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THE SUBGRADE FOR SECTION 30, COLUMBIA PIKE EXPERIMENTAL ROAD

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TABLE OF CONTENTS

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REINFORCING AND THE SUBGRADE AS FACTORS IN THE DESIGN OF CONCRETE PAVEMENTS

A STUDY OF EXPERIMENTAL SECTIONS OF THE COLUMBIA PIKE

By J. T. PAULS, Associate Highway Engineer, United States Bureau of Public Roads

OBSERVATIONS of the Columbia Pike, an experimental road constructed by the United States Bureau of Public Roads near Arlington, Va., after two and a half years' service under traffic have made possible a number of conclusions with regard to the relation between the cracking of concrete roads and the character of the subgrade and steel reinforcing. The observations also reveal a number of interesting facts with reference to the water-holding properties of the soils composing the subgrades and the relation between these properties and resulting changes in volume.

It is found that subgrade materials with a large percentage of clay not only attain a high moisture content during the wet season but retain a high content during the dry season. Materials of this character subjected to the laboratory test for moisture equivalent will be found to have a high moisture equivalent value. Subgrades having a large percentage of sand do not attain high moisture content. It is also found that subgrade materials composed largely of very fine sand have high capillarity and that under this condition free water will very often be found between the pavement and the subgrade.

Subgrades composed largely of clay swell and contract as moisture is added or taken away. The effect of this in the case of swelling is to lift the pavement at the edges and on contraction to take away the support at the edges. The result is that the slab, acting in one case as a simple beam and in the other as a cantilever, is broken at the center by traffic. The conclusion is drawn from the observations that subgrades that show as much as 10 per cent volume change, by laboratory test on an entire sample including coarse material, should be covered with a layer of coarse granular material, and a pavement laid on a subgrade of this character should have a longitudinal joint at the center. Longitudinal cracks in a pavement indicate an unstable subgrade either as to supporting value or movement caused by moisture changes. Adding a granular material to such a subgrade increases its supporting value and modifies the effect of any volume change. Pavements on this type of subgrade should be designed with a center joint.

The observations made indicate that plain concrete slabs will crack transversely because of temperature and moisture changes at intervals of from 40 to 60 feet. Smooth subgrade surfaces increase the distances between cracks, but the thickness of the concrete does not affect the spacing of the contraction cracks.

Judging by the experience with the experimental sections, pavements reinforced longitudinally will develop transverse contraction cracks, the number, spacing, and size of which will be controlled by a number of factors. If the steel reinforcing is not continuous but is separated by joints, it is to be expected that no cracks will form less than 30 feet from any joint, and by a suitable relation of the percentage of steel to the length over which the steel is made continuous the distance may be increased to 60 feet. The position of the cracks will be influenced by the strength of the concrete and the roughness of the subgrade as well as by the percentage and continuous length of the steel. If the spacing of the joints is less than twice the distance in which a crack would form, contraction cracking may be entirely prevented. If the distance between joints is extended, a crack may be anticipated at a distance of 30 to 60 feet from each joint and the area between these cracks may be expected to crack at relatively short intervals. With a high percentage of continuous steel relatively fine, closely spaced cracks may be looked for; with a low percentage breaks in the steel may be expected to permit wider cracks to form at considerable intervals. Mesh reinforcing in amounts as used in these tests is likely to break at intervals and permit open cracks to form.

Attention should be called to the possible danger of the use of too high a percentage of longitudinal steel, for under such conditions numerous fine transverse cracks will develop and there is the possibility that the narrow transverse beams thus formed will crack under traffic.

From the results obtained on sections reinforced longitudinally it would appear that the practice of omitting contraction joints in pavements of this type is questionable. It would appear that where longitudinal steel is used the design should provide for contraction joints from 50 to 100 feet apart with the steel designed to prevent intermediate contraction cracks from forming. Another method that would probably be more satisfactory to the contractor, but which might be subject to other objections, would be to make the concrete continuous and to break the steel one-half inch or 1 inch at intervals where it is desired that contraction cracks shall form.

These conclusions will, no doubt, be modified in certain respects and in others made more conclusive as time permits the action of the various factors to become more effective. The results obtained thus far show conclusively the great importance of subgrade investigations in connection with the design of a pavement. Since the character of the subgrade may vary widely in a comparatively short section of road, it seems logical that the design should be modified accordingly so as best to meet the particular subgrade conditions existing, or that the subgrade should be so corrected as to make it suitable for a uniform design of pavement.

THE CHARACTER OF EXPERIMENTAL SECTIONS

The Columbia Pike includes 32 experimental concrete surfaces of an average length of 200 feet, varying in such features as the thickness, reinforcing, and cross section of the slabs. The nine types of cross section used are shown in Figure 1, the principal dimensions and characteristics of each in Table 1. All sections were made of Potomac River aggregate and cement in the proportion of 1 : 2 : 3.

Generally speaking, the subgrade was very stable. The old macadam construction, loosened, reshaped and compacted, served as the base of some of the sections; others, on account of changes in grade, were laid directly on the earth, with the exception of two sections, Nos. 9 and 28, which were laid on a base constructed of cinders. Sections 30, 31, and 32 were built on a heavy fill in new location. The nature of the soil underlying each of the sections is represented in detail by the analyses of samples taken every 50 feet at the time of construction. These analyses are reported in Table 2.

Capped pipes were embedded in the concrete at intervals when the concrete was placed, and at various times since the pavements were completed samples of the subgrade material have been extracted through these pipes for determination of the moisture content. The results of these determinations are recorded in Table 3.

Besides making these moisture determinations, the condition of the pavement has been observed and a record has been kept of the cracks that have formed. The results of these observations are recorded in Figures 2, 3, and 4, which show, in addition to the record of cracks, the average, maximum, and minimum moisture determinations and the analyses of the subgrade materials.

The general condition of the experimental sections at this time is good. Certain sections, such as 1 and 29 between stations 101 and 103 and stations 105 and 108, which have cracked badly, may give trouble later. Section 9, a 4-inch plain concrete section on a cinder base, is badly cracked over a small area in the vicinity of a joint. The slipping of the concrete at this joint, due to the fact that the joint had been placed off the vertical at the time of construction, may be held responsible for the failure of this section. Other than in these cases the experimental sections are in such condition as to require only the regular periodic joint and crack repair customary on this type of road.

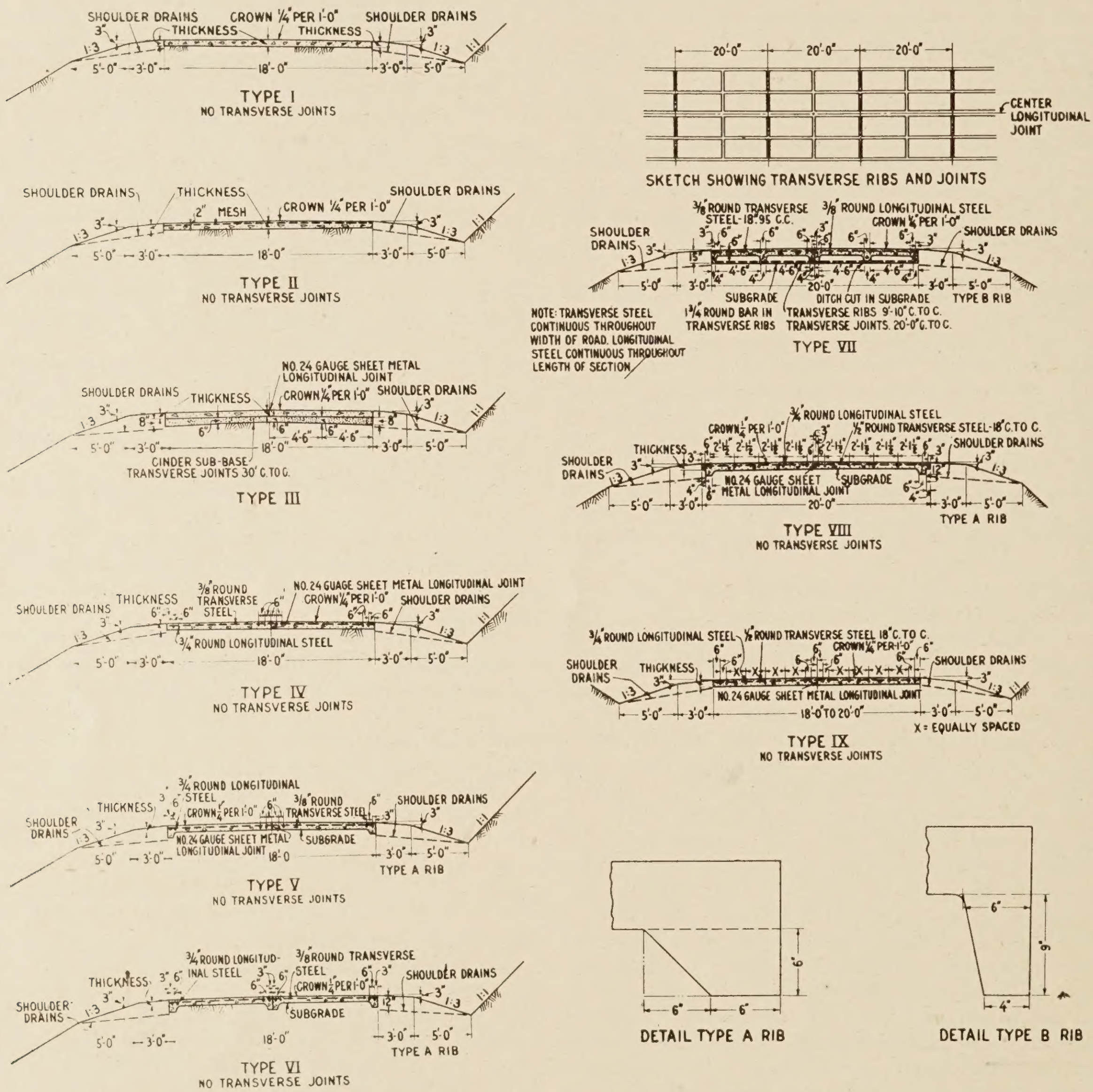


FIG. 1.—The types of cross section and characteristics of sections of the Columbia Pike

Table 1.—*Dimensions and characteristics of experimental sections*

Section No.	Station	Type	Thickness	Reinforcement		Joints		Concrete ribs		Constructed base
				Longitudinal	Transverse	Longitudinal	Transverse	Longitudinal	Transverse	
			Inches							
1	27+55 to 29+55	I	6–7	None	None	None	None	None	None	None.
2	29+55 to 31+55	II	6	None	None	None	None	None	None	None.
3	31+55 to 33+55	II	7	None	None	None	None	None	None	None.
4	33+55 to 35+55	II	8	None	None	None	None	None	None	None.
5	35+55 to 39+55	I	6–7	None	None	None	None	None	None	None.
6	39+55 to 41+55	II	6	25 pounds per 100 square foot mesh		None	None	None	None	None.
7	41+55 to 43+55	II	6	50 pounds per 100 square foot mesh		None	None	None	None	None.
8	43+55 to 45+51	I	7–8	None	None	None	None	None	None	None.
9	45+51 to 47+55	III	4	None	None	Center	30 feet center to center.	None	None	6–8 inch cinder.
10	47+55 to 49+55	IV	6	None	None	do	None	None	None	None.
11	49+55 to 51+55	I	8–9	None	None	None	None	None	None	None.
12	51+55 to 53+55	IV	7	None	None	Center	None	None	None	None.
13	53+55 to 55+55	IV	8	None	None	do	None	None	None	None.
14	55+55 to 59+55	I	6–7	None	None	None	None	None	None	None.
15	59+55 to 61+55	IV	6	8 ½-inch round rods.	⅜-inch round rods, 12 inches center to center.	Center	None	None	None	None.
16	61+55 to 63+55.5	IV	6	do	⅜-inch round rods, 18 inches center to center.	do	None	None	None	None.
17	63+55.5 to 65+56.5	IV	6	do	⅜-inch round rods, 24 inches center to center.	do	None	None	None	None.
18	65+56.5 to 67+56.5	IV	6	4 ½-inch round rods.	None	do	None	None	None	None.
19	67+56.5 to 69+56.5	IV	6	4 ¾-inch round rods.	None	do	None	None	None	None.
20	69+56.5 to 71+56.5	IV	6	4 1-inch round rods.	None	do	None	None	None	None.
21	71+56.5 to 73+56.5	V	6	None	None	do	None	2 Type A	None	None.
22	73+56.5 to 75+56.5	V	8	None	None	do	None	do	None	None.
23	75+56.5 to 77+56.5	V	6	8 ¾-inch round rods.	None	do	None	do	None	None.
24	77+56.5 to 79+57	V	6	do	⅜-inch round rods, 18 inches center to center.	do	None	do	None	None.
25	79+57 to 81+50	VI	6	None	None	do	None	4 Type A	None	None.
26	81+50 to 83+50	VI	6	8 ¾-inch round rods	None	do	None	do	None	None.
27	83+50 to 85+50	VI	6	do	⅜-inch round rods, 18 inches center to center.	do	None	do	None	None.
28	85+50 to 87+00	III	6	None	None	do	30 feet center to center.	None	None	6–8 inch cinder
29	87+00 to 108+00	I	6–7	None	None	None	None	None	None	None.
30	108+00 to 110+00	VII	6	Shown on plan, Fig. 1.		Center	20 feet center to center.	6 type B	9 feet 10 inches center to center.	None.
31	110+00 to 112+00	VIII	6	14 ¾-inch round rods.	½-inch round rods, 18 inches center to center.	do	None	2 type A	None	None.
32	112+00 to 115+50	IX	6	do	do	do	None	None	None	None.

CHARACTERISTICS OF STEEL REINFORCING BARS

The steel reinforcing bars used in the various reinforced sections were deformed round bars made of billet steel for concrete reinforcement, intermediate grade, conforming to A. S. T. M. standard specification A15–14. The mesh reinforcement consisted of main members held together by secondary members at right angles to and twisted about the main members.

The 1-inch rods used were not tested. Tests of the three-eighths, one-half, and three-fourths inch rods revealed the characteristics shown in the following table:

Size of rod	Area of cross section	Ultimate strength	Elongation in 8 inches	Yield point
	Square inches	*Pounds per square inch*	*Per cent*	*Pounds per square inch*
⅜-inch	0.11	72,750	22.6	49,100
½-inch	.20	75,550	22.6	47,750
¾-inch	.44	77,950	25.0	45,500

THE AMOUNT AND EFFECT OF SUBGRADE MOISTURE

Marked variations were observed in the moisture content of the subgrade under the various sections and in the length of time that high percentages of moisture were retained after the advent of dry weather. The highest percentage of moisture recorded was 43 per cent between stations 105 and 108, while the lowest percentage observed at the same time was 8.2 per cent in section 16. The highest moisture content found during a dry period was 28 per cent in section 11 and the lowest observed at the same time was 5.5 per cent in section 29. These figures show that the subgrade in some locations may be almost free of moisture while at the same time the subgrades in other sections may be almost saturated. This occurs in portions of the subgrade where conditions other than the subgrade materials are the same. In view of the change in the supporting value of certain subgrade materials on addition of moisture, it would seem very important to determine how those subgrades compare in their supporting value with other materials that do not attain such high moisture content.

The subgrade materials which show high moisture content are found to be those materials which have a high percentage of clay. Sections 4, 10, 11, 32, 31, 30, and section 29, between stations 89–91, 93–99, and 103–108, have subgrades of this type. Moisture determinations on these subgrades not only show high content during the wet season but also show a large amount retained during the dry season.

TABLE 2.—*Analyses of subgrade materials—Samples obtained from the top 6-inch layer of finished grade along the center line of the pavement*

Section No.	Type	Station	Mechanical analysis[1]: Coarse material: Above ¼-inch	Coarse material: ¼-inch–10 mesh	Sand: 10–20 mesh	Sand: 20–60 mesh	Sand: 60–100 mesh	Sand: 100–200 mesh	Silt	Clay	Moisture equivalent	Capillary moisture	Volume change caused by capillary moisture	Adsorption number
			Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	
1	I	28+00	18.5	4.8	4.9	20.2	6.4	6.4	13.4	25.5	9.2	18.0	10.3	17.0
		28+50	67.9	5.8	3.4	10.3	2.7	2.8	4.0	3.0	1.6	4.6	0.0	2.0
		29+00	35.6	29.8	7.2	6.8	1.8	2.8	6.4	9.6	4.7	9.2	4.1	12.0
		29+50	25.3	13.9	2.8	23.5	11.4	6.0	4.9	12.1	6.3	15.9	4.3	10.0
2	II	30+00	6.8	11.0	5.7	24.6	9.8	9.0	14.7	18.4	7.6	16.9	5.4	9.0
		30+50	9.4	11.2	19.8	41.0	1.9	1.6	4.5	10.4	5.4	11.1	0.0	3.0
		31+50	51.6	20.9	2.8	9.8	2.2	3.0	4.5	5.2	4.3	10.3	2.5	9.0
3	II	32+50	56.3	9.2	4.7	11.4	4.3	4.2	6.4	3.5	1.3	5.0	0.0	3.0
		33+00	38.2	21.1	3.2	15.5	4.6	5.8	6.8	4.7	2.1	7.7	0.0	4.0
4	II	33+50	36.0	9.9	6.0	23.3	4.5	4.0	6.8	9.5	4.6	10.6	0.0	7.0
		34+00	11.0	8.6	11.5	29.8	7.1	6.2	12.9	12.8	6.7	17.9	0.0	9.0
		34+50	6.0	8.3	4.5	23.3	7.3	10.5	22.5	17.5	10.7	18.2	0.0	10.0
		35+00	20.8	4.4	4.3	18.0	7.8	8.8	21.5	14.4	8.1	17.6	0.0	11.0
5_1	I	36+00	28.2	11.9	9.8	23.4	8.5	8.7	6.7	2.8	1.4	8.9	0.0	2.0
		37+00	46.6	5.3	5.2	20.7	5.3	4.9	6.7	5.2	2.4	8.9	0.0	4.0
5_2	I	38+50	42.6	7.6	6.9	24.6	6.6	6.1	3.9	1.8	0.9	8.5	0.0	2.0
		39+50	53.6	7.1	4.2	12.4	4.6	4.4	7.7	6.1	2.4	5.7	0.0	3.0
6	II	40+00	35.4	9.8	5.1	17.5	5.5	6.7	10.8	9.4	3.7	9.5	0.0	6.5
		40+50	50.5	5.2	2.5	12.1	4.9	4.1	9.9	10.8	4.1	7.5	0.0	5.5
		41+00	23.8	6.2	4.6	22.6	7.8	9.7	14.3	11.1	5.6	16.7	0.0	9.0
		41+50	28.8	9.1	5.2	20.7	6.3	7.5	11.9	10.5	4.2	10.5	0.0	6.0
7	II	43+50	37.3	9.1	5.3	16.5	4.4	3.4	5.7	18.3	7.7	13.1	5.1	11.0
8	II	44+00	35.1	4.9	3.0	14.3	5.6	6.1	14.3	16.7	6.8	10.3	0.0	4.0
		44+50	22.0	6.2	4.8	27.1	9.6	10.6	12.3	7.4	2.7	9.8	0.0	2.0
		45+00	30.6	5.3	4.4	20.9	8.4	6.8	11.0	12.5	4.2	11.5	0.0	4.0
		45+50	39.4	9.1	5.5	18.7	5.1	6.2	8.4	7.7	3.1	7.8	0.0	6.0
10	IV	48+00	30.0	6.5	2.9	13.5	5.1	5.7	11.5	24.7	9.8	14.9	8.3	10.0
		48+50	16.7	4.8	3.4	16.8	7.4	6.7	14.6	29.6	11.7	18.5	9.8	13.0
		49+00	7.1	2.8	2.8	12.7	7.0	9.4	14.7	43.6	18.5	27.7	15.7	22.0
11	I	49+50	6.9	6.2	5.0	26.2	6.3	7.5	14.3	27.5	11.7	21.9	9.8	16.0
		50+00	21.4	6.1	2.9	15.6	6.8	7.2	13.7	26.3	10.9	18.0	10.0	13.0
		50+50	26.7	8.1	4.1	16.1	6.2	8.1	11.6	19.0	9.3	15.1	5.9	14.0
12	IV	52+00	18.1	5.6	5.2	20.8	8.2	8.6	17.4	16.1	9.9	18.4	5.2	12.0
		52+50	8.2	11.3	10.1	30.0	8.7	10.9	12.7	8.0	4.8	14.6	0.0	8.0
13	IV	54+50	27.4	5.2	5.4	22.6	8.4	7.4	11.7	11.9	5.9	12.7	0.0	6.0
		55+00	31.2	5.7	4.1	19.4	7.8	6.4	10.3	15.1	5.6	12.6	3.5	9.0
		55+50	29.8	6.5	5.0	20.6	5.2	7.2	9.7	16.0	6.2	12.7	4.6	12.0
14_1	I	56+00	28.9	5.2	3.8	24.8	6.6	6.9	9.3	14.4	7.1	15.9	5.9	12.0
		56+50	3.2	0.2	0.5	48.0	9.0	6.9	9.5	22.8	19.0	40.0	12.7	25.0
		57+00	17.7	2.8	2.6	35.9	7.1	5.7	9.3	18.9	12.2	21.9	9.1	27.0
14_2	I	58+50	37.8	8.6	8.3	19.0	5.5	5.2	8.0	7.5	14.2	8.7	0.0	8.0
		59+00	30.4	8.9	8.7	24.2	7.2	6.5	8.1	6.1	2.8	9.5	0.0	7.0
15	IV	59+50	27.4	6.6	5.6	27.2	5.8	6.2	8.1	12.9	7.6	13.2	3.9	16.0
16	IV	62+50	21.1	8.7	6.0	23.4	7.3	8.7	9.7	15.0	6.9	11.5	0.0	11.5
17	IV	64+50	31.8	9.2	3.8	20.5	4.5	4.7	10.5	15.0	6.4	9.8	6.6	8.0
		65+50	29.6	7.6	4.5	25.8	6.8	5.7	9.6	10.4	4.1	11.6	0.0	6.0
18	IV	66+00	19.6	7.8	4.5	29.6	7.7	7.5	12.0	11.2	5.1	12.9	0.0	6.0
		66+50	15.8	7.7	6.2	31.0	6.5	7.7	11.9	13.3	5.4	12.8	0.0	8.0
		67+00	18.6	9.9	5.8	26.4	7.8	7.2	13.5	10.7	5.5	12.2	0.0	3.0
		67+50	32.6	12.0	6.7	22.4	4.2	5.2	8.2	8.5	4.8	8.7	0.0	8.0
19	IV	68+00	16.9	7.2	4.2	24.3	9.8	13.2	10.6	13.8	6.5	11.7	0.0	9.0
		68+50	5.8	1.9	2.9	25.0	15.4	17.1	14.5	17.4	6.3	18.5	0.0	10.0
		69+00	9.5	3.5	2.6	26.9	13.4	13.4	15.6	15.1	6.7	12.8	0.0	8.0
		69+50	10.8	3.5	3.0	24.9	9.7	9.5	16.3	22.4	8.0	13.9	4.6	9.0
20	IV	70+00	8.8	4.2	2.2	31.8	12.9	16.1	11.1	12.7	4.4	11.5	0.0	6.0
		70+50	21.7	8.0	5.8	29.1	7.3	6.2	10.4	11.5	5.5	12.7	0.0	7.0
		71+00	21.6	7.4	5.7	27.2	6.8	7.7	11.6	11.9	5.4	13.8	0.0	9.9
		71+50	12.4	3.5	2.3	18.7	4.6	5.6	22.2	30.5	10.3	14.5	5.5	9.9
21	V	72+00	17.6	2.1	1.6	17.5	5.4	6.4	25.9	23.4	7.8	13.6	0.0	4.0
		72+50	18.1	5.8	4.6	26.8	9.2	12.0	13.4	10.0	22.6	11.0	0.0	3.0
		73+00	9.0	6.3	6.8	31.9	8.6	10.9	16.0	10.3	4.4	13.7	0.0	4.0
		73+56.5	33.8	8.2	5.0	21.8	6.4	6.1	9.4	9.2	4.0	11.1	1.8	8.0
22	V	74+00	12.8	6.4	3.9	25.0	10.8	10.4	13.8	16.9	6.7	15.1	0.0	7.0
		74+50	10.7	4.6	2.8	24.4	13.7	14.6	12.4	16.9	7.7	15.7	5.1	9.0
		75+00	10.6	5.7	4.0	26.5	11.0	16.1	11.9	14.2	5.0	18.5	0.0	8.0
		75+50	32.6	8.2	4.8	28.4	5.1	3.9	6.5	10.4	5.4	8.0	0.0	8.0
23	V	76+00	3.3	1.5	1.3	27.0	19.3	20.8	14.5	12.1	6.4	17.1	0.0	9.0
		76+50	1.5	1.1	0.9	26.5	20.3	21.8	13.3	14.6	4.6	20.1	0.0	9.0
		77+00	2.6	1.0	0.7	26.5	16.4	23.4	13.2	16.1	7.6	16.2	0.0	9.0

TABLE 2.—*Analyses of subgrade materials—Samples obtained from the top 6-inch layer of finished grade along the center line of the pavement*—Continued

Section No.	Type	Station	Mechanical analysis[1]: Coarse material: Above ¼-inch	Coarse material: ¼-inch–10 mesh	Sand: 10–20 mesh	Sand: 20–60 mesh	Sand: 60–100 mesh	Sand: 100–200 mesh	Silt	Clay	Moisture equivalent	Capillary moisture	Volume change caused by capillary moisture	Adsorption number
			Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	
24	V	77+50	4.4	0.8	0.8	18.4	13.8	20.3	14.8	26.7	9.8	26.0	10.9	12.0
		78+00	8.0	8.2	7.2	31.5	11.3	10.8	14.6	8.2	3.1	22.3	0.0	3.0
		78+50	30.8	10.6	6.2	23.6	5.2	5.9	8.2	9.5	3.9	15.1	0.0	7.0
		79+00	22.7	6.5	5.4	28.8	7.8	7.1	10.4	11.3	4.7	18.0	0.0	6.0
		79+50	32.2	9.1	5.5	21.9	5.5	6.5	9.8	9.5	3.2	14.7	0.0	6.0
25	VI	80+00	4.8	4.2	5.6	32.0	11.6	11.4	19.5	11.0	4.8	21.7	0.0	3.0
		80+50	15.0	5.4	3.3	25.1	9.9	9.2	16.4	15.7	6.6	20.5	0.0	8.0
		81+00	4.4	2.1	2.4	20.5	9.9	11.4	23.0	26.1	9.7	22.1	0.0	7.0
		81+50	22.4	8.2	5.6	25.8	6.6	8.1	9.2	14.0	5.0	15.9	0.0	4.0
26	VI	82+00	8.1	3.7	2.6	17.5	8.7	10.1	20.5	28.8	9.9	21.0	8.6	9.0
		82+50	0.0	0.3	0.4	9.7	4.9	8.3	31.0	45.4	15.5	28.4	11.5	16.0
		83+00	0.0	0.3	0.5	14.7	7.9	11.5	30.9	34.2	12.0	22.7	7.8	12.0
27	VI	83+50	0.0	2.1	0.3	13.4	6.5	9.7	23.2	44.8	17.2	30.4	16.5	28.0
		84+00	0.0	0.4	0.8	15.2	7.1	8.4	20.1	48.0	16.7	27.4	12.0	21.0
		84+50	2.3	1.0	0.9	17.0	8.3	10.4	27.8	32.2	9.6	23.3	12.8	9.5
		85+00	9.2	4.1	2.8	15.6	6.0	7.0	25.2	30.2	7.8	27.1	11.9	8.5
29_1	I	88+00	21.4	7.3	6.2	29.2	7.6	8.9	11.1	8.3	2.5	17.7	0.0	4.0
		88+50	7.0	6.0	7.1	27.8	8.6	10.2	18.1	15.2	6.7	25.1	0.0	11.0
29_2	I	89+00	30.0	5.8	4.8	22.5	6.0	6.6	10.9	13.2	5.1	17.9	7.0	11.0
		89+50	15.9	5.9	5.7	24.8	7.1	9.0	16.6	14.9	6.3	22.8	7.5	10.5
		90+00	8.3	8.2	8.5	33.2	8.0	9.1	14.6	10.2	3.5	22.6	0.0	6.5
		90+50	1.6	0.3	0.7	17.0	8.5	9.2	20.7	42.0	13.7	35.9	12.3	18.0
		91+00	32.0	6.0	2.6	18.8	5.7	5.2	9.8	19.9	7.3	20.2	7.1	12.0
29_3	I	91+50	35.8	4.5	2.7	23.4	5.4	3.8	9.1	15.1	6.5	13.8	4.7	11.0
		92+00	31.2	10.3	5.9	21.9	4.5	4.6	7.0	14.4	4.7	15.8	0.0	11.0
		92+50	28.5	4.9	3.3	21.4	6.3	5.1	12.1	18.3	5.8	19.7	6.1	10.4
29_4	I	93+00	29.0	5.8	3.5	18.9	5.5	5.4	12.2	19.6	7.4	15.8	8.9	13.0
		93+50	27.6	4.0	2.5	20.4	5.8	5.4	13.1	21.2	8.3	17.7	9.4	11.0
		94+00	19.7	2.2	1.4	19.1	6.8	6.8	17.2	26.7	10.7	17.6	7.0	14.0
		94+50	34.8	3.1	1.8	18.6	6.6	5.5	10.3	19.2	8.0	17.3	7.8	17.0
29_5	I	95+00	26.5	8.1	7.7	24.2	7.1	7.4	12.0	6.9	2.6	13.8	0.0	7.0
		95+50	6.4	4.2	5.9	27.7	7.1	7.6	19.5	21.8	9.2	24.5	6.9	11.0
		96+00	7.6	4.5	11.1	29.6	8.8	12.5	17.5	8.4	4.0	20.3	0.0	7.5
		96+50	34.8	6.2	4.2	13.8	3.0	3.5	12.7	21.8	7.7	14.8	5.6	12.0
		97+00	41.9	4.4	3.9	14.4	3.7	3.9	11.3	16.5	6.6	15.4	5.6	10.0
29_6	I	97+50	29.7	6.8	5.5	20.9	3.5	4.3	10.9	18.2	6.3	15.0	5.9	8.0
		98+00	40.3	4.6	3.7	19.7	3.9	3.6	10.4	13.7	5.8	12.7	3.2	8.0
		98+50	34.0	6.2	4.8	19.4	5.4	5.2	11.5	13.3	5.7	15.1	5.3	10.0
		99+00	44.2	9.2	4.8	15.0	3.3	3.6	8.6	11.3	5.5	12.5	3.3	10.0
29_7	I	99+50	35.3	6.9	3.4	20.7	3.6	3.8	9.5	16.6	6.7	17.1	5.8	15.0
		100+00	46.3	11.7	5.2	18.1	2.3	2.6	5.4	8.4	3.8	9.2	0.0	10.0
		100+50	26.7	3.9	3.5	24.0	5.8	5.5	12.9	17.5	8.8	20.5	7.9	13.0
		101+00	32.8	4.2	3.9	21.1	4.0	4.6	10.7	18.7	8.3	15.4	4.7	14.0
29_8	I	101+50	24.4	1.7	2.1	13.8	4.9	8.2	20.3	24.5	14.9	23.5	9.5	40.0
		102+00	23.4	2.1	2.2	12.8	5.6	7.4	19.4	27.1	14.5	21.5	9.0	46.0
		102+50	22.6	2.3	1.7	15.2	6.9	8.3	19.4	23.4	10.6	16.5	6.7	18.0
		103+00	8.1	2.8	3.8	26.0	7.6	9.4	21.9	20.2	9.4	26.3	9.4	15.0
29_9	I	103+50	26.5	2.4	1.6	10.8	4.3	5.3	19.7	29.3	19.2	29.9	14.0	61.0
		104+00	9.1	0.4	0.0	4.8	1.2	7.9	48.7	27.8	31.4	3.1	16.0	42.0
		104+50	15.1	0.3	1.8	6.1	4.7	13.4	33.2	25.3	26.0	35.7	14.5	64.0
		105+00	24.2	1.5	1.2	8.2	4.3	6.8	18.7	35.0	21.0	27.7	15.2	67.0
29_{10}	I	105+50	16.0	3.6	1.8	11.0	4.6	8.0	23.0	31.9	21.8	30.5	17.1	65.0
		106+00	32.8	5.1	3.7	13.3	4.4	6.1	14.6	19.9	11.8	19.9	13.2	32.0
		106+50	22.9	2.1	2.1	11.4	4.3	8.6	20.8	27.7	17.8	27.1	14.8	47.0
		107+00	19.8	4.3	4.9	18.2	8.6	9.9	21.2	12.9	9.4	19.7	4.8	15.0
		107+50	4.7	3.2	4.0	21.2	9.2	16.7	27.0	13.9	8.3	25.8	7.4	24.0
		108+00	5.4	6.7	4.2	20.7	7.7	13.5	19.9	21.8	9.2	23.4	9.8	10.0
30	VII	108+50	5.9	1.3	3.3	10.4	3.4	5.0	22.1	48.5	31.7	39.2	22.1	73.0
		109+00	13.2	3.9	3.3	8.2	2.9	5.4	20.7	42.3	23.1	31.7	19.1	82.0
		109+50	22.0	7.5	4.5	19.3	6.7	10.2	10.9	18.7	9.8	15.5	6.0	13.0
31	VIII	110+00	19.5	8.6	5.7	30.0	7.1	5.9	8.0	15.2	6.6	16.6	4.4	10.0
		110+50	12.2	3.4	2.9	16.1	10.2	14.8	15.5	24.8	9.8	18.3	7.0	13.0
		111+00	8.3	3.3	1.6	13.9	17.3	18.4	14.9	22.1	10.5	21.1	7.8	12.0
		112+00	24.3	12.2	5.6	20.7	6.0	7.5	8.9	14.8	5.9	13.4	0.0	12.0
32_1	IX	112+50	11.6	2.0	2.3	13.2	6.6	11.7	18.9	33.5	14.9	21.9	10.2	27.0
		113+00	9.4	1.1	1.6	5.5	2.9	12.5	24.0	42.9	29.3	36.6	20.8	110.0
		113+50	24.5	4.1	3.8	8.4	1.7	3.3	19.5	34.7	20.7	27.0	15.6	85.0
32_2	IX	114+50	22.7	7.7	3.7	12.5	2.2	4.3	16.5	30.4	15.1	23.9	12.3	23.0
		115+50	23.1	8.8	5.2	23.0	5.8	9.1	10.7	14.2	8.2	18.1	9.4	14.0

[1] Comparing these analyses with those of the soils examined by A. C. Rose as reported in the article entitled "Practical Field Tests for Subgrade Soils," PUBLIC ROADS, volume 5, No. 6, August, 1924, the following differences will be noted: The analyses here reported were made with Tyler standard mesh sieves instead of metric sieves. The samples were analyzed just as they were taken from the subgrade without removing the coarse material. Silt, as defined by Mr. Rose, is material finer than 0.05 millimeter and coarser than 0.005 millimeter; as here used the term refers to material passing a 200-mesh sieve which is not held in suspension by water. Clay, according to Mr. Rose's definition, is material finer than 0.005 millimeter. As here used the term refers to material held in suspension by water. The maximum size of particles is approximately 0.010 millimeter, about twice as great as the maximum size of the particles designated as clay under Mr. Rose's definition.

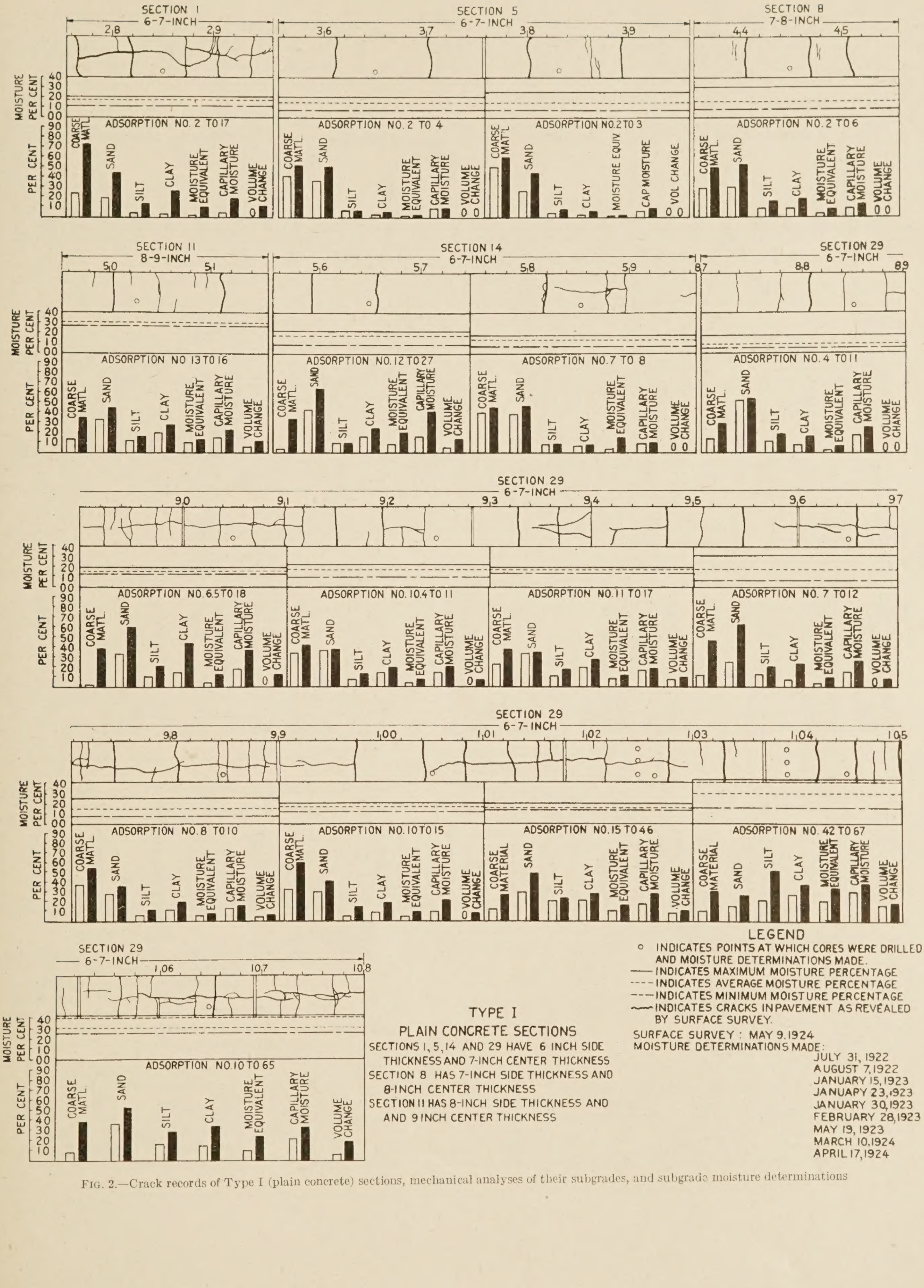
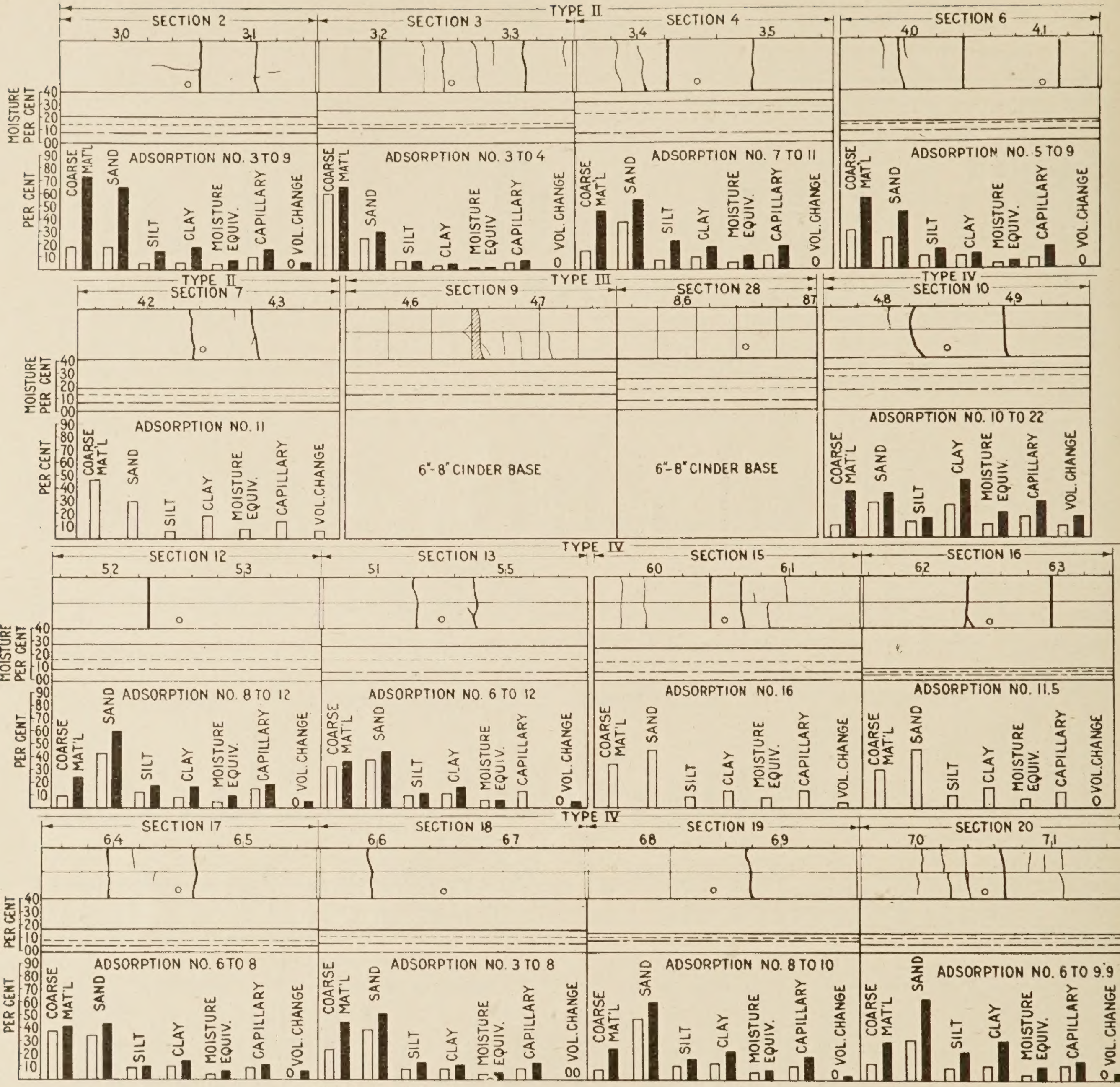


Fig. 2.—Crack records of Type I (plain concrete) sections, mechanical analyses of their subgrades, and subgrade moisture determinations



TYPES II, III, AND IV

TYPE	SECTION	THICKNESS (INCHES)	REINFORCING		JOINTS		CONSTRUCTED BASE
			LONGITUDINAL	TRANSVERSE	LONGITUDINAL	TRANSVERSE	
II	2	6					
	3	7					
	4	8					
	6	6	25 LBS. PER	100 SQ.FT. MESH			
	7	6	50 LBS. PER	100 SQ.FT. MESH			
III	9	4			CENTER	30 FT. C. TO C.	8"-6" CINDER
	28	6			DO	DO	DO
IV	10	6			DO		
	12	7			DO		
	13	8			DO		
	15	6	8-½ INCH ROUND	⅜ INCH ROUND 12" C. TO C.	DO		
	16	6	DO	⅜ INCH ROUND 18" C. TO C.	DO		
	17	6	DO	⅜ INCH ROUND 24" C. TO C.	DO		
	18	6	4-½ INCH ROUND		DO		
	19	6	4-¾ INCH ROUND		DO		
	20	6	4-1 INCH ROUND		DO		

LEGEND

o ... INDICATES POINTS AT WHICH CORES WERE DRILLED AND MOISTURE DETERMINATIONS MADE
—— INDICATES MAXIMUM MOISTURE PERCENTAGE
– – – INDICATES AVERAGE MOISTURE PERCENTAGE
— - — INDICATES MINIMUM MOISTURE PERCENTAGE
~~~ INDICATES CRACKS IN PAVEMENT AS REVEALED BY SURFACE SURVEY

SURFACE SURVEY: MAY 9, 1924
MOISTURE DETERMINATIONS MADE:

JULY 31, 1922 | FEBRUARY 28, 1923
AUGUST 7, 1922 | MAY 19, 1923
JANUARY 15, 1923 | MARCH 10, 1924
JANUARY 23, 1923 | APRIL 17, 1924
JANUARY 30, 1923

FIG. 3.—Crack record of sections of Types II, III, and IV, mechanical analyses of their subgrades, and subgrade moisture determinations
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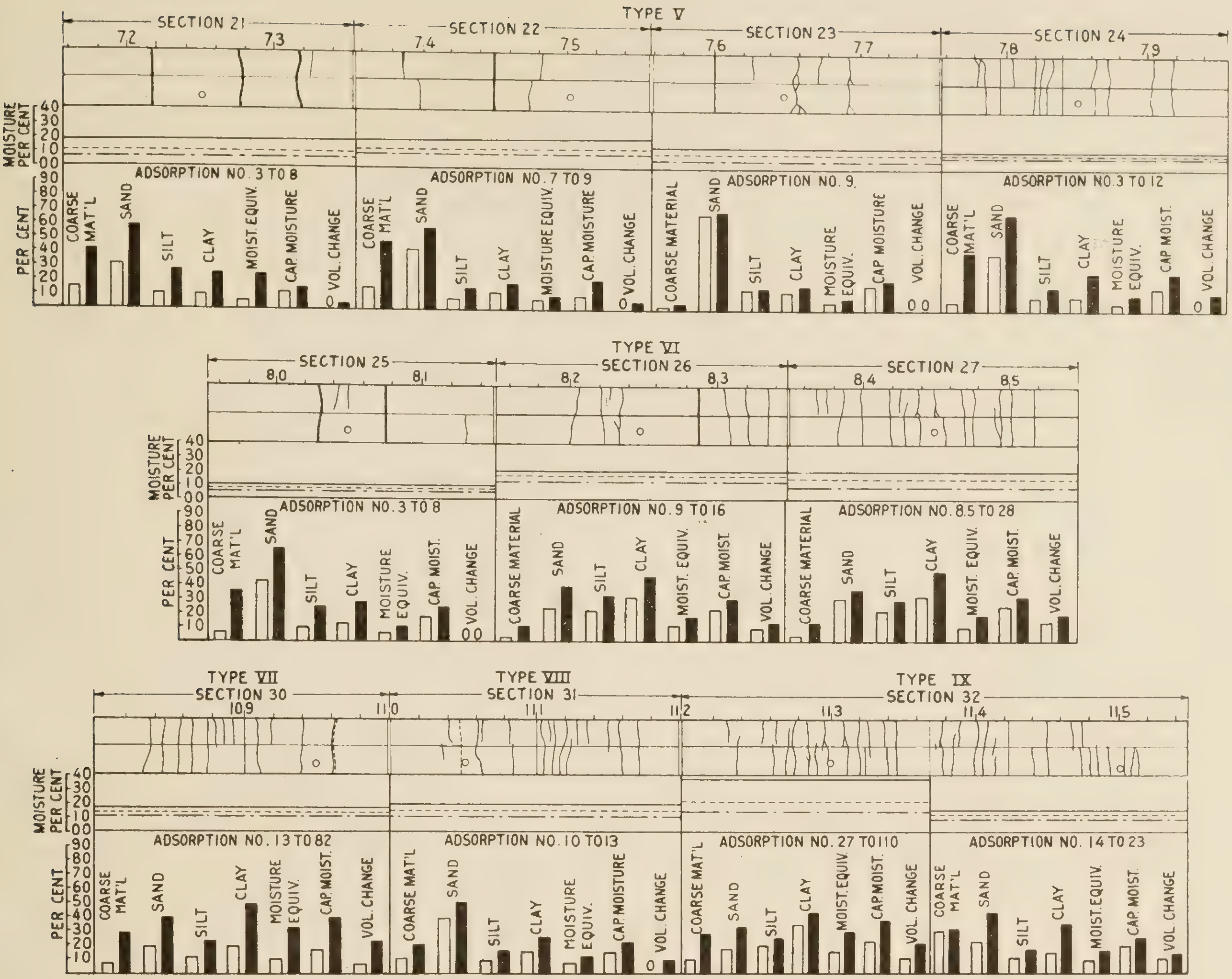

TYPES V TO IX INCLUSIVE

TYPE	SECTION	THICKNESS (INCHES)	REINFORCING		JOINTS		CONCRETE RIBS	
			LONGITUDINAL	TRANSVERSE	LONGITUDINAL	TRANSVERSE	LONGITUDINAL	TRANSVERSE
V	21	6			CENTER	NONE	2 TYPE A	NONE
	22	8			DO	DO	DO	DO
	23	6	8-$\frac{3}{4}$-INCH ROUND		DO	DO	DO	DO
	24	6	DO	$\frac{3}{8}$-INCH ROUND-18 INCH C. TO C.	DO	DO	DO	DO
VI	25	6			DO	DO	4 TYPE A	DO
	26	6	8-$\frac{3}{4}$-INCH ROUND		DO	DO	DO	DO
	27	6	DO	$\frac{3}{8}$-INCH ROUND-18 INCH C. TO C.	DO	DO	DO	DO
VII	30	6	SHOWN ON PLAN		DO	20 FEET C. TO C.	6 TYPE B	9 FT. 10 IN. C. TO C.
VIII	31	6	14-$\frac{3}{4}$-INCH ROUND	$\frac{1}{2}$-INCH ROUND-18 INCH C. TO C.	DO	NONE	2 TYPE A	NONE
IX	32	6	DO	DO	DO	DO	NONE	DO

SURFACE SURVEY: MAY 9, 1924
MOISTURE DETERMINATIONS MADE:
JULY 31, 1922
AUGUST 7, 1922
JANUARY 15, 1923
JANUARY 23, 1923
JANUARY 30, 1923
FEBRUARY 28, 1923
MAY 19, 1923
MARCH 10, 1924
APRIL 17, 1924

LEGEND
○ INDICATES POINTS AT WHICH CORES WERE DRILLED AND MOISTURE DETERMINATIONS MADE
—— INDICATES MAXIMUM MOISTURE PERCENTAGE
- - - - INDICATES AVERAGE MOISTURE PERCENTAGE
—·— INDICATES MINIMUM MOISTURE PERCENTAGE
—— INDICATES CRACKS IN PAVEMENT AS REVEALED BY SURFACE SURVEY

FIG. 4.—Crack record of sections of Types V to IX, inclusive, mechanical analyses of their subgrades, and subgrade moisture determinations

Subgrade materials which have a large percentage of granular material and very little clay not only take up less moisture during the wet season than those which are high in clay but also retain less during the dry season. This is shown in sections 9 and 28 where the subgrade is cinders, and in sections 25, 24, 23, 19, and 16 where a large percentage of clean sand is found.

Whether the subgrade is on a cut or fill has some effect on the amount of moisture it will hold. Subgrades through a cut will attain higher moisture and retain more of it than similar subgrades over a fill. Section 30 on a high fill and 32 in a cut indicate the effect of this difference on the moisture content of a subgrade.

The percentage of capillary moisture as determined by examination of a subgrade sample by the present laboratory test does not represent the maximum amount of moisture that will be held by the particular subgrade. Comparison of results of moisture determinations in Table 3 with capillary values given in Table 2 shows the capillary values to be about 75 per cent of the maximum moisture and about equal to the values obtained in the subgrade during the warm season. This can probably be accounted for by the fact that the laboratory test is made at room temperature and that capillary moisture increases as the temperature is reduced to freezing. It would seem that the laboratory test should be made on the samples when they are close to freezing and not, as is now done, at room temperature.

The lack of stability of subgrades which are composed of materials showing high volume change is very definitely shown by certain sections. In the plain concrete sections those whose subgrade materials show volume change in the laboratory have cracked longitudinally, while those sections whose subgrades show no volume change have not cracked. Section I has cracked badly longitudinally. Although the subgrade shows some volume change, some of the cracks are probably due to one or more springs in the subgrade. Evidence of the existence of such springs is the free water observed along the edges and in the cracks.

Section 29, stations 103–105, has a subgrade similar to the adjoining sections but at the time of construction cement in the proportion of 1 to 28 was mixed to a depth of 6 inches with the soil. This section has no longitudinal cracks. The high moisture readings in the adjoining sections are also obtained in this section but much less moisture is retained in the treated subgrade. The effect of the treatment on this section is quite evident when compared with the two adjoining sections, which have cracked badly longitudinally. This treatment has apparently so modified the character of the subgrade that volume changes so destructive in the adjoining section have not done any damage in this one. It is very likely, however, that a layer of cinders or other granular material such as coarse sand would have provided approximately the same results.

Materials which undergo large changes in volume seem to be those which have a large percentage of clay. The quantity and character of this clay, as indicated by the adsorption number of the material, is also indicative of its action under a change in moisture content. Sections which have a large amount of granular material, as illustrated by sections 3, 4, 5, show no volume change, others show changes as high as the 16.8 per cent in section 27. Other sections which are high in volume change are 10, 11, 15, 26, and 29, stations 89–91, 93–97, 101–108, and sections 30, 31, and 32. Sections which have high volume change have a high adsorption number, while the reverse is true where the volume change is small.

TABLE 3.—*Moisture content of subgrades at intervals since the construction of the Columbia Pike*

Section No.	Station	Moisture content at certain dates									
		July 31, 1922	Aug. 7, 1922	Jan. 15, 1923	Jan. 23, 1923	Jan. 30, 1923	Feb. 28, 1923	May 19, 1923	Mar. 10, 1924	Apr. 17, 1924	May 19, 1924
1	28+50	[1]15.3	[1]13.0	14.0	12.0	[1]18.6	[1]21.3	[1]18.6	[1]17.9	[1]21.5	[1]18.6
2	30+50	7.8	7.0	9.7	15.0	16.4	17.0	17.8	16.2	20.4	21.1
3	32+50	11.9	10.3	10.9	13.3	13.1	12.4	12.8	24.2	15.2	16.8
4	34+50	7.6	6.8	21.0	16.2	26.3	30.1	30.4	28.8	31.9	25.9
5	36+50	15.7	12.3	7.7	10.5	20.9	24.4	20.2	10.9	13.6	21.0
	38+50	7.7	8.1	9.6	9.6	21.9	22.1	20.6	15.2	20.1	19.0
6	41+00	9.1	8.6	13.9	15.4	13.9	14.4	15.3	12.6	8.0	16.0
7	42+50	7.6	6.9	14.8	10.1	12.9	14.9	15.4	17.3	16.9	15.2
8	44+50	9.5	8.9	11.8	10.8	14.5	30.4	24.9	17.0	13.4	12.6
9	46+50	16.7	16.1	22.0	17.6	16.2	16.5	12.2	26.2	15.9	29.7
10	48+50	15.4	18.3	25.0	27.3	27.5	29.3	27.7	30.7	23.4	24.1
11	50+25	29.5	29.9	28.0	29.2	25.2	31.9	30.4	41.2	33.7	33.2
12	52+50	......	9.8	9.1	10.0	18.6	22.2	26.5	18.1	16.4	19.0
13	54+50	6.4	8.8	18.3	18.6	15.9	21.6	20.1	13.2	26.0	24.8
14	56+50	9.1	13.3	16.6	22.5	20.2	22.7	19.9	22.3	22.8	21.9
	58+50	7.2	9.7	14.5	16.7	17.1	17.1	14.7	11.2	18.3	12.1
15	60+50	8.4	7.3	12.9	13.2	12.8	11.2	11.0	20.6	24.0	19.9
16	62+50	7.0	6.8	7.7	7.6	8.1	7.7	7.2	5.9	8.2	6.8
17	64+50	7.5	5.8	7.7	8.0	9.0	9.8	8.9	7.1	15.9	17.1
18	66+50	7.5	7.7	7.0	15.3	9.4	14.9	13.8	13.0	10.2	7.3
19	68+50	8.6	9.6	10.4	10.3	10.4	12.2	14.0	13.8	12.8	11.8
20	70+50	6.5	14.8	10.0	9.9	11.6	10.0	9.0	10.3	12.3	16.5
21	72+50	8.7	7.2	7.6	8.5	17.6	9.2	11.8	9.2	13.7	17.0
22	75+00	7.8	8.0	8.3	9.0	13.7	13.4	12.3	12.5	17.6	10.4
23	76+50	5.7	5.6	5.9	9.6	10.5	8.9	6.8	7.8	10.1	12.2
24	78+50	9.3	9.3	9.9	10.0	9.0	11.2	15.3	8.8	00.0	11.8
25	80+50	6.0	6.7	7.2	7.9	9.6	8.9	9.9	5.9	8.4	9.9
26	82+50	12.1	12.5	12.4	15.5	13.6	19.7	17.4	15.8	19.0	14.5
27	84+50	8.3	9.2	12.4	16.2	15.0	16.4	12.3	13.2	18.0	13.8
28	86+50	8.2	9.7	20.0	21.4	24.8	25.2	16.5	15.5	16.1	23.3
29	88+50	5.2	6.0	7.1	8.5	8.0	8.4	11.4	13.4	16.6	10.4
29_1	90+50	14.9	13.9	19.3	15.1	15.8	12.6	14.0	14.1	18.5	20.6
29_2	92+50	10.3	11.4	17.9	15.9	18.4	15.7	16.7	20.0	22.2	18.4
29_3	94+50	13.8	15.3	17.1	19.0	19.2	22.5	18.9	19.7	17.9	18.4
29_4	96+50	6.7	7.0	26.1	18.2	30.6	19.9	22.8	17.8	25.2	13.0
29_5	98+50	8.2	9.6	21.0	12.0	19.9	20.3	20.4	13.9	18.2	25.4
29_6	100+50	12.6	12.2	17.2	16.9	19.5	20.4	14.9	18.0	17.2	17.8
29_7	102+50	14.0	13.9	17.4	16.3	10.2	18.4	16.8	20.9	00.0	18.4
29_8	104+50	21.1	13.1	41.0	27.3	34.7	30.2	26.8	38.3	43.4	28.9
29_9	106+50	26.4	25.4	43.0	39.8	20.2	31.0	37.6	38.5	26.5	30.6
30	109+50	10.8	12.6	11.4	15.1	13.0	12.0	11.6	11.0	10.1	11.7
31	110+50	13.1	15.8	9.5	13.4	17.8	11.6	19.8	15.6	15.5	12.7
32	113+00	14.7	28.8	36.0	33.7	16.8	18.0	13.9	17.8	14.1	15.1
	115+00	13.1	12.1	14.7	7.9	13.4	12.2	8.5	7.9	00.0	14.5

[1] Free water in subgrade.

LONGITUDINAL JOINTS PREVENT LONGITUDINAL CRACKS

Those sections in which longitudinal joints have been constructed are in all cases free of longitudinal cracks, even when the subgrade material, as in sections 10, 15, 24, 26, and 27, changes in volume. There is this distinction, however, that in the section with a high volume change the longitudinal joints are open, showing that more or less movement has taken place at the edge with the joints acting as a hinge; where there has been no volume change, the appearance of the joint indicates little if any movement about the center axis.

Transverse cracks resulting from contraction occur at intervals which correspond to the distance in which the force of subgrade friction becomes greater than the tensile strength of the concrete. Unevenness of the subgrade such as might be obtained by construction of the pavement on rough macadam, for example, increases the number of transverse cracks by increasing the friction between the subgrade and pavement.

An increase in thickness of the pavement does not materially affect the spacing of the transverse cracks as caused by contraction in the pavement. The additional strength of the thicker section is probably balanced by the additional friction on the subgrade. Types of construction of plain concrete which have the section strengthened by additional thickness or thickened edges, although they do not materially affect the spacing of transverse contraction cracks, do serve to keep down the number of additional cracks caused by heavy traffic.

TRANSVERSE CRACKING NOT PREVENTED BY LONGITUDINAL STEEL

One of the purposes of this experiment was to investigate the behavior of sections with different kinds and quantities of steel reinforcing. The results obtained to date indicate emphatically that longitudinal steel, above a certain amount and installed as it generally is now, does not prevent transverse cracking but does, on the other hand, greatly increase the number of such cracks.

Figures 2, 3, and 4 show that there is a great difference in the appearance and location of the transverse cracks. In sections 27, 30, 31, and 32, where the percentage of longitudinal reinforcing is large, the surface shows transverse cracks every few feet. Sections 30, 31, and 32 having a higher percentage of steel than No. 27 show a greater number of cracks. The effect of the amount of longitudinal reinforcing on the pavement is further indicated in sections 15, 16, 17, 18, 19, 20, 23, 24, and 26. In these sections the amount of reinforcing is varied; comparing them, we notice that 23, 24, and 26, each of which have eight ¾-inch rods, are not cracked so badly as sections 30, 31, and 32 which have a larger number of rods. Section 20, with four 1-inch rods, is cracked slightly less than the sections which have eight ¾-inch rods. Sections 15, 16, 17, 18, and 19, with reinforcing varying in amount from four ¾-inch rods to four ½-inch, are free from the fine cracks. These sections have open cracks similar to those found in a plain concrete section but spaced at greater distances (60 or 70 feet). Sections 6 and 7, which are reinforced with 25 and 50 pounds of mesh per 100 square feet, have only the open cracks spaced at a greater distance than would ordinarily be found in plain concrete.

From the condition of the reinforced sections it would seem that some doubt can be properly expressed as to the value of longitudinal reinforcing as it is now used in the pavement. The longitudinally reinforced sections in this experiment show conclusively:

1. That longitudinal steel up to a certain amount does increase the spacing of the transverse cracks and that these cracks are open on contraction.
2. That longitudinal steel above this amount will produce fine transverse cracks spaced in some cases only a few feet apart.

DEDUCTIONS WITH REGARD TO TRANSVERSE CRACKING

The action of these sections and the results of observations on other concrete pavements seem to support the following deductions:

In plain concrete the movement of the pavement in contraction is resisted by the friction between the slab and the subgrade. This force carried to the pavement is very high, much higher than would be expected as the movement is very slow. On the average subgrade it will crack the pavement in tension at intervals from 40 to 60 feet. In cases where the subgrade is rough and offers high resistance to the movement of the slab, transverse cracks will develop at shorter intervals. An extreme case of this was observed when a concrete pavement was laid on a rough natural rock base, the pavement being badly damaged by numerous transverse cracks. It has been observed that the width of these cracks varies directly as the distance between them. In cases where the transverse cracks are close together and very fine, they become wider as time goes on from the grinding and crushing of the edges of the crack. This is particularly apt to occur in cases where the subgrade happens to be unstable.

In the case of pavement reinforced longitudinally, the resistance to contraction developed by the subgrade is similar to that in the plain concrete. Along with this force there is the added force developed by the bond of the steel. Acting against this pair of forces is the force resulting from contraction limited by the tensile strength of the concrete. Taking the case of a slab with continuous reinforcing, we would have as we move from one end toward the other increasing resistance to movement of the slab over the subgrade and increased force from the restrained contraction of the pavement. These forces increase as the distance from the end of the slab or crack is increased until the tensile strength of the concrete is exceeded, producing a transverse crack in the slab. The stress in the concrete is thereby relieved, but the steel remains in tension and continues to oppose the contraction of the concrete beyond the first break. With respect to this section of the concrete the stress in the steel is an initial force to which the subgrade frictional forces are added, with the result that in a comparatively short distance the strength of the concrete is again exceeded and another crack is formed. Repetition of this process produces a series of closely spaced cracks, until the accumulating tensile force ruptures the steel.

If the steel is not continuous and the joints which separate it are close enough together, it is possible that there may be no cracking of the slab. This condition would be expected in pavements which are separated by joints into sections the length of which is less than twice the distance required for the formation of a first crack. There are no such sections in the Columbia Pike, but sections 24, 26, 27, 30, 31, and 32 supply evidence of the validity of the above theory. Their first cracks occur about 40 feet from the joint, while the other intermediate cracks are spaced from 10 to 15 feet apart.

The effect of accumulating tension in the steel is illustrated by the behavior of sections 6, 7, 16, 17, 18, 19, and 20, in which large open cracks have developed. Opening these cracks at points where the steel was located, it was found that in sections 23, 24, 26, and 27, in each of which the longitudinal steel consists of eight ¾-inch deformed round bars, no change in the diameter of the rods had occurred as far as could be determined. In section 20, which is reinforced with four 1-inch rods, slight reductions in diameter were found. In section 19, with four ¾-inch rods, the diameter was reduced to five-eighths inch, and there was evidence of a nearly complete failure in the steel. In sections 16 and 17, with eight ½-inch rods, the diameter was reduced to approximately three-eighths inch, while in section 18, with four ½-inch rods, there was a complete break with a reduction in the diameter to about one-fourth inch. In sections 6 and 7, with 25 and 50 pounds of mesh per 100 square feet area, complete breaks were found at the cracks.

THE COST OF GRADING WITH FRESNOES

SOME SUGGESTIONS AS TO COMMON LOSSES AND HOW TO AVOID THEM

By J. L. HARRISON, Highway Engineer, United States Bureau of Public Roads

FROM the management standpoint the outstanding problem on a construction job is how to obtain high production. The time required to perform each necessary operation and the degree to which unnecessary operations are eliminated are the determining elements. Large losses in time are more apt to be the result of a small loss on each performance of an operation necessarily repeated many times than of any outstanding error in the performance of an important operation. No foreman of a grading job would think of letting one of his teams stand idle on the job all day, but he may produce much the same effect by disregarding as small a detail as that of allowing all his teams, after dumping, to swing through an arc with a 30-foot radius when a 15-foot radius would do just as well. The loss of time in this case, being small in each instance, is apt to escape notice. High production can be secured only when such losses are reduced to a minimum, and in order to make them as small as possible their causes should be studied in detail.

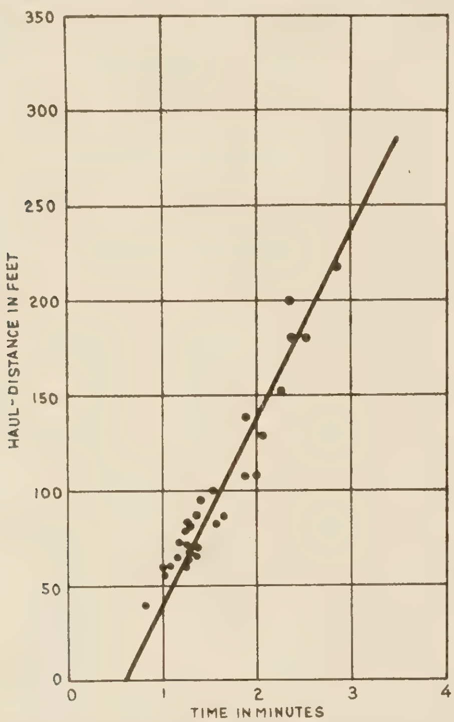

FIG. 1.—A well-managed fresno job. Note consistency of performance and low time loss on turning, loading, etc.

Approaching from this angle the problem of moving earth by fresno, the typical operations are: (*a*) Loading, (*b*) hauling to the dump, (*c*) dumping and spreading, (*d*) returning for the next load, and (*e*) turning to load again. The order of these operations may be varied. For instance, the load may be taken before the turn for the return trip is made; but the sequence is of less importance than the nature of the operations themselves because, whatever the sequence, each takes time and is, therefore, a source of possible loss.

The studies which have been made by the Bureau of Public Roads indicate that the losses suffered by contractors doing fresno work fall within the two categories of managerial losses and bidding losses. The managerial losses are those which can be corrected by carefully systemizing the work. On a fresno job they include those due to improper adjustment of equipment to the work, short loading, time lost in loading, time lost in turning, careless dumping, and a number of miscellaneous losses not directly related to these. The haul itself offers little opportunity for management, as mules set a fairly steady pace which the studies indicate to be little affected by external conditions.

Bidding losses which are, perhaps, as important as the managerial losses, are those which are suffered because of incorrect bidding. They arise principally from a failure to gauge correctly the influence of haul.

SOME CAUSES OF SHORT LOADING

Referring first to the managerial losses, one of the most significant is short loading. It takes just as long to move half a load as it does to move a whole load. Therefore, if a proper output is to be obtained, the loading must be kept up to standard. The bureau's studies indicate that a standard 4-foot fresno loads about a third of a cubic yard per trip. On a well-managed job the fresnoes will be loaded to their standard capacity every trip. A part of the load may be dropped during a long haul, but, from the standpoint of the practical contractor, the effect of this is relatively small, because he can easily allow for it by leaving the grade a little low if much hauling is to be done over it. In any event he generally finds it necessary to go back over his work and correct it for shrinkage, etc., when the final stakes are set; and the material that falls off the fresnoes usually serves to reduce by so much the amount that must be handled in the clean-up. Short loading, however, is a positive loss the causes of which should be sought and eliminated wherever possible. It may be due to the indifference or inefficiency of the drivers. When this is the case the drivers should be trained to load properly or be replaced.

Poor plowing is often an element in short loading. It also affects the time spent in loading. If the soil is properly plowed, a full load can be secured in sand unless it is very dry, in loam, and in clay except where it is unusually heavy. But plowing is relatively expensive and is hard on the stock—particularly plowing in that sort of heavy ground where it is most needed if full loads are to be taken without undue loss of time. In such cases the problem presented to the foreman is a choice between the tangible reduction in output which will result if a fresno is laid off and the mules are hitched to the plow, and the intangible losses which result from slower and lighter loading when there is inadequate plowing.

Light loading and loss of time in loading sometimes result also from failure to remove the roots of trees. If the contract price for earth excavation is 20 cents per cubic yard and labor is 25 cents an hour, an increase in output of $12\frac{1}{2}$ yards will pay for a 10-hour day's work in removing roots. With an ordinary small outfit of six to eight fresnoes working on a 100-foot drag, the loss of as little as 0.05 minute per load or of 2 per cent in the amount loaded is sufficient to justify the employment of an extra laborer to remove roots or obstructions of any other character which interfere with proper loading. The losses due to obstructions are often much greater than this.

On short hauls every effort should be made to secure heavy loading. Whatever additional yardage may be secured in this way is a clear gain. It is not advisable, however, to attempt heavy loading on long hauls, because of the greater amount of work such hauls impose on the stock. In a haul of 100 feet, the round-trip time averages 2.2 minutes. Twenty-seven trips are made in an hour. Only one-half minute of the trip time is spent in hauling the load to the dump, which means that the team is under full load 13½ minutes per hour. On a 300-foot haul the time required per trip is 4.2 minutes, and there are 14 loads per hour. In this case the team is under full load 1½ minutes per trip or 21 minutes per hour. The fact that on the longer haul the stock performs 50 per cent more heavy work during the day than on the short haul offers a reasonable explanation of the fact that—even without extra-heavy loading—stock generally shows a distinct loss of weight if held on long-haul work for a protracted period.

All "pestering" of stock should be rigidly avoided and teamsters who engage in it should be replaced. Some teamsters abuse the stock under the pretense that they are ambitious for more speed. Jobs where this is permitted almost always show an increase in the time taken per trip.

CONSISTENCY OF PERFORMANCE AN IMPORTANT ELEMENT OF GOOD MANAGEMENT

One of the more conspicuous facts brought out by the studies is that the difference between good management and average or poor management is largely a matter of the consistency of the performance. Good management sets a high standard and consistently follows it. Under average or poor management the results secured will at times compare favorably with what would be expected under the best of management, while at other times the results will fall so low as to be classed as poor. The lack of consistency can be traced to a tendency on the part of many foremen to allow the physical conditions surrounding the job to govern performance. If these favor a high output, a high output is secured, but if they tend to create losses no adequate effort is made to avoid their effect. So, if the lay of the work suggests that a long swing at the dump be made, the long swing is not prevented by managerial effort, though the loss of but a tenth of a minute on each load may easily mean the loss of a hundred yards on the day's output. The tendency of foremen to allow physical conditions to establish the details of performance which they should themselves govern may in some cases be due to a lack of energy but it is more apt to be a matter of lack of training. However, the point it is desired to emphasize is that a contractor should view inconsistency in output on a fresno job as indicating weak field supervision and take immediate steps either to develop or to displace a foreman whose record of performance is of that kind.

The difference between good and average management as measured by the results obtained may be illustrated by the following example: A normal fresno job will show an average haul of perhaps 150 feet. On such a job the average output under average management would be about 73 cubic yards per fresno per day of 10 hours. Under good management it would be about 95 cubic yards per fresno per day. With a 10-fresno outfit the difference in output would be 220 cubic yards per day, which, at 25 cents per cubic yard, is $55 per day. In other words, on such a job as this a saving of one-tenth of a minute per load means a saving from $7.50 to $10 a day for the contractor.

So far no significant differences have been found in the rate at which such materials as sand, light gravel, loam, clay, or even heavy clay are moved once the load is taken. There is little variation in the pace of the mules either in moving the material or returning from the dump. There appears to be a little slowing

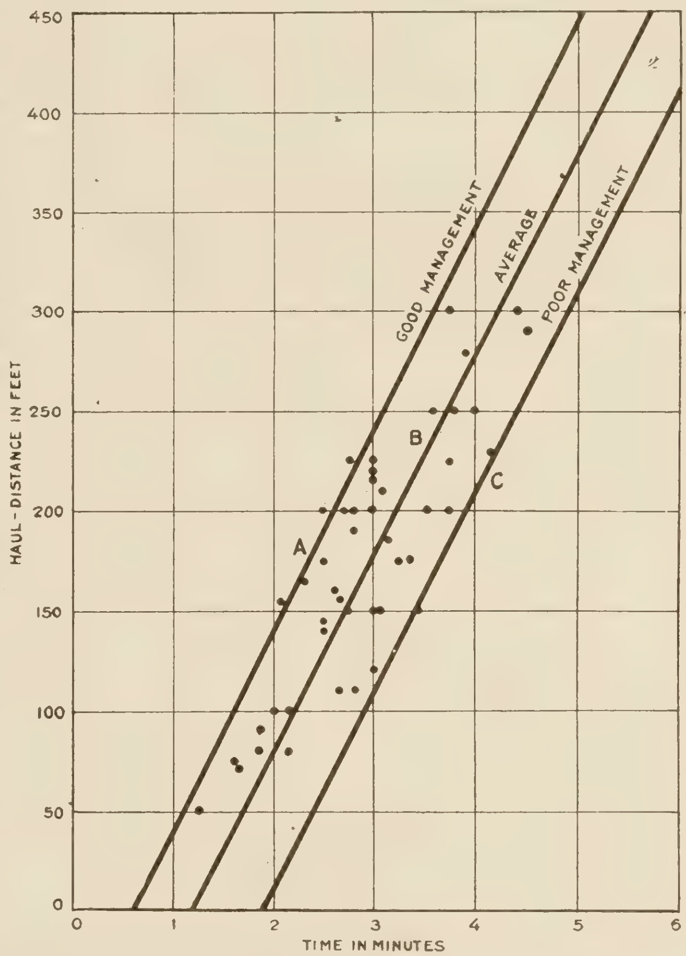

FIG. 2.—Diagram comparing job represented in Figure 1 with other jobs typical of average and poor management. The points shown illustrate lack of consistency on a job where the control was weak

down in loose, dry sand and a little speeding up on material which packs readily and gives a particularly smooth, hard footing, but in the Mississippi Valley, where the bureau's studies have been made, such materials are exceptional. On the other hand, very stiff clay loads with difficulty even when well plowed, and there is a distinct tendency for the men to tire when handling it. Graphs, similar to Figure 1, based on morning and afternoon observations on the same well managed job, show the effect of the increased time required for loading, unloading, and turning in the afternoon by increases in the X intercept. The uniform pitch of the rate lines brings out the fact that on heavy work the stock continues to move at its regular pace after the men have begun to show fatigue. Under conditions involving heavy work in loading it is well to put on an extra man to load the fresnoes in order to relieve the teamsters.

WET WEATHER LOSSES LARGELY AVOIDABLE

Among the managerial losses those incident to wet weather should be mentioned. It is customary to stop work whenever there is rain and to delay reopening until the ground has dried out more or less. During such delays the stock has, of course, to be fed. Many contractors also feed their men without charge, and it occasionally happens that labor conditions are such that the men also must be paid. It should, therefore, be noted that fresno work can be performed without great difficulty even in mud so deep that the mules sink above their fetlocks. Nor is the work as much delayed by such conditions as might be supposed. The mules move a little more slowly and there is a tendency to lose time in loading and dumping, but outputs can be readily secured which more than cover operating costs. The

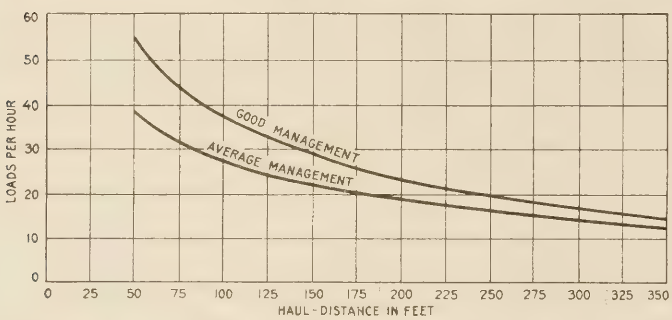

FIG. 3.—Diagram showing number of loads per hour under good management and average management.

dead loss of the corral and the mess is thus avoided even though no profit is made. After a heavy shower work should be started at once. The surface of the ground will be muddy but a single cut will remove the mud, and even if the wet earth must be cleared from the path over which the teams move the actual cost of this operation is small as compared with the cost of delay. As soon as the wet surface is removed, a fresno job can be made to run as smoothly as in dry weather.

Another loss that can be easily avoided is the maintenance of extra stock. It is not uncommon to find that an outfit is maintaining 3 to 5 extra animals in the corral as against 25 to 30 on the job. On the other hand, one job (not a fresno job) now under study maintains 44 horses on the job with a reserve of one in the corral. With proper attention to the selection of good drivers and with proper attention to the stock there is small need for more than one extra animal unless the outfit is unusually large. At present, feeding a mule costs from 60 to 70 cents a day or about $20 a month. Any animal which has a tendency to "go lame" or injure itself on the work is, therefore, too expensive to keep.

Figure 2 compares the results of good, average, and poor management. Graph A shows the results on a job where the foreman used conspicuous care in avoiding time losses of all kinds. Short diameter turns were insisted on and the equipment was kept in motion. Graph B shows the average results on a considerable number of fresno jobs. The intercept on the X axis, a constant which covers loading, unloading, and turning, with incidental losses of time, is 0.6 minute per load on the well-managed job as compared with 1.2 minutes under average conditions and 1.9 minutes with poor management. There is no significant difference in the rate at which the teams move to and from the dump. The effect of the differences which these graphs show as between the "loading, dumping, and turning time" under good and average management is of considerable importance as the following table, based on a 100-foot haul, shows.

	Good management	Average management
Time per load, 100-foot haul, minutes	1.6	2.2
Loads per fresno per 10-hour day	375	272
Loads per day for eight-fresno outfit	3,000	2,176
Per cent of average performance	138	100
Quantity of excavation per day, cubic yards	1,000	725
Teams required to produce 1,000 cubic yards per day	8	11

In Figure 3 the value of close attention to the elimination of lost time in loading, unloading, and turning, with incidental losses, is shown in another way in two graphs which give the number of full loads per hour per fresno which can be expected under good management and under average management. Using these graphs any foreman engaged on fresno work can determine the efficiency of his outfit by the following simple process:

1. Count the number of loads dumped during an hour.
2. Divide the number of loads dumped by the number of fresnoes at work to obtain the average number of loads dumped per fresno.
3. Find the average distance over which the loads dumped during the count were moved.[1]
4. Compare the average number of loads per fresno, obtained as indicated in 1 and 2, with the number shown by the graphs (or Table 1) for the distance determined as indicated in 3, to ascertain whether the management is above or below average.

Table 1 gives the same information contained in Figure 3 for intervals of 25 feet.

TABLE 1.—*Fresno output*

[Standard 4-foot fresnoes]

Haul distance (feet)	Loads per fresno per hour: Good management	Loads per fresno per hour: Average management	Haul distance (feet)	Loads per fresno per hour: Good management	Loads per fresno per hour: Average management
50	54½	38	200	23	19
75	44	31	250	19½	16
100	37½	27	300	16½	14
125	32½	24½	350	14½	13
150	28½	22	400	13	11½
175	25½	20			

BIDDING LOSSES

Aside from the losses incurred through faulty management, contractors often suffer serious losses because

[1] In practice, when moving earth from a cut to a fill the area over which loads are being secured and the area over which dumping is being done are often quite sharply defined. For the purpose of testing the efficiency of operation it is accurate enough to measure from the center of the area over which loading was done during the count to the center of the dumping area, a procedure which has been found by experience to give results agreeing within a few feet of the average distance obtained by taking the distance on each load and averaging these distances.

of careless bidding on fresno work. It is customary to call for bids for highway excavation on the cubic-yard basis, and quite commonly contractors are influenced in determining the amount of their bids by the unanalyzed cost of other jobs. Presumably this practice is based on the presumption that moving one cubic yard will cost about as much as moving another, or, in any event, that the average work required in moving a cubic yard of earth on one project will be approximately equal to the average work required on another project. The fact is that the unanalyzed cost of work done on a past project is a dangerous criterion of the cost of prospective work because it takes no account of the haul which, in practice, varies from mile to mile on an individual project, and varies considerably as between different projects. The force of this statement will be seen by a little study of the typical fresno loop (see fig. 4) or by a study of the time-distance graph (fig. 2), and is clearly developed in the right-hand column of the detailed time schedule and estimate on page 15.

If the drag or distance moved (see fig. 4) is long, a large amount of time is consumed in dragging one load and returning for another. If the drag distance is zero, then theoretically a load can be picked up and dumped at any point on the approximate circle resulting from a combination of the two turning swings with small loss of time over what is required in driving around a circle 30 to 50 feet in diameter. This is about what actually takes place when material is moved from the ditches to build up the subgrade. With allowances for the extra effort required in climbing from the ditch to the top of the embankment, as well as for loading and dumping, operations that result in changing the path of travel from a circle to a short loop, the time per trip may, with good management, be kept as low as 1 minute per load, and 1.2 minutes per load is a fairly common performance. Work handled in this way is known as "in-and-out" work.

As against this, a 300-foot drag will require with good management about 3.6 minutes and with average management 4.2 minutes. In other words, a drag approaching the maximum limit of the fresno's range of efficiency takes nearly four times as much time as "in-and-out" work. This illustration will serve to make it clear that the price bid on one job is not a valid criterion of the price which ought to be bid on the next job, unless it is known that the average haul is the same in both cases or that the average time required for a load is the same. Estimates should, therefore, be made up from the plans for the job itself. In no case should they be made from the unanalyzed bids on other jobs.

A METHOD OF SCHEDULING HIGHWAY GRADING

Another matter which ought to receive more attention in connection with highway work is the time scheduling of construction operations. A first-class building contractor will schedule a million-dollar building so closely that he will know its date of completion almost to a day. In doing this he lays down in advance the force his superintendent will be authorized to employ, and, on this basis, calculates the time that will be required in order to do each part of the work. Once this is done, to insure himself that a job is being built at the assumed cost, he has to examine only three things:

1. His materials purchases to see that in quantity and in price these agree with his estimate. As these are usually protected in advance of making his bid, he should find no trouble here.

2. The wages that are being paid. Prevailing wages are, of course, known before the estimate is made, and the superintendent should be appropriately limited in this field.

3. The amount of work done.

If the first two are as planned and the job, in time, is up to schedule, the contractor knows without consulting detailed cost data that the job is being profitably conducted and what the profit is.

The graphs presented in this article offer a ready means of preparing such a schedule for highway grading with standard 4-foot fresnoes and at the same time estimating the cost of fresno work in such a manner that the contractor may at all times feel confident of the results which his field forces are securing. The methods are as follows:

1. To determine the time required on any job—

(1) Multiply the total yardage (excavation plus borrow)[2] by the time required for "loading, dumping, and turning," which is, for average performance, 1.2 minutes.[3]

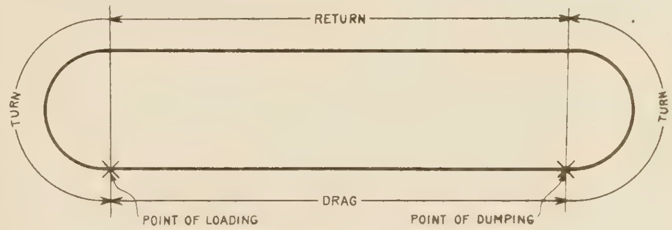

FIG. 4.—Typical loop made by fresno outfit

(2) Add to this the product obtained by multiplying the net yardage of each cut, or borrow pit, by the average distance in stations (100 feet) that it must be hauled.[3] If no haul distance is shown for borrow multiply borrow yardages by 0.5 (one-half station).[4]

(3) Divide the sum obtained above by 200 to obtain the number of days' work to be done, if the working day is 10 hours (180 for a 9-hour day and 160 for an 8-hour day). The method here described is adopted for the sake of simplicity. To obtain the correct time of the job in minutes the extensions under (1) and (2) should also be multiplied by 3, in which case the sum should be divided by 600.

(4) Divide this by the number of fresnoes it is planned to work and the result will be the number of days' work which will be required of the outfit under normal conditions.

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[2] Borrow as used here covers material secured outside the limits of the standard cross section—generally though not always by widening ditches.

[3] This product is one-third of the time required for the operation, since it is obtained by multiplying yardage by the time required to handle one-third of a cubic yard. See explanation under (3).

[4] Theoretically side borrow involves only transverse haul. In practice, because ditch lines and the road section generally must be kept reasonably uniform, whereas the ground will vary a good deal, more or less longitudinal haul is required.

(5) To this add one-fifth to cover Sundays, holidays, and wet weather, in order to obtain the working period. For example:

		Good management	Average management
Loading, dumping, and turning time, minutes		0.6	1.2
Excavation, cubic yards	70,000		
Borrow, cubic yards	30,000		
Total earth to be moved, cubic yards[1]	100,000	60,000	120,000
Cut 20,000 cubic yards moved 2.47 stations		49,400	49,400
Cut 10,000 cubic yards moved 3.16 stations		31,600	31,600
Borrow 30,000 cubic yards moved 1.46 stations		43,800	43,800
Cut 10,000 cubic yards moved .74 stations		7,400	7,400
		192,200	252,200
Divide sum above by 200 to give days' work to be done		961	1,261
Divide days' work to be done by number of fresnoes in outfit to give days' work for outfit (in this case 10)		96	126
Add one-fifth for Sundays, holidays, and bad weather		19	25
Working period, days		115	151

[1] By reason of side-hill work and excavation from ditches (in-and-out work requiring only loading, dumping, and turning time) this quantity will generally be greater than the sum of the cuts and borrow pits listed below.

2. To convert this estimate of the time needed to accomplish a specific amount of work into a bidding estimate the next step is to determine the cost per day of operating an outfit of the size used in making the time estimate in this case 10 fresnoes. The costs and wages given below are for illustration only. The contractor must use those actually prevailing.

	Mules	
A typical outfit would be—		
10 fresnoes, 3 mules each	30	
2 plows, 4 mules each	8	
Extra stock	2	
	40	[5] $35
Add one-fifth for corral cost on Sundays, holidays, and bad weather		7
Superintendent (one-fifth added)		10
Foreman (one-fifth added)		5
Labor:		
12 teamsters, at $3		36
2 plow holders		7
1 laborer (water boy, etc.)		2
Total cost per day		102

On long hauls one plow can be converted into a fresno outfit which gives a small working margin. The above is illustrative only. The contractor must develop his estimate of the cost of a day's work on the basis of the outfit he has and wages prevailing when the work is to be done.

3. Continuing with the example, the time estimate shows the days' work for the outfit to be 96 days under good management and under average management, 126. Multiplying these by the total cost per day, $102, as indicated above, gives a direct expenditure of $9,792 under good management and $12,852 under average management. To this direct expenditure it is essential that four items be added, as follows:

4. The proportional part of the cost of wintering the stock which is properly allocable to the job. These are: Good management, $1,200; average management $1,600.[7]

5. Depreciation on stock and equipment: A 40-mule outfit generally represents an investment of $8,000 to $10,000. Few contractors can get an average of more than five years' work out of a mule. He must then be sold and replaced. His sale value will seldom be above $50. The annual depreciation per animal is, therefore, in common practice, from $30 to $40. That part of this depreciation properly chargeable to this job should be added, as should the probable repairs and depreciation on equipment. The latter is not a heavy item on fresno jobs; so, for the purposes of this example, the depreciation to be added may be assumed to be: Good management, $1,400; average management, $1,600.[6]

6. Moving costs: It costs a good deal to get from one job to the next. If the job is long enough to constitute a season's work the cost of getting back to winter quarters should be added. The costs are estimated as follows: Good management, $250; average management, $500.

7. Profit, estimated as follows: Good management, $3,000; average management, $3,000.

Recapitulating, the various items of cost are as follows:

	Good management	Average management
Direct expenditure for job	$9,792	$12,852
Wintering stock	1,200	1,600
Depreciation of stock and equipment	1,400	1,600
Moving costs	250	500
Profit	3,000	3,000
Total value of the job	15,642	19,552

Dividing these total costs by the total yardage moved, which is 100,000 cubic yards (excavation plus borrow), the prices which should be bid under the two conditions are as follows:

Good management	15½ cents per cubic yard.
Average management	19½ cents per cubic yard.

This estimate is based on the presumption that nothing but plowing is done by the plow teams. With small outfits—about 6 fresnoes—this is generally true, the plow team being allowed to stand idle a good deal of the time whenever the haul is long and the amount of earth which must be moved by the fresnoes is correspondingly small. With larger outfits, as the haul increases plow teams can be cut off and put at hauling. However, as outfits operating more than 10 fresnoes in a unit are seldom encountered on highway grading work the better practice in estimating is to consider the plow teams as used only on plowing, because there is little assurance that any large part of the time of the plow teams can be diverted to hauling.

Heavy clay, gumbo, and other extra-heavy materials, if dry, require more than the ordinary plow equipment, and, even after good plowing, load more slowly. This difference can be covered in the estimate of the cost of a day's work (1) by allowing for the extra stock and men required to do the extra-heavy plowing and (2) by allowing for extra men to do the loading, thereby relieving the teamsters and enabling them to keep the job up to schedule in other respects.

It is worthy of note that if a job is scheduled and estimated according to the plan outlined above, no extra allowance is needed for overhaul, as the overhaul is included in the bid price.

Such items as clearing and grubbing, maintenance of grade during the construction period, the cost of the clean-up, etc., are given no treatment here because the specifications of the various States differ so materially

[5] This item should cover all corral costs such as labor in caring for animals, feed, veterinary services, etc., but not depreciation on stock or equipment.

[6] For purposes of illustration it has been assumed that under average management this would be a full season's work, but that under good management it would not.

in these particulars that it has been considered advisable merely to refer to them and to state that such items, when included as part of the grading should be set up in the estimate as separate items just as depreciation, getting onto the work, etc., are set up.

A SAMPLE GRADING SCHEDULE

As a more complete illustration of these methods, there follows a schedule and an estimate based on an actual project. This schedule and the estimate, for purposes of illustration, are based on the presumption that fresnoes will be used throughout the job and that the management is equal to the average. In practice both wheelers and fresnoes are likely to be used on a job of this sort, the former being used wherever the haul much exceeds 350 feet. There is a little haul of this kind on this project but as it in no way affects the principles which are being discussed, this fact has been disregarded in preparing this schedule.

Detailed time schedule and estimate for a typical highway grading job using a 10-fresno outfit

Mile	Work	Quantity	Time units	Days work one fresno	Days work for outfit	Progress schedule	Unit price per cu. yd.
				Days	*Days*		*Cents*
1	Excavation	4,363.8				Start May 1.	
	Borrow	2,028.4					
	Total excavation	6,392.2×1.2	7,670.6				
	Borrow	2,028.4×0.5	1,014.2				
	Cut	145.4×2.7	392.6				
	Do	121.6×1.6	194.6				
	Do	167.6×1.75	293.3				
	Do	1,480.1×3.0	4,440.3			Finish May 8.	
		Divide by 200	)14,005.6(	70.0	7		17.1
2	Excavation	8,534.5					
	Borrow	2,549.1					
	Total excavation	11,083.6×1.2	13,300.3				
	Borrow	2,549.1×0.5	1,274.5				
	Cut	297.5×3.0	892.5				
	Do	302.5×2.0	605.0				
	Do	778.5×2.16	1,681.6				
	Do	1,193.2×2.10	2,505.7				
	Do	93.6×1.75	163.8				
	Do	12.6×3.0	37.8				
	Do	1,029.3×2.5	2,573.3				
	Do	1,735.4×2.17	3,765.8				
	Do	1,404.7×2.0	2,809.4			Finish May 26.	
		Divide by 200	)29,609.7(	148.0	15		21.2
3	Excavation	8,358.8					
	Borrow	2,224.9					
	Total excavation	10,583.7×1.2	12,700.4				
	Borrow	2,224.9×0.5	1,112.5				
	Cut	536.5×1.5	804.8				
	Do	902.5×1.75	1,579.4				
	Do	1,530.0×2.98	4,559.4				
	Do	69.4×1.95	135.3				
	Do	1,329.9×2.74	3,643.9				
	Do	2,504.8×2.6	6,512.5			Finish noon, June 14.	
		Divide by 200	)31,048.2(	155.2	15½		23.0
4	Excavation	8,096.6					
	Borrow	4,239.9					
	Total excavation	12,336.5×1.2	14,803.8				
	Borrow	4,239.9×0.5	2,119.9				
	Cut	1,084.2×3.0	3,252.6				
	Do	968.5×2.5	2,421.3				
	Do	348.5×2.85	993.2				
	Do	1,661.3×2.86	4,751.3				
	Do	285.9×1.25	357.4				
	Do	615.0×2.00	1,230.0				
	Do	2,860.4×3.00	8,581.2				
		Divide by 200	)38,510.7(	192.5	19	Finish noon, July 9.	24.1

Detailed time schedule and estimate for a typical highway grading job using a 10-fresno outfit—Continued

Mile	Work	Quantity	Time units	Days work one fresno	Days work for outfit	Progress schedule	Unit price per cu. yd.
				Days	*Days*		*Cents*
5	Excavation	2,911.9					
	Borrow	5,761.9					
	Total excavation	8,673.8×1.2	10,408.6				
	Borrow	5,761.9×0.5	2,880.9				
	Cut	648.0×3.0	1,944.0				
	Do	218.2×3.6	785.5				
	Do	302.8×2.98	902.3				
		Divide by 200	)16,921.3(	84.6	8½	Finish July 18.	15.3
6	Excavation	12,773.0					
	Borrow	961.8					
	Total excavation	13,734.8×1.2	16,481.8				
	Borrow	961.8×0.5	480.9				
	Cut	146.6×1.5	219.9				
	Do	646.8×3.0	1,940.4				
	Do	1,705.4×3.0	5,116.2				
	Do	1,581.7×1.5	2,372.6				
	Do	896.1×1.5	1,344.2				
	Do	1,237.0×1.5	1,855.5				
	Do	2,658.7×3.1	8,242.0				
	Do	1,736.9×3.0	5,210.7				
	Do	430.5×3.3	1,420.7				
		Divide by 200	)44,684.9(	223.4	22½	Allow 1 day for bad weather; finish noon Aug. 15.	25.7
7	Excavation	7,340.1					
	Borrow	8,273.5					
	Total excavation	15,613.6×1.2	18,736.3				
	Borrow	8,273.5×0.5	4,136.7				
	Cut	229.8×1.75	402.2				
	Do	286.3×3.01	861.8				
	Do	1,493.5×2.66	3,972.7				
	Do	2,801.6×2.80	7,844.5				
	Do	187.7×1.7	319.1				
		Divide by 200	)36,273.3(	181.4	18	Finish noon Sept. 6.	18.0
8	Excavation	6,339.5					
	Borrow	9,237.7					
	Total excavation	15,577.2×1.2	18,692.6				
	Borrow	9,237.7×0.5	4,618.8				
	Cut	280.0×1.5	420.0				
	Do	705.8×2.9	2,046.8				
	Do	1,074.0×2.58	2,770.9				
	Do	1,510.5×3.0	4,531.5				
	Do	308.3×2.6	801.6				
	Do	502.2×2.22	1,114.9				
		Divide by 200	)34,997.1(	175.0	17½	Allow 1 day for bad weather; Finish Sept. 27.	17.6
	Totals: Excavation	58,718.2	246,050.8	1230.1	123		20.5
	Borrow	35,277.2					

Total earth to be moved, 93,995.4.

123 days work @ $102	[1] $12,550
Winter coast of corral	1,600
Depreciation on stock and equipment	1,600
Moving to and from job	500
Profit	3,000
Bid price for job	19,250
Bid price per day ($19,250 divided by 123 days work)	156.50
Bid price per cubic yard ($19,250 divided by 93,995.4 cubic yard)	.205

[1] See estimate of value of day's work, page 14. Extensions to nearest $10 only.

The bid price per cubic yard which is developed by the above method is an average price in which all haul, including overhaul, is merged and in which no differentiation is made between borrow and excavation. Most contractors make little or no difference between borrow and excavation in presenting their bids and overhaul is generally underbid where the free-haul limit is within the fresno haul limit. Engineers are

(Continued on page 23.)

SAND-CLAY AND SEMI-GRAVEL ROADS STUDIED

PROGRESS REPORT OF COOPERATIVE RESEARCH ON SELECTED ROADS IN GEORGIA

IN JULY, 1922, a cooperative study of 29 Federal-aid projects in Georgia was begun by the State Highway Department of Georgia and the United States Bureau of Public Roads, the University of Georgia also participating. The work has been under the immediate supervision of C. M. Strahan, director of research, State Highway Department of Georgia, who in a progress report gives the scope of the work and the progress thus far made.

The 29 projects selected for study comprise a total of 184 miles, distributed generally over the State, and are representative of the variations in topography, climate, and soil conditions in the State. The research has for its economic aim the collection of data as to first cost, annual maintenance, residual worth, traffic density served, and period of efficient service with which to make economic comparison with other types of road, to fix the limits of cost and traffic density to which these types are best adapted, and to guide judgment as to replacement with the same or other materials. It has for its purpose in the physical field the determination of the causes of strength and weakness, the development of tests and standards, and the hope of experimental discovery whereby improved and more dependable road service can be secured from abundant and inexpensive sources of road-building material.

At the beginning of the investigation complete records were available on the cost of each project which had been constructed under engineering supervision in accordance with specifications approved for Federal-aid construction in Georgia. Since completion the projects had been maintained by the counties but were then in process of being put entirely under State control, thus permitting control of maintenance methods and allowing complete cost records to be kept.

The research was planned to cover the following program:

1. Inspection trips each spring and fall. On the spring inspection a record is made by means of notes and photographs of any weaknesses developed during the winter and the effect of current maintenance is observed. On the fall inspection samples of road surfacing are secured at selected points on each project for laboratory analysis and study and at the same time road conditions are noted.

2. The taking of systematic traffic counts (separating passenger cars, trucks, and horse-drawn vehicles). The plan has been to secure a 12-month daylight count on each project during eight days of each month, using two days in each week (Sunday and Monday of the first week, Tuesday and Wednesday of the second week, and so on).

3. The collection and study of maintenance costs on each project in the light of traffic counts.

4. The study in the laboratory of the several soil ingredients—i. e., gravel, clay, silt, and sand—with a view to determining more scientifically the physical, chemical, and mass action of such mixtures; to differentiate the binding values of various clays, if possible; and to investigate the interlocking bond strength of the coarser and fine aggregates involved.

It is planned to continue this line of research until the projects have reached the stage of reconstruction or substantial replacement, at which time an estimate of their residual or salvage value can be made to complete the economic showing desired.

The investigation has now been under way for two years and a considerable mass of data has been collected, but it is felt that it is too early to attempt to draw conclusions or to present a detailed analysis of the figures. Table 2, however, serves to give an idea of the character of information being collected. The figures under mechanical analysis of surfacing material represent the average composition of the surface on each project and are based on a total of 184 samples analyzed in duplicate.

For purposes of comparison, the following classification of sand-clay and top-soil mixtures by the Bureau of Public Roads is given.

TABLE 1.—*Characteristics of typical sand-clay and top-soil mixtures*

	Class A (hard)	Class B (medium)	Class C (soft)
	Per cent	*Per cent*	*Per cent*
Sand	65–80	60–70	55–80
Silt	0–15	10–20	10–20
Clay	9–18	15–25	10–25

CHANGES NOTED IN THE COMPOSITION OF ROAD SURFACES

A table similar to that given for the year 1922 has been prepared for the year 1923, and it is interesting to note the changes which have taken place in a year's time as indicated by samples taken at identical locations. The roads are maintained by road machines and drags which dress the shoulders as well as the surface, and the effect of this must be taken into consideration as well as that of wind and rain. Changes in coarse material were only such as might be expected from variation in sampling. The reduction in coarse sand (above No. 60 sieve), anticipated on account of the grinding and wear of traffic, was found to average 4 per cent.

An average loss of 2 per cent in total sand and a gain of 2 per cent in clay are both contrary to expectations. There is no change in the average amount of silt. The increase in clay occurs in all roads except three red-pebble, gravelly roads, in which losses in clay from 1.2 to 2.5 per cent are shown. Another year's wear may change the clay indications, but, assuming that this increase may be confirmed next year, the following comment is pertinent: With red-clay shoulders thinly covered or unprotected by soil, increase of clay would naturally follow from machining the surface. But the roads with sandy shoulders likewise show increase of clay, accompanied usually by the gradual formation of a loose sand covering on the wheelway. The assumption has been that the chief losses from these surfaces would be in silt and clay, the gathering of loose sand being taken as proof. If dry-weather loss from wind and dust alone be considered, this is probably true. But it is conceivable that traffic friction most easily dislodges the sand grains from embedment in the clay and silt; that any loosened clay and silt may tend to rebind aided by dews and light rain; that the washing rains act chiefly on the loose sand which they move to the side ditches. Hence the net result of both wind and water losses may be to remove a larger weight of sand than of clay. This is the more probable, since the total clay and silt is only 30 per cent while the total sand is from 60 to 70 per cent of the weight of the surfacing. Future samples from the loose material and from the upper 3 inches of the road surface will be taken to throw light on this seeming anomaly of increase in clay content.

A record has been made of the loss in thickness of the road surfaces. Here, too, such losses are the combined result of traffic wear and maintenance operations. The record shows no case where the original thickness has been increased. A maximum loss of 2 inches on one low-grade soil is shown for the year. Those projects with considerable amounts of gravel show the least loss of thickness. The average for all sample points was 0.6 inch. No consistent relation of thickness lost to traffic counts has appeared during the first year's wear. The roads with heavy traffic generally show a larger loss than those with light traffic, where the materials are reasonably comparable.

The figures for 1922 on annual cost per mile for maintenance of roadway and shoulders are based in part on the judgment of the division engineers, having been compiled during the first year of the State's control of maintenance while its organization and equipment were in the formative stage. For succeeding years these figures will rest on a direct account kept for each project. It will be readily seen how such cost data will be cumulatively useful in guiding budget estimates, in measuring the cost of traffic service, and in fixing the proper economic ratio of service to traffic density.

They bear also on problems of partial betterment to prolong the life of these roads under rapidly increasing traffic. Judging from the present record and from observed field conditions, it is tentatively thought that the better grades of road soils may be expected to show satisfactory service for a traffic density of 300 to 400 vehicles per day during a life of from five to seven years with an upkeep figure approaching $250 per mile.

The research studies are already leading to some conclusions which are thought to be well worth considering in designing these types of roads. It is believed that the maximum allowable amount of clay of 18 per cent in class A surfaces (Table 1) may be increased somewhat where more than 10 per cent of material above the 10-mesh sieve is present. Field observations show that the amount of clay desirable is not a fixed percentage of the total amount of fine material or binder since it is affected by the amount of coarse material with which the binder is associated. The extreme maximum of clay seems to be about 25 per cent of the mixture including gravel with a preferable amount of 20 per cent. Reduced to a percentage of the total amount of fine material, this would give a limit of 20 per cent clay where gravel constituted 10 per cent of the surfacing and 25 per cent clay for 30 per cent or more gravel. It has been observed that corrugations and potholes develop most commonly in those surfaces in which the clay percentage is low.

COARSE MATERIAL AND SMOOTH GRADING IMPORTANT

It is uniformly found that the greatest density of road surfaces of this type is in the upper 3 or 4 inches, with decreasing density and increasing moisture as the subgrade is approached. It is very desirable to know the variations of these factors in the road soil surfaces. The real sources of road efficiency in these materials vests in the mass action by which concentrated pres-

TABLE 2.—*Data collected on 29 projects in 1922*

Project No.	County	Length	Type of surface	Thickness of surfacing	Mechanical analysis of surfacing material [1]: Gravel in total sample	Sand retained on No. 60 sieve	Analysis of material passing No. 10 sieve: Sand	Silt	Clay	Daily traffic: Passenger vehicles	Trucks	Annual cost per mile for maintenance
		Miles		*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*			
1b	Henry	7.0	Top soil	7.5	11	50	76	11	13	800	110	$306
4	Walton	10.8	do	8.2	12	49	71	11	18	90	6	103
5	Bacon	17.6	Sand-clay	5.1	7	64	77	6	17	170	12	182
6a	Hall	5.7	Top soil	5.1	21	45	68	15	17	140	50	203
18	Dooly	10.4	Sand-clay	3.5	3	48	73	10	17	60	25	217
19	Bleckley	9.1	do	5.8	7	43	72	9	19	90	25	
22	Milton	10.0	Top soil	6.0	18	36	63	19	18	430	170	156
41	Douglas	6.6	do	3.8	8	52	72	15	13	230	65	88
49	Mitchell	13.4	Pebble	7.4	20	37	73	14	13	140	18	157
59	Jackson	4.3	Top soil	6.8	16	52	69	12	19	280	44	162
60	Stephens	11.3	do	5.4	9	46	74	11	15	230	22	100
68	Walton	9.1	do	9.7	7	53	73	14	13	140	16	114
76	Wheeler	9.9	Sand-clay	8.8	9	54	75	7	18	120	18	134
77	Charlton	19.2	Clay-gravel	8.0	35	60	72	6	22	190	13	188
124	Washington	5.9	Top soil	6.8	6	42	74	8	18	130	14	149
131	Jackson	7.1	do	6.4	12	54	72	10	18	280	44	166
134	Coweta	19.9	do	8.5	5	55	75	13	12	110	13	121
144	Macon	3.8	Sand-clay	6.2	0	39	76	12	12	210	22	207
145	Montgomery	12.4	Pebble	7.4	16	44	68	9	23	140	15	98
146	Tift	6.0	do	4.0	12	45	71	14	15	250	30	200
151	Floyd	7.3	Chert		50	34	50	20	30	120	20	61
178	Murray	6.6	do	8.1	38	25	40	25	35	160	20	50
179	Greene	2.7	Semigravel	10.9	23	50	65	21	14	260	26	122
189	Hart	3.8	do	9.9	9	56	75	11	14	310	22	201
196	Early	9.0	Sand-clay	7.8	6	52	74	8	18	120	15	149
197	Bulloch	4.8	Pebble	7.7	14	45	77	9	14	140	10	215
199	Quitman	5.0	Gravel	5.8	26	46	72	11	17	200	28	160
205	Effingham	4.2	Top soil		4	40	77	8	15	210	50	350
S-10-14	Richmond	8.4	Clay-gravel	6.3	36	68	75	4	21	300	46	128

[1] The No. 10 sieve is taken as the dividing line between course material and fine material. In analysis of fine material the sample is taken as 100 per cent after the removal of the coarse material.

sures and impacts are distributed and resisted, and in the ability of certain soil mixtures to maintain this mass action in the presence of notable percentages of water or their ability to so limit the ingress of water that the mass strength is not seriously diminished. The power of the road surface to resist water absorption and to avoid a too serious loss of mass stability under impact is the reason why certain soil mixtures are reliable and stable while others are hopelessly worthless.

Convinced by close observation that well-selected class C soils will pack into slabs of ample strength and possess in themselves sufficient resistance to undue water saturation, the Georgia Highway Department has recommended on several projects where class A materials were not available the following type of construction: An 8-inch bed of available class B or class C local soil to be covered with 2 inches of imported fine gravel, screenings, chats, or like hard material. The latter is applied before the soil bed is well packed. In the outcome the harder surface is securely bonded to the soil bed, consolidation of the road proceeds rapidly and evenly, and a firm slab of excellent durability is formed. The hard surface layer, though thin, protects the soil bed from direct abrasion and from dust and washing losses, which are the chief defects of these lighter grades of road soils. The mass action of the whole slab is strong and satisfactory.

The studies and observations are leading to a greater confidence in the influence on road soil and gravel types of the small-sized hard material lying between three-eighths and one-tenth inch in diameter. Particular attention is called to project 179, Greene County, where a class A binder is reinforced by 23 per cent of hard quartz particles all of which lie between a No. 4 and a No. 10 sieve. This road, which is easily mistaken for a paved road, has borne for three years without perceptible wear a traffic density of 281 vehicles per day, and the maintenance cost has been chiefly for building and upkeep of the shoulders.

Similar evidence is accumulating on some of the clay-gravel roads where material above the No. 4 sieve is less than 20 per cent, the deficiency being made up by the hard material just above the No. 10 sieve.

There seems to be an increase of density and a more uniform mass action where soil and gravel mixtures grade smoothly from coarse to fine. Within certain limits, the absolute size of the coarse aggregate seems less important than the uniformity of the grading. In this climate there appears to be less tendency to corrugation and pitting on gravel roads where a liberal amount of binder unites a graded gravel which starts at or below a 1-inch maximum size.

Close observations are being made on this important point. The local gravel deposits of Georgia are characterized by smaller sizes than the glacial gravels of the North and West. They are more commonly associated with clay binders which grade as class A material. They have shown excellent durability, smoothness, and low cost of upkeep.

EFFORTS MADE TO DEVISE TESTS FOR QUALITY

One of the major purposes of the research is to perfect methods of test for the control of these types of material as used in road construction. Attention has been directed to a review of present standards and laboratory methods and to devising new tests and apparatus in promising directions in addition to the field experiments carried on in connection with the current construction and maintenance work of the Georgia Highway Department. Much attention has been directed toward the study of the behavior of the clay ingredients. Experiments on clay have been made by cementing thin brass plates with clay films. The plates, 1 inch wide, are lapped to give 1 square inch of cemented surface and the break is made by a pull parallel to the surface. It was hoped to measure thus the adhesion of the clay to the plates, or the shear in the clay film, according to the observed fracture. Quantitatively this series of tests has been disappointing. Qualitatively it has been significant of the influence of water on clay behavior. The greatest adhesion was shown by wet films of stiff mud consistency. On drying in air, or completely by artificial heat, adhesion to the plates was rapidly lessened and completely destroyed when dry. The plates were clamped together strongly while drying, but to no avail. The test suggests that the plasticity and adhesion of clay are in large part the result of colloidal films established when water is present.

Wet soil briquettes 1 square inch in area were molded in a divided mold and broken by a tension pull. Using increasing percentages of water, the breaking weights increased to a maximum and then diminished to zero. Considerable differences are noted in the water capacity of different soils in that part of the curve where the tenacity is greatest; also in the absolute amount of the breaking weights.

Of the soils tested, those with low clay content reached a maximum and stayed high with 22 per cent moisture, while those with a larger clay percentage yielded under less pull with about 15 per cent water. The test is quickly made and simple and has value in selecting soils which preserve their stability with high percentages of water.

The test is applicable to dry briquettes, but here the tension figure steadily increases with the percentage of clay and thus draws no line for samples carrying excess clay.

A series of tests, consisting of the breaking of 1 by 1 inch molded bars of soil 5 inches long, has also been made. The soils were mixed to normal consistency—i. e., plastic enough to prevent packing when placed in the molds. The bars were dried in air or on a water bath. The 5-inch bars were broken on a 4-inch span by a transverse load applied at the third points. Most of the bars ruptured at the center of the span or at the loaded points. Each material was tested in duplicate bars. The short broken pieces, four for each material, were further broken in shear by a center knife-edge load on a 1-inch span. Fifty-four soils were tested. The range of breaks for the 4-inch spans was from 2,700 to 15,000 grams. About 60 per cent of these showed a satisfactory agreement of the duplicate breaks (within 10 per cent of the average).

Those which broke in the center of the 1-inch span yielded in every case by diagonal shear toward the point of support. The range of breaks was from 6,000 to 30,000 grams. The four tests on each material gave stisfactory agreement in 50 per cent of the soils tested. The extremely low breaks were associated with the poorer roads, but the medium and high breaks followed no significant relation to the road history of the samples.

A disk shear or punching test was devised and tried as follows: Disks of the soil 2 inches in diameter and

1 inch thick were molded from stiff mud under 3,000 pounds total compression. When dried, the disks were placed in a punching cylinder to fit and fractured by a 1-inch diameter steel plunger acting in line with a 1-inch hole in the bottom of the cylinder. Three disks were made for each material and 31 soils were tested. The breaking loads ranged from 425 pounds minimum to 3,340 pounds maximum. The average agreement of the three disks for each material was fairly good. It was felt, however, that the ratio between the areas of plunger and disk is too large for best results. It is planned to repeat the test using 4-inch disks and a three-quarter inch plunger. A test of this kind is very promising, but the labor of making the disks is considerable.

A METHOD FOR THE DYE-ADSORPTION TEST

The dye-adsorption test for clays, as originally devised by the Bureau of Public Roads, has been applied in the Georgia work. Both crystal violet and methylene blue have been used as the adsorption color media. The standard filter tubes, as used by the bureau, and a colorimeter method have been used. The colorimeter adsorption method was devised in order to use directly a liquid sample from the clay washings. Comparative tests on clay which had been recovered by evaporation to dryness with the same clay tested while still moist showed much higher adsorption in the latter case. Hence it was felt that if the clay washings could be used directly much time in recovering the clay would be saved and the adsorption figures would more closely represent the nature of the original clay. The method proceeds as follows: The clay washings are caught in a large jar and brought to known volume (say 2 liters) with distilled water. From the freshly and vigorously shaken liquid quickly take 25 cubic centimeter samples by a standard pipette. Evaporate one sample to dryness to determine the amount of clay in each sample. Place the other samples in flat-bottom glass flasks and add an excess of standard methylene blue solution (1:1000), recording the amount added. Bring the flask to vigorous boiling and continue for three minutes. Transfer and rinse the flask liquid into a cylindrical graduate and dilute with distilled water to fixed volume, 50 or 100 cubic centimeters. Allow this to settle until perfectly clear and take an accurate pipette sample for use in the colorimeter tube and compare by the usual colorimeter procedure. Having thus determined the residual amount of methylene blue not adsorbed, this amount subtracted from the original addition will give the amount of methylene blue adsorbed.

A batch of 12 samples is easily carried on by this method and duplicate determinations are very close. The adsorption test is very delicate, dealing with minute actual weights of coloring matter and probably influenced by the presence of small amounts of ionizing salts. To be significant in distinguishing clays, the test conditions must be very carefully standardized. It is, however, one of the most promising tests yet proposed for differentiating substances like the clays.

Following the Bureau of Public Roads methods for separation of the total clay into coarse clay and suspension clay by means of the centrifuge, some work has been done on the Georgia road soils. As compared with other records, these clays seem to be quite low in suspension clay. The tests are being continued and some of them repeated. An experiment is in progress to show whether the suspension clay is a fixed quantity in a given sample or whether the coarser particles of clay can be made to pass into the colloidal suspension state.

Another test has been recently planned for field use as the result of preliminary laboratory tests on large slabs of road soil. It may be called a penetration or a bearing-power test. A simple apparatus was devised by which a steel rod one-tenth inch in diameter could be loaded to a ¼-inch penetration when resting on a road fragment about 6 by 6 inches in size. Unit pressures from 4,000 to 7,000 pounds per square inch were recorded on the dry specimens, and 1,600 pounds was held by a road soil with 15 per cent moisture as tested in the field.

A field apparatus similar to an automobile jack but operated smoothly by screw power is being made, with a penetration needle (three sizes), a pressure indicator and a cut-off to regulate the penetration distance. Using the rear-axle weight of his car, the observer with this apparatus will be able to take a large number of bearing-power readings on the surface of the road and at successive depths as desired by digging off the top material. He will also take samples to be put into air-tight bottles for laboratory determination of moisture corresponding to the bearing-power tests. This proposal is similar to the more elaborate subgrade apparatus in use by other investigators for bearing-power tests.

HIGHWAY RESEARCH BOARD TO STUDY CONCRETE REINFORCEMENT

The Highway Research Board of the National Research Council announces the beginning of an investigation of the economic value of reinforcement in concrete roads. Director Charles M. Upham reports that the various State highway departments will cooperate with the board in conducting the investigation. Inspections of pavements will be made in the States of New Jersey, Ohio, New York, Pennsylvania, Delaware, Wisconsin, Iowa, Illinois, and California, and in Wayne County, Mich., and Milwaukee County, Wis.

An effort will be made to determine from a survey of existing roads the influence of steel reinforcement on the resistance of the slab to traffic, subgrade, and climatic conditions; the conditions under which steel reinforcement is especially beneficial to a concrete slab; the effect of slab design on the efficiency of reinforcement; and finally the relative cost of plain and reinforced concrete roads, considering the initial investment, and the annual maintenance and renewal charges.

The procedure will consist of a personal examination of a sufficient number of existing road surfaces to cover different slabs, traffic, and climatic conditions. It is proposed to supplement the examination by photographs, sketches, soil determinations, and other available data. In each case attention will be given to a study of the subgrade to determine its general characteristics and properties as well as the existing drainage conditions.

It is expected that a progress report will be ready for the annual meeting of the Advisory Board on Highway Research to be held at the National Research Council Building, Washington, D. C., December 4 and 5, 1924.

CAUSES OF NONUNIFORMITY OF CONCRETE

A SYMPOSIUM

In May, 1924, the Bureau of Public Roads forwarded to each of its district offices and to all State highway departments a tabulation showing the results of compression tests on cores drilled from several concrete pavements in each of four widely separated States. The test results indicated a lack of uniformity in strength of the concrete taken from the several projects, and the bureau requested those to whom the tabulation was sent to reply with suggestions as to ways and means of improving the uniformity of concrete.

Replies were received from 27 State highway departments and practically all of the district offices. The replies have been analyzed and are now presented in tabular form for the information of those who participated in the discussion and for highway engineers in general. It was hoped by this means to bring out corrective measures of immediate practical application. While the opinions expressed are interesting in their variety, it must be conceded that the majority of the remedies suggested are either quite obvious or lacking in the means of application. The divergence of the opinions is, at least, significant of the complexity of the problem of uniform concrete production.

The subject as a whole is one of prime importance, and it is thought that publishing the information collected will suggest new lines of thought for some and perhaps stimulate investigation and research. All of this should ultimately be conducive to better concrete and better pavements.

Abstract of suggestions of State highway departments and district offices with regard to nonuniformity of concrete as demonstrated by the compressive strength of cores drilled from pavements

Causes of nonuniformity	Suggested by—		Remedies for nonuniformity	Suggested by—	
	States	District offices		States	District offices
1. Cement:			1. Cement:		
(*a*) Variation in cement from one plant	2	1	(*a*) Beyond reasonable control	2	
(*b*) Variation between different brands	3	2	(*b*) Use only one brand on a project		1
(*c*) Storage of cement		1			
2. Water:			2. Water:		
(*a*) Variation in quality	2				
(*b*) Acidity or alkalinity	2		(*b*) Test with litmus paper	1	
3. Aggregates:			3. Aggregates:		
(*a*) Variation in grading	7	6	(*a*) Educate material plant personnel	1	
			Beyond reasonable control	1	1
			Rigid inspection		2
			Control by tests	2	
(*b*) Variation in cleanliness	2	2	(*b*) Beyond reasonable control	1	2
(*c*) Variation in quality	2	2	(*c*) Beyond reasonable control	1	1
			Control by tests	1	
(*d*) Segregation of coarse aggregate in stock piles	2	1	(*d*) Stock pile in layers	1	1
			Combine separate sizes at proportioning plant	2	1
(*e*) Bulking of sand due to moisture	5	1	(*e*) Weigh aggregates and correct for moisture	1	1
4. Construction processes:			4. Construction processes:		
(*a*) Variation in consistency of concrete	21	6	(*a*) Slump test	2	
			Beyond reasonable control	1	1
			Weigh aggregates		1
			Accurately measure and proportion water	4	1
			Accurately control water for each batch	3	
			Add hydrated lime to cement	1	
			Use moderately wet mix and then squeeze out excess water.	1	
			Closer observation by inspector and foreman	1	
			Use mixer which will discharge concrete of proper consistency.		1
			Correct leaky valves on mixer		1
(*b*) Inaccuracy in measuring aggregates	12	5	(*b*) Proportion by weight	3	3
			Measure accurately and uniformly by means of independent measuring devices.	1	
			Use accurate measuring devices	1	
			Use batchers	1	
(*c*) Segregation of concrete in mixer and in pavement	5		(*c*) Improve control of consistency	1	
			Avoid central mixing plants	1	
			Decrease maximum size of stone	1	
			Change design of mixers	2	1
(*d*) Variation in time of mixing	9	2	(*d*) Rigid inspection		1
			Require 1½-minute mixing period	1	
			Require 2-minute mixing period	1	
			Use batch meter	2	
			Require uniformly definite time of mixing	1	
(*e*) Overstanding	1	1	(*e*) Proportion aggregates by weight		1
(*f*) Arbitrary proportions	2	1	(*f*) Use scientific proportions	1	
(*g*) Too much tamping	1		(*g*) Tamp slightly with not too dry a mix	1	
(*h*) Insufficient tamping	5		(*h*) Use mechanical tamper	3	
			Tamp uniformly	1	
(*i*) Use of boom-and-bucket mixer	1				
(*j*) Variable cement factor	1		(*j*) Careful inspection	1	
(*k*) Equipment in poor condition	1				
(*l*) Variation in manipulation	4	1	(*l*) Avoid shoveling and handling after dumping	2	
			Use boom-and-bucket mixer	1	
(*m*) Variation in pavement thickness	1	1	(*m*) Accurate subgrade and rigid inspection		1
5. Curing:			5. Curing:		
(*a*) Variation in later curing due to wet and dry subgrades.	1	1	(*a*) Beyond reasonable control	1	1
(*b*) One section shaded, another exposed to sun	1	1	(*b*) Beyond reasonable control	1	1
(*c*) Nonuniform curing	6	3			
(*d*) Variation in nature of material used for cover	1	2			
(*e*) Variable absorption of subgrade	2	2	(*e*) Use tar paper on subgrade		1
(*f*) Inadequate curing	5	2	(*f*) Pay more attention to proper curing	1	
			Keep pavement wet for at least 14 days	1	
			Have contractor cover this item in his bid	1	
(*g*) Absorption by dry subgrade	1		(*g*) Wet the subgrade	1	
(*h*) Temperature changes during curing period		1			
(*i*) Temperature changes during construction period	3		(*i*) Protect work adequately during early fall	1	
(*j*) Rise or fall of temperature during settling period	1				
(*k*) Effect of freezing	2		(*k*) Lay no pavement at 40° F. when temperature is falling	1	

Abstract of suggestions of State highway departments and district offices with regard to nonuniformity of concrete as demonstrated by the compressive strength of cores drilled from pavements—Continued

Causes of nonuniformity	Suggested by— States	Suggested by— District offices
6. Engineering supervision:		
(a) Poor control and inspection	8	3

Remedies for nonuniformity	Suggested by— States	Suggested by— District offices
6. Engineering supervision:		
(a) Require carefully supervised system of inspection and testing	1	
Need high-class, intelligent inspection and constant engineering control	1	
More rigid specifications and enforcement of same through efficient inspection	1	
Rigid inspection		1
Adequate inspection		1
Intelligent inspection		1
More efficient and competent inspection	1	
Rigid inspection and control of materials, also accurate and complete inspection record	1	

7. Miscellaneous:	States	District offices
(a) The main feature yet to be perfected is the accurate measurement of the mixing water	1	
(b) Study of design and specifications required to get uniform working conditions		1
(c) Extend the laboratory work by relating the test record as nearly as practicable to construction conditions	1	
(d) Study means of controlling variable moisture contents of subgrade due to weather and soil conditions	1	
(e) Investigate means of allowing for bulking of sand due to moisture	1	
(f) Surface finish of pavement considered to be a factor of supreme importance	1	
(g) Adequate and competent engineering supervision, and accurate proportioning of materials are the whole solution		1
(h) Uniform concrete is being obtained as a result of uniform consistency and accurate measurement of materials on all contracts	1	

Abstract of comments on the compression test of cores and the use of the test results as an indication of the quality of concrete in pavements

Comments on compression test	Suggested by— States	Suggested by— District offices	Remedies for inaccuracies of compression test	Suggested by— States	Suggested by— District offices
1. Ends of specimens not true planes perpendicular to vertical axis.	3	1	1. Grind ends of caps	1	
2. Size of specimen not proportional to maximum size of coarse aggregate.	1		2. Cast some sections of pavement 12 inches thick for drilling cores.	1	
3. Variation in speed of test	3	1	3. Use greater care to get uniformity		1
4. Age of specimens not given proper consideration	2	3			
5. Variable diameter of the core		1			
6. Variation in height of cores	2				
7. Injury to cores by drill	3		7. Use molded cylinders for early ages	1	
8. Personal equation of test operators	1				
9. Strain in specimen due to temperature stresses	1		9. Do not drill close to a fracture	1	

Miscellaneous comment (From three States and two district offices):

1. Does the compression test on cores indicate the quality of concrete in the pavement?
2. The core test is unreliable as a criterion of the strength of concrete.
3. The nature of the specimen and the method of test make the core test unreliable.
4. More reliance should be placed on the tensile-strength test than on the compression test.
5. Use compression test only in conjunction with test on beams.
6. Use average of three cores from one locality for the equivalent of one core result.
7. Investigation required to determine reliability of core test.
8. Use beam tests instead of test on cores.

The first and larger part of the tabulation is devoted to the opinions of those who accept the results of the compression test on drilled cores as a measure of the strength of the concrete in the pavement. The numerous recommendations for rigid inspection and close field control stand out prominently among the remedies suggested. This factor, undoubtedly of vital importance, is mentioned by some States in which inspection and control are notoriously weak, which suggests the question as to why, if the weakness is recognized, the necessary steps are not taken to correct the faulty conditions. By comparison of paving work in various localities it is quite apparent that the additional cost of intelligent inspection and a well-planned control of materials and construction is a profitable investment, which is reflected in the quality of the pavements. Without adequate control no scheme for producing better pavements can be properly administered or enforced.

It is of special interest to note that one State claims to obtain uniform concrete as a result of uniform consistency and accurate measurement of materials. This claim is supported by the results of core tests on a number of pavements.

The second part of the tabulation deals with the validity of the compression test on cores. Certain factors are pointed out which may influence the test results to a considerable extent, and it is well that attention be directed toward them. Definite objection is also expressed to the policy of using the results of core tests as an indication of the strength of the concrete. It still remains to be demonstrated that the beam test is a better or more reliable criterion than the compression test on cores. If it is eventually accepted that the beam test is superior, it will then be necessary to develop some convenient and practical method for obtaining beam specimens from the pavement. It is not contended that pavements fail under static load by direct compression, but on the basis of present information it seems reasonable to use the results of tests on cores in a comparative manner for any one pavement as a measure of the strength of the concrete.

The methods employed in constructing the average concrete pavement are in themselves the most convincing evidence that there must be a considerable nonuniformity of concrete. The variation in grading of the coarse aggregate, the lack of accuracy in measuring aggregates, the variable consistency, the shoveling of mortar from fresh batches to supply deficiencies in preceding batches, and inadequate or nonuniform curing are some of the main factors which can be verified by mere inspection and which are apt to contribute to variation in the strength of the concrete. How to correct or improve on these conditions is the problem.

ROAD MATERIAL TESTS AND INSPECTION NEWS

SIEVE LIMITS FOR THE MECHANICAL ANALYSIS OF SUBGRADE SOILS

The subgrade laboratory of the Bureau of Public Roads has been endeavoring for some time to standardize the tests used in determining the physical properties of subgrade soils. One of the essential tests is the mechanical analysis of the soil, and in the consideration which has been given to this test the principal question has been that of the choice of sieves.

The United States Bureau of Soils in all its analyses has employed metric sieves, which separate the soil into constituents the size limits of which are expressed in millimeters. The sieves generally used in highway practice for the analysis of sands, topsoils, and sand-clay mixtures have sizes of opening which are defined by the number of meshes to the inch, and the sizes of opening thus defined differ materially from those employed by the Bureau of Soils.

Notwithstanding the fact that much of the work of previous soil investigators has been based on the metric system, it has been deemed advisable, after careful consideration, to employ the same sieves for subgrade soil investigations as for the analysis of other highway materials, mainly because they are already a part of the equipment of highway testing laboratories, and because analyses made with them are more intelligible to the highway engineer.

The analyses published in this issue of Public Roads, in the article entitled "Reinforcing and the subgrade as factors in the design of concrete pavements," were made with Tyler standard mesh sieves. It will be noted that the openings are defined by the number of meshes to the inch. The fractions designated as clay and silt are those which pass the 200-mesh sieve, and they are separated by subsidence in water according to a method which classifies particles whose diameter is 0.010 millimeter and less as clay and above that size as silt. The desirability of adopting one method for all subgrade analyses will be perceived if the reader will compare the above analyses with those published in the August issue of this magazine in the article entitled "Practical field tests for subgrade soils." The latter analyses were made with metric sieves according to the method followed by the Bureau of Soils.

It will be noted, for example, that in the important matter of determining the percentage of clay the two methods of analysis give different percentages because the particles designated as clay by the two methods have different maximum sizes. As stated in the footnote on page 4, the maximum size of the clay particles in the Bureau of Public Roads analysis is twice as great as the size of particles so designated by the Bureau of Soils method using metric sieves. It naturally follows that the percentage of clay indicated by the Bureau of Public Roads method is considerably greater than the Bureau of Soils method shows. The difficulty of comparing the results of the two methods is illustrated by the parallel grouping of the limits of the two methods in the next column.

Relation between metric sieves and Tyler standard mesh sieves

Analysis by metric sieves		Size of Tyler mesh	Analysis by Tyler standard mesh sieves	
Designation	Sieve size		Sieve No.	Designation
	Millimeters	*Millimeters*		
	2.000			
				Coarse material.
		1.651	10	
Fine gravel				
	1.000			
Coarse sand		0.833	20	Sand.
	0.500			
Medium sand				
	0.250			
		0.221	60	
Fine sand				
		0.147	100	
	0.100			
Very fine sand		0.074	200	
	0.050			
Silt				Silt.
		0.010		
	0.005			
Clay	0.000	0.000		Clay.

By plotting the mechanical analysis obtained by either method as a cumulative percentage curve it is possible to convert one analysis into terms of the other, although the values obtained are, of course, only approximate. The abscissae of such a curve would represent the sieve sizes in millimeters, and the ordinates cumulative percentages passing the respective sieves. The conversion is made by reading from the curve the cumulative percentages represented by the ordinates of the points at which the curve is cut by perpendiculars to the horizontal axis representing the size limits of the other system of sieves. By subtracting the cumulative percentages thus obtained for each sieve from the corresponding percentages for the next larger sieves the percentage retained on each sieve is obtained.

The result of converting two of the analyses in Table 2, page 4, into the form in which they would appear if the analyses had been made with metric instead of with Tyler standard mesh sieves is shown below.

Mechanical analysis

Section No.	Station	Millimeters, per cent between—							
		Over 2	2-1	1-0.5	0.5-0.25	0.25-0.1	0.1-0.05	0.05-0.005	0.005-0
		Coarse gravel	Fine gravel	Coarse sand	Medium sand	Fine sand	Very fine sand	Silt	Clay
1	28+00	22.5	3.8	10.0	10.7	11.4	6.1	22.5	13.0
2	30+00	17.0	4.0	11.5	12.5	17.7	7.3	20.0	10.0

The analyses published in this issue were made with Tyler sieves and these analyses have been used by way of example in the foregoing comparisons. These sieves are used at present by the Bureau of Public Roads laboratory, but the bureau does not wish to imply that the Tyler scale should be adopted as the standard for highway work. On the contrary, as stated in the April issue of Public Roads, the bureau believes that the Bureau of Standards screen scale should be generally adopted and is prepared to conform to that scale as soon as it is accepted. In all likelihood it will ultimately be adopted by the American Society for Testing Materials and the American Association of State Highway Officials, and the bureau is hopeful that such action will be taken with as little delay as possible.

(Continued from page 15)

prone to take advantage of bidding of this sort and often widen cuts to secure extra material because, under the accepted bid, it is as cheap to do this as it is to borrow alongside and, in their opinion, better for the road. As a matter of fact, under an average unit price bid such as the above, the contractor is fairly certain to lose on the longer hauls involved in extra work built with material secured in the cuts. On the other hand, under this system of bidding the State pays too much for extra material secured from side borrow. Hence, it would seem to be better for every one concerned if a correct differentiation were made between excavation and borrow and if overhaul were bid in at a correct figure. The following indicates the method which may be pursued in making such a differentiation and shows the relationship which really exists between the cost of side-borrow and excavation on an ordinary fresno project. Overhaul is also calculated as a separate item, a 200-foot free haul (center to center of mass rule) being assumed. The details of the recast are not given.

Item	Unit	Amount	Time units	Days work for outfit of 10 fresnoes assumed	Bid price on work at \$156.50 per day	Computed unit price bid
		Cu. yds.		*Days*		*Cents*
Borrow	Cubic yards	35,277.2	59,971.2	30	\$4,695	13.3
Excavation	do	58,718.2	153,251.7	76.6	11,988	20.4
Overhaul	Cubic yards per station.	32,827.9	32,827.9	16.4	2,567	7.8
Total			246,050.98	123	19,250	

From the standpoint of the officials responsible for design, bids which differentiate between excavation and borrow should be of considerable value for they would at once place a large premium on careful design involving the avoidance of all unnecessary haul if fresnoes are to be used. From the standpoint of the contractor a bid of this kind is equally of advantage for while working under it field changes can not deprive him of his profit as sometimes happens when borrow and excavation are taken at the same price and the engineer decides to substitute excavation (wider cuts) for a large part of the borrow. This style of bidding is, then, of value to the State and is a real protection to the contractor's profits.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

SEPT. 30,1924

STATES	FISCAL YEARS 1917-1924			FISCAL YEAR 1925									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1924			PROJECTS COMPLETED SINCE JUNE 30, 1924			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 4,598,721.63	$ 2,186,247.54	464.1	$ 575,007.24	$ 284,651.86	86.1	$ 15,530,308.90	$ 7,534,554.61	836.6	$ 79,416.15	$ 39,708.07	0.6	$ 1,207,800.92	Alabama
Arizona	8,338,365.41	4,287,683.88	527.8	713,173.87	406,825.70	37.5	1,600,010.00	968,451.46	127.3	222,983.72	136,265.33	32.7	1,696,474.63	Arizona
Arkansas	11,094,751.31	4,424,345.63	944.4	122,739.41	50,850.57	4.9	7,923,828.33	2,920,437.11	394.2	544,200.78	239,497.48	49.2	1,427,269.21	Arkansas
California	12,999,075.03	5,647,148.17	533.7	2,715,077.91	1,473,319.61	90.2	12,759,091.73	6,642,729.58	439.3	435,211.04	256,774.51	1.3	3,073,334.13	California
Colorado	8,108,070.31	4,029,898.97	502.6	323,679.33	174,547.77	28.9	5,582,952.15	3,015,081.10	193.8	35,740.10	20,057.34	2.9	2,320,295.82	Colorado
Connecticut	3,062,872.02	1,269,558.60	73.6	199,024.56	98,423.00	4.9	3,262,054.48	1,045,804.04	54.1				967,409.36	Connecticut
Delaware	3,056,832.22	1,007,714.83	72.5	462,969.09	197,825.82	13.8	608,205.95	214,911.10	13.8	710,582.29	289,320.00	19.3	29,758.25	Delaware
Florida	961,134.07	461,470.92	48.8	441,141.73	220,570.86	12.8	9,225,920.22	4,498,526.04	248.7	398,642.71	171,123.94	10.3	935,195.24	Florida
Georgia	17,167,373.32	7,955,805.20	1214.2	534,460.21	265,330.76	46.2	9,828,434.90	4,879,632.73	757.1	1,784,431.08	843,312.03	108.9	505,816.28	Georgia
Idaho	8,181,697.92	4,092,395.52	506.8	197,306.67	113,420.29	18.7	1,644,869.99	949,544.65	118.4	809,078.31	533,904.13	50.8	988,447.41	Idaho
Illinois	26,964,706.06	12,279,546.33	804.7	663,460.35	327,523.92	20.6	16,436,813.06	8,167,813.86	558.5				2,661,607.89	Illinois
Indiana	7,577,444.16	3,655,540.97	225.7	1,182,752.58	581,613.20	41.2	15,951,610.62	7,770,338.46	517.5				2,304,899.37	Indiana
Iowa	23,195,778.19	9,237,031.86	1682.9	282,803.25	113,620.08	20.2	8,697,847.92	4,038,449.47	632.0	297,707.62	147,800.00	25.6	1,799,235.59	Iowa
Kansas	17,084,136.48	6,043,176.80	502.7	3,806,022.20	1,404,334.99	108.9	15,347,582.65	6,104,078.33	618.0	1,476,394.85	706,878.71	92.2	1,040,820.17	Kansas
Kentucky	10,822,980.31	4,613,947.28	429.4	596,589.15	241,290.72	23.2	8,917,340.25	4,119,294.28	338.4	680,318.04	307,498.91	33.5	1,089,707.81	Kentucky
Louisiana	8,488,463.18	3,636,143.36	661.2	738,057.99	358,026.14	51.5	5,254,523.47	2,602,920.59	338.3	271,338.99	135,669.49	9.1	532,682.42	Louisiana
Maine	6,911,058.78	3,299,935.38	230.7	260,943.30	129,792.71	10.0	1,613,995.65	802,565.56	62.3	235,761.31	93,583.78	8.7	764,094.57	Maine
Maryland	6,760,044.42	3,213,321.78	243.2	230,745.50	115,372.74	10.8	2,382,355.91	1,017,083.73	73.5	741,077.98	294,426.81	21.5	8,744.94	Maryland
Massachusetts	10,191,202.02	4,105,727.22	232.8	88,434.91	28,760.00	1.4	6,377,796.76	2,041,497.59	106.5	485,367.78	146,756.38	7.2	1,597,038.81	Massachusetts
Michigan	13,434,135.07	6,060,612.23	494.5				14,428,493.46	6,913,576.62	507.5				2,905,583.15	Michigan
Minnesota	24,037,561.24	9,885,843.07	2292.0	675,006.59	304,379.76	53.2	9,973,348.31	4,631,929.97	842.3	443,530.95	196,400.00	80.2	299,866.20	Minnesota
Mississippi	7,888,193.89	3,828,941.39	655.0	357,385.12	178,692.55	18.0	8,235,805.97	4,103,716.98	493.2	601,622.36	299,873.30	43.9	1,120,048.78	Mississippi
Missouri	11,352,027.70	5,245,899.18	803.5	592,423.54	295,591.85	49.2	18,273,683.60	8,752,448.06	766.2	3,797,210.31	1,775,686.46	196.3	1,870,562.45	Missouri
Montana	8,867,279.16	4,384,335.12	791.4	108,869.05	87,529.84	12.4	2,884,950.84	2,115,151.46	268.2	1,085,404.95	615,745.03	122.2	3,763,655.55	Montana
Nebraska	7,876,337.16	3,714,691.59	1440.4	200,802.81	100,401.40	22.6	7,694,728.23	3,751,494.73	788.4	1,028,632.63	514,316.26	120.0	3,370,042.02	Nebraska
Nevada	3,460,245.52	1,853,624.98	225.6	587,270.26	511,310.71	43.3	4,870,499.36	4,077,547.69	412.6	168,117.37	146,631.94	19.3	301,205.68	Nevada
New Hampshire	3,076,750.19	1,487,867.58	171.3	20,801.55	10,308.46	1.1	1,443,234.25	678,912.21	47.3	138,590.95	67,471.46	4.6	190,404.27	New Hampshire
New Jersey	7,623,795.12	2,661,531.49	148.7	298,751.09	111,360.00	5.6	9,724,981.71	2,795,603.75	73.4	796,997.14	205,677.49	10.6	815,074.27	New Jersey
New Mexico	5,306,286.45	2,758,849.68	714.3	381,468.92	231,065.57	51.3	6,620,900.78	4,348,083.59	656.8	389,438.39	239,504.59	27.7	1,011,828.57	New Mexico
New York	18,862,742.49	8,257,844.44	572.7	468,489.72	191,631.63	14.5	29,534,066.96	11,183,066.72	659.6	5,016,794.00	1,427,433.75	92.9	5,648,171.46	New York
North Carolina	12,567,732.97	5,676,757.66	884.7	241,113.23	112,077.78	12.5	11,200,653.19	4,208,856.02	316.5	1,719,594.43	807,790.46	39.9	1,488,769.08	North Carolina
North Dakota	9,088,973.11	4,418,505.42	1587.9	376,098.20	187,597.07	81.0	3,641,820.37	1,785,327.54	565.6	184,426.85	92,213.39	20.8	1,880,012.58	North Dakota
Ohio	33,122,751.43	11,879,917.99	962.5	1,205,099.59	468,814.13	30.3	13,210,878.97	5,014,387.60	364.1	2,187,800.71	741,115.00	68.8	2,035,929.28	Ohio
Oklahoma	12,986,865.26	5,888,852.03	497.3	1,187,137.45	572,302.34	52.8	8,234,521.73	3,959,190.42	364.5	2,269,348.24	998,272.23	99.3	1,118,085.98	Oklahoma
Oregon	12,082,873.17	5,819,093.79	655.6	513,232.52	303,539.04	48.5	3,347,304.72	1,980,045.65	181.6				403,480.52	Oregon
Pennsylvania	36,825,248.98	14,114,694.79	729.7	415,284.23	176,961.99	9.7	22,662,779.55	6,306,421.25	381.1	2,673,574.86	742,641.25	48.7	3,260,896.72	Pennsylvania
Rhode Island	1,774,397.25	779,227.96	46.0	157,932.08	62,600.00	3.1	1,243,889.16	438,306.80	25.2	380,178.15	92,700.00	6.2	560,206.24	Rhode Island
South Carolina	9,016,476.73	4,124,045.22	924.4	824,054.97	372,624.85	142.2	5,783,649.52	2,402,577.79	423.0	603,658.77	190,330.15	57.7	597,967.99	South Carolina
South Dakota	8,674,597.86	4,244,636.27	989.8	845,706.40	453,412.58	120.0	7,426,924.78	3,701,926.79	1000.0	366,912.37	183,456.16	56.4	135,248.20	South Dakota
Tennessee	6,805,683.35	3,313,936.07	259.6	1,261,420.91	630,710.45	37.5	14,016,675.13	6,393,573.37	450.3	1,782,093.23	809,835.64	88.6	876,581.47	Tennessee
Texas	42,341,998.56	16,190,624.91	3122.8	2,121,853.15	951,671.72	186.0	25,591,436.99	10,134,114.56	1562.0	3,111,221.73	1,212,001.07	175.4	3,235,800.74	Texas
Utah	3,304,423.75	1,895,805.92	219.0	719,066.79	390,456.24	91.2	3,658,519.93	2,411,856.29	212.9	1,211,778.50	839,681.79	78.1	578,672.76	Utah
Vermont	1,922,114.16	942,769.12	74.4				1,999,561.44	956,178.98	50.8	165,057.47	73,677.20	5.1	561,353.70	Vermont
Virginia	10,035,301.48	4,801,782.43	562.5	312,421.07	140,321.52	8.9	9,242,088.09	4,299,993.56	323.7	1,238,244.82	476,332.88	37.6	874,522.61	Virginia
Washington	11,384,615.67	5,290,895.45	457.0	89,400.58	42,436.47	11.2	3,720,136.20	1,697,300.00	146.0	594,429.73	283,000.00	20.0	573,046.08	Washington
West Virginia	5,489,747.95	2,365,041.53	255.6	898,083.14	402,583.56	24.7	4,428,254.78	1,998,238.67	155.6	528,260.11	232,245.33	18.6	756,022.91	West Virginia
Wisconsin	18,753,903.15	7,441,033.57	1325.3	604,737.13	291,645.40	31.1	4,363,817.49	2,181,492.74	184.6	219,299.22	66,905.70	7.0	3,697,373.59	Wisconsin
Wyoming	6,127,625.61	3,078,098.70	687.6	412,650.54	225,012.57	57.1	4,652,110.25	2,951,679.84	320.6	77,217.69	49,921.00	11.7	382,638.89	Wyoming
HAWAII													365,625.00	HAWAII
TOTALS	$ 549,655,391.27	$237,852,399.82	32452.9	$30,010,949.88	$ 14,393,140.24	1849.8	$411,055,258.70	$ 185,506,713.94	18971.9	$ 41,987,688.68	$ 17,743,436.44	2031.4	$ 69,629,309.56	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 66,150,590.24 Federal aid $ 30,561,567.30 Miles 3,259.7

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

REPORTS

Report of the Director of the Bureau of Public Roads for 1918.
Report of the Chief of the Bureau of Public Roads for 1919.
Report of the Chief of the Bureau of Public Roads for 1920.
Report of the Chief of the Bureau of Public Roads for 1921.
*Report of the Chief of the Bureau of Public Roads for 1922. 5c.
*Report of the Chief of the Bureau of Public Roads for 1923. 5c.

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
*347. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370. The Results of Physical Tests of Road-Building Rock. 15c.
386. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
*389. Public Road Mileage and Revenues in the Central, Mountain, and Pacific States, 1914. 15c.
390. Public Road Mileage in the United States, 1914. A Summary.
*393. Economic Surveys of County Highway Improvement. 35c.
407. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463. Earth, Sand-Clay, and Gravel Roads. 15c.
*532. The Expansion and Contraction of Concrete ana Concrete Roads. 10c.
*537. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*555. Standard Forms for Specifications, Tests, Reports, and Methods of Sampling for Road Materials. 10c.
*583. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*586. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1916. 10c.
*660. Highway Cost Keeping. 10c.
670. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
*691. Typical Specifications for Bituminous Road Materials. 15c.
*704. Typical Specifications for Nonbituminous Road Materials. 5c.
*724. Drainage Methods and Foundations for County Roads. 20c.
No.*1077. Portland Cement Concrete Roads. 15c.
*1132. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
1216. Tentative Standard Methods of Sampling and Testing Highway Materials.

DEPARTMENT CIRCULAR

No. 94. TNT as a Blasting Explosive.

FARMERS' BULLETINS

No.*338. Macadam Roads. 5c.
*505. Benefits of Improved Roads. 5c.
*597. The Road Drag. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

No.*727. Design of Public Roads. 5c.
*739. Federal Aid to Highways, 1917. 5c.
*849. Roads. 5c.

OFFICE OF PUBLIC ROADS BULLETIN

No. *45. Data for Use in Designing Culverts and Short-span Bridges. (1913.) 15c.

OFFICE OF THE SECRETARY CIRCULARS

No. 49. Motor Vehicle Registrations and Revenues, 1914.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
74. State Highway Mileage and Expenditures for the Calendar Year 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D-2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D-3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 20, D-4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D-6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

*Department supply exhausted.

Have you ever Stood By the Seashore?

HAVE YOU EVER STOOD BY THE SEASHORE and watched the progress of an ocean liner way off near the horizon? If you have you will remember that it scarcely seemed to move at all, although you were sure it was moving, and moving swiftly too. You recall that when you turned your eyes away for a time and then looked back you found it in an entirely different quarter.

PROGRESS IN THE IMPROVEMENT OF AMERICA'S HIGHWAYS is very much like the motion of that ship. The immensity of the task ahead makes our best efforts, somehow, seem poor and weak indeed. Although Federal-aid roads are being completed at the rate of nearly 10,000 miles a year and an equal mileage, perhaps, is built on the Federal aid system each year without Government assistance, the ultimate improvement of the main system and the necessary auxiliary roads seems sometimes discouragingly remote. When you feel that way just look back ten years and you will find in the tremendous improvement of American roads during that period the encouragement you need. There is now a definite program of construction in all States. Every road improved in accordance with the program forges another link in the system. Section by section the gaps in the projected network are filled in. Mile by mile the sections of improved roads draw toward each other; and one fine day we shall come to the realization that we have been building better and more swiftly than we knew.

ASK ANY OF THE STATES that stand near the head of the highway procession and they will tell you that there is only one opinion about highway improvement when the gaps begin to close.