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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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General View of Deflectometer Used in Tests of Six-Wheel Trucks

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THE SIX-WHEEL TRUCK AND THE PAVEMENT

TESTS CONDUCTED BY THE UNITED STATES BUREAU OF PUBLIC ROADS TO DETERMINE THE RELATIVE EFFECT OF FOUR AND SIX WHEEL TRUCKS

Reported by L. W. TELLER, Associate Engineer of Tests, United States Bureau of Public Roads

TESTS conducted recently by the United States Bureau of Public Roads indicate definitely that the tensile stress set up in a concrete pavement by a six-wheel truck is only about half as great as the stress produced by a four-wheel truck of the same gross load. The tests show also that the stress produced in the pavement by the six-wheel vehicles is a function of the load on the wheels and not of the axle spacing. This seems to be true for all spacings of the rear axles greater than 3 feet, there being some indication that when the wheels are closer together than 36 inches the stress produced in the pavement may be increased.

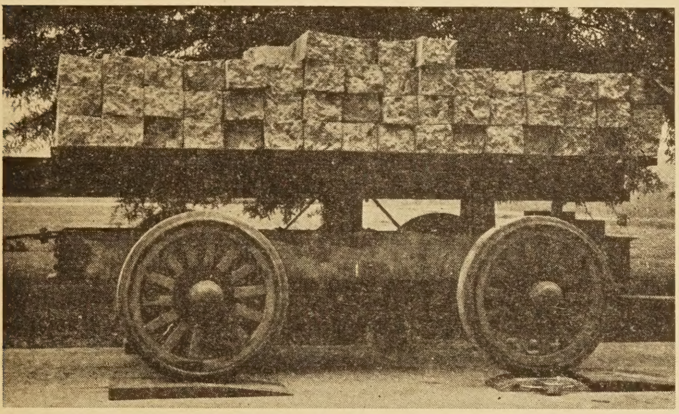

FIG. 1.—Special trailer with 86-inch axle spacing. The rear wheels are resting on the loadometers, the front wheels on the leveling platforms

It is also shown that under six-wheel as well as four-wheel trucks the maximum tensile stress occurs in the bottom of the slab, regardless of the axle spacing. Even though there is counterflexure of the pavement between the wheels, the tension developed in the top of the slab is of less magnitude than the tension developed in the bottom of the slab directly under the wheels. This latter tension, therefore, is the critical stress for six-wheel as well as four-wheel vehicles.

Loads passing over the pavement 21 inches from the edge were found to produce an average stress less than 50 per cent as great as that produced by the same loads passing 9 inches from the edge.

The pavement on which the tests were made was a concrete slab of uniform thickness, and the maximum deformation of this slab was found to occur along the edge under both four and six wheel vehicles. Subjected to loads of varying magnitude applied at the same point, both the deflection and the deformation of the slab were found to be proportional to the load.

The six-wheel truck is the motor-vehicle designers' solution of the problem presented by the demand for vehicles which will carry heavy "pay loads" without violation of the restrictive regulations adopted for the protection of the highways. The regulations are the outgrowth of experience and research, which show that heavy concentrated loads unwarrantably increase the cost of pavement construction and maintenance. Such regulation limits the gross load, the axle load, the wheel load, the load per inch of tire width supported by the pavement, or several of these factors.

Facing such restrictions, the motor-vehicle designers have sought to increase the legal carrying capacity of the single vehicle by using four wheels instead of two at the rear, where the bulk of the load is carried, thus doubling the number of axles, wheels, and inches of tire width by which the load is transmitted to the pavement. Such vehicles comply with the regulations and can legally carry heavier loads than the four-wheel vehicles, and the increase in "pay load" accommodated is much greater than the increase in dead load necessitated by the new design.

While it has seemed reasonable to assume that the addition of the extra pair of wheels would accomplish the purpose intended, the Bureau of Public Roads has felt that the effect upon the pavement should be determined experimentally, and to that end the tests reported in this article have been conducted to study the behavior of a concrete pavement under both six and four wheel trucks.

OBJECTS AND METHODS OF TEST

The principal objects of the tests were to determine:

1. The relative stresses produced in the pavement by four and six wheel vehicles having (*a*) the same gross load, and (*b*) the same wheel load.
2. The effect of the axle spacing of six-wheel vehicles on the stress produced in the pavement.

In order to study the effect of various axle spacings a special trailer was constructed consisting of a frame provided with two axles, which could be spaced at any distance from 3 to 10 feet apart, the load being equally supported by all four wheels. The front end of the platform that carried the cargo rested on a knife-edge placed transversely on the rear end of the motor truck which was used to move the trailer. Thus the essentials of a six-wheel vehicle were reproduced and axle spacings of from 3 to 10 feet could be had at will. As the trailer was to be run only at very low speeds over a smooth pavement, no springs were provided. The wheels were equipped with 40 by 6 inch single solid rubber truck tires. Figure 1 shows the general appearance of the unit. The four-wheel, 5-ton capacity truck (fig. 2), equipped with similar tires, was used for comparative tests. In addition to the equipment described above two other trucks were used, one a six-wheel and one a four-wheel, class B Army truck of

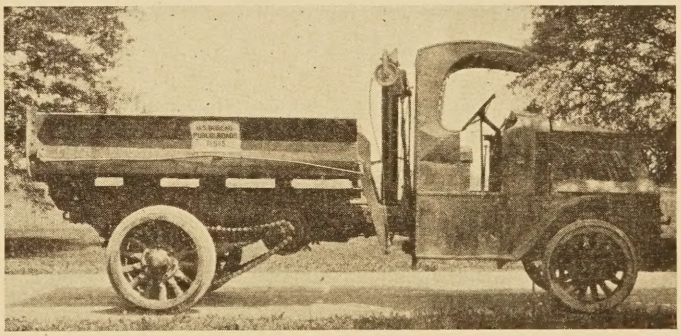

FIG. 2.—Four-wheel truck with solid tires

identical design except for the rear end construction. These two trucks, which were equipped with pneumatic tires, are shown in Figures 3 and 4.

Two "loadometers" were used to measure the wheel loads, these devices having been calibrated prior to their use in this investigation. The vehicle being weighed was supported in a level position by small wheel platforms, as shown in Figure 1.

FIG. 3.—Six-wheel truck with pneumatic tires

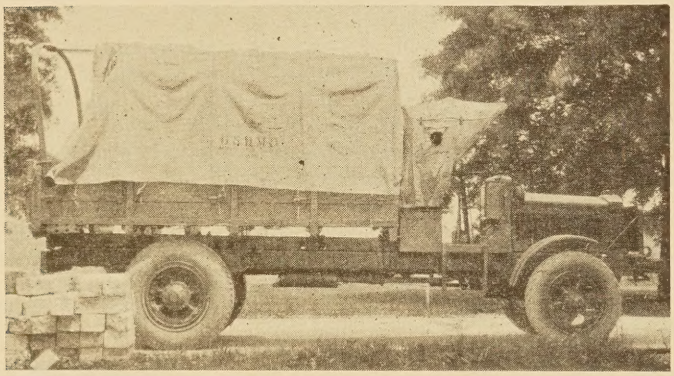

FIG. 4.—Four-wheel truck with pneumatic tires

The test pavement was a 6-inch plain concrete slab of uniform cross section. The width of the slab was 18 feet and the length between transverse joints was 30 feet. The tests were made at one corner of this slab, the transverse joint being a poured joint, without expansion material.

The deflection data were secured with an instrument called a "deflectometer," which, as shown on the cover, consists of a series of pen arms supported by the pavement. Each pen arm traces a continuous record on a moving paper of the deflection of the slab at that point as the truck wheel moves over the slab. Thus, each curve is an influence line of deflection for the particular point at which the pen arm is supported. The vertical movement of the slab is multiplied fifty times on these curves by a direct lever action. From a series of such curves it is possible to obtain the elastic curve of the pavement for any position of the wheel. The apparatus is provided with electric contacts which record on the same paper the instant at which the truck passes a given point. The speed of the truck and of the paper are determined by a time-recording mechanism which is simply a pendulum whose known period is recorded electrically on the moving paper record. Details of this part of the apparatus are shown in Figure 6.

Graphic strain gauges, illustrated in Figure 6,[1] were installed along the upper and lower edges of the slab in the positions shown in Figure 7. These gauges, which recorded the maximum compressive and tensile deformations occurring in the outer fibers of the concrete pavement during each passage of the truck, were protected by a canvas sunshade during the tests.

Two traffic lines were painted on the pavement—one 9 inches, the other 21 inches from the edge. During the test runs the outer wheels of the vehicle were kept centered over one or the other of these lines. From three to five runs were made under each test condition, depending upon the manner in which the recording apparatus functioned.

Complete data were obtained on nearly 100 runs, the conditions being varied according to the following program:

I. Four-wheel trucks:
  - (*a*) Pneumatic tires, 44 by 10 inches, 9 inches from edge of pavement.
    1. 4,000-pound wheel load.
    2. 8,000-pound wheel load.
  - (*b*) Single solid tires, 40 by 6 inches, 9 and 21 inches from edge of pavement, with 4,000, 5,000, 6,000, 7,000, and 8,000 pound wheel loads.

II. Six-wheel trucks:
  - (*a*) Pneumatic tires, 38 by 9 inches, with 50-inch axle spacing, 9 inches from edge of pavement.
    1. 4,000-pound wheel load.
    2. 4,300-pound wheel load.
  - (*b*) Pneumatic tires, 40 by 6 inches, with 4,350-pound wheel load.
    1. Nine inches from edge of pavement and 36, 40, 44, 48, 54, 60, 72, 86, and 118 inch axle spacings.
    2. Twenty-one inches from edge of pavement and 36, 40, 44, and 54 inch axle spacings.

ELASTIC CURVES DEVELOPED FROM DEFLECTION MEASUREMENTS

As previously remarked, the deflection data obtained consisted of deflection influence lines for a series of points along the edge of the pavement. Each of these curves showed the deflection of a point of the pavement as the truck passed over it. By relating the movement

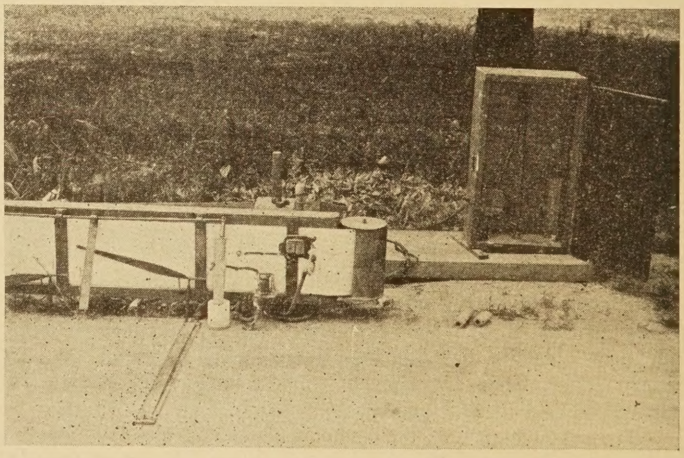

FIG. 5.—Details of pendulum and road contacts of deflectometer

of the load to the curve, the deflection of that particular part of the pavement for any position of the load can be obtained. With a series of such influence curves representing the deflection of a series of points along the edge of the pavement, it is possible to determine the form assumed by the pavement under the load, or, in other words, the elastic curve of the pavement.

[1] For a description of the graphic strain gauge see Engineering News-Record, vol. 90, No. 13, Mar. 29, 1923, p. 575.

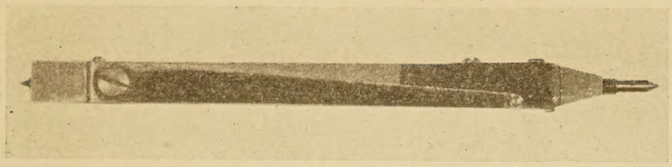

FIG. 6.—Graphic strain gauge developed by the Bureau of Public Roads. The gauge is inserted in a slot cut into the concrete with the two points bearing against the concrete at the ends of the slot. Deformation of the concrete causes a throw of the lever, which causes a point at its end to trace a line on a smoked-glass slide. The length of the line is proportional to the deformation

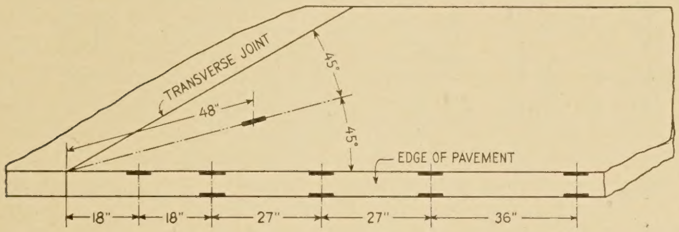

FIG. 7.—Layout of graphic strain gauges

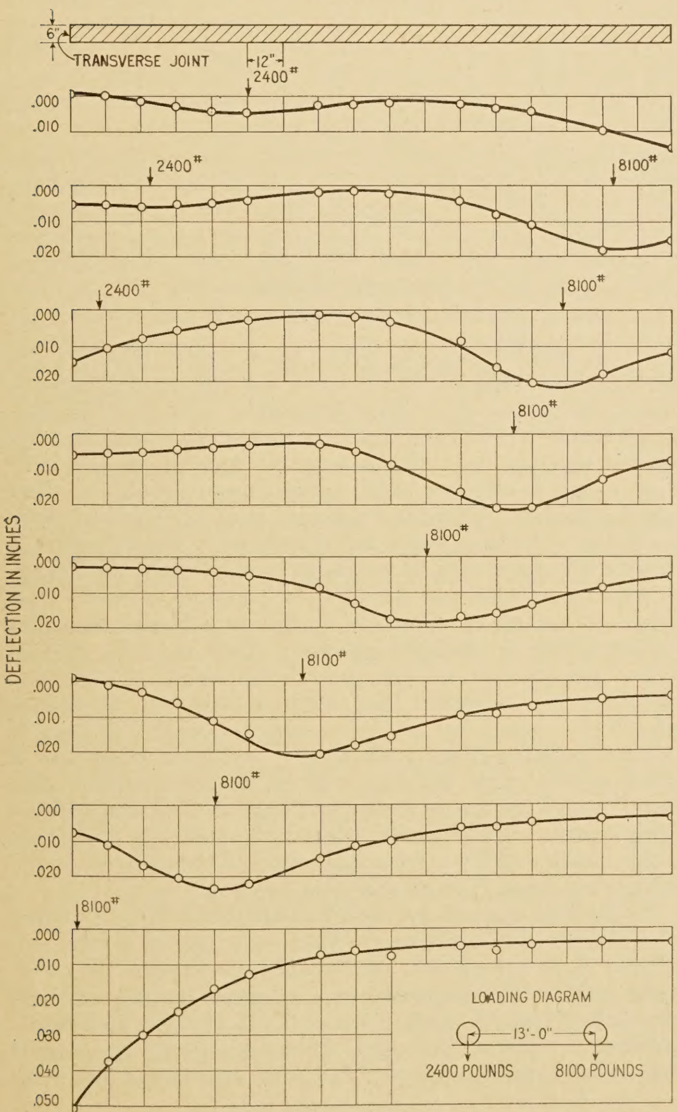

FIG. 8.—Elastic curves of 6-inch concrete pavement under 4-wheel truck in various positions

While so far as this investigation is concerned the elastic curve of the pavement is not of primary importance, two such curves have been included (figs. 8 and 9) to show the extent and nature of the deflection which actually occurs under the four and six wheel trucks respectively.

From these curves it will be seen that the deflection is directly proportional to the load. For instance, a 2,400-pound wheel load at the corner of the slab produced a maximum deflection of 0.015 inch. The rear wheel load of 8,100 pounds passing the same point produced a maximum deflection of 0.051 inch. The same is true for deflections along the edge of the slab away from the corner, although the deflection is only about 40 per cent as great. Very little trouble was experienced with the deflection-measuring apparatus and the quality of the records obtained was excellent.

The data from the strain gauges consisted of graphic records showing the maximum tensile and compressive deformations occurring at the gauge position for each load application. These records in each case were reduced to the maximum unit deformation, an average

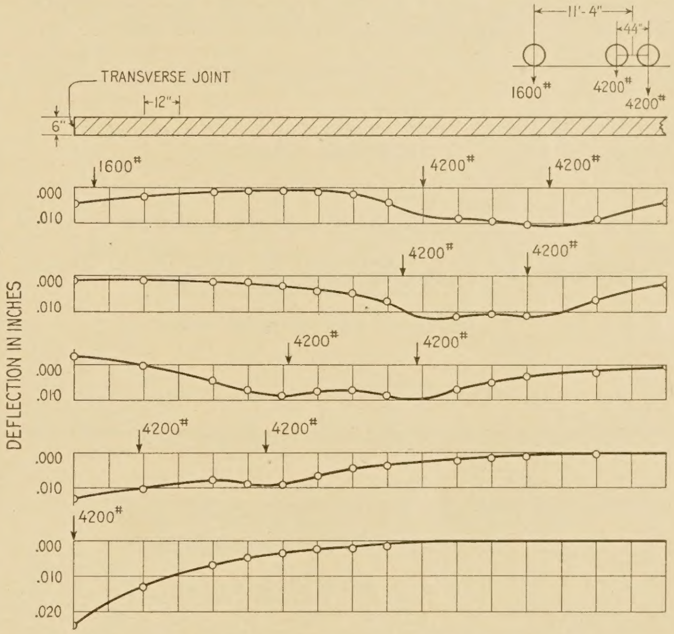

FIG. 9.—Elastic curves of 6-inch concrete pavement under 6-wheel truck in various positions

value over the 6-inch gauge length, and were corrected slightly for the position of the gauge with respect to the surface of the pavement by locating the position of the neutral plane of the pavement and assuming a straight-line variation of stress. These corrections were usually small, as the gauges were placed as near the surface of the slab as possible.

Examining the data from all of the runs with the various vehicles, the stress produced along the edge of the pavement by a wheel load 21 inches from the edge is found to be 47 per cent of that produced by the same load 9 inches from the edge. This percentage varied from 43 to 51 per cent for the various groups of comparable results.

COUNTERFLEXURE UNDER SIX-WHEELERS FAILS TO PRODUCE HIGH TOP STRESSES

The variation in stress along the edge of the pavement for a given load is shown by Figure 10. The diagram shows the maximum edge stresses at the several points resulting from the load as it passes along the edge.

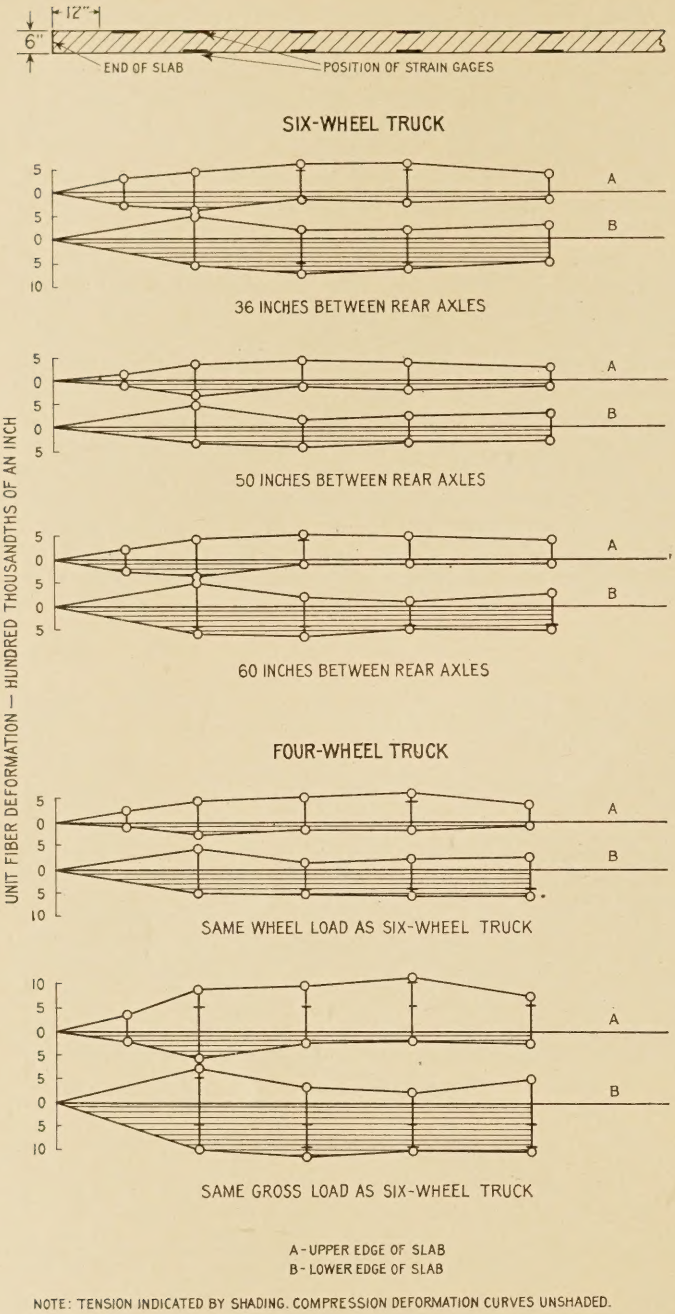

FIG. 10.—The variation in stress along the edge of the pavement resulting from the passage of 6-wheel and 4-wheel trucks. The stresses indicated are the maxima produced at the several points by the loads in any position

The variation has the same characteristics that were shown in a number of tests previously conducted by the Bureau of Public Roads. The significance of these diagrams lies in the fact that they show that for six-wheel trucks the maximum tensile stress occurs in the bottom of the slab regardless of the axle spacing. This means that even though there is counterflexure of the pavement between the wheels the tension developed in the top of the slab is of less magnitude than the tension developed in the bottom of the slab directly under the wheels. This latter tension, then, is the critical stress for the six-wheel as well as the four-wheel vehicle.

This leads to a study of the relative fiber deformations produced by four-wheel and six-wheel vehicles,

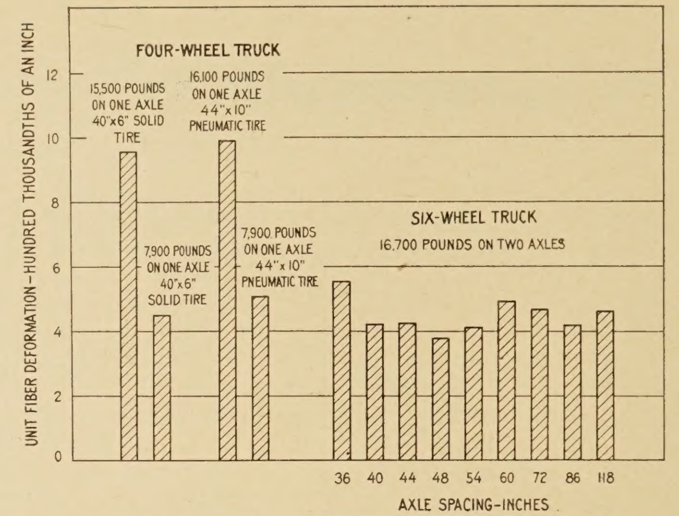

FIG. 11.—Unit fiber deformations of 6-inch concrete slab by 4-wheel and 6-wheel trucks

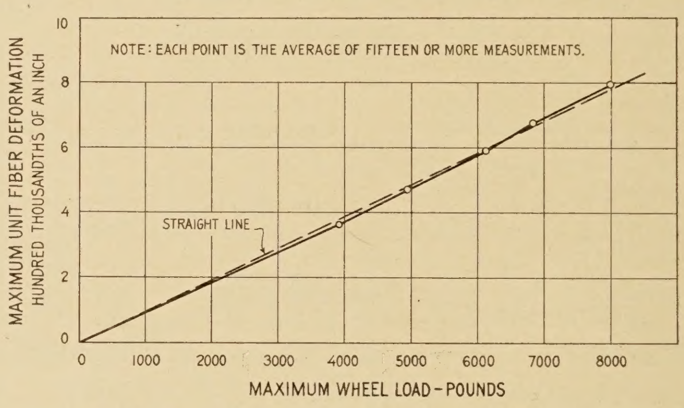

FIG. 12.—The relation between wheel load and fiber deformation

having, first, the same gross loads, and, second, the same wheel loads. These data are presented graphically in Figure 11. The difference between the two four-wheel trucks was only in the area of contact between the tire and the pavement due to the type of tire, and it is obvious that this has no appreciable effect on the stress. The data indicate that for a given gross load the six-wheel truck will cause only about one-half the tensile deformation produced by the four-wheel truck. This seems to be true for all axle spacings with the possible exception of the 36-inch, for which the recorded deformation is slightly greater. This may be due to some peculiarity of this particular test, although it seems more probable that with wheels but 36 inches apart the condition is beginning to approach that of a concentrated load. However, as it is doubtful if such a close axle spacing will be met in practice, this small increased stress would not appear to be important. The significant indication is that the stress in the pavement produced by a six-wheel truck is a function of the wheel load and not of the axle spacing.

Sufficient data were obtained in these tests to show the relation between wheel load and fiber deformation. These data are shown graphically in Figure 12. Fifteen or more measurements were averaged for each of five-wheel loads (4,000, 5,000 6,000, 7,000, and 8,000 pounds). These average values lie along a straight line, with a maximum deviation of less than 4 per cent.

(Continued on page 184)

RAILROAD ABANDONMENTS AND THEIR RELATION TO HIGHWAY TRANSPORTATION

Reported by HENRY R. TRUMBOWER, Economist, UnitedStates Bureau of Public Roads

IN 1916 there were 254,037 miles of railroad in the United States. That was the peak year. Every year since has shown a net decrease in the mileage, so that at the end of 1923 there were only 250,222 miles, or 3,815 miles less than the peak mileage. As during the seven-year period there was some mileage added as well as abandoned, the actual mileage abandoned was greater than 3,815 miles.

This reduction in the railroad mileage of the country has been called to the attention of the public by some writers as an indication of a condition of general stagnation of the railroad business. More recently it has been intimated to be the result of the increased operation of motor vehicles over the improved highways.

In order to determine the significance and economic importance of these abandonments, and especially to ascertain the influence of motor vehicle operation, the Bureau of Public Roads has made a study of the circumstances surrounding the abandonments since 1920, the results of which lead to the conclusion that in but few instances has highway competition been the primary cause of the abandonment. It would be difficult, if not impossible, to determine the causes of the curtailments made before 1920, but since that year the information has been made available as a result of the passage of the transportation act.[1] That act provides for the issuance of certificates of public convenience and necessity by the Interstate Commerce Commission in all cases of contemplated abandonment of lines by any railroad. The facts involved in these abandonment proceedings are fully set forth in the published opinions of the commission, and in that way are made matters of public information.

AVERAGE ABANDONMENT 500 MILES A YEAR SINCE 1920

From 1920, when the act was passed, to May 1, 1925, the Interstate Commerce Commission granted certificates of public convenience and necessity authorizing the abandonment of 120 lines and branches aggregating 2,438.95 miles. The details follow in Table 1.[2]

TABLE 1.—*Mileage of railroad lines and branches abandoned from 1920 to 1925*

Year	Lines		Branches		Total	
	Number	Miles	Number	Miles	Number	Miles
From Nov. 1, 1920	1	30.30	3	23.97	4	54.27
1921	14	548.12	20	144.82	34	692.94
1922	9	387.56	15	196.05	24	583.61
1923	6	305.30	14	152.39	20	457.69
1924	7	180.70	19	248.38	26	429.08
To May 1, 1925	4	157.65	8	63.71	12	221.36
Total	41	1,609.63	79	829.32	120	2,438.95

The line abandonments consist of all those which involved the abandonment of a whole line of railroad; the branch abandonments include those which involved the abandonment of a portion of a company's line of railroad or one of its branches. These authorizations by the commission covered during this period 41 lines of railroad totaling 1,609.63 miles and 79 branches constituting 829.32 miles. Of all the abandonments, in other words, 66 per cent were branches and 34 per cent were lines. The branches, however, constituted only 34 per cent of the abandoned mileage and the lines constituted 66 per cent. The average length of an abandoned line was 39.25 miles and of an abandoned branch only 10.49 miles.

[1] Transportation act, 1920, sec. 1, par. 18.

[2] Finance Reports, Interstate Commerce Commission; vols. 65, 67, 70, 71, 72, 76, 79, 82, 86, 90, and 94.

EAST NORTH CENTRAL STATES SHOW LARGEST ABANDONMENT

The abandoned mileage is not confined to any particular section of the country; in every State except eight there were some abandonments. The States in which no mileage was relinquished were Delaware, Idaho, Kansas, Kentucky, Maryland, Missouri, Nebraska, and Utah. The distribution by geographic sections is shown in Table 2.

TABLE 2.—*Distribution of railroad abandonments by geographic sections* [1]

Group of States	Lines			Branches			Total		
	Number	Miles	Average length	Number	Miles	Average length	Number	Miles	Average length
New England	----	--------	-----	11	54.47	4.95	11	54.47	4.95
Middle Atlantic	1	37.00	37.00	7	54.65	7.81	8	91.65	11.46
East North Central	8	633.78	79.22	18	176.48	9.81	26	810.26	31.16
West North Central	2	153.05	76.52	7	64.08	9.15	9	217.13	24.13
South Atlantic	10	220.95	22.09	8	27.65	3.45	18	248.60	13.81
East South Central	4	166.61	41.65	4	87.98	21.99	8	254.59	31.82
West South Central	10	233.34	23.33	10	120.56	12.06	20	353.90	17.69
Mountain	2	33.50	16.75	8½	[2] 118.37	13.92	10½	[2] 151.87	14.46
Pacific	4	131.40	32.84	5½	[2] 125.08	22.74	9½	[2] 256.48	26.99
Total	41	1,609.63	39.25	79	829.32	10.49	120	2,438.95	20.32

[1] New England States: Maine, New Hampshire, Vermont, Massachusetts, Connecticut, and Rhode Island. Middle Atlantic States: New York, New Jersey, and Pennsylvania. East North Central States: Ohio, Indiana, Illinois, Michigan, and Wisconsin. West North Central States: Minnesota, Iowa, Missouri, North Dakota, South Dakota, Kansas, and Nebraska. South Atlantic States: Delaware, Maryland, West Virginia, Virginia, North Carolina, South Carolina, Georgia, and Florida. East South Central States: Kentucky, Tennessee, Alabama, and Mississippi. West South Central States: Arkansas, Oklahoma, Louisiana, and Texas. Mountain States: Montana, Idaho, Wyoming, Utah, Colorado, New Mexico, Arizona, and Nevada. Pacific States: Washington, Oregon, and California.

[2] One abandoned branch extended between California and Nevada.

The smallest mileage abandoned was in the New England States, where only 54.47 miles were relinquished in 11 branches, of 4.95 miles average length. In the East North Central States there were 26 abandonments, involving 810.26 miles, or an average length of 31.16 miles; this is the largest mileage found in any of the sections, and is due in a large measure to the abandonment of eight whole lines in this section. The abandonment of the Indiana coal division of the Chicago & Eastern Illinois Railroad, comprising 162.1 miles, and the Chicago, Peoria & St. Louis Railroad in Illinois with its 234.3 miles of line, constituted a large portion of the abandoned mileage in this section.

In analyzing this situation the abandoned mileage in the various sections should be compared with the total railroad mileage. In this period, extending over almost five years, the 2,438 miles of abandoned railroads appear large when considered alone. If the trackage abandoned were brought together and placed end to end it would reach from New York City to Sacramento, Calif. In other words, it is equivalent to a transcontinental line of railroad, or, compared in another way, it is almost equivalent to the Wabash Railway system with its 2,476 miles of line. But when comparison is made with the total railroad mileage of the country these abandonments do not take on such a large aspect. On December 31, 1919, which was just prior to the passage of the law making it necessary for the railroads to file applications for the right to abandon lines with the Interstate Commerce Commission, the railroad mileage of the country was reported as 253,152 miles. For the country as a whole, therefore, the mileage abandoned since 1920 is less than 10 miles for every 1,000 miles in existence at the end of 1919.

In Table 3, in which the abandoned mileage is compared with the total railroad mileage as of December 31, 1919, for the various groups of States, it will be seen that the abandonments in the Middle Atlantic States were relatively the smallest, being only 4.1 miles per thousand, although the relative abandonments were approximately the same for the West North Central States. The greatest abandonment relative to the 1919 mileage took place in the East North Central States, where the abandoned mileage amounted to 18 miles for every thousand miles in existence in 1919.

TABLE 3.—*Comparison of abandoned railroad mileage with the total railroad mileage as of December 31, 1919*

Group of States	Total mileage of railroad, Dec. 31, 1919	Total mileage abandoned, 1920–1925	Abandoned mileage per 1,000 miles of road
	Miles	*Miles*	*Miles*
New England	7,985.59	54.47	6.8
Middle Atlantic	22,362.17	91.65	4.1
East North Central	44,868.75	810.26	18.0
West North Central	52,176.46	217.13	4.2
South Atlantic	32,434.56	248.60	7.6
East South Central	17,783.91	254.59	14.3
West South Central	32,972.64	353.90	10.7
Mountain	25,314.65	151.87	6.0
Pacific	17,253.44	256.95	14.8
Total	253,152.17	2,438.95	9.6

CAUSES OF SECTIONAL DIFFERENCES IN MILEAGE ABANDONED

The relative abandonments differ rather widely in the several groups of States because of the variations in the local conditions under which the railroad lines were built. In general it can be said that the relative abandonments in the States east of the Mississippi River were greater than in the States west of the Mississippi. For every thousand miles of railroad in the Eastern States 11.6 miles were abandoned and for every thousand miles west of the Mississippi only 7.6 miles were abandoned. The railroad mileages in these two broad sections of the country at the end of 1919 were almost the same; in the Eastern States there were 125,434.98 miles and in the Western States 127,717.19 miles. But the abandonments in the Eastern States were 1,459.47 miles, 49 per cent greater than the abandonment of 979.48 miles in the Western States. This difference can be largely explained by the fact that the railroads are older in the Eastern States, and a great many of the abandonments consisted of lines or branches of railroads built originally for the purpose of shipping to market various natural resources, the supply of which later became exhausted and no other traffic had been developed to warrant the continued operation. That type of railroad extension and development was not prevalent to the same extent in the Western States.

While the average length of the abandoned lines and branches has already been touched upon, it is of interest to determine how many were relatively short, and how many were more or less extensive. It will be readily seen from the classification in Table 4 that by far the largest number consisted of short stretches. Fifty-seven of the abandoned roads, constituting 47.5 per cent of the total number abandoned, were less than 10 miles long; those between 10 and 29 miles in length, 44 in number, were 36.6 per cent of the total. The remainder, or 19, ranging in length from 30 to 234 miles, constitu ed only 15.9 per cent of the abandonments. The lines abandoned are naturally of much greater average length than the branches.

TABLE 4.—*Classification of abandoned railroad lines and branches according to length, 1920–1925*

Length	Number of lines	Number of branches	Total
Miles			
0–4		25	25
5–9	4	28	32
10–14	9	9	18
15–19	8	7	15
20–24	2	3	5
25–29	3	3	6
30–34	1	1	2
35–39	1	1	2
40–44	2		2
50–54	4	1	5
55–59		1	1
75–79	1		1
90–94	2		2
95–99	2		2
160–164	1		1
230–234	1		1
Total	41	79	120

LOGGING AND MINING ROADS A MAJORITY OF THOSE ABANDONED

While the purposes for which these abandoned roads were built are not indicated in every case, it is clear that a majority were originally constructed as logging or mining roads. In some cases railroads already in operation built extensions or branch lines into certain sections or territories for the main purpose of tapping timber or mineral resources and in that manner sought to secure traffic consisting of the raw materials or products of such resources in their finished or semifinished state. In a great many cases new lines of railroad were built for this purpose and connected with lines or branches already in existence. The natural resources exhausted and traffic of other sorts having failed to develop, these lines and branches have been left without sufficient business to warrant continuance. That is the simple explanation of the majority of the abandonments.

In Table 5 the abandonments are classified according to the purpose of construction in order to show clearly the relative number of logging and mining roads.

The logging and mining roads, both lines and branches, aggregated 1,351.30 miles, or 55.4 per cent of the total abandoned mileage; and of these 1,351.30 miles 62.8 per cent were logging roads. Although the abandoned mileage of logging roads was greater than that of the abandoned mining roads, the mining roads were of greater average length. The lines built as logging roads and abandoned in this period averaged 25 miles in length; the lines built as mining roads averaged 53.3 miles. The same relative difference is noted in the average length of branch roads. The logging roads averaged 9.2 miles in length and the mining roads 16 miles. Of the 28 lines of logging and mining roads which were abandoned 24 were logging roads and only 4 mining roads. The tendency to build separate and individual lines of railroad to tap mineral resources was not as marked as the tendency to build such lines in order to market timber products.

TABLE 5.—*Railroad abandonments classified according to purpose of construction*

	Lines		Branches		Total	
	Number	Miles	Number	Miles	Number	Miles
Logging roads	24	600.66	27	248.59	51	849.25
Mining roads	4	213.20	18	288.85	22	502.05
Logging and mining roads	28	813.86	45	537.44	73	1,351.30
All others	13	795.77	34	291.88	47	1,087.65
Total	41	1,609.63	79	829.32	120	2,438.95

In most instances the lines of logging and mining railroads were projected and constructed by companies or interests engaged in the lumber or mining business and the railroad companies were in fact subsidiaries of such industrial companies. The branch lines abandoned were constructed in most cases by railroad companies which sought in this manner to obtain additional traffic and to meet the industrial and transportation needs of shippers.

MOTOR VEHICLE COMPETITION A MINOR CAUSE OF ABANDONMENT

The causes leading to the abandonment of this railroad mileage are in some instances simple and easily determined; in other instances the causes are more or less complicated. The lack of sufficient traffic to continue making the operation profitable or even possible is the fundamental reason for abandonment. The causes for the lack of traffic are varied. The primary causes cited are: (1) Exhaustion of natural resources; (2) competition of other railroads; (3) competition of motor vehicles operating on highways; (4) rearrangement of lines of railroad; and (5) miscellaneous causes. The reasons for the abandonments, grouped according to this classification, are summarized in Table 6.

TABLE 6.—*Primary causes of lack of traffic*

Cause	Number of railroads	Percentage of number	Length	Percentage of length
		Per cent	Miles	Per cent
Exhaustion of natural resources	78	65.0	1,411.20	57.8
Competition of other railroads	14	11.7	713.34	29.3
Competition of motor vehicles	10	8.4	104.46	4.3
Rearrangement of lines of railroad	5	4.1	32.64	1.3
Miscellaneous	13	10.8	177.31	7.3
Total	120	100.0	2,438.95	100.0

The outstanding fact deduced from this analysis and summary is that the exhaustion of the natural resources which furnished the bulk of the traffic for these abandoned lines and branches of railroads accounted for 65 per cent of the number of abandonments and for 57.8 per cent of the abandoned mileage. This one factor was the principal cause of the abandonment of considerably more than half of the total abandoned mileage. Competition of other railroads resulted in traffic losses which brought about the abandonment of almost 30 per cent of the total abandoned mileage, and it is of special interest to note that less than 5 per cent of the abandoned mileage can be attributed to motor-vehicle competition as the primary cause.

REASONS FOR ABANDONMENT ILLUSTRATED

To illustrate these primary causes typical cases are cited. The Alabama & Mississippi Railroad Co. was authorized October 21, 1921, to abandon its line from Vinegar Bend, Ala., to Pascagoula, Miss., a distance of 75.5 miles.[3] The railroad was built to market timber products. At the time of abandonment the territory traversed consisted largely of cut-over timber lands. The agricultural development was so small that only three carloads of agricultural products originated on the line in 1920. The passenger traffic was negligible. For five years prior to 1920 the line incurred net operating deficits. The principal communities on the line were served by other railroads and suffered no hardship as a result of the abandonment. The road was in the hands of a receiver and no objection was made to the proposed plan of abandonment.

For similar reasons the Pere Marquette Railway Co. was authorized October 21, 1921, to abandon a branch line from Harrison to Leota, Mich., 9.8 miles in length.[4] It had been built in 1897 as a logging road. The timber had become exhausted and the country tributary to the branch was sparsely populated. No cities or towns had grown up, and after the lumber was all cut and moved the traffic came practically to a standstill, making the operation of the branch line no longer necessary. No opposition to abandonment was encountered. This is an illustration of what happened in most cases where branch roads, built primarily to take care of the transportation of certain natural resources, were abandoned when these resources became exhausted and no other kind of traffic had developed in amount sufficient to keep up railroad operation.

An example of the abandonment of a line of railroad brought about primarily by the competition of other railroads is the case of the Chicago, Peoria & St. Louis Railroad, extending from Pekin to East St. Louis, a distance of 234.30 miles, through one of the richest agricultural sections of Illinois.[5] This was the longest section of railroad abandoned in the five-year period. It passed through 35 cities and villages, 20 of which had no other rail transportation facilities. It was an old line of railroad, a part of which was constructed prior to the Civil War. It had passed through several receiverships and was in the hands of a receiver again at the time the abandonment proceedings were brought.

The principal reason advanced for the decrease in traffic was the loss of freight tonnage to other railroads brought about through short routing. The local business was not sufficient to maintain the road in operation and the connecting railroads were not willing to divert any of their through traffic voluntarily. During the war the road's freight-soliciting agencies in outside

[3] 70 I. C. C. 531. [4] 70 I. C. C. 535. [5] 76 I. C. C. 801.

cities were done away with, and when the road was returned to its owners the business formerly secured in that manner could not again be obtained. The lack of earnings also resulted in undermaintenance of road bed and rolling stock, so that the service became poorer and poorer. Motor-vehicle competition was also cited as one of the reasons for a loss of a part of the local traffic. Highway transportation could not, however, have been regarded as the primary cause leading to abandonment. The highways in the territory were greatly improved and these improved highways made it possible for communities dependent on this line of railroad for service to have their transportation needs satisfied without any undue hardships. The average distance to other railroads from the 20 points which were not on another line was only 6.6 miles. The main reason leading to abandonment was that the railroad was located in too highly competitive railroad territory. The lack of sufficient traffic was evident even prior to the time when competition of the highways became pronounced.

ABANDONMENTS DUE TO HIGHWAY COMPETITION CITED

The abandonment of the Ocean Shore Railroad in California was caused primarily by highway competition, according to the commission's report.[6] The road extended from San Francisco to Tunitas Glen, a distance of 53.6 miles. It was constructed in 1909, but in recent years the traffic had diminished steadily on account of the increasing competition of motor vehicles. Before the road was in operation two years, receivership proceedings were had. According to the evidence in the case, the gross revenues never exceeded the operating expenses. In this case it is a question whether the railroad could have been successfully operated even if highway transportation had not entered as a competitive factor.

The Boston & Maine Railroad Co. was authorized to abandon several of its branch lines leading from the main line to summer resorts.[7] The branch extending to the Profile House, N. H., 9.11 miles in length, was allowed to be abandoned because automobile traffic had reduced the passenger traffic to almost nothing. This was brought about through the construction of a good highway from the Profile House to stations on the company's main line. The year's revenue was given at $1,713 and the operating expenses $11,227. It is questionable whether this section of railroad, operated only during the summer months, ever yielded revenues in excess of operating expenses.

The construction of an improved highway the length of Barnegat Island, N. J., made it possible for the Pennsylvania Railroad Co. to discontinue operating its branch line, 8.12 miles long, on that island.[8] The highway paralleled the whole length of the railroad. Trucks and automobiles meet all the traffic demands. The record shows that the railroad was operated at a loss continually since 1894, when it was constructed. When the improved highway took so much of the traffic that there was scarcely any left for the railroad to carry, it also made it possible for the railroad company to be relieved of a financial burden which had been a drain upon the net revenues of the system as a whole. The public appeared to be satisfied with the abandonment, since there was no strenuous opposition to the discontinuance of service from any source at the time of the hearing.

[6] 67 I. C. C. 760. [7] 70 I. C. C. 266. [8] 79 I. C. C. 506.

In a few cases abandonments were brought about through rearrangements in line or through new construction which resulted in the shortening of routes, making the old stretches of railroad no longer necessary. An example was the abandonment of a branch of the Delaware & Hudson Co. in New York which formerly was a part of its main line. A short line was built which made this section of the road a branch, with no through traffic to carry.[9] Improved highways have been constructed by which the public's transportation needs are adequately met, and it was thus made possible to abandon this branch without interfering with the public convenience and necessity. There were five abandonments of this character.

A number of abandonments were authorized where it was shown that there was not sufficient traffic, though no definite reasons were given for traffic deficiencies or decreases. A case in point was the abandonment of the Orangeburg Railway, a line 17.7 miles long, extending from Orangeburg to North, S. C.[10] The road, built in 1913, was originally planned to extend to Charleston. It never earned its fixed charges. As early as 1916 it was already in the hands of a receiver. There was no evidence that traffic was taken away from it by other railroads or by highways. The plain facts were that there never was enough business for a railroad in the territory and that it should not have been built in the first place.

Another example of an abandonment of this character was that of the branch of the Chicago, Milwaukee & St. Paul Railway extending from Cogswell to Harlem, N. Dak., a distance of 5 miles.[11] Harlem at one time had two general stores and two grain elevators. The stores have been discontinued; one of the elevators was dismantled, and the other was destroyed in a windstorm and was not rebuilt. The farmers operating in the territory traversed by this branch do their business in the towns on the main line of the railroad. Thus it was no longer necessary to continue the operation of the branch, because nobody's convenience and necessity was served by it.

THE RIGHT TO ABANDON REFUSED IN SOME INSTANCES

Not every application for abandonment was granted by the commission. One line of railroad with a mileage of 40.80 miles and 8 branches, involving a mileage of 75.76 miles, was denied the authority to abandon. In the case of the Frankfort & Cincinnati Railway, which desired to abandon its line from Frankfort to Paris, Ky., the commission found that, while there had been a loss in traffic due to the closing down of distilleries and the use of motor vehicles on the highways, it seemed possible for the company, by making additional efforts, to get more through business and that a large number of people were dependent on the railroad because of poor roads in certain sections served by it.[12] It was further brought out that the Louisville & Nashville Railroad Co. owned the majority of the stock of this road, and that it could therefore be regarded as that road's subsidiary.

The other denials all involved large railroad companies which desired to abandon certain branch roads because of their unprofitableness.[13] In denying au-

[9] 94 I. C. C. 376.
[10] 67 I. C. C. 789.
[11] 82 I. C. C. 274.
[12] 86 I. C. C. 740.
[13] 71 I. C. C. 225; 72 I. C. C. 25; 72 I. C. C. 267; 72 I. C. C. 303; 79 I. C. C. 431; 86 I. C. C. 637; 86 I. C. C. 609; 94 I. C. C. 624.

thority to abandon these branches the commission invariably found that the branches were necessary to meet the present and future transportation demands of the communities affected and that, while operating deficits were shown, "a railroad company could not claim the right to earn a net profit from every section into which its road might be divided.

The jurisdiction and power of the Interstate Commerce Commission to grant authority to a railroad to abandon a line of railroad or a part thereof was challenged in several instances by State authorities. After the commission had issued its order [14] authorizing the abandonment of the line owned and operated by the Eastern Texas Railroad Co., the State of Texas brought a suit in the United States district court against the commission, the railroad company, and others in which the annulment of the commission's order was sought. The district court decided against the State, whereupon the State appealed the case to the United States Supreme Court. The Eastern Texas Railroad was wholly an intrastate road, and it was claimed that paragraph 18, section 402 of the transportation act did not give the Interstate Commerce Commission the power to authorize the abandonment of a railroad with respect to intrastate commerce. The Supreme Court in reversing the order of the lower court held [15] that the continued operation of the road in intrastate service was of local concern and that any shortage of earnings would not affect or place a burden upon interstate commerce because no carrier engaged in interstate commerce would be required to make up any deficits. The court interpreted this section of the transportation act to mean that the commission did not have the authority to grant abandonment certificates so far as purely intrastate business was concerned and where neither interstate nor foreign commerce was burdened thereby.

Whether the railroad in this case was entitled to discontinue its intrastate business was not considered by the court in that particular proceeding. This question, however, had to be decided by the court within a period just long enough for another appealed case to be brought to the court's attention.

RIGHT TO ABANDON UNPROFITABLE INTRASTATE OPERATIONS AFFIRMED

The Eastern Texas Railroad having been authorized to abandon its interstate traffic now proceeded to discontinue its intrastate business as well as its interstate business. Two injunction suits were filed forthwith in the United States district court; in one the State of Texas asked for an injunction against the railroad company to prevent it from dismantling and abandoning its railroad; in the second suit an injunction was sought by the railroad company to prevent the State authorities from interfering with the company's abandonment plans. The district court granted the injunction applied for by the railroad company and denied the State's request. In the appeal to the United States Supreme Court it was there argued by the State, as in the lower court, that the company's charter required it to operate for a period of 25 years, which had not yet expired, and that it was liable for damages if it failed to continue operating. A law passed by the Texas Legislature which required "all railroads within the State to operate at least one passenger train a day" was also cited as being a part of the company's charter requirements, even though passed subsequent to the granting of the charter. In the testimony in the district court it was shown that the operating costs were $84,000 a year and that the estimated revenues would not be in excess of $50,000, of which not more than $20,000 would be derived from intrastate business. The company maintained that a requirement to operate would, in the light of these facts, deprive it of its property without due process of law and in violation of the provisions of the fourteenth amendment. The Supreme Court affirmed the action of the lower court and held that the charter was in the nature of a permit and did not require its holder to operate at a loss nor could the "train-a-day" law be considered as more than one of the regulations of the State under its police powers.[16]

After the first of these Texas Railroad Co. decisions by the Supreme Court the Interstate Commerce Commission, in granting certificates authorizing the abandonment of lines of railroad situated wholly within a State, always limited its orders to the discontinuance of operations with respect to interstate commerce.

In opposing the application of the Southern Pacific Co. for authority to abandon one of its branches located wholly in Nevada the authorities of that State claimed that the Interstate Commerce Commission was without jurisdiction because of the intrastate character of the branch.[17] It was shown that the branch was operated at a loss and that there was no great demand for its service. In that case the commission held that even though the branch was located wholly within the State it constituted a part of the Southern Pacific Co.'s interstate system of transportation and that the losses incurred on this branch would be reflected in the accounts of the company and might in that manner be regarded as a possible burden upon interstate commerce.

Although, as has been shown, the Interstate Commerce Commission has authorized the abandonment of a large railroad mileage in the last five years, it should be borne in mind also that during this same period [18] certificates of public convenience and necessity were issued by the commission authorizing the new construction of 2,673.36 miles of railroad. This involved the construction of 17 lines aggregating 1,159.52 miles and 67 branches aggregating 1,513.84 miles. How many miles were actually constructed can not be determined from the official reports, but it is reasonable to assume that a substantial portion of this mileage was built and put into operation.

It is generally believed that highway transportation will be considered an important element in the future railroad development of the country. Highway transportation may make it necessary to make railroad extensions into certain territories and it will also make it possible to relinquish the operation of certain low-traffic lines and branches, thus saving money for the railroads and providing the public with reasonably adequate transportation service.

[14] 65 I. C. C. 436.
[15] 258 U. S. 204.
[16] 264 U. S. 79; 283 Fed. 584.
[17] 72 I. C. C. 404.
[18] Nov. 1, 1920, to May 1, 1925.

THE ACTION OF SULPHATE WATER ON CONCRETE[1]

A SUMMARY OF TESTS OF SPECIMENS IMMERSED FOR ONE YEAR IN MEDICINE LAKE, S. DAK.

Reported by DALTON G. MILLER, Drainage Engineer, United States Bureau of Public Roads

THE field work of 1919 and 1920 [2] resolved the so-called concrete-alkali problem of Minnesota, as related to the use of concrete drain tile into a problem of the effect of the sulphates of magnesium ($Mg\,SO_4$) and sodium ($Na_2\,SO_4$). Subsequent to this work a cooperative laboratory was established at University Farm at St. Paul, Minn., in 1921, and experiments have been there conducted by the department of agriculture of the University of Minnesota, the State Department of Drainage and Waters, and the United States Bureau of Public Roads. These laboratory experiments were designed principally to aid in the general improvement of farm drain tile and particularly to develop tile that will endure under the wide range of soil conditions peculiar to Minnesota. The results are applicable, however, to many other sections of the United States and to concrete culvert pipe subjected to the action of sulphate waters under conditions similar to those to which drain tile are subjected.

At the cooperative laboratory several hundred 2 by 4 inch concrete and mortar cylinders of various types were made and stored in earthenware jars containing solutions of the two sulphate salts. Much has been learned from this work and certain conclusions seem to be warranted. It is recognized, however, that any laboratory results to be of value must be supported by tests under field conditions, and it was with this in view that the cylinders upon which this report is based were installed in Medicine Lake, S. Dak., 17 miles northwest of Watertown. In choosing a suitable site for field installations this lake was selected as being very nearly ideal for the purpose in the essentials of kinds and quantities of salts and, also, as being reasonably accessible.

[1] University of Minnesota Paper No. 562, Journal Series.

[2] Report of concrete-alkali investigations in Minnesota, 1919–20. Department of Drainage and Waters, State of Minnesota.

ANALYSES OF MEDICINE LAKE WATER

Analyses of water samples taken from Medicine Lake at different seasons of the year are given in Table 1. As indicated, the total salt content has ranged between 2.34 and 4.72 per cent, consisting almost entirely of a combination of magnesium and sodium sulphates in which the magnesium salt greatly predominates. The larger of these two figures represents water with a total salt content slightly in excess of any soil water so far found in contact with any drain tile in Minnesota and may safely be assumed to be as severe as is ordinarily encountered in agricultural lands anywhere.

As in the previous laboratory tests the specimens used were 2 by 4 inch cylinders, the majority of concrete, but a number also of Ottawa sand mortar. As suggested by the previous laboratory work, in which it was found that proper curing is one of the most essential factors in developing resistance to the action of sulphate waters, the samples installed in the lake represent a wide range of curing conditions, including storage for varying periods in the moist closet, in steam, in water vapor at various temperatures, in tap water and distilled water, and in air, and various combinations of the several treatments.

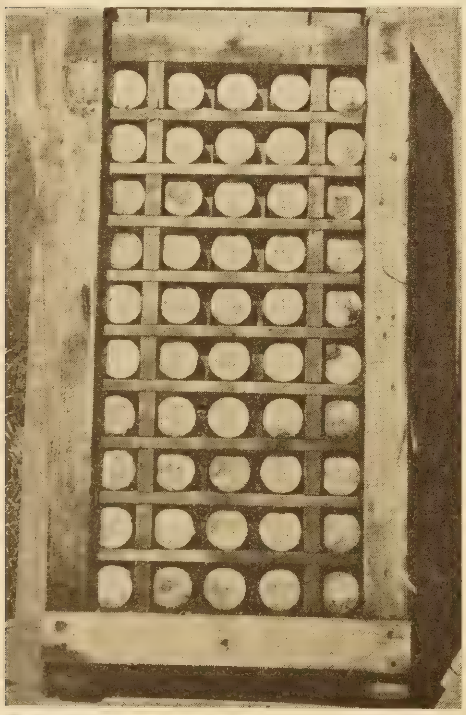

FIG. 1.—Cylinders from series 77 to 81, cured in water vapor at 212° F.

FIG. 2.—Cylinders from series 82 to 86, cured in water vapor at 155° F.

FIG. 3.—Cylinders from series 87 to 91, cured in water vapor at 100° F.

Portland cement concrete cylinders after 18 months in Medicine Lake, S. Dak. Cylinders in each crate were identical except for curing

Table 1.—*Analyses of water from Medicine Lake, S. Dak.*

(Analyses by the Water and Beverage Laboratory, Bureau of Chemistry, U. S. Department of Agriculture)

Sample No.	Date taken	Radicals								
		Na (Calc.)	Ca	Mg	NO_3	Cl	SO_4	CO_3	HCO_3	Total
		Milligrams per liter (parts per 1,000,000)								
778.2	Dec. 10, 1923	3,755	1,006	5,012	1	580	28,898	84	282	39,618
781.2	Feb. 14, 1924	4,344	1,082	6,116	Trace.	596	34,533	54	490	47,215
803.2	Apr. 29, 1924	395	344	4,276	Trace.	396	17,585	98	323	23,417
808.2	July 1, 1925	3,649	436	4,910	Trace.	464	27,067	114	158	36,828
		Percentage reacting values								
778.2		13.05	4.01	32.94	0.00	1.31	48.10	0.22	0.37	100.00
781.2		12.67	3.62	33.71	.00	1.13	48.22	.12	.53	100.00
803.2		2.23	2.22	45.55	1.00	.45	47.44	.42	.69	100.00
808.2		13.58	1.86	34.56	.00	1.12	48.25	.41	.22	100.00

Besides varying the curing treatment several different cements were tried including three high-alumina brands, and a number of the specimens were designed to test the effect of various admixtures and impregnations. The number of types of cylinders installed in the lake has purposely been kept to the minimum, keeping in mind as the principal purpose the development of a concrete suitable for use in the commercial manufacture of ordinary farm drain tile, and following the lines indicated as most hopeful by the laboratory experiments. Because of these conditions, the 2 by 4 inch concrete cylinders referred to as standard laboratory cylinders, which constitute the majority of the specimens, were made with a well-graded aggregate the screen size of which was less than three-eighths inch. Also, in nearly all cases, the mix was 1:3 and the relative consistency 1. A 1:3 mix is as rich as is ordinarily used for drain tile, and it has been found by the laboratory work that a leaner mix than this is very much less resistant. A richer mix, while more resistant, is rarely used, as the cost of such tile generally is prohibitive. The relative consistency of 1 is as wet as can be used in machine-molded tile, and it has been found that consistencies greatly varying from this are considerably less resistant.

In addition to these standard laboratory cylinders of concrete a number of mortar cylinders made with standard Ottawa sand were also placed in the lake with the idea that they would fairly well represent the poorly graded aggregate that is too often used in the smaller sizes of tile.

HIGH-ALUMINA AND STEAM-CURED CYLINDERS BEST AFTER ONE YEAR IN LAKE

All data with respect to each series of specimens are given in Tables 2 and 3. No effort will be made at this time to analyze in detail all the results of the examinations and tests of the cylinders which have now been stored in the lake for one year, as the exposure period is too short to warrant the drawing of definite conclusions with respect to many of the series. From even a cursory study of the data, however, certain significant facts are evident and these may be briefly summarized as follows:

1. None of the high-alumina cement cylinders showed evidence of any deleterious action. One hundred and twenty-five of such cylinders in 25 series, variously cured and stored in the lake for one year, were tested for compression, together with the same number of check cylinders stored for the same period in the laboratory tank. In 21 series the strength ratio of the cylinders from the lake in comparison with those from the tank exceeded 100 per cent, in 1 series it was 99 per cent, in 2 it was 98 per cent, and in 1 it was 94 per cent. The maximum for any series was 144 per cent, while the average for the 25 series was 115 per cent. It is possible that the 15 per cent excess strength of the cylinders from Medicine Lake is due, at least in part, to the generally lower temperature of the lake water as compared with that of the laboratory tank. It is known that the reaction to temperature of high-alumina cement is considerably different from that of standard Portland cement, as the time required for both the initial and final set of the high-alumina brands is materially shortened by reduction, within certain limits, of the temperature of the curing medium, which is contrary to the action of standard Portland cement. It is also possible that the sulphates of the lake water may have had some accelerating action.

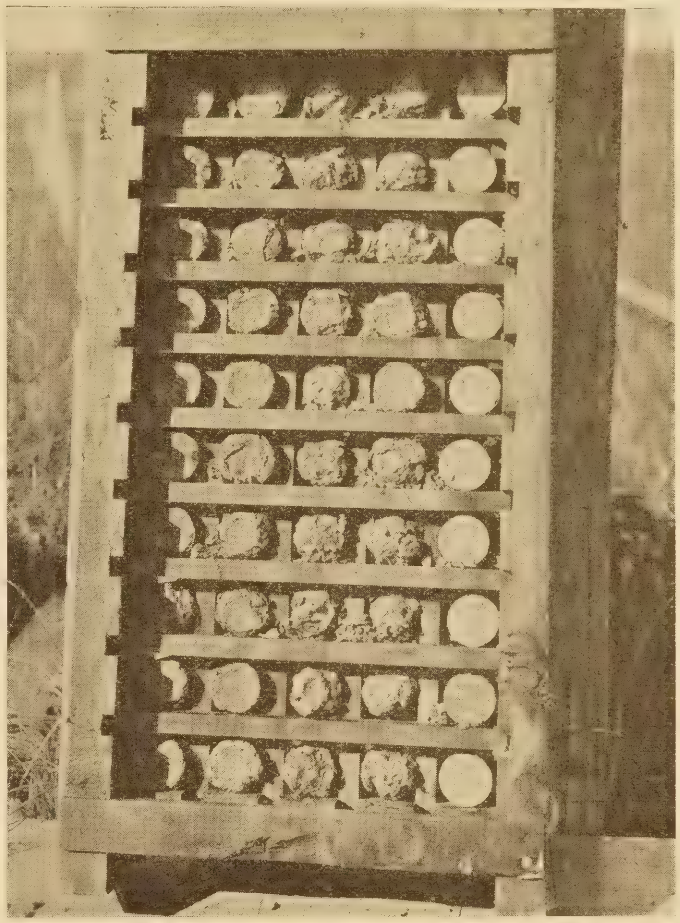

Fig. 4.—From left to right, after 24 hours in the moist closet, cylinders in each row were cured as follows:

Series 92.—Tap water 27 days, air 28 days
Series 93.—Moist closet 2 days, air 25 days
Series 94.—Water vapor at 100° F. 2 days, air 25 days
Series 95.—Water vapor at 155° F. 2 days, air 25 days
Series 96.—Water vapor at 212° F. 2 days, air 25 days

Portland cement concrete cylinders after 18 months in Medicine Lake, S. Dak. Cylinders were identical except for the curing conditions.

2. None of the standard Portland cement cylinders cured in steam at a temperature of 212° F., regardless of the length of time they were so cured or of other variables, showed any surface action, those from the lake looking fully as good at one year as the ones from the laboratory tank. Ninety-five cylinders from 19 series after one year in the lake had an average strength

TABLE 2.—*Description and tests of all laboratory standard concrete cylinders*

(Fineness modulus of aggregate for all cylinders 4.67. Mix 1 : 3 except as noted for 4 series)

Series No.	Cement laboratory No.[1]	Cement	Admixture or impregnation	Curing method: Time in moist closet	Curing method: Time in steam at 212° F.	Curing method: Time in water vapor	Curing method: Temperature of water vapor	Curing method: Time in water	Curing method: Time in air	Absorption at 21 days	Average compression tests: 7 days	Average compression tests: 28 days	1 year, Tank specimens: Number	1 year, Tank specimens: Strength	1 year, Lake specimens: Number	1 year, Lake specimens: Strength	Strength ratio	Length increase: Number	Length increase: Increase
				Hours	*Hours*	*Hours*	*° F.*	*Days*	*Days*	*P. ct.*	*Lbs.*	*Lbs.*		*Pounds*		*Pounds*	*Per cent*		*Inch*
77	11.4	One-half Northwestern, one-half Universal.		3	69				25	7.95	1,110	1,250	5	1,250	5	1,210	97	10	+0.0006
78	11.4	do.		6	66				25	7.18	2,260	3,040	5	3,130	5	2,840	91	10	−.0008
79	11.4	do.		12	60				25	6.86	3,400	4,120	5	4,550	5	4,410	97	10	−.0015
80	11.4	do.		24	48				25	6.94	3,060	4,020	5	4,600	5	4,330	94	10	−.[illegible]5
81	11.4	do.		48	24				25	6.68	2,910	3,850	5	4,630	5	4,410	95	10	−.0016
82	11.4	do.		3		69	155		25	7.0	3,365	4,210	5	5,380	5	500	9		
83	11.4	do.		6		66	155		25	6.9	3,840	4,720	5	5,700	5	490	9		
84	11.4	do.		12		60	155		25	6.7	4,440	4,960	5	5,250	5	1,030	20		
85	11.4	do.		24		48	155		25	6.6	4,355	4,550	5	6,460	5	1,220.	19		
86	11.4	do.		48		24	155		25	6.7	3,545	4,210	5	6,250	5	1,660	27		
87	11.4	do.		3		69	100		25	6.5	3,730	4,120	5	5,110	5	1,430	28		
88	11.4	do.		6		66	100		25	6.7	3,590	4,320	5	4,960	5	1,370	28		
89	11.4	do.		12		60	100		25	6.7	4,050	4,680	5	5,270	5	1,290	25		
90	11.4	do.		24		48	100		25	6.7	4,080	4,220	5	5,240	5	1,630	31		
91	11.4	do.		48		24	100		25	6.8	3,510	4,060	5	5,010	5	1,920	38		
92	11.4	do.		24				[2] 27	28	6.7	2,620	4,540	5	5,690	5	3,780	66	10	−.0001
93	11.4	do.		72					25	7.1	3,380	3,560	5	5,120	5	3,150	62	9	+.0014
94	11.4	do.		24		48	100		25	6.8	3,810	4,000	5	5,640	5	2,020	36	9	+.0021
95	11.4	do.		24		48	155		25	6.4	4,240	4,370	5	5,510	5	1,660	30	7	+.0059
96	11.4	do.		24	48				25	6.5	3,770	4,410	5	4,560	5	4,300	94	10	−.0004
97	11.4–8.4	do.		24		48	155		25	6.3	3,900	4,770	5	5,690	5	1,880	33	7	+.0031
98	11.4–8.4	do.		24	24	24	155		25	6.6	4,180	4,120	5	5,020	5	4,340	87	10	−.0004
99	11.4–8.4	do.		24		72	155		24	6.4	4,490	4,830	5	5,410	5	1,530	28	10	+.0085
100	11.4–8.4	do.		24	24	48	155		24	6.2	4,310	4,960	5	5,730	5	5,540	97	10	−.0001
101	11.4–8.4	do.		24	48	24	155		24	6.4	4,110	4,710	5	4,890	5	4,640	95	10	−.0007
102	17.4	do.		24		48	155		25	6.2	4,550	4,730	5	5,580	5	980	18	10	+.0501
103	17.4	do.		24	24	24	155		25	6.0	4,600	5,510	5	5,290	5	4,980	94	10	+.0004
104	17.4	do.		24		72	155		24	6.1	4,910	5,180	5	5,180	5	1,270	25	10	+.0162
105	17.4	do.		24	24	48	155		24	6.0	4,820	5,070	5	4,910	5	5,220	106	10	+.0001
106	17.4	do.		24	48	24	155		24	5.7	5,110	5,900	5	5,320	5	5,250	99	10	−.0002
137	14.4	Ciment electrique.[3]		24		48	155		25	6.25	3,250	3,550	5	2,830	5	3,400	120	10	−.0006
138	14.4	do.		24	24	24	155		25	6.15	3,150	3,670	5	2,860	5	3,320	116	10	−.0004
139	14.4	do.		24		72	155		24	6.09	3,160	3,840	5	2,640	5	3,790	144	10	−.0010
140	14.4	do.		24	24	48	155		24	6.13	3,190	3,890	5	2,730	5	3,790	139	10	−.0007
141	14.4	do.		24	48	24	155		24	6.09	3,360	3,800	5	3,020	5	3,670	121	10	−.0006
147	14.4	do.		24				[4] 27		6.25	4,560	4,890	5	4,560	5	5,220	114	10	−.0005
148	14.4	do.		72					25	7.00	5,020	4,810	5	5,310	5	5,960	112	10	+.0011
149	14.4	do.		24		48	100		25	6.55	5,370	5,510	5	5,240	5	5,450	104	10	+.0015
150	14.4	do.		24		48	155		25	6.16	3,360	3,870			5	3,400		10	−.0014
151	14.4	do.		24	48				25	6.07	3,300	3,750	5	2,840	5	3,650	129	10	−.0011
152	14.4	do.		24				[4] 20	343	6.35	4,570	5,040	5	5,100	5	5,040	99		
153	14.4	do.		24				[4] 20	56	6.49	4,830	4,910	5	5,120	5	6,280	123	10	−.0001
154	14.4	do.		24				[4] 20	28	6.51	4,760	4,850	5	5,450	5	5,620	103	10	+.0005
155	14.4	do.		24				[4] 20	14	6.54	4,250	4,960	5	4,570	5	5,770	126	10	+.0004
156	14.4	do.		24				[4] 20		6.34	4,710	4,860	5	4,380	5	4,960	113	10	−.0005
157	18.4	One-half Northwestern, one-half Universal.	Blast furnace slag admixed.	21				[4] 20		6.52	2,420	3,960	5	6,030	5	4,230	70	10	+.0003
158	18.4	do.	do.	24				[4] 20	35	5.97	2,880	4,180	5	5,440	5	5,170	95	10	−.0008
159	18.4	do.	do.	24				[4] 20	35	6.66	2,410	4,190	5	5,670	5	5,560	98	10	−.0011
160	18.4	do.	do.	24		48	155		35	6.38	3,850	4,010	5	5,300	5	4,800	91	10	+.0004
161	18.4	do.	do.	24		48	155		35	6.85	3,800	4,020	5	5,450	5	4,830	89	10	+.0001
162	18.4	do.	Calcium chloride admixed.	24				[4] 20		5.72	2,870	4,330	5	5,530	5	3,910	71	10	+.0020
163	18.4	do.	do.	24				[4] 20	35	7.06	2,200	3,010	5	4,280	5	3,550	83	10	+.0020
164	18.4	do.	do.	24				[4] 20	35	5.94	2,940	4,000	5	5,450	5	5,050	93	10	−.0003
165	18.4	do.	do.	24		48	155		35	7.09	3,800	3,640	5	4,600	5	3,300	72	10	+.0042
166	18.4	do.	do.	24		48	155		35	5.82	4,840	4,920	5	4,990	5	4,460	89	10	+.0015
167	18.4	do.	Cal admixed	24				[4] 20		6.02	3,200	4,150	5	4,860	5	3,910	80	10	+.0018
168	18.4	do.	do.	24				[4] 20	35	5.84	3,180	4,290	5	5,570	5	4,620	83	10	+.0001
169	18.4	do.	do.	24				[4] 20	35	5.84	3,250	4,280	5	4,550	5	4,800	105	10	−.0006
170	18.4	do.	do.	24		48	155		35	5.96	4,990	4,670	5	5,030	5	5,010	100	10	+.0008
171	18.4	do.	do.	24		48	155		35	5.97	4,550	4,690	5	5,140	5	4,980	97	10	+.0015
172	18.4	do.		24				[4] 20	35	5.57	2,870	4,190	5	5,300	5	4,370	82	9	−.0001
173	18.4	do.	Ironite admixed	24				[4] 20	35	5.87	2,950	3,980	5	4,890	5	5,020	102	9	−.0004
174	18.4	do.		24		48	155		35	6.11	4,020	4,390	5	4,330	5	3,910	90	10	+.0004
175	18.4	do.	Ironite admixed	24		48	155		35	6.06	3,880	4,440	5	5,510	5	4,630	84	10	+.0006
176	18.4	do.	do.	24		48	100		35	6.08	3,680	4,220	5	5,250	5	5,190	99	10	+.0003
177	18.4	do.		24				[4] 20	35	5.91	2,520	4,200	5	4,930	5	3,670	74	10	−.0010
178	18.4	do.	Volcanic ash admixed.	24				[4] 20	35	6.79	1,900	3,210	5	5,450	5	2,960	54	10	−.0015
179	18.4	do.		24		48	155		35	5.83	4,110	4,510	5	5,210	5	3,820	73	10	+.0009
180	18.4	do.	Volcanic ash admixed.	24		48	155		35	7.59	3,240	3,340	5	4,440	5	3,600	81	10	−.0003
181	18.4	do.	do.	24		48	100		35	7.28	2,710	2,820	5	5,130	5	4,290	84	10	−.0001
182	18.4	do.		24				[4] 20	35	5.98	2,802	4,010	5	5,780	5	4,450	77	10	−.0005
183	18.45	do.	Alkagel "A" admixed.	24				[4] 20	35	5.31	2,260	3,560	5	4,570	5	3,520	77	10	−.0011
184	18.4	do.		24		48	155		35	5.96	4,070	4,530	5	5,100	5	4,240	83	10	+.0002
185	18.4	do.	Alkagel "A" admixed.	24		48	155		35	4.97	3,280	3,940	5	4,570	5	3,300	72	10	−.0001
186	18.4	do.	do.	24		48	100		35	5.43	3,120	3,730	5	3,820	5	3,550	93	10	−.0006
208	25.4	Atlas lumnite [3]		24				[4] 20	35	4,87	6,360	6,500	5	6,260	5	7,420	118	10	−.0007
209	25.4	do.		24				[4] 20	35	4.96	6,310	6,470							
210	25.4	do.		24				[4] 20	35	4.94	7,200	7,100							
211	25.4	do.		24				[4] 20	35	4.95	7,440	6,970							
212	25.4	do.		24				[4] 20	35	4.91	6,180	6,870							
243	28.4	Super cement [5]		24				[4] 20	35	6.15	2,840	3,800	5	5,400	5	4,020	74	10	−.0015
244	28.4	do.		24				[4] 20	35	5.96	2,900	3,740	5	5,300	5	4,100	77	10	−.0027
245	28.4	do.		24				[4] 20	35	6.23	2,768	3,730	5	5,250	5	3,600	69	10	−.0006
246	28.4	do.		24				[4] 20	35	6.25	2,980	3,690	5	5,750	5	4,250	74	10	−.0019

TABLE 2.—*Description and tests of all laboratory standard concrete cylinders*—Continued

Series No.	Cement laboratory No.[1]	Cement	Admixture or impregnation	Curing method: Time in moist closet	Curing method: Time in steam at 212° F.	Curing method: Time in water vapor	Curing method: Temperature of water vapor	Curing method: Time in water	Curing method: Time in air	Absorption at 21 days	Average compression tests: 7 days	Average compression tests: 28 days	1 year, Tank specimens: Number	1 year, Tank specimens: Strength	1 year, Lake specimens: Number	1 year, Lake specimens: Strength	Strength ratio	Length increase: Number	Length increase: Increase
				Hours	*Hours*	*Hours*	*° F.*	*Days*	*Days*	*P. ct.*	*Lbs.*	*Lbs.*		*Pounds*		*Pounds*	*Per cent*		*Inch*
247	28.4	Super cement		24				[4] 20	35	6.31	3,070	3,880	5	5,130	5	4,170	81	10	−0.0008
252	58.4	Ciment fondu [3]		24				[4] 20	35	5.40	7,410	7,250							
253	58.4	do		24				[4] 20	35	5.40	8,040	7,200							
256	61.4	One-half Northwestern, one-half Universal.		24				[4] 20	35	5.84	2,780	3,950							
257	63.4	Northwestern		24				[4] 20	35	5.84	2,720	4,290							
258	62.4	Universal		24				[4] 20	35	5.78	2,960	3,790							
259	33.4	Lehigh		24				[4] 20	35	5.41	3,340	4,780							
260	34.4	Hawkeye		24				[4] 20	35	6.14	3,340	4,540							
[6] 286	27.4	Atlas lumnite [3]		24				[4] 20	35										
287	27.4	do		24				[4] 20	35										
288	27.4	do		24				[4] 20	35										
[7] 289	27.4	do		24				[4] 20	35										
[8] 290	27.4	do		24				[4] 20	35										
[8] 291	58.4	Ciment fondu [3]		24				[4] 20	35										

[1] For tests of cement see Table 5
[2] Tap water.
[3] High-alumina cement.
[4] Distilled water.
[5] Special cement.
[6] Laboratory standard cylinder except in mix, which is 1:2.
[7] Laboratory standard cylinder except in mix, which is 1:4.
[8] Laboratory standard cylinder except in mix, which is 1:5.

TABLE 3.—*Description and tests of all standard Ottawa sand mortar cylinders*

(Fineness modulus of aggregate for all cylinders 3. Mix 1 : 3 except as noted for 4 series)

Series No.	Cement laboratory No.[1]	Cement	Admixture or impregnation	Curing method: Time in moist closet	Curing method: Time in steam at 212° F.	Curing method: Time in water vapor	Curing method: Temperature of water vapor	Curing method: Time in water	Curing method: Time in air	Absorption at 21 days	Average compression tests: 7 days	Average compression tests: 28 days	1 year, Tank specimens: Number	1 year, Tank specimens: Strength	1 year, Lake specimens: Number	1 year, Lake specimens: Strength	Strength ratio	Length increase: Number	Length increase: Increase
				Hours	*Hours*	*Hours*	*° F.*	*Days*	*Days*	*P. ct.*	*Lbs.*	*Lbs.*		*Pounds*		*Pounds*	*Per cent*		*Inch*
107	17.4	One-half Northwestern, one-half Universal.		24		48	155		25	9.8	2,660	3,080	5	3,580	4	960	27	8	+0.0126
108	17.4	do		24	24	24	155		25	9.8	2,590	2,910	5	2,930	5	2,540	87	10	+.0002
109	17.4	do		24		72	155		24	9.6	2,670	3,220	5	3,490	5	930	27	10	+.0141
110	17.4	do		24	24	48	155		24	9.5	2,600	3,160	5	2,940	5	2,720	93	10	−.0010
111	17.4	do		24	48	24	155		24	9.7	2,510	3,130	5	2,630	5	2,450	93	10	−.0002
112	17.4	do		24				[2] 27		9.9	1,650	2,630	5	3,540	5	1,790	51	10	+.0002
113	17.4	do		72					25	10.7	2,000	2,000	5	2,980	3	1,120	38		
114	17.4	do		24		48	100		25	10.1	2,270	2,790	5	3,040	3	1,040	34		
115	17.4	do		24		48	155		25	10.1	2,730	2,900	5	3,400	4	1,070	32	8	+.0416
116	17.4	do		24	48				25	10.3	2,220	2,470	5	2,500	5	2,430	97	10	+.0005
127	17.4	do		24		48	155		53	9.06	2,530	2,710	5	3,460	5	2,280	66	10	+.0012
128	17.4	do		24	24	24	155		53	9.08	2,350	2,480	5	2,830	5	3,180	112	10	+.0003
129	17.4	do		24		72	155		52	8.95	2,700	3,120	5	3,320	5	1,970	59	10	+.0009
130	17.4	do		24	24	48	155		52	9.07	2,750	2,720	5	2,830	5	3,080	109	10	+.0013
131	17.4	do		24	48	24	155		52	9.16	2,630	2,470	5	2,600	5	2,560	98	10	+.0008
132	14.4	Ciment electrique.[3]		24		48	155		25	8.10	2,250	2,910	5	2,760	5	2,790	101	10	−.0010
133	14.4	do		24	24	24	155		25	8.02	2,280	2,680	5	2,080	5	2,900	139	10	−.0013
134	14.4	do		24		72	155		24	7.98	2,470	2,850	5	2,440	5	2,840	116	10	−.0015
135	14.4	do		24	24	48	155		24	7.82	2,620	2,960	5	2,650	5	2,590	98	10	−.0013
136	14.4	do		24	48	24	155		24	7.77	2,550	2,930	5	2,630	5	2,580	98	10	−.0004
142	14.4	do		24				[4] 27		8.67	2,910	3,600	5	3,590	5	4,060	113	10	−.0004
143	14.4	do		72					25	9.34	4,220	4,430	5	4,220	5	4,310	102	10	+.0016
144	14.4	do		24		48	100		25	9.13	4,320	4,640	5	3,750	5	4,490	120	10	+.0022
145	14.4	do		24		48	155		25	8.11	2,200	2,760	5	2,780	5	2,620	94	10	−.0011
146	14.4	do		24	48				25	7.77	2,600	2,870	5	2,500	5	3,010	120	10	−.0008
248	10.4	Super cement [5]		24				[4] 20	35	9.70	1,430	2,770	5	3,730	5	3,310	89	10	−.0020
249	10.4	do		24				[4] 20	35	9.21	1,450	2,710	5	3,870	5	3,280	85	10	−.0022
250	10.4	do		24				[4] 20	35	9.48	1,410	2,720	5	3,650	5	3,240	89	10	−.0016
254	58.4	Ciment fondu [3]		24				[4] 20	35	9.23	5,030	4,540							
255	58.4	do		24				[4] 20	35	9.36	4,990	5,750							
[6] 276	27.4	Atlas lumnite [3]		24				[4] 20	35	6.33	6,210	7,680							
277	27.4	do		24				[4] 20	35	8.31	4,940	4,810							
278	27.4	do		24				[4] 20	35	8.28	5,190	4,840							
[7] 279	27.4	do		24				[4] 20	35	10.25	3,380	3,090							
[8] 280	27.4	do		24				[4] 20	35	11.75	2,160	1,920							
[8] 292	58.4	Ciment fondu [3]		24				[4] 20	35										
293	19.4	One-half Northwestern, one-half Universal.		24				[4] 20	35										
294	19.4	do	Sulphur impregnated.[9]	24				[4] 20	35										
295	19.4	do	Sulphur admixture.	24				[4] 20	35										

[1] For tests of cement see Table 5.
[2] Tap water.
[3] High alumina cement.
[4] Distilled water.
[5] Special cement.
[6] Standard Ottawa sand mortar cylinder except in mix, which is 1 : 2.
[7] Standard Ottawa sand mortar cylinder except in mix, which is 1 : 4.
[8] Standard Ottawa sand mortar cylinder except in mix, which is 1 : 5.
[9] At eighth week of curing the cylinders were placed, in melted sulphur for 15 minutes.

ratio of 97 per cent, as compared with the tank specimens, the minimum being 87 and the maximum 114 per cent. The fact that these cylinders after one year's exposure to the water of the lake average slightly weaker than the corresponding cylinders stored in the laboratory tank may be due to slight retardation of the natural curing processes, because of the lower temperatures of the lake water, or may be due to the influence on the curing of the sulphates in the lake, or to chemical action. In any event a strength ratio of 97 per cent is so near normal as to render further tests necessary before drawing definite conclusions.

3. Curing in water vapor at temperatures of 155° F. and 100° F. has been of no value. Ninety cylinders from 18 series in which a temperature of 155° F. was used had a strength ratio of but 38 per cent as compared with check cylinders from the laboratory tank, while 35 cylinders from 7 series in which a temperature of 100° F. was used in the curing had a strength ratio of 31 per cent. Actually the results are not even as favorable for a temperature of 155° F. as these figures would indicate, for when the strength ratios of 35 cylinders from 7 of the 155° F. series are compared with the same number of cylinders from the 100° F. series—the cylinders from the two groups being identical in all respects except that of the temperature of the curing vapor—the results are 21 and 31 per cent, respectively. This is a surprisingly poor showing for the cylinders cured at 155° F. It would naturally be expected that, since the cylinders cured at 212° F. showed up so well, those cured at 155° F. would be at least more resistant than those cured at 100° F. (see figs. 1, 2, 3, and 4). It is evident from this that there is a critical temperature point, somewhere above 155° F. in the curing of concrete made of standard Portland cement if the purpose be to develop resistance to sulphate waters.

4. Of the cylinders of series 92, 93, 112, 113, 172, 177, and 182 cured in the laboratory in the moist closet, followed either by further curing in water or by storing in air, or both, before exposure to the water of Medicine Lake, it is noted that the strength ratio of 50 such cylinders is 64 per cent when compared with cylinders from the same series stored in the laboratory tank. The temperatures under which these cylinders were cured ordinarily ranged around 70° F., and it appears that this temperature has been more effective than either 155° F. or 100° F. in retarding the action of sulphate waters (see fig. 4), although considerable deterioration is evident in all cylinders of these series after exposure to the lake water for one year.

5. In series 157 to 186, most of which contained admixtures, the condition of the specimens after one year in the lake indicates that under some conditions additions of blast-furnace slag, calcium chloride, Cal, and ironite have increased the resistance of the concrete while additions of volcanic ash and Alkagel "A" have been of little or no benefit. All these results are somewhat involved, and tests after longer periods of exposure will be necessary before conclusions will be of value. However, the curing conditions which produced the best result after one year's exposure of cylinders containing each of the foregoing admixtures are shown in Table 4, together with the results of tests for strength and change of length. No surface deterioration was evident in any of the cylinders of the series included in this table.

TABLE 4.—*Best curing conditions for specimens containing admixtures as indicated by tests after exposure for one year to Medicine Lake water*

Series No.	Admixture		Curing conditions after 24 hours in moist closet			Strength ratio, lake to tank specimens	Length change
	Amount	Material	Time in water	Time in water vapor	Time in air		
	Per cent		*Days*	*Days*	*Days*	*Per cent*	*Inch*
159	40	Blast-furnace slag	20		35	98	−0.0011
164	4	Calcium chloride	20		35	93	−.0003
169	4	Cal	20		35	105	−.0006
173	20	Ironite	20		35	102	−.0004
181	20	Volcanic ash		2	35	84	−.0001
186	3	Alkagel "A"		2	35	93	−.0006

SPECIMENS INSTALLED IN WOODEN CRATES

The specimens were installed in the lake in copper-nailed wooden crates, 50 cylinders to a crate, as shown in the illustrations. The crates were marked by copper tags and otherwise permanently identified by burned-in numbers, and each specimen had a positional identity within the crates. The crates were installed in the lake in about 5 feet of water; consequently the specimens were not subjected to any weathering action due to freezing and thawing.

In general, the cylinders were crated and shipped by express to Medicine Lake when 8 weeks old and were placed in the lake within three days after leaving the laboratory. Those cylinders which were not cured 8 weeks were placed in earthenware jars containing 10 liters (2.6 gallons) of Medicine Lake water, 20 cylinders to a jar, and at 8 weeks were crated and shipped to Medicine Lake. On the same day the cylinders were placed in Medicine Lake water other cylinders of the same series were placed in tap water in the laboratory tank.

In all cases the mix used in the cylinders was proportioned by volume with the weight of cement assumed at 100 pounds per cubic foot. Where an admixture was used the amount of cement and aggregate remained the same and the material added was measured as a percentage by weight of the cement used. Each batch was mixed by hand at least 1½ minutes dry and 2 minutes after adding the water. The materials, after mixing, were tamped in three 3-gang brass molds in four layers, each layer being tamped 20 times. For tamping the laboratory standard cylinders a pointed rod was used which was three-eighths inch in diameter by 15 inches long; for the standard Ottawa sand cylinders a flat-ended ⅝-inch rod was used which weighed 337 grams. Round-head brass screws 1 inch long were set in neat cement on the ends of three cylinders, and measurements between these screwheads served as the basis for the record of change of length. The other six cylinders of each batch were left with blank ends and were used in the various compression tests.

TESTS OF CEMENT AND AGGREGATES

The aggregate, mix, and consistency of all laboratory standard cylinders were identical with the few exceptions noted in Table 2, the variables being curing, cements, and admixtures. The cement used, unless otherwise stated in Table 2, was a mixture of equal parts of Northwestern and Universal Portland cement. For physical tests of all cements used see Table 5.

TABLE 5.—*Summary of tests of cements used in experimental cylinders placed in Medicine Lake*

Laboratory No.	Lot No.	Brand of cement	Time of set: Vicat, Initial		Vicat, Final		Gilmore, Initial		Gilmore, Final		Steam test	Fineness	Water required for normal consistency	Specific gravity	Tensile strength of standard briquettes: 3 days	7 days	28 days
			Hours	*Minutes*	*Hours*	*Minutes*	*Hours*	*Minutes*	*Hours*	*Minutes*		*Per cent*	*Per cent*		*Pounds*	*Pounds*	*Pounds*
8.4	8	One-half Northwestern and one-half Universal	3	10	6	20	4	10	7	20	O. K.	21.6	24.0	3.11	149	230	342
10.4	------	Super cement	4	40	18	30	5	30	18	30+	O. K.	7.7	24.25	--------	122	184	324
11.4	11	One-half Northwestern and one-half Universal	2	45	5	40	3	25	7	0	O. K.	17.9	23.0	3.13	150 131	235 186	357 316
14.4	------	Ciment electrique [1]	3	50	6	35	4	35	6	45	O. K.	16.8	26.0	3.22	278	316	--------
17.4	12	One-half Northwestern and one-half Universal	2	15	6	0	3	40	6	15	O. K.	20.0	25.0	3.13	130	217	317
18.4	13	-----do	3	15	6	50	5	30	7	20	O. K.	22.0	25.0	3.13	126 168	186 243	307 382
25.4	------	Atlas lumnite [1]	9	0	11	0	9	0	11	0	O. K.	7.8	22.5	--------	466	458	472
27.4	------	-----do	5	10	7	10	5	30	7	25	O. K.	14.0	22.5	--------	465	452	398
28.4	------	Super cement	------	------	------	------	------	------	------	------	------	------	24.25	--------	109	176	281
33.4	------	Lehigh	3	30	5	25	3	10	5	25	O. K.	17.9	22.5	3.13	209	259	387
34.4	------	Hawkeye	5	20	7	55	6	10	8	40	O. K.	9.1	28.5	3.14	224	285	403
58.4	------	Ciment fondu [1]	5	0	7	15	5	30	7	55	O. K.	6.6	22.5	3.11	439	424	376
61.4	15	One-half Northwestern and one-half Universal (mixture of Nos. 49.4, 50.4, 43.4, and 44.4)	4	40	7	20	5	5	7	30	O. K.	14.5	23.5	3.12	151	218	336
62.4	16	Universal (mixture of Nos. 43.4 and 44.4)	4	50	7	5	6	25	7	25	O. K.	17.0	24.0	3.14	172	231	326
63.4	17	Northwestern (mixture of Nos. 49.4 and 50.4)	4	50	7	30	6	15	8	5	O. K.	18.2	23.5	3.18	134	218	335

[1] High-alumina cement.

The aggregate, obtained from the J. L. Shiely Co. gravel pit at St. Paul, Minn., passed all standard physical tests, including a test for shale, which showed less than 0.4 per cent by volume. The various sizes were separated by screening and recombined for each batch, as shown in Table 6. The combination used in series Nos. 77 to 106 differed slightly from the normal but the fineness modulus was 4.67.

TABLE 6.—*Screen analysis and quantities of aggregate for each 9-cylinder batch of laboratory standard cylinders*

(Unit room-dry weight of aggregate, 124 pounds per cubic foot)

Screens	Percentage	Coarser than—	Percentage	Amount required
				Grams
Retained on 3/8 inch	0.0	3/8 inch	0.0	--------
Passing 3/8 inch, retained on No. 4	38.2	No. 4	38.2	1,350
Passing No. 4, retained on No. 8	25.8	No. 8	64.0	910
Passing No. 8, retained on No. 16	13.3	No. 16	77.3	470
Passing No. 16, retained on No. 30	13.3	No. 30	90.6	470
Passing No. 30, retained on No. 50	6.8	No. 50	97.4	240
Passing No. 50, retained on No. 100	2.4	No. 100	99.8	[1] 91
Passing No. 100, retained on pan	.2	----------	----------	--------
	100.0	----------	467.3	[2] 3,531

[1] Passing No. 50 and retained on pan.
[2] This quantity is for a 1:3 mix; for a 1:4 or 1:5 mix the quantities were increased 10 per cent for each size of aggregate, making a total of 3,884 grams.

Fineness modulus=4.67.

The standard Ottawa sand cylinders differed in no respect other than the aggregate from the standard concrete cylinders. As in the latter, the standard mix was 1:3 and the relative consistency 1, while the variables, as in the other group, were curing, cements, and admixtures. The cement used, unless otherwise stated in Table 3, was a mixture of equal parts of Northwestern and Universal Portland cement.

Compression tests were made in a 50,000-pound Reihle machine, using a bottom spherical bearing block. The cylinders tested at 7 and 28 days were cured in the same manner as the cylinders to be placed in Medicine Lake except those which were intended to be cured 24 hours in the moist closet and 20 days in water. These were kept in water until tested.

The absorption test results are based on the A. S. T. M. standard absorption test for drain tile, which stipulates drying the specimen to complete dryness in a temperature of not less than 230° F., followed by boiling for five hours.

DESCRIPTION OF ADMIXTURES USED

Series 157 to 161, blast furnace slag.—The slag used in these series was obtained from the Universal Portland cement plant at Duluth, Minn. Before using, it was ground in a coffee-mill grinder in the State Bureau of Mines at the University of Minnesota until it approached the fineness of cement. The mill used was not heavy enough to crush the iron pebbles and these were rejected. The material, after grinding, was screened on sieves Nos. 100 and 200 and recombined in the proportion of 1 part passing the No. 100 and retained on the No. 200 sieve and 5 parts passing the No. 200 sieve. It was mixed dry with the cement and aggregate.

Series 162 to 166, calcium chloride.—The calcium chloride used in these series was a chemically pure product, dissolved in the mixing water.

Series 167 to 171, Cal.—Cal, used in these series, is the trade name for a powdered manufactured product. Quoting from United States Bureau of Standards, Technologic Paper No. 174, "Cal is a material obtained by pulverizing the dried or undried product resulting from a mixture of either quicklime or hydrated lime, calcium chloride and water."

Series 172 to 176, ironite.—This is the trade name for a powdered waterproofing product. The recommended method of use is by brush coat or coats, applied to the surface of the concrete after the removal of the forms. In series 173, 175, and 176 it was mixed dry with the cement and aggregate. Such use of this material as an admixture, so far as is known, is not advocated by the manufacturers.

(Continued on page 183)

CLAY SOILS IN RELATION TO ROAD SUBGRADES

By HUGH H. BENNETT, Soil Scientist, United States Bureau of Soils

ACCORDING to tests which have been recently made,[1] unfavorable road subgrade conditions are frequently due to the presence of an excessive amount of clay soil of a plastic nature. Ordinarily the quantity of the clay present in the soil determines the quality of the subgrade, but there are a number of exceptions to this rule, particularly in the soils of tropical regions and certain soils of the more southerly latitudes in the United States. In this connection some observations relating to fundamental differences in the character of clays, with emphasis on their effect upon stability or serviceability as subgrade material,[2] may be of interest to highway engineers.

THE CHARACTER OF CLAY SOILS IN THE UNITED STATES

Clay,[3] as commonly known, is a very fine-grained and dense soil which absorbs water slowly and holds it tenaciously because of the lag of surface tension and other forces which impede the movement of moisture through the minute pore spaces between the grains. Well-known properties of clay include pronounced stickiness and plasticity when wet, and hardness, often accompanied by shrinking and cracking when dry. These conceptions will usually fit the conditions in the humid sections of the United States, although there are some variations in these characteristics in different localities, corresponding with the differences between the soil types.

It might be supposed from the accepted definition that those clays having the higher percentages of sand would be also the least impervious to water. This, however, is not invariably true. Permeability is often due more to the degree of the weathering of the soil and the consequent character of the colloids [4] rather than to the size and textural grading of the constituents. In some highly weathered clays, such as exist in the more southerly latitudes, the flocculent character of the extremely fine particles provides a more open and better-drained soil, even in the case of extraordinarily fine-grained clays, than is found in more northerly regions, even in sandy clays, where the colloids are not of the kind that cling together to form soil aggregates.

OUTSTANDING ILLUSTRATIONS OF ADVANCED WEATHERING

The most outstanding illustrations of the effect of advanced weathering upon soil character are to be found in the humid tropics, where, under a constantly warm temperature and heavy rainfall, soil weathering over large areas has proceeded to a stage considerably in advance of that prevailing in the North Temperate Zone. There are soils in Central America with 90 per cent of the particles as small as the finest clay that are as friable and open-natured as the mellowest loam. On the contrary, clays approximating this fineness in the United States may be counted upon to be exceedingly impervious to water, subject to swelling and shrinking with variations in moisture content, and of a poor quality for highway subgrades.

Humid tropical clay—that is, the older clay, which has existed in place for a long time—has the permeability previously referred to. This condition results chiefly from the weathering of the colloids until they are characterized by a marked tendency of the particles to group into clusters, from which the individual grains strongly resist detachment by water. This aggregating of the minute particles gives rise to an open, granular structure, while the refusal of the grains to break away and float off in water prevents the soil pore space from clogging through a process of downward transposition of the particles by percolating water (eluviation). Nearly all the rainfall passes downward through the soil, and the little which runs off usually shows only a slight turbidity. In Cuba the famous red soil of Matanzas and other provinces absorbs the rain to such an extent that over broad areas no streams whatever have developed.

The soils of the humid tropics show very little tendency to swell and shrink with variations in moisture content, they strongly resist erosion, and are only slightly sticky when wet. In extreme cases, as in eastern Costa Rica, even plowing in the rain results in a fine tilth.[5] Material of this kind is usually a product of the climate and the duration of the weathering. Friability and resistance to erosion are much more nearly normal properties of the fine soils of a warm, wet climate than of soils in a cool, wet climate. The plastic nonfriable clays of the wet tropics, and frequently also of the Temperate Zone, are the young clays, such as poorly drained alluvial clay and freshly decomposed shale material.

DELINEATION OF THE REPRESENTATIVE MELLOW-CLAY AREAS OF THE UNITED STATES

There are in the United States no extensive areas of mellow clays precisely like the severely weathered types of the tropics, but there are large areas, notably in the Piedmont Plateau from New Jersey to central Alabama, in which the clay soils have, in some degree, similar properties. Studies have shown that the colloids in the red granitic soils of the Piedmont (Cecil soils) have some of the characteristics or an approximation of the characteristics of the tropical colloids. The Cecil soils, including chiefly the clay, clay-loam, and sandy-loam types, have everywhere a brittle, red-clay subsoil, usually containing some particles of quartz derived from the parent rocks. In places quartz veins cut through the clay, these representing the most resistant part of the original country rock. Often the underlying granite or gneiss in its upper weathered part consists of "rotten rock," i. e., friable material which *in*

[1] Practical Field Tests for Subgrade Soils. PUBLIC ROADS, vol. 5, No. 6, August, 1924.
Field Methods Used in Subgrade Soils. PUBLIC ROADS, vol. 6, No. 5, July, 1925.

[2] This article will discuss the direct effect of the grading and size of the soil grains and not the indirect effect of hardpan, compact layers, etc., upon the behavior of the overlying material under varying conditions of moisture. Nor will sandy clays be discussed, since these are usually permeable and stable.

[3] Clay soils, according to the textural definition, have the following proportions of ingredients: 30 to 100 per cent clay particles (diameter range, 0.005 mm. and less); less than 55 per cent sand (1 to 0.05 mm.); less than 55 per cent silt (0.05 to 0.005 mm.); and more than 50 per cent silt and clay.

[4] Exceedingly fine particles of the soil, sometimes referred to as ultraclay.

[5] Soils of this group are related to that kind of laterite which has a high content of the hydroxides of iron and alumina, and a relatively low content of silica and the bases (lime, magnesium, potash, and soda).

situ has preserved the skeleton structure of the original, unaltered rock.[6] Probably the most valuable properties of these soils, from the standpoint of the highway engineer, are the slight tendency to swell and shrink on wetting and drying, and a sufficient permeability to prevent water-logging, provided ordinary drainage outlets are maintained. The clay hardens on drying, but seldom cracks, even in cuts exposed to the air.

Essentially the same properties characterize the Porters soils of the Blue Ridge Mountain region. These are reddish brown at the surface, with clay subsoil much like the subsoil of the Cecil. They also are derived from granite and gneiss, show little tendency to shrink or swell, and admit of fairly free passage of water. On soil of this nature subgrade difficulties are likely to be of negligible importance. Clays of uniform red, brown, and deep-yellow colors are better oxidized than those of whitish, grayish, bluish, and pale-yellow colors, or mottlings of these colors, and they almost invariably are better drained, shrink less on drying, swell less on wetting, and are much firmer (less miry) in the presence of excessive moisture. The colloidal clay of the slightly oxidized light-colored and mottled soils has more of the swelling and shrinking properties, and the soils consequently are characterized by larger volumetric changes under varying moisture conditions. In dry weather an experienced soil man can readily identify clays such as the Susquehanna, Montrose, Iredell, Lufkin, Coxville, and Eldon, wherever exposed long enough to dry out, by the peculiar cracking ("checking") occasioned by shrinking.

REPRESENTATIVE HEAVY CLAY AREAS IN THE SOUTHERN STATES

Water passes slowly through the light-colored and mottled plastic clays; they become saturated with penetrating moisture, and eventually soft and unstable. These properties have given rise to such local names as "pipe clay," "beeswax," "soggy soil" (or "sobby soil"), "spouty clay," and so on. Even some of the mottled clays containing considerable sand, such as the Leaf (a stream-bench soil of the Coastal Plain region east of the Coastal Prairies), and the Elkton of the Chesapeake Bay region, behave unfavorably between the wet and dry ends.

The clay of the plastic, impervious Susquehanna soils is poorly oxidized, as shown by its mottled red, yellow, and bluish color. Frequently the upper part is red, with mottled clay beneath a thinly oxidized zone, the red color decreasing with the depth. These are very extensive soils in the Coastal Plain region, east of central Texas. Especially are they of abundant occurrence in west-central Georgia, southern Alabama, throughout the uplands of Mississippi, southern Arkansas, and east Texas. They are also scatteringly present as far north as the Susquehanna River locality of Maryland. The clay type and the clay subsoil of all types constitute unfavorable soil material for any purpose. When saturated it is slippery, plastic, and extremely miry; on drying it hardens, shrinks, and cracks. The Montrose clay occurring in association with the Susquehanna, on flat areas, has the properties of the latter, with some accentuation of the bad features, due to retarded oxidation caused by poor surface drainage. No red color is present in the subsoil, which consists of yellow and bluish-gray clay of extreme density and imperviousness. Quite similar features are exhibited by the gray Lufkin clay of the flatwoods regions of Texas, Arkansas, Mississippi, eastern North Carolina, and other parts of the Coastal Plain, and also by the clay subsoil of the Coxville types occurring through the Atlantic flatwoods from southeastern Georgia northward.

The Iredell soils, locally referred to as "pipe clay," "bull tallow," and "black jack" land, are the most extensive bad road soils of the Piedmont region. They have been formed from the decay of basic igneous rocks, chiefly diorite. Their subsoils consist of yellow or greenish-yellow, very sticky, plastic clay, which is impervious to water and swells or contracts as the water content rises or falls. The clay on drying always cracks badly.

In those parts of the Piedmont underlain by sedimentary rocks of Triassic age certain claystone horizons give rise to an unfavorable clay, White Store clay, that practically duplicates the properties of the Susquehanna clay. In the prairie regions of west-central Missouri the subsoil of the dark-colored Eldon series also essentially repeats the characteristics of the Susquehanna, including the unfavorable features relating to subgrades.

CONCLUSION

Space does not permit discussion of all the numerous clay types, most of which probably will be found unsatisfactory as subgrades for highways. However, it is believed that an application of the principles involved in the varying properties outlined above to the clay soils should roughly classify most of them as to their suitability or unfitness for subgrade material, at least for the humid regions of the United States. With these properties in mind, certain clays probably can be eliminated at once from the list of suitable materials, others can immediately be set down as favorable, while those of doubtful classes can be subjected to the necessary physical tests.

It should be observed in connection with texture that certain materials occurring in both the humid and arid regions, much more generally in the latter, will show in their physical analysis a composition as fine as the heaviest clay and yet have markedly different properties from normal clays. These are the chalky materials composed largely of lime carbonate or of lime sulphate or of both, and known locally by such names as "caliche," "plains marl," "gyp-beds," and "rotten limestone."[7]

All of these are low in content of those colloids that impart plasticity and water-logging properties common to the plastic clays. With plentiful moisture, the tendency is to remain firm rather than to soften. They are not true clays, and under normal conditions with respect to artificial removal of excess rain water they probably will function satisfactorily as subgrade material for roads.

[6] This material has been used with good results for surfacing roads.

[7] The "rotten limestone" variety of chalk is largely found as the basal formation beneath the Houston soils of the prairie region of Alabama and Mississippi (Selma chalk) and the Black Waxy Belt of central Texas (Austin chalk). The other classes of materials outcrop or lie near the surface over vast stretches of the Great Plains and arid regions. A subsoil accumulation of lime of the caliche order really marks the boundary between the prairies and plains.

HIGHWAY RESEARCH BOARD ACTIVITIES

EXPLAINING the service rendered by the Highway Research Board of the National Research Council in a recent radio address under the auspices of the National Research Council and Science Service, Prof. S. S. Steinberg, of the University of Maryland, remarked that whereas a few years ago a road contract 2 miles long was considered a large undertaking, contracts are now let in 10 and 20 mile sections. In one day recently, according to Professor Steinberg, Pennsylvania let road contracts totaling $12,000,000. The previous record was that of Illinois, with a letting of $9,000,000 in one day. Such stupendous programs, it has been truly said, will make the construction of highways the most active American industry during the current year.

Continuing, Professor Steinberg said that—

Fortunately, highway engineers realize the necessity of having this great development of our highway system proceed on sound principles of engineering and economics. This is evident from the fact that the Federal Government, the State highway departments, and many universities are engaged in an extensive program of research into the problems affecting highway finance, construction, and maintenance. A recent census shows that there are almost 500 highway research projects under way throughout the country. It is the function of the Highway Research Board of the National Research Council to coordinate these researches, spread as they are over such a wide territory; to prevent duplication of effort, by putting the workers into touch with one another; and finally, to make known to each State the findings which may be immediately applied in practice; thus resulting not only in the better construction and maintenance of highways but also in a great saving in the taxpayer's dollar.

The Highway Research Board thus occupies a unique position as a service organization to highway engineers and to the public, and it enjoys the complete confidence of all agencies throughout the country engaged or interested in highway development. As indicative of the personnel of the board and interests represented, we might cite that Dean A. N. Johnson, of the University of Maryland, is chairman; W. H. Connell, of the Pennsylvania State Highway Department, is vice chairman; and C. M. Upham, State highway engineer of North Carolina, is director. The executive committee also includes Prof. T. R. Agg, of Iowa State College; A. J. Brosseau, president of Mack Trucks (Inc.); Dr. H. C. Dickinson, of the United States Bureau of Standards; Thomas H. MacDonald, chief of the United States Bureau of Public Roads; and W. Spraragen, of the National Research Council.

CONCRETE REINFORCEMENT STUDY NEARING COMPLETION

Where research of a general character needs to be undertaken, which a single organization would not be justified in conducting, the Highway Research Board has been assigning an expert to carry on such investigation. The results are then made available to all interested agencies. As an example, there is now nearing completion a nation-wide survey of the economic value of steel reinforcement in concrete roads. In some States it is the practice to incorporate in the concrete steel rods or mesh, using half a pound or more to every square foot of surface. In other States no reinforcement is used. As this item alone may amount to many thousands of dollars per mile of road, it is necessary for the highway engineer to know whether the use of reinforcement is justified, and if so, the proper amount needed for the greatest economy.

Another investigation, just commenced under the board's auspices, deals with the development of earth roads. Although we may now travel with speed and comfort from one end of the country to the other, we must not forget that 85 per cent of all our rural highways are still in a state of nature, and many of them impassable for several months of each year. It may help us to realize what a great economic loss this means when we consider that one-third of all the automobiles in use are owned by farmers most of whom are compelled to use these earth roads. This problem is an especially significant one in the Western States where there is a great mileage of highways to improve and maintain and the funds are so limited that any extensive program of surfacing with the more costly types of pavements is out of the question.

Many attempts have been made to solve this problem. For instance, in North Carolina, where there is a sea of sand stretching for hundreds of miles along the coast, the State Highway Commission has developed, as a result of research, a sand-asphalt surface made up of a mixture of 93 per cent local sand and 7 per cent asphalt. This renders very satisfactory service to that locality. In South Carolina, Illinois, and California, tars and oils are being used, either by mixing with the earth or as surface applications. In the Southern States either the top soil of the fields or a mixture of sand and clay is used. In Iowa, Missouri, and South Dakota experiments are being conducted in which the natural soil is mixed with hydrated lime, or in some cases with Portland cement, in an attempt to stabilize the natural soil.

Realizing that this is one of the most important problems confronting highway engineers, the Highway Research Board is attempting to coordinate completed and current research on this subject with the hope of developing a low-cost road surface that will be suitable for light traffic.

Professor Steinberg, who has been designated as the acting secretary of this new investigation, has been head of the department and professor of civil engineering at the University of Maryland since 1918. He was previously connected with the New York, Delaware, and South Carolina highway departments, and served also on subway construction in New York and railroad construction in Central America. During several summers he has been employed by the Bureau of Public Roads on details at the University of Maryland in connection with research conducted there by Dean A. N. Johnson. During the summer of 1924 he was assistant director of the Highway Research Board.

Another investigation which has been begun under the auspices of the board is that on culvert pipe. The objects of this study are to correlate the various tests and experiments on pipe loads and stresses, to set up a standard with which to compare the life of different kinds and classes of culvert pipe, and to determine the most probable conditions of service for which standards should be designed.

CRUM APPOINTED TO HEAD CULVERT PIPE INVESTIGATION

R. W. Crum, engineer of materials and tests, Iowa State Highway Commission, has been designated as chairman of the culvert-pipe investigation. Mr. Crum was for 12 years associate professor of civil engineering at the Iowa State College, during which time he was engaged on research work for the State engineering experiment station. He has held his present position with the State highway commission since 1919. He is the author of a number of important research papers, and is an active member of several research committees of the American Society of Civil Engineers and the American Society for Testing Materials.

In view of the constantly enlarging program of highway development in Latin America, the board is extending a cordial invitation to the Latin American universities to designate honorary representatives, who shall serve as liaison officers between the universities and the board. Similar contacts now exist with the universities in the United States.

These invitations have been presented personally by Dean A. N. Johnson, who is now in South America as one of the United States delegates to the Pan American Congress of Highways held in Buenos Aires in October.

THE PREPARATION OF CONCRETE CORES FOR COMPRESSION TESTS

Reported by C. E. PROUDLEY, Assistant Engineer of Tests

THE division of tests of the Bureau of Public Roads has received a number of requests for information relative to the best method of preparing concrete cores for compression tests. The following information may be of assistance to those who have already taken up this line of investigation, as well as those who anticipate doing so in the future.

It has been found that capping the cores with neat cement is the most satisfactory method, whether the ends of the core are very irregular or not. Plaster of paris has been used with success where the specimens had especially smooth end surfaces, but in general this practice is not advised. It has been found from comparative tests that cores capped with plaster of paris may show a reduction in strength as great as 50 per cent below those similar in character capped with Portland cement.

The method adopted in the laboratory of the Bureau of Public Roads is to remove any very sharp or abnormal projections from the end of the core by means of a carborundum saw or the skillful use of a hammer, after which the surface is cleaned with hydrochloric acid, followed by rinsing with a jet of water. Unless this cleansing process is carefully done the cap will not adhere to the core. No attempt should be made to prepare specimens with heights equal to or exactly twice their diameters. It has been found that more accurate results are obtained when the original core is capped and tested without any attempt to build up its height with the capping material. The necessary correction for height may easily be made by applying the correction factor given in A. S. T. M. Tentative Standard Method of Securing Specimens of Hardened Concrete from a Structure, serial designation C45–21T.

The surface to be capped is thoroughly brushed with a soft paste of neat cement in order to insure a good bond for the cap. The specimen should be kept wet at all times, and a minimum period of time should elapse between the various operations in order to prevent premature drying of the unhydrated cement in the cap. Several methods of applying the cap may be followed, depending upon the irregularities of the surface. If the end of the core is comparatively smooth neat cement paste of approximately normal consistency is heaped upon the specimen and the latter then inverted onto a piece of plate glass which has been oiled and is free from scratches. New plate glass is recommended, as any scratches or roughness may cause the cement to stick to the glass, thereby removing the cap from the core. The excess cement is squeezed out by pressing the core firmly down against the plate, and a trowel is used to point up the cement around the edges of the core. Care should be taken that the axis of the specimen is perpendicular to the glass surface.

Another method which is used in the case of specimens having irregular ends consists of wrapping a piece of thin sheet metal, which has been oiled, around the core so as to form a mold. This may be held in place with several wire bands. The highest point of the end of the core should be flush with the end of this mold, so that when the cement paste is applied there will be neither an excessive thickness of cap nor any projection from the core. The form thus prepared is filled with cement paste, after which a piece of oiled plate glass is placed upon it so as to form a plane bearing surface.

The selection of oil in these operations is important, as some oil seems to lack those qualities which prevent adhesion of the cement to the glass or metal surfaces. Several brands should be tried for the purpose of selecting that which gives the best results. Mobiloil B and tractor crank-case oil have been used in the bureau's laboratory with satisfaction.

Lumnite cement, either as a stiff paste or in a 1:1 mortar mixture, should give satisfactory results if sufficient care is taken to supply all the water required for its hydration. When Lumnite cement is used the freshly capped specimens should be kept in a moist closet or thoroughly surrounded by saturated burlap bags for at least 12 hours. If the initial curing is done properly, cores capped with Lumnite cement may be tested 24 hours after capping. Portland cement paste gives satisfactory results, especially if a 4 per cent solution of calcium chloride is used as the mixing water. This accelerates the hardening of the cement sufficiently to allow a compression test to be made after three days of curing.

The chief requisites of a material suitable for capping specimens are that its strength at the time of the test shall be at least equal to that of the specimen and that the modulus of elasticity shall be as nearly that of the concrete under test as it is feasible to obtain.

(Continued from page 179)

Series 177 to 181, volcanic ash.—The ash is a siliceous material from Sheridan County, Nebr. It was mixed in the batch dry with the cement. Insoluble in cold water and free from organic impurities, it has the following screen analysis:

Screen sizes	Per cent
Passing No. 30, retained on No. 50	0.0
Passing No. 50, retained on No. 100	0.4
Passing No. 100, retained on No. 200	8.4
Passing No. 200	91.2
	100.0

Series 182 to 186, Alkagel "A".—This is the trade name for a colloidal product manufactured at Duluth, Minn. It was dissolved in the mixing water and added to the batch.

Series 294 to 295, sulphur.—Powdered commercial sulphur was used in series 295 and the same material was used in the impregnated series 294.

COMMITTEE ON TESTS AND INVESTIGATIONS MEETS

THE subcommittee on tests and investigations of the committee on standards of the American Association of State Highway Officials met in Washington at the Bureau of Public Roads on September 29 and 30.

A part of the time of the committee was taken up with discussion of revision of United States Department of Agriculture Bulletin No. 1216, "Tentative Standard Methods of Sampling and Testing Highway Materials." Several changes relating to methods of procedure were recommended. The new standard screen scale for mechanical analysis of fine aggregate for concrete recently adopted by the American Society for Testing Materials was also adopted by the subcommittee. The former requirement was for screens with square openings, whereas round openings are now optional for the ¼-inch size and larger. The new screen scale permits of a greater variety of sieves than the old scale.

The committee received and discussed several research reports, such as Investigations of the Relation Between the Transverse and Compressive Strength of Concrete, The Value of Stone Screenings in Place of Sand as Fine Aggregate in Concrete, The Use of Rail Steel for Reinforcing in Concrete, Methods of Determining the Proportions of Hardened Concrete, Methods for the Determination of the Asphaltic Content of Road Oils, and Methods for Determining Toughness of Bituminous Mixtures. Specifications for concrete culvert pipe and for paints were also discussed.

The following members were in attendance:

H. A. Hall, Florida.
H. F. Clemmer, Illinois.
R. W. Crum, Iowa.
Mark Morris, Iowa.
F. C. Lang, Minnesota.
W. F. Purrington, New Hampshire.
J. G. Bragg, New Jersey.
F. H. Bauman, New Jersey.
W. L. Blaum, New York.
J. E. Myers, New York.
A. S. Rea, Ohio.
R. I. Giles, North Carolina.
O. H. Hansard, Tennessee.
J. V. Keily, Rhode Island.
H. S. Mattimore, Pennsylvania (chairman).
M. H. Ulman, Pennsylvania.
Shreve Clark, Virginia.
R. B. Dayton, West Virginia.
J. R. Boyd, Bureau of Public Roads.
I. B. Mullis, Bureau of Public Roads.
A. C. Rose, Bureau of Public Roads.
L. G. Carmick, Bureau of Public Roads.
H. M. Milburn, Bureau of Public Roads.
F. H. Jackson, Bureau of Public Roads (secretary).
F. H. Janda, National Research Council.

(Continued from page 168)

BOTTOM EDGE TENSION DOUBLE TOP CORNER TENSION

The data indicate that with a load passing along a path 9 inches from the edge of the pavement the tension produced in the top of the slab by a load at the corner is always less than the tension in the bottom of the slab directly under the wheel. The relative magnitude of these two tensions was compared by expressing the deformation recorded along the diagonal in the top of the slab as a percentage of the average maximum deformation recorded by four gauges in the bottom edge of the slab with the following result:

Wheel load	Ratio of stress along diagonal to stress along edge
4-wheel truck:	*Per cent*
4,000 pounds	42
5,000 pounds	40
6,000 pounds	45
7,000 pounds	41
8,000 pounds	57
6-wheel truck:	
Average all loads .... pounds	48

The gauge in the diagonal was located 48 inches from the corner, or about 35 inches from the wheel, when the maximum stress was produced. Shifting the position of this gauge along the diagonal might change the recorded stress, although data from previous tests were used in locating it. These percentage values are merely indications, and are included only to give an idea of the approximate relation between these stresses in a uniform slab where there is little or no support across the transverse joint.

Summing up, the following may be stated as the indications of this investigation:

1. The deflection of a concrete pavement is directly proportional to the load applied (within the limits of this investigation).
2. A load passing along the test pavement (uniform thickness, 6 inches) 9 inches from the edge of the pavement produced approximately twice the fiber deformation in the edge of the pavement that was caused by the same load passing along a path 21 inches from the edge.
3. The tension produced in the top of a pavement due to counterflexure between the wheels of a six-wheel vehicle is less than the tension produced in the bottom of the pavement directly under the wheel regardless of the axle spacing.
4. In the case of six-wheel vehicles the maximum tension produced in the pavement seems to be a function of the wheel load and not of the axle spacing, at least between the limits of 3 feet and 10 feet.
5. Within the deformation limits obtained in this investigation (maximum unit fiber deformation of about 0.0001 inch per inch) the fiber deformation in the pavement is directly proportional to the load.
6. In a pavement slab of uniform thickness the maximum deformation occurs along the edge of the slab for both four-wheel and six-wheel vehicles.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets or to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

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

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
*347. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370. The Results of Physical Tests of Road-Building Rock. 15c.
386. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
390. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463. Earth, Sand-Clay, and Gravel Roads. 15c.
*532. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*586. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1916. 10c.
*660. Highway Cost Keeping. 10c.
670. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
*691. Typical Specifications for Bituminous Road Materials. 10c.
*704. Typical Specifications for Nonbituminous Road Materials. 5c.
*724. Drainage Methods and Foundations for County Roads. 20c.
*1077. Portland Cement Concrete Roads. 15c.
*1132. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
1216. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
No. 1259. Standard Specifications for Steel Highway Bridges adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULAR

No. 94. TNT as a Blasting Explosive.

FARMERS' BULLETINS

No. *338. Macadam Roads. 5c.
*505. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

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

OFFICE OF PUBLIC ROADS BULLETIN

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

OFFICE OF THE SECRETARY CIRCULARS

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

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

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

* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

SEPTEMBER 30, 1925

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1926									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1925			PROJECTS COMPLETED SINCE JUNE 30, 1925			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 5,970,097.71	$ 2,863,197.86	611.8	$ 2,596,734.31	$ 1,227,424.29	134.4	$ 13,119,144.41	$ 6,363,120.19	694.2	$ 378,592.64	$ 175,050.00	11.6	$ 2,179,863.66	Alabama
Arizona	9,580,133.43	5,016,119.94	613.8	237,926.80	145,397.06	7.0	1,969,799.31	1,249,914.42	157.3	276,880.71	180,420.97	35.5	1,969,488.61	Arizona
Arkansas	13,310,190,08	5,380,181.73	1048.9	1,680,021.97	749,315.78	60.1	6,722,805.84	3,147,432.39	415.0	1,076,958.63	506,303.04	85.8	554,664.06	Arkansas
California	22,346,175.99	10,719,249.61	894.8	1,026,130.48	509,752.08	24.4	10,429,810.75	5,129,483.09	270.5	1,650,336.56	732,415.86	89.1	2,497,208.36	California
Colorado	11,876,703.94	6,067,814.34	651.2	563,724.78	301,210.67	20.9	4,592,631.05	2,283,554.88	222.6	196,031.87	109,947.34	13.6	2,182,900.77	Colorado
Connecticut	4,558,639.29	1,819,368.66	101.6	575,452.78	171,947.14	9.5	1,909,405.24	609,235.48	29.4	38,649.99	19,324.00	1.5	1,240,377.72	Connecticut
Delaware	4,281,559.81	1,496,190.65	107.1	383,163.86	180,583.85	12.1	1,056,711.56	420,745.85	22.9				10,912.65	Delaware
Florida	2,959,273.72	1,405,487.97	96.3				8,970,318.13	4,370,709.60	251.8	593,589.09	204,181.77	11.8	1,207,389.66	Florida
Georgia	20,156,002.37	9,406,366.46	1478.3	1,533,977.67	752,849.49	108.9	11,197,627.34	5,428,240.05	693.9	1,386,866.86	435,218.24	91.0	428,089.76	Georgia
Idaho	9,394,676.80	4,815,332.26	600.1	176,117.58	101,367.48	21.0	2,711,794.26	1,707,146.35	176.5	680,788.60	411,380.96	37.3	587,810.95	Idaho
Illinois	40,010,481.10	18,640,076.28	1236.2	688,967.89	340,741.22	25.0	8,598,101.74	4,165,939.64	289.9				3,509,824.86	Illinois
Indiana	13,639,172.65	6,562,455.68	422.1	1,132,976.57	544,542.84	41.2	17,042,952.77	7,969,148.90	471.2	112,160.81	56,080.40	4.2	1,136,237.18	Indiana
Iowa	27,272,285.21	11,107,492.99	1996.9	102,280.50	39,495.95	16.3	7,671,624.74	3,441,026.98	463.7	1,502,174.74	696,327.65	122.9	2,140,750.43	Iowa
Kansas	26,399,695.77	9,755,273.32	831.4	2,492,110.88	1,067,579.28	115.1	10,837,265.30	4,627,627.02	530.9	1,532,784.09	465,023.99	103.5	1,476,741.39	Kansas
Kentucky	14,832,324.28	6,205,994.59	584.9	1,356,407.10	565,020.08	45.6	7,950,326.58	3,550,910.53	306.1	79,653.87	39,826.93	7.0	1,434,247.87	Kentucky
Louisiana	11,939,424.97	5,279,870.86	927.6	875,723.58	411,312.04	65.0	2,772,603.62	1,325,240.49	108.8	567,118.64	283,056.89	21.8	972,163.72	Louisiana
Maine	8,174,281.31	3,907,870.33	281.4	173,275.15	85,733.59	6.6	1,311,335.31	551,480.38	42.2	973,159.26	392,833.36	35.6	843,336.34	Maine
Maryland	8,132,506.90	3,849,383.15	294.4	271,012.37	128,725.85	9.7	2,919,045.57	1,293,531.65	131.3				18,792.35	Maryland
Massachusetts	14,047,656.22	5,467,661.28	300.6	505,080.98	165,015.60	11.0	5,894,811.24	1,566,221.98	84.9	523,237.49	173,035.13	8.1	1,647,737.01	Massachusetts
Michigan	16,234,000.80	7,328,316.91	612.6	2,807,574.86	1,334,319.27	115.3	13,614,403.05	6,276,891.49	389.2	309,916.00	113,210.57		3,072,208.76	Michigan
Minnesota	30,415,685.89	12,738,642.04	2721.2	1,605,828.93	748,422.07	176.7	8,420,751.61	3,475,400.00	687.8	844,036.79	52,800.00	48.5	446,347.89	Minnesota
Mississippi	10,292,285.79	4,988,702.73	803.4	1,037,377.99	517,960.94	76.8	8,275,985.70	4,131,737.19	434.5	866,500.44	428,250.19	68.5	768,163.95	Mississippi
Missouri	17,368,156.57	8,219,411.43	1118.9	1,765,817.26	852,178.33	85.7	26,543,802.25	10,764,686.97	788.7	1,854,369.22	145,005.53	42.9	398,306.74	Missouri
Montana	10,156,600.41	5,317,523.15	921.6	448,444.40	390,460.39	49.5	1,823,262.71	1,278,779.76	150.7	882,285.54	461,565.89	102.7	4,424,895.81	Montana
Nebraska	9,306,374.36	4,389,523.50	1570.6	1,112,300.53	537,102.43	99.6	8,779,538.78	4,358,961.31	880.9	2,594,699.03	1,268,245.81	228.6	2,493,273.95	Nebraska
Nevada	4,917,465.69	3,088,299.78	357.3	176,699.88	131,175.94	18.2	5,229,691.47	4,383,698.11	448.4	111,809.24	98,079.05	19.5	145,644.12	Nevada
New Hampshire	4,165,687.86	1,986,226.87	208.1				1,232,186.05	581,902.15	40.3	56,895.78	22,710.00	1.5	213,027.98	New Hampshire
New Jersey	11,961,357.45	3,820,679.99	219.1	568,121.43	131,911.06	8.1	10,212,826.36	3,529,508.12	83.5				50,612.83	New Jersey
New Mexico	8,717,999.18	4,914,070.61	1081.3	2,086,565.12	1,394,254.27	170.9	2,560,206.42	1,625,371.41	252.8	272,777.67	190,930.39	17.4	1,660,495.32	New Mexico
New York	28,597,760.67	12,229,076.53	831.5	2,744,698.59	1,066,085.79	67.9	31,735,025.75	10,375,109.05	641.2	8,859,600.00	2,222,502.50	147.5	4,506,255.13	New York
North Carolina	21,014,450.41	8,746,454.59	1119.8	2,066,128.35	867,034.98	62.1	7,708,532.34	3,340,365.95	193.2	1,449,872.60	596,457.85	37.2	458,338.63	North Carolina
North Dakota	10,829,263.82	5,268,930.47	1917.5	573,986.80	286,862.87	141.9	3,749,139.68	1,890,221.73	479.3	1,012,332.91	506,162.18	117.7	1,602,761.75	North Dakota
Ohio	41,572,252.81	15,244,993.93	1191.1	1,843,360.94	644,759.37	56.4	12,024,843.47	4,554,772.20	363.0	1,574,151.83	488,740.22	36.7	2,021,493.28	Ohio
Oklahoma	20,787,024.94	9,672,890.34	852.2	1,529,982.01	684,504.73	52.3	7,147,660.01	3,415,280.80	338.5	795,052.10	267,008.83	31.4	267,857.30	Oklahoma
Oregon	14,388,188.70	7,142,364.63	794.6	424,964.52	235,853.72	26.1	3,531,754.07	2,049,888.22	179.3	508,820.95	137,767.25	23.0	130,528.18	Oregon
Pennsylvania	43,054,835.19	16,222,023.97	850.3	1,834,714.33	513,286.76	32.5	35,159,598.89	9,353,875.14	612.5	3,935,957.11	1,344,278.66	111.8	558,396.47	Pennsylvania
Rhode Island	2,628,496.20	1,119,688.09	64.8				2,543,268.84	681,939.59	37.2				500,316.32	Rhode Island
South Carolina	11,163,347.84	5,121,267.54	1235.9	1,051,972.51	357,678.74	58.5	6,010,468.85	2,824,091.68	328.9	892,715.51	381,496.01	61.5	64,997.03	South Carolina
South Dakota	12,091,434.67	5,989,879.00	1447.9	1,220,287.86	585,884.32	141.9	6,871,535.15	3,272,429.89	972.2	169,162.07	44,213.78	28.2	52,185.01	South Dakota
Tennessee	13,789,140.98	6,732,079.77	497.9	2,555,128.53	1,261,638.03	84.4	9,707,053.23	4,327,533.01	338.4	386,669.75	149,846.57	17.5	1,191,074.62	Tennessee
Texas	54,120,970.83	21,057,940.12	3907.1	3,903,731.98	1,588,337.77	229.0	22,953,818.50	10,048,344.36	1354.9	4,093,076.77	1,818,327.52	309.6	1,666,564.23	Texas
Utah	6,259,159.41	3,818,836.91	423.1	132,178.99	99,880.35	5.9	3,224,624.68	2,193,626.23	255.5	23,746.72	17,054.73	1.0	841,129.78	Utah
Vermont	3,015,174.51	1,452,894.45	107.8	153,910.93	76,897.43	3.2	1,777,483.29	777,488.36	38.3	145,406.47	72,703.23	4.8	522,898.53	Vermont
Virginia	13,099,720.01	6,271,998.20	676.2	3,336,056.31	1,597,695.44	139.2	7,989,756.56	3,488,740.13	252.2	1,204,196.14	560,587.67	44.9	136,640.56	Virginia
Washington	13,352,504.18	6,117,211.87	526.7	1,315,032.75	550,338.64	64.1	3,080,756.36	1,427,900.00	98.5				920,245.49	Washington
West Virginia	7,343,200.86	3,230,293.33	326.7	252,664.30	123,013.89	12.8	6,780,995.84	2,722,663.90	179.1	981,982.41	367,090.29	35.2	115,513.59	West Virginia
Wisconsin	21,807,140.91	8,919,640.62	1451.7	107,355.47	53,526.17	6.6	5,681,321.94	2,794,796.59	277.6	484,386.26	238,519.50	35.7	3,562,070.12	Wisconsin
Wyoming	8,809,819.33	4,739,096.67	982.0	622,379.97	374,550.58	50.4	3,650,537.43	2,297,442.53	251.2	73,606.61	47,255.00	9.9	172,335.22	Wyoming
Hawaii							342,277.22	97,440.00	6.5				637,088.00	Hawaii
TOTALS	740,140,790.92	325,654,346.00	41898.3	53,648,350.79	24,493,628.57	2769.8	396,041,221.25	171,541,585.74	16717.4	45,943,009.77	16,921,235.75	2263.5	59,639,203.94	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 104,492,191.53 Federal aid $ 46,448,700.02 Miles 4,531.4