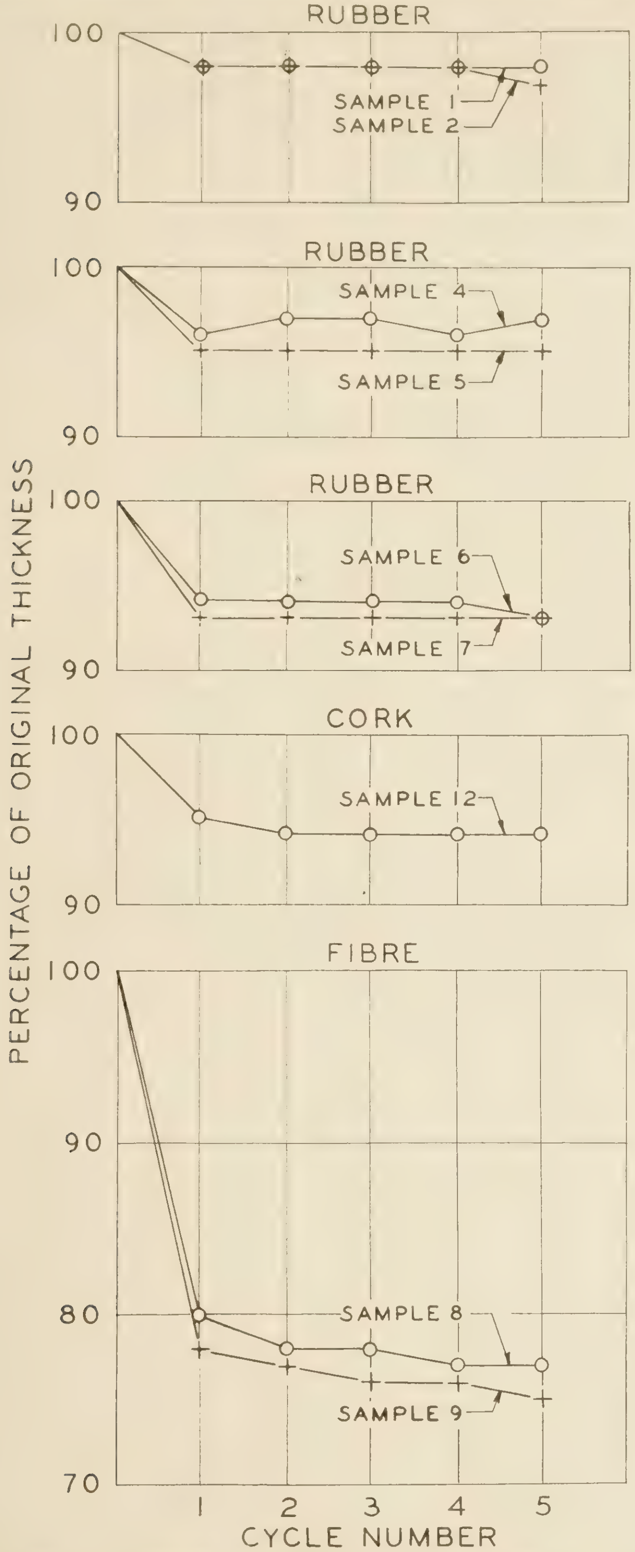

Figure 6.—Thickness at 5 Minutes After Each of Five Applications of Momentary Compression to One Half the Original Thickness. No Restraint of Edges.

In most cases the load required to produce compression to one half the original thickness was but little greater with three edges restrained than with all edges free. The asphalt-rubber samples and samples 6 and 7, of sponge rubber, however, required a considerably greater load with 3 edges restrained. Recovery of specimens after the two tests was approximately the same, as shown by figure 9.

SAMPLES EXPOSED TO NATURAL AND ARTIFICIAL WEATHERING

Tests to determine the effect of weathering were made by placing specimens between steel plates and exposing them to the weather. Sample 13 was not tested because it was received too late to obtain extended exposure without delaying the report. The cork joint, sample 12, was also received late and was exposed only 6 months. Samples were exposed to the weather for periods of 3, 6, 9, and 12 months and then subjected to three applications of momentary compression to one half of their original thickness at hourly intervals and after a 24-hour recovery period the thickness was measured. The average results are shown in figure 10. The sponge-rubber and fiber fillers apparently were not affected by the year's exposure to weathering. The cork filler showed no deleterious effect of weathering at an age of 6 months.

Considerable variation in the test results was found for the samples of asphalt-rubber filler. These samples, numbers 10 and 11, contained large pieces of vulcanized rubber. Some variation from sample to sample would probably be expected with small samples and it is believed that this variation accounts for the erratic results obtained. Considering the entire series of tests on the asphalt-rubber filler it may be said that there is no evidence of the effect of weathering.

A record of the temperatures to which the test specimens were exposed was obtained from a maximum and minimum thermometer placed beside the specimen racks. Figure 11 shows rather high maximum temperatures for the greater portion of the year. The test specimens were placed on the roof of a building near the wall of an adjoining building, where they were protected from the prevailing wind but exposed to the direct sunshine. The weather was mild and in only 5 weeks did the temperature fall below freezing.

Since expansion joint fillers would be subjected to more severe weathering in certain portions of the country, samples of the materials were exposed to both heat and frost action in artificial weathering tests. At the completion of each treatment, the samples affected were given three momentary compressions, and the recovery measured (24-hour recovery). Figure 12 shows that exposure to a temperature of 140° F. for a period of 7 days was found to have no appreciable effect on any of the samples tested.

In the freezing test, five cycles of freezing and thawing in water seriously affected some of the test specimens. One of the cane fiber specimens was completely separated into two parts, and others were partially split open. Failure, was of course, on a plane parallel to the surface of the material. Sponge-rubber fillers 4 and 5 were also affected by the frost action, the felt backing separated from the rubber and the rubber itself warped out of plane and, in some cases, expanded longitudinally. Reference to figure 12 shows, however, that the resilient properties of the damaged specimens had been affected only to a very slight extent.

Sustained compression tests were made as a measure of the enduring properties of the materials. Samples of each sponge-rubber filler were tested for periods of from 7 days to 1 year. Tests on the cork filler are now available for only 6 months since the material was received at a later date than the other samples. The great loss in resiliency resulting from the momentary compression of asphalt-rubber and cane-fiber fillers did not warrant further tests on these materials. However, tests on these types were made after sustained com-

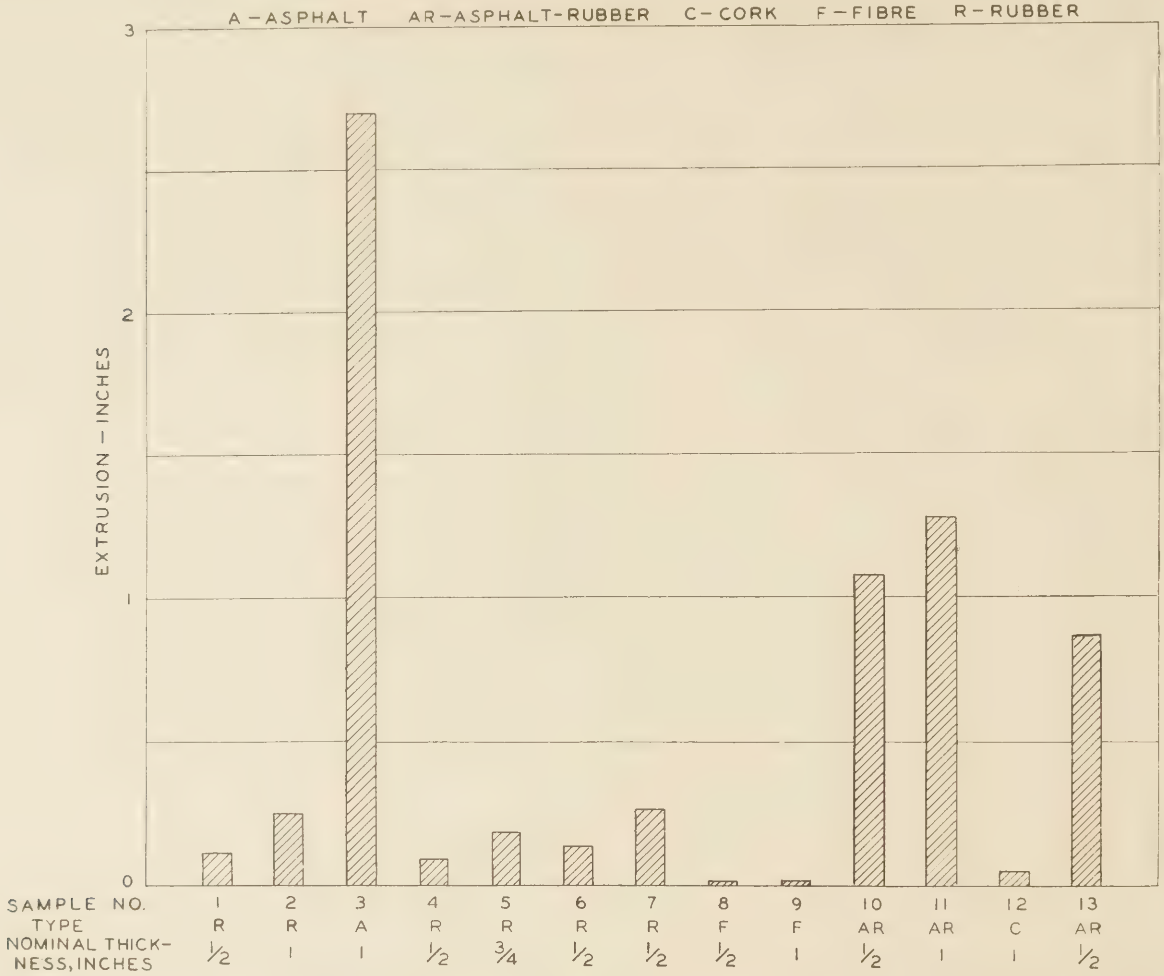

FIGURE 7.—EXTRUSION OF SPECIMENS WHEN COMPRESSED TO ONE HALF THE ORIGINAL THICKNESS AND WHILE RESTRAINED ON THREE EDGES.

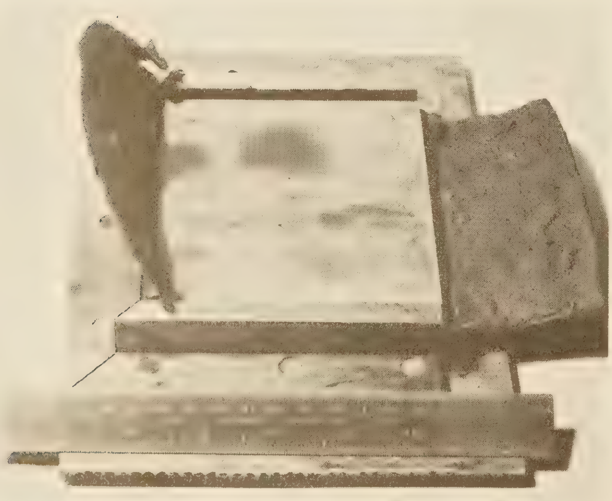

FIGURE 8.—SAMPLE OF ASPHALTIC FILLER AFTER EXTRUSION TEST.

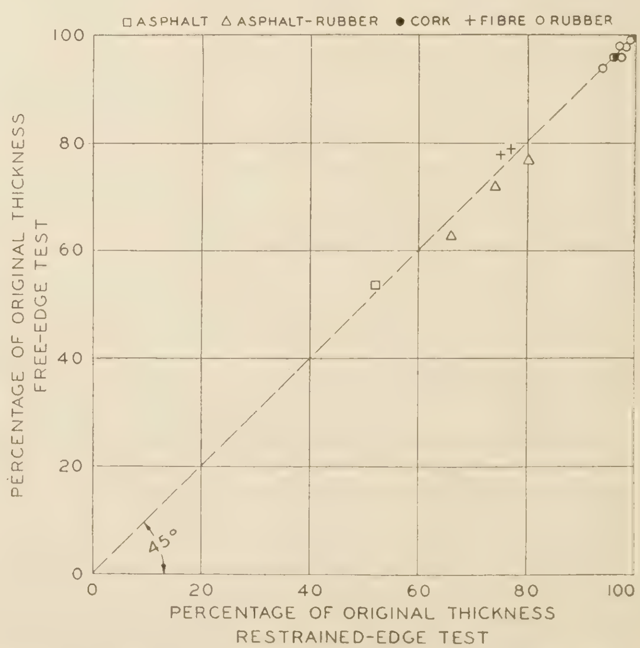

FIGURE 9.—RELATION BETWEEN RECOVERY OF MATERIALS AFTER THE FREE-EDGE AND THE RESTRAINED-EDGE TESTS.

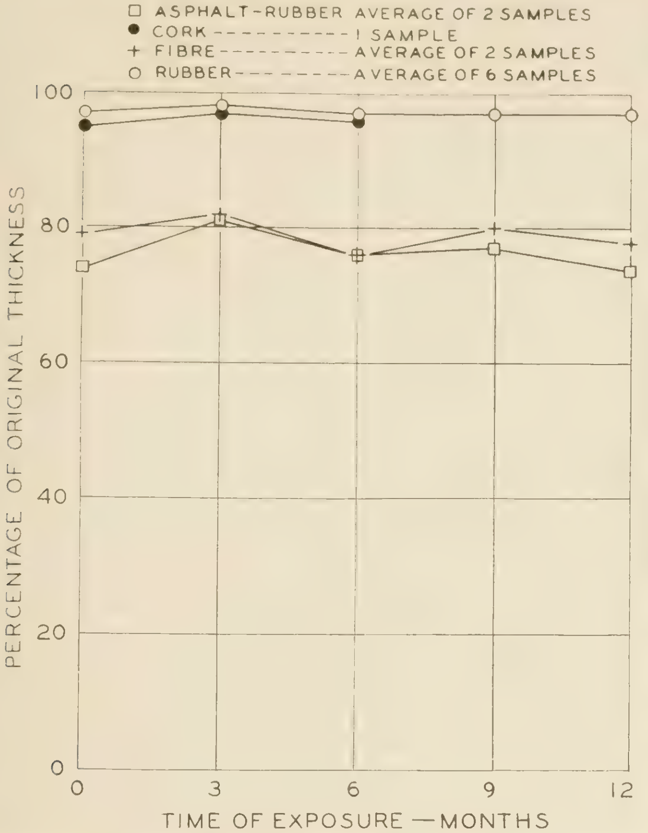

Figure 10.—Effect of Weathering on Recovery of Specimens. Samples were Exposed for Various Periods, Then Compressed Momentarily to One Half the Original Thickness Three Times at Intervals of 1 Hour and Recovery Measured After a 24-Hour Rest Period.

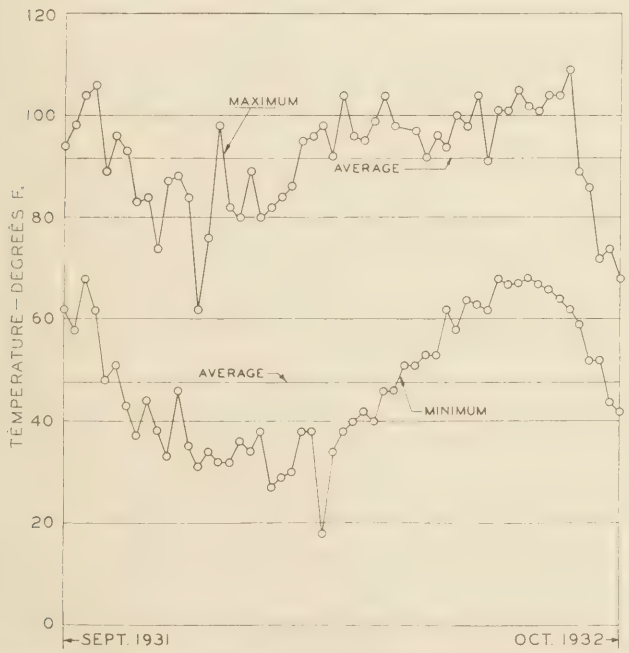

Figure 11.—Temperature Record During Exposure Test Period.

pression for 7 days and 3 and 6 months. Asphalt-rubber filler no. 13 was not tested due to the lateness of receipt and the small amount of material submitted.

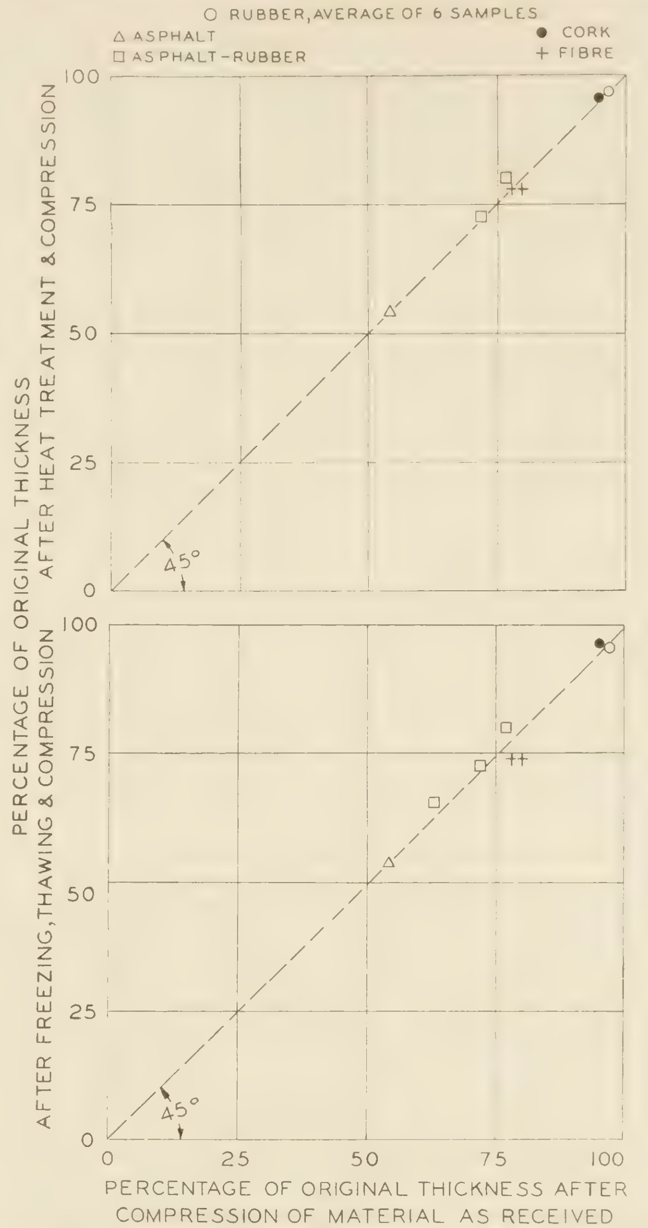

Figure 12.—Recovery After Compression of Unweathered Specimens Compared With Recovery After Compression of Samples Subjected to Artificial Weathering.

ALL SAMPLES SHOW PROGRESSIVE LOSS IN RESILIENCY WITH SUSTAINED COMPRESSION

The recovery of specimens after sustained compression is shown in figures 13 and 14. Figure 13 shows average values for each type of material while the results for each sample of sponge-rubber filler are shown in figure 14. A recovery period of 24 hours was used in all cases. The cork filler showed a progressive loss in resiliency with increase in time of compression. After being compressed for 7 days, the cork filler recovered to 90 percent of its original thickness, but after 6 months compression a recovery to only 63 percent was found. The sponge-rubber filler showed on the average somewhat better recovery at 7 days, but at 6 months the recovery was practically the same as for the cork. After being under compression for 1 year, the sponge rubber in general had practically no recovery. Con-

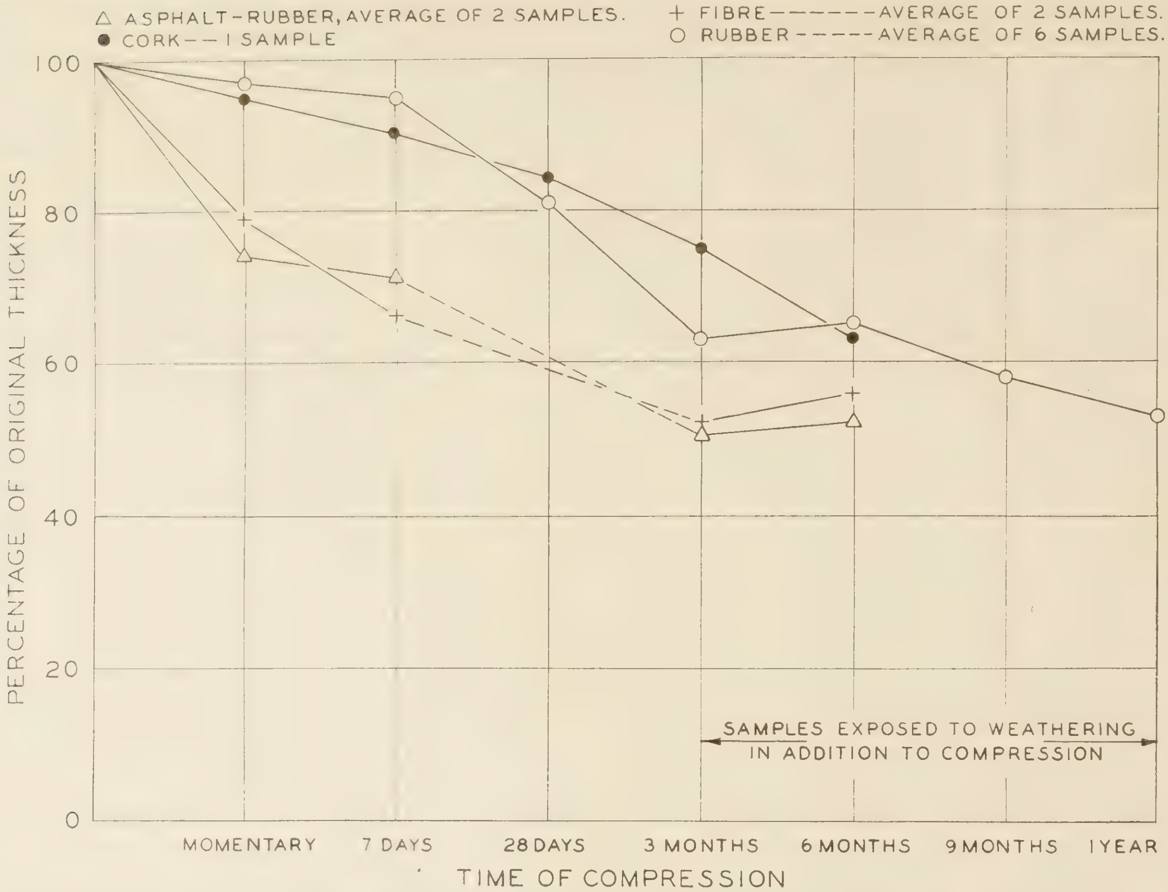

Figure 13.—Recovery of Specimens in 24 Hours After Various Times of Compression and Weathering.

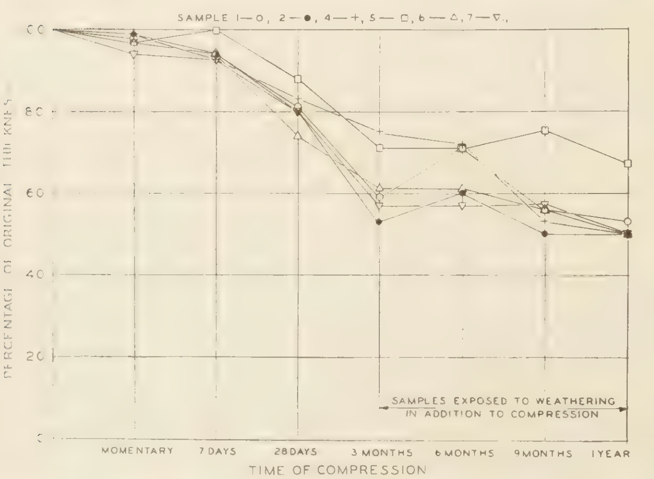

Figure 14.—Recovery of Individual Specimens of Sponge Rubber in 24 Hours After Various Times of Compression.

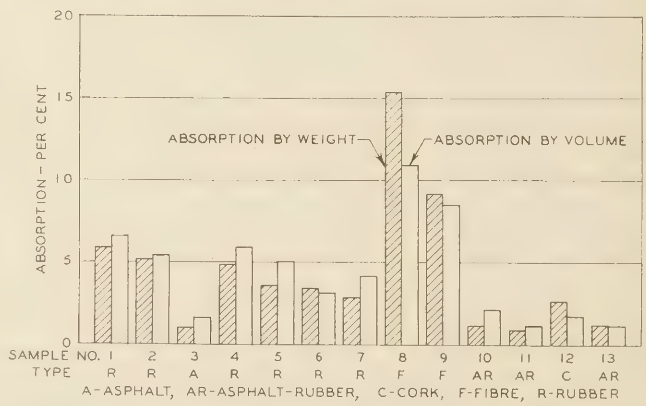

Figure 15.—Absorption of Specimens in 24 Hours.

sideration of the individual samples of sponge rubber indicates that one sample is somewhat superior to the others in resistance to fatigue. This sample, no. 5, showed a recovery to 67 percent after compression for 1 year. Sample 4, made by the same manufacturer, showed no recovery, and it is possible that the results for sample 5 may be somewhat in error. Four of the other samples showed no change in thickness upon release of pressure, and one recovered slightly to 53 percent of its original thickness. After compression for 28 days, all samples of sponge rubber showed recoveries of 74 percent of their original thicknesses or better. The fiber and asphalt-rubber fillers had relatively little recovery even after only 7 days compression, while at 3 and 6 months the recovery was practically negligible.

Absorption tests were made on all samples. Test specimens were weighed, immersed in water for 24 hours, wiped surface-dry and reweighed. The results given in figure 15 show the cane fiber filler to be the most absorptive and the asphaltic and asphalt-rubber fillers to be the least. The sponge-rubber fillers show absorptions varying from 2.8 percent to 5.8 percent by weight. The cork filler absorbed only 2.6 percent water by weight. In the case of the sponge-rubber and asphaltic samples the felt or burlap backing probably absorbed an appreciable amount of water.

While the tests reported in this investigation cannot be said to be complete, it is believed that sufficient data have been obtained to furnish a fairly accurate knowledge of the capabilities of the various materials studied. It must be said that the perfect expansion-joint filler is yet to be developed. None of the materials tested show all of the qualities desired in a filler. So far as resiliency is concerned, however, the majority of the materials tested are considerably superior to the plastic asphaltic fillers which have been used for many years.

Of the materials tested, the sponge-rubber and cork fillers appear to combine to the highest degree the features of resiliency, durability, and resistance to extrusion which are considered desirable in expansion joint fillers. The chief question regarding the sponge-rubber filler is the probable service life of the material. One year's exposure to the weather failed to cause any apparent change in the characteristics of the sponge rubber. However, no definite information is available beyond this period. No conclusions can therefore be drawn at this time regarding the relative durability of fillers of this type.

The cane or vegetable fiber filler possesses the best resistance to extrusion of any material included in this investigation. It is not very resilient and cannot be considered as efficient from this point of view as either sponge rubber or cork. Although the fiber joint is easily damaged by frost action, tests on frozen samples show no appreciable loss of resiliency. It is possible that after the material is installed frost action would have little deleterious effect on its performance.

Information obtained from various sources indicates that changes in length of as much as one quarter inch may be expected in a 40- to 50-foot concrete slab due to variations in temperature and moisture content. These tests indicate the desirability of using 1 inch of cork or sponge rubber filler for each 40- to 50-foot slab if it is desired to make allowance for the maximum expansion which may occur and at the same time have the joints tightly filled with desirable material.

The asphalt-rubber fillers appear to be the least desirable of the materials studied. These fillers show relatively little resiliency and have large amounts of extrusion. In these respects they are little better than the plastic asphaltic joint fillers.

CONCLUSIONS

The following conclusions appear to be warranted by the test results.

1. Considering the essential features of the resilient types of filler as determined by the tests described in this report, the different types studied may be rated as follows:
   (*a*) Sponge rubber and cork
   (*b*) Fiber
   (*c*) Asphalt-rubber
2. The different samples of sponge-rubber filler exhibit a considerable range in physical characteristics, which warrants careful investigation of any particular material prior to use.
3. A compression test with 3 edges restrained is believed suitable for testing resilient expansion joint fillers. Measurements of the recovery, extrusion, and applied load can be made in a single test.

(Continued from p. 16)

area extending not less than 1 foot beyond the edge of the pavement and to a depth of not less than 2 feet. Excavation for the entire width of roadway and backfilling with selected material is preferable to the use of drains in excavated trenches.

3. The same preventive measures should be adopted for bituminous road surfaces as are employed for concrete road surfaces.

4. A porous granular material of the A–3 soil group should be substituted for excavated frost-heave soil. In locations where the cost of this type of material is excessive, topsoil may be used. The topsoil should be carefully selected from weathered soil layers and should be free from accumulations of lime.

5. Drains do not prevent frost heave in typical frost-heave soils such as the silts, silt loams, silty clays, and silty clay loams. According to the results of tests, most of these soils possess the physical properties of the group A–4 soils.

6. Frost heave caused by blocked drains or a high water table in porous A–3 soils may be prevented by the installation of drains. A thorough investigation of the soil conditions at each location should be made and the drainage system designed accordingly.

7. Frost heaving in profiles *C* and *E* (fig. 5) may be prevented by drainage. In order to eliminate detrimental frost action under the conditions represented by the remainder of the profiles, the frost-heaving soils should be excavated (or grade line raised) and selected material placed beneath the surfacing. The type of treatment should be governed by the permanence of the surfacing and the traffic requirements.

8. Information furnished by soil surveys, made prior to construction, should be utilized in fixing highway grades in such a manner as to correct the deficiencies of the natural soil profiles, with artificially constructed soil profiles. This practice will result in greater economy than the correction of defects in the subgrade after the grading and surfacing have been completed.

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION

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

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

AS OF FEBRUARY 28, 1934

STATE	PUBLIC WORKS FUNDS ASSIGNED FOR CLASS I PROJECTS ON THE FEDERAL AID HIGHWAY SYSTEM	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR NEW CLASS I PROJECTS
		Total cost	Public works funds	Regular Federal aid	Mileage	Estimated total cost	Public works funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	Public works funds allotted	Mileage	
Alabama	$ 4,185,067					$ 4,989,020.89	$ 2,755,265.47	$ 2,233,755.42	33.5	276.3	$ 235,186.30	28.0	$ 1,194,615.23
Arizona	3,804,731	$ 303,587.42	$ 303,587.42		21.6	3,299,720.65	2,835,025.45		34.3	231.1	68,539.45	.9	597,578.68
Arkansas	3,374,167	18,339.52	9,849.52	$ 8,490.00	.6	1,674,984.39	1,215,892.85	459,091.54	40.5	87.7	849,387.64	58.9	1,299,036.99
California	7,803,677	251,200.24	217,552.52		5.8	8,337,853.95	6,283,859.92		30.3	250.5	585,311.58	17.1	716,952.98
Colorado	3,437,265	728,229.51	728,229.51		53.4	1,478,907.19	1,468,326.84		33.1	59.1	282,296.35	3.7	958,412.30
Connecticut	1,404,213					1,230,901.72	1,055,122.78	175,778.94	6.7	26.6	346,048.17	6.4	3,042.05
Delaware	909,544	133,532.30	133,532.30		15.4	742,815.70	742,815.70		21.7	17.4			33,196.00
Florida	2,615,917	176,839.08	97,261.49	79,577.59	6.4	2,519,238.84	1,817,932.22	700,696.52	41.6	95.7	516,102.72	13.9	184,620.57
Georgia	5,045,592	130,924.84	130,924.84		5.5	2,146,766.41	2,146,766.41		36.9	129.0	745,464.53	56.9	2,022,436.22
Idaho	2,166,858	284,272.42	283,033.41		35.6	1,345,164.12	1,331,659.97		44.5	95.0	146,518.90	13.4	405,645.72
Illinois	4,431,348					1,038,589.29	1,038,589.29		18.8	24.1	567,012.04	8.0	2,825,746.67
Indiana	4,717,786					1,974,416.80	1,974,416.80		32.6	67.5	1,159,878.89	32.2	1,583,490.31
Iowa	5,027,830	704,773.93	704,400.00		30.0	4,153,957.36	3,848,330.00		29.7	223.2	358,250.00	15.7	116,850.00
Kansas	5,044,802	340,082.76	340,082.76		67.7	3,724,965.95	3,632,252.56		38.3	299.4	1,046,189.45	31.9	26,277.23
Kentucky	3,608,332	145,472.40	145,472.40		12.0	1,815,380.73	1,815,380.73		35.9	136.7	1,094,106.25	85.2	553,372.62
Louisiana	2,914,295	38,528.74	38,528.74		1.0	2,088,854.29	1,582,203.29		20.8	39.7	700,076.70	27.7	593,486.27
Maine	1,617,560					1,161,321.30	1,125,754.35		36.7	35.5	242,775.60	8.5	249,030.05
Maryland	1,782,263					801,143.78	791,495.03		8.2	14.8			990,767.97
Massachusetts	1,101,716	153,936.[illegible]	83,952.64	69,984.24	4.2	1,160,630.24	844,970.60	315,659.64	17.9	32.9	64,430.41	.7	108,362.35
Michigan	5,094,491	63,000.00	63,000.00		1.8	2,784,250.00	2,784,250.00		28.5	146.0	2,190,066.00	79.9	57,175.00
Minnesota	5,115,153	1,148,243.24	1,148,243.24		263.0	2,097,128.05	2,097,128.03		41.2	301.1	366,605.03	105.6	1,503,176.70
Mississippi	3,489,337					3,216,675.60	1,733,202.37	1,444,841.29	15.9	158.2	629,537.53	83.5	1,126,597.10
Missouri	6,090,153	318,173.69	219,506.32		20.1	4,063,325.13	3,723,740.33	50,048.60	37.1	130.4	871,040.85	32.4	1,275,865.50
Montana	4,463,849	221,309.34	221,309.34		34.0	3,744,987.76	3,472,108.88	192,878.88	37.5	268.7	640,792.56	40.9	129,638.22
Nebraska	3,914,481	381,081.97	255,420.54		144.8	3,915,261.27	3,036,632.94	108,538.00	41.9	180.1	460,859.94	53.4	161,567.58
Nevada	2,909,387	272,235.60	272,235.60		60.3	1,466,705.48	1,466,705.48		33.9	131.5	758,995.53	42.1	411,450.39
New Hampshire	725,739					573,516.95	567,831.95		29.8	9.8	68,469.25	1.1	89,437.80
New Jersey	3,065,137	12,742.75	12,742.75		.2	1,207,980.25	1,207,980.25		7.9	19.6	1,381,124.01	17.7	463,289.99
New Mexico	2,896,467	1,072,779.31	1,072,779.31		139.0	1,873,686.82	1,737,327.34	102,921.33	12.2	138.7			86,360.35
New York	10,182,526	117,100.00	117,100.00		3.8	9,631,250.00	8,658,575.00	315,000.00	25.0	205.7	1,248,590.00	28.0	158,261.00
North Carolina	4,761,147	567,604.75	321,021.30	246,583.45	44.0	1,839,373.64	1,533,949.37	305,424.27	23.5	196.7	1,306,568.04	240.0	1,599,608.29
North Dakota	2,902,224	506,686.40	506,682.89		258.9	873,842.71	873,842.71		15.3	228.3	1,193,654.38	380.1	328,044.02
Ohio	6,968,066	261,790.00	261,790.00		5.5	5,801,895.00	5,633,595.00		35.1	150.2	1,072,681.00	33.0	
Oklahoma	4,608,399	189,356.42	189,356.42		8.0	3,418,432.44	3,418,432.44		34.6	218.1	509,439.37	53.5	491,170.77
Oregon	3,053,448	350,457.26	322,563.76		40.4	2,352,972.89	2,204,545.69	58,505.67	28.3	111.2	412,750.38	29.9	113,588.17
Pennsylvania	5,757,978	12,791.10	12,791.10		.2	4,043,732.67	4,043,732.67		13.8	97.2	1,701,454.23	27.8	
Rhode Island	999,354	18,830.63	18,830.63		.6	969,153.21	969,153.21		26.7	20.0	10,468.37		901.79
South Carolina	2,729,583	4,168.89	4,168.89			1,826,973.64	1,824,244.02	2,729.62	35.4	169.9	282,842.64	22.5	618,327.45
South Dakota	3,005,739	871,013.41	865,096.92	5,916.49	144.9	1,101,467.48	1,069,443.19	32,024.29	11.2	177.2	498,160.33	59.3	573,038.56
Tennessee	4,246,309	131,887.64	72,591.00	59,296.64	5.1	2,691,692.56	2,174,971.82	516,720.74	29.6	98.7	1,431,281.52	59.1	567,464.66
Texas	12,122,012	651,879.59	494,793.34		157.2	7,967,735.36	7,063,543.71	-	29.1	639.2	1,933,798.20	201.6	2,629,876.75
Utah	2,374,205	869,670.21	869,670.21		101.6	1,058,173.84	1,057,778.94		31.6	80.9	219,567.89	10.6	227,187.96
Vermont	928,184	47,983.81	47,983.81		6.2	735,362.91	707,209.66		30.3	34.7	20,000.00	.5	152,990.53
Virginia	3,708,379	157,156.81	139,973.94	17,182.87	10.2	1,680,515.57	1,680,103.26	412.31	39.6	62.8	1,742,368.98	83.5	145,932.82
Washington	3,057,934	212,506.32	212,506.32		14.1	2,129,434.43	2,126,077.61	3,356.82	26.6	81.4	341,660.72	14.2	377,689.35
West Virginia	2,013,405	194,881.82	194,881.82		6.4	1,492,378.55	1,492,378.55		36.7	63.6	254,963.13	2.3	71,181.50
Wisconsin	4,862,441	298,477.25	288,174.42	1,190.00	15.7	2,581,337.60	2,559,238.07	21,500.00	37.2	101.7	664,009.68	63.1	1,351,018.83
Wyoming	2,250,663	680,509.21	593,153.00	60,000.00	118.6	1,565,534.20	1,411,700.00	18,000.00	21.8	325.8	29,682.61	11.5	216,127.39
District of Columbia													
Hawaii	1,683,956	25,988.65	12,994.33	12,994.32	.9	1,429,747.99	1,197,920.88	181,827.36	11.4	28.6	186,604.30	3.3	286,436.49
TOTALS	184,973,109	13,070,026.11	12,027,768.75	561,215.60	1,864.7	125,819,157.60	111,833,403.63	7,239,711.24	29.9	6,718.2	31,435,611.45	2,189.5	29,676,325.17

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION

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

CLASS II—PROJECTS ON EXTENSIONS OF THE FEDERAL-AID HIGHWAY SYSTEM INTO AND THROUGH MUNICIPALITIES

AS OF FEBRUARY 28, 1934

STATE	PUBLIC WORKS FUNDS ASSIGNED FOR CLASS II PROJECTS IN MUNICIPALITIES	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR NEW CLASS II PROJECTS
		Total cost	Public works funds	Regular Federal aid	Mileage	Estimated total cost	Public works funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	Public works funds allotted	Mileage	
Alabama	$ 2,092,533	$ 29,479.63	$ 29,479.63		0.8	$ 299,579.46	$ 299,579.46		31.0	8.5	$ 297,352.30	9.7	$ 1,466,121.61
Arizona	781,794	6,427.92	6,427.92		.5	125,980.32	125,980.32		9.5	2.9	96,354.42	2.8	553,031.34
Arkansas	1,687,084					573,074.48	501,179.27	$ 71,895.21	28.3	16.1	373,068.64	10.9	812,836.09
California	3,901,839	107,821.54	96,674.88		3.1	2,184,283.71	1,922,291.90		20.7	27.9	836,261.89	8.9	1,046,610.33
Colorado	1,718,633	128,443.81	128,443.81		3.3	378,285.92	378,285.92		13.7	7.0	337,147.08	6.7	874,756.19
Connecticut	802,407	52,696.86	52,696.86		.5	430,230.83	430,230.83		30.2	6.9	255,987.28	2.3	63,492.03
Delaware	454,772	75,309.50	75,309.50		.9	149,600.00	149,600.00		5.3	1.0	50,710.00	.8	179,152.50
Florida	1,307,959					1,043,278.43	835,513.22	207,765.21	20.1	10.6	472,445.78	6.9	
Georgia	2,724,620	27,184.02	27,184.02		1.1	458,238.40	458,238.40		25.5	16.5	280,542.75	13.8	1,958,654.83
Idaho	1,197,829	22,257.32	22,049.37		1.7	330,125.37	326,160.97		16.2	5.5	82,133.43	4.1	767,485.23
Illinois	6,377,199	45,749.72	45,749.72		.5	2,905,279.75	2,905,279.75		11.8	30.9	3,254,737.89	33.9	671,431.64
Indiana	4,813,165	24,698.54	24,698.54		.4	18,031.73	18,031.73		22.2	.4	1,170,404.06	26.0	3,605,030.67
Iowa	2,315,585	244,716.01	242,350.00		8.6	776,439.00	732,415.00		14.9	15.4	681,750.00	19.0	1,159,070.00
Kansas	2,522,401	27,227.72	27,227.72		2.6	1,411,651.30	1,411,651.30		8.2	19.8	1,083,521.98	15.1	
Kentucky	2,029,687	13,307.85	13,307.85		.2	139,663.07	139,663.07		2.9	1.9	505,435.11	12.7	1,371,280.97
Louisiana	1,457,148	34,714.92	34,714.92		.2	323,725.57	323,725.57		59.9	6.4	167,260.18	7.6	931,447.33
Maine	909,378	79,296.57	79,296.57		1.6	350,433.41	350,433.41		33.4	8.5	312,148.18	4.7	167,999.84
Maryland	891,132					16,788.76	12,638.76		79.1	.6			878,493.24
Massachusetts	5,007,199	53,896.49	30,896.49	$ 23,000.00	1.4	3,150,737.00	3,136,637.00	14,100.00	14.3	12.7	250,293.88	1.1	1,589,372.23
Michigan	4,457,679	180,700.00	180,700.00		2.3	1,056,370.00	1,054,220.00		12.6	19.6	1,492,646.00	9.9	1,730,113.00
Minnesota	3,410,102	471,834.86	471,834.86		34.5	832,848.47	832,848.47		46.8	33.5	530,014.28	14.4	1,575,404.39
Mississippi	1,744,669					355,998.02	329,830.58	26,167.44	26.7	12.4	306,452.56	13.3	1,108,385.86
Missouri	3,045,077	4,587.13	3,305.88		.3	992,398.10	979,369.87		39.2	15.5	834,386.19	19.0	1,228,015.06
Montana	1,115,962	2,878.39	2,878.39		.1	374,033.85	374,033.85		25.1	10.4	151,076.18	9.1	587,973.58
Nebraska	1,957,240	29,267.12	28,577.83		3.6	1,013,524.35	1,013,524.35		32.6	16.5	834,675.77	14.1	80,462.05
Nevada	500,051	50,304.27	50,304.27		1.2								449,746.73
New Hampshire	706,640					353,802.08	353,802.08		12.4	7.5	352,837.92	9.0	
New Jersey	3,217,442	70,309.86	70,309.86		.8	1,782,488.30	1,782,488.30		24.2	12.5	820,908.72	7.7	543,735.12
New Mexico	1,448,234	10,526.42	10,526.42		1.0	490,082.67	490,082.67		22.4	9.2	471,729.58	11.6	475,895.33
New York	8,530,099	129,750.00	129,750.00		2.2	5,007,430.00	4,881,145.00		17.9	41.5	2,400,830.00	16.4	1,118,374.00
North Carolina	2,380,573	52,434.88	52,434.88		4.8	201,332.71	201,332.71		27.3	9.1	577,075.42	26.5	1,549,729.99
North Dakota	1,451,112	2,153.29	2,153.29		1.0	49,955.24	49,955.24		50.0	6.2	409,969.28	14.5	989,034.19
Ohio	4,645,378	66,830.00	59,355.00		2.0	1,399,084.43	1,309,793.43		21.0	16.0	2,485,759.00	36.0	790,470.57
Oklahoma	2,304,200	24,890.38	24,890.38		1.0	465,104.34	465,104.34		13.5	12.0	1,045,656.11	20.0	768,549.17
Oregon	1,526,724	23,731.40	23,731.40		.6	754,954.56	740,638.83	14,315.73	14.7	14.6	383,580.87	8.2	378,772.90
Pennsylvania	5,416,051	120,976.76	120,976.76		4.4	1,217,636.69	1,208,335.32		21.2	22.2	1,565,479.15	22.8	2,521,259.77
Rhode Island	499,677					155,048.79	155,048.79		3.2	2.4	106,304.22	1.8	238,323.99
South Carolina	1,364,791	24,919.48	24,919.48		1.7	307,304.30	306,899.29	405.01	16.3	15.3	138,038.95	6.3	894,933.28
South Dakota	1,502,870	75,816.40	75,816.40		4.7	158,605.22	158,605.22		13.9	5.9	209,590.01	8.7	1,058,858.37
Tennessee	2,123,155	19,992.06	19,992.06		1.2	566,629.72	566,629.72		19.2	7.3	380,995.19	6.7	1,155,538.03
Texas	6,061,006	124,266.13	104,642.19		9.2	1,710,575.57	1,580,519.44		25.1	58.4	1,676,566.00	25.5	2,699,278.37
Utah	771,826	369,762.28	369,685.28		8.8	187,426.30	186,467.30		34.3	4.9	58,995.40	2.0	156,678.02
Vermont	500,509	3,854.07	3,854.07		.6	298,777.26	297,549.64		21.8	8.2	153,245.15	3.6	45,860.14
Virginia	1,854,189	54,390.92	44,739.04	9,651.88	2.0	1,030,170.29	826,530.98		17.9	9.5	778,497.57	14.3	204,421.41
Washington	1,877,571	169,589.66	169,589.66		4.9	924,480.62	919,765.02		9.7	21.2	685,191.81	6.1	103,024.51
West Virginia	1,342,270	15,323.43	15,323.43		.4	362,465.66	338,119.92		17.2	5.6	526,204.30	8.5	462,622.35
Wisconsin	2,431,220	151,529.60	151,529.60		4.8	637,207.36	637,207.36		37.8	15.3	681,025.52	17.2	961,457.52
Wyoming	1,125,332	53,504.69	53,004.69		1.1	390,535.51	390,534.55		10.5	9.4	432,018.12	7.1	249,774.64
District of Columbia	959,235	295,510.62	295,510.62		2.4	570,729.00	570,729.00		15.4	1.7	92,921.15	.6	74.23
Hawaii													
TOTALS	114,268,748	3,572,862.52	3,496,843.54	32,651.88	129.0	38,712,425.92	37,458,647.15	334,648.60	20.2	623.3	31,090,224.67	547.9	42,223,032.64

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

CLASS III—PROJECTS ON SECONDARY OR FEEDER ROADS

AS OF FEBRUARY 28, 1934

STATE	PUBLIC WORKS FUNDS ASSIGNED FOR CLASS III PROJECTS ON SECONDARY HIGHWAYS	COMPLETED			UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR NEW CLASS III PROJECTS	STATE
		Total cost	Public works funds	Mileage	Estimated total cost	Public works funds allotted	Percentage completed	Mileage	Public works funds allotted	Mileage		
Alabama	$ 2,092,533				$ 93,344.43	$ 93,344.43	16.1	5.3	$ 83,707.07	7.6	$ 1,915,481.50	Alabama
Arizona	625,435				433,887.46	370,530.68	19.7	32.1			254,904.32	Arizona
Arkansas	1,687,084				331,611.51	331,611.51	8.4	21.6	211,195.30	38.9	1,144,277.19	Arkansas
California	3,901,838	$ 122,219.50	$ 85,059.65	1.7	1,714,358.42	1,443,414.25	17.3	112.1	886,688.28	28.7	1,486,675.82	California
Colorado	1,718,632	50,000.00	50,000.00	12.2	825,545.21	825,545.21	60.8	115.1	250,634.51	29.7	592,452.28	Colorado
Connecticut	659,120				664,915.58	659,120.00	2.1	14.5				Connecticut
Delaware	454,772								161,554.00	1.8	293,218.00	Delaware
Florida	1,307,958				1,203,489.02	1,203,489.02	30.8	68.3	28,436.47	.1	76,032.51	Florida
Georgia	2,320,973				359,481.66	359,481.66	12.5	27.3	338,604.91	19.7	1,622,886.43	Georgia
Idaho	1,121,562	67,404.64	63,302.38	7.0	810,860.83	778,839.50	44.3	117.6			279,420.12	Idaho
Illinois	6,262,223	78,787.21	78,787.21	19.3	1,824,519.04	1,824,519.04	14.7	154.0	1,974,498.84	109.6	2,384,417.91	Illinois
Indiana	501,892				189,508.90	189,508.90	16.9	42.4	168,415.02	11.6	143,968.08	Indiana
Iowa	2,212,245	85,897.35	83,000.00	27.3	625,320.05	595,300.00	32.8	53.3	387,300.00	52.9	1,146,645.00	Iowa
Kansas	2,522,401	11,865.47	11,865.47	3.8	1,107,780.33	1,107,780.33	9.7	99.0	1,387,307.18	61.4	15,448.02	Kansas
Kentucky	1,879,340	13,990.90	13,990.90	2.8	1,215,225.29	1,215,225.29	25.4	155.4	630,101.56	61.0	20,022.25	Kentucky
Louisiana	1,457,148				381,879.69	381,879.69	16.0	15.6	325,115.56	19.9	750,152.75	Louisiana
Maine	842,479	465,424.08	464,188.60	56.3	424,543.68	367,221.58	82.5	40.3	8,989.70	.2	2,079.12	Maine
Maryland	891,132				211,934.07	211,934.07	13.2	15.6	410,405.85	29.3	268,792.08	Maryland
Massachusetts	488,185				469,741.41	469,741.41	35.6	15.2			18,443.59	Massachusetts
Michigan	3,184,057	88,500.00	88,500.00	7.3	1,821,450.00	1,821,450.00	7.9	139.4	1,030,550.00	87.3	243,557.00	Michigan
Minnesota	2,131,314	46,377.25	46,377.25	34.7	1,388,608.93	1,308,177.93	34.2	199.9	298,501.02	34.4	478,257.80	Minnesota
Mississippi	1,744,669				435,000.00	435,000.00	43.2	54.6			1,309,669.00	Mississippi
Missouri	3,045,076	18,488.65	18,488.65	4.0	2,260,881.93	2,260,881.93	27.6	386.0	643,261.03	96.2	122,444.39	Missouri
Montana	1,859,937	79,392.69	79,392.69	16.0	704,516.02	704,516.02	5.4	68.0	827,038.34	111.3	248,989.95	Montana
Nebraska	1,957,240				1,757,064.92	1,755,202.60	28.2	250.3	202,037.40	33.6		Nebraska
Nevada	1,136,479				1,006,971.41	1,006,971.41	32.9	97.0			129,507.59	Nevada
New Hampshire	477,460				491,997.62	451,173.77	30.6	25.1	26,210.05	.5	76.18	New Hampshire
New Jersey	63,460				56,550.52	56,550.52	7.1	.5			6,909.48	New Jersey
New Mexico	1,448,234	9,000.00	9,000.00	4.0	1,071,950.00	1,071,950.00	38.9	681.2	63,900.00	73.0	303,384.00	New Mexico
New York	3,617,476	144,300.00	144,300.00	4.6	3,809,913.19	3,378,488.19	18.7	93.1	85,975.00	1.9	8,712.81	New York
North Carolina	2,380,573	145,298.61	145,298.61	18.4	1,086,726.77	1,086,726.77	35.6	98.4	17,611.55	.5	1,130,936.07	North Carolina
North Dakota	1,451,112								95,781.83	16.8	1,355,330.17	North Dakota
Ohio	3,871,148	133,950.00	133,950.00	44.6	2,548,530.00	2,433,394.90	22.0	228.4	1,083,260.00	34.8	220,543.10	Ohio
Oklahoma	2,304,199				152,421.47	152,421.47		10.9	611,975.75	100.8	1,539,801.78	Oklahoma
Oregon	1,526,724				775,189.12	757,716.98	14.9	27.2	653,650.00	62.3	115,357.02	Oregon
Pennsylvania	7,716,975	32,266.00	32,266.00	4.0	5,979,370.20	5,924,931.75	25.7	585.9	871,403.68	57.5	888,373.57	Pennsylvania
Rhode Island	499,677				72,438.03	72,438.03	2.8	7.0	209,313.33	19.1	217,925.64	Rhode Island
South Carolina	1,364,791				1,202,607.25	1,202,607.25	24.2	135.7			162,183.75	South Carolina
South Dakota	1,502,870	39,421.46	39,421.46	16.1	43,072.35	43,072.35	95.2	14.8	182,724.57	62.9	1,237,651.62	South Dakota
Tennessee	2,123,155				654,723.26	654,723.26	18.8	60.8	558,578.16	39.2	909,853.58	Tennessee
Texas	6,061,006	43,529.18	34,878.83	17.9	4,908,038.45	3,991,905.75	34.3	597.7	890,209.15	138.4	1,144,012.27	Texas
Utah	1,048,677	81,770.38	81,543.36	11.7	730,506.46	727,802.41	38.2	138.6	87,910.24	6.1	151,420.99	Utah
Vermont	438,880				327,772.79	309,469.49	21.3	28.8	110,432.98	7.0	18,977.53	Vermont
Virginia	1,854,189	28,000.00	28,000.00	20.5	1,407,746.40	1,339,583.75	36.7	178.3	366,594.40	14.8	120,010.85	Virginia
Washington	1,180,362	49,041.41	49,041.41	11.7	933,440.79	923,440.79	17.9	46.0	49,953.47	7.4	157,926.33	Washington
West Virginia	1,118,559				397,153.15	397,153.15	17.4	23.0	622,835.20	36.7	98,570.65	West Virginia
Wisconsin	2,431,220	26,056.01	26,056.01	.6	1,460,663.83	1,421,229.11	22.4	95.0	471,368.93	43.4	512,565.95	Wisconsin
Wyoming	1,125,332	32,972.04	31,300.00	4.1	705,187.44	690,965.00	39.1	104.9	259,453.20	30.3	143,613.80	Wyoming
District of Columbia	959,234	116,450.00	116,450.00	2.0	681,145.93	658,254.90		4.4	176,689.50	1.4	7,839.60	District of Columbia
Hawaii	187,106								180,042.06	4.9	7,063.94	Hawaii
TOTALS	94,758,143	2,010,402.83	1,954,458.48	352.6	51,759,584.81	49,466,686.05	25.7	5,485.6	17,930,215.09	1,595.4	25,406,783.38	TOTALS

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, 1924. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1927. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1928. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1929. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1932. 10 cents.

DEPARTMENT BULLETINS

No. 136D . . Highway Bonds. 20 cents.

No. 347D . . Methods for the Determination of the Physical Properties of Road-Building Rock. 10 cents.

No. 532D . . The Expansion and Contraction of Concrete and Concrete Roads. 10 cents.

No. 583D . . Reports on Experimental Convict Road Camp, Fulton County, Ga. 25 cents.

No. 660D . . Highway Cost Keeping. 10 cents.

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

TECHNICAL BULLETINS

No. 55T . . Highway Bridge Surveys. 20 cents.

No. 265T . . Electrical Equipment on Movable Bridges. 35 cents.

MISCELLANEOUS CIRCULARS

No. 62MC . . Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects. 5 cents.

No. 93MC . . Direct Production Costs of Broken Stone. 25 cents.

MISCELLANEOUS PUBLICATION

No. 76MP . . The results of Physical Tests of Road-Building Rock. 25 cents.

No. ——— . . Federal Legislation and Regulations Relating to Highway Construction. 10 cents.

REPRINT FROM PUBLIC ROADS

Reports on Subgrade Soil Studies. 40 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.

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).

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.

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION

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

SUMMARY OF CLASSES I, II, AND III

AS OF FEBRUARY 28, 1934

STATE	TOTAL APPORTIONMENT OF PUBLIC WORKS FUNDS	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR ALL NEW PROJECTS
		Total cost	Public works funds	Regular Federal aid	Mileage	Estimated total cost	Public works funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	Public works funds allotted	Mileage	
Alabama	$ 8,370,133	$ 29,479.63	$ 29,479.63		.8	$ 5,381,944.78	$ 3,148,189.36	$ 2,233,755.42	32.7	290.1	$ 616,245.67	45.3	$ 4,576,218.34
Arizona	5,211,960	310,015.34	310,015.34		22.1	3,859,588.43	3,331,536.45		31.7	266.1	164,893.87	3.7	1,405,514.34
Arkansas	6,748,335	18,339.52	9,849.52	$ 8,490.00	.6	2,579,670.38	2,048,683.63	530,986.75	32.4	125.4	1,433,651.58	108.7	3,256,150.27
California	15,607,354	481,241.28	399,287.05		10.6	12,236,496.08	9,649,566.07		26.4	390.5	2,308,261.75	54.7	3,250,239.13
Colorado	6,874,530	906,673.32	906,673.32		68.9	2,682,738.32	2,672,157.97		38.9	181.2	870,077.94	40.1	2,425,620.77
Connecticut	2,865,740	52,696.86	52,696.86		.5	2,326,048.13	2,144,473.61	175,778.94	10.0	48.0	602,035.45	8.7	66,534.08
Delaware	1,319,088	208,841.80	208,841.80		16.3	892,415.70	892,415.70		18.9	18.4	212,264.00	2.6	505,566.50
Florida	5,231,834	176,839.08	97,261.49	79,577.59	6.4	4,766,006.29	3,856,934.46	908,461.73	33.6	174.6	1,016,984.97	20.9	260,653.08
Georgia	10,091,185	158,108.86	158,108.86		6.6	2,964,486.47	2,964,486.47		32.2	172.8	1,364,612.19	90.4	5,603,977.48
Idaho	4,486,249	373,934.38	368,385.16		44.3	2,486,150.32	2,436,660.44		40.7	218.1	228,652.33	17.5	1,452,551.07
Illinois	17,570,770	124,536.93	124,536.93		19.8	5,768,388.08	5,768,388.08		14.0	209.0	5,796,248.77	151.5	5,881,596.22
Indiana	10,037,843	24,698.54	24,698.54		.4	2,181,957.43	2,181,957.43		31.2	110.3	2,498,697.97	69.8	5,332,489.06
Iowa	10,055,660	1,035,387.29	1,029,750.00		65.9	5,555,716.41	5,176,045.00		27.9	291.9	1,427,300.00	87.6	2,422,565.00
Kansas	10,089,604	379,175.95	379,175.95		74.1	6,244,397.58	6,151,684.19		26.2	418.2	3,517,018.61	108.4	41,725.25
Kentucky	7,517,359	172,771.15	172,771.15		15.0	3,170,269.09	3,170,269.09		30.4	294.0	2,229,642.92	158.9	1,944,675.84
Louisiana	5,828,591	73,243.66	73,243.66		1.2	2,794,459.55	2,287,808.55		25.5	61.7	1,192,452.44	55.2	2,275,086.35
Maine	3,369,917	544,720.65	543,485.17		57.9	1,936,298.39	1,843,409.34		45.2	84.3	563,913.48	13.4	419,109.01
Maryland	3,564,527					1,029,866.61	1,016,067.86		10.1	31.0	410,405.85	29.8	2,138,053.29
Massachusetts	6,597,100	207,833.37	114,849.13	92,984.24	5.6	4,781,108.65	4,451,349.01	329,759.64	17.2	60.8	314,723.69	1.8	1,716,178.17
Michigan	12,736,227	332,200.00	332,200.00		11.4	5,662,070.00	5,659,920.00		18.9	305.0	4,713,262.00	177.1	2,030,845.00
Minnesota	10,656,569	1,666,455.35	1,666,455.35		332.2	4,318,585.45	4,238,154.43		40.1	534.5	1,195,120.33	154.4	3,556,838.89
Mississippi	6,978,675					4,007,673.62	2,498,032.95	1,471,008.73	22.1	225.2	935,990.09	96.8	3,544,651.96
Missouri	12,180,306	341,249.47	241,300.85		24.4	7,316,605.16	6,963,992.13	50,048.60	34.3	531.9	2,348,688.07	147.6	2,626,324.95
Montana	7,439,748	303,580.42	303,580.42		50.1	4,823,537.63	4,550,658.75	192,878.88	31.5	347.1	1,618,907.08	161.3	966,601.75
Nebraska	7,828,961	410,349.09	283,998.37		148.4	6,685,850.54	5,805,359.89	108,538.00	36.1	446.9	1,497,573.11	101.1	242,029.63
Nevada	4,545,917	322,539.87	322,539.87		61.5	2,473,676.89	2,473,676.89		33.5	228.5	758,995.53	42.1	990,704.71
New Hampshire	1,909,839					1,419,316.65	1,372,807.80		25.6	42.4	447,517.22	10.6	89,513.98
New Jersey	6,346,039	83,052.61	83,052.61		1.0	3,047,019.07	3,047,019.07		17.4	32.6	2,202,032.73	25.4	1,013,934.59
New Mexico	5,792,935	1,092,305.73	1,092,305.73		144.0	3,435,719.49	3,299,360.01	102,921.33	22.4	829.1	535,629.58	84.6	865,639.68
New York	22,330,101	391,150.00	391,150.00		10.6	18,448,593.19	16,918,208.19	315,000.00	21.7	340.3	3,735,395.00	46.3	1,285,347.81
North Carolina	9,522,293	765,338.24	518,754.79	246,583.45	67.2	3,127,433.12	2,822,008.85	305,424.27	28.4	304.2	1,901,255.01	267.0	4,280,274.35
North Dakota	5,804,448	508,839.69	508,836.18		259.9	923,797.95	923,797.95		17.2	234.5	1,699,405.49	411.4	2,672,408.38
Ohio	15,484,592	462,570.00	455,095.00		52.1	9,749,509.43	9,376,783.33		29.7	394.6	4,641,700.00	103.8	1,011,013.67
Oklahoma	9,216,798	214,246.80	214,246.80		9.0	4,035,958.25	4,035,958.25		30.8	241.0	2,167,071.23	174.3	2,799,521.72
Oregon	6,106,896	374,188.66	346,295.16		41.0	3,883,116.57	3,702,901.50	72,821.40	22.8	153.0	1,449,981.25	100.4	607,718.09
Pennsylvania	18,891,004	166,033.86	166,033.86		8.6	11,240,739.56	11,176,999.74		20.9	705.3	4,138,337.06	108.1	3,409,633.34
Rhode Island	1,998,708	18,830.63	18,830.63		.6	1,196,640.03	1,196,640.03		22.2	29.4	326,085.92	20.9	457,151.42
South Carolina	5,459,165	29,088.37	29,088.37		1.7	3,336,885.19	3,333,750.56	3,134.63	29.6	320.9	420,881.59	28.8	1,675,444.48
South Dakota	6,011,479	986,251.27	980,334.78	5,916.49	165.7	1,303,145.05	1,271,120.76	32,024.29	14.4	197.9	890,474.91	130.9	2,869,548.55
Tennessee	8,492,619	151,879.70	92,583.06	59,296.64	6.3	3,913,045.54	3,396,324.80	516,720.74	25.8	166.8	2,370,854.87	105.0	2,632,856.27
Texas	24,244,024	819,674.90	634,314.36		184.3	14,586,349.38	12,635,968.90	—	30.3	1,295.3	4,500,573.35	365.5	6,473,167.39
Utah	4,194,708	1,321,202.87	1,320,898.85		122.1	1,976,106.60	1,972,048.65		34.3	224.4	366,473.53	18.7	535,286.97
Vermont	1,867,573	51,837.88	51,837.88		6.8	1,361,912.96	1,314,228.79		26.3	71.7	283,678.13	11.1	217,828.20
Virginia	7,416,757	239,547.73	212,712.98	26,834.75	32.7	4,118,432.26	3,846,217.99	412.31	33.9	250.6	2,887,460.95	112.6	470,365.08
Washington	6,115,867	431,137.39	431,137.39		30.7	3,987,355.84	3,969,283.42	3,356.82	20.7	148.6	1,076,806.00	27.7	638,640.19
West Virginia	4,474,234	210,205.25	210,205.25		6.8	2,251,997.36	2,227,651.62		30.3	92.2	1,404,002.63	47.5	632,374.50
Wisconsin	9,724,881	476,062.86	465,760.03	1,190.00	21.1	4,679,208.79	4,617,674.54	21,500.00	32.8	212.0	1,816,404.13	123.7	2,825,042.30
Wyoming	4,501,327	766,985.94	677,457.69	60,000.00	123.8	2,661,257.15	2,493,199.55	18,000.00	24.8	440.1	721,153.93	48.9	609,515.83
District of Columbia	1,918,469	411,960.62	411,960.62		4.4	1,251,874.93	1,228,983.90		16.1	6.1	269,610.65	2.0	7,913.83
Hawaii	1,871,062	25,988.65	12,994.33	12,994.32	.9	1,429,747.99	1,197,920.88	181,827.36	11.4	28.6	366,646.36	8.2	293,500.43
TOTALS	394,000,000	18,653,291.46	17,479,070.77	593,867.48	2,346.3	216,291,168.33	198,758,736.83	7,574,359.84	27.0	12,827.1	80,456,051.21	4,332.8	97,306,141.19

U. S. GOVERNMENT PRINTING OFFICE: 1934