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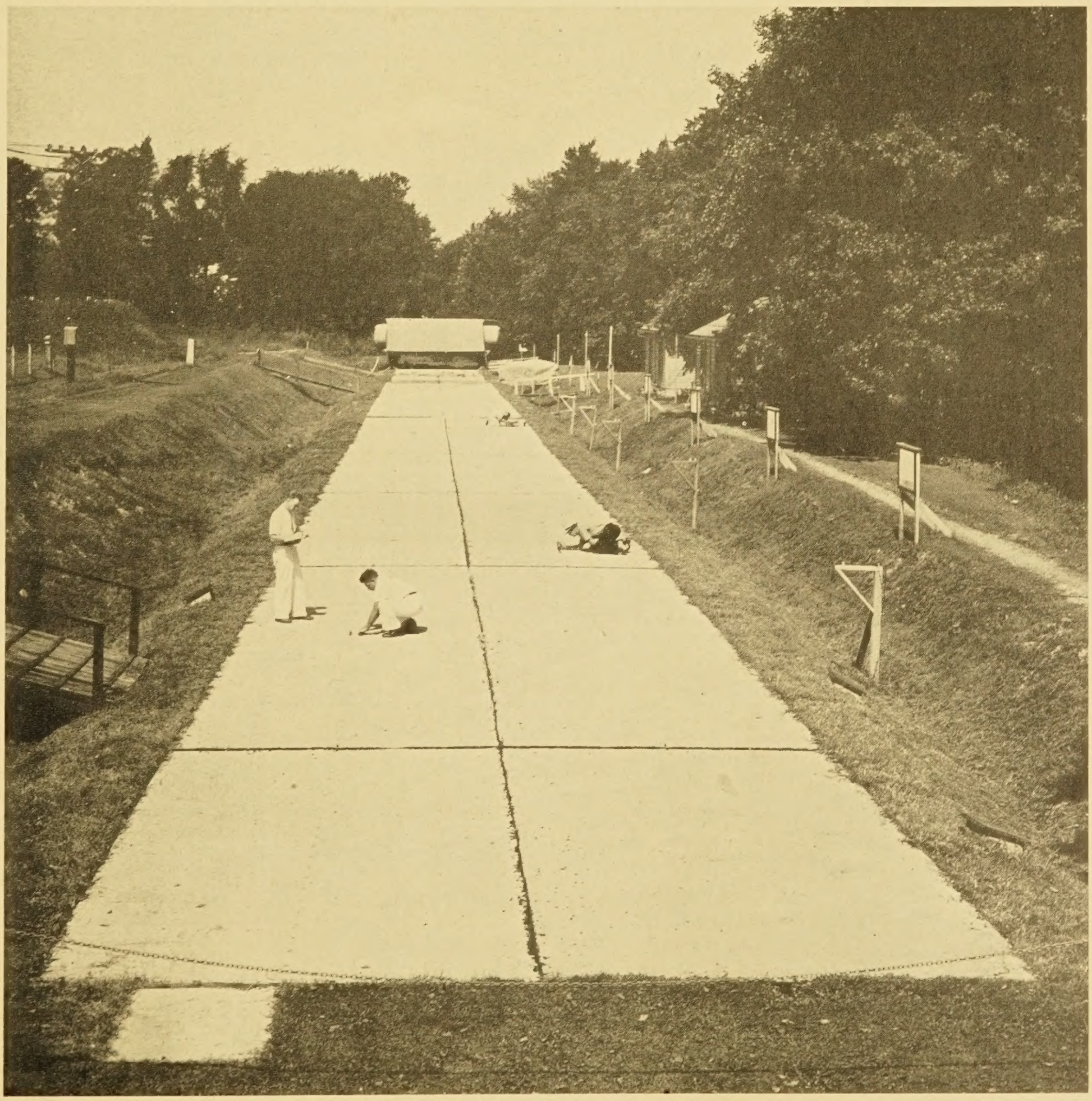

FULL-SIZE PAVEMENT SLABS TESTED AT ARLINGTON EXPERIMENT FARM

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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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THE STRUCTURAL DESIGN OF CONCRETE PAVEMENTS

BY THE DIVISION OF TESTS, BUREAU OF PUBLIC ROADS

Reported by L. W. TELLER, Senior Engineer of Tests, and EARL C. SUTHERLAND, Associate Highway Engineer[1]

PART 1.—A DESCRIPTION OF THE INVESTIGATION[2]

SINCE 1930, the Bureau of Public Roads has been conducting at the Arlington Experiment Farm, Va., an extensive investigation with the general objective of developing information that will be of assistance in better understanding the structural action of concrete pavement slabs.

More specifically, the research was planned to study the following four main subjects:

1. The effects of loads placed in various ways on pavement slabs of uniform thickness.
2. The "balance of design" or relative economy of typical pavement slab cross-sections.
3. The behavior under load and comparative structural effectiveness of typical longitudinal and transverse joint designs.
4. The effects of temperature conditions and of moisture conditions on the size, shape, and load-carrying ability of pavement slabs.

The study of the effects of loads placed in various ways on slabs of uniform thickness was intended primarily as an experimental verification of the only rational theory of pavement slab stresses thus far advanced, i. e., the Westergaard analysis.[3] The program was accordingly planned in such a way that each of the factors that theoretically might influence the load-stress relation could be examined experimentally and the observed effects compared with those predicted by the theory. In addition, this study was expected to indicate rather definitely what the shape of the slab cross section should be if the design were so balanced that a given load would produce a certain definite maximum stress regardless of the position of the load on the slab.

The study of the balance of design of typical pavement slab cross sections was planned, first, for the purpose of showing the relative economy of the various designs, and second, to provide data upon which to base conclusions as to the proper shape for a perfectly balanced cross section. The data obtained in the load tests on slabs of uniform thickness mentioned in the last part of the preceding paragraph necessarily form an important part of the study of the balance of the cross-section design.

The almost complete lack of data concerning the structural behavior of the various types of longitudinal and transverse joint designs existing at the time this research was planned and the importance of a knowledge of this action in any consideration of the structural design of pavements made a study of the subject imperative. That part of the investigation dealing with joint design was planned to yield data showing the structural effectiveness of most of the commonly used types of joints and also information regarding the effect of dowel spacing and joint width on the structural action of joints.

The fourth part of the investigation, that is, the study of the effects of temperature conditions and of moisture conditions on the size, shape, and load-carrying ability of pavement slabs, was planned to provide information, not heretofore available, on the complex relations created by temperature and moisture variations, and the practical significance of these relations with respect to the design of the pavement slab as a load-carrying structure.

In order to carry out the studies contemplated in this investigation the group of 10 full-size concrete pavement slabs shown on the cover page was constructed. Each of these slabs is 40 feet in length, 20 feet in width, and has a particular cross section. Each slab is divided by a longitudinal and a transverse joint of a particular design and each slab is definitely separated from those adjoining it, in most cases by a 2-inch open joint. The concrete was uniform throughout the group and all slabs were without steel reinforcing. Special efforts were made to obtain subgrade uniformity under the entire group of sections.

The load tests and other studies designed to develop the information desired have been made on these 10 slabs. Some idea of the magnitude of the work of testing may be had when it is realized that, in round numbers, some 30,000 strain measurements, 25,000 deflection observations, 65,000 measurements of slab expansion (or contraction), and 30,000 temperature measurements were made in the course of the investigation. Approximately 10 percent of these were made during the night or early morning hours.

Figure 1 shows the details of the several designs of pavement slab cross section included in the investigation. It will be observed that these include the rather massive edge design suggested a number of years ago by the American Association of State Highway Officials, 3 designs of the conventional thickened-edge type in which the edge thickening is decreased uniformly to zero over a distance of 3 feet, a design in which the upper and lower boundaries of the section are parabolas diverging so as to give a thickened edge, 2 lip-curb sections (with and without the conventional edge thickening), and 4 sections of uniform thickness. The area of the cross section (in square feet) for a 20-foot width pavement of each design is noted in this figure.

The details of the several designs of transverse joint included are shown in figure 2. The ordinary butt-type open joint with 3 different dowel spacings and 2 different widths of joint opening, the continuous steel plate key with 2 widths of joint opening, the thickened slab end (without dowels or other connection), and the "plane of weakness" both with and without dowels

[1] Roscoe Lancaster, Harry D. Cashell, Arthur L. Catudal, and Ernest G. Wiles, junior highway engineers, gave able assistance in carrying on the work reported. They contributed valuable suggestions as to procedure and made observations at all hours and under all weather conditions.

[2] A series of five articles has been planned. The first three will probably be published in consecutive issues. Parts 4 and 5 may not be published in issues consecutive with the rest of the series as they are dependent upon work yet to be completed.

[3] Stresses in Concrete Pavements Computed by Theoretical Analysis, by H. M. Westergaard, PUBLIC ROADS, vol. 7, no. 2, April 1926, and Analytical Tools for Judging Results of Structural Tests of Concrete Pavements, by H. M. Westergaard, PUBLIC ROADS, vol. 14, no. 10, December 1933.

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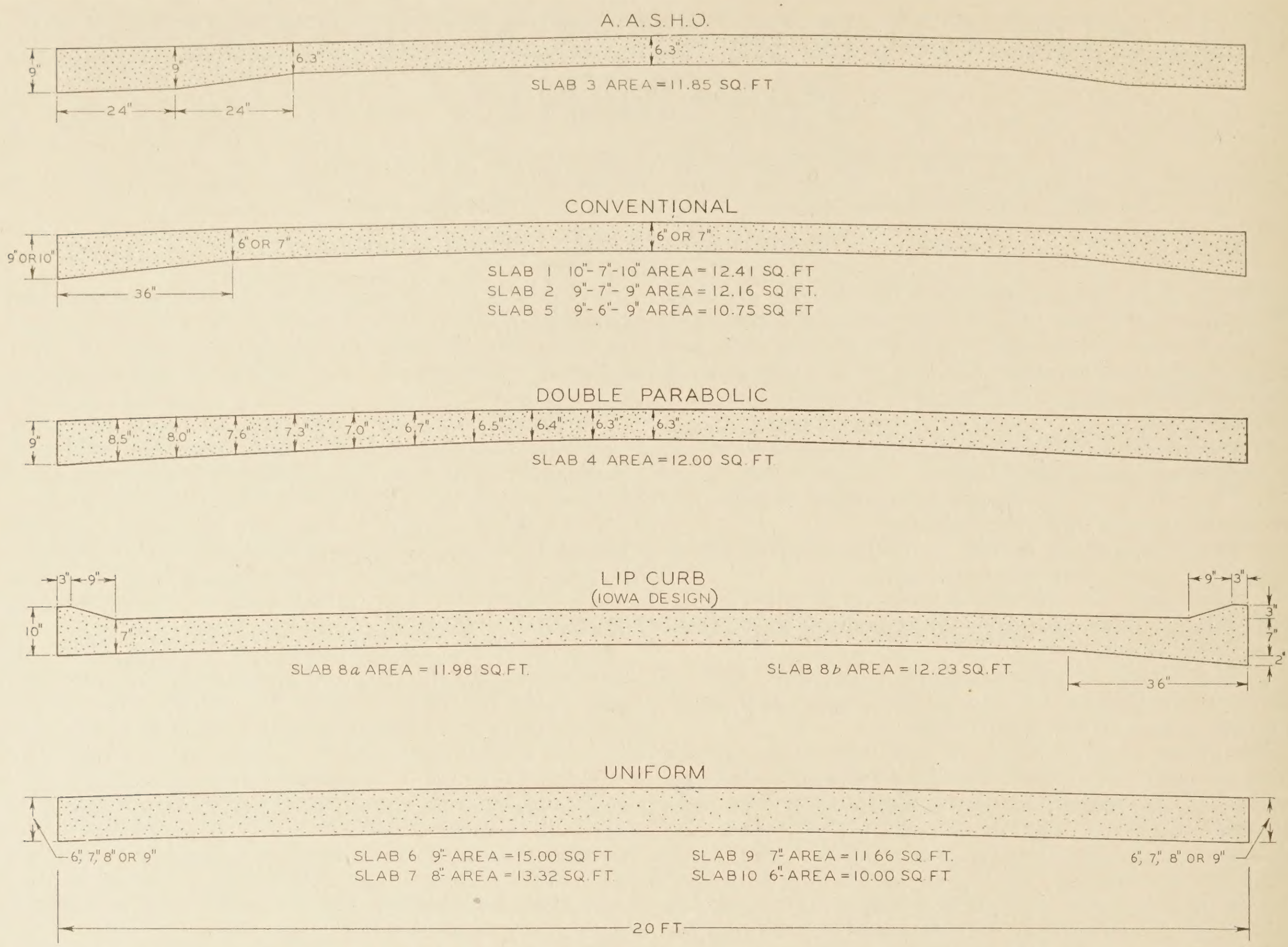

Figure 1.—Designs of Cross Sections Investigated.

comprise the types that have been tested. All of the joints have been kept filled with a typical poured bituminous joint filler.

The structural features of the several types of longitudinal joints used in the 10 slabs are shown in figure 3. In 4 of these, separation of the 2 slabs was accomplished with a deformed plate of heavy sheet metal; in 4 others the slabs were laid half width at a time and bond between the halves of the slab was prevented by a sheet of tarred felt. The other 2 slabs were grooved to create a longitudinal plane of weakness that was intended to crack through and form a separation between the 2 halves of the slab.

TEST SLABS CAREFULLY CONSTRUCTED

The subgrade.—The importance of subgrade uniformity in any test of the structural action of pavement slabs was recognized from the first. The site used was selected with this in mind and a detailed soil survey was made to determine the conditions existing in the area involved, with the result that the soil was classified as a uniform brown silt loam (class A-4). The uniformity of the soil is indicated by the results of tests made in the laboratory on the samples taken during the survey, as shown in table 1.

The original surface of the area having been disturbed by earlier experiments, the subgrade material was entirely removed until the new subgrade was entirely an undisturbed soil formation. On this surface the line of the pavement was laid out. In order to insure proper drainage, deep side ditches with suitable outlets were provided.

Table 1.—*Test data from subgrade samples*

Sample no.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
1	24	8	18	1.8	28	20
2	22	5	21	1.7	23	19
3	25	8	19	1.8	26	21
4	24	9	19	1.8	25	20
5	25	9	19	1.8	26	19
6	24	6	18	1.8	28	19
7	23	8	19	1.8	20	18
8	25	9	17	1.8	25	21
9	23	7	18	1.8	19	20
10	32	12	24	1.6	35	28
11	24	7	20	1.8	26	20
12	28	9	24	1.7	34	23
13	24	9	20	1.8	24	20
14	27	12	16	1.8	23	20
15	25	10	17	1.8	25	19
16	25	10	17	1.8	25	19
17	24	8	19	1.8	23	19
18	25	9	18	1.8	24	20
19	24	9	18	1.8	25	20
20	24	7	20	1.8	27	20
21	23	8	18	1.8	23	20
22	24	8	18	1.8	27	19
23	30	14	19	1.8	22	21
24	28	13	18	1.8	27	21
25	29	11	16	1.8	31	23

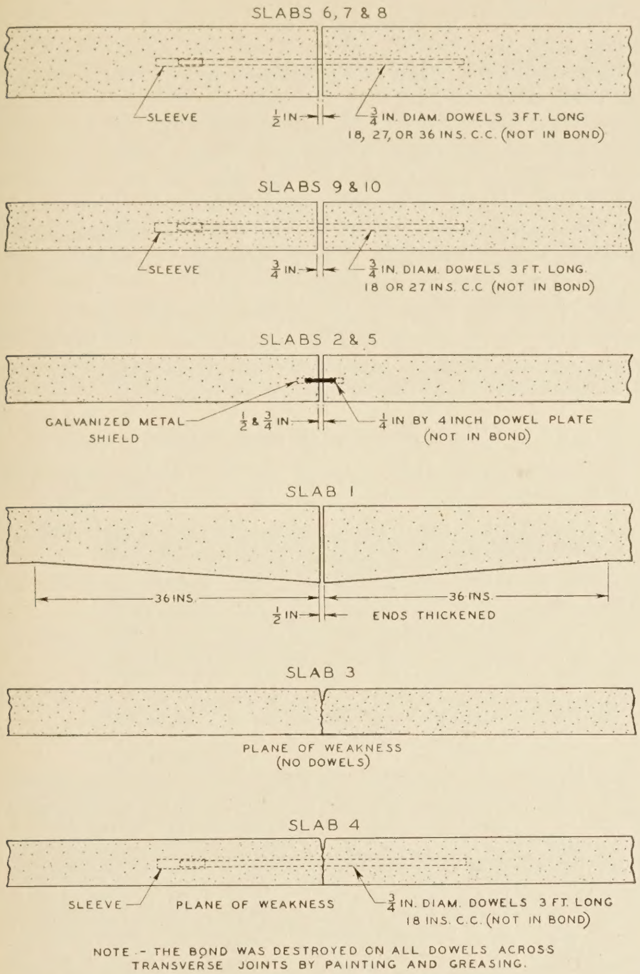

FIGURE 2.—DESIGNS OF TRANSVERSE JOINTS INCLUDED IN THE INVESTIGATION.

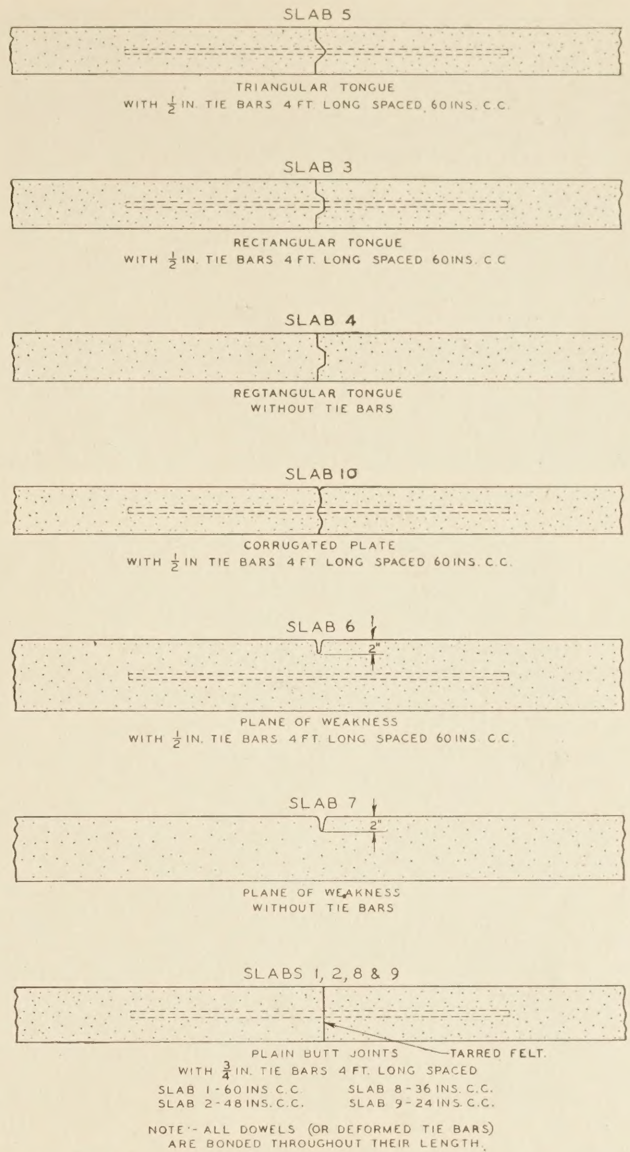

FIGURE 3.—DESIGNS OF LONGITUDINAL JOINTS INCLUDED IN THE INVESTIGATION.

The subgrade where the slabs were to be located was next plowed to a depth of approximately 10 inches. It was left in this loose condition for a period of about 4 weeks, during which it was broken up and agitated several times with a disk harrow. The soil was finally compacted, first with a 5-ton tandem roller and then with the wheels of a loaded 5-ton motor truck. The appearance of the subgrade after this manipulation is shown in figure 4. On this compacted soil the forms were set and the final grading completed. Because of the purpose for which the slabs were to be used, great care was taken to have the final subgrade surface exactly to grade and very smooth in order that the thickness of the completed slab would be known definitely. The appearance of the subgrade at the time the concrete was placed is shown in figure 5.

The moisture content of the subgrade was maintained by sprinkling daily and the particular portion on which concrete was to be placed was given an additional light sprinkling immediately prior to placing concrete.

The concrete.—The materials used for the concrete were carefully selected and the mix designed to give high flexural strength. The cement was a standard portland cement of satisfactory quality, and all came from one bin at the plant.

The fine aggregate was a rather coarse, angular quartz sand, containing some grains of chert, feldspar, gneiss and mica. The average fineness modulus of the sand as determined by a number of tests was 3.26. The source of this material is near Fredericksburg, Va.

The coarse aggregate was a blue limestone obtained from near Martinsburg, W. Va. It was shipped to the job in three sizes and recombined at the proportioning plant to give the desired grading. The proportions used were:

Size:	*Percent*
1¼ to 2¼ inches	50
¾ to 1¼ inches	25
¼ to ¾ inch	25

When combined in this way the average fineness modulus of the coarse aggregate was 7.65.

The proportions fixed for the concrete were 1:2:3½, using dry-rodded volume as the basis of measurement. Actually, in batching materials for the mixer, these proportions were controlled by weighing all of the constituents except the water. Figure 6 shows the

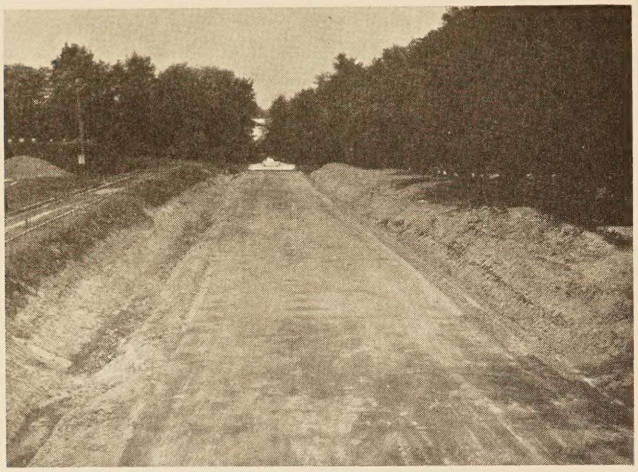

Figure 4.—Appearance of the Subgrade After Rolling Had Been Completed.

proportioning plant used in the construction of the slabs.

Moisture determinations were made on samples from the stock piles each morning and necessary adjustments were made in the batch weights and water content. The water-cement ratio decided upon as a result of trial mixes was 0.85 by volume.

Concrete was mixed for 1½ minutes in a modern paving mixer (size 27–E). At the beginning of each day's run, a preliminary half-size batch was run through the mixer and discarded, the purpose being to coat the interior of the mixer drum and obtain uniformity in subsequent batches. The concrete was dumped on the subgrade and distributed in the usual manner. Compacting and finishing were accomplished with a 2-screed finishing machine, without tamping. The final finish was obtained with a hand belt and edging tools. A double layer of wet burlap was applied immediately after the final belting and this was kept wet for 24 hours, after which it was replaced with a layer of earth 3 to 4 inches thick, which was kept wet for 20 days and then removed. Figure 7 shows the equipment used in mixing and placing the concrete.

In order to have concrete available for such later studies of the physical properties of the concrete as might be necessary, three short extra sections of pavement were cast during the construction of the test slabs.

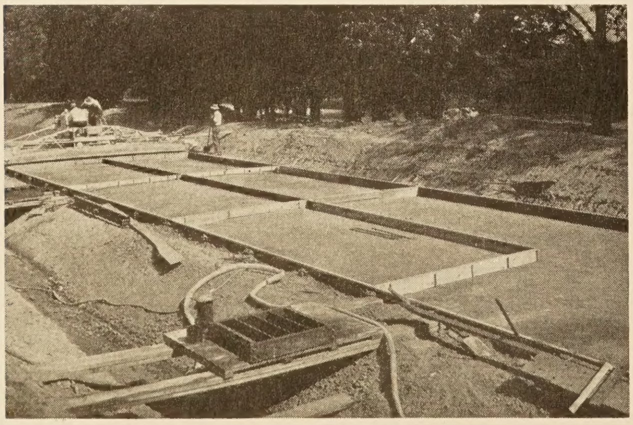

Figure 5.—Appearance of the Subgrade After Form Setting and Completion of the Final Grading.

For an early determination of concrete strength, 8 beams and 5 cylinders were made for each of the 10 pavement slabs. These specimens were cast from the concrete after it had been dumped on the subgrade. The beams were 7 by 7 by 30 inches and the cylinders 6 by 12 inches in size. These specimens were protected from moisture loss during the first 24 hours, after which the cylinders were removed to the damp room and the beams were buried in the earth shoulder beside the slab. All of these specimens were tested at the age of 28 days.

The average flexural strength of the beams (80 specimens) was 765 pounds per square inch and the average compressive strength of the cylinders (46 specimens) was 3,525 pounds per square inch.

TEST PROCEDURE DESCRIBED

The tests and observations made in this investigation may be divided into three groups, as follows:

1. Load tests on the pavement slabs, in which definite loadings were applied to the various sections according to a plan and the resulting deflections and strains were measured. These tests form the basis of—

(*a*) The examination of the Westergaard analysis.

(*b*) The study of the pavement cross-sections.

(*c*) The determination of the structural efficiency of the different joint designs.

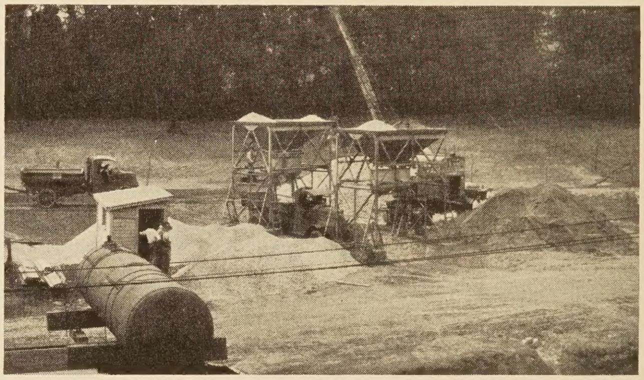

Figure 6.—The Proportioning Plant.

2. Observations made on the slabs to determine the effects of variations in temperature and moisture conditions on their size, shape, and load-carrying ability. These observations included the determination of—

(*a*) Temperature conditions within and surrounding the pavement.

(*b*) The expansion, contraction, and warping of slabs due to temperature changes and to changes in moisture condition.

(*c*) The strains induced in the concrete through the tendency of the slab to change its size and shape.

3. *Auxiliary tests.*—This group comprises a considerable number of collateral investigations carried out principally in the laboratory, to develop information essential to the interpretation of the data obtained in the tests on the slabs. In this group will be found tests to determine—

(*a*) The physical characteristics of the subgrade.

(*b*) The physical properties of the concrete.

(*c*) The thermal properties of the concrete.

(*d*) The effect of moisture conditions on the strength and stiffness of the concrete.

PROGRAM OF LOAD TESTS BASED ON CAREFUL PRELIMINARY INVESTIGATIONS

Before beginning the general program of load tests, preliminary tests were made on a slab of uniform thickness and on one having a conventional thickened edge. The purpose of these preliminary tests was to determine—

1. The proper points at which to apply the loads for the various studies.
2. The proper position for the strain gages if the critical strain was to be measured for each loading.
3. The extent of the deflections to be measured for each loading.

The information obtained in these tests made possible the detailed planning of the tests that were to follow.

An important development of this preliminary work was the conclusion that relative deflections, as measured in these tests, may not always be a true indication of relative stresses. Since deflection and stress are directly related theoretically, it seems probable that the deflection data, while apparently accurate, are actually quite crude when compared to the strain data. Thus, differences in elastic curvature that are not detectable in the deflection data may cause large differences in stress.

This conclusion made it necessary to depend almost entirely upon the stress data as a basis for the comparisons that it was desired to make.

As soon as the preliminary tests had been completed detailed plans for the load-testing program were developed. Figure 8 shows the plan and elevation of one of the test sections and the points where loads were applied for the different studies and the positions of the strain gages in relation to the load points. Figure 9 is a similar drawing showing the points where loads were applied and the lines along which the deflection curves were determined. This figure also shows the location of the points where the opening and closing of the joints due to temperature were measured.

In the studies of the balance of the designs of slab cross-section the load was applied successively at points 1 foot apart along the transverse axis of one of the quarter-slab panels (points 1 to 10, inclusive, along the line A′—B′ in fig. 8).

For the investigations of joint design, loads were applied at the joint edge, at the center of the slab panel, and at the free edge of each slab, thus permitting a comparison to be made between the maximum stresses developed by a given load acting at the joint edge and those developed by the same load at the other two points, points that represent the extreme limits of slab continuity. These stress data make it possible to set up a rational measure of the structural effectiveness of joint designs.

In the case of the longitudinal joints the load points A, H, and B were used, while for transverse joints the loads were applied at I, H, and G. As the program advanced it was found that some joint designs were not equally effective at all points nor at all times of the year, so that additional loads were applied along lines parallel to the slab axes but displaced from the center line. In all cases, however, the complete data were obtained for the three positions of the load.

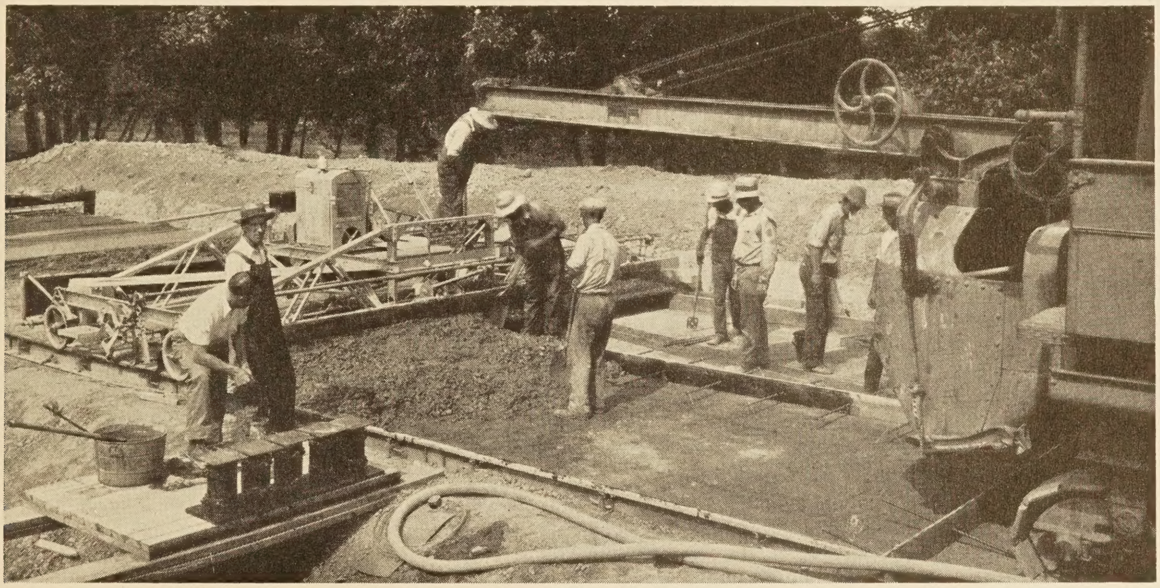

Figure 7.—The Mixing and Placing of the Concrete.

When Westergaard prepared his analysis of the stresses in a concrete pavement slab, he developed a mathematical treatment covering three important cases of loading that were: Case I, a wheel load acting at the free corner of a slab; case II, a wheel load acting at the interior of a slab and at a considerable distance from the edges; case III, a wheel load acting at the edge of a slab and at a considerable distance from a corner.

The loadings applied to the test slabs in the study of the three cases of the Westergaard analysis are shown in figure 8. These tests were made only on the slabs of uniform thickness. For every loading the significant stress and deflection data were obtained. Figure 10 shows the appearance of one of the test sections after the installation of the gage points for the strain and deflection measurements.

METHOD OF APPLYING LOADS DESCRIBED

It was considered desirable to use loads that would create maximum stresses of approximately one-half of the modulus of rupture of the concrete. Since some of the slabs were of rather heavy design, reactions of some

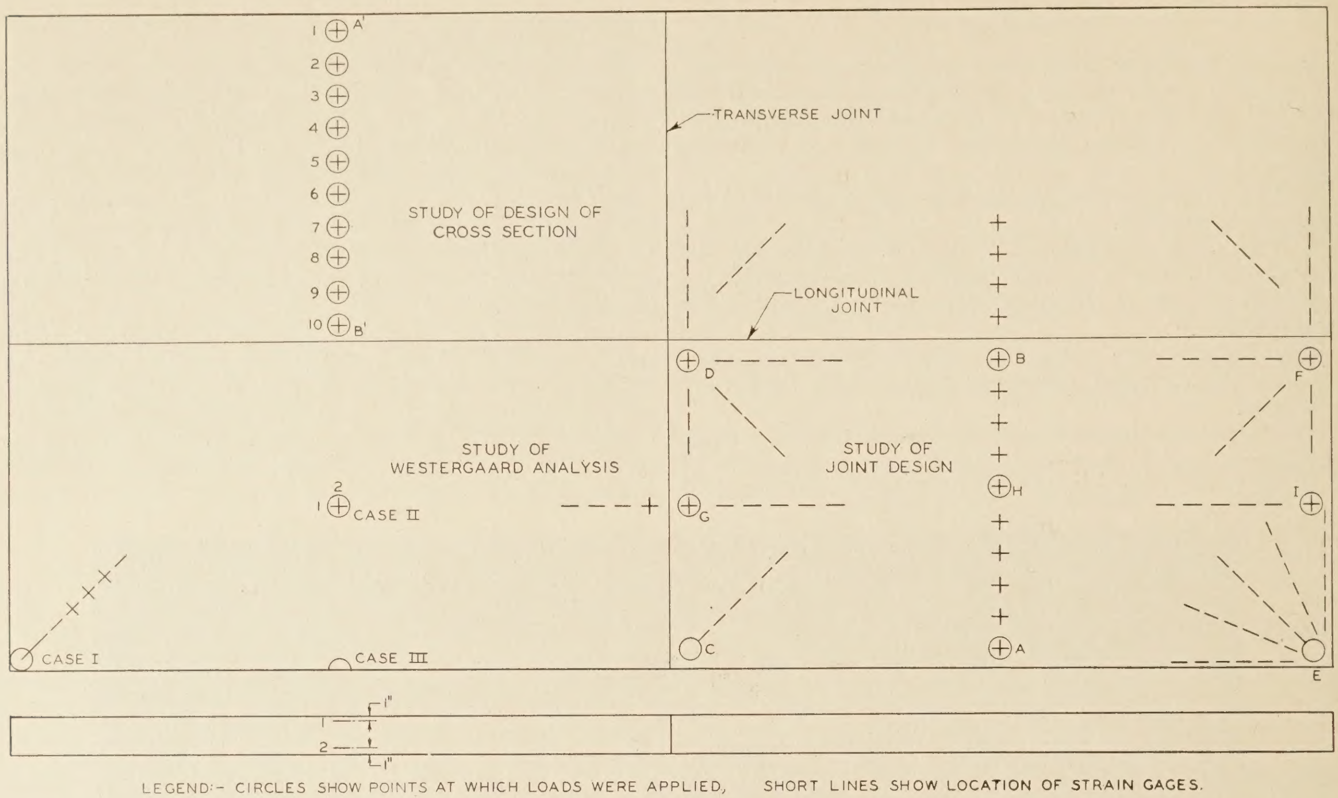

Figure 8.—Plan and Elevation of a 20- by 40-Foot Test Section, Showing the Points Where Loads Were Applied and the Locations of the Strain Gages.

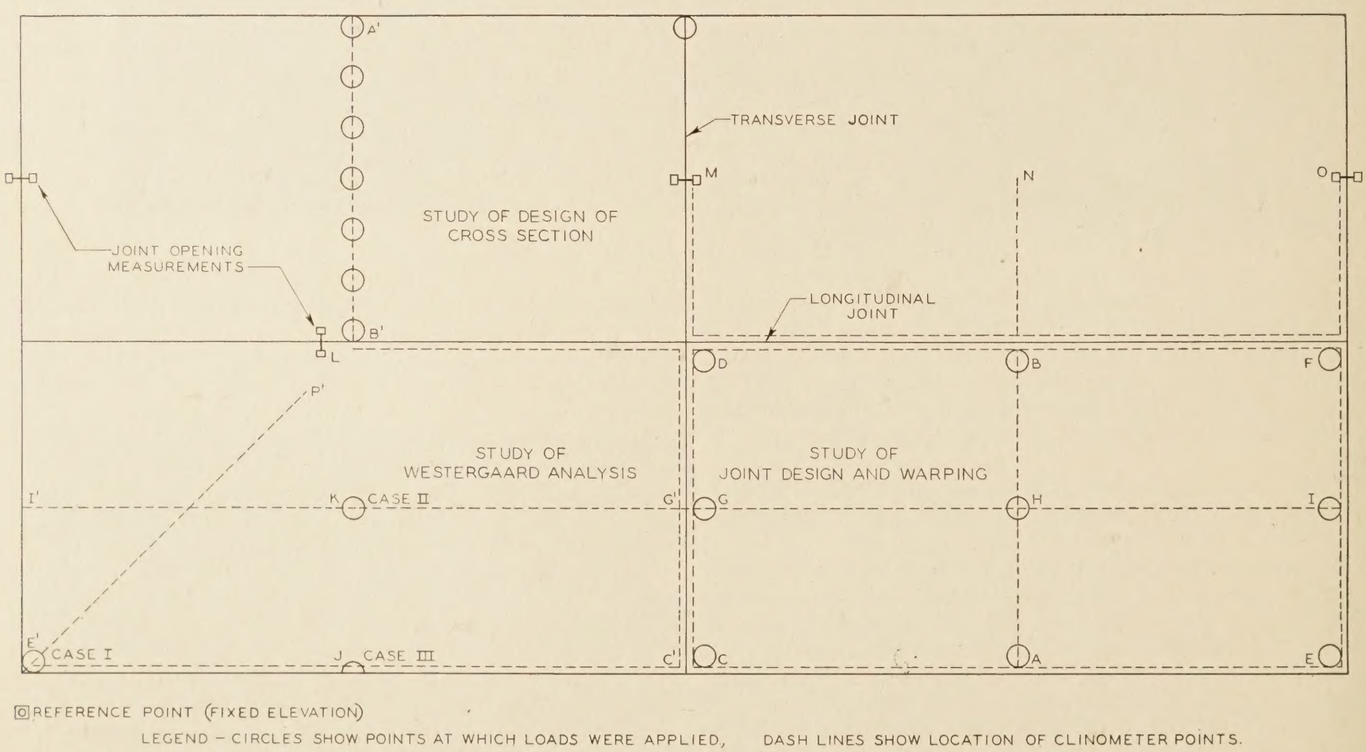

Figure 9.—Plan of a 20- by 40-Foot Test Section Showing the Points Where Loads Were Applied and the Lines Along Which Deflections Were Measured.

magnitude were necessary to produce such stresses. It was also highly desirable that at the time the test load was being applied no other loads be on the slab, in order that the observed effects could be attributed definitely to a known reaction system. These two considerations and the availability of a large cylindrical steel tank led to the adoption of the loading equipment shown in figure 11. The tank, 30 feet long and 6 feet in diameter, was mounted in a structural steel frame or cradle, supported by two transverse end frames 22 feet apart. Each end frame was provided with a pair of heavy cast-iron wheels of small diameter and these rested upon a railway laid along the earth shoulders parallel to the pavement edge. The tank spanned the slab completely and could be moved longitudinally over the test sections at will.

A heavy wooden bolster or pad was fitted to the lower surface of the tank and so arranged that it could be shifted to any position from one end frame to the other, and thus be placed over any desired point on any of the sections.

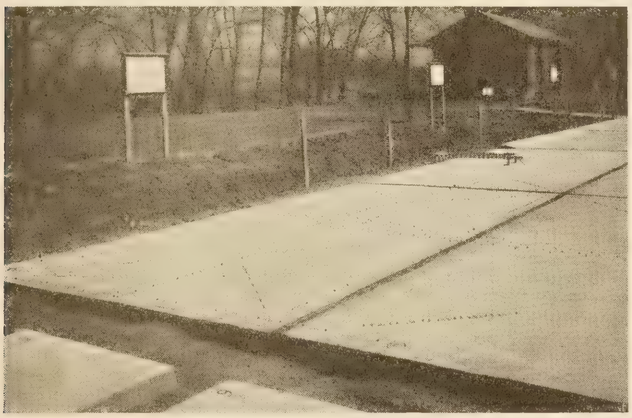

Figure 10.—Complete Installation of Gage Points for Strain and Deflection Measurements on One of the Test Sections.

By partially filling the tank with water a reaction in excess of any load required for the loading of the slabs became available. To develop the load on the slab the device shown in figure 12 was constructed. In this figure, A is the wooden bolster that bears against the bottom of the tank, B is a steel facing plate on the lower surface of the bolster, C is a hardened steel knife edge, D is a pair of heat-treated steel beams whose load-deflection rate being known through calibration enables the operator to determine the load on them at any instant by reading the micrometer dial that measures their deflection, E is a ball-bearing screw jack used for developing the thrust, F is a spherical bearing block that prevents eccentricity of loading on the bearing plate G, and H is a sponge-rubber pad to take up surface irregularities on the slab and assure a uniform intensity of load over the entire area of the bearing block.

The capacity of the loading device shown in this figure is approximately 25,000 pounds. One division on the micrometer dial is equivalent to a load increment of about 30 pounds and periodic calibrations have led to the conclusion that the load measurement by this means can be depended upon to be accurate within 100 pounds, which makes the percentage of error small for loads of the magnitudes used in these tests.

The bearing blocks that received the thrust of the jack and applied it to the pavement were of two types and of several sizes. For the study of the Westergaard

Figure 11.—Loading Equipment in Place Over One of the Test Sections. The Slab is Covered with Straw and Shaded to Prevent Warping.

theory it was necessary to use blocks having both circular (fig. 13 A) and semicircular bearing areas (fig. 13 B), circular for the interior and corner loadings and semicircular for the edge, in order to meet the assumptions of the analysis. Also it was necessary to use several sizes of each in order to investigate the effect of the size of the bearing area on the maximum stress caused by a given load. The diameters of the blocks selected were 6, 8, 12, 16, and 20 inches. The majority of the tests were made with the 8-inch diameter circular block. For the corner loadings the full circular plates were used. When the larger plates were used, distribution of the load was obtained by pyramiding the plates as shown at the right hand side of figure 13 A and also in figure 12.

In a number of the tests, such as those at the interior of the pavement slab, it was necessary to measure the strain in the concrete directly under the bearing plate. For these tests special blocks, provided with a groove across the bottom face large enough to accommodate a

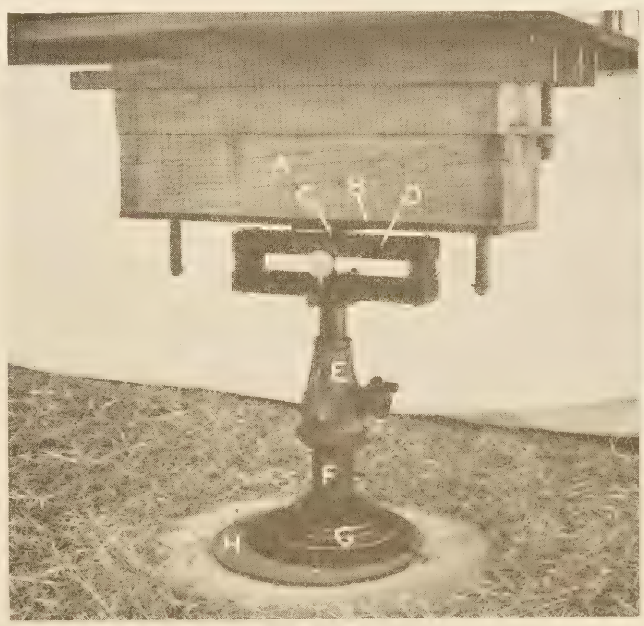

Figure 12.—Apparatus for Applying the Load and for Measuring Its Magnitude.

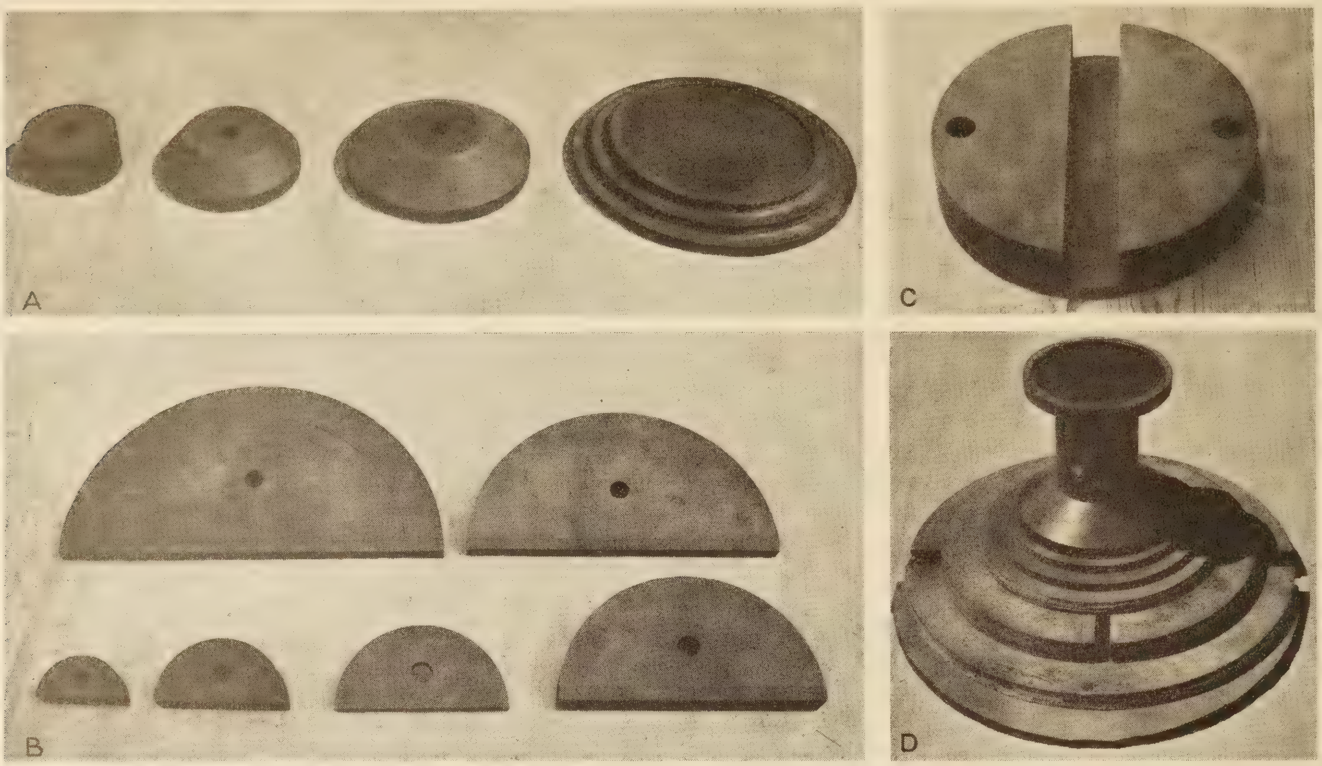

Figure 13.—Bearing Plates: A, Circular Bearing Plates Used in the Corner Loading Tests; B, Semicircular Plates Used for the Edge Loading Tests; C, Small Circular Bearing Block Grooved to Provide Space for a Strain Gage. The Block Is Shown Inverted. D, Large Circular Bearing Block With Space for a Strain Gage in the Lower Plate.

strain gage, were used. The smaller blocks of this type were built as shown in figure 13 C. For the larger areas two segments of the proper size were placed on the sponge-rubber pads on either side of the strain gage and the load was distributed to these by superimposed circular plates as shown in figure 13 D.

Both circular and semicircular plates were used as bearing blocks for the tests at the edge of the pavement slab. The circular plates were the same ones used in the corner loading tests and the semicircular plates were those shown in figure 13 B.

All of the bearing blocks were made of steel and were so designed that the deflection under load produced a negligible effect on the uniformity of load distribution. The effect of the groove in the bearing block on the maximum stress in the slab was investigated and the tests showed that for a given load the grooved block caused the same maximum strain as a block of the same diameter without the groove.

A study was made of the effect of load duration on the magnitude of the strain developed in the concrete. It was found that in some positions, on some of the pavement designs, essentially the maximum strain was developed after the load had been maintained for 1 or 2 minutes, while at other points, 4 or 5 minutes was necessary before this equilibrium was established. As a result of this study the procedure of maintaining the load for 5 minutes before making any strain measurement was adopted for all of the tests. Conversely, 5 minutes was allowed for recovery after the release of each load before the application of the next load. The strains reported in the papers that are to follow are, therefore, maximum strains for the particular loads and are all definitely larger than would be caused by momentary loads of the same magnitude.

ACCURATE MEASUREMENTS OF STRAIN OBTAINED WITH SPECIAL GAGES

Throughout the investigation the strains in the concrete were measured with the recording strain gage shown in figure 14. The gage and its characteristics have been described in detail elsewhere [4] and will be dealt with only briefly here. It consists of a body or frame about 6 inches in length carrying a simple bell crank lever with arms of unequal length. The short arm of this lever is moved by any displacement of the gage points between which the gage is mounted. This motion is transmitted to the long arm of the lever and of course magnified by the ratio of the lengths of the two arms. The long arm of the bell crank carries a stylus point at its free end which makes a trace on the smoked surface of a small glass plate, thus recording a displacement of the end of the arm. The trace on the record slide is thus proportional to the displacement of the gage points and its length is measured, either directly with a comparator or by optical magnification in a projection apparatus. The mechanical magnification in the gage is about 60:1 and ordinarily another magnification of about 30:1 is had in the projection apparatus.

The gages were designed to eliminate ordinary temperature effects. The gage body from tip to tip is made of the alloy "invar", and further compensation is obtained through the use of a pair of dissimilar metals in the long (or stylus) arm of the bell-crank lever.

The accuracy of the gages is sufficient to permit the determination of stress in concrete to within 20 or 25 pounds per square inch, where dependence is placed upon a single observation.

[4] An Improved Recording Strain Gage, by L. W. Teller, Public Roads, vol. 14, no. 10, December 1933.

The strain gage is approximately 6 inches in length. Early in the consideration of the program the question was raised as to whether or not a gage of this length would record maximum strains when used under bearing blocks of the sizes that it was desired to use. This matter was investigated rather thoroughly by using special gages of various lengths placed under bearing blocks of a range of sizes, and the data obtained indicated quite conclusively that the gages would record the maximum strain, provided that the entire gage and gage points were within the circumference of the bearing plate and that the axis of the gage lay along one of the diameters of the plate. Theoretically, the stress is not exactly uniform across the area of the slab under the bearing plate, and it is probable that, had it been possible to measure strains with greater precision, the variation due to length of gage would have been detected. With the apparatus described, however, the same maximum unit strain was indicated by at least 3 different lengths of gage under bearing plates of several sizes, so long as the gages were placed in accordance with the 2 provisions mentioned above. As a result of these tests, it was concluded that the

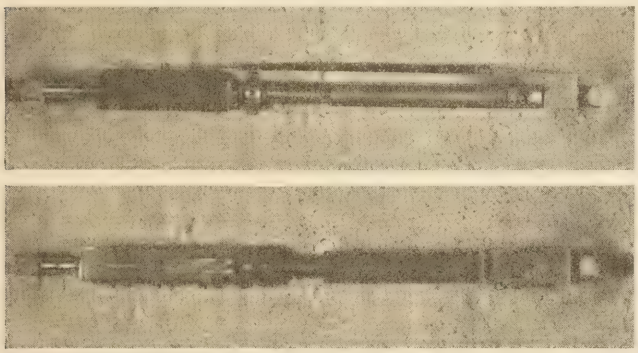

Figure 14.—Recording Strain Gage of the Type Used in This Investigation Mounted Between Two Gage Points.

6-inch gage would record approximately the maximum strain if used with bearing plates with a diameter of 6 inches or more.

In use the gages were installed between two small brass posts containing drilled and reamed gage holes. These posts, ¼ by ¼ by 1½ inches in size, were set into small holes drilled in the surface of the concrete to a depth of about 1 inch immediately before each test, being held in place with plaster of paris. Various other cementing materials were tried, but it was found that with time the posts tended to work loose with all of them and that, for a temporary setting, plaster of paris was as satisfactory as any of them and considerably more convenient to use.

As usually installed the axis of the strain gage was one-fourth of an inch from the surface of the concrete. This caused the recorded strain to be greater than the strain at the surface of the pavement by an amount that depended upon the relative distances of the gage and that surface from the neutral plane of the pavement slab. In most of the measurements, it was therefore necessary to apply a small correction to the observed strains in order to compensate for the gage position. Figure 15 shows an installation of the gages for a load test at the corner of a slab of constant thickness.

DEFLECTIONS MEASURED WITH CLINOMETERS

The deflection measurements in this investigation were made with the clinometer or "level-bar" shown in

Figure 15.—Arrangement of Strain Gages for a Load Test at the Corner of a Slab of Uniform Thickness.

figure 16. This instrument was built especially for the project from a design developed by the Bureau in connection with a recent highway bridge research,[5] the principle of the instrument being the same as that of the clinometer loaned by the American Society of Civil Engineers for the tests of the Yadkin River Bridge.[6]

It consists of a rigid, horizontal steel frame carrying a very sensitive spirit level in its upper face and supported by a vertical leg at each end. One of these legs is of fixed length while the length of the other is adjustable by means of a fine pitch screw operated by a knurled hand nut at the top of the instrument. The amount of adjustment made with this nut is indicated in thousandths of an inch by a micrometer dial on the front of the frame. In order that the position assumed by the instrument when it is placed on the clinometer points shall always be the same, a third or steadying leg is provided, projecting at right angles from the center of the frame and turning down at the outer end where it terminates in an adjustable foot.

Small brass cylinders were grouted into holes drilled in the pavement surface at 10-inch intervals along the

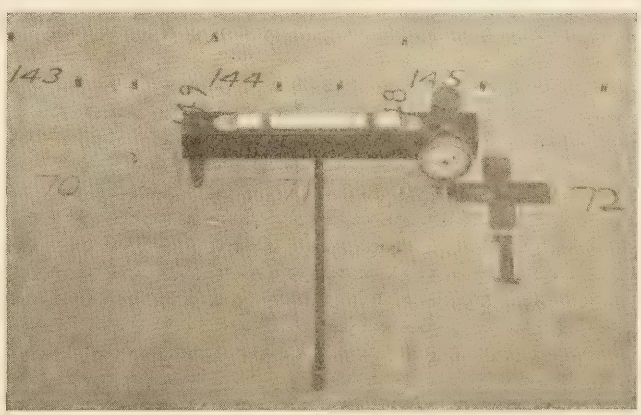

Figure 16.—Special 10-Inch Clinometer Used for Measuring Slab Deflections. The Instrument Is Shown Resting on Small Brass Clinometer Points Set Into the Surface of the Pavement.

lines of desired deflection measurements. The upper or exposed face of each of these contained a small verti-

[5] Application of the Freyssinet Method of Arch Construction to the Rogue River Bridge in Oregon, by Albin L. Gemeny and Conde B. McCullough. Technical Bulletin No. 2 of the Oregon State Highway Commission, Salem, Oreg.
[6] Loading Tests on a Reinforced Concrete Arch, reported by Albin L. Gemeny and W. F. Hunter, Public Roads, vol. 9, no. 10, December 1928.

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cally drilled hole and also a narrow horizontal groove or slot with beveled edges. The direction of the slot was made parallel to the long axis of the clinometer frame when the instrument rested on the points. The lower ends of the 2 main clinometer legs are sharp-pointed cones and in setting up the instrument 1 of the legs is set in the drilled hole in the top of 1 of the clinometer points and the other leg is placed in the slot in the adjacent point. Any expansion or contraction of either the instrument or the concrete causes only a very slight horizontal displacement of the leg that rests in the slot and this movement produces no error in the measurement being made.

After the legs of the clinometer are properly set in the gage points the instrument is carefully adjusted to a level position by rotating the knurled hand nut. When level, the micrometer dial is read. The clinometer is then moved 10 inches to the next gage point and the operation repeated. Any deflection of the slab due to load or to warping will change the relative elevation of the clinometer points and this change will be measured by the difference in the adjustments necessary to level the clinometer as indicated by differences in the readings of the micrometer dial before and after the deflection occurred. The operation is simply one of precise leveling along the line of installed points. While the micrometer dial reads directly in thousandths of an inch it was found practicable to estimate ten-thousandths. The design of the adjusting mechanism is such that thread wear and backlash cannot introduce an error in successive measurements.

Benchmarks or reference points completely independent of the pavement were used to fix the datum for the pavement surfaces.

TEMPERATURES MEASURED WITH THERMOCOUPLES

In practically all of the load tests it was necessary to reduce the influence of slab warping to a minimum. It was found that if the slab was kept shaded from all direct sunlight and covered with several inches of dry straw, the temperature differential between the upper and lower surfaces became negligible and the warping of the slab was so small that its influence on stress was not important. Therefore, these precautions were taken in all tests as a matter of regular procedure. The shade and straw covering are shown in figure 11.

Observations to determine the effects of the temperature and moisture conditions within and surrounding the test sections were started soon after the pavement was laid and have been continued to the present time. These observations included extensive temperature measurements, moisture determinations, measurements of the changes in size and shape of the slabs resulting from temperature and moisture variations, and measurements of the strains caused by these variations in various parts of the slab structure.

When the test sections were built a number of resistance coil thermometers were placed in the slabs at selected points to furnish the temperature data then thought necessary. The original installations proved to be inadequate in extent and several of the resistance coils ceased to function for some reason that could not be determined. It was found also that the coils used had a time lag in their operation that was very undesirable for the work to be done.

It became necessary to make other provision for measuring the temperatures in the concrete. The plan adopted was to build two small slabs of concrete of the same materials and proportions as were used in the test sections and to install in these copper-constantan thermocouples for temperature determination. These slabs were each 4 feet square and one was 6 and the other 9 inches in depth. The thermocouples were installed in the center of the slab area. Before placing the concrete two thermocouples were placed in the subgrade under each small slab at depths of 2 inches and ⅛ inch respectively and, as the concrete was being placed, additional thermocouples were placed at 1-inch intervals from the bottom of each slab to the top. With this installation it was possible to determine not only the differential existing between the upper and lower surfaces but also the complete temperature gradient from one to the other.

Thermocouples were also placed at the top and bottom surfaces of the four constant-thickness slabs.

The "average" temperature of the pavement slabs as used in connection with the expansion and contraction measurements was developed from the data obtained with the thermocouple installations in the small slabs. A pavement slab having a thickness of 6 inches or 9 inches was assumed to have an "average" temperature equal to the mean temperature of the small slab of the same thickness and the "average" temperature of sections having a thickness between 6 and 9 inches was obtained by interpolation, assuming a straight-line variation between the mean temperatures of the 6-inch and 9-inch slabs.

MICROMETERS USED TO MEASURE CHANGES IN LENGTH

Measurements were made to determine the extent of both the daily cycle and the annual cycle of dimensional changes in the slabs. These measurements served to show the magnitude of the changes in slab dimensions that were caused by the daily and annual variations in temperature and moisture content, and they also provided a means for determining the relative restraint to expansion and contraction offered by the various joint designs.

To determine the absolute changes in length of the slab sections, the movements of the slab ends with respect to fixed reference points were measured with a micrometer, while the degree of restraint offered by the joint designs was determined by comparing the movement at these joints with that at the free ends of the same slab.

The fixed reference points referred to were installed in concrete posts cast in heavy foundation blocks several feet below the surface of the ground, the posts themselves being completely protected from lateral earth pressure.

Figure 17 shows the 7-inch micrometer built for this purpose, together with the invar reference bar used for a standard of length in these measurements. The guaranteed coefficient of thermal expansion for this material is 0.8×10^{-6} per degree centigrade. Its change in length for air temperature ranges is so small that for the purpose of the tests its length could be considered as being constant throughout the year.

The measurements with this micrometer were made between the tips of conical gage points of stainless steel set horizontally in the upper ends of short steel posts cemented into the slab surface, one on either side of the joint (or slab end) and approximately 7 inches apart. These details can be seen in figure 17.

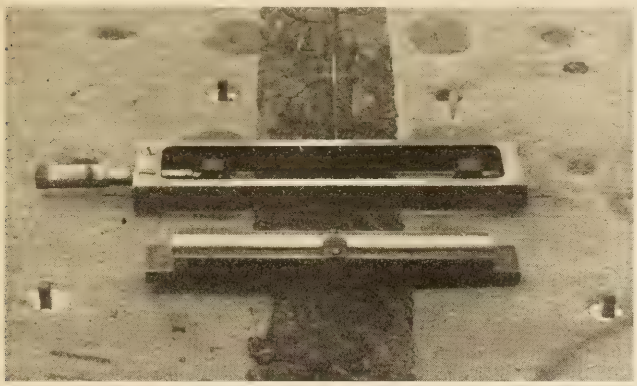

Figure 17.—The Special 7-Inch Micrometer Frame and the Invar Reference Bar Used for Measuring the Expansion and Contraction of the Test Sections.

Some additional data on the length changes occurring in the pavement slabs were obtained with 3 electric telemeters that were embedded in 3 of the slab panels at the time the concrete was placed. These instruments[7] were installed at mid-depth at the center of the longitudinal axis of three of the 10- by 20-foot panels. They were intended to provide data in connection with one of the designs but, because of certain difficulties that will be discussed later, they failed to do so. They did, however, furnish valuable information regarding elongation caused by both temperature and moisture.

ACTION OF SLABS DURING WARPING STUDIED

The magnitude of the temperature warping in the various sections was determined on numerous occasions over a period of about 3 years. Measurements were made to determine the warped shape of an entire 10- by 20-foot panel. The degree of restraint to free warping caused by the different joint designs was studied at selected points by means of measurements of warping over a limited area near the joint involved.

The necessary temperature data for these studies were obtained from the thermocouple installations and the shape of the warped surface was determined by clinometer measurements along the lines of points shown in figure 9. The measurements of warping with the clinometer were referenced to fixed points or bench marks set into the earth shoulders. Because of the time that was necessary to take readings around the entire perimeter of a 10- by 20-foot slab, frequently the shape of the slab changed sufficiently to develop a considerable error of closure. Care had to be taken to make these long series of measurements at a time when the conditions producing the warping were not changing too rapidly.

In the study of warping some attention was given to the strains in the concrete produced by the forces set up by the warping action of the slab itself and also to the relative strains produced in a slab of given design by a given load when the slab was both warped and unwarped. The procedure for the loading and strain measurement involved only one feature that was different from the rest of the strain measurements. To produce warping the straw cover was removed and the pavement was exposed to the direct rays of the sun for a number of the tests.

[7] For a description see Technologic Paper No. 247, U. S. Bureau of Standards, A New Electric Telemeter, by Burton McCollum and O. S. Peters.

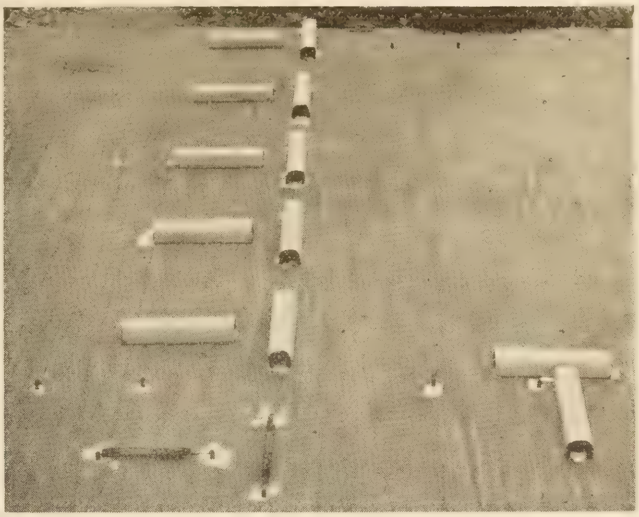

Figure 18.—A Strain Gage Installation for Measuring Strains Caused by Warping.

The very nature of these tests required a wide range of temperature and strain observations that were continuous over the complete warping cycle. It was thought desirable, therefore, to protect the strain gages from direct sunlight with small semicylindrical covers as shown in figure 18. These covers, to be described in a subsequent paper, permitted the free circulation of air around the gages but were so constructed as to resist heat absorption. The purpose of this protection was to keep the gages at the same temperature as the concrete in the pavement.

DIFFICULTY ENCOUNTERED IN MEASURING PRESSURE OF SLABS AGAINST THE SUBGRADE

A group of nine soil pressure cells was placed beneath the 6-inch and 9-inch slabs of constant thickness, arranged in the pattern of a 90° cross. These cells were installed in the subgrade with their diaphragm side down in carefully scraped recesses so that perfect bearing was obtained. The recesses were sufficiently deep to cause the back of the cell to be flush with the general level of the subgrade. The concrete of the slab was placed on these cells but no anchorage was provided to fix the cells to the concrete. The purpose of these installations was to obtain data on the distribution of a load to the subgrade by the two thicknesses of slab. Unfortunately, these data were never obtained as the cells under both slabs failed to record pressure before the load tests were made.

For a short time after the construction of the sections the cells operated and such pressure data as were obtained during this period indicated the normal fluctuations as the slab warped during the day. In the course of a few weeks the cells ceased to record pressure, indicating that a separation between the bottom of the slab and the back of the cells had occurred. Whether this was due to a settlement of the cells or a swelling of the subgrade that raised the slab more than it did the cells is not known. The cells were not embedded because it was desired to maintain the full flexural strength of the slab. Perhaps some anchorage attachment on the backs of the cells that would have held them to the slab without reducing the slab strength would have made a better installation, but this is by no means certain.

AUXILIARY TESTS MADE

A consideration, either theoretical or experimental, of the structural action of a concrete pavement slab lying on an earth subgrade, necessitates either assumptions regarding or a knowledge of the physical properties of the concrete and of the subgrade. Obviously, definite data developed by tests are to be preferred to any assumptions that may be made.

The auxiliary tests made in connection with this investigation were planned to develop information concerning:

(*a*) The concrete—
  1. Strength in compression and in flexure.
  2. Stress-strain relation in compression and in flexure.
  3. Effect of moisture content on the strength and elastic properties.
  4. Thermal properties.

(*b*) The subgrade—
  1. Resistance offered by the subgrade to horizontal slab movement.
  2. Resistance offered by the subgrade to vertical slab movement (or deflection) including an attempt to evaluate the support offered to the slab by the subgrade under the test sections.

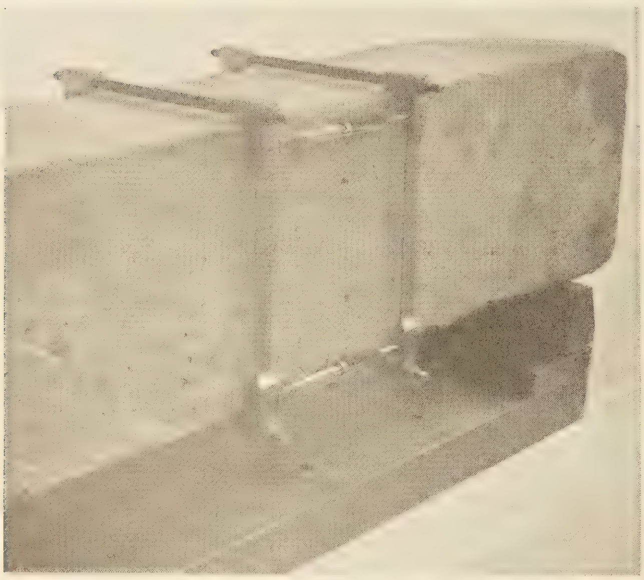

FIGURE 19.—APPARATUS USED FOR DETERMINING THE STRESS-STRAIN RELATION FOR CONCRETE IN FLEXURE.

Although the auxiliary tests are grouped here in the description of test procedure, most of the tests were carried out as separate investigations and the discussions of the data obtained will appear in the particular parts of the report with which they are concerned. In one or two cases these collateral investigations proved to be sufficiently comprehensive and general to warrant a more detailed presentation elsewhere and, in those cases, only the facts that have a direct bearing on the major research will be included in this report, leaving the detailed description of what was done to a separate report.

STRESS-STRAIN RELATION AND COEFFICIENT OF EXPANSION OF CONCRETE DETERMINED

The strength tests of the concrete were made to determine the ultimate strength in compression and in flexure so that safe working stress limits might be fixed.

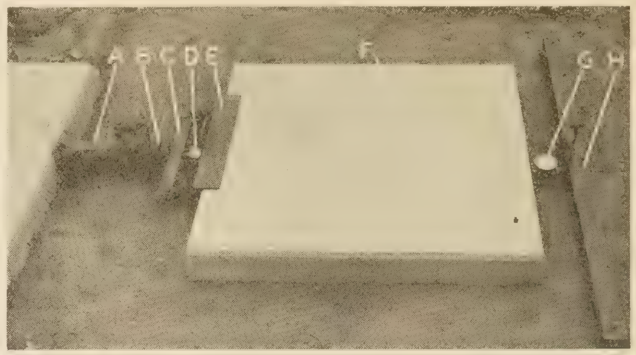

FIGURE 20.—APPARATUS USED FOR DETERMINING THE RESISTANCE OF THE SUBGRADE TO HORIZONTAL MOVEMENT OF THE PAVEMENT.

The procedure followed in making these tests was simply that of good testing practice and included no unusual features. It will not be described therefore. The data obtained have already been given on page 148.

The determination of the stress-strain relations in compression and in flexure was a matter of considerable importance because of the direct application of the data to the analysis of the slab tests. Effort was made to have the tests comprehensive as to scope and precise as to technique to develop thoroughly reliable data. The stress-strain relation in compression was determined from tests on cores, using an extensometer of the Martens' type.[8] These cores were drilled from the small sections provided for the purpose and the program included tests on both wet and dry specimens.

For the determination of the stress-strain relation in flexure on the sawed beams, use was made of equipment designed in the bureau for this particular purpose and shown in figure 19. This apparatus consists of two frames that are clamped around the flexure specimen either side of the midspan and far enough apart to permit the installation of a recording strain gage near the top and bottom on each side of the beam. Each frame makes contact with the specimen at two points on each side of the beam. These points are small chisel-edged studs projecting from the inside of the frame directly opposite the points where the ends of the strain gages make contact with the frame and are held tightly to the specimen by tightening the transverse tie bolts of the frame. When the specimen is flexed the strain gages record the amount of length change that occurs between the frames on either side of the beam and in both tension and compression. The load was applied at the third points of the span and the deflection of the specimen at mid-span was measured with micrometer dials arranged on either side. The equipment for load application and for deflection measurement was omitted in figure 19 so that the details of the strain-measuring apparatus could be seen to better advantage.

Flexure specimens in both the wet and dry states were included in these tests. Since the concrete in the pavement slabs contained moisture and the moisture content of the concrete was assumed to vary, it seemed desirable to develop any information possible relative to the variations in moisture in the pavement slabs and also as to the effect of moisture condition on the physical properties of concrete similar to that used in making the slabs. The moisture content of the slabs was determined by the rather crude but direct

[8] For a description see Handbook of Testing Materials, by Prof. Adolf Martens, 1st ed. 1899 (authorized translation by Gus C. Henning).

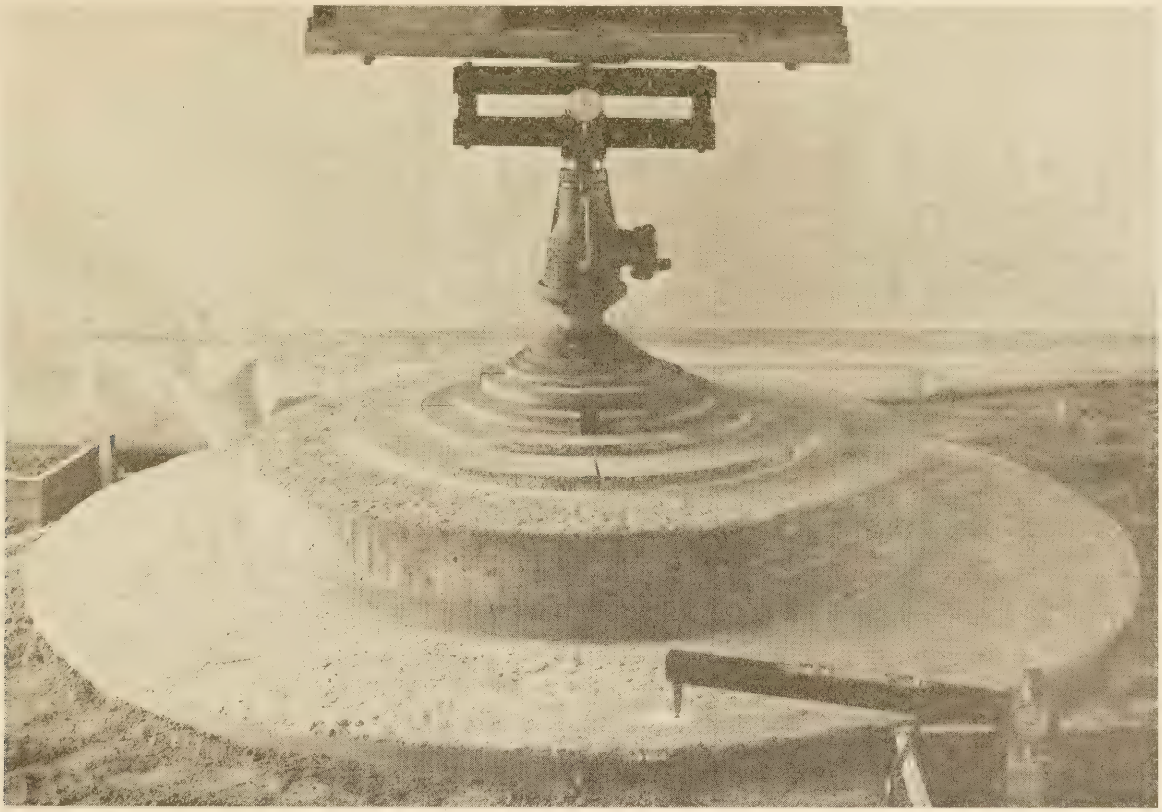

FIGURE 21.—APPARATUS USED FOR DETERMINING THE RESISTANCE OF THE SUBGRADE TO VERTICAL DISPLACEMENT.

method of weighing and drying fragments of concrete broken from the short sections of pavement provided for test specimens of all kinds. The moisture content of the test specimens was also determined by weighing.

The coefficient of thermal expansion of the concrete was determined by a method developed for these tests. Concrete of the same materials and proportions as were used in the pavement sections was placed in a cylindrical mold 12 inches in diameter and 24 inches high and made of a very light-gage sheet copper. This mold was in reality a large can with a watertight bottom and top. An electric telemeter was installed at the midpoint of the longitudinal axis of the cylinder. The concrete was introduced through an opening in the top of the can and as soon as the surface of the concrete was within about half an inch of the top of the mold this opening was sealed off by soldering on a cover plate. This effectively retained within the copper jacket all of the moisture originally in the concrete.

After the concrete had set and the heat of setting had been dissipated, the sealed specimen was placed in an insulated water bath, the temperature of which was placed at different levels within the range of normal air temperatures, and maintained at each until complete temperature equilibrium was established. The unit changes in length accompanying the various changes in temperature were measured with the embedded telemeter. This furnished a simple and apparently satisfactory method for the determination. The mass of concrete was large enough to be representative and the usual difficulties due to moisture changes in the specimen were avoided by using a watertight envelope.

RESISTANCE OF SUBGRADE TO HORIZONTAL AND VERTICAL MOVEMENT STUDIED

The tests made on the subgrade in place were planned with two objects in mind: First, to determine the character of the resistance that the subgrade offered to horizontal movement of the slab; and second, to find out what resistance the subgrade offered to vertical movement of the slab (such as deflection under load) and, if possible, to develop some means for evaluating this resistance in terms that would be applicable to pavement design.

The first tests were made with 4 slabs, each 4 feet square and 6 inches thick, placed on the same subgrade as the large test sections. The general method of test was to move these small slabs horizontally, very slowly, alternately forward and backward by total amounts that equalled approximately the annual cycle of expansion and contraction of the large pavement sections. The thrust necessary to cause horizontal movement and the magnitude of the displacement caused by the thrust were measured from the time that the first detectable movement took place until the total desired displacement had been attained.

Figure 20 shows the apparatus set up for one of these tests. In this figure, A is a jack used to develop the thrust, B is a spherical bearing block that controls the line of the thrust, C is a steel beam whose deflection, as indicated by the micrometer dial D, measures the magnitude of the thrust, E is the frame supporting the steel beam and the dial, F is the small slab, G is the micrometer dial that measures the horizontal displacement of the slab, and H is a rigid member used to support the dial G.

The second group of subgrade tests may be described generally as load-deflection tests on circular bearing plates in intimate contact with the subgrade. The diameters of the plates used were 2, 4, 6, 8, 12, 16, 20, 26, 36, 54, and 84 inches. The two larger sizes were concrete disks cast on the subgrade, the others were steel plates bedded in a ½-inch layer of mortar placed on the subgrade, but separated from it by a layer of waterproofed paper so that the moisture content of the soil would not be altered.

Figure 21 shows the loading equipment set up over the 84-inch bearing plate. It will be observed that the loads were applied on these plates in the same manner as on the pavement sections. For the very small plates a system of dead-load increments was used. Vertical displacements were measured with respect to fixed reference points by means of the clinometer shown in the figure. Generally these measurements were made at three points 120° apart around the periphery of the plates, although in some cases the measurements were made at the midpoint. In general the loads applied were such that a vertical displacement would be produced that approximated in magnitude the observed deflections of the pavement slabs. The deflection of a slab corner, for example, was found to be approximately 0.05 inch, the slab edge 0.02 inch, and the interior of the panel 0.01 inch for a load that did not overstress the concrete.

In making the tests on the subgrade, vertical displacements of 0.005, 0.010, 0.020, 0.035, and 0.050 inch were obtained in nearly all cases. For each displacement value, several loads were applied until a given load produced the same deflection each time it was applied. Each load was applied for 5 minutes and then released for 5 minutes before being applied again. In planning these tests it seemed desirable to arrange the procedure so that the subgrade would be subjected to the same conditions as would obtain under the loaded pavement slab, as nearly as possible. For this reason the rate of loading and the orders of magnitude of subgrade deformation were made to correspond very closely as noted above. The moisture content of the soil was determined before and after each test and protection from sunshine and rainfall was provided.

ROAD-BUILDING LIMEROCKS

BY THE DIVISION OF TESTS, U. S. BUREAU OF PUBLIC ROADS

Reported by R. C. THOREEN, Assistant Highway Engineer

LIMESTONES include those rocks that are composed principally of calcium carbonate with varying amounts of other materials, chiefly silica, magnesia, alumina, and iron oxide. Their physical characteristics vary from the crystalline marbles and consolidated true limestones to the unconsolidated shell marls. This report is concerned with the less consolidated grades of limestone, known locally as "limerocks", that, because of their availability, are used extensively in the construction of road surfaces and base courses in the Southeastern States.

ORIGIN OF LIMEROCKS DISCUSSED[1]

Geologically, the limestones belong to the sedimentary group of rocks, or those that are composed of materials from older rocks that have undergone disintegration and have been redeposited either on land or in water by physical, organic, or chemical agencies. The following, more-detailed discussion of this process deals only with the formation of limestone in water, because the limerocks discussed in this report are chiefly of marine origin.

Lime (CaO) is widely distributed throughout the earth's crust. Because of its chemical activity it is always found in combination with other elements, chiefly as calcium carbonate ($CaCO_3$), in the form of limestone, calcite, and marine and fresh-water shells. It is not readily soluble in pure water, but is soluble in water charged with carbon dioxide (CO_2), resulting in carbonic acid (H_2CO_3).

Rain water becomes carbonic acid by absorbing carbon dioxide from the air or by contact with decaying vegetable matter. This acidulated water percolating through soil and rock materials takes calcium carbonate into solution as calcium bicarbonate $CaH_2(CO_3)_2$. While still in solution the lime is carried by streams and rivers to the ocean where conditions are favorable for deposition.

The deposition of calcium carbonate to form limestone is effected through two main processes: First, the activity of organisms that remove calcium carbonate from solution to form shells; and second, the removal of carbon dioxide by chemical and physical agencies, such as evaporation, aeration, and activity of bacteria and algae, with resultant supersaturation with and precipitation of calcium carbonate.

In an ideal case with both of these processes functioning, the sea floor becomes covered with more or less worn and broken shells and the remains of lime-secreting sea plants, bound together by an amorphous mass of calcium carbonate precipitated by different chemical reactions. If the bed becomes buried under newer sediments due to sinking of the sea floor, the pressure causes further consolidation and partial crystallization. If it becomes exposed by rising of the sea floor, rain water will leach out the calcium carbonate from the upper layers and redeposit and partially crystallize it in the lower layers, thus effecting further consolidation. The degree of consolidation has much to do with the classification of limestones, which differ widely in composition, hardness, texture, and color. The one property common to all limestones is the predominance of calcium carbonate.

With respect to physical appearance, the limerocks of the Southeastern States may be grouped as follows:

1. *Semicrystalline.*—Hard, rather compact limestone, originally soft but recrystallized and consolidated by the action of water. Examples are found in all limerocks that have been exposed to weathering and percolating water.
2. *Fossiliferous.*—Any limestone composed chiefly of fossil shells or other animal remains. All limerocks are fossiliferous in varying degrees.
3. *Shell.*—Limerock composed almost entirely of shells or shell fragments.
4. *Chalky.*—Partly consolidated limestone composed of microscopic shells and shell fragments.
5. *Oolitic.*—Limerock composed of tiny nodules of crystalline calcium carbonate held together by a calcareous cement.
6. *Sandy.*—Limerock into which sand has been incorporated by the action of water currents and waves.
7. *Cherty or flinty.*—Limerock containing nodules or bands of chert or flint. Chert is amorphous silica formed from spicules of sponges and other siliceous matter by the action of ground water.
8. *Marl.*—An indefinite term applied to any soft, earthy mass containing varying quantities of lime, clay, sand, and carbonaceous material. The lime content frequently is in the form of marine or fresh-water shell fragments.

LOCATIONS AND CHARACTERISTICS OF LIMEROCKS DESCRIBED

The limerocks are found in greatest abundance throughout Florida and in the southern portions of Georgia and Alabama. The principal workable deposits in Florida are known as the Ocala, Marianna, Miami Oolite, Tampa, Glendon, and Coquina formations, and also include extensive deposits of undifferentiated marls. The physical characteristics and general locations of these deposits are as follows:

1. *Ocala.*—The Ocala formation consists of a cream-white, soft, porous, granular limestone that bleaches to a chalk-white on exposure (see fig. 1). Some portions are hard and semicrystalline, and in some localities it contains nodules and layers of chert. On the whole it is very uniform in texture and composition, containing as high as 99.6 percent calcium carbonate.

Although the Ocala limestone presumably underlies the whole State to a thickness of more than 400 feet, it is exposed over a comparatively limited area about 50 by 150 miles in extent in the northwestern portion of the peninsula, adjacent to and paralleling the Gulf of Mexico, and in a small area in western Florida just below the Alabama line.

2. *Marianna.*—The Marianna is a very pure, soft, chalk-like limestone, cream-yellow when fresh and chalk-white when bleached by exposure. Recent excavations have revealed that it contains a considerable amount of argillaceous material scattered throughout. The Marianna overlies the Ocala formation to a known thickness of 33 feet. It is exposed over a small area

[1] This discussion of the origin of limerocks and the distribution of the Florida deposits is based on A Preliminary Report on the Limestones and Marls of Florida, by Stuart Mossom, sixteenth annual report, Florida Geological Survey, 1925.

surrounding the town of Marianna in Jackson County, Fla.

3. *Glendon.*—The Glendon limestone consists of minute fossils bound together by amorphous lime material. It is much more compact and not as powdery as the Ocala limestone. The outcroppings of this rock cover a considerable area in western Florida and extend into southeastern Alabama.

4. *Tampa.*—The Tampa limestone varies from a hard, semicrystalline, cream-colored or light-gray rock to a rather soft, amorphous material containing scattered masses of dense, crystalline limestone. The principal deposits are located in the western portion of the peninsula near the city of Tampa, also in the extreme northeastern portion of the peninsula, and to a limited extent in western Florida.

5. *Miami Oolite.*—The typical rock of this formation is a soft, white, pure (95 percent calcium carbonate), oolitic limestone with occasional irregular layers of calcite (see fig. 1). In places rounded grains of white sand are intermixed, sometimes occurring in lenses or pockets. The rock hardens on exposure to air and water. Miami Oolite, known locally as "Ojus" rock, occurs in a rather narrow strip along the coast of southeastern Florida.

6. *Coquina.*—The Coquina limestone occurs in three principal phases. The first is composed entirely of small, clean shells and shell fragments, usually unconsolidated but with hard ledges of firmly cemented shells near the top of the deposit (see fig. 1). The second phase consists of finely crushed shells closely cemented into a hard, compact rock. The third phase contains finely crushed shells and a large quantity of sand grains, cemented together by the calcium carbonate of the shells. In the latter phase it is more a calcareous sandstone than a sandy Coquina. The Coquina deposits are located chiefly in a narrow strip along the eastern coast of the peninsula.

7. *Marls.*—Marl deposits of various compositions are distributed over an extensive area in the lower third of the peninsula and in scattered localities in northern and western Florida.

In Georgia the principal limerock deposits are found in a comparatively narrow strip extending from the southwestern corner of the State in a northeasterly direction to the Atlantic coast. The limerocks in most of this strip are of the Jackson group, of which Ocala limestone is the upper formation, and therefore they are somewhat similar to Ocala limestone in physical characteristics. The limestones in the northern portion of this strip are principally marls that are also distributed in scattered localities throughout the southeastern portion of the State.

Outcroppings of the Ocala, Glendon, and Marianna formations are also found in Alabama. The Glendon limerock appears as an extension of the Florida deposits in the southeastern portion of the State. The Ocala and Marianna formations are found in the extreme southeastern corner of the State as extensions of the Florida deposits, and they also appear in limited areas in the southwestern portion of the State east of the Tombigbee River. As in Florida and Georgia, marl deposits are also distributed in scattered areas throughout the southern portion of the State.

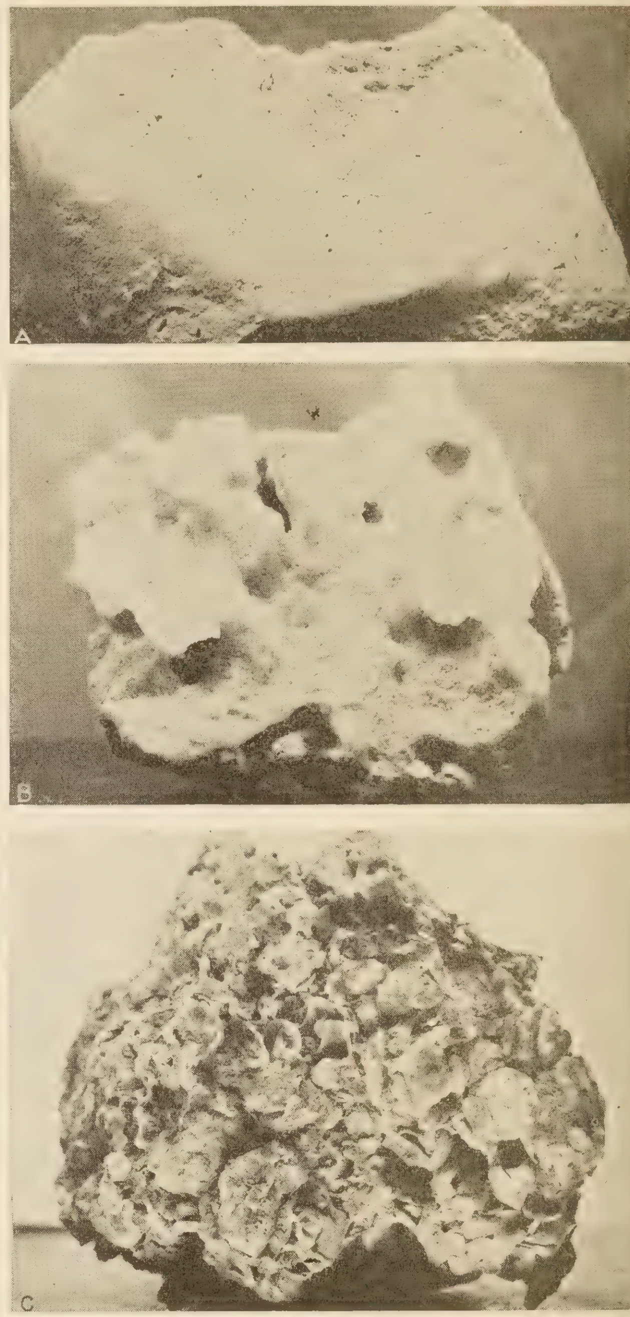

FIGURE 1.—TYPICAL SAMPLES OF LIMEROCKS: A, OCALA LIMEROCK. THE LEFT HALF IS HARD AND SEMICRYSTALLINE, THE RIGHT HALF IS SOFT AND POROUS. B, MIAMI OOLITE; C, SHELL-PHASE COQUINA LIMEROCK.

ROAD CONSTRUCTION METHODS USING LIMEROCK

Mining of the limerocks is all open-pit work after the overburden of soil has been removed by power shovels (see fig. 2). Although the rock is sometimes so soft that it can be scraped from the face of the pit by the shovels, it is usually fractured and loosened by blasting to facilitate handling. It is conveyed to crushers, crushed to the fineness required, and either loaded directly or stored. Although in most of the deposits the limerock is soft and powdery, crushing is usually necessitated by the occurrence of harder masses. Most pits are worked to the depth of the water table. However, Miami Oolite is mined by means of dredges to a depth of as much as 30 feet below the water table.

Some of the harder, semicrystalline portions of limerocks, of which limited quantities occur in a number of the principal formations, are suitable for use in mac-

FIGURE 2.—METHODS OF MINING LIMEROCKS: A, OPEN-PIT MINING OF OCALA LIMEROCK; B, DREDGING OF MIAMI OOLITE.

adam surfacing or as concrete aggregate. The bulk of the limerocks are too soft for these purposes, however, and are used in granular form in the construction of base courses and low-type road surfaces.

Limerock base courses are usually constructed in two layers. The construction procedure, while subject to minor variations depending upon the type of limerock being used, is substantially as follows:

Side forms of sufficient height to confine the uncompacted material are placed (see fig. 3). Final preparation of the subgrade is completed by means of templets. The limerock first arriving is dumped adjacent to and then spread over the prepared subgrade to a depth equal to approximately half the thickness of the finished base. Subsequent loads are hauled over and dumped on material already placed and spread on the subgrade as at the start of construction, care being taken at all times to avoid dumping the rock directly on the subgrade.

During the dumping and spreading operations the limerock is thoroughly saturated with water unless it is sufficiently moist when received. Immediately after spreading, the limerock is thoroughly compacted by rolling. Not more than 1 day's spread of material is placed before the second course is started.

Before placing the second layer, the first layer is bladed to a uniform surface and thoroughly watered to obtain bonding between the two layers. The material for the second layer is spread to a depth sufficient to insure the required thickness of finished base. It is then thoroughly watered and rolled until the entire base is dense and unyielding.

After the second course has been rolled, the entire surface is scarified, shaped to the desired cross section, and again watered and thoroughly rolled. Rolling is continued until the entire depth of base course is bonded and compacted and its surface is true to grade and cross section.

The surface is then checked with a templet and straightedge, and the thickness of the base is checked by borings at regular intervals. Any deficiencies in smoothness and thickness are corrected, and the base course is then opened to traffic for a curing period.

After the curing period the base is given a light blading to correct any pitting caused by traffic; following this a bituminous prime coat is applied. After the prime coat has cured for a short period, the bituminous wearing course is placed (see fig. 4).

LIMEROCKS GROUPED ACCORDING TO QUALITY

For the purpose of determining the tests most suitable for distinguishing between the satisfactory and unsatisfactory varieties of limerock, samples representative of conspicuous performance and of roads under different climatic and traffic conditions were tested in laboratories of the Bureau. The tests ordinarily made on similar road-building materials were performed.

Samples were obtained from quarries or pits that supplied material for base courses of known performance and from base courses actually constructed in Florida, Georgia, and Alabama. These samples were furnished by the State highway departments. Information on the quality or performance of the materials represented was supplied by the highway departments and by a Bureau representative.

The various degrees of quality or performance of the materials investigated have been designated by the terms "excellent", "good", "fair", or "poor"

The limerocks considered as excellent provide stable base courses under practically all conditions They have no plasticity and do not shrink appreciably on drying. They are readily machined and finished, and practically any faults resulting from errors in construction can be corrected without detriment to the quality of the material.

The limerocks considered as good are usually satisfactory if ordinary care is observed in construction. These materials have little shrinkage and are only slightly plastic. They provide a dull, closely-knitted surface when wetted and rolled, and are not slippery when wet. Excessive watering during construction, however, may result in their breaking down under manipulation to the extent that proper curing cannot be effected. Also, hauling over such materials when newly laid sometimes prevents bonding.

The limerocks classified as fair are similar in most respects to the good varieties and are just as capable of giving satisfactory service when first-class construction methods are used. However, they are less resistant than the good limerocks to failure in the presence of moisture such as would result from continued wet weather.

The limerocks considered as poor have never given entirely satisfactory results as base course materials. Field observations show that these materials are highly plastic and are slippery when wet. In finishing base courses constructed of some of these limerocks a scum is formed, often to a depth of one-half inch. This scum shrinks and cracks on drying and must be removed by blading. None of the poor limerocks can be used except when dry. They cannot be used with any assurance of satisfactory results except possibly in the lower layer of base courses on unusually good sandy subgrades, or when combined with at least an equal proportion of sand or other granular material.

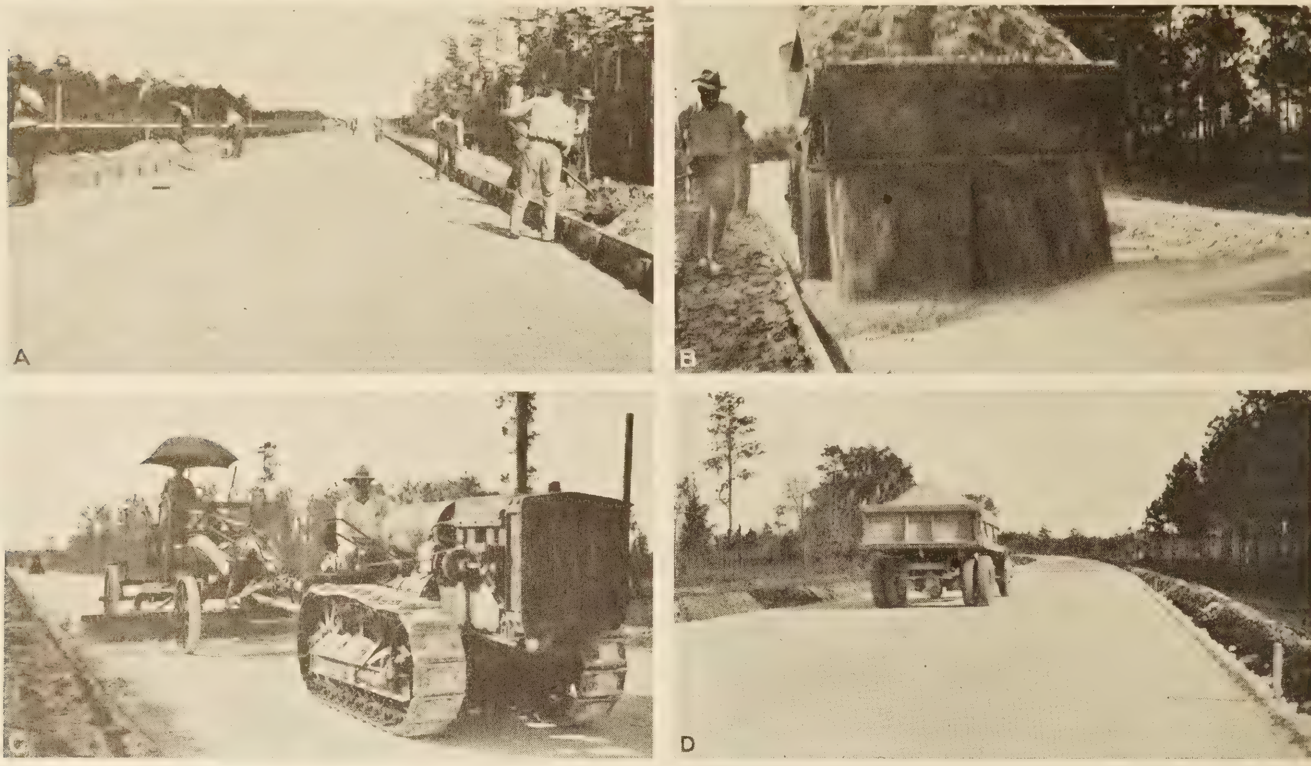

FIGURE 3.—BASE-COURSE ROAD CONSTRUCTION OPERATIONS USING LIMEROCK; A, SUBGRADE PREPARATION. THE SIDE FORMS ARE IN PLACE. B, DUMPING, SPREADING, AND WATERING LIMEROCK; C, BLADING AND FINISHING OPERATIONS; D, FINISHED BASE COURSE READY FOR PRIME COAT.

The materials investigated were grouped according to the foregoing designations, the groups being numbered consecutively from 1 to 4 in the order of their discussion.

VARIOUS LABORATORY TESTS PERFORMED

The laboratory examinations of the limerock samples included chemical analyses, mechanical analyses, the Page tests, and determinations of the subgrade soil constants.

The chemical analyses disclosed the percentages of calcium carbonate, magnesium carbonate, and combined silica, alumina, and iron oxide.

The Page tests, the procedures for which are described in United States Department of Agriculture Bulletin 347, disclosed the cementing value and time of slaking.

The subgrade soil constants determined for the limerocks were liquid limit, plasticity index, shrinkage limit, shrinkage ratio, centrifuge moisture equivalent, field moisture equivalent, volumetric change, and flocculation factor.

The significance of all these constants, except the flocculation factor, and the procedures for their determination are discussed in detail elsewhere.[2]

The flocculation factor is determined as follows: Five cubic centimeters (absolute volume) of the powdered material are thoroughly dispersed in 39 cc of distilled water and 1 cc of chemical deflocculent in a 50-cc glass-stoppered graduate and permitted to settle. The flocculation factor is the ratio of pores to solids in the accumulated sediment at the end of 24 hours.

The tests were performed on material passing a no. 40 sieve, as it was found that with few exceptions the samples could be reduced to this size with little effort. This material may be considered representative of the fine or binder portions of the base courses produced by the crushing action of the rolling and other manipulation during construction. Whether or not this is entirely true does not invalidate the results for comparative purposes, since the same method of preparation was used for all samples.

Chemical composition.—The results of the chemical analyses given in table 1 show that the samples of excellent limerocks contained a high percentage of calcium carbonate, averaging 98.3 percent. However, table 1 shows that the fair limerocks contained an average of 97.1 percent calcium carbonate, or practically as much as the excellent limerocks contained. On the other hand, the good limerocks (excluding sample no. 24, which is obviously not representative of the chemical composition of this group) contained considerably less calcium carbonate than the excellent limerocks, averaging 91.1 percent. Moreover, the poor limerocks contained practically the same amount of calcium carbonate as the good limerocks.

Likewise, the percentages of combined silica, alumina, and iron oxide did not vary according to the quality of the material, these percentages for the good limerocks being practically the same as those for the poor limerocks. The average magnesium carbonate contents of the inferior materials were slightly greater than those of the other groups, but the variation was not consistent enough to serve as a means of identification.

It is thus apparent that the chemical analysis is not an efficient indicator of the quality of different grades of limerock in general. However, it should be stated that a knowledge of the calcium-carbonate content has been found by field experience to be a satisfactory guide

[2] See PUBLIC ROADS, vol. 12, nos. 4, 5, and 8, June, July, and October 1931.

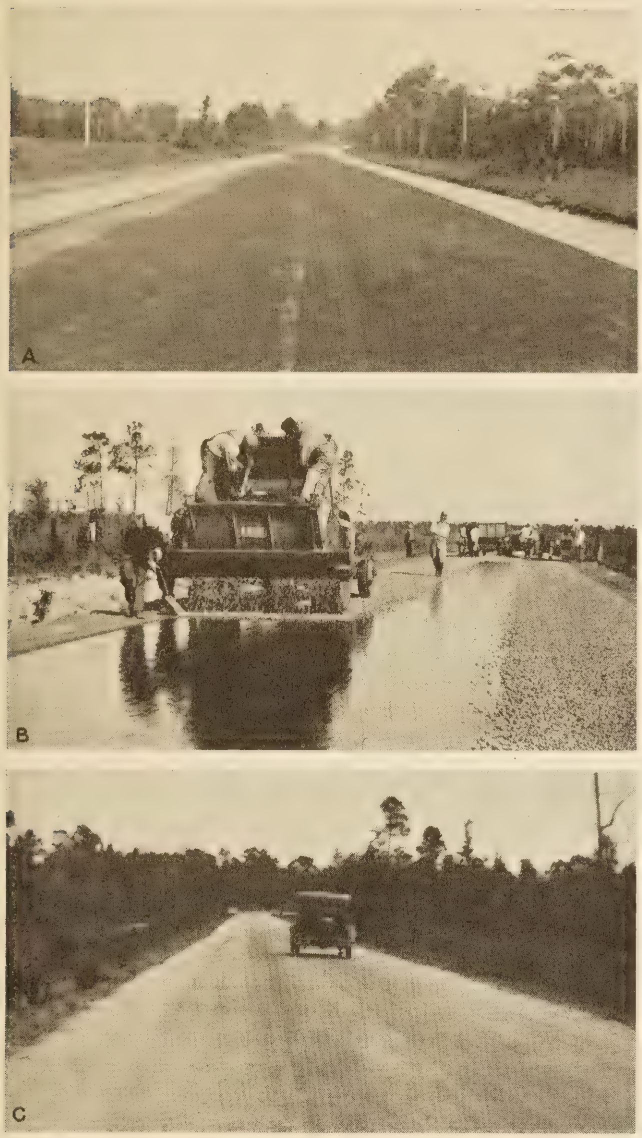

FIGURE 4.—A, COMPLETED LIMEROCK BASE COURSE WITH PRIME COAT; B, APPLYING SURFACE TREATMENT; C, SURFACE TREATMENT COMPLETED AND HIGHWAY IN USE.

TABLE 1.—*Chemical composition, cementing value, and time of slaking of limerocks*

Sample no.	Chemical composition			Cementing value	Time of slaking
	Silica, alumina, and iron oxide	Calcium carbonate	Magnesium carbonate		
	Percent	*Percent*	*Percent*		*Minutes*
GROUP 1. EXCELLENT					
6	0.40	98.66	0.87	14	60+
8	.50	98.66	.76	14	15
11	.65	98.04	.76	11	10
13	.90	98.13	.76	12	60+
14	.90	98.20	.83	53	16
15	1.35	97.85	.80	15	11
Average	.78	98.26	.80	20	----------
GROUP 2. GOOD					
16	8.40	90.89	0.72	87	10
17	3.50	96.07	.68	79	13
18	11.00	87.50	.61	88	13
19	9.10	90.00	.83	71	13
20	8.80	90.00	.72	75	11
21	11.10	87.76	.83	110	20
22	9.60	88.75	.68	57	11
23	9.20	89.55	.87	80	18
24	[1] 30.80	[1] 67.50	[1] .61	102	20
25	3.70	95.00	.76	50	14
26	7.20	91.78	.71	86	10
27	4.45	94.82	.76	95	23
Average	7.82	91.10	.74	83	15
GROUP 3. FAIR					
31	2.65	95.34	1.51	168	30
33	.50	98.75	.83	32	60+
36	3.20	95.63	.76	11	60+
37	1.65	97.14	.91	34	5
38	.35	98.66	.80	39	60+
Average	1.67	97.10	.96	57	----------
GROUP 4. POOR					
43	10.00	88.32	0.68	387	9
44	9.45	87.68	1.44	101	25
45	9.40	88.04	.68	229	9
46	7.90	89.73	1.02	210	18
47	8.25	90.00	.98	149	16
48	7.45	90.45	1.25	110	17
49	8.30	89.38	.76	249	8
50	6.80	91.16	.91	115	15
51	7.35	90.45	.95	107	12
52	7.90	90.36	.83	235	22
53	6.65	91.60	.91	67	11
54	10.00	88.75	.76	196	9
Average	8.29	89.66	.93	180	14

[1] Excluded from averages.

in the selection of materials within certain given deposits or formations.

Cementing and slaking values.—The lack of correlation between the cementing value and reported performance is at once apparent from the wide variation of cementing values within each of the groups. Because of this variation the average value for each group has little significance. It is interesting to note, however, that the average cementing value of the poor limerocks was nine times that of the excellent limerocks. The usual interpretation of this test, that the higher the cementing value the better the material, not only did not apply in this instance but was completely reversed. The erratic results obtained indicate that the cementation test, however interpreted, is a very uncertain method for distinguishing between satisfactory and unsatisfactory limerocks.

The average times of slaking for the poor limerocks were practically equal to those for the good limerocks. Also, there was no consistent difference between the slaking times for the samples of excellent and fair limerocks. Consequently, this test likewise did not serve to differentiate between the different grades of limerock.

Mechanical analyses.—The mechanical analysis of a soil yields information as to the presence of materials varying not only in size but also in character, since the different fractions are not merely the results of mechanical reduction in size of any one material such as sand, but are also the product of chemical reactions caused by weathering, oxidation, and the like. The mechanical analysis of a homogeneous material such as limerock, in contrast, merely reflects the degree to which the material was crushed in preparation, and therefore does not have the same significance as for a soil.

Therefore, the purpose of grinding the limerocks in preparation for test was not, as in the case of soils, to effect separation of particles already differing in size and character, but only to reduce the mass of material to such size as to make possible the performance of the physical tests.

A number of mechanical analyses were performed, however, because it was desired to determine whether or not limerocks differing in quality would break down to different degrees of fineness under the uniform method of preparation used, and whether differences in quality thus would be indicated by the mechanical analysis. The similarity in the gradings of the samples representing the excellent limerocks to those representing the fair quality limerocks indicates that the degree of fineness does not vary substantially with the quality of material and, therefore, that the mechanical analysis is not significant for identification purposes. (See table 2.)

GOOD LIMEROCKS HAD LOW PLASTICITY INDEXES

Subgrade soil constants.—The results of the subgrade soil tests, shown in table 3, show that the shrinkage limits and shrinkage ratios were not consistently different for the various grades of limerock, and that volumetric change on drying out after being wetted to the field moisture equivalent was either entirely absent or so small as to have little significance. Although some shrinkage of the poorer grade limerocks was observed in the field, it is evident that the shrinkage is not always reflected in the laboratory tests and therefore that the usual method of determining shrinkage is of little value in limerock identification.

Likewise, the liquid limit considered apart from its relation to the other tests, while, on the average, increasing with decreasing quality of limerock, did not vary consistently for the different grades when individual determinations are considered.

TABLE 2.—*Mechanical analyses of limerocks*

GROUP 1. EXCELLENT

Sample no.	Particle size smaller than—			
	0.42 mm	0.05 mm	0.005 mm	0.001 mm
	Percent	*Percent*	*Percent*	*Percent*
1	100	38	16	4
2	100	57	17	5
3	100	56	21	3
4	100	56	22	4
5	100	45	19	6
7	100	34	16	5
9	100	50	22	6
10	100	45	22	8
12	100	42	20	6
Average	100	47	19	5
GROUP 3. FAIR				
31	100	70	21	8
33	100	64	22	4
37	100	46	21	4
38	100	74	28	4
39	100	48	22	6
40	100	51	24	6
41	100	40	21	8
42	100	40	20	4
Average	100	54	22	6

The results show that the poor limerocks had higher plasticity indexes than any of the good or fair quality limerocks and that, without exception, the excellent limerocks had no plasticity whatever. In addition, they show that the centrifuge and field moisture equivalents of the fair and poor limerocks were on the whole greater than those of the excellent and good limerocks. The variation thus shown is highly significant in that the difference in water-absorbing properties of the good and fair limerocks thus indicated corresponds to the difference in their reported field performances in the presence of water, and thus offers a basis of distinction between these two groups.

TABLE 3.—*Subgrade soil test constants for different limerocks*

GROUP 1. EXCELLENT

Sample No.	Liquid limit	Plasticity index	Shrinkage limit	Shrinkage ratio	Moisture equivalent		Flocculation factor	Volumetric change
					Centrifuge	Field		
1	26	0	35	1.4	21	22	1.3	0
2	25	0	29	1.5	17	20	1.3	0
3	25	0	28	1.5	23	24	1.4	0
4	25	0	27	1.5	21	23	1.3	0
5	22	0	24	1.6	16	20	1.2	0
6	20	0	26	1.6	17	20	1.2	0
7	20	0	26	1.6	15	19	1.1	0
8	20	0	25	1.6	16	20	1.2	0
9	19	0	23	1.6	15	18	1.1	0
10	19	0	22	1.7	19	17	1.3	0
11	18	0	23	1.6	15	19	--------	0
12	18	0	22	1.7	14	17	1.0	0
13	18	0	21	1.7	15	18	1.2	0
14	17	0	22	1.7	15	18	--------	0
15	17	0	21	1.7	17	16	1.0	0
Average	21	0	25	1.6	17	19	1.2	0
GROUP 2. GOOD								
16	23	0	23	1.6	15	21	1.2	0
17	22	0	19	1.8	15	20	1.2	2
18	21	6	21	1.7	16	18	1.3	0
19	21	5	21	1.7	20	20	1.6	0
20	21	5	21	1.7	19	22	1.6	2
21	21	5	21	1.7	17	20	1.5	0
22	20	5	19	1.8	16	19	1.5	0
23	19	6	18	1.8	16	15	1.4	0
24	19	5	17	1.8	18	17	1.2	0
25	19	4	19	1.7	16	18	1.5	0
26	18	4	16	1.8	15	16	1.3	0
27	18	0	25	1.6	12	20	--------	0
Average	20	4	20	1.7	16	19	1.4	(1)
GROUP 3. FAIR								
28	27	7	24	1.6	22	23	1.5	0
29	26	7	23	1.7	23	23	1.4	0
30	26	6	25	1.6	22	23	1.5	0
31	26	5	26	1.5	28	30	1.9	6
32	25	6	22	1.6	21	23	1.5	2
33	25	5	26	1.6	27	22	1.7	0
34	25	5	24	1.6	23	24	1.2	0
35	25	4	22	1.6	19	25	1.3	5
36	24	5	24	1.6	21	26	1.6	3
37	24	5	24	1.6	21	25	1.3	2
38	24	3	26	1.6	30	24	1.9	0
39	23	3	27	1.5	23	--------	1.4	--------
40	22	3	26	1.6	23	21	1.5	0
41	22	0	29	1.5	20	--------	1.2	--------
42	20	0	26	1.6	19	--------	1.2	--------
Average	24	4	25	1.6	23	24	1.5	1
GROUP 4. POOR								
43	34	17	21	1.7	31	24	2.3	5
44	31	13	24	1.6	27	23	2.0	0
45	30	15	19	1.7	23	22	2.1	5
46	30	10	24	1.6	26	22	1.8	0
47	28	11	21	1.7	24	22	1.8	2
48	28	10	24	1.6	28	24	1.9	0
49	28	9	22	1.7	26	22	--------	0
50	25	9	19	1.8	23	21	1.8	4
51	25	8	21	1.7	23	20	2.0	0
52	24	8	19	1.8	21	20	1.4	2
53	24	8	19	1.7	26	20	1.4	2
54	23	8	16	1.8	18	17	1.6	2
Average	28	11	21	1.7	25	21	1.8	2

[1] Negligible.

Since the centrifuge and field moisture equivalents are apparently of like significance for this purpose, it would seem desirable to include only the simpler of the two tests, the field moisture equivalent, in the testing procedure.

The test results also show that, in general, the flocculation factors increase with decreasing quality of material.

CONCLUSIONS

The test results indicate that the tests of greatest value for predetermining the quality of road-building limerocks are the plasticity, the field moisture equivalent, and the flocculation tests. While present information does not warrant the establishment of rigid limiting values, a plasticity index of 0 to the exclusion of all other determinations appears sufficient to indicate those limerocks that are likely to perform satisfactorily as base-course material under practically all conditions; a plasticity index from 1 to 7 and a field moisture equivalent not exceeding 20 indicate limerocks that will perform satisfactorily in carefully constructed base courses under average conditions; a plasticity index from 1 to 7 and a field moisture equivalent greater than 20 indicate limerocks that will perform satisfactorily in carefully constructed base courses under fairly dry conditions; and a plasticity index of 8 or more indicates limerocks that are likely to prove troublesome.

Use of the foregoing tests does not eliminate the desirability of using the chemical analysis in certain cases, such as those in which the quality of material within a given deposit or formation has been found to vary with the calcium carbonate content.

The flocculation test seems of sufficient significance to be useful for the preliminary field examination of limerocks. A flocculation factor exceeding about 1.9 will indicate material unsuitable for base-course construction, thus eliminating the necessity for any further tests. Of course as is shown, for example, by the test results on samples no. 52 and 53 (see table 3), a flocculation factor under 1.9 will not necessarily indicate a satisfactory material and must be supplemented by plasticity and field moisture equivalent tests for more definite identification. Use of the flocculation test in the field, however, will in some cases eliminate the inconvenience of sampling and transporting materials to a laboratory for further testing.

NEW SPECIFICATIONS FOR HIGHWAY MATERIALS AND HIGHWAY BRIDGES AVAILABLE

A revised edition of the Standard Specifications for Highway Materials and Methods of Sampling and Testing is now available from the American Association of State Highway Officials, 1220 National Press Building, Washington, D. C.

This publication contains standard specifications for portland cement, high-early-strength portland cement, different grades of bituminous materials, aggregates for various highway uses, steel reinforcement, corrugated metal pipe culverts, wire rope and fittings for guard rails, premolded asphalt plank, reinforced concrete culvert pipe, and paint for traffic lines. This material covers 64 pages. Two hundred and forty-three pages are devoted to standard methods of sampling and testing highway materials. Included in this material are nine new methods and tests relating to sampling and testing subgrade soils. The price of the publication is $2 per copy.

The new edition of Specifications for Highway Bridges adopted by the American Association of State Highway Officials will be available within a short time. The sections of this publication are General Provisions, Materials, General Construction, Special Construction, Design, Appendices containing tables of moments, shears, etc., for H–20 loading and a guide to grading structural timbers, and arc welding metal bridge structures. The price of this publication is $2 per copy.

The Bureau of Public Roads has cooperated with the association in preparing both of these publications and the specifications are recommended by the Bureau for general use in highway construction. Copies are not available from the Bureau.

HIGHWAY RESEARCH BOARD TO MEET IN DECEMBER

The Fifteenth Annual Meeting of the Highway Research Board of the National Research Council will be held in Washington, D. C., on December 5 and 6, 1935. A program of reports on research investigations is to be announced in the near future.

CURRENT STATUS OF UNITED STATES PUBLIC WORKS ROAD CONSTRUCTION

AS PROVIDED BY SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT (1934 FUNDS) AND BY THE ACT OF JUNE 18, 1934 (1935 FUNDS)

CLASS 1.—PROJECTS ON THE FEDERAL-AID HIGHWAY SYSTEM OUTSIDE OF MUNICIPALITIES

AS OF SEPTEMBER 30, 1935

STATE	APPORTIONMENTS		COMPLETED				UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS	
	Sec. 204 of the Act of June 16, 1933 (1934 Fund)	Act of June 18, 1934 (1935 Fund)	Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	Estimated Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds
Alabama	$ 3,947,753	$ 2,129,921	$ 7,230,252	$ 3,660,004	$ 469,752	394.9	$ 1,690,593	$ 287,749	$ 1,227,477	84.9		$ 17,945			$ 414,747
Arizona	3,855,555	1,381,051	5,222,449	3,829,744	681,541	381.6	783,475	15,039	676,051	13.3				$ 10,772	23,459
Arkansas	3,334,167	1,714,000	4,147,331	2,802,924	640,429	185.9	1,567,696	492,798	935,379	80.9		32,191	0.1	38,445	106,001
California	7,912,928	3,713,643	12,045,142	7,790,341	1,040,465	395.0	3,257,955	121,228	2,525,800	52.8		11,203	.5	1,360	136,175
Colorado	3,437,265	2,424,504	5,879,938	3,359,613	2,286,625	285.2	219,568	77,653	128,808	7.6					9,072
Connecticut	1,404,213	607,500	1,592,908	1,398,014		30.8	670,946		554,726	7.2				6,199	52,774
Delaware	877,566	461,697	1,355,369	877,566	456,600	48.8	1,102		1,102						3,996
Florida	2,469,369	1,116,600	3,892,937	2,373,654	687,741	147.2	307,215	59,479	220,479	9.2		184,895	2.2	36,236	23,484
Georgia	5,045,592	2,556,745	5,681,091	4,301,508	1,128,394	377.6	1,323,490	585,971	654,495	50.8	$ 5,136	19,417	.3	152,977	754,439
Idaho	2,166,858	1,131,910	2,823,180	2,158,872	563,910	231.6	353,991		343,338	16.5		1,670	.1	7,986	222,992
Illinois	4,408,827	2,408,778	3,595,766	3,048,230	490,658	61.3	3,052,792	1,336,281	1,715,696	45.9		202,424		24,316	
Indiana	5,018,921	2,688,632	4,631,907	4,360,519	105,148	129.1	3,127,293	582,353	2,491,450	152.4		12,920		76,049	79,114
Iowa	5,027,830	1,963,361	7,174,580	4,988,630	1,697,936	421.9	319,000	39,200	264,440	17.5					985
Kansas	5,044,802	2,354,131	6,180,011	5,004,394	997,730	670.1	1,386,053	31,530	1,331,976	92.8		24,425	.9	8,878	
Kentucky	3,751,605	1,302,209	4,226,537	3,512,269	451,162	307.1	1,035,501	182,976	723,447	39.3	8,570	5,000	9.0	47,790	122,599
Louisiana	2,693,135	1,380,419	3,093,008	2,481,839	93,648	79.4	1,088,499	34,283	1,047,066	26.2	40,221	23,404	1.1	136,792	216,302
Maine	1,567,012	782,195	2,163,254	1,544,248	588,897	57.5	197,482	5,179	192,289	6.8				17,585	1,010
Maryland	1,782,263	332,836	1,222,286	1,097,249	123,526	21.7	724,472	594,801	129,672	16.1		28,558	1.4	90,213	51,081
Massachusetts	1,101,716	1,582,874	2,075,114	1,048,966	530,306	47.9	575,991	52,687	523,304	9.7		499,386	2.1	63	29,877
Michigan	6,051,533	3,126,284	7,264,721	5,938,494	955,100	292.3	2,235,900	108,800	2,102,336	109.7		68,848	3.1	4,239	
Minnesota	4,561,011	2,533,733	7,021,257	4,461,058	2,290,026	1,066.3	311,885	93,673	180,120	32.4		3,074		6,280	60,512
Mississippi	3,489,337	2,832,182	6,104,844	2,853,429	837,683	321.9	2,499,073	556,664	1,556,044	133.3	20,193	64,295	4.1	59,051	374,160
Missouri	5,237,532	2,890,666	5,323,643	4,462,251	247,559	218.9	3,479,822	775,281	2,487,285	103.4		60,436	2.7		95,386
Montana	4,463,849	2,714,208	6,944,034	4,305,635	2,060,410	622.3	729,270	111,796	617,474	30.8	33,476	6,374	7.3	12,942	29,949
Nebraska	3,914,481	1,982,182	5,774,256	3,867,871	795,629	428.7	1,444,942	18,006	1,175,618	50.4				28,604	10,934
Nevada	2,909,387	1,350,356	3,841,442	2,758,996	1,001,569	399.0	523,181	127,315	334,320	83.9		2,850		23,075	11,617
New Hampshire	692,119	465,404	1,181,047	692,119	440,896	23.3									24,508
New Jersey	3,173,019	951,379	3,092,894	2,793,057	109,956	46.7	933,099	345,862	490,464	5.9		160,316	7.5	34,100	190,643
New Mexico	2,846,648	1,676,769	4,241,639	2,711,206	1,375,407	408.9	349,999	62,413	287,586	22.7	37,087			35,941	13,777
New York	10,234,915	3,673,231	13,475,251	9,641,772	1,057,480	248.2	5,533,630	576,975	2,440,202	104.4	12,000	1,000	.2	4,168	174,549
North Carolina	4,761,147	1,930,365	5,932,907	4,069,839	727,112	720.4	1,179,344	349,255	780,496	80.4	64,450	350,113	8.8	277,603	72,644
North Dakota	2,902,224	1,469,483	3,972,735	2,721,258	641,515	1,345.9	187,085	54,799	67,755	42.2	46,291	275,180	38.5	79,875	485,035
Ohio	7,277,758	3,539,256	9,302,754	7,123,972	1,581,609	241.4	1,736,287	136,761	1,468,057	27.0	1,500	322,726	5.3	15,525	166,863
Oklahoma	4,608,399	2,342,590	5,580,299	4,262,946	1,151,757	357.1	1,501,714	339,781	1,020,413	45.3		93,545	6.8	5,672	76,875
Oregon	3,053,448	1,426,910	4,152,473	2,905,698	812,953	224.5	768,746	102,555	574,540	24.7				45,195	39,417
Pennsylvania	6,641,194	4,554,082	10,613,249	6,488,594	3,629,165	203.9	994,231	136,292	827,482	20.2	2,126	97,435	.2	14,182	
Rhode Island	988,230	474,772	1,528,596	988,230	464,572	34.9	3,800		3,800						6,400
South Carolina	2,729,583	940,954	2,917,808	2,601,428	249,246	250.7	517,305	72,536	444,769	27.0	11,220	72,537	.5	44,399	174,402
South Dakota	3,005,739	1,523,822	4,392,586	2,717,329	1,136,690	737.6	481,902	287,327	194,575	80.7		108,915	15.9	1,083	83,641
Tennessee	4,246,309	2,105,453	5,786,328	4,105,518	982,164	222.6	994,654	70,497	827,756	32.4	11,254	183,445	2.0	59,040	112,088
Texas	11,588,643	6,858,253	15,189,673	11,436,596	3,078,680	1,321.3	3,651,524	147,758	3,299,383	233.0		432,897	8.9	4,288	47,293
Utah	2,367,205	1,066,345	3,232,606	2,322,418	634,740	284.7	409,671	37,000	277,450	18.3		117,000	1.5	7,787	37,155
Vermont	928,184	466,042	1,365,864	912,376	344,959	64.2	128,565		114,302	3.1	3,922	2,713		11,887	4,068
Virginia	3,731,207	1,916,178	5,334,896	3,495,658	1,363,583	228.1	682,226	170,553	443,015	33.0	31,855	36,343	7.2	33,141	73,237
Washington	3,057,934	1,553,206	3,453,175	2,859,191	567,860	117.2	1,285,755	198,743	960,335	13.2		6,678			18,333
West Virginia	2,013,405	1,140,167	2,468,769	1,946,307	469,952	90.5	446,018	41,447	401,067	15.1	6,984	3,000	.1	18,668	266,147
Wisconsin	4,697,518	1,818,970	6,090,771	4,660,540	1,166,047	275.7	716,056	25,000	587,928	33.5	7,300	58,559	.4	4,678	6,437
Wyoming	2,250,663	1,686,368	3,426,209	2,105,140	1,156,834	656.4	654,160	141,021	486,981	78.1		37,872		4,502	4,681
District of Columbia															
Hawaii	1,693,344	598,778	1,326,739	952,902		27.3	1,124,830	681,972	283,004	13.4	20,973	10,000	.3	37,497	305,774
TOTALS	184,963,342	93,641,394	244,566,525	172,800,412	44,365,621	15,755.5	57,187,788	10,269,289	40,421,552	2,227.4	364,558	3,639,589	139.0	1,529,083	5,214,632

CURRENT STATUS OF UNITED STATES PUBLIC WORKS ROAD CONSTRUCTION

AS PROVIDED BY SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT (1934 FUNDS) AND BY THE ACT OF JUNE 18, 1934 (1935 FUNDS)

CLASS 2.—PROJECTS ON EXTENSIONS OF THE FEDERAL-AID HIGHWAY SYSTEM INTO AND THROUGH MUNICIPALITIES

AS OF SEPTEMBER 30,1935

STATE	APPORTIONMENTS		COMPLETED				UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS	
	Sec. 204 of the Act of June 16, 1933 (1934 Fund)	Act of June 18, 1934 (1935 Fund)	Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	Estimated Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds
Alabama	$ 2,389,928	$ 1,064,961	$ 2,384,804	$ 2,063,925	$ 300,204	61.5	$ 445,135	$ 282,675	$ 162,460	13.6	$ 11,886	$ 201,702	3.7	$ 31,441	$ 400,594
Arizona	756,982	289,673	743,863	622,804	86,475	15.1	327,960	129,321	161,999	.4				4,857	41,199
Arkansas	1,964,534	857,025	2,207,912	1,733,378	371,635	52.3	471,808	100,471	370,685	9.9	116,801	65,733	1.4	13,883	48,972
California	4,213,986	2,219,360	5,537,987	3,857,146	956,332	71.9	2,900,148	356,840	1,087,744	8.6		61,500	.7		113,784
Colorado	1,718,633	190,000	1,937,326	1,670,277	170,641	40.0	11,229	11,229		.8				37,127	19,359
Connecticut	802,407	426,500	838,564	802,407	9,362	10.3	193,692		142,521	1.6		261,170	1.8		13,447
Delaware	460,409	230,849	560,829	460,282	92,040	9.2						42,012		127	96,797
Florida	1,459,649	501,200	1,885,653	1,441,744	146,700	21.8	299,430		259,531	3.3		9,922		17,905	85,047
Georgia	2,724,620	1,278,373	2,437,627	2,229,933	170,446	76.8	671,956	266,401	382,267	16.1		31,607	1.2	228,285	694,053
Idaho	1,197,829	321,126	1,225,345	1,150,878	28,989	21.3	290,885		257,428	3.0				46,951	34,709
Illinois	7,381,910	2,230,350	6,623,661	6,283,943	8,850	66.3	2,706,002	1,083,605	1,622,397	14.8		102,041	1.5	14,362	497,062
Indiana	4,287,050	2,248,858	4,118,138	3,627,832	315,172	77.7	2,186,194	601,075	1,584,942	26.8	558	138,072	1.6	57,586	210,672
Iowa	2,614,472	1,280,000	3,216,653	2,443,068	631,635	76.4	614,808	171,365	403,200	5.8				39	245,165
Kansas	2,522,401	1,432,949	3,136,698	2,473,483	509,645	48.3	927,627		912,325	8.8	48,918	10,980	1.2		
Kentucky	1,927,828	958,599	1,959,805	1,579,834	306,055	40.2	713,670	316,559	377,205	7.9		102,091	1.7	31,435	173,248
Louisiana	1,708,577	744,560	975,918	792,968	172,967	24.0	1,229,217	896,730	290,445	15.0	10,666	218,453	2.2	8,212	62,694
Maine	960,426	484,379	1,060,684	910,266	144,844	19.4	378,629	45,122	319,605	3.8				5,038	19,930
Maryland	891,132	452,514	434,293	422,624		3.8	1,279,775	460,911		2.2				7,597	452,514
Massachusetts	5,007,199	847,600	3,236,487	3,004,220	157,851	18.5	2,092,009	1,974,333	117,676	1.4		464,671	.1	28,646	107,402
Michigan	3,500,637	1,613,142	4,014,733	3,322,717	501,900	48.3	1,166,700	155,650	998,550	11.4		91,200	.7	22,271	21,492
Minnesota	3,719,143	1,421,494	3,724,614	3,118,591	533,952	121.6	1,067,111	514,701	456,767	11.8	2,310	50,385	.6	83,541	380,391
Mississippi	1,744,669	354,022	1,346,607	1,192,935	132,313	42.9	690,415	509,356	110,023	18.6		12,352	1.1	42,378	99,335
Missouri	4,019,501	919,152	3,330,569	3,175,479	53,094	56.8	1,174,210	692,890	455,539	10.7		410,520	.9	151,132	
Montana	1,115,962	113,092	1,155,934	1,047,354	66,149	41.0	41,853	35,859	5,994	2.6				32,750	40,949
Nebraska	1,957,240	991,091	2,572,043	1,915,583	618,581	46.5	296,930	25,623	271,307	4.8	16,035		.4		101,203
Nevada	500,051	100,000	539,499	473,788	56,234	10.8	67,552	26,150	41,402	.6				113	2,364
New Hampshire	740,334	242,465	853,411	668,379	181,465	19.0	46,123		46,123	.3	22,006		.4	49,949	14,877
New Jersey	3,117,921	1,809,500	3,247,132	2,983,109	117,435	23.1	1,178,660	10,297	929,815	5.7		626,855	1.0	124,515	135,394
New Mexico	1,674,158	529,506	1,824,716	1,560,253	249,529	41.9	183,963		183,963	1.1				113,905	96,014
New York	8,092,500	3,961,690	9,127,508	7,835,941	717,700	69.8	3,099,630	250,500	2,793,440	16.6		57,000		6,059	393,550
North Carolina	2,380,573	1,210,236	3,228,526	2,235,534	953,794	105.3	300,711	120,471	180,240	6.9		42,413	.5	24,568	33,789
North Dakota	1,451,112	734,742	1,496,382	1,302,987	184,645	64.2	206,717	96,271	110,446	10.0	51,854	138,959	8.3		300,691
Ohio	4,335,686	2,359,503	5,422,788	4,237,739	677,374	71.7	1,499,530	93,352	1,284,740	11.7		347,700	1.5	4,595	49,689
Oklahoma	2,304,200	1,171,295	2,761,523	2,262,279	401,011	53.4	606,689	39,634	551,372	6.7		96,583	.9	2,288	122,328
Oregon	1,526,724	778,728	2,085,475	1,482,285	541,885	40.1	270,997		234,525	2.2	518			43,921	2,319
Pennsylvania	4,837,988	2,397,703	6,557,054	4,791,003	1,467,935	84.5	291,409		290,470	.9	40,000	244,787	1.8	6,985	394,510
Rhode Island	512,665	285,760	650,675	508,370	140,964	8.9								4,295	144,796
South Carolina	1,364,791	488,000	1,298,762	1,233,572	53,000	40.6	345,969	124,414	216,005	10.8	6,805	15,412	.1		203,584
South Dakota	1,502,870	761,910	1,385,834	1,243,836	141,263	43.2	360,408	65,622	294,786	21.3	6,853	62,905	3.8	186,559	262,957
Tennessee	2,123,155	1,121,790	2,462,842	2,054,124	322,219	31.7	654,103	69,031	585,072	4.8		58,746	.6		155,753
Texas	6,642,863	1,795,000	6,309,642	5,722,509	454,734	142.3	1,659,538	666,033	813,546	19.4	2,192	485,881	5.8	252,129	40,839
Utah	778,826	533,173	1,122,592	649,146	364,373	27.8	333,602	129,130	167,800	6.5		1,000		550	
Vermont	500,509	240,611	730,862	486,227	161,761	17.2	66,356		66,296	1.5		8,129	.4	14,262	4,425
Virginia	1,948,780	956,021	2,519,547	1,704,936	565,758	39.3	547,137	220,395	276,424	7.8	9,099	69,057	2.5	14,349	44,781
Washington	1,977,260	776,603	2,564,806	1,974,755	568,864	45.9	175,618		175,618	1.6		1,415		2,505	30,706
West Virginia	1,342,270	570,085	1,178,195	1,112,989	28,109	18.6	590,094	229,220	331,388	8.6		110,272	.1	61	100,316
Wisconsin	2,596,143	1,379,513	3,727,717	2,509,104	1,140,176	70.6	241,862	53,457	183,352	3.6		55,985	1.2	33,582	
Wyoming	1,125,332	29,416	1,104,126	1,098,148	2,784	23.5	36,783	22,068	14,132	2.5		10,284		5,116	2,216
District of Columbia	946,445	181,051	1,127,496	946,445	181,051	6.6									
Hawaii															
TOTALS	115,370,280	47,885,170	123,993,857	102,421,129	16,159,895	2,211.9	34,940,844	10,846,761	20,521,565	354.8	346,501	4,707,794	49.7	1,755,889	6,494,916

CURRENT STATUS OF UNITED STATES PUBLIC WORKS ROAD CONSTRUCTION

AS PROVIDED BY SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT (1934 FUNDS) AND BY THE ACT OF JUNE 18, 1934 (1935 FUNDS)

CLASS 3.—PROJECTS ON SECONDARY OR FEEDER ROADS

AS OF SEPTEMBER 30, 1935

STATE	APPORTIONMENTS		COMPLETED				UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS	
	Sec. 204 of the Act of June 16, 1933 (1934 Fund)	Act of June 18, 1934 (1935 Fund)	Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	Estimated Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds
Alabama	$ 2,032,452	$ 1,064,960	$ 2,430,557	$ 1,921,746	$ 486,536	169.3	$ 576,844	$ 61,735	$ 515,109	31.6		$ 63,180	2.3	$ 48,971	$ 135
Arizona	599,423	971,211	1,189,294	530,962	554,239	99.4	439,543	59,762	363,778	34.9				8,699	53,194
Arkansas	1,449,634	857,024	1,599,763	1,337,753	249,811	197.4	649,483	79,018	568,398	72.4		31,680	3.1	32,864	7,135
California	3,480,440	1,999,203	4,762,400	3,280,107	718,369	210.3	1,429,520	196,170	1,147,259	35.6				4,163	133,575
Colorado	1,718,632	871,502	2,682,390	1,608,6[illegible]	623,474	276.9	472,158	110,000	248,028	34.8					
Connecticut	659,120	420,868	920,180	659,120	235,769	19.5						125,141	1.9		59,958
Delaware	481,113	230,849	529,649	277,564	230,280	68.8	203,783	203,549		2.5					569
Florida	1,302,816	1,043,543	1,692,424	1,276,273	394,194	83.3	664,116		612,569	42.1		15,858		26,543	20,923
Georgia	2,320,973	1,278,373	2,027,502	1,892,659	96,570	149.4	732,023	335,910	396,113	50.5				92,404	785,690
Idaho	1,121,562	824,450	1,888,884	1,094,530	611,868	217.5	136,457		136,457	6.7				27,032	76,125
Illinois	5,780,033	4,282,273	6,167,139	5,051,536	1,054,618	323.4	3,518,485	715,399	2,803,086	173.1		104,970	2.7	13,098	319,599
Indiana	731,872	151,473	531,688	460,033	15,004	56.6	326,965	232,843	94,122	43.3				38,996	42,346
Iowa	2,413,358	1,875,000	3,476,969	2,390,379	982,345	564.3	921,993	22,027	854,300	139.4		25,600	2.3	952	12,755
Kansas	2,522,401	1,330,595	2,820,002	2,483,248	333,645	255.5	1,036,103	39,153	996,950	58.5					
Kentucky	1,837,926	1,557,503	2,538,453	1,813,980	578,415	303.1	974,615		908,100	106.1		65,505	3.0	23,946	5,482
Louisiana	1,426,879	838,953	1,275,331	1,085,083	177,832	54.1	908,081	337,357	570,724	42.0	$ 3,135	74,211	2.5	1,303	16,186
Maine	842,479	445,012	1,393,836	842,403	429,831	105.7	13,424		13,424	.8				75	1,757
Maryland	891,132	1,024,708	1,218,325	876,320	283,690	79.8	209,560		209,560	6.9	9,800	196,094	6.3	5,012	335,364
Massachusetts	488,185	920,000	482,233	474,504		15.2	837,333		837,333	20.4				13,681	82,667
Michigan	3,184,057	1,713,142	3,552,903	3,025,292	278,850	222.2	1,482,552	117,227	1,344,325	78.0		84,300	3.3	41,538	5,667
Minnesota	2,376,415	1,470,324	3,615,283	2,178,974	1,183,512	399.9	388,150	133,322	227,827	16.3		37,523	.1	64,119	21,462
Mississippi	1,744,669	354,023	1,383,690	1,367,190	16,500	150.3	456,153	349,342	106,811	36.4		197,261	17.9	28,137	33,451
Missouri	2,923,273	2,363,922	3,841,407	2,828,144	926,260	816.4	1,418,891	93,414	1,279,720	230.7		157,941	10.7	1,716	
Montana	1,859,937	942,434	2,585,856	1,739,224	836,801	310.8	166,200	68,565	97,635	16.1				52,149	7,998
Nebraska	1,957,240	991,091	2,598,664	1,953,891	621,814	449.3	358,629		314,125	51.9		55,152	2.8	3,349	
Nevada	1,136,479	852,000	1,737,865	1,109,579	578,758	211.9	240,641	8,443	232,198	24.7				18,457	41,043
New Hampshire	477,386	261,593	754,680	475,526	229,412	33.4	22,102		20,000	.8				1,860	12,181
New Jersey	55,099	460,000	56,528	55,099		.5	107,525		107,525	1.7		213,355	1.1		139,121
New Mexico	1,272,129	735,425	1,920,315	1,272,129	646,136	295.1	73,332		73,332	2.6					15,956
New York	4,002,686	3,693,000	5,963,461	3,538,046	1,616,289	217.0	2,622,520	327,200	1,996,050	166.9				137,440	80,661
North Carolina	2,380,573	1,700,340	3,131,693	2,246,077	880,817	346.9	908,856	90,285	818,571	82.7	44,161		.9	50	952
North Dakota	1,451,112	734,742	1,437,412	1,293,948	142,627	444.2	317,008	95,421	221,587	76.4	31,234	262,180	64.2	30,508	108,348
Ohio [1]	3,871,148	1,966,253	4,637,208	3,762,196	622,944	361.2	1,034,673	37,360	943,929	53.9	47,700	188,740	13.3	23,892	210,640
Oklahoma	2,304,199	1,171,295	2,560,728	2,154,542	146,861	290.4	1,075,747	147,071	849,239	43.3		29,343	.3	2,586	145,852
Oregon	1,526,724	892,176	2,470,262	1,494,881	771,691	172.3	59,551	19,526	4,700	1.6				12,317	115,785
Pennsylvania	7,411,822	2,639,003	8,180,744	6,578,394	1,342,592	628.7	2,035,033	669,709	1,272,269	126.2		15,701		163,719	8,441
Rhode Island	497,813	254,040	497,631	447,809	39,789	34.2	211,374		211,374	7.1	50,004		3.0		2,876
South Carolina	1,364,791	1,342,000	1,328,650	1,089,302	233,769	136.9	1,318,840	249,891	1,030,599	156.3		70,760	3.6	25,598	6,872
South Dakota	1,502,870	761,911	1,873,256	1,424,764	443,659	536.1	255,492	69,046	186,446	75.8		119,584	28.0	9,060	12,222
Tennessee	2,123,155	1,075,748	2,409,369	1,951,410	366,515	165.0	510,131	103,127	407,004	22.0	3,663	138,374	3.2	64,955	163,855
Texas	6,012,518	3,638,000	8,067,322	5,987,309	1,588,118	942.8	2,040,331		2,040,331	104.2		1,000	14.2	25,209	8,551
Utah	1,048,677	533,173	1,730,899	1,040,016	393,173	236.1	166,380		140,000	16.0				8,661	
Vermont	438,880	241,354	811,648	435,360	240,452	53.2								3,520	903
Virginia	1,736,770	893,188	2,007,451	1,565,227	383,311	240.8	452,637	129,548	313,003	45.1	17,600	148,154	14.9	24,396	48,721
Washington	1,080,673	776,603	1,672,985	1,080,673	550,517	117.8	226,087		226,087	7.1					
West Virginia	1,118,559	570,083	951,249	856,050	50,375	46.8	601,424	257,170	344,254	31.9		103,572	6.7	5,338	71,883
Wisconsin	2,431,220	1,743,354	3,451,065	2,341,749	844,852	205.9	957,325	68,000	779,377	30.9		117,484	.2	21,471	1,641
Wyoming	1,125,332	571,928	1,443,025	1,122,741	302,796	242.1	269,134		269,132	35.1				2,590	
District of Columbia	972,024	792,791	1,449,096	971,729	477,367	12.2	104,932		104,932	.7		75,395	.6	296	135,097
Hawaii	177,718	351,000	178,209	177,718		4.9	150,713		143,867	4.6					207,133
TOTALS	93,666,378	58,473,436	117,931,573	86,921,821	24,813,047	11,574.8	34,082,849	5,426,590	27,381,559	2,448.6	207,297	2,718,058	213.1	1,110,670	3,560,772

PUBLICATIONS of the BUREAU OF PUBLIC ROADS

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Department and as the Department does not sell publications, please send no remittance to the United States Department of Agriculture.

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Report of the Chief of the Bureau of Public Roads, 1924. 5 cents.

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

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

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

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

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

Report of the Chief of the Bureau of Public Roads, 1933.

Report of the Chief of the Bureau of Public Roads, 1934.

DEPARTMENT BULLETINS

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

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

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

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

TECHNICAL BULLETINS

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

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

MISCELLANEOUS CIRCULARS

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

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Federal Legislation and Regulations Relating to Highway Construction. 10 cents.

Supplement No. 1 to Federal Legislation and Regulations Relating to Highway Construction.

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The Taxation of Motor Vehicles in 1932. 35 cents.

REPRINT FROM PUBLIC ROADS

Reports on Subgrade Soil Studies. 40 cents.

Single copies of the following publications may be obtained from the Bureau of Public Roads upon request. They cannot be purchased from the Superintendent of Documents.

SEPARATE REPRINT FROM THE YEARBOOK

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

TRANSPORTATION SURVEY REPORTS

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

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

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

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

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

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

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

CURRENT STATUS OF UNITED STATES PUBLIC WORKS ROAD CONSTRUCTION

AS PROVIDED BY SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT (1934 FUNDS) AND BY THE ACT OF JUNE 18, 1934 (1935 FUNDS)

SUMMARY OF CLASSES 1, 2, AND 3.

AS OF SEPTEMBER 30, 1935

STATE	APPORTIONMENTS		COMPLETED				UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS	
	Sec. 204 of the Act of June 16, 1933 (1934 Fund)	Act of June 18, 1934 (1935 Fund)	Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	Estimated Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds
Alabama	$ 8,370,133	$ 4,259,842	$ 12,045,614	$ 7,645,675	$ 1,256,493	625.7	$ 2,712,573	$ 632,159	$ 1,905,046	130.1	$ 11,886	$ 282,827	5.9	$ 80,412	$ 815,476
Arizona	5,211,960	2,641,935	7,155,606	4,983,510	1,322,254	496.1	1,550,978	204,122	1,201,829	48.7				24,328	117,852
Arkansas	6,748,335	3,428,049	7,955,007	5,874,055	1,261,875	435.6	2,688,987	672,287	1,874,462	163.3	116,801	129,603	4.7	85,192	162,108
California	15,607,354	7,932,206	22,345,528	14,927,594	2,715,166	677.2	7,587,623	674,238	4,760,803	97.1		72,703	1.2	5,522	383,534
Colorado	6,874,530	3,486,006	10,499,653	6,638,521	3,080,740	602.1	702,954	198,882	376,836	43.3				37,127	28,431
Connecticut	2,865,740	1,454,868	3,351,652	2,859,541	245,131	60.5	864,638		697,247	8.8		386,311	3.7	6,199	126,179
Delaware	1,819,088	923,395	2,445,848	1,615,412	778,920	126.8	204,885	203,549	1,102	2.5		42,012		127	101,361
Florida	5,231,834	2,661,343	7,471,014	5,091,671	1,228,636	252.2	1,270,761	59,479	1,092,579	54.5		210,675	2.2	80,684	129,453
Georgia	10,091,185	5,113,491	10,146,219	8,424,100	1,395,409	603.8	2,727,468	1,188,282	1,432,874	117.5	5,136	51,025	1.6	473,666	2,234,183
Idaho	4,486,249	2,277,486	5,937,410	4,404,280	1,204,767	470.4	781,333		737,223	26.1		1,670	.1	81,970	333,825
Illinois	17,570,770	8,921,401	16,386,565	14,383,710	1,554,126	451.0	9,277,279	3,135,284	6,141,179	233.7		409,435	4.2	51,777	816,661
Indiana	10,037,843	5,088,963	9,281,733	8,448,384	435,324	263.5	5,640,452	1,416,271	4,170,514	222.5	557	150,992	1.6	172,631	332,132
Iowa	10,055,660	5,118,361	13,868,202	9,822,077	3,311,917	1,062.6	1,855,801	232,592	1,521,940	162.7		25,600	2.3	991	258,904
Kansas	10,089,604	5,117,675	12,136,711	9,961,125	1,841,020	974.0	3,349,784	70,683	3,241,251	160.1	48,918	35,405	2.2	8,878	
Kentucky	7,517,359	3,818,311	8,724,794	6,906,083	1,335,632	650.3	2,723,785	499,535	2,008,753	153.2	8,570	172,596	13.6	103,171	301,330
Louisiana	5,828,591	2,963,932	5,344,257	4,359,891	444,446	157.5	3,225,797	1,268,371	1,903,235	83.2	54,022	316,069	5.8	146,308	295,182
Maine	3,369,917	1,711,586	4,617,774	3,296,917	1,163,572	182.6	589,535	50,301	525,318	11.4				22,699	22,696
Maryland	3,564,527	1,810,058	2,874,905	2,396,193	407,217	105.4	2,213,807	1,055,712	339,230	25.3	9,800	224,652	7.7	102,822	838,959
Massachusetts	6,597,100	3,350,474	5,793,833	4,527,690	688,157	81.7	3,505,333	2,027,020	1,478,313	31.5		964,057	2.2	42,389	219,947
Michigan	12,736,227	6,452,568	14,832,357	12,286,502	1,735,850	562.8	4,885,152	381,677	4,445,212	199.0		244,348	7.1	68,048	27,159
Minnesota	10,656,569	5,425,551	14,365,154	9,758,623	4,007,490	1,587.8	1,767,147	741,696	864,714	60.5	2,310	90,982	.6	153,940	462,364
Mississippi	6,978,675	3,540,227	8,835,141	5,413,555	986,496	515.1	3,645,641	1,415,362	1,772,878	188.2	20,193	273,907	23.1	129,565	506,946
Missouri	12,180,306	6,173,740	12,495,619	10,465,874	1,226,912	1,092.2	6,072,923	1,561,585	4,222,545	344.8		628,897	14.2	152,848	95,386
Montana	7,439,748	3,769,734	10,685,824	7,092,213	2,963,360	974.1	937,323	216,219	721,104	49.5	33,476	6,374	7.3	97,840	78,896
Nebraska	7,828,961	3,964,364	10,944,963	7,737,345	2,036,024	924.5	2,100,502	43,628	1,761,050	107.0	16,035	55,152	3.2	31,953	112,137
Nevada	4,545,917	2,302,356	6,118,807	4,342,364	1,636,561	621.7	831,374	161,908	607,920	109.1		2,850		41,645	55,025
New Hampshire	1,909,839	969,462	2,789,138	1,836,024	851,772	75.7	68,226		66,123	1.0	22,006		.4	51,809	51,566
New Jersey	6,346,039	3,220,879	6,396,554	5,831,264	227,391	70.3	2,219,284	356,159	1,527,804	13.2		1,000,526	9.6	158,615	465,158
New Mexico	5,792,935	2,941,700	7,986,670	5,543,588	2,271,072	745.9	607,295	62,414	544,881	26.5	37,087			149,846	125,747
New York	22,330,101	11,327,921	28,566,220	21,015,760	3,391,469	535.0	11,255,780	1,154,675	7,229,692	287.9	12,000	58,000	.2	147,666	648,761
North Carolina	9,522,293	4,840,941	12,293,127	8,551,450	2,561,723	1,172.6	2,388,911	560,011	1,779,307	169.9	108,611	392,526	10.2	302,221	107,385
North Dakota	5,804,448	2,938,967	6,906,529	5,318,194	968,787	1,854.3	710,810	246,491	399,788	128.7	129,380	676,319	111.0	110,383	894,074
Ohio	15,484,592	7,865,012	19,362,750	15,123,908	2,881,927	674.4	4,270,490	267,473	3,696,727	92.6	49,200	859,166	20.1	44,012	427,192
Oklahoma	9,216,798	4,685,180	10,902,549	8,679,766	1,699,630	700.8	3,184,150	526,486	2,421,024	95.4		219,471	8.1	10,546	345,055
Oregon	6,106,896	3,097,814	8,708,211	5,882,864	2,126,528	437.0	1,099,294	122,081	813,765	28.5	518			101,434	157,521
Pennsylvania	18,891,004	9,590,788	25,351,047	17,857,990	6,439,693	917.0	3,320,673	806,002	2,390,221	147.3	42,126	357,923	2.0	184,886	402,951
Rhode Island	1,998,708	1,014,572	2,676,902	1,944,409	645,325	78.0	215,174		215,174	7.1	50,004		3.0	4,295	154,072
South Carolina	5,459,165	2,770,954	5,545,219	4,924,302	536,015	428.2	2,182,114	446,841	1,691,372	194.1	18,025	158,709	4.2	69,998	384,857
South Dakota	6,011,479	3,047,643	7,651,676	5,385,928	1,721,612	1,316.8	1,097,802	421,994	675,808	177.8	6,853	291,404	47.7	196,703	358,820
Tennessee	8,492,619	4,302,991	10,658,539	8,111,052	1,670,897	419.3	2,158,888	242,654	1,819,832	59.2	14,917	380,566	5.8	123,995	431,696
Texas	24,244,024	12,291,253	29,566,638	23,146,414	5,121,532	2,406.4	7,351,392	813,792	6,153,261	356.7	2,192	919,778	28.8	281,626	96,683
Utah	4,194,708	2,132,691	6,086,097	4,011,580	1,392,286	548.6	909,653	166,130	585,250	40.9		118,000	1.5	16,998	37,155
Vermont	1,867,573	948,007	2,908,374	1,833,963	747,172	134.5	194,921		180,597	4.6	3,922	10,842	.4	29,688	9,396
Virginia	7,416,757	3,765,387	9,861,894	6,765,821	2,312,652	508.2	1,682,000	520,496	1,032,442	85.9	58,554	253,554	24.6	71,886	166,739
Washington	6,115,867	3,106,412	7,690,965	5,914,619	1,687,241	280.9	1,687,460	198,743	1,362,039	21.9		8,093		2,505	49,038
West Virginia	4,474,234	2,280,335	4,598,213	3,915,346	548,436	156.0	1,637,535	527,838	1,076,709	55.6	6,984	216,843	6.9	24,066	438,347
Wisconsin	9,724,881	4,941,837	13,269,553	9,511,393	3,151,075	552.1	1,915,243	146,457	1,550,656	68.0	7,300	232,028	1.9	59,732	8,078
Wyoming	4,501,327	2,287,712	5,973,360	4,326,029	1,462,414	922.0	960,076	163,089	770,245	115.7		48,156		12,208	6,898
District of Columbia	1,918,469	973,842	2,576,592	1,918,173	658,419	18.8	104,932		104,932	.7		75,395	.6	295	135,097
Hawaii	1,871,062	949,778	1,504,948	1,130,620		32.2	1,275,543	681,972	426,870	18.0	20,973	10,000	.3	37,497	512,908
TOTALS	394,000,000	200,000,000	486,491,955	362,143,362	85,338,563	29,542.2	126,211,481	26,542,640	88,324,676	5,030.8	918,356	11,065,441	401.8	4,395,642	15,271,320

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