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APPARATUS USED IN MAKING EMBANKMENT STUDIES

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

In This Issue

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RESEARCH ON THE CONSTRUCTION OF EMBANKMENTS

A REPORT ON COOPERATIVE RESEARCH IN INDIANA AND OHIO

By HENRY AARON, Highway Engineer, Division of Tests, Public Roads Administration, W. T. SPENCER, Soils Engineer, State Highway Commission of Indiana, H. E. MARSHALL, Geologist, Ohio Department of Highways

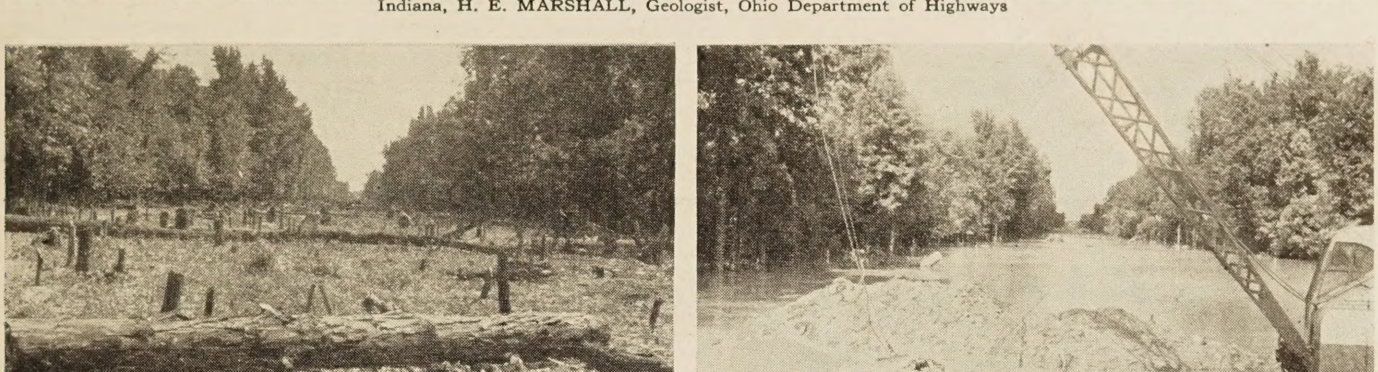


FIGURE 1.—EMBANKMENT SITE IN INDIANA (A) AFTER CLEARING AND (B) AFTER HEAVY RAINS WHICH OCCURRED DURING JULY 1938.

CONTROL OF EMBANKMENT CONSTRUCTION in accordance with laboratory compaction test data is a relatively recent development. The methods of performing both the laboratory tests and the field control tests vary with different organizations,[1] but in all cases the test results disclose the relation between moisture content and density of a soil which results from a given method of compaction. A standard procedure for making the test for compaction and density of soil was adopted by the American Association of State Highway Officials in 1938[2] and by the American Society for Testing Materials in 1942.[3]

This test discloses the moisture content of a soil at which the density obtained by a given method of compaction is higher than for any other moisture content. This is called the optimum moisture content. Greater stability of fills has been obtained wherever this test has been adopted as a basis for controlling compaction. However, many questions of importance arose in connection with its practical application.

Among these were the efficiency of different types of rollers in obtaining the desired results, the relation of the depth of layer to type of roller, the economics and limitations of moisture control, and the performance of the completed fill with respect to settlement and change in stability. In an effort to find answers to these questions, two cooperative projects, one by the State Highway Commission of Indiana and another by the Ohio Department of Highways, were undertaken with the Public Roads Administration cooperating.

[1] Proceedings of the Eighteenth Annual Meeting of the Highway Research Board. Compaction of Earth Embankments, Part II (1938).

[2] Standard Specifications for Highway Materials and Methods of Sampling and Testing, published by the American Association of State Highway Officials, 1220 National Press Building, Washington, D. C., 1938 and 1942 Editions, Method T 99–38.

[3] A. S. T. M. Standards Designation D 698–42 T, Tentative Method of Test for Moisture-Density Relations of Soils, published by the American Society for Testing Materials, 260 South Broad Street, Philadelphia, Pa. 1942.

As a result of this work, two independent reports were prepared entitled Fill Construction Experiment in Indiana and Fill Construction Experiment in Ohio. W. T. Spencer, soils engineer of the State Highway Commission, participated in the preparation of the Indiana report. H. E. Marshall, geologist of the Department of Highways, participated in the preparation of the Ohio report. Lt. Col. R. R. Litehiser, former chief engineer of tests, and K. B. Woods and Capt. C. H. Shepard, former assistant engineers of the Ohio Department of Highways, co-operated in planning and directing the Ohio project. For the Public Roads Administration Lt. Henry Aaron, former highway engineer, participated in the preparation of both reports.

The experiments were similar in a number of respects including principal objectives, soil variables, control

FIGURE 2.—LOCATION OF FILL CONSTRUCTION EXPERIMENT IN GIBSON COUNTY, INDIANA.

procedures, and types of compacting equipment used. The results of both experiments lead to similar conclusions. As a result much of the material included in one of the reports is duplicated in the other. For convenience of publication the reports have been combined into one presentation with such editorial changes as were needed to eliminate repetition.

PROJECTS DESCRIBED

The Indiana project, a relocated section of State Route 57 southwest of Buckskin, Gibson County, begins at a point about 600 feet southwest of Pigeon Creek and extends in a northeasterly direction for a distance of approximately 0.8 mile. The area is flat,

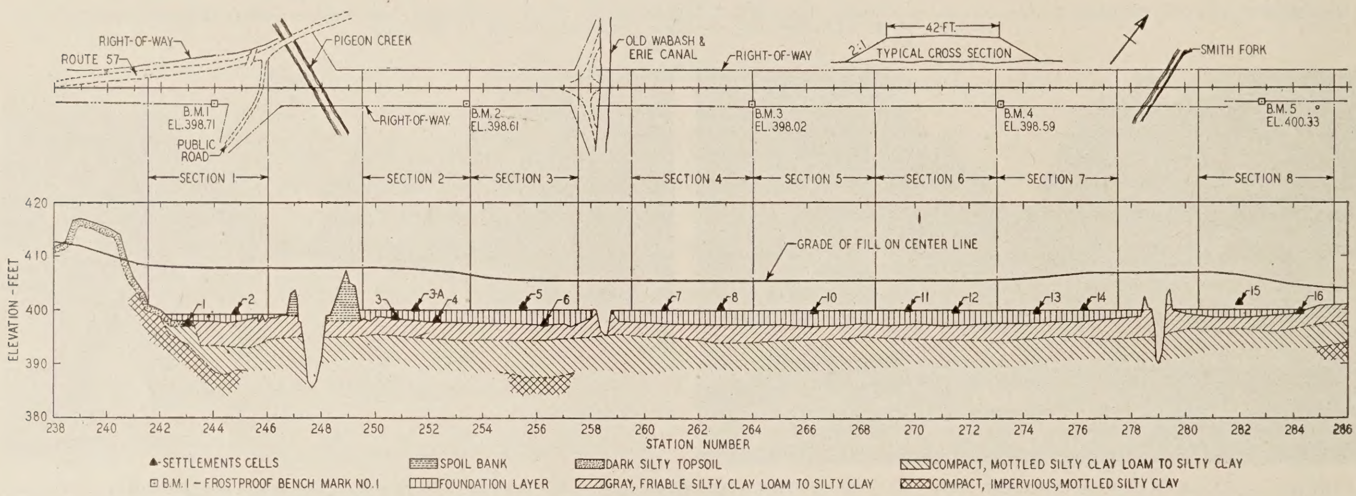


Figure 3.—Plan, Profiles, and Details of Experimental Section in Gibson County, Indiana.

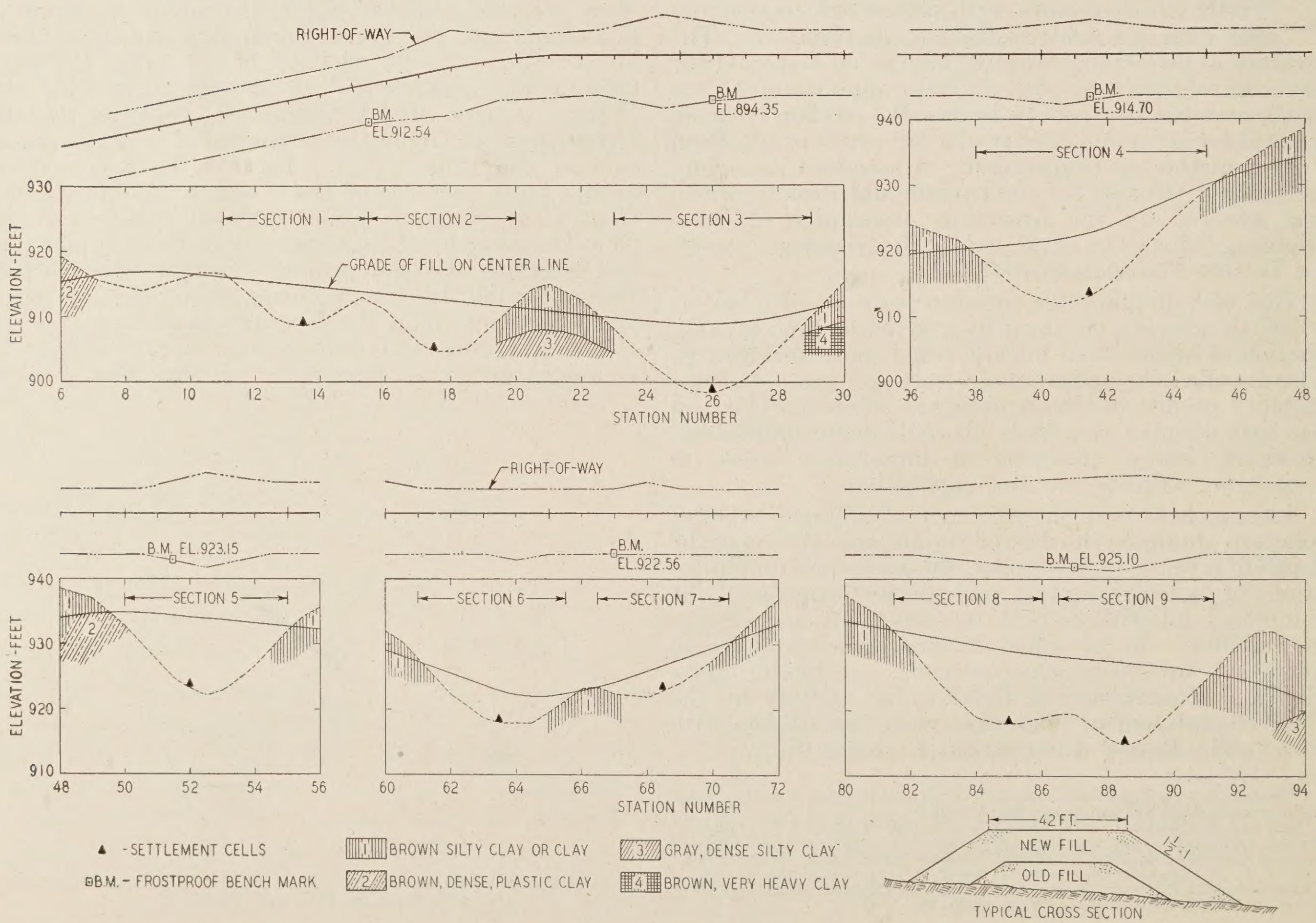


Figure 4.—Plan, Profiles, and Details of Experimental Sections in Delaware County, Ohio.

poorly drained, and subject to inundation. Figure 1 shows the site of the fill under dry and wet conditions. The location is shown in figure 2. The line crosses the low bottom lands bordering Pigeon Creek and Smith Fork.

The Ohio project, located on U. S. Route 23 in Delaware County, begins at a point about one-half mile north of the city limits of Delaware and extends in a northerly direction for a distance of approximately 1.5 miles. It follows the location of an old brick road. Changes in grades and widening of the roadway, however, required the placing of considerable fill material over the old road.

The topography is rolling and surface drainage is good. The area is drained by the Olentangy River which runs parallel to the project a short distance to the east.

In all, 17 sections were constructed; 8 in Indiana, varying in length from 400 to 500 feet, and 9 in Ohio, varying in length from 400 to 700 feet. The test sections have a top width of 42 feet and side slopes of 2 to 1 on the Indiana project and 1½ to 1 on the Ohio project (see figs. 3 and 4).

The embankments were constructed during the summer and fall of 1938. A portland cement concrete pavement was constructed on the Indiana project in 1939, and a brick pavement with a concrete base and curbs was placed on the Ohio project in 1938.

Table 1 gives details concerning the various sections of the Indiana project and table 2 gives similar information for the Ohio project.

Three different types of rollers were used in each State. The Indiana fill was constructed in layers ranging from 6 to 12 inches, and the Ohio fill in layers ranging from 6 to 9 inches in thickness.

On all sections of the Indiana project and sections 8 and 9 of the Ohio project, it was required that rollers compact each layer to a density equal to 95 percent of the maximum density of the soil as determined by the standard compaction test.[4] On sections 1 to 7 of the Ohio project, it was required that each layer of soil be compacted to maximum density.

On section 1 of the Indiana project and all sections of the Ohio project, it was required that the moisture content of the layer at the time of compaction should be within 1 of the optimum moisture content of the soil used in the layer.

TABLE 1.—*Details of experimental sections in Gibson County, Indiana*

Section No.	Station limits	Loose thickness of layer	Type of roller	Moisture control
		Inches		
1	241+50 to 246+00	6	Sheepsfoot	Optimum±1.
2	249+50 to 253+50	6	3-wheel	None.
3	253+50 to 257+50	6	Sheepsfoot	None.
4	259+50 to 264+00	9	3-wheel	None.
5	264+00 to 268+50	9	Pneumatic tire	None.
6	268+50 to 273+00	12	3-wheel	None.
7	273+00 to 277+50	12	Pneumatic tire	None.
8	280+50 to 285+50	6	Pneumatic tire	None.

TABLE 2.—*Details of experimental sections in Delaware County, Ohio*

Section No.	Station limits	Loose thickness of layer	Type of roller	Moisture control
		Inches		
1	11+00 to 15+50	6	Sheepsfoot	Optimum±1.
2	15+50 to 20+00	6	3-wheel	Do.
3	23+00 to 29+00	6	Pneumatic tire	Do.
4	38+00 to 45+00	6	Sheepsfoot	Do.
5	50+00 to 55+00	9	3-wheel	Do.
6	61+00 to 65+50	9	Sheepsfoot	Do.
7	66+50 to 70+50	9	Pneumatic tire	Do.
8	81+50 to 86+00	6	Sheepsfoot	Do.
9	86+50 to 91+00	6	3-wheel	Do.

THREE TYPES OF ROLLERS USED

The compacting equipment included sheepsfoot, three-wheel, and pneumatic tire rollers. The sheepsfoot rollers were of two general types, A and B, with the dimensions, weights, and working pressures as given in table 3. Common to both types were two drums, which rotated independently of one another. The drums were connected by a frame in such a manner as to permit the rollers to adapt themselves to uneven ground.

TABLE 3.—*Details of sheepsfoot rollers*

	Indiana, type A	Ohio, type A	Ohio and Indiana, type B
Number of drums	2	2	2
Diameter of drums (inches)	40	40	44
Length of drums (inches)	48	48	48
Distance between drums (inches)	10	10	8
Total width of tamped area (inches)	106	106	104
Number of feet per drum	88	88	112
Number of feet on ground	8	8	8
Length of feet (inches)	7	7	7
Tamping area of each foot (square inches)	5.5	5.5	5.25
Weight, drums empty (pounds)	5,100	6,250	7,350
Weight, loaded with water (pounds)	9,200	9,800	12,200
Ground pressure, drums empty (pounds per square inch)	116	142	175
Ground pressure, loaded with water (pounds per square inch)	209	223	290

Twenty-two rows of tamping feet studded each drum on type A rollers (fig. 5, *A*). Each row had four tamping feet so located that they were staggered with respect to the feet in the adjacent rows. Each tamping foot had an enlarged elliptical contact surface of 5.5 square inches.

Type B (fig. 5, *B*) had 28 rows of tamping feet. Each foot had a rectangular cross section with the longer dimension increasing with distance from the drum and the shorter dimension uniformly 1.5 inches throughout its entire length. The tamping feet were attached to a ⅜-inch removable circumferential band and could be replaced with feet of different sizes.

The three-wheel rollers (fig. 5, *C*), weighed 10 tons each. The one used in Indiana had rear wheels 23 inches wide, producing a ground pressure of 325 pounds per inch of width.

The one used in Ohio had rear wheels 20 inches wide producing a ground pressure of 350 pounds per inch of width.

The same type of pneumatic tire roller was used on both projects. It consisted of a loading platform (fig. 5, *D*) mounted on two axles equipped with nine smooth truck tires, four on the front axle and five on the rear axle. The tires on the front and rear axles were staggered with respect to each other so that they covered the entire strip, 60 inches wide, over which the roller traveled. The tires were inflated to a pressure of 35

[4] In Indiana the basis was the maximum wet weight per cubic foot. In Ohio the basis was maximum dry weight per cubic foot resulting from the compaction test. As long as the moisture content of the soil does not vary more than about 2 percent from the optimum, the results furnished on either basis should be essentially the same.

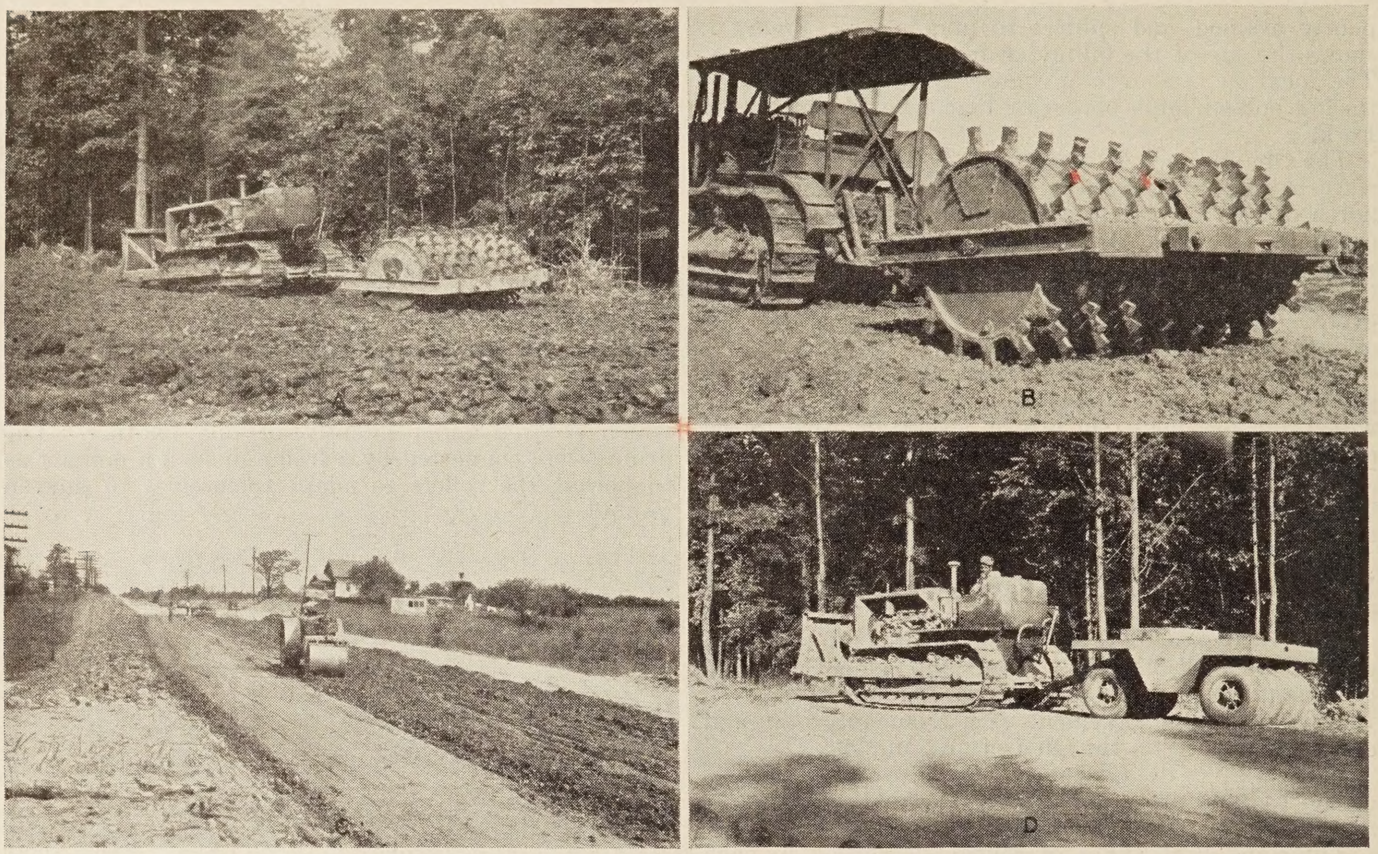


Figure 5.—Types of Rollers Used (A) Sheepsfoot Roller Type A in Indiana, (B) Sheepsfoot Roller Type B in Ohio, (C) Three-Wheel, 10-Ton Roller in Ohio, and (D) Pneumatic Tire Roller in Indiana.

pounds per square inch. The roller had a net weight of 2,680 pounds, but the platform was loaded so that under working conditions the roller gave a pressure of about 225 pounds per inch of width of tire surface in contact with the ground.

TESTS OF SOILS SHOW A CONSIDERABLE RANGE IN PROPERTIES

Prior to the construction of the embankment, subgrade surveys were made on both projects to determine the character of the soil in the fill areas and in deposits available for excavation.

Indiana Experiment.—Borings were made with soil augers and representative samples, obtained from the different soil layers, were tested in the laboratory of the State Highway Commission. The results of these tests are given in table 4.

The soil profile along the center line of the project is shown in figure 3. The top 3 to 4 feet of the soil in the embankment area varies from a gray friable silty clay loam to silty clay. This is underlaid by a more compact, mottled material of the same texture. At depths of from 9 to 12 feet, it grades into a compact, impervious silty clay. Free water was found in many places on top of the compact layers of soil. According to the test data shown in table 4, the soils have physical properties of the A–4 and A–7 groups.

The material used in the embankment was obtained from three sources in the upland areas adjacent to the fill locations; the cut from station 238 to station 241, borrow pit A about 1,000 feet right of station 246, and borrow pit B to the right of station 336. The borrow pit locations are shown on figure 2.

Soil profiles at the three sources were similar in character. The upper layer, about 12 inches thick, was a brownish friable silt loam or silty clay loam having physical properties of the A–4 group. The underlying soils to depths of more than 14 feet varied from a mottled gray and brown to a grayish silty clay or clay, which was friable when dry, crumbly when moist, plastic when wet, and had physical characteristics similar to those of the A–4 and A–7 groups.

Table 4.—*Results of tests* [1] *of samples of soil from embankment and borrow pit areas in Gibson County, Indiana*

Location	Depth	Mechanical analysis						Physical constants			
		Particles larger than 2 mm.	Sand, 2 to 0.05 mm.	Silt, 0.05 to 0.005 mm.	Clay, smaller than 0.005 mm.	Colloids, smaller than 0.001 mm.	Passing No. 40 sieve	Liquid limit	Plasticity index	Shrinkage limit	Field moisture equivalent
Embankment area:	*Feet*		*Pct.*	*Pct.*	*Pct.*	*Pct.*					
Station 244	0–4	0	12	65	23	12	90	27	6	17	27
	4–9.5	0	8	65	27	10	99	27	8	17	27
	9.5–12	0	6	68	26	10	95	38	19	21	25
Station 268	0–3	0	9	60	31	15	99	28	11	18	23
	3–12	0	5	64	31	12	99	30	12	18	21
Cut from Station 238 to Station 241: Station 239	0–1	0	6	74	20	6	100	33	9	26	32
	1–18.5	0	5	42	53	15	100	33	13	18	26
Borrow pit A:											
Hole No. 2 [2]	0–1	0	5	65	30	14	100	31	13	19	26
	1–12	0	3	57	40	16	100	46	25	16	31
Hole No. 3 [3]	1–7	0	6	66	28	10	100	35	17	17	20
	10–14.5	0	2	60	38	14	100	39	17	19	26
Borrow pit B:											
Station 335	0–1	0	24	50	26	15	100	26	6	19	25
	1–6	0	8	53	39	18	100	36	16	19	30

[1] Tests performed in laboratory of State Highway Commission of Indiana.
[2] Station 3+00 on center line of borrow pit.
[3] Station 6+00 on center line of borrow pit.

Ohio Experiment.—The existing road on the Ohio project was surfaced and, therefore, borings were made with soil augers along the shoulders and ditches. Samples obtained from the different soil layers were tested in the laboratory of the State Highway Department at Columbus.

Four types of soils were found on the experimental section as shown on the soil profile in figure 4. The mechanical analyses and physical properties of these soils are given in table 5. Type 1 is a brown silty clay or clay having physical properties of the A–4 group. Type 2, a brown dense plastic clay, has physical characteristics similar to those of the group A–7 soils.

Type 1 is underlain in some locations by type 3, which is a gray, dense silty clay having physical properties of the A–4 group.

Type 4, a very heavy clay of the A–7 group, was found in making the borings but was not used in any of the embankments owing to its occurrence below the depth of excavation.

TABLE 5.—*Average results of tests performed on 37 samples of typical soil materials found in the subgrade survey in Delaware County, Ohio*

Soil type	Mechanical analysis					Physical constants		Number samples tested
	Particles larger than 2 mm.	Particles smaller than 2 mm.						
		Coarse sand, 2 to 0.25 mm.	Fine sand, 0.25 to 0.05 mm.	Silt, 0.05 to 0.005 mm.	Clay, smaller than 0.005 mm.	Liquid limit	Plasticity index	
		Percent	Percent	Percent	Percent			
1	10	18	11	38	33	32	15	24
2	4	18	9	38	35	41	23	9
3	12	13	9	42	36	31	15	3
4	5	20	12	32	36	62	41	1

The soil in a borrow pit located about 1,000 feet to the right of station 18 was similar to the material found in the roadway cuts. Medium-sized boulders were found in all the soils.

MOISTURE-DENSITY CURVES PREPARED FOR USE IN CONTROLLING CONSTRUCTION OPERATIONS

In Indiana the maximum densities and optimum moisture contents of representative samples of the materials used in the embankment were determined by a compaction test similar to that described under method T 99–38 of the American Association of State Highway Officials. It differed from the standard method in that a separate portion of the sample was used for each change in moisture content.

Tests were performed in a field laboratory located at Buckskin. Results of these tests, table 6, were used to control the compaction of all sections and moisture content of the soil in section 1 during the construction of the fill. The results are shown graphically in figures 6 to 11.

During the construction of lifts 5 to 14 in section 2, lifts 5 to 14 in section 3, and the foundation layers in sections 4 and 5 of the Indiana project the power shovel was excavating material from a cut face extending from the surface to a depth of 8 feet in the part of borrow pit A east of the public road. The upper 4.5 feet at this location was represented by sample 2–A, and the lower 3.5 feet by sample 5–B. The average values of the maximum densities and optimum moisture contents of these two samples were used to control the compaction on the lifts when the material represented by these samples was placed.

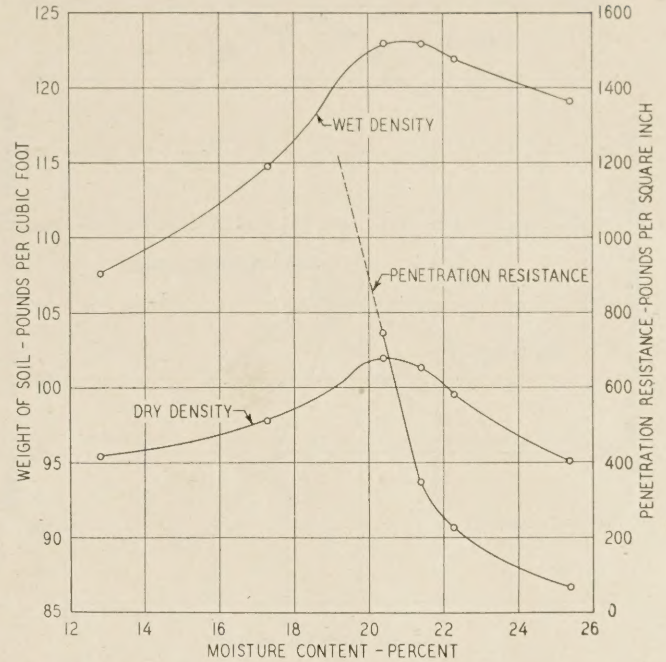


FIGURE 6.—DENSITY AND PENETRATION CURVES, SAMPLE 1–A IN INDIANA.

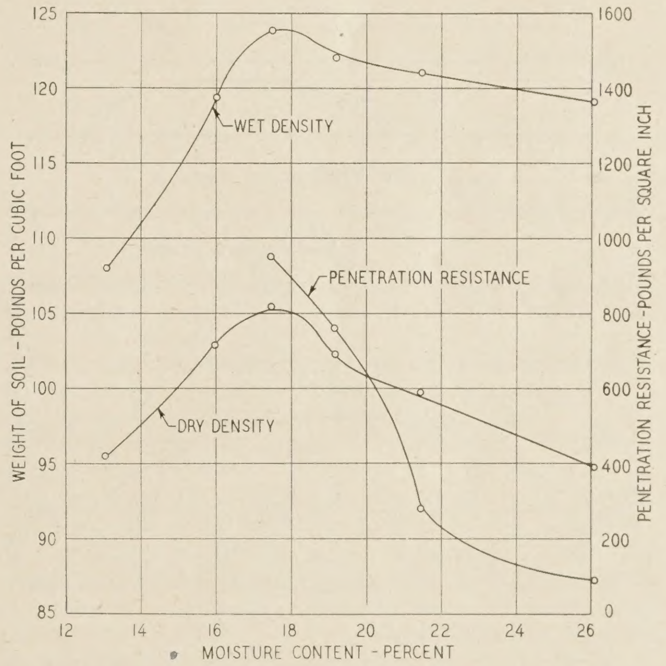


FIGURE 7.—DENSITY AND PENETRATION CURVES, SAMPLE 2–A IN INDIANA.

The average values were: Maximum wet density, 126.5 pounds per cubic foot; maximum dry density, 107.9 pounds per cubic foot; and optimum moisture content, 17.3 percent.

In Ohio the compaction tests were performed in the laboratory at Columbus and were in strict accordance with the standard method, A. A. S. H. O. designation T 99–38.

The results of the compaction tests performed on seven samples representative of the soils used in the

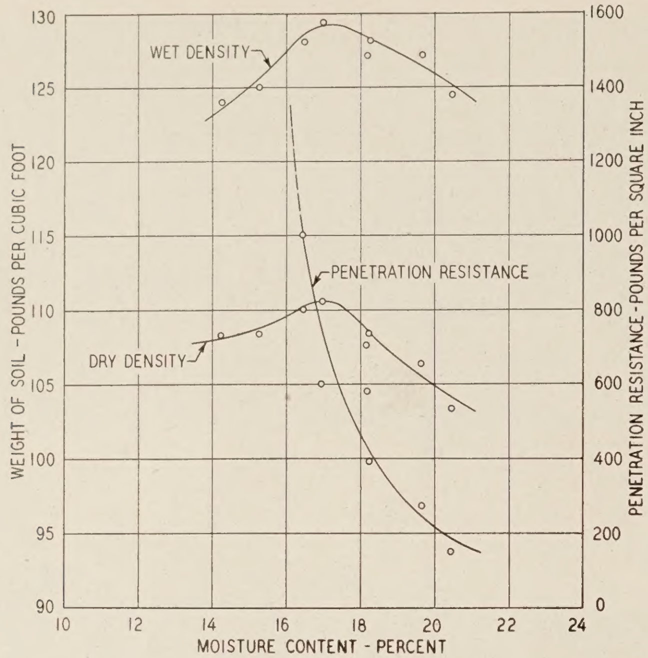


Figure 8.—Density and Penetration Curves, Sample 5–B in Indiana.

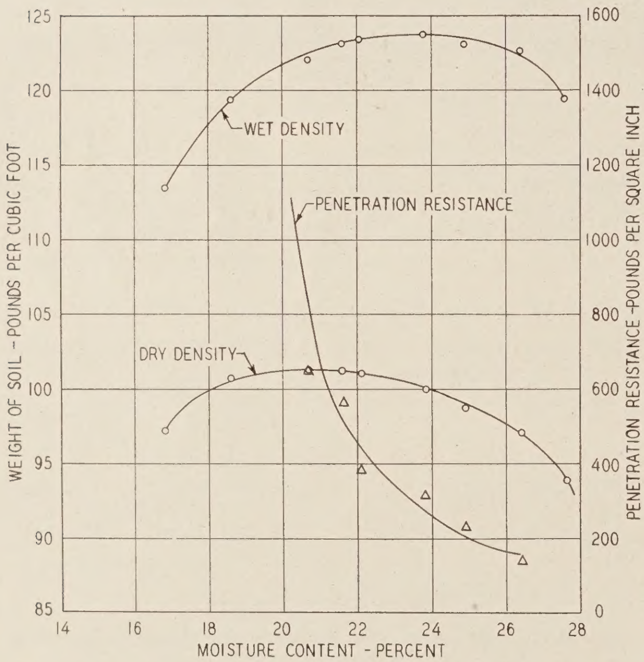


Figure 9.—Density and Penetration Curves, Sample 6 in Indiana.

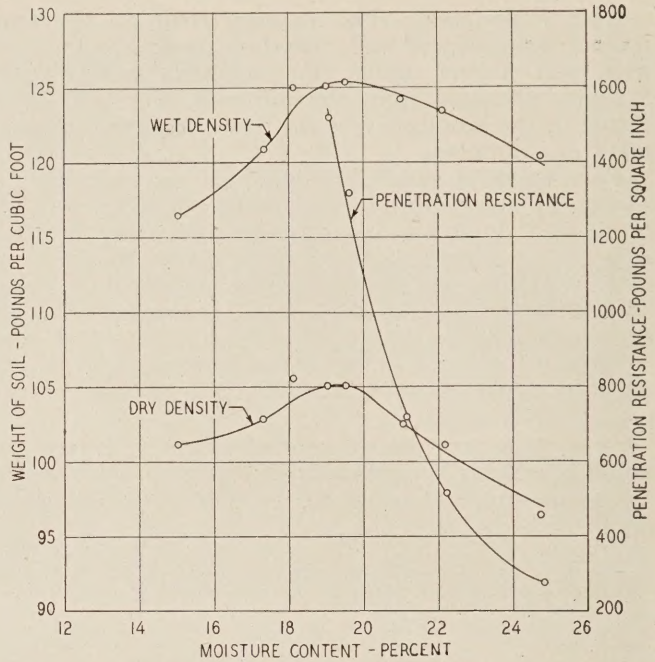


Figure 10.—Density and Penetration Curves, Sample 7–A in Indiana.

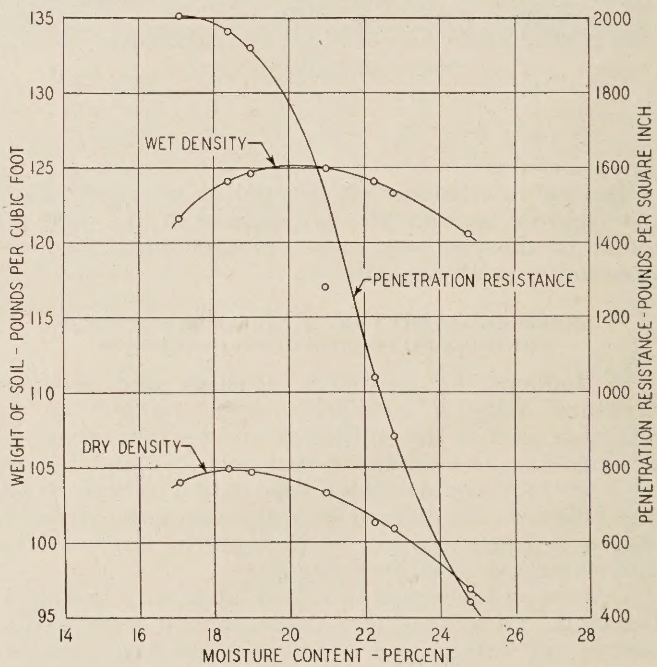


Figure 11.—Density and Penetration Curves, Sample 9–A in Indiana.

embankments in Ohio are shown graphically in figures 12 to 18 and together with their gradations, liquid limits, and plasticity indexes are given in table 7. These results are presented here in order to give a general idea of the character of the soils comprising the various fill sections.

A total of 81 samples were tested. The results were used to control the compaction and moisture content of the soil during the construction of the test sections.

CONSTRUCTION OF THE INDIANA PROJECT DESCRIBED

Foundation Layer.—Clearing of the embankment area in Indiana was completed during the first week of July. Construction of the fill was delayed until July 26 by heavy rains which flooded the low bottom lands with as much as 3 feet of water (fig. 1, *B*). After the water receded, the soil in the low area was so wet and soft that it would not support the construction equipment.

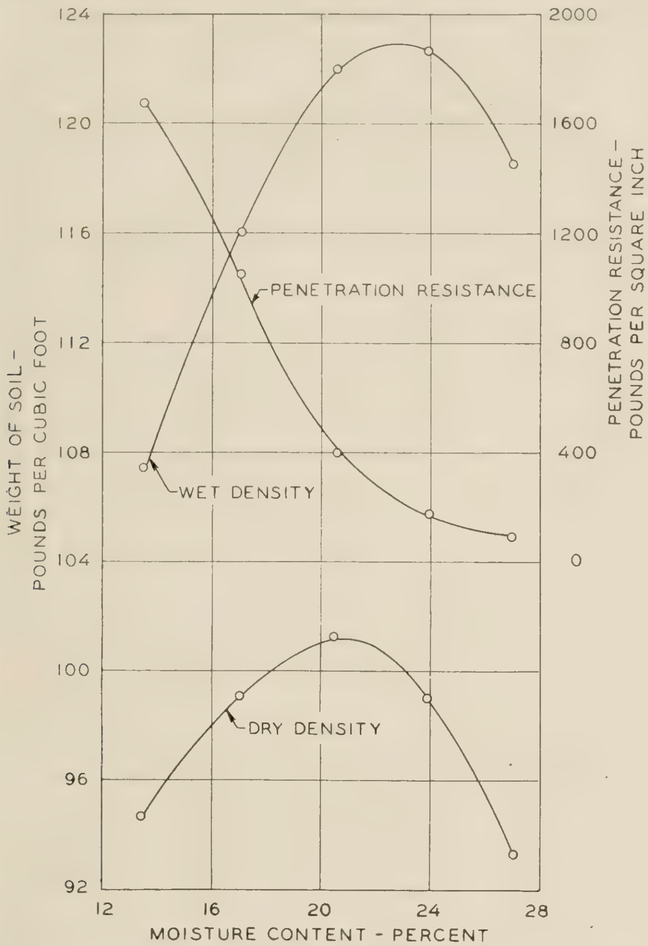


FIGURE 12.—DENSITY AND PENETRATION CURVES, SOIL No. 13840 IN OHIO.

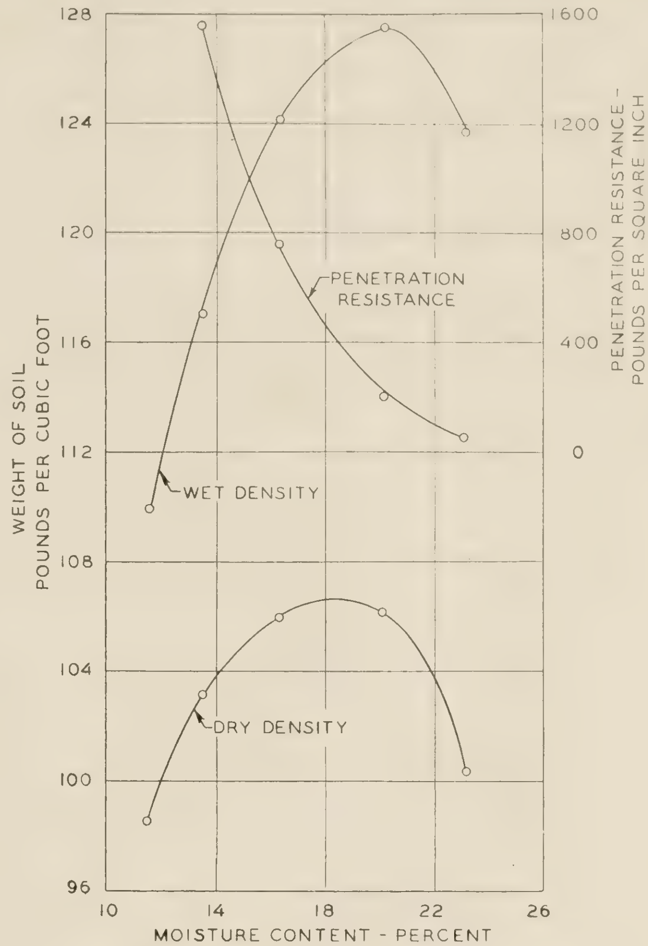


FIGURE 13.—DENSITY AND PENETRATION CURVES, SOIL No. 14035 IN OHIO.

The condition of the soil where a tractor broke through is illustrated in figure 19.

As the soil dried very slowly a foundation layer was placed as shown in figure 3 to facilitate the operation of the equipment used in the construction of the experimental sections. The foundation layer was constructed to elevation 400 by end dumping from trucks, spreading the material with a bulldozer, and compacting with a sheepsfoot roller. The layer was placed in lifts having loose thicknesses of about 10 inches maximum.

TABLE 6.—*Results of compaction tests on embankment materials in Gibson County, Indiana*

Sample No.	Location	Depth	Maximum wet density	Maximum dry density	Optimum moisture content at maximum dry density	Resistance to penetration at optimum moisture content
		Feet	*Pounds per cubic foot*	*Pounds per cubic foot*	*Percent*	*Pounds per square inch*
1-A	Cut between station 238 and station 241.	0-4.5	123.0	102.0	20.8	580
2-A	Borrow pit A, east of public road.	0-4.5	123.9	105.3	17.6	940
5-B	Borrow pit A, east of public road.	4.5-8.0	129.2	110.5	17.0	730
6	Borrow pit A, east of public road.	8.0-15.0	123.7	101.2	20.6	520
7-A	Borrow pit A, west of public road.	0-5.0	125.2	105.0	19.6	1,280
9-A	Borrow pit B	0-5.5	125.0	104.8	18.4	1,960

Considerable difficulty was encountered in operating the equipment on the first lift of the foundation layer due to the surface irregularities produced by the subsidence and displacement of the soft undersoil. The sheepsfoot roller was very useful at this stage of the work since it would adapt itself to the uneven surface and could be backed out on soft soil which would not support the tractor. However, it was necessary to vary the amount of water in the drums to adjust the weight of the roller to fit the soft undersoil conditions.

Most of the foundation layer on sections 2 and 3 was placed with trucks having a capacity of 8 cubic yards. These were replaced by trucks holding 1.5 cubic yards which were found to be more satisfactory for the conditions existing on this job.

The foundation layer and the equipment used in its construction are shown in figure 20, *A*.

Construction Schedule.—Construction of sections 1 and 8 was commenced on September 24. The embankment area had dried out sufficiently by this time so that no foundation layer was required. However, the first two lifts on these sections were quite irregular and should be considered as a foundation layer.

The only road available for hauling material from borrow pit A to the embankment passed through section 1, making it necessary to delay the construction of this section until sections 2 to 7 were completed.

Construction of the fill proper started on sections 2 and 3, located between Pigeon Creek and the Old Wa-

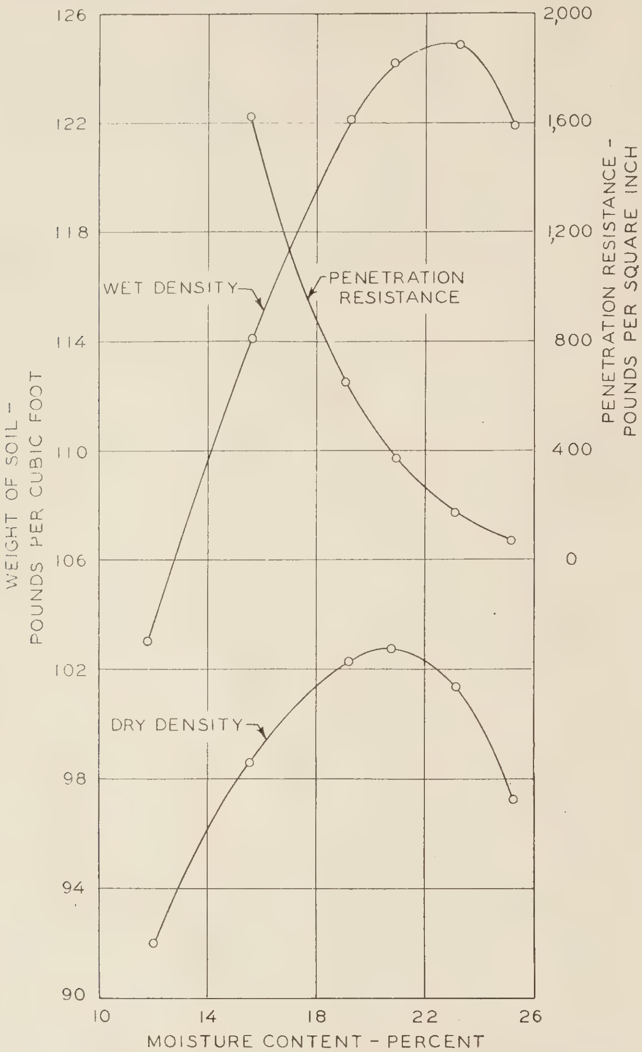


Figure 14.—Density and Penetration Curves, Soil No. 14056 in Ohio.

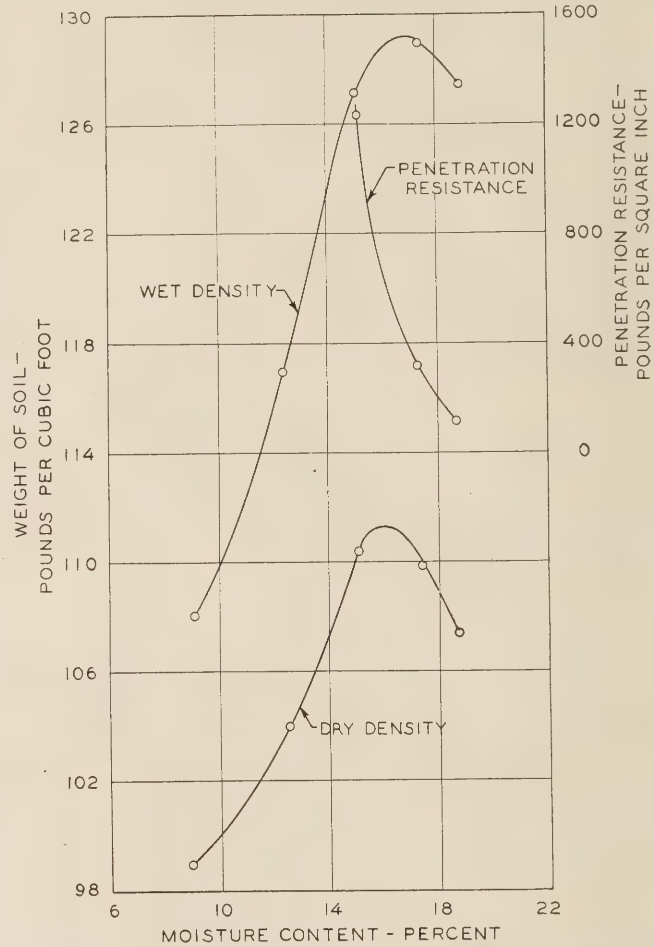


Figure 15.—Density and Penetration Curves, Soil No. 14058 in Ohio.

bash River and Erie Canal. Corresponding lifts on each section were constructed concurrently, the material for section 3 being hauled over section 2. Hauling of material and moving of equipment was not permitted on section 3 (compacted with sheepsfoot roller) during or subsequent to construction, except as necessary to construct the section.

The access road to sections 4, 5, 6, and 7 entered the fill area near the beginning of section 4. For this reason section 7 was constructed first, followed in order by sections 6, 5, and 4. Dates of construction of the different sections are given in table 8.

Table 8.—*Dates experimental sections were constructed in Gibson County, Indiana*

Section No.	Construction period in 1938			
	Foundation layer		Fill	
	Started	Completed	Started	Completed
1			Sept. 26	Oct. 12.
2	July 26	July 26	Aug. 10	Sept. 12.
3	July 28	July 28	Aug. 10	Sept. 17.
4	Aug. 13	Aug. 14	Sept. 9	Sept. 17.
5	Aug. 19	Aug. 21	Sept. 6	Sept. 9.
6	Aug. 21	Aug. 23	Sept. 2	Sept. 6.
7	Aug. 23	Aug. 24	Aug. 25	Sept. 2.
8			Sept. 24	Oct. 6.

Table 7.—*Results of tests performed on seven samples representative of soils used in embankments in Delaware County, Ohio*

Sample No.	Gradation					Liquid limit	Plasticity index	Maximum wet density	Maximum dry density	Optimum moisture content	Penetration resistance at optimum moisture content
	Passing 1-inch sieve	Passing No. 4 sieve	Passing No. 10 sieve	Passing No. 60 sieve	Passing No. 200 sieve						
	Percent	*Percent*	*Percent*	*Percent*	*Percent*			*Lb. per cu. ft.*	*Lb. per cu. ft.*	*Percent*	*Lb. per sq. in.*
13840		100	97	84	76	40	18	123.0	101.2	20.5	410
14035	100	99	97	89	81	39	17	127.6	106.6	18.6	400
14056	100	89	74	38	33	45	19	124.9	102.8	20.6	440
14058	100	95	75	18	16	37	16	129.2	111.3	15.8	800
14140		100	96	80	74	36	15	126.7	105.6	19.2	700
14147	100	94	92	75	65	32	14	129.6	110.8	16.0	610
14148	100	98	93	79	68	30	12	133.3	114.4	16.0	370

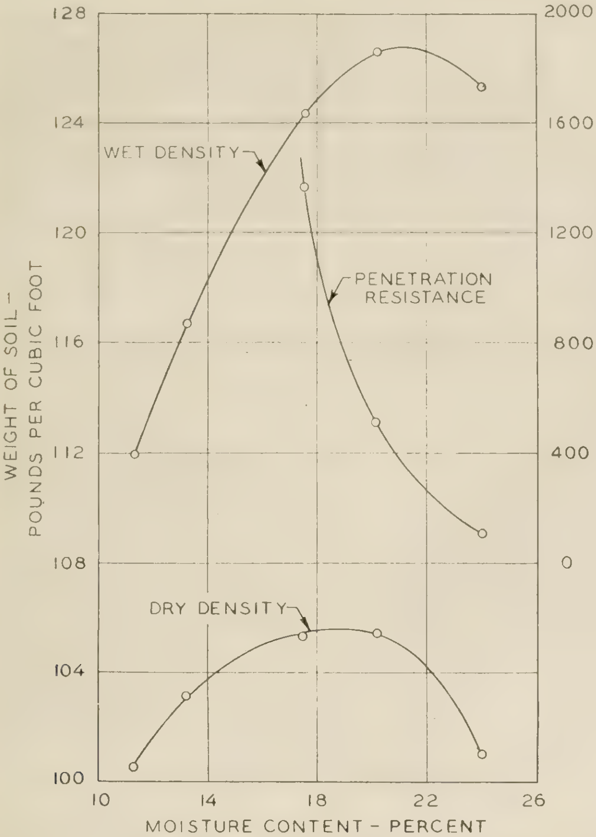


FIGURE 16.—DENSITY AND PENETRATION CURVES, SOIL No. 14140 IN OHIO.

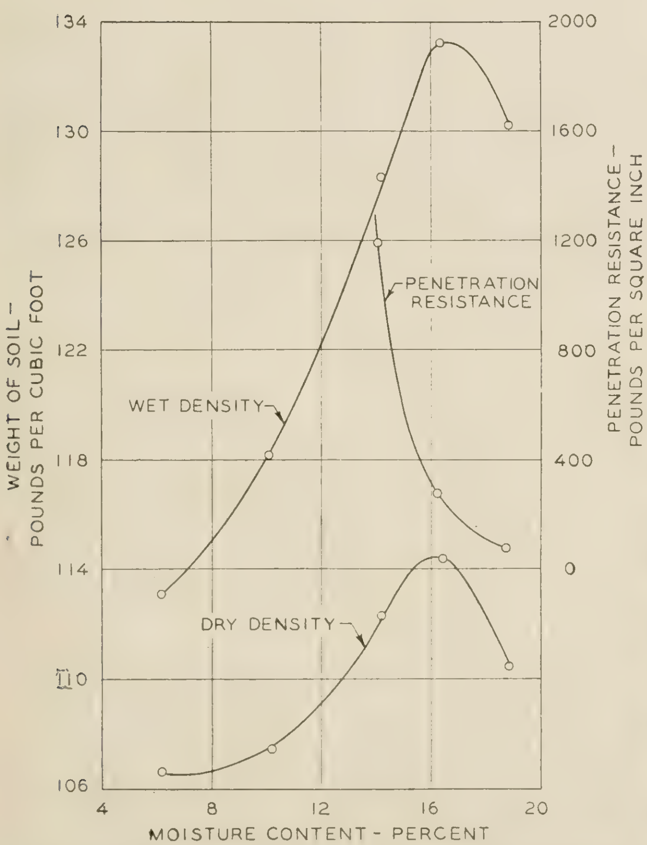


FIGURE 17.—DENSITY AND PENETRATION CURVES, SOIL No. 14148 IN OHIO.

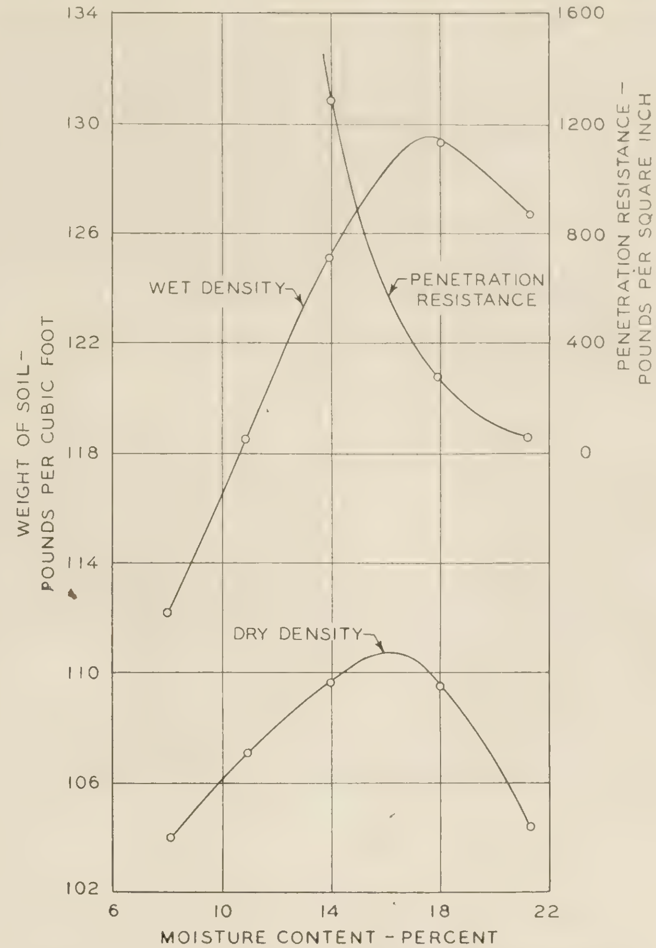


FIGURE 18.—DENSITY AND PENETRATION CURVES, SOIL No. 14147 IN OHIO.

Construction Procedure.—Fill material for section 1 was placed in lifts 6 inches thick, loose measurement, by scrapers (fig. 20, *B*) having a capacity of about 8 cubic yards. The first two lifts were not uniform in thickness and compaction owing to the unevenness of the ground surface and the tendency of the fill material to be dis-

FIGURE 19.—SOFT CONDITION OF SOIL IN FILL AREA AFTER THE FLOOD WATERS RECEDED. A TRACTOR WAS MIRED AT THIS LOCATION IN INDIANA.

FIGURE 20.—CONSTRUCTION OF GRADE IN INDIANA SHOWING: (A) FOUNDATION LAYER OVER SOFT SOIL; (B) LARGE SCRAPER PLACING FILL MATERIAL ON SECTION 1; (C) COMPACTING WITH THREE-WHEEL 10-TON ROLLER; AND (D) COMPACTING WITH PNEUMATIC TIRE ROLLER.

placed as a result of the spongy character of the undersoil.

Following the placing of the first lift over the full width of the embankment area, each succeeding lift was constructed in two parts, each part being one-half the width of the fill section. One-half of the lift was compacted while material was placed on the adjoining half. This method was adopted in order to avoid compacting the spread material by any means other than by the roller specified for the section.

Prior to compaction, the soil on each half lift was tested to determine if its moisture content was within the tolerance of 1 of the optimum, as required by the specifications for this section. If the soil contained more than the specified amount, the soil was permitted to dry before compaction. A disk harrow was used to facilitate the drying. The disking served also to pulverize the clods of soil.

If the soil did not have sufficient moisture to comply with the specifications, water was added with a pressure distributor. Each application of water, approximately 0.5 gallon per square yard, was mixed with the soil by a disk harrow. Water was added and the disking continued until the required amount of moisture was uniformly distributed through the soil. The lift was then compacted with the sheepsfoot roller until it attained a density equal to at least 95 percent of the maximum as determined by the compaction test.

Fill material for sections 2 to 8 was excavated from the borrow pits by power shovels and hauled to the embankment in 1.5-ton trucks, spread with a bulldozer and road machine into layers of specified thickness, disked to pulverize the clods and loosen soil packed by equipment, and compacted to the required density by rolling. Moisture control was not used on these sections. When the soil was placed and compacted the soil had a moisture content as it came from the borrow pit. At times when the soil contained too much moisture to permit the operation of the compaction equipment, it was allowed to dry until the rollers could perform satisfactorily.

Section 3 like section 1 was compacted with a sheepsfoot roller; sections 2, 4, and 6 with the three-wheel roller (fig. 20, *C*); and sections 5, 7, and 8 with a pneumatic tire roller (fig. 20, *D*).

METHODS USED IN OHIO VERY SIMILAR TO THOSE IN INDIANA

Since the Ohio project was located over an old road it was necessary to remove the old road surfacing, the sod, and other objectionable material from the shoulders and side slopes before placing new fill. Figure 21, *A*, shows the old fill at the location of test section 4 cleared and ready for widening and raising of the grade. The widening was accomplished by placing material on each side of the old fill in horizontal layers of specified loose thickness. While the soil was being deposited on one side, the material on the other side was compacted.

After the widening had been completed, the material for each lift was placed for the full width of the roadway. Hauling was not permitted over any lift until it had been compacted to the required density.

Fill material for sections 1, 2, 3, 4, 8, and 9 was placed in lifts 6 inches thick, loose measurement, and in 9-inch lifts on sections 5, 6, and 7. Most of the fill material was excavated by power shovels hauled to the embankment in dump trucks (fig. 21, *B*) and spread with a bulldozer (fig. 21, *C*) into layers of specified

FIGURE 21.—TEST SECTION 4 IN OHIO SHOWING: (A) OLD FILL CLEARED AND READY FOR WIDENING AND RAISING OF THE GRADE; (B) HAULING WITH DUMP TRUCKS; (C) SPREADING WITH BULLDOZER; AND (D) HAULING AND SPREADING WITH SCRAPER.

FIGURE 22.—DISKING AND APPLYING WATER WITH A PRESSURE DISTRIBUTOR IN OHIO.

thickness. The excavation, hauling, and spreading of the soil used in some of the lifts on sections 1, 8, and 9 was done by 12-yard scrapers (fig. 21, *D*). The soil layer was then disked (fig. 22) to pulverize the clods.

Required moisture content was obtained in the same manner as described for section 1 of the Indiana project after which the lift was compacted to the required density. The pressure distributor used in Ohio is shown in figure 22.

DENSITY OF LIFTS DETERMINED

On both projects the number of trips that the particular roller was required to make in order to obtain the necessary compaction was governed by the specified density of the compacted lift.

Procedure in Indiana.—The density of the compacted lift in the Indiana experiment was determined in the following manner: The location where the density test was to be made was shaped with a shovel to give a level surface. A soil-collecting box, 18 inches square, having a 4.5-inch hole in the center was set in place on the leveled surface and a hole was bored to the bottom of the compacted layer with a 4-inch post-hole auger which passed through the hole in the soil-collecting box. This operation is illustrated in figure 23, *A*. The soil-collecting box served to collect the soil which spilled through the sides and over the top of the auger while making the boring. This material, together with the soil removed by the auger, was placed in a bucket and weighed. The weight of the bucket, subtracted from the weight of bucket and soil, gave the weight of the soil removed from the hole. For determination of the moisture content a representative sample of about 80 gm. was placed in a container and sealed. The moisture content was later determined in the field laboratory at Buckskin.

Standard Ottawa sand of known weight per cubic foot, loose measurement, was poured into the hole from a canvas sack. The combined weight of the sack and sand was determined before placing any sand in the hole. When the hole was filled to the top of the lift

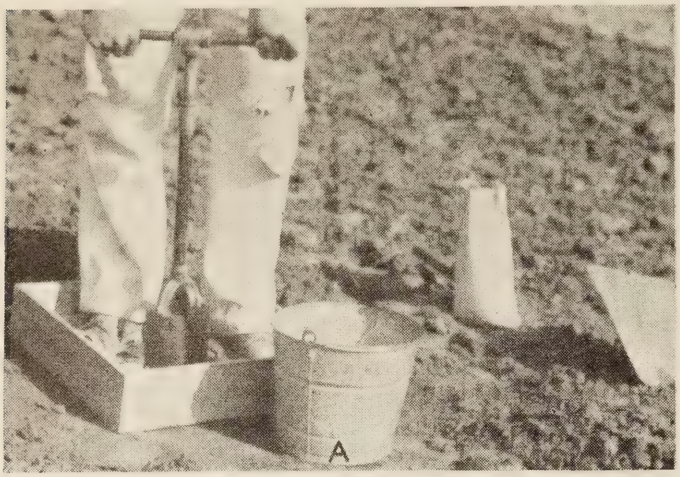


FIGURE 23.—REMOVING SOIL WITH POST-HOLE AUGER TO DETERMINE DENSITY OF COMPACTED LIFT AND DETERMINING THE VOLUME BY FILLING THE HOLE WITH SAND OF KNOWN WEIGHT PER UNIT VOLUME (INDIANA PROJECT).

(fig. 23, *B*), the sack and the sand remaining was weighed. The difference between the two weights gave the weight of sand in the hole.

All weighing was done with a balance having a capacity of about 30 pounds and accurate to 0.5 ounce.

The weight of sand required to fill the hole was divided by its weight per cubic foot to determine the volume of the hole. The weight of the soil removed from the hole was then divided by the volume of the hole to obtain the wet density of the compacted lift in pounds per cubic foot.

The dry density of the compacted lift in pounds per cubic foot was calculated by the formula:

$$\text{Dry density} = \frac{\text{wet density} \times 100}{100 + \text{percentage of moisture in soil}}$$

Determination of Dry-Sand Volume.—The weight per cubic foot of the loose dry Ottawa sand used in the density tests was calculated in the following manner: A container of known volume having approximately the same dimensions as the auger hole was filled with sand poured from a canvas bag. The mouth of the bag was constricted in such a manner that the sand was discharged in a loose stream having a diameter of about 1.5 inches. The bag was raised as the container filled in order to provide a constant height of fall of 12 inches.

The difference in the weights of the container, empty and filled with sand, equals the weight of sand in the container. This weight of sand divided by the volume of the container gives the density of the loose dry sand in pounds per cubic foot.

During the operation care was exercised to avoid vibration of the container which might cause the sand to settle.

The procedure used in pouring the sand into the container for determination of its weight per cubic foot was followed in placing the sand in the auger holes.

Procedure in Ohio.—With several exceptions the method used in Ohio was similar to that used in Indiana. Only the exceptions need to be noted.

Instead of the collecting box 18 inches square, a pan 14 inches square with a 4.5-inch circular opening in the center was used. After the sample had been collected, a metal cover was fitted over the opening in the pan in order to prevent the loss of soil, the pan and soil were weighed, and the weight of the soil was calculated by subtracting the known weight of the pan from the combined weight of the pan and soil.

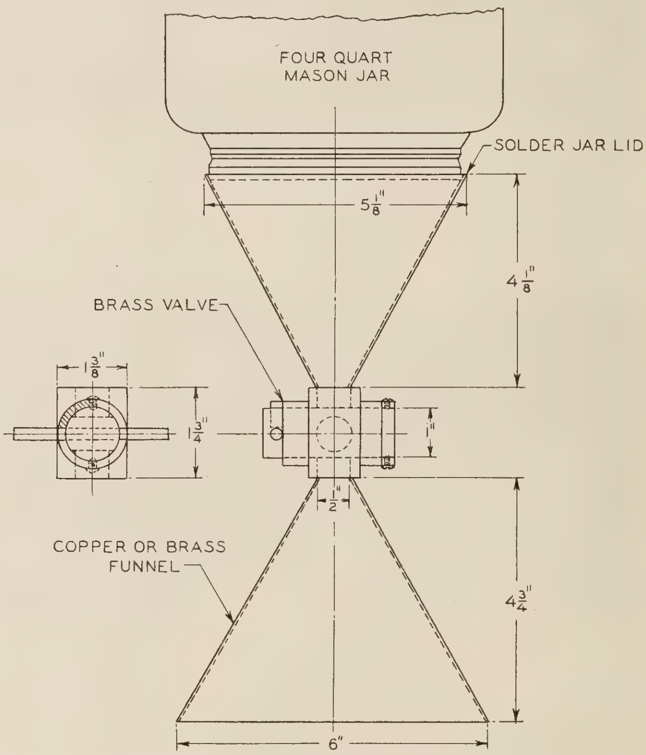


FIGURE 24.—DETAILS OF JAR-CONE DEVICE USED IN MAKING DENSITY TEST IN DELAWARE COUNTY, OHIO.

Standard Ottawa sand of known weight per cubic foot, loose measurement, was poured into the hole from a glass jar fitted with a detachable apparatus consisting of a metal cone, a shut-off valve, and a funnel as shown in figure 24. The combined weight of this jar-cone apparatus and the sand contained therein was determined before placing sand in the hole. The jar-cone apparatus was placed in an inverted position over the hole (see cover illustration) and the valve opened. The sand flowed until the hole and funnel were completely filled. The valve was closed and jar-cone apparatus with the sand remaining in it weighed.

The difference between the two weights gave the weight of the sand in the hole and funnel. By subtracting the constant weight of the sand in the funnel, the weight of the sand required to fill the hole was found. All weighing was done with a balance having a capacity of about 35 pounds and accurate to 0.01 pound.

Calibration of Sand.—Before conducting any tests on the embankment, the dried sand was calibrated for weight per cubic foot by placing the jar-cone apparatus, with funnel up and valve closed, on a vibrationless support. The funnel was then filled and the valve opened. As the sand flowed into the jar more sand was poured into the upper funnel, maintaining the sand in the funnel level with the top at all times. When the jar and cone were full, the valve was closed and the excess sand poured from the funnel.

The weight of sand contained in the jar and cone was determined by obtaining the difference in weight of the full and empty apparatus. This weight divided by the volume of the apparatus, previously determined with water, gave the weight per cubic foot of the sand under standard flow conditions.

To calibrate the constant weight of dry sand contained in the 6-inch funnel, the weighed apparatus was inverted on a smooth surface and sand allowed to flow into the funnel until full. The difference between the weight of the apparatus with the remaining sand and the weight of the apparatus full of sand gave the weight of the sand in the funnel.

Correction for Coarse Material.—The standard compaction test used in the laboratory showed the maximum density of that portion of the soil which passed the No. 4 sieve. Inclusion of material larger than the No. 4 sieve would have the effect of increasing this density owing to the higher specific gravity of the stone, approximately 2.65, as compared with the bulk specific gravity of the compacted soil, about 1.76 for soil having an average dry density of 110 pounds per cubic foot. Lifts, compacted to specified densities based on the laboratory tests, and which contain material retained on the No. 4 sieve, will have densities higher than those indicated by the standard test.

In this case the specified densities applied to the material passing the No. 4 sieve and consequently, a correction was made for the effect of the material retained on the No. 4 sieve. This correction was made in the following manner: Instead of using the total weight of soil extracted from the auger hole in computing the density of a lift, a value equal to this weight minus one-third of the weight of the material retained on the No. 4 sieve was used (ratio of specific gravities given above is 3 to 2).

All field tests in Ohio were performed in a portable laboratory (fig. 25) stationed on the job.

FIGURE 25.—PORTABLE FIELD LABORATORY IN OHIO.

CONTROL OF THE MOISTURE CONTENT ON INDIANA PROJECT

On sections 2 to 8 where moisture control was not specified records of the moisture content of each lift were obtained as a part of the density determination previously described.

Two samples were taken from each half lift on section 1, and the moisture contents of the samples were determined by drying them in pans over an open flame, care being taken to constantly stir the soil to prevent burning. The sample (about 80 gm.) was dried in approximately 10 minutes. The weighings were made on a triple beam balance having a capacity of 111 gm. and sensitive to 0.01 gm.

Additional samples were taken from locations where visual inspection indicated important differences in moisture content.

When the soil of section 1 needed either wetting or drying to give it the specified moisture content, moisture determinations were made after each application of water or during the disking to expedite evaporation. It was seldom necessary to dry the soil placed in section 1. Of the 20 lifts comprising the fill 17 required the addition of water in amounts ranging from 0.5 to 3 gallons per square yard.

COMPACTION CONTROL AND DENSITY DETERMINATIONS ON OHIO PROJECT

Use of Typical Curves.—The soil delivered to the fills on the Ohio project was obtained from the cuts and borrow pits in such a manner that it was a mixture of the materials represented by the samples for which moisture-density curves were determined. It was decided to adopt the method previously used with satisfactory results in Ohio [5] when such variations were found. Selection was made from the group of curves available as a result of the tests made on samples from different parts of the cuts and borrow pits, the one most representative of the material being placed. Selection was made in the following manner: The soil to be used in the fill was compacted in the compaction cylinder according to the standard procedure and its wet weight per cubic foot was determined. Penetration by the Proctor plasticity needle then disclosed its penetration resistance

[5] SOIL MECHANICS APPLIED TO HIGHWAY ENGINEERING IN OHIO. Engineering Experiment Station Bulletin 99, Ohio State University, July 1938.

in pounds per square inch. The observed wet density corresponds to a range of moisture contents on the group of moisture-density curves and the observed penetration resistance corresponds to a range of moisture contents on the group of moisture-penetration-resistance curves. Using the observed penetration and density the moisture-density and the moisture-penetration curves for each sample were examined and the corresponding moisture determined on each curve. The soil for which these two values were most nearly equal was considered to be representative of the soil being placed and its moisture-density curve was used in further determinations.

Moisture Control.—The moisture determinations were made as the soil was delivered on the fill and after the various applications of water.

The typical moisture-density and moisture-penetration-resistance curves having been selected for a soil to be used in a particular lift, the density determinations were made in the usual manner. For checking the moisture content of the compacted material the Proctor plasticity needle, which disclosed the penetration resistance of the compacted lift, was used (see cover illustration). The moisture content of the soil in the fill was taken as the moisture content corresponding to the observed density on the typical moisture-density curve. This value was checked by using the observed penetration to determine moisture on the moisture-penetration curve.

Compaction Control.—As in the case of moisture control, the compaction was controlled in accordance with the moisture-density curves selected as described above. Table 9 shows that as many as 22 different curves were used on a section. The average maximum density determined from all curves used in each section, together with the range in maximum densities and the variations in compaction obtained in the fills are given in table 10. The table shows that compaction in excess of 100 percent of maximum was obtained on parts of every section. This indicates that actual field compaction, at places, exceeded that of the standard compaction test.

DETAILED RECORD MADE OF CONSTRUCTION OPERATIONS

In view of the experimental nature of the construction, an attempt was made to control the thickness of the fill layers as accurately as possible. The loose thickness of each lift was measured by taking elevations at the same spots before and after spreading soil. The difference in elevation was taken as the loose thickness of lift.

In Indiana the measurements were made at locations 50 feet apart along lines through the quarter points of the lift. If the measured thickness differed from the specified thickness by more than 0.1 foot for a distance of 50 feet, material was added or bladed off as required. An engineer's level was used in taking elevations. All measurements in the Ohio experiment were made in a particular area of the fill section about 15 feet to the right of the center line at the station where the fill was a maximum in height and where the settlement cells, described later in this report, were placed.

In order to obtain data relative to the uniformity of the embankment and the performance of the different types of compacting equipment, the following records were made during the construction of each section on both projects:

TABLE 9.—*Variation between moisture content of compacted fill and optimum in Delaware County, Ohio*

Section No.	Optimum moisture content [1]			Variation from optimum			Lifts below optimum [3]	Moisture-density curves
	Average [2]	Minimum	Maximum	Average	Minimum	Maximum		
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Number*
1	18.8	14.2	22.0	−2.0	−0.8	−4.9	94	12
2	18.2	15.5	22.0	−2.2	−.3	−4.2	96	16
3	17.6	14.4	20.6	−2.0	0	−4.8	97	17
4	18.4	15.2	22.0	−1.0	+.1	−4.0	74	18
5	20.0	16.0	23.0	−2.0	−.9	−4.4	96	11
6	16.6	15.3	18.2	−.8	−.4	−3.5	83	10
7	18.7	16.0	23.0	−.5	−.3	−1.7	80	5
8	17.7	12.7	23.0	−1.8	−.1	−5.5	95	22
9	17.6	14.7	20.2	−2.0	0	−5.2	94	11

[1] Determined in laboratory by means of standard compaction test.
[2] Average from curves used in test section.
[3] Percentage of lifts compacted at moisture contents below optimum.

TABLE 10.—*Variation in densities in Delaware County, Ohio*

Section No.	Maximum dry densities [1]			Compaction [3]				Moisture-density curves
	Average [2]	Minimum	Maximum	Average	Minimum	Maximum	Specified	
	Lb. per cu. ft.	*Lb. per cu. ft.*	*Lb. per cu. ft.*	*Percent*	*Percent*	*Percent*	*Percent*	*Number*
1	106.1	102.4	111.3	99.5	91.0	108.5	100	12
2	108.3	102.4	113.7	105.2	93.5	113.7	100	16
3	109.4	102.7	114.5	102.9	93.9	111.9	100	17
4	107.4	102.4	113.7	101.5	94.4	106.9	100	18
5	105.5	102.3	111.7	104.2	96.5	111.4	100	11
6	110.3	107.0	114.4	97.1	92.5	100.7	100	10
7	107.2	102.3	111.7	101.4	100.0	102.5	100	5
8	109.4	99.2	118.7	97.3	83.0	107.5	95	22
9	109.3	103.1	113.5	101.1	91.6	108.6	95	11

[1] Determined in laboratory by means of standard compaction test.
[2] Average for curves used in test section.
[3] Percentage of maximum dry density obtained in the compacted fill.

1. Type of roller used for compaction.
2. Maximum density and optimum moisture content of the material comprising each lift.
3. Number of lifts.
4. Loose thickness of each lift.
5. Moisture content of soil in each lift.
6. Amount of water added to fill material.
7. Number of trips with roller to obtain specified density.
8. Thickness of lift after compaction.
9. Density of compacted lift.
10. Moisture content of compacted lift.

The following time studies were made to obtain information on the efficiency of the different rollers as indicated by the production in volume of material compacted per hour of rolling time:

1. Total available working time for section.
2. Actual working time of rollers.
3. Delays due to weather conditions, equipment repairs, waiting for material, and other operations.
4. Operating speed of rollers and other equipment.
5. Time required for various construction operations in connection with moisture control on section 1 of the Indiana project and on all sections of the Ohio project.

A summary of the construction records relating to the amount and percentage of compaction, the moisture contents of the compacted lifts, and the amount of rolling required to obtain the desired density are given in table 11 for the Indiana work, and in table 12 for the Ohio work. The results of the time and production studies presented in tables 13 and 14 were summarized from data furnished by H. L. Arbenz and W. R. Lytz, respectively, of the Division of Construction, Public Roads Administration.

TABLE 11.—*Construction record summary in Gibson County, Indiana*

Section No.	Type roller	Number lifts	Loose thickness of lifts specified	Average thickness of lifts constructed: Loose	Average thickness of lifts constructed: Compacted	Moisture content: Optimum [1]	Moisture content: Compacted fill: Average	Moisture content: Compacted fill: Minimum	Moisture content: Compacted fill: Maximum	Wet density: Maximum [2]	Wet density: Average, compacted fill	Percentage of maximum wet density: Required	Percentage of maximum wet density: Average obtained	Number trips with roller to obtain required density: Average	Number trips with roller to obtain required density: Minimum	Number trips with roller to obtain required density: Maximum	Remarks
			Feet	*Feet*	*Feet*	*Percent*	*Percent*	*Percent*	*Percent*	*Pounds per cubic foot*	*Pounds per cubic foot*	*Percent*	*Percent*				
1	Sheepsfoot	2															Foundation layer.
		1	0.50	0.56	0.54	21.6	23.3	22.2	24.4	123.7	124.1	95	100.3	4.5			Lift 11.
		17	.50	.50	.48	19.6	19.5	16.5	22.4	125.2	122.6	95	97.9	8.0	6	18	Lifts 3 to 10 and 12 to 20.
2	3-wheel	4															Foundation layer.
		10	.50		.50	17.3	22.9	20.8	24.7	126.5	123.3	95	97.5	2.1	2	3	Lifts 5 to 14.
		4	.50	.56	.48	21.6	24.8	24.6	27.9	123.7	123.9	95	100.2	2.0	2	2	Lifts 15 to 18.
		2	.50		.46	19.6	20.8	20.5	21.2	125.2	124.5	95	99.5	1.8	1	2	Lifts 19 and 20.
3	Sheepsfoot	4															Foundation layer.
		10	.50		.51	17.3	22.4	21.2	24.6	126.5	121.8	95	96.3	7.8	6	13	Lifts 5 to 14.
		2	.50	.52	.44	21.6	26.6	24.5	28.7	123.7	118.1	95	95.5	7.2	5	12	Lifts 15 and 16.
		2	.50		.42	19.6	20.4	20.1	20.7	125.2	120.4	95	96.2	5.5	5	6	Lifts 17 and 18.
4	3-wheel	3															Foundation layer.
		1	.75			21.6	20.4			123.7	128.2	95	103.6	2.0			Lift 4.
		6	.75	.77	.72	19.6	21.0	17.1	25.3	125.2	126.7	95	101.2	2.5	2	5	Lifts 5 to 10.
5	Pneumatic tire	3															Foundation layer.
		8	.75	.76	.69	21.6	22.9	20.2	25.7	123.7	120.5	95	97.4	3.9	3	12	Lifts 4 to 11.
6	3-wheel	2															Foundation layer.
		7	1.00	.95	.87	21.6	24.3	21.5	26.3	123.7	123.6	95	99.9	2.0	2	2	Lifts 3 to 9.
7	Pneumatic tire	2															Foundation layer.
		8	1.00	.99	.84	21.6	27.1	23.0	29.5	123.7	120.4	95	97.3	2.7	2	3	Lifts 3 to 10.
8	Pneumatic tire	2															Foundation layer.
		13	.50	.55	.52	18.4	20.4	14.9	26.3	125.0	123.7	95	99.0	2.9	2	6	Lifts 3 to 15.

[1] Determined from compaction tests performed in laboratory.
[2] Determined from compaction tests performed in laboratory. Figures given are maximum values for the compaction method used in the test and not absolute maximums.

TABLE 12.—*Construction record summary in Delaware County, Ohio*

Section No.	Type roller	Number lifts	Thickness loose material specified per lift	Average thickness per lift constructed: Loose	Average thickness per lift constructed: Compacted	Average moisture content: Optimum [1]	Average moisture content: Compacted fill	Average dry density: Maximum [1]	Average dry density: Compacted fill	Maximum dry density: Required	Maximum dry density: Average obtained	Number trips with roller to obtain required density: Average	Number trips with roller to obtain required density: Minimum	Number trips with roller to obtain required density: Maximum	Number moisture-density curves
			Feet	*Feet*	*Feet*	*Percent*	*Percent*	*Pounds per cubic foot*	*Pounds per cubic foot*	*Percent*	*Percent*				
1	Sheepsfoot	21	0.50	0.54	0.42	18.8	16.8	106.1	105.6	100	99.5	9.1	8	12	12
2	3-wheel	31	.50	.61	.49	18.2	16.0	108.3	113.9	100	105.2	2.6	2	4	16
3	Pneumatic tire	32	.50	.51	.39	17.6	15.6	109.4	112.6	100	102.9	4.8	3	6	17
4	Sheepsfoot	28	.50	.54	.38	18.4	17.4	107.4	109.0	100	101.5	6.3	5	10	18
5	3-wheel	28	.75	.66	.54	20.0	18.0	105.5	109.9	100	104.2	4.1	2	8	11
6	Sheepsfoot	13	.75	.78	.62	16.6	15.8	110.3	107.1	100	[2] 97.1	6.0	6	6	10
7	Pneumatic tire	10	.75	.90	.68	18.7	18.2	107.2	108.7	100	101.4	6.0	4	8	5
8	Sheepsfoot	38	.50	.51	.40	17.7	15.8	109.4	106.4	95	97.3	8.4	3	20	22
9	3-wheel	35	.50			17.6	15.6	109.3	110.5	95	101.1	3.3	1	8	11

[1] Average of all optimum moisture contents and maximum densities as determined in laboratory on the different soils placed in the test section.
[2] Two types of sheepsfoot rollers used on this section. All lifts compacted with 6 roller trips and densities recorded for comparison of compaction produced by each roller.

CONTROL OF MOISTURE CONTENT WITHIN A NARROW RANGE FOUND TO BE PRACTICABLE

Indiana Project.—As shown in table 11 the average moisture content of 17 of the 20 lifts in section 1 was only 0.1 below the optimum for the material used. On one lift the average was 1.7 higher than the optimum. Although in a few instances the variation from the optimum was somewhat greater than the 1 specified, it is evident that the moisture control practiced on this section produced a high degree of uniformity in moisture content. Variations from the optimum were found to be greater on the other sections where the fill material was placed as it came from the borrow pit without regard to its moisture content.

Ohio Project.—It was not practicable to keep the moisture content within the range of 1 of the optimum value. An attempt was made to maintain the moisture within the specified limits, but wider variations were permitted where satisfactory compaction was obtained and rigid control of the moisture content would have affected the progress of the work to such an extent as to be impracticable.

Variations between the average moisture contents of the compacted fills and the optimum moisture contents of the soils are given in table 9. The average variation for all the sections was 1.7 below the optimum, while average variation for the different sections ranged from 0.5 to 2.2 below the optimum. Although the variations in individual cases were somewhat higher, reaching a maximum of 5.5 below the optimum in section 8, the average values indicate that the moisture content of the soil can be maintained reasonably close to the optimum.

It is of interest to observe that on most of the work the compacted soil did not have a moisture content greater than the optimum. The number of lifts compacted at moisture contents below the optimum amounted to 92 percent on the nine test sections and varied from 74 percent on section 4 to 97 percent on section 3. This condition occurred in spite of the addition of water to all but a few of the lifts placed. The additional moisture was not sufficient, in most cases, to raise the moisture content to the optimum.

WEATHER INFLUENCED THE MOISTURE CONTENT OF FILLS IN INDIANA

The optimum moisture contents of the different soil materials used in the fill, the average variations from the optimum, and minimum and maximum variation from the optimum of samples of compacted soil are given in table 15, together with dates of construction of the various lifts and the sources of the fill material.

A study of table 15 and the weather records indicate that, in the absence of moisture control, the moisture content of the compacted lifts was influenced largely by weather conditions. The variation from the optimum was greatest in the portions of sections 2, 3, and 7 constructed in August, when there was a considerable amount of rain. In most cases the moisture content was higher than the optimum which was to be expected since in addition to the August rains the rains in July had been so heavy and frequent that practically no construction work could be performed. In contrast relatively small variations were found on sections 4, 5, and 6; lifts 19 and 20 of section 2; and lifts 17 and 18 of section 3 placed during September 2 to 17.

MOISTURE CONTROL DOES NOT CAUSE SERIOUS DELAYS IN CONSTRUCTION

Indiana Project.—One of the major delays in the construction of fills was waiting for material. This was due to an insufficient number of hauling units. Hauling operations were never delayed because of rolling. The time lost waiting for material (table 13) was greater than the actual rolling time required to compact the fill. Each of the rollers readily compacted all of the material delivered to the fill.

The hauling equipment was delayed on section 1 to correct the moisture content of the soil. Sprinkling of dry soil, drying of wet soil, and necessary disking resulted in a delay of 25.46 equipment hours. Addition of water by a distributor operating at a speed of about 3 feet per second took an average of 25 minutes per lift although as much as an hour was required on some lifts. Mixing of the soil and water was done with a disk harrow which operated at an average speed of 200 feet per minute. The average time for disking was 35 minutes per lift, but this varied according to the width of the lift and amount of water added.

In only one instance was it necessary to reduce the moisture content of the soil delivered to the fill prior to compaction. The soil was spread and left to dry with occasional disking to facilitate the drying. Six hours of intermittent drying and disking were required to process the soil in this manner.

Delays resulting from the moisture control occurred only on the first six lifts on section 1. As the various operations of wetting and disking the fill material became more familiar to the construction crew, delays were avoided.

Ohio Project.—The actual number of hours that the rollers were in operation was a small percentage of the time that they were on the job. Most of the time was consumed in waiting for material and for other operations in connection with hauling, spreading, disking,

TABLE 13.—*Summary of time and production studies of rolling in Gibson County, Indiana*

Section No.	Type roller	Thickness loose material per lift	Average speed of roller	Use of available working time: Rolling[1]	Use of available working time: Waiting for material	Use of available working time: Other operations	Volume compacted: Total for section	Volume compacted: Per hour of rolling	Rolling time per cubic yard compacted	Average roller trips to obtain required density
		In.	*Ft. per min.*	*Hr.*	*Hr.*	*Hr.*	*Cu. yd.*	*Cu. yd.*	*Sec.*	
1	Sheepsfoot	6	256	32.97	41.98	97.05	11,250	341	10.55	7.8
2	3-wheel	6	160	25.28	35.78	9.94	6,150	244	14.80	2.0
3	Sheepsfoot	6	185	19.08	26.85	5.07	4,855	255	14.11	7.4
4	3-wheel	9	150	11.08	15.67	9.25	5,074	458	7.85	2.4
5	Pneumatic tire	9	175	12.40	13.60	10.00	5,432	438	8.21	3.9
6	3-wheel	12	144	8.73	17.17	9.10	4,974	570	6.33	2.0
7	Pneumatic tire	12	220	11.27	29.08	6.40	6,280	557	6.44	2.7
8	Pneumatic tire	6	250	18.60	52.27	23.48	6,720	361	9.97	2.9

[1] Actual number of hours rollers were used for compacting.

TABLE 14.—*Summary of time and production studies in Delaware County, Ohio*

Section No.	Type roller	Thickness loose material per lift	Average speed of roller	Use of available working time: Rolling[1]	Use of available working time: Waiting for material	Use of available working time: Other operations	Volume compacted: Total for section	Volume compacted: Per hour of rolling	Rolling time per cubic yard compacted	Average roller trips to obtain required density
		In.	*Ft. per min.*	*Hr.*	*Hr.*	*Hr.*	*Cu. yd.*	*Cu. yd.*	*Sec.*	
1	Sheepsfoot	6	270	15.43	1.92	15.39	2,794	181	19.9	9.1
2	3-wheel	6	210	21.94	------	66.59	3,967	181	19.9	2.6
3	Pneumatic tire	6	280	41.74	2.33	41.28	9,159	220	16.4	4.8
4	Sheepsfoot	6	240	29.29	25.27	34.74	8,156	279	12.9	6.3
5	3-wheel	9	295	25.60	11.06	29.38	6,315	247	14.6	4.1
6	Sheepsfoot	9	320	8.73	2.50	6.10	2,232	255	14.1	6.0
7	Pneumatic tire	9	280	6.15	10.25	13.99	1,705	277	13.0	6.0

[1] Actual number of hours the rollers were used for compacting.

TABLE 15.—*Dates of placing material from different sources and variations from optimum moisture content in Gibson County, Indiana*

Section No.	Lift No.	Source of fill material: Borrow pit	Source of fill material: Depth below surface	Optimum moisture content	Variation from optimum: Average	Variation from optimum: Minimum	Variation from optimum: Maximum	Construction dates
			Feet	*Percent*	*Percent*	*Percent*	*Percent*	
1	11	A	8–15	21.6	[1]+1.7	+0.6	+2.8	Oct. 4.
	3 to 10	A	0–5	19.6	–.1	–3.1	+2.8	Sept. 28–Oct. 4.
	12 to 20	A	0–5	19.6	–.1	–3.1	+2.8	Oct. 5–12.
2	5 to 14	A	0–8	17.3	+5.4	+3.5	+7.4	Aug. 10–19.
	15 to 18	A	8–15	21.6	+3.2	+3.0	+6.3	Aug. 30–Sept. 1.
	19 to 20	A	0–5	19.6	+1.2	+.9	+1.6	Sept. 12.
3	5 to 14	A	0–8	17.3	+5.1	+3.9	+7.3	Aug. 10–19.
	15 to 16	A	8–15	21.6	+5.0	+2.9	+7.1	Aug. 30.
	17 to 18	A	0–5	19.6	+.8	+.5	+1.1	Sept. 12 and 17.
4	4	A	8–15	21.6	–1.2	------	------	Sept. 9.
	5 to 10	A	0–5	19.6	+1.4	–2.5	+5.7	Sept. 9–17.
5	4 to 11	A	8–15	21.6	+1.3	–1.4	+4.1	Sept. 6–9.
6	3 to 9	A	8–15	21.6	+2.7	–.1	+4.7	Sept. 2–9.
7	3 to 10	A	8–15	21.6	+5.5	+1.4	+7.9	Aug. 23–Sept. 2.
8	3 to 15	B	0–5.5	18.4	+2.0	–3.5	+7.9	Sept. 27–Oct. 6.

[1] Plus and minus signs indicate moisture contents above and below optimum.

and sprinkling. Construction work was never delayed because of the rolling. Each of the rollers readily compacted all of the material delivered to the fills.

It was necessary to add water to almost all lifts in order to comply with specifications for moisture control. This was done with an 800-gallon pressure distributor and two disk harrows. When moving at the speed of 250 feet per minute, the water was applied at the rate of 0.5 gallon per square yard. Two trips with a disk harrow operating at 200 feet per minute was generally required for mixing the water and soil. While these operations sometimes delayed the roller, they were not responsible for delays in hauling or spreading.

REQUIRED COMPACTION READILY OBTAINED

The construction records summarized in tables 11 and 12 show that the required compaction was obtained with all types of rollers.

The average density and average number of trips by rollers of each type required to obtain compaction are shown in table 16.

TABLE 16.—*The average density and number of trips needed for compaction*

Roller type	Number trips: Indiana	Number trips: Ohio	Maximum wet density in Indiana	Maximum dry density in Ohio
			Percent	*Percent*
Sheepsfoot	7.6	7.6	97.2	98.6
3-wheel	2.1	3.4	99.5	102.9
Penumatic tire	3.1	5.1	98.1	102.9

Indiana Project.—The three-wheel and pneumatic tire rollers gave equal compaction with the same effort regardless of whether the loose thickness of the lift was 6, 9, or 12 inches. The data disclose also that variations in average moisture content of the soil of as much as 5.5 above the optimum, and more than 7 in a few instances, did not increase the amount of rolling necessary to obtain the specified density.

Extreme cases of 18 trips with the sheepsfoot roller, 5 trips with the three-wheel roller, and 12 trips with the pneumatic tire roller were recorded. This occurred in only one instance with each type of roller and had little effect on the average values. As a matter of fact, the specified density was obtained with 6 or less trips on 69 percent of the lifts compacted with the sheepsfoot roller, with 11 to 13 trips on 24 percent of the lifts, and with 8 or 9 trips on 5 percent of the lifts. Likewise, 2 or less trips were required on 88 percent of the lifts compacted with the three-wheel roller, and 3 or less trips on 88 percent of the lifts compacted with the pneumatic tire roller.

Several tests were made on section 6 to determine if there was any difference in density between the upper and lower halves of a 12-inch lift compacted by a three-wheel, 10-ton roller. The results of these tests, given in table 17, show that satisfactory compaction was produced throughout the entire thickness of the lift and that the lower half is likely to have a density equal to or greater than the upper half. The moisture content of the soils in this section ranged from 0.1 below to 4.7 above the optimum, and averaged 2.7 above the optimum. It has been observed that these relatively high moisture contents facilitate compaction of soils such as were found on this project.

Ohio Project.—With the three-wheel and pneumatic tire rollers slightly more rolling was necessary on the 9-inch than on the 6-inch lifts in order to obtain equal compaction.

Extreme cases of 20 trips with the sheepsfoot roller and 8 trips with the three-wheel and pneumatic tire

TABLE 17.—*Results of density determinations on upper and lower halves of 12-inch lift compacted with 3-wheel 10-ton roller in Indiana*

Lift No.	Station	Number trips with roller	Maximum wet density: Upper half	Maximum wet density: Lower half
			Percent	*Percent*
7	271+02	2	105.9	101.0
8	270+32	2	105.1	111.2
8	270+35	2	102.3	109.8
9	271+02	2	100.4	106.6

rollers were recorded. This occurred on only one lift in the case of the sheepsfoot roller and on three lifts with each of the other rollers. The construction records show that the specified density was obtained with 8 or less trips on 77 percent of the lifts compacted with the sheepsfoot roller, 4 or less trips on 74 percent of the lifts compacted with the three-wheel roller, and 6 or less trips on 93 percent of the lifts compacted with the pneumatic tire roller.

The average densities of the compacted fills were very satisfactory. Although the minimum densities in some cases were considerably below the percentage specified, it will be seen that all lifts, with the exception of those in section 8, were compacted to a density of more than 90 percent of the maximum as indicated by the compaction test. Only three lifts in section 8 fell below this mark. These three lifts had densities equal to 83, 88.8, and 89.5 percent of the maximum.

ROLLER PRODUCTION DISCUSSED

Despite the fact that the required compaction was obtained by fewer trips of the three-wheel roller than by other types of rollers higher production in cubic yards compacted per hour was obtained with the pneumatic tire and sheepsfoot rollers on the Indiana project where the fill material was placed in 6-inch lifts and on the Ohio project for both 6- and 9-inch lifts. Differences in width and speed of roller account for this.

For example, sheepsfoot roller type A covered a strip 106 inches wide as compared with 60 inches for the pneumatic tire roller, while for the three-wheel roller the aggregate width of the rear rolls was 46 inches on the Indiana project and 40 inches on the Ohio project. At equal speeds the sheepsfoot roller could roll a given area once in 57 percent of the time required by the pneumatic tire roller, in 43 percent of the time required by the three-wheel roller in Indiana and in 38 percent of the time required by the three-wheel roller in Ohio.

Stated in another manner at equal operating speeds the sheepsfoot roller could roll an area approximately 1.8 times as fast as the pneumatic tire roller, 2.3 times as fast as the three-wheel roller in Indiana, and 2.6 times as fast as the three-wheel roller in Ohio.

Speed of Rollers in Indiana Project.—The average speed of the sheepsfoot roller of 256 feet per minute on section 1 was the maximum obtainable with the 40-horsepower tractor used to pull the roller. This tractor did not have the power necessary to operate at a higher speed.

Speed of the pneumatic tire roller was influenced by safety considerations. The roller had a tendency to tip over, so it was necessary to operate at slow speed along the edges and sides of the fill. Speed was increased on the central portion of the fill. This difficulty was not encountered with the sheepsfoot roller which could readily adapt itself to uneven and irregular surfaces.

In accordance with the specification requirements, the three-wheel roller was operated in extreme low gear at approximately 3 feet per second. On one lift of section 2 the speed was increased to about 6 feet per second. The density determination showed that the compaction was undiminished at the higher speed and that an increase in roller capacity corresponding to the increase in speed could be obtained.

Relative Production in Indiana Project.—Table 13 shows that the pneumatic tire roller compacted a cubic yard of soil in 6-inch lifts in less time than the other rollers. The greatest number of trips of actual rolling was recorded for the three-wheel roller on section 2. The maximum production of 361 cubic yards per hour was obtained with the pneumatic tire roller on section 8 as compared with 244 for the three-wheel roller on section 2, and 341 and 255 for the sheepsfoot roller on sections 1 and 3, respectively.

With respect to the production of the sheepsfoot roller on sections 1 and 3, the greater production was obtained on section 1 even though the average number of trips required to obtain the specified density was 5 percent higher. The increase in production of 33 percent is attributed entirely to the increase of 38 percent in the roller speed.

On the sections where the material was placed in lifts having loose thicknesses of 9 and 12 inches, greater production was obtained with the three-wheel roller than with the pneumatic tire roller. The sheepsfoot roller was used only on the 6-inch lifts. A very definite increase in production was accomplished by increasing the lift thickness. The influence of lift thickness on the roller production is shown in figure 26.

Ohio Project.—Table 14 shows that on the sections where soil was placed in 6-inch lifts 12.9 seconds was required to compact a cubic yard of soil with the sheepsfoot roller on section 4 and 19.9 seconds on section 1. These figures compare with 16.4 seconds for the pneumatic tire roller on section 3 and 19.9 seconds for the three-wheel roller on section 2. The least amount of time necessary to compact 1 cubic yard on the sections constructed in 9-inch lifts was 13 seconds with the pneumatic tire roller.

Maximum production of 279 cubic yards compacted per hour was obtained with the sheepsfoot roller on section 4. On section 1, however, the sheepsfoot roller was able to compact only 181 cubic yards per hour. The greater production on section 4 indicates that the higher moisture content of the soil was significant in facilitating compaction. The average variation from the optimum moisture content on section 4 was −1 as compared with −2 on section 1. The percentages of the total number of lifts compacted at moisture contents below the optimum on sections 1 and 4 were, respectively, 94 and 74 percent.

The average number of cubic yards compacted per hour for the different rollers are summarized as follows:

Sheepsfoot roller:	
6-inch lift	245 cu. yd.
9-inch lift	255 cu. yd.
All sections	247 cu. yd.
Pneumatic tire roller:	
6-inch lift	220 cu. yd.
9-inch lift	277 cu. yd.
All sections	227 cu. yd.
Three-wheel roller:	
6-inch lift	181 cu. yd.
9-inch lift	247 cu. yd.
All sections	216 cu. yd.

These data disclose maximum production with the sheepsfoot roller on the 6-inch lifts and with the pneumatic tire roller on the 9-inch lifts. Although the increase in production obtained with the sheepsfoot roller as a result of increasing the lift thickness is very small, the production of the pneumatic tire and three-wheel rollers was increased materially. The influence of lift thickness on roller production is shown in figure 27.

Variations in production and compaction corresponding to different operating speeds of the rollers, as summarized from the daily records of the production studies, are given in table 18. Although it was im-

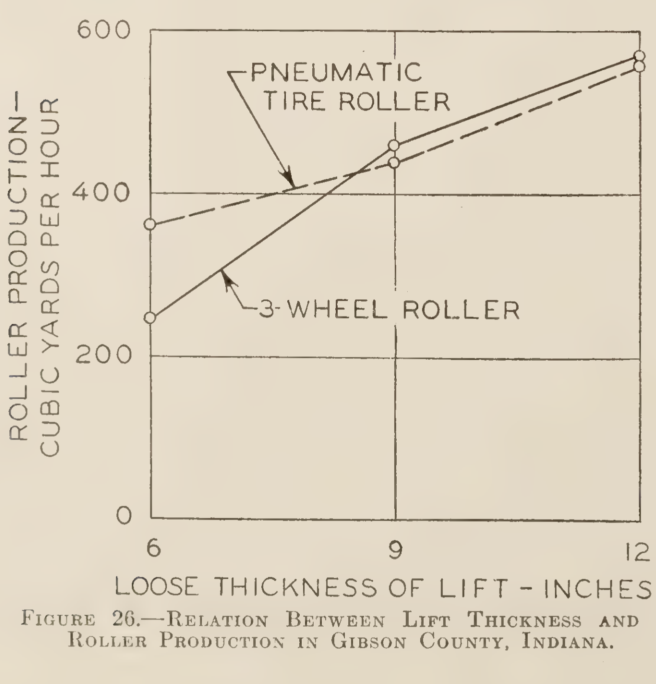


Figure 26.—Relation Between Lift Thickness and Roller Production in Gibson County, Indiana.

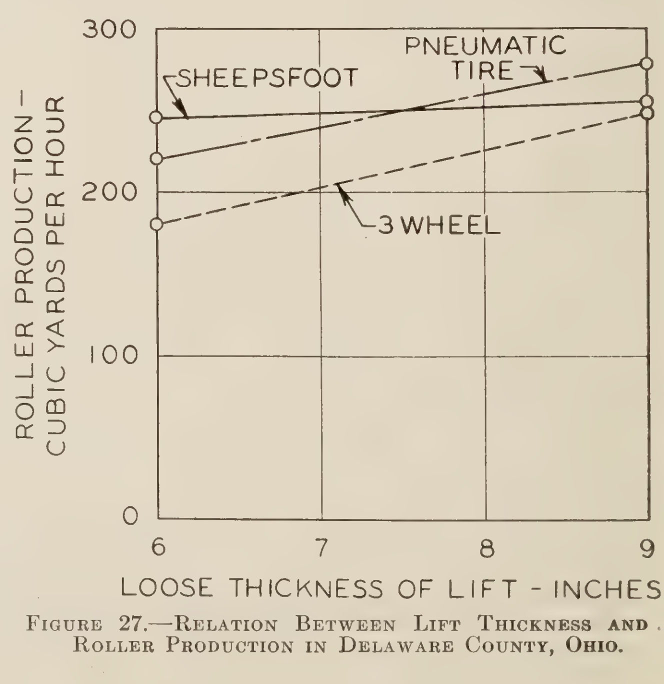


Figure 27.—Relation Between Lift Thickness and Roller Production in Delaware County, Ohio.

possible to maintain a constant rate of speed under the various construction conditions, it is believed that the speeds indicated represent the average speed during the period of observation. The data show a wide variation in volume of fill compacted per hour for the different roller speeds. There is, however, a definite indication that greater fill production was obtained at the higher speeds without any sacrifice in the quality of the compaction.

PERFORMANCE OF SHEEPSFOOT ROLLERS COMPARED WITH THAT OF OTHER ROLLERS

Ohio Project.—Of the 100 lifts on which the sheepsfoot rollers were used, 87 were compacted with type A roller and 13 with type B roller. The average densities obtained with each roller and the average number of trips required to obtain this density were as follows: Type A roller with 7.8 trips produced 98.7 percent of maximum dry density and type B roller with 6.8 trips produced 97.7 percent.

Type B roller was used only on eight lifts of section 4, and five lifts of section 6. The remaining lifts on these sections were compacted with type A roller. On section 4 both rollers were loaded to capacity and an attempt was made to obtain the specified 100 percent compaction.

On section 6, in addition to using both rollers with the drums full of water, roller type B was used with the drums about half full of water. While in this condition all lifts were compacted by six trips and the soil densities determined. The densities obtained were considered satisfactory.

The results obtained on sections 4 and 6 are summarized in table 19. These data indicate that slightly better results were obtained with roller type A than with roller type B, and that roller type B was more effective with the heavier load. Since there were variations in soil and moisture content at the time of compcction and the data are limited, it seems that the differences in compaction are not great enough to be significant.

TABLE 18.—*Effect of roller speed on production and compaction in Delaware County, Ohio*

Section No.	Type roller	Roller speed	Compaction [1]	Volume compacted per hour of rolling	Average roller trips to obtain compaction
		Feet per minute	*Percent*	*Cubic yards*	
1	Sheepsfoot	250	100.1	172	8
		300	96.8	238	8
		300	91.8	172	10
		300	103.0	172	12
2	3-wheel	100	108.2	75	3
		150	104.6	232	3
		200	110.1	264	3
		200	105.1	138	4
		300	104.3	294	3
3	Pneumatic tire	200	98.7	166	2
		200	105.6	164	5
		300	92.5	250	3
		300	101.7	223	4
		300	105.8	262	5
		300	104.0	244	6
4	Sheepsfoot	200	100.1	222	[illegible]
		200	106.4	158	7
		230	100.3	257	6
		240	102.2	332	6
		240	101.3	176	9
		250	100.7	300	6
		270	101.6	408	6
		300	100.8	441	5
		330	96.0	380	6
5	3-wheel	200	106.7	191	2
		240	104.0	236	3
		250	99.7	244	2
		300	107.7	269	4
		300	109.3	174	5
		330	99.2	290	5
		340	102.9	274	5
		400	109.8	317	5
6	Sheepsfoot	200	100.0	194	6
		360	95.9	285	6
7	Pneumatic tire	300	100.0	319	6
		300	101.2	306	7
		400	102.2	258	8

[1] Compaction in percentage of maximum dry density.

FIGURE 28.—COMPACTED FILL IN OHIO AFTER RAIN. THE SURFACE IS SLICK AND SLUSHY BUT NOT RUTTED.

TABLE 19.—*Summary of roller trips and densities on sections 4 and 6 in Delaware County, Ohio*

Roller type	Quantity of water in drum	Number lifts	Number trips	Maximum dry density [1]
				Percent
Section 4:				
A	Full [2]	20	5.9	101.9
B	Full [3]	8	7.3	99.1
Section 6:				
A	Full [2]	8	6	97.9
B	Partly filled [4]	3	6	94.7
B	Full [3]	2	6	95.6

[1] Average values.
[2] Ground pressure of 223 pounds per square inch.
[3] Ground pressure of 290 pounds per square inch.
[4] Ground pressure of 232 pounds per square inch.

OPERATION OF SHEEPSFOOT ROLLERS IN INDIANA IMPROVED BY LOAD ADJUSTMENT

To meet the requirements of the specifications relating to the ground pressure under each tamping foot of sheepsfoot roller type A, it was necessary to fill the drums with water. There was a tendency at times for the tamping feet to pick up the soil and tear material loose rather than compact it. This was overcome by drawing out part of the water and reducing the ground pressure under each foot from 209 to 164 pounds per square inch. The required compaction was readily obtained in this manner.

Sheepsfoot roller type B was used only on 6 of the 20 lifts in section 1. It was first tried with the drums full of water. Under this load the feet dug up the soil and satisfactory compaction could not be obtained. It was then used with the drums half filled with water and gave good results. The most efficient operation was obtained when the drums were empty. In this condition the ground pressure under each tamping foot was approximately equal to that of sheepsfoot roller type A partially filled with water.

FIGURE 29.—EROSION IN INDIANA (A) OF THE SHEET AND GULLY TYPE IN BORROW PIT A, (B) CHANNELS DRAIN INTO OUTLET IN BORROW PIT A, (C) IN FILL SLOPE BEFORE GOOD COVER OF LESPEDEZA WAS ESTABLISHED, AND (D) TYPICAL CONDITION OF SHOULDERS AND SLOPES ON ENTIRE PROJECT.

COMPACTION PRODUCED BY BULLDOZERS AND TRACTORS IN INDIANA COMPARED

The compaction produced by construction equipment other than rollers was investigated on lifts 5 and 6 of section 7. Density tests were made in the path of the bulldozer and tractors spreading the material deposited by trucks. These tests were in addition to those performed after compacting with the pneumatic tire roller, which was used on portions of lifts 5 and 6, as well as on the other lifts comprising the fill section. The results of these tests are presented in table 20.

The normal movements of the 17-ton bulldozer in spreading the soil on lift 5 produced a density of 94.4 percent of the maximum wet density. Two passages of the 11-ton tractor increased the density to 97.5 percent. This compares with 101.1 percent obtained with three trips of the pneumatic tire roller.

On lift 6 the density obtained with the 17-ton bulldozer was 93.3 percent of the maximum wet density. In one location, two passages of the 11-ton tractor did not increase the density, while in another instance the density was increased to 96.2 percent. This difference may be accounted for by the fact that in the former case the lower 2 inches of the lift was loose and uncompacted.

Two trips of the pneumatic tire roller over the bulldozed material did not serve to increase the density on lift 6. However, an average density of 97.3 percent was obtained on this fill section with the pneumatic tire roller, and in no case were more than three trips necessary to obtain the required density of 95 percent. The moisture contents of the fill material in this section ranged from 1.4 to 7.9 above the optimum and averaged 5.5 above the optimum.

COMPACTION PREVENTED SOFTENING FROM RAINS BUT DID NOT PREVENT EROSION

Removal or mixing with drier soil of any layer of the compacted embankment, which became softened

TABLE 20.—*Compaction produced by bulldozer, tractors, and pneumatic tire roller on lifts 5 and 6 of section 7 in Gibson County, Indiana*

Type of equipment	Number trips	Lift No.	Maximum wet density
			Percent
17-ton bulldozer	Spreading [1]	5	94.4
11-ton tractor	2 [2]	5	97.5
Pneumatic tire roller	3 [3]	5	101.1
17-ton bulldozer	Spreading [1]	6	93.3
11-ton tractor	2 [2]	6	[4] 93.1
Pneumatic tire roller	2 [3]	6	93.4
11-ton tractor	2 [2]	6	96.2

[1] Normal movement of bulldozer spreading material. Number of trips indefinite.
[2] Tractor moved forward and backward in same path after bulldozer had completed spreading.
[3] Roller used on portions of lifts after bulldozer had completed spreading.
[4] Loose, uncompacted material found in lower 2 inches of lift.

by rains was a requirement of the specifications for the Ohio work. Although heavy rains occurred during the construction of the test sections, they did not cause softening of the compacted lifts. The surface of the fill became slick and a small amount of slush was formed under the action of traffic as shown in figure 28 but there was no detrimental rutting. Only removal of the slushy material by blading was needed to accommodate traffic over the fills without any inconvenience.

The contractor was able to continue grading operations within a few hours after it stopped raining. Unsatisfactory condition developed only on the lifts compacted with the sheepsfoot roller. The small depressions left by the feet of the roller held water and became quite soft, necessitating considerable drying before additional material could be placed. This condition was overcome by smoothing out the surface with the three-wheel or pneumatic tire roller just as the rain was commencing.

As illustrated in figure 29, *A* and *B*, the soils used in the construction of the fills in Indiana were susceptible to severe sheet and gully erosion when not protected by a good cover of vegetation. A great many washouts and gullies were observed on the shoulders and slopes of all the sections soon after the fills were completed. Following the construction of the concrete pavement, the shoulders and slopes were repaired and planted with Lespedeza. Some erosion occurred (fig. 29, *C* and *D*) soon after planting but this has been reduced to negligible amounts since a satisfactory cover has been established.

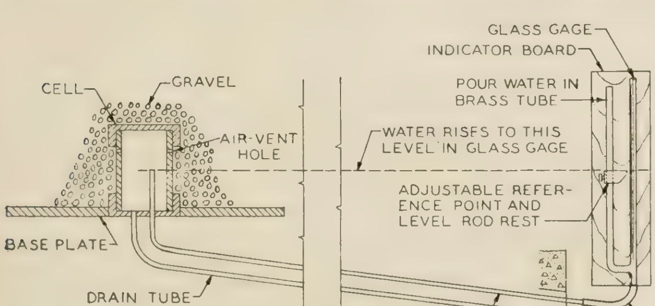
Embankment slopes in Ohio were left unprotected for the purpose of observing the resistance to erosion of the sections compacted by different types of rollers.

On neither of the projects was there a significant difference in condition of the various fill sections indicating that the edges of the fills were equally well compacted by all the rollers insofar as resistance to erosion was concerned.

SETTLEMENT CELLS INSTALLED

For the purpose of measuring the subsidence of the fills into the undersoil use was made of settlement cells. Total settlement of the fills including subsidence was determined from measurements made on reference stakes, plugs, and marked locations on the pavement.

The cells used to measure the subsidence are similar to the Ames settlement cell described in Bulletin 112 of the Iowa Engineering Experiment Station, published in 1933. The cell, and its accessory parts, illustrated



ure 30.—Diagram Showing Operation of Settlement Cell.

Figure 31.—Placing Settlement Cell in Ohio.

in figure 30, operates on the principle that the level of liquids in two vessels connected by a tube will be the same, provided the tube is large enough to allow the liquid to flow freely.

The cell (fig. 31) consists of a small chamber welded to a steel plate 12 inches square. It can be placed at any point in the embankment. Within the chamber are two orifices at different levels. The upper orifice, consisting of a tube screwed to the base of the cell, is coupled to a copper tube extending out through the fill to a convenient place where it is attached by means of a rubber hose to a glass gage tube mounted on an indicator board (fig. 30) which may be moved from one location to another. The lower orifice, flush with the base of the cell, is connected with another copper tube and serves merely as an overflow outlet for the system.

When taking readings to determine the changes in elevation of the cell, the indicator board with the attached glass gage and brass tubes is set in a vertical position near the toe of the fill slope, as illustrated in figure 32. By means of a rubber hose, the glass gage is connected to the copper tube leading to the upper orifice in the cell chamber. The other tube is left open to serve as a drain. Water is poured into the brass tube until it overflows the upper orifice and flows out from the outlet pipe. Water is added until the system is entirely free of air as indicated by a uniformly continuous flow from the drain tube. The level of the water in the glass gage is then the same as the upper orifice in the cell chamber. The adjustable reference point is moved vertically on the brass tube to coincide with the level of the water in the glass gage.

The adjustable reference point also served as a rest for the level rod when determining the elevation of the water in the glass gage.

Figure 32.—Pouring Water Into Gage on Ohio Project.

Air-vent holes drilled near the top of the cell serve to break suction or siphonic action when water is poured into the tube quite rapidly.

Cells were installed either on the original ground surface under the fill or on top of the foundation layer on the Indiana project and on top of the old roadway fill on the Ohio project. All measurements were referenced to frostproof bench marks situated at convenient locations outside of the fill areas.

The locations of the cells and bench marks are shown on figures 3 and 4. The design details of the frostproof bench marks and their method of installation are illustrated in figure 33.

The arrangement of the settlement cells and accessories on the Indiana project is illustrated in figure 34, *A*, and on the Ohio project in figure 34, *B*. A trench about 12 inches wide was excavated from the center line where the cell was to be placed perpendicular to the center line to a point just outside the fill slope. The bottom of the trench was sloped toward the outer end to give a drop of about 18 inches.

The two ¼-inch copper tubes, which were of sufficient length to reach beyond the fill slope, were coupled to the cell and placed along the bottom of the trench.

On the Indiana project a thin layer of gravel was spread over an area slightly greater than would be covered by the base plate of the cell. The cell was seated firmly on the gravel or on the ground (in Ohio), and the tubing was straightened to eliminate kinks. Gravel (about 1 cu. ft. in Indiana) or crushed stone (about 2 cu. ft. in Ohio) was placed over the cell to provide air space around the air-vent holes, and the

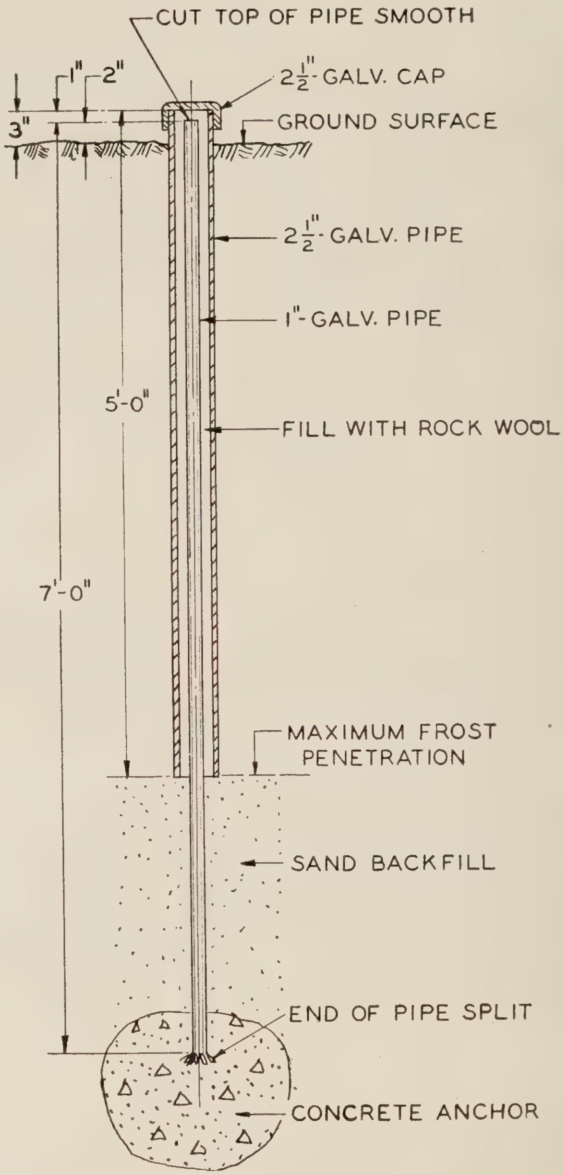


Figure 33.—Bench Mark.

trench containing the copper tubes was backfilled with soil and tamped by hand.

A mound of soil about 15 inches high was placed over the cell and tubes to protect them from damage by construction equipment. Soil was delivered by trucks and deposited on each side of the tubes. The soil was spread by a bulldozer, care being taken not to pass over the cell until the mound was completed. The protective mound of soil was from 15 to 20 feet in width.

The copper tubes were protected where they emerged from the fill by the concrete head wall illustrated in figure 32.

REFERENCE STAKES, PLUGS, AND MARKS PLACED ON TOP OF FILLS

In Indiana the settlement occurring in the fill prior to the construction of the concrete pavement was determined by taking elevations on reference stakes set in the top of the completed embankment directly over the points where the settlement cells were placed. The stakes were cut in lengths of about 16 inches from 4-inch green saplings and were painted with motor oil to retard cracking. A 2-inch boat spike was driven in the top of each stake.

A hole, 8 inches in diameter and 12 inches deep, was dug with a post-hole digger and the stake, after being again painted with oil, was placed on undisturbed soil in the hole. A 2-inch layer of loose soil was placed in the hole around the stake and oil was poured on the soil. After the oil had penetrated the soil, the layer was tamped, and another layer was placed in the same manner.

Oil-saturated rock wool was used instead of oil-treated soil for the stakes located over cells 12, 13, and 14. The rock wool was mixed with oil and tamped in place in layers having a compacted thickness of 1 inch. The hole was filled with rock wool to within 4 inches of the top, and filling was completed with oil-treated soil.

All these precautions were taken to eliminate the effects of frost action. In addition, a covering of straw was placed over the stakes during the winter months. Three 2-inch-square guard stakes were placed around each settlement stake to protect it from damage by cars or equipment.

After the concrete pavement was constructed during August 1939, a brass plug was installed at the location of each cell, at the quarter points of the pavement. The details of the plug and its installation are shown in figure 35.

In Ohio settlements occurring in the different fill sections were measured on the east curb of the pavement at points located at the same stations as the settlement cells. These points on the curb were marked by a 2-inch cross chiseled in the concrete and painted with red enamel. The marks were cut in the curb during the period from October 24 to 28, 1938, and the elevations determined at that time.

SETTLEMENT AND SUBSIDENCE CAREFULLY OBSERVED

Indiana Project.—Readings on the settlement cells were taken two or three times a week while the fill was being placed; once every 10 or 15 days from the completion of the fill section until the completion of the entire job on October 12, 1938; and subsequently on November 10 to 14, 1938, May 22 to 23, 1939, April 26 to 28, 1940, June 10 to 12, 1940, December 10 to 11, 1940, and November 17 to 19, 1941.

Readings on the reference stakes on top of the fill were taken when the readings on the settlement cells were taken and, in addition, on March 20 and May 8, 1939. The last measurements were made on the stakes on May 22 and 23, 1939, just prior to the construction of the concrete pavement. Readings on the reference plugs were taken on June 11 and 12, 1940, December 10 and 11, 1940, and November 17 and 19, 1941. The results of the measurements on cells, stakes, and plugs are summarized in table 21.

The total settlement of the fill sections as measured on the brass plugs installed in the pavement ranged from zero to 0.03 foot with an average of 0.01 foot. These movements were recorded during the period from June 1940 to November 1941.

The maximum settlement as measured on the wooden stakes from completion of the fill to the construction of the pavement was 0.09 foot at the location of cell No. 8 in section 4. The average for sections 2 to 8 was 0.05 foot. An average upward movement of 0.015 foot was recorded in section 1.

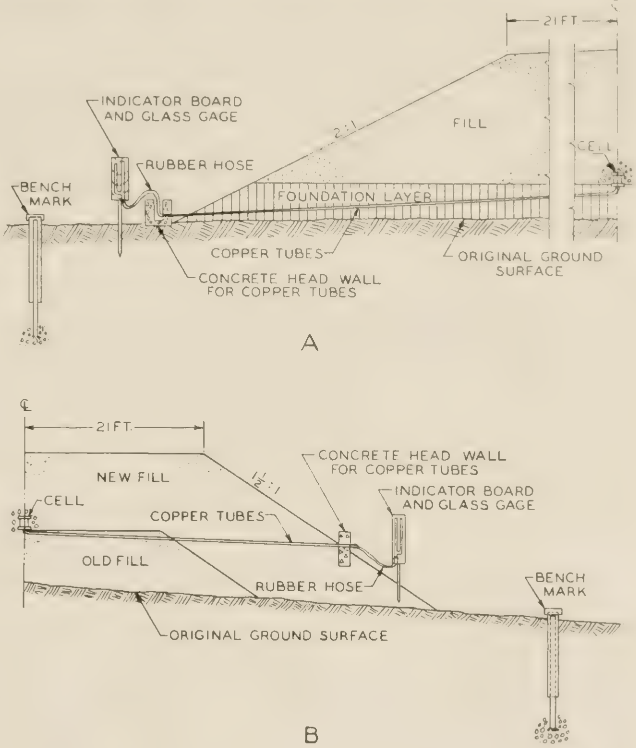


FIGURE 34.—INSTALLATION OF SETTLEMENT CELL AND ACCESSORIES IN INDIANA (A) AND IN DELAWARE COUNTY, OHIO (B).

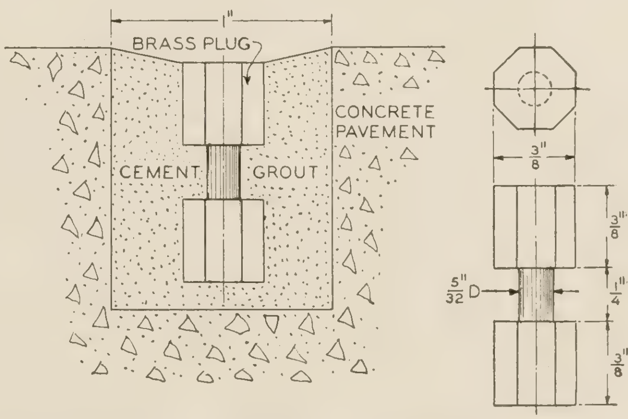


FIGURE 35.—DETAILS OF BRASS PLUGS SET IN PAVEMENT IN GIBSON COUNTY, INDIANA.

Ohio Project.—Settlement cell readings were taken each day while the fill was being placed, once a week during the period from the date each section was completed until the date of completion of the entire project on October 24, 1938, and subsequently in February, April, July, and November 1939, July and December 1940, and January 1942. After October 28, 1938, measurements at the marked locations on the pavement curb were taken on the same dates as the readings on the settlement cells. The results of the readings on the settlement cells and on the east curb are plotted in figure 36 and summarized in table 22.

Although the concrete base and curb were constructed almost immediately after completion of the embankment (2 to 28 days), the total settlement taking place within the body of the fill as measured on the east curb did not exceed 0.08 foot. The curb in sections 1 and 6

TABLE 21.—*Summary of measurements of vertical movements of settlement cells under fill, wooden stakes in top of fill, and brass plugs in pavement in Gibson County, Indiana*

Section No.	Cell No.	Movement of settlement cell: During fill construction	Movement of settlement cell: Total to May 1939	Movement of settlement cell: Total to November 1941	Total movement of wooden stakes	Movement of cells during period of total movement of wooden stakes	Total movement of brass plugs [1]	Movement of cells after brass plugs were installed [2]
		Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
1	1	[3]−0.15	−0.22		[4]+0.01	−0.07		
	2	−.12	−.08	0.15	+.02	+.04	0.00	−0.03
2	3-A	+.26	−.70	.60	−.08	−.96	−.02	−.42
3	5	−.70	−1.29	−.43	−.06	−.66	−.01	.00
4	7	−.12	−.12	−.05	−.05	.00	.00	+.24
	8	−.09	−.37	−.52	−.09	−.28	−.01	+.38
5	10	−.16	−.23	−.24	−.06	−.07	−.01	+.24
6	11	−.32	−.45	−.29	−.05	−.13	−.02	−.14
	12	−.25	−.59	−.23	−.06	−.34	.00	+.03
7	13	−.23	−.26	−.22	−.05	−.03	.00	−.01
	14	−.18	−.23	−.17	−.05	−.05	.00	+.02
8	15	−.05	−.01	−.08	−.01	+.04	−.03	−.10
	16	−.10	−.08	−.06	.00	+.02	−.02	+.01

[1] Values are averages of measurements made on two plugs installed at quarter points on each side of center line for the period from June 1940 to November 1941.
[2] Differences in elevation from June 1940 to November 1941.
[3] − denotes downward movement.
[4] + denotes upward movement.

started to move upward soon after construction and at the last readings had not returned to its original elevation. However, the maximum uplift (see fig. 36) never exceeded 0.04 foot at any time during the period of observations. The average settlement on six of the nine sections was 0.025 foot while an average upward movement of 0.017 foot was recorded on the other three sections.

BEHAVIOR OF SETTLEMENT CELLS DESCRIBED

Indiana Project.—Cell Nos. 3, 4, and 6, placed directly on the natural ground surface in sections 2 and 3 which were the first constructed, were soon put out of operation by action of the grading equipment in placing the foundation layer. Cell No. 1 in section 1 and cell No. 16 in section 8 were installed on the natural ground surface after a prolonged dry period. The remaining cells were placed on top of the foundation layer.

Cell No. 1 functioned satisfactorily until May 22, 1939, after which it could not be made to operate and all readings were suspended at this location.

Cell Nos. 2, 7, 8, 10, 13, 14, and 15 were functioning in a satisfactory manner when the readings were taken in November 1941. A uniform flow of water was always obtained through the cell and the level of the water in the glass gage remained constant. When water was run through the cell a second time, the first reading was readily checked. The entire operation required about 15 minutes.

The readings taken in November 1941 disclosed that cell Nos. 2, 7, and 8 were becoming sluggish in their operation.

Cell No. 16 did not respond readily when first installed. A considerable amount of water was added before any flowed from the outlet. After excavating some of the fill material near the toe of the slope, it was found that the tubes were curved upwards. As soon as they were straightened, the flow became uniform and the normal amount of water was required.

Cell No. 3–A behaved very sluggishly. A steady flow of water was obtained through the cell, but the level in the gage fluctuated over a range of as much as 5 inches when the water was cut off. The fluctuation in water level gradually decreased to a range of one-half inch in 15 minutes. When this fluctuation was reduced to about one-fourth inch, the indicator was set at the average level and a check was made. Several determinations were sometimes necessary before a reading was checked.

More time was required to determine the elevation of cell No. 5 than for any of the others. When apparently at equilibrium, the water level in the gage dropped suddenly and then slowly moved upward, coming to rest after about 45 minutes.

The action of cell Nos. 11 and 12 wa quite similar. Each required the ordinary amount of water to obtain a uniform flow through the cell. When the water supply was shut off, the level fluctuated. The fluctuation was very irregular and did not gradually decrease as in the case of cell No. 3–A. The level would drop quickly and rise slowly, or vice versa, and the change in level might be 6 inches or a very slight amount. A constant level might be reached in a few minutes or it might take nearly an hour.

Ohio Project.—Erratic readings were attributed in part to adjustments in the seating of the cells resulting from the operation of heavy equipment in placing and compacting the fill material. Evidence of such action is the large amount of settlement occurring, for example, in section 2, on the first day following the installation of the cell. There seems to be no reason for this behavior except that the foundation for the cell was disturbed when the cell was installed.

Also, it is very likely that kinks had developed in the soft copper tubes extending through the fill. When

TABLE 22.—*Summary of readings taken on settlement cells and on east curb in Delaware County, Ohio*

Section No.	Construction period in 1938: Fill: Started	Construction period in 1938: Fill: Finished	Construction period in 1938: Base and curb: Started	Construction period in 1938: Base and curb: Finished	Time from completion of fill to construction of base and curb	Initial readings in 1938: Cell: Date	Initial readings in 1938: Cell: Elevation	Initial readings in 1938: Curb: Date	Initial readings in 1938: Curb: Elevation	Vertical movement of cell [1]: During construction of fill	Vertical movement of cell [1]: During construction of fill and pavement	Vertical movement of cell [1]: Total to January 1942	Vertical movement of cell [1]: From date of initial reading on curb to January 1942	Total movement of east curb to January 1942 [1]
					Days		*Feet*		*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
1	9-19	9-25	10-18	10-19	24	9-20	907.88	10-24	914.73	−0.04	−0.09	−0.10	−0.01	+0.01
2	9-25	10-5	10-19	10-20	15	9-27	905.18	10-24	912.83	−.24	−.29	−.29	.00	−.01
3	9-24	10-5	10-21	10-22	17	9-28	898.51	10-28	908.83	−.01	−.02	−.09	−.07	−.03
4	10-4	10-19	10-24	10-25	6	10-5	913.47	10-28	922.99	−.05	−.07	−.08	−.01	−.08
5	10-14	10-24	10-26	10-26	2	10-16	923.09	10-28	934.27	−.08	−.08	−.13	−.05	−.01
6	10-11	10-15	10-27	10-27	12	10-12	917.88	10-28	923.07	−.05	−.05	−.07	−.02	+.03
7	10-7	10-10	10-15	10-15	5	10-8	922.71	10-24	927.13	−.05	−.07	−.04	+.03	−.01
8	8-31	9-19	10-10	10-10	21	9-5	918.47	10-24	928.79	−.18	−.24	−.27	−.03	−.01
9	8-26	9-12	10-10	10-10	28	8-30	916.95	10-24	927.02	−.12	−.20	−.22	−.02	+.01

[1] Figures show difference in elevation between dates of initial and final readings for period covered regardless of fluctuations during that period. (See figure 36.)

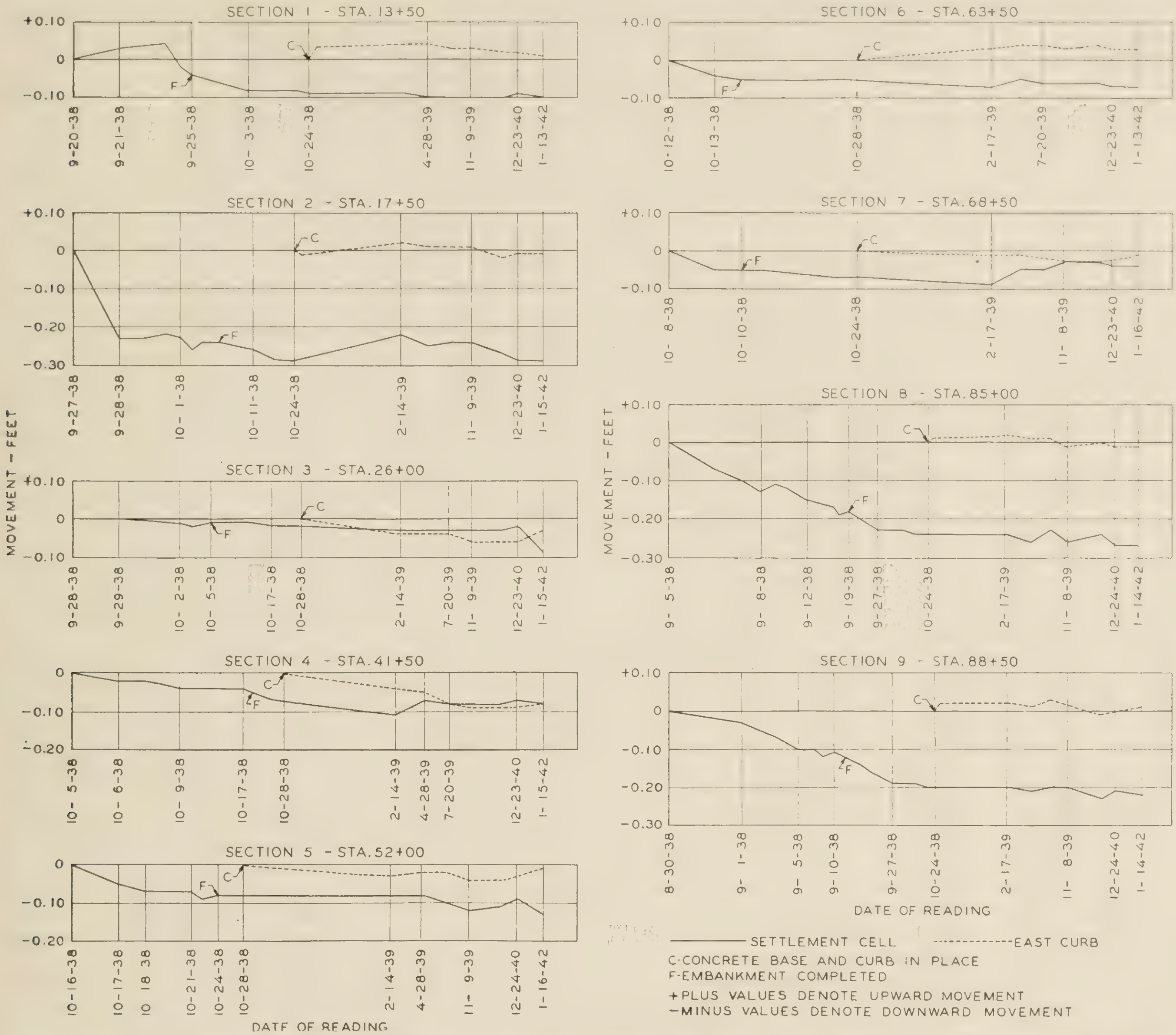


Figure 36.—Results of Measurements on Settlement Cells and East Curb Fill Construction Experiment, Delaware County, Ohio.

testing the operation of a cell prior to installation under the fill, it was observed that kinks in the line interfered with the free flow of water through the system and made it difficult to check readings.

Both Projects.—The purpose of the measurements of settlement on the top of the fill or pavements was to indicate the relative stability of the various fill sections compacted by different rollers and with various thicknesses of layers.

It had been assumed that differences between the total settlement as measured on the top of fill or pavement and the settlement as measured by the cell readings would disclose the amount of consolidation that had occurred within the fill. However, the data presented in tables 21 and 22 show that the subsidence indicated by the cells had no direct relation to the total settlements on either project. Upward movements of the cells were recorded at the same time that downward movements were measured on the stakes, plugs, and pavement curb, and vice versa.

Upward movements were recorded on the settlement gages at different times during the entire period of observation. Those occurring during construction may have been caused by adjustments in the seating of the cells or displacements of the soft undersoil resulting from the operation of the heavy equipment. However, the upward movements found a year or more after the completion of the embankment, cannot be attributed to foundation support but must be due to faulty operation of the cells or their accessory parts. As a result, the elevation measured on the gage outside of the fill did not accurately represent the elevation of the cell.

The behavior of cells is a strong indication that kinks developed in the soft copper tubes to the extent that they interfered with the free flow of water through the system.

Since the installation of the cells in these experiments, several improvements [6] have been made in the design of the system and in the method of seating the

[6] Consolidation of Embankment and Foundation Materials, Progress Report of Subcommittee No. 2 of the Committee of the Soil Mechanics and Foundations Division on Earth Dams and Embankments. Proceedings of the American Society of Civil Engineers, vol. 66, No. 8, part 1, October 1940.

cell which tend to eliminate the difficulties described above.

CONCLUSIONS

The following conclusions seem justified in view of the fact that there have been neither indications of detrimental results from settlements nor indications of instability on any of the sections of either the Ohio or Indiana projects since the construction of the pavements.

Both Projects.—1. The compaction test may be utilized generally to control the construction of embankments regardless of the type of compacting equipment. It offers a practical means of determining when a layer of soil is satisfactorily compacted.

2. Data from compaction tests performed in the laboratory and from density tests made on the compacted fill offer a means of determining the moisture content and lift thickness at which satisfactory compaction may be obtained most economically with a given type of roller.

Indiana Project.—3. A density equal to 95 percent of the maximum as determined in the laboratory in accordance with method T 99–38 of the American Association of State Highway Officials is apparently sufficient to produce highway fills of satisfactory stability when the dimensions of the fills are similar to those on this project. The maximum height of fill in the various test sections ranged from 7 to 10 feet.

4. Soils similar to those comprising the fills on this project may be readily compacted to the desired density by any of the rollers used in this experiment if the moisture content of the soil is within the proper limits. With proper moisture content, equally good compaction was obtained throughout the entire thickness of a compacted layer regardless of whether the soil was compacted by the tamping action of the sheepsfoot roller, the kneading action of the pneumatic tire roller, or the compression produced by the three-wheel roller.

5. The moisture content of fills may be controlled within 1 of the optimum value when the soil is obtained from a borrow pit as was done on this project. Such rigid control, however, does not seem justified in light of the densities and stabilities obtained in this experiment with moisture contents ranging from 3.5 below to 7.9 above the optimum.

6. The high degree of stability attained at the wide range in moisture contents indicates that control of compaction on the basis of density alone will produce satisfactory results provided the soil contains sufficient moisture for the rollers to be effective and is not so wet as to interfere with the operation of the rollers. Definite limits above and below the optimum will depend entirely on the character of the soil. These limits may be determined by observing the performance of the rollers in conjunction with density tests.

7. The amount of fill material compacted per hour with a roller of a particular type depends on the type of soil, the moisture content of the soil, the thickness of the layer, and the speed of the roller. With soils and moisture contents typical of this project, it was found that when the loose thickness of the soil layer was increased from 6 to 12 inches and when the rollers were operated at their maximum speeds, a corresponding increase in roller capacity was obtained without any sacrifice in compaction.

Ohio Project.—8. Highway fills of satisfactory stability may be obtained when constructed in layers compacted to maximum density as determined on the same types of soils in accordance with the standard compaction test (Method T 99–38 of the American Association of State Highway Officials). Such embankments may be paved immediately following construction without danger of detrimental settlement.

9. Soils similar to those comprising the fills studied may be readily compacted to the desired density by any of the rollers used in this experiment when spread in layers having loose thicknesses of 6 to 9 inches. The soils ranged from silty clay loams to clays having physical properties of the A–4 to A–7 groups. They were of friable to hard consistency when dry but were plastic when wet.

10. A density equal to 95 percent of the maximum obtained in the compaction test is apparently satisfactory when the type of soils and the depth of the fills (5 to 11 feet over the old fill core and 7 to 21 feet on the widened portions) are similar to those on this project and when the moisture contents are reasonably close to the optimum.

11. Control of the percent of moisture within 1 of the optimum value is not practicable where the soil delivered to the fill is a mixture of materials varying in moisture-density characteristics. It is desirable that a greater permissible variation in moisture content from the optimum be established and this permissible variation should depend on the type of soil.

12. The use of moisture-density curves typical of the soils to be used and the selection of a particular curve in the manner described in this report facilitates the use of compaction test data under variable conditions, and is a practicable method of field control.

13. The construction of embankments may be controlled in accordance with the moisture contents and densities indicated by compaction test data without causing delay in construction operations.

14. The compaction of areas rolled at maximum roller speed is equal to that obtained at slower speeds. The higher speeds result in a corresponding increase in roller capacity.

PROSPECTING FOR GRAVEL DEPOSITS BY RESISTIVITY METHODS

BY THE DIVISION OF PHYSICAL RESEARCH AND TESTS, PUBLIC ROADS ADMINISTRATION

Reported by R. WOODWARD MOORE, Associate Civil Engineer

IN the spring of 1940 field tests were made by the Public Roads Administration in the vicinity of Beltsville, Md., to further investigate the usefulness of the earth resistivity method of test in locating gravel deposits.[1] Considerable quantities of gravel were needed in local road improvements within the Beltsville Research Center grounds. Shallow test pits had failed to indicate an adequate supply of gravel in an area where a remnant of an old gravel-capped plateau was known to exist and where gravel deposits seemed most likely to occur.

Digging test pits in sandy gravel formations is often hampered by serious caving within a few feet of the surface. Deep pits require relatively large openings and shoring of the walls for safety. Meeting these requirements is costly and time consuming and a more rapid and less expensive means of exploring for gravel is desired.

A number of routine field tests were made with the resistivity equipment in the area prospected by the shallow test pits to locate, if possible, the outlines of gravel deposits or pockets that might offer a saving in the construction work nearby. The results of these geophysical tests indicated that the earth resistivity method of test may be applied with considerable success in areas where a sandy gravel is underlain by a clayey sand, a sandy clay, or a clay stratum.

RESISTIVITY TESTS INDICATE LOCATION OF GRAVEL

The resistivity test, as the name implies, involves measuring the resistivity of the materials lying below the ground surface. The presence and approximate depth to any material having resistivity characteristics markedly different from that of the overlying or surrounding formations is revealed by variations in the resistivity values obtained. These are governed by the relative quantities of the materials involved.

The apparatus used by the Public Roads Administration is shown in figure 1.[2] The three units, the instrument containing the potential and current-measuring devices, the battery box and the reels of wire, can be readily carried over any ordinary terrain. The battery box, which contains sufficient batteries to supply approximately 200 volts to the current circuit, has compartments for carrying incidental equipment such as tools, tapes, electrodes, etc. The potentiometer, mounted on the left-hand portion of the instrument panel, is equipped with a sensitive galvanometer (0.125 microamperes per mm. on its own scale) and has a maximum range of 1.10 volts. The milliammeter, mounted on the right-hand portion of the panel, is a sensitive unit equipped with shunts to give full-scale readings of 0–15,

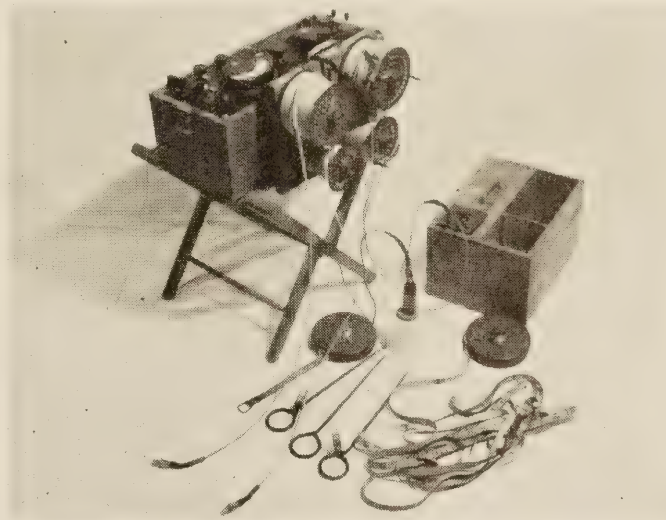


FIGURE 1.—RESISTIVITY APPARATUS USED BY THE PUBLIC ROADS ADMINISTRATION.

0–75, 0–150, and 0–300 milliamperes. The potential electrodes consist of porous porcelain pots, such as the one shown in figure 1, and they are filled with a saturated solution of copper sulphate in which is immersed a coil of heavy copper wire (No. 8 or larger). The current electrodes (not shown in figure 1) are heavy iron rods about 2 feet long and 1¼ inches in diameter.

In making a resistivity test the apparatus is set up on an ordinary camp stool and the current electrodes are driven into the ground so spaced that one-third of the distance separating them constitutes the approximate desired depth of investigation. The two potential electrodes are placed at the third points between the current electrodes and all four electrodes are connected to the instrument by the wires provided for that purpose. A current of a few milliamperes strength is introduced into the ground, and a zero reading is obtained on the galvanometer of the potentiometer circuit by adjusting the potentiometer controls. The value of the potential drop set up in the potentiometer to balance that existing between the porous pots resting on the ground surface, together with the value of the current flowing through the ground as obtained from the milliammeter, are inserted in an appropriate formula and the apparent or average resistivity of the ground is obtained. The underlying theory and a diagram of the electrical connections are presented in the earlier report. The method has been referred to previously as the direct current method.

There are two tests commonly made with the resistivity apparatus, the step traverse and the depth test. In making a step traverse, resistivity data are obtained that relate to the formations in the subsurface within a fixed depth which is controlled by the spacing of the

[1] PROSPECTING FOR ROAD MATERIALS BY GEOPHYSICS. Stanley W. Wilcox. Engineering News Record, February 21, 1935, p. 271.
[2] See *Subsurface Exploration by Earth Resistivity and Seismic Methods*, by E. R. Shepard, published in the June 1935 issue of PUBLIC ROADS. Figure 4 of this earlier report shows the circuit used.

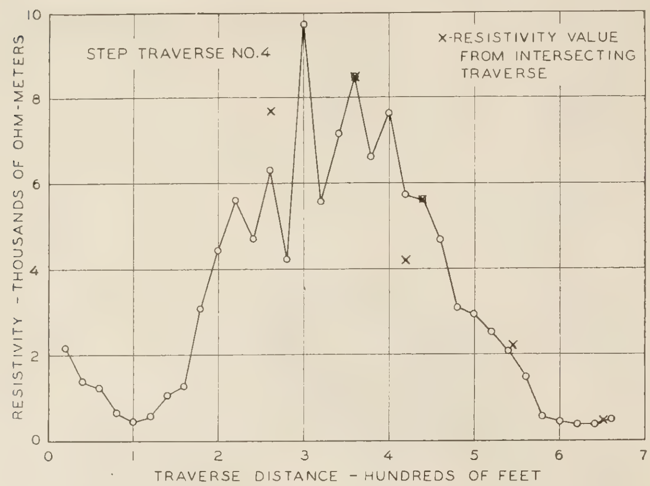


FIGURE 2.—STEP TRAVERSE OVER DEPOSIT OF SANDY GRAVEL WITH ELECTRODE SPACING OF 20 FEET.

electrodes on the ground surface. Experience has developed the empirical relation that the depth of the layer explored is approximately equal to the electrode spacing used. Thus, a 20-foot electrode spacing may be expected to yield information concerning a layer of material approximately 20 feet in depth. By maintaining this 20-foot spacing, and making tests at each 20-foot station or interval along a selected line of exploration, variations in resistivity are obtained which indicate increasing or decreasing amounts of material of high resistance, such as a sandy gravel, within the depth involved. If the resistivity values are plotted as ordinates against traverse distances as abscissas, the variations in the subsurface formations within the 20-foot layer are reflected with fair accuracy by variations in the resistivity graph. A step traverse over a deposit of sandy gravel underlain by clay and clayey sand is shown in figure 2.

When using this method of test in prospecting for gravel, the normal procedure is to make a series of tests along a random traverse line passing over likely locations. The proper electrode spacing is governed by the depth to which the information is desired. If a high resistivity zone is found (fig. 2), other tests are made on lines parallel to the random line at 50- to 100-foot intervals. Additional tests are made along lines, similarly spaced, but differing in orientation by approximately 90 degrees. This results in data for points that approximate a grid and it is possible to plot a resistivity contour map as shown in figure 3.

When a pocket of gravel is indicated, further exploration is made by depth tests, and it is desirable to make a few direct tests with a 6- or 8-inch post-hole auger. Samples of the material excavated from the bore holes show the nature of the material and can be used for determining the physical properties of the underlying formations.

In the depth test, data are obtained indicating the resistivities of the successive strata as one probes deeper into the ground. In this test the resistivity of the sub-

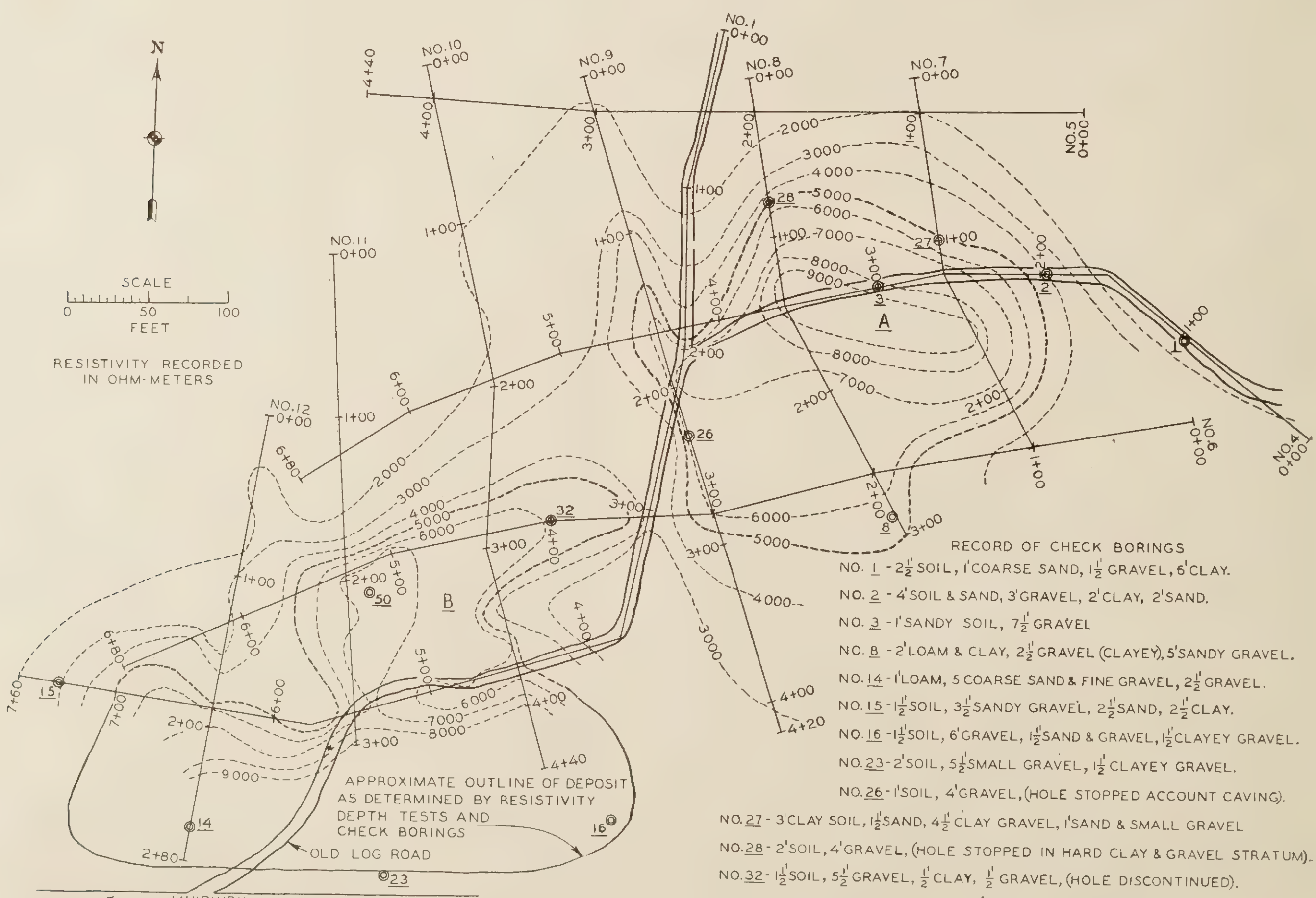


FIGURE 3.—RESISTIVITY CONTOUR MAP OF A DEPOSIT OF SANDY GRAVEL.

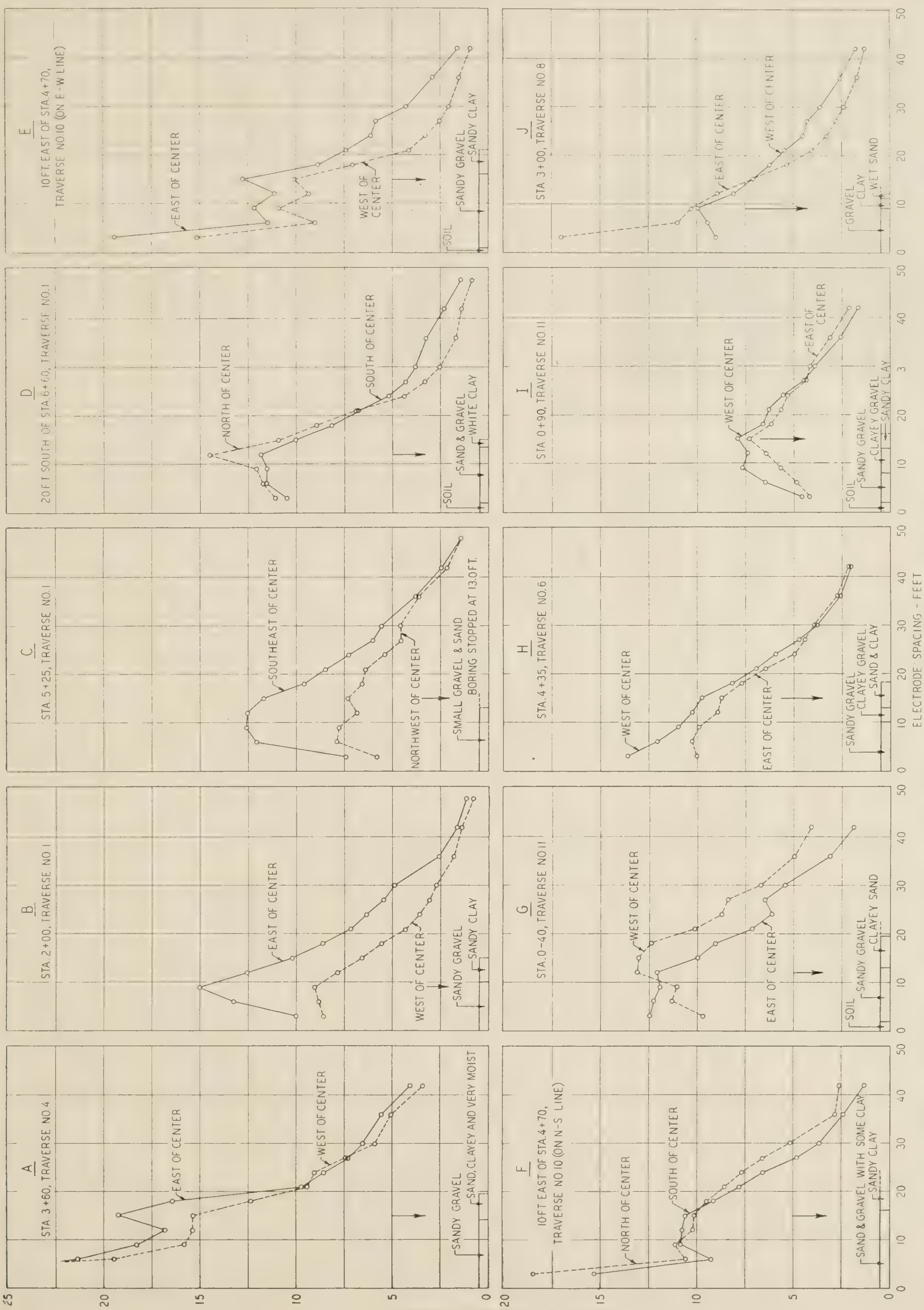
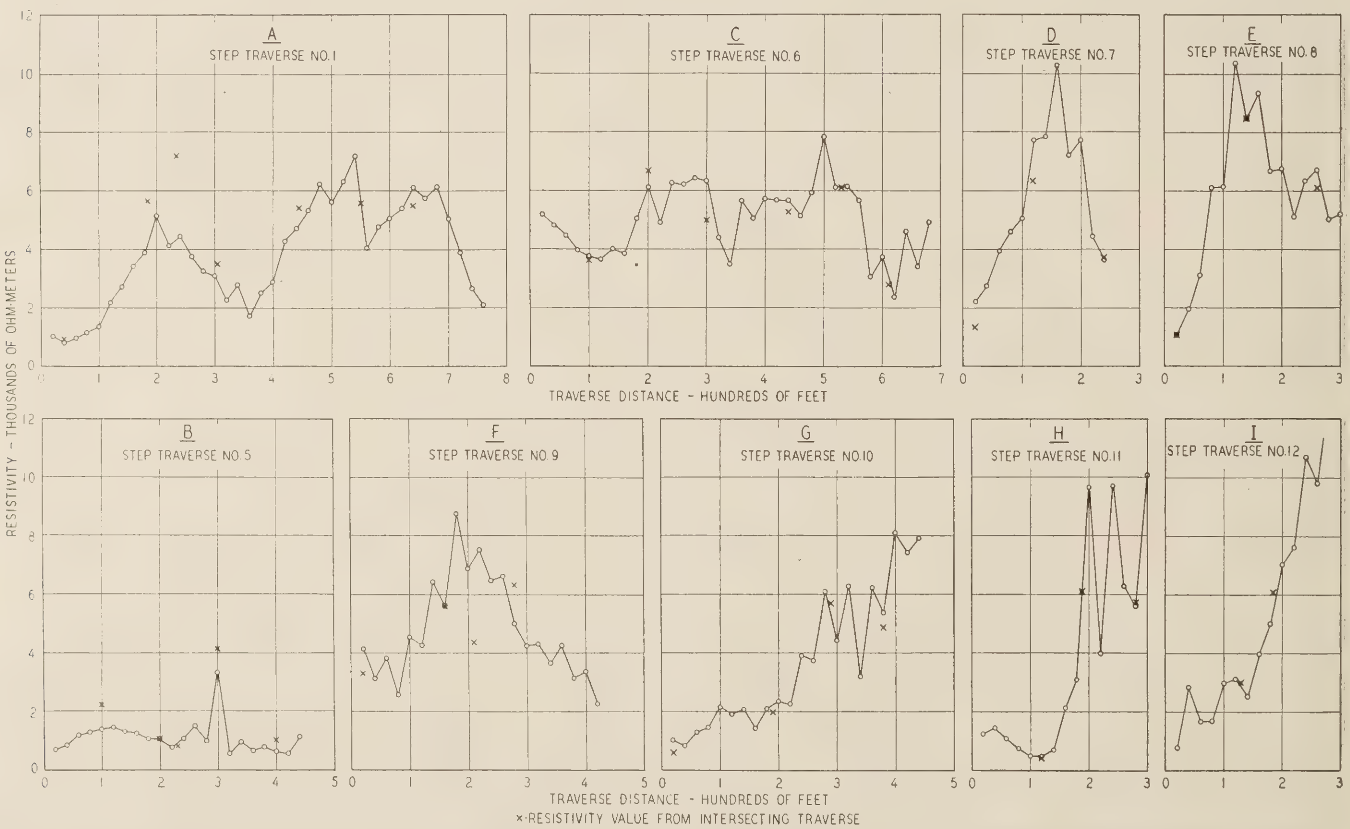


FIGURE 4.—RESULTS OF RESISTIVITY DEPTH TESTS IN A SANDY GRAVEL FORMATION UNDERLAIN BY CLAY OR CLAYEY SAND. TWO SETS OF READINGS WERE TAKEN AT EACH POINT, USING THE LEE PARTITIONING METHOD. RESULTS OF TEST BORINGS SHOWN IN LOWER LEFT CORNER OF EACH GRAPH.

Figure 5.—Boring With a Post-Hole Auger to Determine the Character of the Subsurface Materials.

surface material is determined for a succession of gradually increasing electrode spacings. As the spacing is increased, the depth of material controlling the resistivity increases correspondingly. A common procedure in shallow exploration is to use an initial electrode spacing of 3 feet and subsequent spacings of 6 feet, 9 feet, 12 feet, etc. Each increase in the spacing of the electrodes probes approximately 3 feet deeper into the ground. The data are plotted with resistivity as ordinates and electrode spacing as abscissas and any inflection in the resistivity curve indicates the depth at which there is a change in the character of the formation. A number of such depth curves are shown in figure 4. In these tests there was a change from a gravel of high resistivity to clay or clayey sand with low resistivity. This change is indicated in the curves by the rather sharp decrease in resistivity at electrode spacings varying from 9 to 15 feet. In these tests the gravel lay within a few feet of the surface and the depth to the lower boundary of the gravel is indicated by the arrow appearing in each of the graphs. It is usually possible to pick the top horizon of the gravel also when the depth of the overburden is of sufficient importance to warrant it.

Borings made at the locations of several of the depth tests showed that this method of test and analysis indicated the lower horizon of the gravel rather accurately. Under favorable conditions holes 20 to 30 feet in depth may be bored with only moderate difficulty with an ordinary 6-inch post-hole auger (fig. 5). Two men may be expected to sink 4 or 5 holes to depths of 12 to



A - ALONG OLD LOG ROAD FROM A POINT ABOUT 400 FEET NORTH OF RIDGE TO A POINT ON SUMMIT AND TURNING APPROXIMATELY 100° WEST TO A POINT OVER THE RIDGE AND SOME 360 FEET DISTANT.
B - ABOUT 100 FEET NORTH OF TRAVERSE NO. 4 - CROSSING NOS. 7, 8, 9, AND 10 AT STA. 0+20 AND TRAVERSE NO. 1 AT 0+60 (0+60, NO. 1 = 2+30, NO. 5).
C - 100 FEET SOUTH OF TRAVERSE NO. 4 - CROSSING NO. 7 AT 2+40, NO. 8 AT 2+60, NO. 9 AT 2+80, NO. 10 AT 2+90, NO. 11 AT 1+90 AND NO. 12 AT 1+30.
D - FROM A POINT 20 FEET NORTH OF STA. 1+00, NO. 5 TO STA. 1+00 NO. 6 - STA. 1+20 NO. 7 = 2+60, NO. 4.
E - FROM A POINT 20 FEET NORTH OF STA. 2+00, NO. 5, TO 40 FEET SOUTH OF STA 2+00, NO. 6 STA. 1+40, NO. 8 = 3+60, NO. 4.
F - FROM A POINT 20 FEET NORTH OF STA. 3+00, NO. 5, TO 140 FEET SOUTH OF 3+00, NO. 6, - STA. 1+60, NO. 9 = 4+40 NO. 4, STA. 2+10 = 2+35 NO. 1, STA. 2+80 = 3+00 NO. 6.
G - FROM A POINT 20 FEET NORTH OF STA. 4+00 NO. 5 TO A POINT 150 FEET SOUTH OF STA. 4+40, NO. 6 STA. 1+90, NO. 10 = 5+45 NO. 4: STA. 2+90 NO. 10 = 4+40, NO. 6 STA. 3+80 NO. 10 = 4+45 NO. 1.
H - FROM A POINT 120 FEET NORTH OF NO. 4 TO A POINT 20 FEET SOUTH OF NO. 1 - STA. 1+20 = 6+50 NO. 4 - STA. 1+90 = 5+30 NO. 6, STA. 2+80 = 5+50 NO. 1.
I - FROM A POINT 130 FEET NORTH OF NO. 6 TO A POINT 75 FEET SOUTH OF NO. 1 - STA. 1+30 = 6+10 NO. 6 - STA. 1+85 = 6+40 NO. 1.

Figure 6.—Effect of Gravel Deposit Upon the Resistivity Traverse (or constant depth test) Data With Electrode Spacing of 20 Feet.

15 feet in an 8-hour day provided excessively hard cemented strata are not encountered.

GRAVEL DEPOSITS SUCCESSFULLY LOCATED AT BELTSVILLE, MARYLAND

The tests were made at a point about 1.6 miles east of Muirkirk, Md., in a wooded area on the north side of the road which links Muirkirk with the Bowie road. A trial traverse was laid out along an old logging road passing through the area investigated. Resistivity tests were made at each 20-foot station along this line (traverse No. 1, fig. 3 and graph A, fig. 6). Two areas of high resistivity were located by this traverse. Another line of tests was run along another old road, crossing the first traverse near station 2+00 where the peak in the plotted graph occurs (traverse No. 4, fig. 3). A more positive indication of gravel was obtained in this second line of tests the results of which are plotted in figure 2. Referring to this figure, it will be seen that the resistivities encountered, involving as they do a zone extending approximately 20 feet to the right and to the left of the line of tests and some 20 feet in depth, were comparatively low for the first eight test points. A test boring at station 1+00 showed successively 2½ feet of sandy soil, 1 foot of coarse sand, 1½ feet of wet gravel, 5 feet of blue clay, and 1 foot of a heavy white clay. Other borings and resistivity depth tests made along this traverse showed a gradually increasing amount of gravel to be present as the peak of resistivity shown between stations 1+60 and 5+00 was approached. Beyond station 5+80 the tests indicate that the material within 20 feet of the surface is largely clay. Graphs A and B in figure 4 show comparisons of the results of borings with those of depth tests made at station 3+60 and at a point 10 feet south of station 4+20 (shown in graph B as station 2+00, traverse No. 1). The log of the boring is given in the lower left-hand corner of the graph. The close correlation between the resistivity depth test data and the data from borings is characteristic of all the tests made in the immediate vicinity. Graphs C to J show results of depth tests at other locations where check borings were made.

Only a few borings are normally required for use in interpreting the results of resistivity tests in a given area. A relatively large number were made in this study since there was no likelihood of being able to make a visual inspection of actual pit-face conditions at an early date and a direct and detailed check on the resistivity data was desired.

In a further expansion of the information concerning the gravel deposit other traverse lines were run parallel to the one shown in figure 2 (graphs B and C, fig. 6), and also a number of lines were run at approximately right angles to these lines and parallel to the one shown in graph A (see graphs D to I, fig. 6). Referring to figures 2 and 6, it is apparent that where one traverse crosses another there is usually similarity of the resistivity values obtained in the two separate test lines at their point of intersection. The plotted crosses indicate the values of resistivity obtained on the lines that crossed the traverse shown in the graph. In only one case, that where traverse No. 9 crosses No. 1, is there any considerable difference between the resistivity values obtained. It is believed that the information obtained with the resistivity apparatus is dependable when properly interpreted.

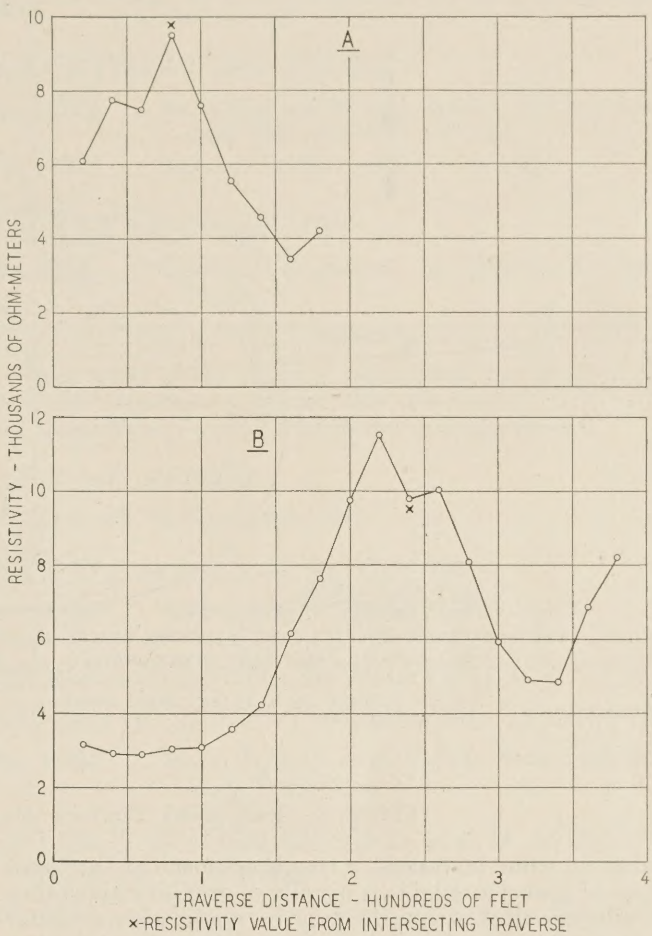


FIGURE 7.—DATA FROM TWO RESISTIVITY TRAVERSE LINES REQUIRED TO LOCATE AND OUTLINE A SMALL POCKET OF SANDY GRAVEL WITH ELECTRODE SPACING OF 20 FEET.

From the data obtained on the several traverse lines a resistivity contour map was made covering the entire area under investigation. This map is shown in figure 3 and it is used in somewhat the same manner as an ordinary surface contour map. The various contour lines represent resistivity and the area of highest resistivity is found to coincide with the thickest part of the gravel deposit.

Resistivity depth tests and check borings both indicated that an area roughly bounded by the 5,000-ohm-meter contour contained a gravel stratum averaging 8 to 10 feet in thickness. The data indicated also that the gravel would be found within the first 12 feet of depth over much of the area and a working face of 15 to 20 feet giving a fair mixture of sand, gravel, and clay for use in low cost road construction would be possible. The clay, for the most part, would be found in the lower 5 or 6 feet and in the form of clayey sand, sandy clay, or a comparatively pure clay.

An examination of the depth curves and data from test borings in figure 4 shows that the resistivity test data indicates the lower horizon of the gravel with a rather high degree of accuracy. The underlying material may vary from a sand with some clay particles to a heavy clay stratum and because of its variable nature it is not so readily defined. While a definite knowledge of the character of the underlying material may be of

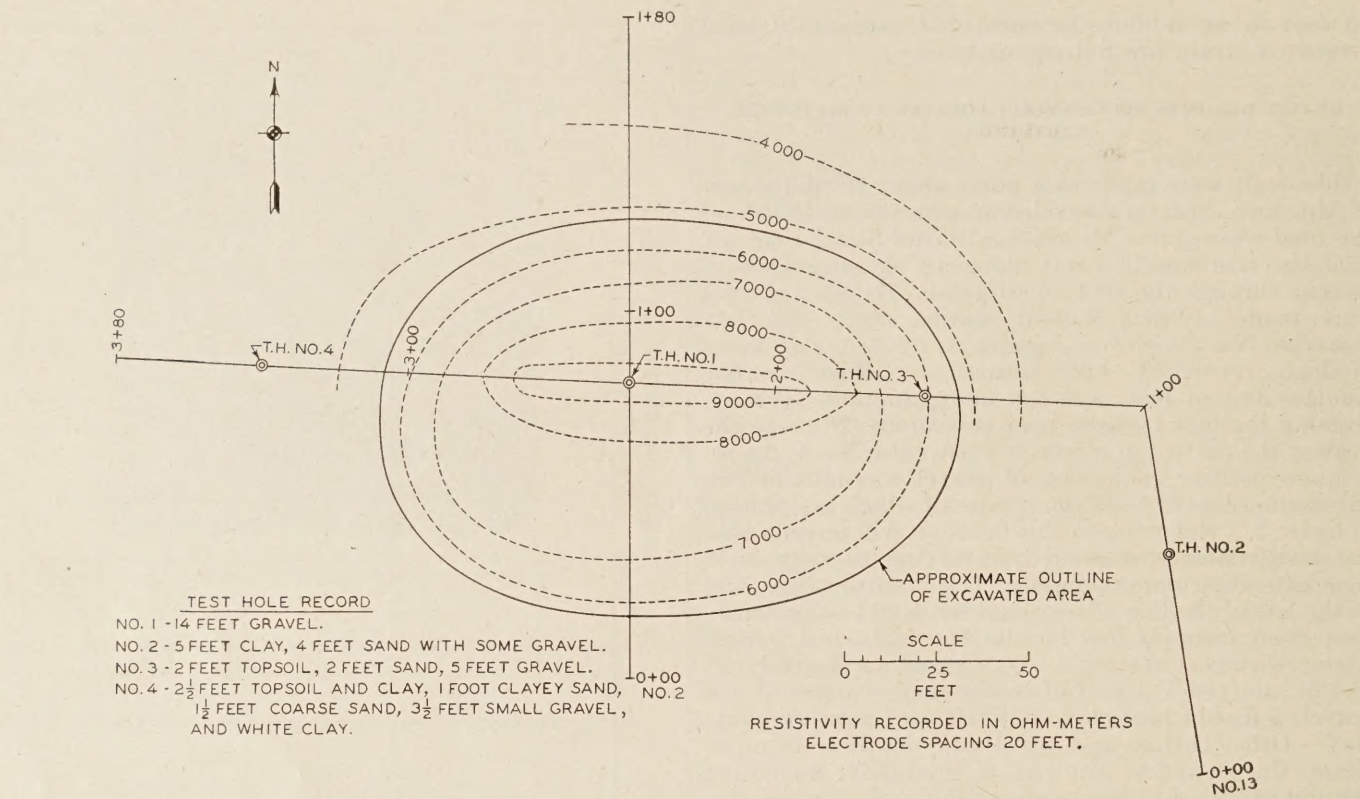


Figure 8.—Resistivity Contour Map of a Small Pocket of Sandy Gravel.

value, in some instances, a rough estimate of the quantities of gravel available is usually of greater importance. It appears that estimates may be made with considerable confidence when based upon resistivity data augmented by the results of a few test borings.

From the data obtained from the tests that have been described, it is estimated that the area marked "A" in figure 3, should contain approximately 10,000 cubic yards of a sandy gravel with some clay and sand spots, all within about 15 feet of the surface. Area "B" is somewhat less uniform and should contain about 15,000 cubic yards of gravel.

Other tests made to the south of the area described in the preceding paragraphs, in an open field where some scattered excavations had already been made, indicated the presence of a pocket of gravel of considerable extent. A random traverse line was run (graph A, fig. 7) and a test boring made at the peak of the resistivity curve at station 0+80. Another traverse line was laid out approximately at right angles to the first and crossing it at the location of the test boring. Graphs A and B in figure 7 show the data from parts of the two traverse lines and figure 8 shows the resistivity contour map obtained from them.

Considering the data shown in figure 8, an area bounded by a contour between the 5,000- and 6,000-ohm-meter contours was selected as the approximate boundary of the most valuable portion of the pocket of gravel.

Subsequent to and because of the indications of the resistivity survey, the gravel pocket was excavated and about 6,000 cubic yards of material were removed. The solid line on the contour map of figure 8 gives approximately the outline of the area excavated.

CONCLUSIONS

The resistivity method of exploration as used in this study offers a rapid and inexpensive means of locating and delineating gravel deposits that are underlain by clay. The most useful field for this method of test is in rapid reconnaissance surveys over relatively large areas where surface indications are few or unreliable. Routine subsurface tests along a projected highway location may be used to locate small local deposits of road-building material satisfactory for construction purposes, thus eliminating the expense of transporting materials from more distant sources. In the work described above where 6,000 cubic yards of material was obtained, the necessary field work was completed by one engineer and two inexperienced helpers in about 8 hours. With the usual field party of two engineers and two experienced helpers a faster rate is possible.

While the resistivity method has proved to be entirely satisfactory in many instances in the past, it should be emphasized that this method of test is not always practical and that there are some field conditions under which resistivity tests will be of little value. A dry, loose overburden may possess such high resistivity as to completely mask the effect of sand, gravel, or rock in the deeper strata. Clay soils overlying or underlying clay-filled gravels may have resistivities so nearly the same as that of the gravel that it is difficult to interpret the resulting resistivity curves accurately. Formations that are inherently heterogeneous yield data that, for obvious reasons, are of little value.

In spite of these limitations, it is believed that the resistivity method of subsurface exploration has a definite field of usefulness in the solution of certain highway problems.

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