PUBLIC ROADS

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BUREAU OF PUBLIC ROADS

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THE BRADLEY LANE EXPERIMENTAL ROAD

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THE BRADLEY LANE EXPERIMENTAL ROAD

By the Division of Tests, United States Bureau of Public Roads

BRADLEY LANE is a highway in Maryland near the District of Columbia line, and connecting Connecticut Avenue with Wisconsin Avenue, which until recently served as a main traffic outlet from Washington to the West. The thoroughfare is 3,800 feet long and has always carried heavy traffic.

Prior to 1911 it was maintained as a limestone macadam road. During the period between 1911 and 1915 various surface treatments were tried.[1] In 1915 the macadam was widened and a bituminous macadam surface applied. In 1923 the road was further improved by the construction of a concrete shoulder along the south edge of the pavement. A general plan of the road at the present time is shown in Figure 1. Figure 2 shows in detail the surface improvements which were made at different times.

SURFACE TREATMENT FIRST APPLIED IN 1911

Prior to 1911 the concrete shoulder on the south side of the road had not been constructed and the cross and longitudinal drains, shown in Figure 1, had not been installed. The concrete sidewalk shown on the north side of the road, however, had been built. The width of the surface was 10 to 12 feet.

Numerous failures, due to an insufficient amount of road metal and to the absence of means for removing surface water, made the road almost impassable at certain seasons of the year during its early life. This difficulty was gradually eliminated by the adjoining farm owners, who for several years hauled in materials and filled the holes as the soft spots developed. By 1911 the road had developed into a smooth, well bonded, and fairly stable structure with a smooth limestone wearing course.

Surface treatment was first applied in 1911. Two types of material—semiasphaltic oils, and a molasses-lime mixture, were used on locations as shown in Figure 2. The west end of the road (440 feet) adjoining Wisconsin Avenue was not treated.

SEMIASPHALTIC OIL TREATMENTS (EXPERIMENT 1)

Experiment 1 was divided into three sections, which varied primarily in the amounts of oil used and, to some extent, in the method of treatment.

Before applying the oil the limestone macadam surface was swept to remove dust and loose particles of stone. The oil, which was applied at a temperature of 170° F., was spread by hand brooming and covered with stone chips (three-fourths to one-fourth inch) after each application. Rolling of the surface completed the operation.

Section 1 began 50 feet west of the west rail of the Connecticut Avenue car tracks, extended west 811 feet and comprised 1,216 square yards. The oil was spread in two applications of 0.61 and 0.31 gallon per square yard, respectively, and each application was followed by an application of stone chips. The characteristics of the oil used are given in Table 1.

[1] Reports describing the construction and behavior of these experiments are included in Office of Public Roads Circulars 98 and 99, U. S. Department of Agriculture Bulletins 105, 257, 407, and 586, and U. S. Department of Agriculture Circular 77.

CONDITION OF SURFACE AFTER SURFACE TREATMENT IN 1911

Section 2 was 256 feet long and comprised 384 square yards. The oil was spread in a single application of 0.61 gallon per square yard. Construction details were identical with those of section 1.

Section 3 was 1,188 feet in length and had an area of 1,784 square yards. The treatment of this section was identical with that of section 1.

TABLE 1.—*Analysis of the refined semiasphaltic oil*[2] *used in the 1911 surface treatment of experiment 1*

Specific gravity, 25°/25° C	0. 972
Specific viscosity, Engler, 100 c. c., 100° C	9. 5
Per cent loss at 105° C., 11 hours, 20 grams	2. 1
Per cent loss at 163° C., 5 hours additional	. 68
Float test on residue[3] at 32° C. (seconds)	288
Bitumen insoluble in 86° B. naphtha (per cent)	8. 72
Fixed carbon (per cent)	7. 36
Bitumen soluble in carbon disulphide (per cent)	99. 79
Organic matter insoluble (per cent)	. 19
Inorganic matter insoluble (per cent)	. 02

MOLASSES-LIME MIXTURE (EXPERIMENT 2)

The molasses-lime mixture used in experiment 2 was selected because of the fairly satisfactory results secured with a similar treatment at Newton, Mass. This type of treatment was suggested by the knowledge that a mixture of molasses and lime results in the formation of calcium sucrates which are tough, sticky salts, not readily soluble in water. Laboratory results have shown that properly proportioned mixtures of molasses, lime, and rock dust or clay will develope a high binding value. Waste molasses, known as black strap, could be obtained at a comparatively low cost at sugar refineries, and it was thought that an efficient and economical method of surface treatment for localities near refineries might be developed.

The molasses-lime mixture experiment, which was contiguous to experiment 1, was 1,060 feet in length, averaged 13 feet in width, and comprised approximately 1,590 square yards. At the time of applying the treatment, the macadam surface was well bonded and in good condition, and was swept and then given

[2] Viscous fluid, fairly stable; contained some water.
[3] Granular surface, very viscous, fairly sticky.

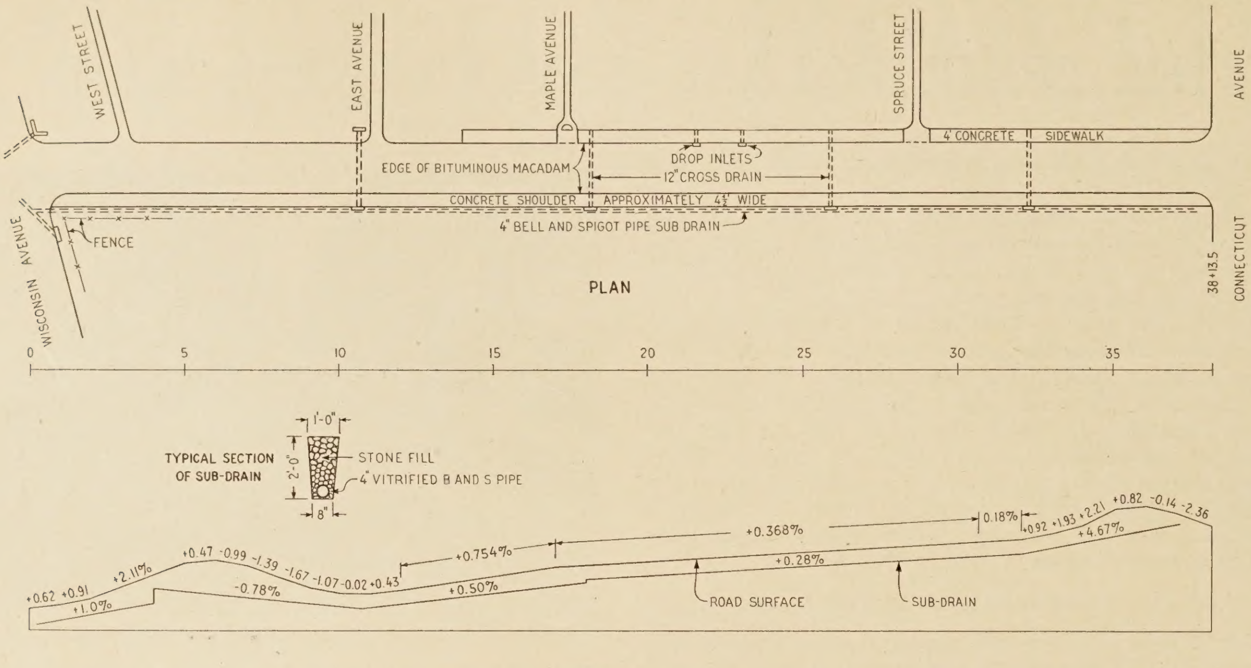


Figure 1.—Plan and Profile of Bradley Lane Showing Drainage System Installed in 1915

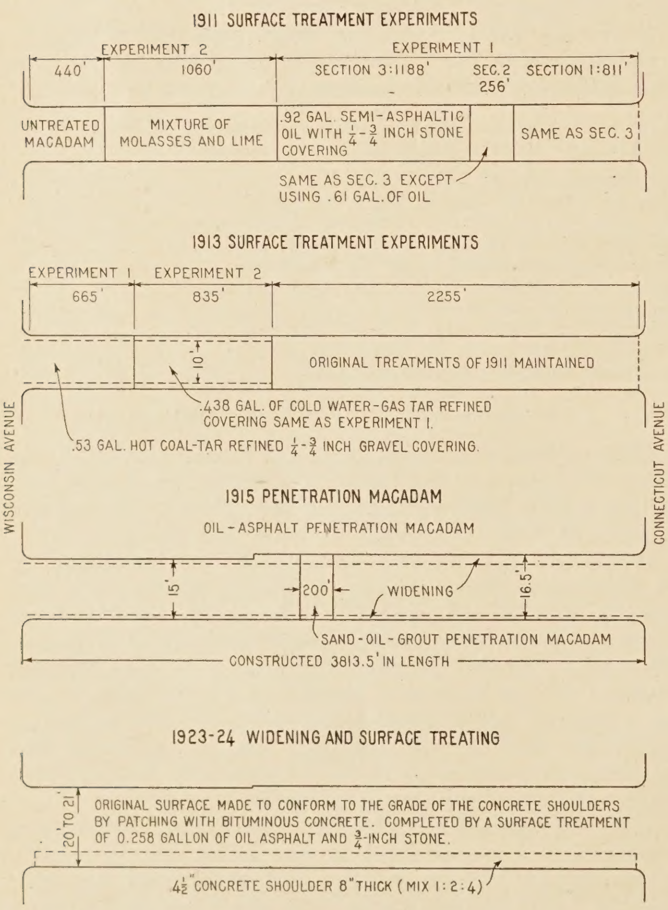


Figure 2.—Plan and Details of the Bradley Lane Experiments, 1911 to 1928

Condition of Road in May, 1912

a light sprinkling of water to lay and saturate the remaining dust.

Laboratory tests were made to determine the proportions of the materials to use in the mixture. It was found that 360 pounds of lime, 150 gallons of molasses, and sufficient water to yield a volume of 450 gallons produced a mixture which was quite sticky, and yet could readily be sprayed on the road with an ordinary sprinkler.

The method of operation was crude and, as the work was done in December, the difficulty experienced in mixing was largely increased by the cold weather and resulting high viscosity of the molasses. The lime was slaked in barrels to form a thick cream and then mixed with molasses and the requisite amount of water. The solution was then sprayed on the road in an amount sufficient to saturate the surface thoroughly, which was found to be a rate of approximately 0.26 gallon per square yard. As it dried quickly, it was unnecessary to detour traffic.

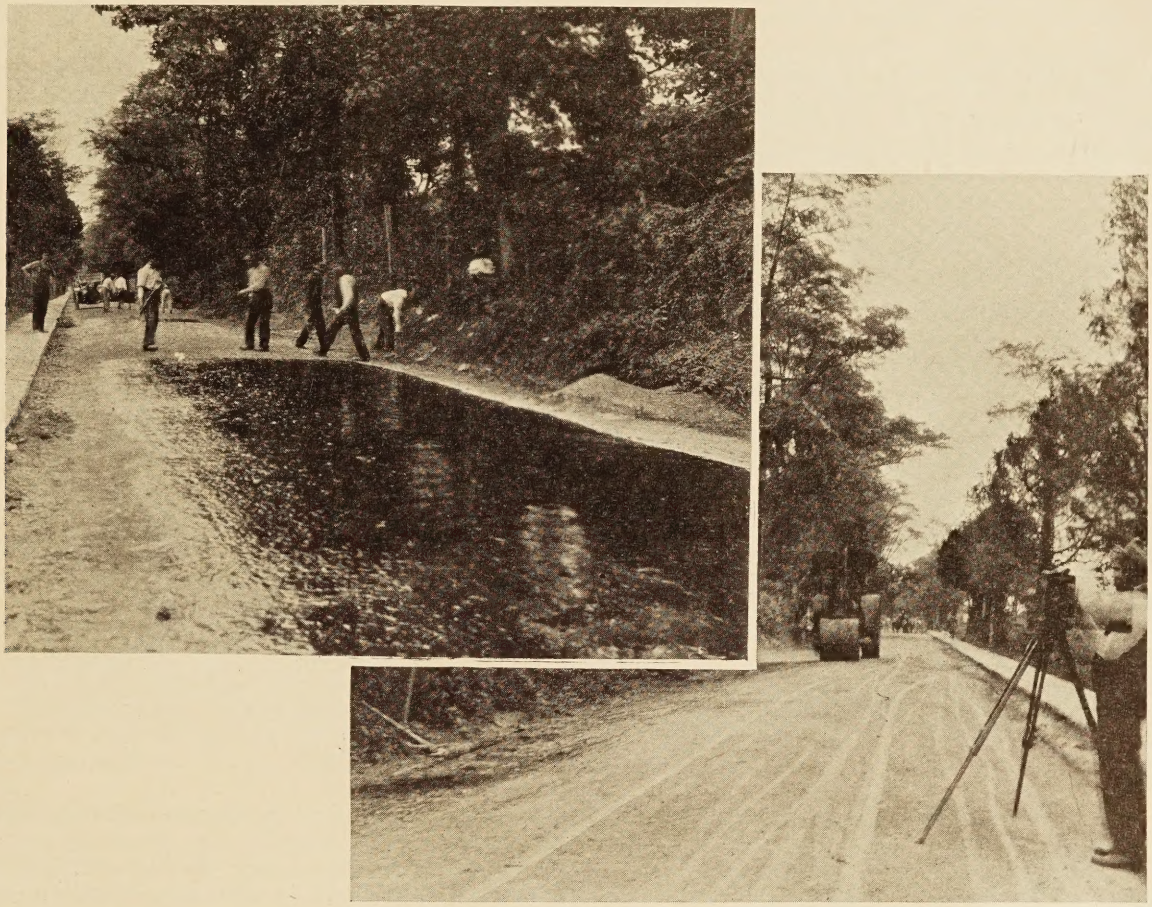
At the time these experiments were conducted, traffic over the road was classed as heavy for section 1 of experiment 1, and as ordinary suburban traffic over the other sections.

Sections of the first experiment were in only fair condition one year after construction. Traffic was concentrated in a single lane because of the narrowness of the roadway, and resulted in the formation of two broad, shallow ruts, particularly in the thick, matted covered sections, 1 and 3, of experiment 1. These ruts developed primarily from the shoving of the surface mat, but in a few places because of insufficient foundation support. Sections 1 and 3 suffered greater surface displacement than did section 2, on which the single application of oil was spread. There is no doubt that the instability of sections 1 and 3 was due primarily to the use of too much bituminous material, and insufficient stone chips since section 2 remained in excellent condition.

An attempt was made to stabilize the bituminous mat on a portion of section 1 by spreading a light course of crushed rock over the surface. This treatment was somewhat effective but it was not considered sufficiently successful to warrant its use on the whole of sections 1 and 3.

By the end of 1913 excessive shoving and rutting had developed in the traffic lane on these sections. The bituminous surface mat was cut through for long distances and many depressions had occurred. The surface mat which had been displaced was used in filling the worst depressions and when tamped into place and covered with a layer of screenings, developed into a firm and satisfactory patch. Additional bare places were covered with a light application of water-gas tar and screenings. This repair work greatly improved the condition of the road. Traffic for the year 1913 averaged 232 vehicles daily.

Experiment 2, on which the molasses-lime mixture was used, still showed evidences of the treatment one month after construction although several heavy rains had occurred. Later rains during the year removed all traces of it, and the experiment took on the appearance of the adjoining section on the west, which had been maintained as untreated macadam.



Applying Surface Treatment in 1911

SURFACE TREATMENT APPLIED IN 1913

This work, as shown by Figure 2, consisted of surface treating both that portion of the road which in 1911 had received no treatment and also the area treated with the molasses-lime mixture.

The badly rutted and worn limestone surface was first scarified, reshaped, and rolled, after which a layer of crushed limestone, 3 to 3½ inches thick when compacted, was added and finished as a water-bound macadam surface, 10 feet wide. Later in the fall, after the surface had become thoroughly bonded, bituminous materials were applied by means of a pressure distributor and were covered with a layer of gravel.

On experiment 1 (665 feet in length) refined coal tar was applied hot at the rate of 0.530 gallon per square yard, and on experiment 2, consisting of 835 feet adjoining on the east, 0.438 gallon per square yard of water-gas tar was used. Analyses of the tars used are given in Table 2.

One year after construction these two experiments on the west end of Bradley Lane showed the detrimental effect of concentrated traffic. Large quantities of building material were being hauled by horse-drawn steel-tired vehicles. Vehicles, in passing, were compelled to run off the road metal, causing excessive wear and breaking down on the edges, thereby reducing the effective road width.

By 1915 the condition of the entire road had become such that it was impossible, with ordinary maintenance methods, to prevent the occurrence of rutting in appreciable amounts, the formation of potholes and shearing and cutting of the edges by traffic. Insufficient width of pavement, lack of adequate foundation, and lack of artificial drainage were considered as causing the undesirable conditions referred to. It is probable that the large volume of steel-tired traffic caused a more rapid deterioration than would have been the case with rubber-tired traffic which, at about this time, began to replace the steel-tired traffic. The old experiments were discontinued and arrangements made to rebuild the road as a bituminous macadam constructed by the penetration method.

Table 2.—*Analyses of tars used in the 1913 surface treatment experiments*

	Experiment 1	Experiment 2
Material	(1)	(2)
Specific gravity, 25°/25° C	1.204	1.121
Float test at 32° C. (seconds)	92	------
Specific viscosity, Engler, 50 c. c. at 50° C	------	17.5
Free carbon (per cent)	14.93	3.33
Distillation, per cent by weight:		
Water	[3] 1.3	Trace.
110° C	Trace.	[4] 0.2
110°–170° C	0.4	[5] 2.4
170°–270° C	[6] 11.9	[4] 24.3
270°–315° C	[7] 6.3	[4] 12.4
Pitch residue	[8] 79.8	60.2
Total	99.7	99.5

[1] Refined coal tar.
[2] Refined water-gas tar.
[3] From leaky coils in tank car.
[4] Clear.
[5] Turbid.
[6] Four-fifths solid.
[7] One-fourth solid.
[8] Fairly glossy; brittle.

BRADLEY LANE RECONSTRUCTED IN 1915

The work consisted of installing an artificial drainage system for removing the free water and the construction of a bituminous penetration macadam wearing surface. As shown by Figure 2, the new pavement was made 16½ feet wide throughout the eastern portion of the road (2,404 feet) and 15 feet wide throughout the western portion (1,409.5 feet).

Considerable effort was made to secure better drainage by increasing the stability of the foundation. A French drain with 4-inch tile was placed at a depth of approximately 2 feet, 11½ feet from the center of the road on the south side, for a distance of 3,646 feet. All existing culverts were cleaned out, and, where necessary, the inlets and outlets paved with spalls. The two catch basins, which were deemed inadequate, were reconstructed and enlarged and were supplemented by the addition of two new ones at other points. Figure 1 shows the plan of the drainage structures upon completion of the work done in 1915.

Prior to constructing the new wearing course, the old surface-treated road was scarified, reshaped, and widened by the addition of new material to form the foundation course for the new surface.

After the foundation course was thoroughly compacted by rolling, the new wearing surface was constructed. With the exception of a 200-foot section, the new wearing surface consisted of a 3-inch layer of limestone (loose measure) constructed as an ordinary penetration macadam. One and one-half gallons of oil asphalt per square yard was applied to the compacted limestone surface after which screenings were spread and the surface rolled. A seal coat of approximately one-half gallon per square yard of the same bituminous material and limestone chips was added.

In the construction of the 200-foot section referred to (stations 19 to 21), it was decided to deviate from the straight penetration method by adding sand to the hot oil and applying it as a grout. The section was divided into two parts, one containing 50 per cent of sand by volume in the mix and the other 33 per cent. Some difficulty was encountered in obtaining a uniform mixture of sand and oil and also in obtaining proper penetration of the grout. The amount of bitumen used was 1.83 gallons per square yard and the slight saving in bituminous material was less than the cost of heating and mixing the materials. During the year following construction no distinction was apparent either in appearance or behavior between this section and the plain penetration macadam. In subsequent maintenance the whole project has been considered as a single experiment, no account being taken of the variation in construction.

The same bituminous material was used throughout the experiment, the analysis of which is given in Table 3. The cost of construction, exclusive of the wearing course was 27.6 cents per square yard. The wearing surface cost 32.84 cents per square yard for the straight penetration and 39.42 cents for the sand-oil grout method.

The top dressing of limestone chips rapidly ground up under traffic, leaving the wearing course stone visible over the entire road. In July, 1916, it was given a surface treatment of residual asphaltic petroleum, the characteristics of which are given in Table 4. The oil was applied at the rate of 0.195 gallon per square yard and covered with a hard, waterworn, river gravel five-eighths inch in size. The cost of this treatment was 7.29 cents per square yard.

In the spring of 1917 the surface was in excellent condition. No distinction was apparent either in

Table 3.—*Analysis of the oil asphalt [4] used in the penetration macadam experiment of 1915*

Specific gravity, 25°/25° C	1.024
Melting point (° C.)	50
Penetration, 25° C., 100 gm., 5 seconds	105
Loss at 163° C., 5 hours, 20 grams (per cent)	0.19
Penetration of residue [4]	64
Bitumen insoluble in 86° B. naphtha (per cent)	25.77
Fixed carbon (per cent)	15.74
Soluble in carbon disulphide (per cent)	99.70
Organic matter insoluble (per cent)	.18
Inorganic matter insoluble (per cent)	.12

[4] Fairly hard, glossy, sticky semisolid.

Table 4.—*Analyses of residual asphaltic petroleums used in surface treating Bradley Lane*

	1916	1919	1924
Specific gravity, 25°/25° C	0.993	0.988	1.006
Specific viscosity, Engler, 50 c. c., 100° C	13.5	16.0	53.4
Float test, 50° C. seconds	28	--------	--------
Float test, 32° C. seconds	--------	54	355
Flash point, °C	--------	64	210
Loss, 163° C., 5 hours, 20 grams, per cent	17.31	13.1	0.52
Float test on residue, 50° C. seconds	151	175	129
Bitumen insoluble in 86° B. naphtha, per cent	18.63	17.5	19.2
Fixed carbon, per cent	11.25	--------	--------
Soluble in carbon disulphide, per cent	99.89	99.90	99.90
Organic matter insoluble, per cent	.08	.10	.10
Inorganic matter insoluble, per cent	.03	.00	.00

Present Condition of Bradley Lane. The Drainage on the Left is Toward the Road Surface

appearance or wear between the plain penetration macadam and the sand-oil grouted sections. The maintenance cost for 1917 was 0.2 cents per square yard and for 1918 was 1.02 cents. The maintenance work consisted in strengthening and repairing the surface in local places, using a coal-tar product and stone.

In 1919 the roadway was given a surface treatment of hot residual asphaltic petroleum applied at the rate of 0.248 gallon per square yard and covered with ½-inch stone chips.

In spite of a large increase in traffic, the maintenance costs remained relatively low, being 0.38, 1.99, and 0.83 cents per square yard for the years 1920, 1921, and 1922, respectively. The average number of vehicles passing over the road daily had increased to such an extent that it became necessary to provide a wider surface, which was done in 1923.

ROADWAY WIDENED IN 1923

In 1923 the roadway was widened to approximately 21 feet throughout its length by the construction of a concrete shoulder along the south edge 4½ feet wide and approximately 8 inches thick. The shoulder was built to a new grade which was adjusted to allow for a layer of bituminous concrete on the old surface and a reduced crown. The layer of bituminous concrete varied from one-half inch to as much as 2½ inches in thickness, due to the variation in the crown and grade of the macadam. The new bituminous surface had a rough and nonuniform appearance, which was eliminated in 1924 by applying a surface treatment consisting of 0.258 gallon per square yard of hot oil asphalt and ¾-inch stone.

The surface has since continued to remain in good condition generally. The maintenance has consisted only of light patching, no surface treatments having been required since 1924. Figure 3 shows the average daily traffic passing over the road during the years 1915 to 1922, inclusive, and for 1927, together with the accumulated annual cost of maintenance. The low maintenance cost during the last few years is remarkable in view of the large increase in traffic. The average number of vehicles increased from 279 in 1915 to 1,048 in 1922 and to 3,139 in 1927. During the years 1916 to 1922, when the pavement was only 16½ feet wide, the average annual maintenance cost per mile per average daily vehicle was approximately 51 cents, while during the years 1924 to 1927 the corresponding cost for a 21-foot width was only 35 cents. The annual maintenance cost per daily vehicle mile for the years 1916 to 1928 is given in Table 5 and Table 6 gives the cost and description of the work done.

In considering these costs it should be remembered that prices for labor and materials increased greatly, resulting in a greatly decreased purchasing power of the dollar, during the period covered by this report. The costs for the later years therefore are not directly comparable to those of the earlier years.

The traffic in 1927 was ten times that of 1915 and it will probably continue to increase in the future. The present condition of the macadam is generally good except for a few places were slight settlement has occurred. How long the macadam, which up to this time has proved sufficiently stable to overcome lack of drainage, will continue to support further increased traffic can not be estimated. However, in view of the present condition of the road it is reasonable to expect that under existing conditions it will continue indefinitely to give good service at a moderate maintenance cost. The present condition of the road is shown by the illustrations.

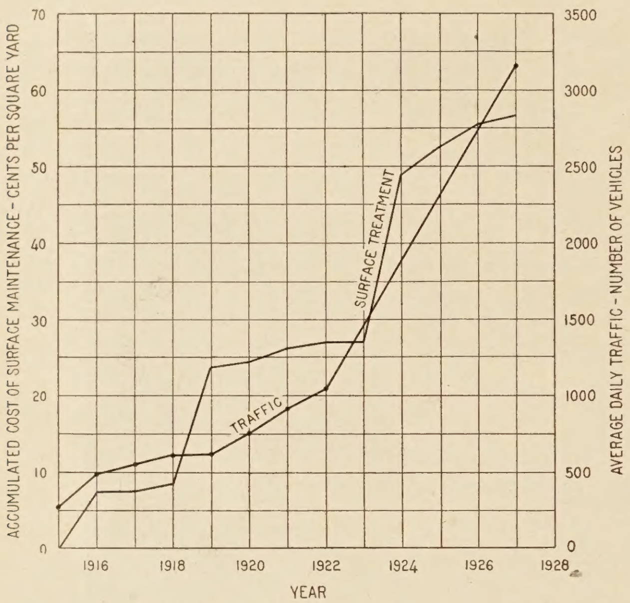


FIGURE 3.—RELATION BETWEEN TRAFFIC AND SURFACE MAINTENANCE COST ON BRADLEY LANE

TABLE 5.—*Average annual maintenance cost per vehicle-mile*

Year	Area	Maintenance cost per square yard	Average daily traffic	Annual cost per daily vehicle mile [1]
	Sq. yds.	*Cents*		
1915	([2])			
1916	6,788	7.29	485	$1.415
1917	6,788	.20	550	.034
1918	6,788	1.02	615	.156
1919	6,788	15.38	615	2.353
1920	6,788	.38	750	.048
1921	6,788	1.99	910	.205
1922	6,788	.83	1,048	.074
1923	([2])			
1924	8,678	21.96	[3] 1,900	1.382
1925	8,678	3.44	[3] 2,300	.179
1926	8,678	3.16	[3] 2,750	.138
1927	8,678	1.06	3,139	.040

[1] Average annual cost per daily vehicle mile for years 1916 to 1922, inclusive, $0.513; for years 1924 to 1927, inclusive, $0.351.
[2] Reconstruction.
[3] Estimated.

TABLE 6.—*Description and cost of work done on Bradley Lane from 1915 to January, 1928*

Year	Area	Cost of reconstruction per square yard	Maintenance: Surface treatments: Bituminous material	Covering	Cost per square yard	Patrol cost per square yard	Total cost per square yard	Total cost of wearing surface from 1915 to 1928
	Sq. yds.	*Cents*			*Cents*	*Cents*	*Cents*	*Cents*
1915	6,788	[1] 60.74						60.74
1916	6,788		Residual asphaltic petroleum, 0.195 gallon per square yard.	Pea gravel	7.29		7.29	68.03
1917	6,788					0.20	.20	68.23
1918	6,788					1.02	1.02	69.25
1919	6,788		Residual asphaltic petroleum, 0.248 gallon per square yard.	½-inch stone	14.76	.62	15.38	84.63
1920	6,788					.38	.38	85.01
1921	6,788					1.99	1.99	87.00
1922	6,788					.83	.83	87.83
1923	{6,788 {1,890	[2] 44.13 [3] 403.68						210.27
1924	8,678		Oil asphalt (hot), 0.258 gallon per square yard.	¾-inch stone	10.60	11.36	21.96	232.23
1925	8,678					3.44	3.44	235.67
1926	8,678					3.16	3.16	238.83
1927	8,678					1.06	1.06	239.89

[1] Widening pavement from approximately 10 feet to 16½ feet and surfacing as penetration macadam.
[2] Heavy patching on original surface required to bring the grade to that of the concrete shoulder.
[3] Constructing concrete shoulder approximately 4½ feet wide on the south side.

TABLE 7.—*Pavement thickness and classification of soil samples* [1]

Station	North side: Pavement thickness [2]	North side: Field moisture content	North side: Laboratory group No.	Center: Pavement thickness [2]	Center: Field moisture content	Center: Laboratory group No.	South side: Pavement thickness [2]	South side: Field moisture content	South side: Laboratory group No.
	Inches			*Inches*			*Inches*		
0+50	12	Wet [3] [4]	4	12	Wet [3] [4]	4	12	Wet [3] [4]	4
3+50	10½	do. [3] [4]	4	7	do. [3] [4]	4	13½	Rocky [3]	4
6+00	13	Rocky [5]	3	14	Rocky [5]		15	do. [5]	
8+50	12	Wet	4	9	No sample		12	Wet	5
11+00	11	Plastic	4	12	do		15	do	4
13+50	12		4	12½	do			do	2
16+00	8½	Plastic	6	9½	do		12	Plastic	6
18+50	7	Wet	4	11½	do		12	do	4
21+00	12½	Plastic	4	11	do		13½	do	4
23+50	10	Wet	2	10½	do		13	Wet	4
26+10	10	Plastic	4	11	do		10	Plastic	4
28+50	7	Wet [4]	4	11	do		8	Wet [4]	4
31+00	8½	do	4	11	do		14½	do	4
33+50	12½	do. [4]	4	12	do		12	do	4
36+00	8	do. [4]	4	12	do		14	do. [4]	1
37+97	8½	Wet	4	8½	do		10½	Rocky	4

[1] Unless otherwise indicated, soil samples were taken on the earth shoulder just outside the pavement slab.
[2] Obtained by cutting through pavement slab.
[3] Sampled from holes through pavement slab.
[4] Micaceous.
[5] Unable to get sample.

STUDY MADE TO DETERMINE EFFECT OF SUBGRADE VARIABLES

In making a subgrade study during the fall of 1927 holes were cut through the pavement at points near each edge and at the center every 250 feet. The thickness of the pavement as thus obtained is given in Table 7 and Figure 4. At the center it ranged from 7 to 14 inches in thickness with an average of 10.9 inches. On the south side, where heavy patching was made following the construction of the concrete shoulder in 1923, the thickness varied from 8 to 15 inches and averaged 12.5 inches. On the north side it was slightly less than that of the center, varying from 7 to 13 and averaging 10.2 inches. These depths do not include such granular material as was found to have been worked into the subgrade soil. In some cases, even to depths of 1 foot below the pavement, stone fragments of 3-inch maximum size were found to be present in amounts sufficient to resist the penetration of both the auger used for

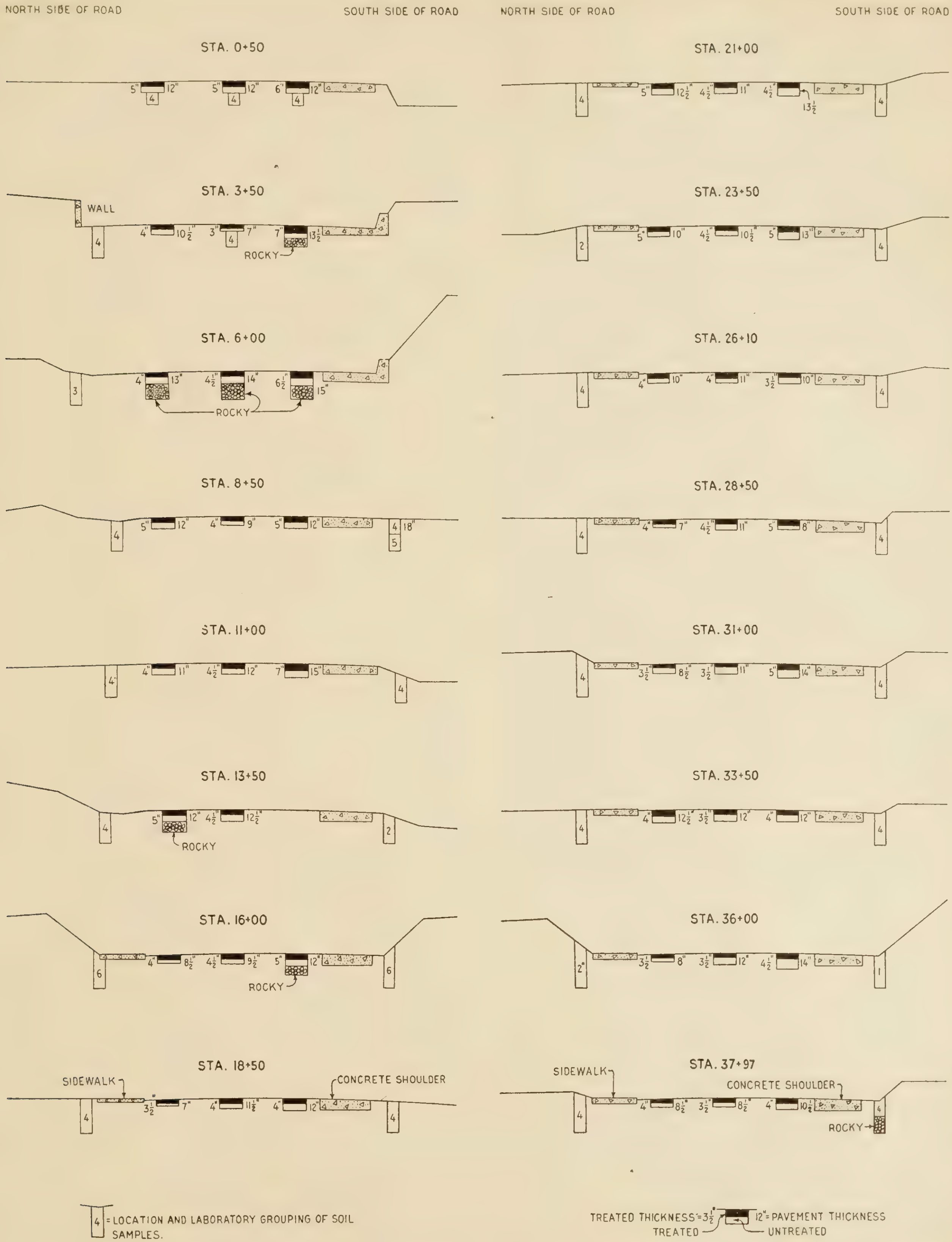


FIGURE 4.—CROSS SECTIONS OF BRADLEY LANE SHOWING PAVEMENT THICKNESS AND LOCATION OF SOIL SAMPLES

securing soil samples and a crowbar. The thickness of the bituminous wearing surface was found to be quite uniform, averaging 4.4 inches in depth.

The drainage on this road has been bad throughout most of the period covered by the experiments. The improvements in drainage made in 1915 were of temporary value only, as the gradual building up of the adjacent property resulted in a decided change in topography. Not only was the artificial drainage affected but the natural drainage as well. The road surface, which formerly had some natural drainage, now serves to a large extent as a drain for the adjoining area.

The south side of the road lies in a cut throughout its length except for a short distance. In building the concrete shoulder the cut was not widened, with the result that the south edge of the pavement extends to the toe of the slope eliminating the shoulder ditch. Surface water, which formerly was drained away from the edge of the pavement, now flows directly to it. The French drain, which lies almost directly under the edge of the pavement, slowly carried off the water, which no doubt affects the support of the subgrade near the edge. On the north side of the roadway the

Present Condition of Bradley Lane at a Point Where Drainage is Toward the Road from Both Sides

sidewalk, which is a trifle higher than the surface, forms a curb with the macadam surface as a gutter. Due to the comparatively flat grade, water stands in this gutter for some time after a rain. Although a few depressions have occurred on this side, caused no doubt by the seepage of some of this water into the foundation, there is little evidence to indicate that the presence of free water is seriously jeopardizing the life of the pavement. A comparison of Table 7 and Figure 4 shows no relation between the slab thickness and the apparent maintenance and drainage conditions.

Soil samples were taken from the test holes at the center and side of the road at stations 0 + 50 and 3 + 50 and at the other 250-foot intervals they were taken only from holes just beyond the edge of the pavment on each side. The samples were taken along the sides of the road because the incorporation of stone fragments in the subgrade under the pavement made sampling at these locations, except as noted, impractical. Such small amounts of soil as were obtained, however, from the subgrade under the pavement were found to be similar to that obtained from either one side or both at the same station. As the samples were taken the field moisture content was determined and from this information they were grouped roughly into three classes, wet, plastic, or dry. The data obtained are given in Figure 4 and Table 7.

Observations made at the time the samples were taken revealed two phases of fine silty loam, one highly micaceous and the other only slightly so. The former was found to occur principally between stations 0 and 8 and in the vicinity of station 35. The soil ranges from a dark gray to a greenish-gray. The slightly micaceous soil was a reddish brown and, in place, gave the appearance of lack of supporting value when wet. Below 2 feet, the soil grades into a red clay subsoil. Typical analyses of the soils are given in Table 8.

Table 8.—*Typical analyses of subgrade soils*

Laboratory Group No.	Lower liquid limit	Lower plastic limit	Plastic index	Shrinkage		Moisture equivalent		Volumetric shrinkage at field moisture equivalent
				Limit	Ratio	Centrifuge	Field	
1	36	36	0	29	1.4	14	32	4
2	32	20	12	19	1.7	23	25	10
3	37	27	10	26	1.5	20	30	6
4	39	24	15	23	1.6	26	29	10
5	48	27	21	22	1.6	32	35	22
6	49	29	20	21	1.7	39	45	41

According to this table the subgrade soils vary widely with respect to affinity for moisture, range of plasticity, and volumetric shrinkage. Comparing the centrifuge moisture equivalent with the shrinkage limit values, one sees that the Group 1 soils gave up their moisture most readily, retaining after centrifuging less than half the moisture contained at the shrinkage limit where the soil has its minimum volume. The soils of Group 3 gave up moisture readily but not to the same extent as those of Group 1. The soils of the other groups in contrast failed to give up moisture so readily. A centrifugal force of one thousand times gravity failed not only to reduce the moisture content to the shrinkage limit but also to the lower plastic limit, above which the supporting properties of soils rapidly decrease. The variation in the plasticity index indicates a variation in the clay content of the soils. Accepting the lineal shrinkage values of 3 and 5 as the boundaries between low and medium and medium and high shrinkage, respectively, the volumetric shrinkages shown in Table 8, translated into terms of lineal shrinkage,[5] show the soils of Groups 1 and 3 to have low shrinkage, those of Groups 2 and 4 to have medium shrinkage, and those of Groups 5 and 6 to have high shrinkage properties.

In addition to the differences just discussed, the test results in Table 8 show the soils of Groups 1, 3, and 6, to be sufficiently spongy to interfere with the proper compaction of a water-bound macadam pavement or base and their efficiency after construction. This sponginess is indicated in slightly different ways, de-

[5] Lineal shrinkage values of 3 and 5 correspond to volumetric shrinkage values of 10 and 18, respectively.

pending upon the soil. For instance, according to the test results, the Group 1 soils consist of fine sand or coarse silt containing an appreciable percentage of coarse, flat flakes. The plasticity index of zero and the low moisture equivalent of 14, both show the absence of clay. The presence of flat particles only can explain the relatively high field moisture equivalent of 32 and lower liquid limit of 36 in the absence of fine silt or clay particles. This is substantiated by the high shrinkage limit of 29, since a round-grained sand only in its loosest possible state could have a void volume represented by this percentage of moisture content. This type of soil is spongy when either wet or dry.

According to the test results, the soils of Groups 5 and 6 are fine micaceous silts or coarse clays. Those of Group 6 contain a larger percentage of small flat particles than those of Group 5. The high plasticity indices of 20 and 21 indicate the absence of coarse particles. The relatively low centrifuge moisture equivalent of 32 of the Group 5 soils compared with the high liquid limit of 48 indicates the absence of very fine clay particles (less than about 0.002 millimeter). The higher centrifuge moisture equivalent of the soils of Group 6 as compared with that of Group 5 soils could be explained by higher clay content, substitution of fine for coarse clay particles, or presence of more flat particles. Of these three possibilities, the first is excluded by absence of increase in the plasticity index; the second is excluded because only the presence of flat particles can cause the field moisture equivalent to increase at a faster rate than the centrifuge moisture equivalent. Furthermore, these flat particles must be relatively small (less than about 0.10 millimeter), otherwise the plasticity index would be appreciably reduced. An increase in the percentage of flat particles sufficient to cause the field moisture equivalent to increase from 35 to 45 while the plasticity index remains unchanged, is sufficient to increase very much the spongy properties of the subgrade soil.

The predominating soil type (Group 4) is a silt loam containing a small percentage of both clay and mica. Soils of this type are not as spongy as those of Groups 1, 3, and 6, nor quite as firm as the soils of Group 5.

SUBGRADE VARIABLES FOUND TO BE LARGELY ELIMINATED BY STAGE CONSTRUCTION

The extent to which water capacity, plasticity, and shrinkage of soils exert an influence on subgrade behavoir depends upon field conditions. Spongy support, in contrast, is dependent upon the presence of certain raw materials, irrespective of field conditions.

Notwithstanding these wide differences in subgrade soil properties, Tables 7 and 9 show that but little relationship exists between slab thickness and soil characteristics. The slab thickness over the soils of Group 4, which was the predominating soil group, varied from 7 to 12½ inches. The slab thickness over Group 6, the soils having the highest shrinkage properties, was less than 10 inches, while that over Group 1, the soils having the lowest shrinkage properties, was above 12 inches.

It is more difficult to work the spongy properties out of soils containing little or no clay (Groups 1 and 3) than out of one which contains an appreciable amount of clay (Group 6), and this fact seems to be reflected by the road thickness of 12 inches on Groups 1 and 3 soils shown in Table 9. Except for this limited indication, unfavorable subgrade conditions seem to have been generally eliminated by the incorporation of coarse granular material during the early progressive construction of the pavement under traffic.

This is not true of the narrow strip along the north edge which was not progressively constructed as was the central portion. Evidently the subgrade has not been so well stabilized since the greater part of the patrol maintenance required on the road is confined to this area.

One of the Worst Conditions of Surface on Bradley Lane at the Present Time

In comparing the unsatisfactory behavior of the surface treatments prior to 1915 with the entirely successful ones subsequent to that year, it is evident that the factors which at that time were considered as the principal causes of failure were only contributing factors and that the method of treatment was primarily responsible for their failure.

From past experiences it is a well-established fact that a prerequisite to a successful surface treatment of the mat type is a stable, well-bonded surface to which the applied treatment will readily adhere. To insure adherence of the mat an intermediary application of bituminous material, which will penetrate appreciably, should ordinarily precede the application of the heavier mat-forming material.[6] In the case of the treatments

[6] There are many examples where surface treatments have been successfully applied without a primer coat. These treatments have been on clean, well-bonded surfaces containing relatively large aggregate.

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QUALITIES REQUIRED IN PAVING CONCRETE[1]

By F. H. JACKSON, Senior Engineer of Tests, United States Bureau of Public Roads

A concrete pavement, from a structural standpoint, consists of a series of flat slabs several feet in width, only a few inches in thickness, and of indefinite length, resting upon a support of more or less uncertain character. It is subjected not only to traffic loads of greatly varying intensity but also to the stresses and weathering effects produced by wide ranges in temperature and moisture conditions. Few concrete structures are subjected to such a variety of destructive forces. In order, therefore, to afford the greatest possible resistance to all such forces the utmost efforts to obtain the best possible product are justified. That so many of the concrete pavements built years ago are still carrying traffic with a reasonable maintenance cost is a tribute to the inherent worth of concrete as a paving material. That so many built within recent years have failed to measure up to the high standard of service required of them should serve as a warning that the best engineering control of design and construction is necessary if satisfactory results are to be obtained.

DESTRUCTIVE AGENCIES CONTINUALLY AT WORK

The destructive forces that are continually at work upon concrete pavements may be divided into two general classes—those due to natural causes, such as variation in temperature, moisture, etc., and those due to traffic loads.

The natural forces produce direct tensile and compressive stresses and a number of complex stresses or "weathering effects," which sometimes cause partial or complete disintegration. It is to resist these latter disintegrating forces that we strive to produce what we term durable concrete. The direct stresses that result from natural causes are commonly produced by the resistance offered by the subgrade to the free expansion or contraction of the concrete slab. They may be induced either by changes in temperature or moisture content, or both. They are present practically from the moment the pavement is laid and they continue throughout its existence.

To combat these natural forces concrete should possess high resistance to both compression and tension. Resistance to compression has, at all times, been considered an important property and for many types of structures it has been deemed the most, if not the only, essential property. However, in the case of pavements, tensile strength is fully as important and perhaps of greater importance. For it is by tensile stress that transverse cracks are formed early in the life of the pavement; and in spite of many opinions to the contrary, a crack in a concrete pavement must be considered as a structural defect. This is particularly true of unreinforced pavements in which an open crack is, in effect, an unsupported edge and, as such, for a given thickness of concrete, becomes the weakest part of the structure.

It has been shown[2] that a direct relation exists between the tensile strength of the concrete and the amount of transverse cracking which will take place in a pavement slab. Other things being equal, the spacing of the transverse contraction cracks will be directly proportional to the tensile strength of the concrete at the time contraction begins.

Much could be written regarding the necessity for continued and adequate curing of concrete in order that its tensile strength may be built up before the pavement is allowed to dry out and tensile stresses are induced which will tend to crack it. The discussion here will be limited to pointing out that thorough curing, especially during the period immediately following the casting of the slab, should tend to reduce the number of contraction cracks, especially those surface cracks or checks induced by the drying out of the surface of the concrete at a greater rate than the mass.

Frequent repetition of stress, whether caused by natural forces or traffic loads, is known to produce fatigue effects; and it is also known that under sustained load or force, concrete is capable of a certain flow by which the stress is relieved. These phenomena are as yet not well understood and require much further investigation.

As the result of research work by the Portland Cement Association and other organizations, the water-cement ratio law governing the strength of concrete has been established. The writer believes that this principle, when correctly applied, affords the simplest and most practical method yet devised for designing concrete mixtures for a given strength. But, simply because the average of many thousands of tests shows that a certain crushing strength may be obtained with a given water-cement ratio, it may not be assumed that the water-cement-ratio theory is an adequate basis of design for paving concrete.

As previously stated, crushing strength is not the most important property of paving concrete. Tensile strength and flexural strength are more important; and we are not at all sure that the water-cement ratio is the most important factor in controlling these properties.

Experiments by the Bureau of Public Roads[3] and other organizations have shown that the strength of concrete in tension is affected to a much greater degree than the compressive strength, by the character of the aggregates employed. However, these experiments indicate that for a given combination of aggregates and cement, the water-cement ratio governs the tensile and flexural strength as closely as it governs the compressive strength. The best solution is believed to be the so-called trial method of proportioning which was described in a recent issue of PUBLIC ROADS.[4]

[1] Paper presented under the title Special Characteristics of Concrete for Pavements at annual meeting of American Concrete Institute, Chicago, Ill., Feb. 12 to 14.

[2] A. T. Goldbeck, PUBLIC ROADS, August, 1925, The Interrelation of Longitudinal Steel and Transverse Cracks in Concrete Roads.

[3] Comparative Tests of Crushed Stone and Gravel Concrete in New Jersey, F. H. Jackson, U. S. Bureau of Public Roads, PUBLIC ROADS, February, 1928.

[4] The Design of Pavement Concrete by the Water Cement Ratio Method, reported by F. H. Jackson, Bureau of Public Roads, PUBLIC ROADS, August, 1928.

SOUND AGGREGATE REQUIRED TO PREVENT WEATHERING

The slowly acting but continued destructive effects of weathering and frost action on concrete pavements, though admittedly of great importance, are still but slightly understood. We speak somewhat vaguely of "durability" as essential in all classes of concrete exposed to the weather or to corrosive action of any sort, but we are not nearly so certain as yet of the factors which affect durability as we are of those which affect strength. The question arises: Is strength a measure of durability, and if not what factors affect durability which do not affect strength? In an effort to throw light on this subject, alternate freezing and thawing tests of concrete are being conducted in a number of laboratories. These tests indicate that unsound aggregates invariably produce unsound concrete; but there are certain types of unsound aggregates which appear to produce concrete of satisfactory strength though still unsound.

In general, however, the factors which affect durability also appear to affect strength. This applies particularly to the amount of mixing water used; and it may be definitely stated that, given sound aggregates, the best insurance against frost action and weathering effects appears in general to be the use of as low a water-cement ratio as is consistent with the requirements of workability.

Such materials as shale and certain varieties of flint occurring either in the fine or coarse aggregate, must be avoided if sound concrete is to be obtained. It is surprising, however, what a wide variety of aggregate types may be used to produce durable, sound concrete, provided the latter is properly designed and fabricated. It seems safe to say that any aggregate which will pass the sodium sulphate soundness test [5] will contribute to satisfactory durability in concrete which is otherwise of good quality. Conversely, failure in this test should subject the aggregate to suspicion until a thorough field investigation at the source has convinced the engineer that it is a safe material to use. Control of aggregates, however, will go for naught unless proper care is exercised in fabrication, and it is the failure of the field man to recognize one or more of the fundamental principles of good concrete construction that accounts, in the author's opinion, for most of the failures which we are apt to attribute to lack of durability.

SCALING LARGELY DUE TO POROUS MORTAR ON SURFACE

Surface scaling is not well understood, although there is considerable evidence to indicate that it is almost entirely the result of frost action. That scaling is not observed in the South, but is confined entirely to the Northern States substantiates this view. It has been possible to simulate the surface scaling of concrete as observed in service by subjecting laboratory specimens to alternate freezing and thawing. Surface disintegration almost always begins on the top of the specimen, and is probably due to the inability of the relatively weak, porous mortar surface to resist disintegration to the same extent as the mass of the concrete.

If this view of the case is correct, it indicates that effort should be made to prevent the formation of such a porous mortar top on the surface of the pavement to minimize the danger of scaling. The use of a stiff concrete, just sufficiently plastic to settle into place without tamping; the use of a fine aggregate containing as little silt or other fine material as possible; and the removal of the thin surface with lutes just prior to final finishing; these measures should go far toward preventing this type of destructive action.

Surface scaling followed by disintegration may sometimes be due to the use of unsound aggregates, but it appears that failures from this cause are more or less local and do not compare in extent to the scaling which may be attributed to the presence of a porous mortar top.

The Effect of Shale in Gravel

An Example of Surface Scaling

EFFECT OF TRAFFIC LOADS CONSIDERED

The major stresses in concrete pavement are undoubtedly the bending stresses produced by traffic loads, applied either statically or with impact. In general, these stresses are a maximum along the edges and at the corners of the slabs. Much work has been done in establishing the most economic pavement cross section for uniform slab strength. It is not the purpose of this paper to discuss slab design, but it may be well to point out that the adequacy of the design depends entirely upon how closely the strength of the concrete as actually placed conforms to the strength assumed in the design. In all formulas for pavement slab design a unit flexural strength of about 300 pounds per square inch is ordinarily assumed, and the thickness of slab necessary to carry a given maximum wheel

[5] U. S. Dept. Agri. Bul. 1216, rev. p. 20.

load is calculated on this basis. For a factor of safety of two, this requires the design of a concrete mixture having a modulus of rupture at 28 days of 600 pounds per square inch, which is equivalent to a crushing strength of about 3,000 pounds per square inch.

It has been stated that the water-cement ratio principle may be used to design concrete of a given flexural strength, provided it is recognized at the outset that such factors as angularity and surface texture of aggregates affect the flexural strength to a greater extent than they affect the compressive strength. Tests are now being made by the Bureau of Public Roads, in which 17 typical varieties of coarse aggregates including trap, limestone, granite, calcareous and siliceous gravels and blast furnace slags are being investigated. They show that for a given mix, grading, and water-cement ratio, the maximum variation from the average flexural strength of concrete made with these aggregates is 25 per cent. An illustration from one of the four mixes employed in the investigation may be cited.

The flexural strength (modulus of rupture) of a field, volumetric 1 :2 :3 mix, using a fixed consistency and with the water-cement ratio falling within a total range of 0.05, varied from 530 pounds per square inch to 650 pounds per square inch at 28 days, due entirely to the kind of coarse aggregate employed. This difference in strength is equivalent to that which would result from a change in the water-cement ratio of 0.15, a variation which should certainly not be ignored in applying the water-cement ratio law. The change in flexural strength was not accompanied by any significant difference in crushing strength, indicating that even quite wide variation in the character of the coarse aggregate had no appreciable effect upon compressive strength for a given water-cement ratio. This point is emphasized to show that in designing pavement concrete mixtures, other factors besides water content must be considered.

SURFACE WEAR NOT A CRITICAL FACTOR

Many engineers believe that high resistance to wear is no longer important and point to pavements carrying heavy traffic which were constructed with relatively soft aggregates and which still show the original finishing marks. On the other hand, the use of steel, non-skid chains in winter does produce wear. This is about the only destructive agency of this type left ,since the steel wagon tire has practically disappeared from our highways, except in certain restricted regions. On the whole, the writer is inclined to believe that surface wear is not a critical factor and that, in general, a concrete mixture which has been properly designed as to strength and durability will also be satisfactory from the standpoint of surface wear.

QUALITY OF PAVEMENT CONCRETE AFFECTED BY CONSTRUCTION METHODS

Attention has been called to the fact that the quality of concrete depends fully as much upon the care used in construction as upon the materials employed or the proportions established in the design. Satisfactory construction can not be secured by the type of engineering control which very carefully specifies the quality and grading of aggregates to be employed, sets a mix which under laboratory conditions will give the desired strength, and then employs an incompetent inspector on the job. Such inspection is unfair to the contractor, as well as to the public, because it often leads to arbitrary and unreasonable interpretation of specifications and results in unnecessary delays and increased cost.

It is believed that there are cases where quality in construction is sacrificed in order to increase the production. Maximum efficiency is, of course, much to be desired in all construction operations, and studies by the bureau have shown that there are many instances where production can be increased without sacrifice of quality. It must be remembered, however, that mere speed should not be permitted to become the controlling factor beyond the point where it is possible to maintain the highest standards of workmanship. Furthermore, no established practice as regards construction should be modified for the purpose of increasing production without first determining very definitely that the proposed modification does not adversely affect the quality of the finished product.

We should also be alive to the possibilities of improving the quality of our concrete, especially if we can do so without increasing cost of production. With this thought in mind, the Bureau of Public Roads is now actively promoting certain principles in connection with production of concrete for pavements which will, we believe, result not only in more uniform quality but will also tend to reduce the ultimate cost to the public. These principles are as follows:

1. The abandonment of volumetric proportioning of aggregates and the adoption of proportioning by weight. Inundation will be recognized as a permissible alternate method for fine aggregate, but weighing will be preferred.
2. Maintenance of the lowest water-cement ratio which, with the type, grading, and proportions of aggregate and methods of finishing employed, will produce a workable, dense, and uniform concrete.
3. The scientific grading of coarse aggregate by combination of separated sizes in each batch in the proportions which will produce the maximum practical density.
4. The abandonment of hand-finishing methods.

The State specifications previously approved by the bureau for Federal-aid concrete pavements provide for certain standard proportions of cement and fine and coarse aggregate. As a result of recent tests the bureau is now convinced that better and more economical concrete may be produced in some instances by increasing the proportion of coarse aggregate previously specified, providing the density and uniformity of the mix are not impaired. It has, therefore, announced that where adequate engineering control is assured, the coarse aggregate proportions previously approved may be increased if, by combination of separated sizes in each batch, a well-graded aggregate is produced and the resulting concrete is dense and uniform, workable by the methods of finishing employed, and of a quality at least equal to that produced by the approved standard mix.

The bureau feels that improvement in uniformity as well as increased economy will be accomplished by measuring coarse aggregates in two or more separate sizes. This practice will insure the maintenance of a uniformly low void content in the coarse aggregate and make it possible to reduce the amount of mortar below that necessary under the present practice where quite wide variations in voids occur frequently from batch to batch, due to inefficient mixing of sizes at the producing plant or stock pile segregation. The practicability of this procedure has been demonstrated in actual

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CONCRETE IN TENSION

By A. N. JOHNSON, Dean of Engineering, University of Maryland

Articles in preceding issues of Public Roads[1] have reported various phases of the general investigation of the strength characteristics of concrete conducted by the engineering experiment station of the University of Maryland, with the cooperation of the United States Bureau of Public Roads and the Maryland State Roads Commission. This article presents the results of a study of the behavior of concrete when subjected to direct tension. Data are given to show the relation between the tensile and compressive strength of mortar and concrete at various ages, as well as the comparative elastic behavior of the materials in both tension and compression.

SPECIAL APPARATUS DEVISED FOR TENSION TESTS

A special form of test piece was devised for the purpose of studying concrete in direct tension. This form was used to cast specimens with a central section 4½ inches in diameter and 9 inches long. At each end of this secton there was an inverted frustum of a cone, making a total length of specimen of nearly 21 inches. This permitted tension measurements on a cross section of the same size as that used in compres sion tests.

The molds were made of brass in two parts and bolted together. They were first oiled and then the concrete was placed and tamped, the mold at the same time being struck with a wooden mallet to insure freedom from small air pockets in the concrete. The shape and dimensions of the specimen are shown in Figure 1.

Two brass pulling heads or grips were provided for testing the specimens in tension. One was placed over each end of the specimen, leaving an annular space of about one-quarter inch between the inside of the pulling head and the outside of the specimen. This space was filled with molten rosin.

Rosin was selected after considerable experimentation with various materials such as pitch, asphalt, plaster, etc. When the rosin reached the proper temperature it became very fluid and poured readily and it was very easy to clean the grips, but little rosin being wasted.

The brass pulling heads were centered by first placing a split aluminum cylinder around the shank of the specimen and holding it temporarily in place with an ordinary wooden clamp (this clamp is not shown in the diagram). This aluminum cylinder has a machined surface at each end where the brass ends or grips bear against it (fig. 1) and it is possible to align the two on the same axis. The grips were attached to links, arranged to form a universal joint, so that the line of force would coincide with the longitudinal geometric axis of the specimen. This was readily attained with considerable accuracy.

Rupture occurred usually in the 4½-inch cylindrical section, although a number of breaks occurred at the transition to the conical portion. Most of these were on the mortar specimens, the concrete specimens usually breaking within the central cylindrical portion.

CRUSHING AND TENSILE STRENGTHS COMPARED

The first series of tests was to obtain a comparison of the strength of mortar and concrete in tension and compression. The results are given in Tables 1 and 2.[2] The results are also shown graphically in Figure 2. This series of tests was made in 1925. The mixture for the mortar was 1 : 2 and for the concrete specimens was 1 : 2 : 3, using a crushed limestone with maximum size of three-fourths inch. All proportions were by volume.

The tables and diagrams indicate that the ratio of tensile to compressive strength is about the same for mortar specimens as for concrete specimens.

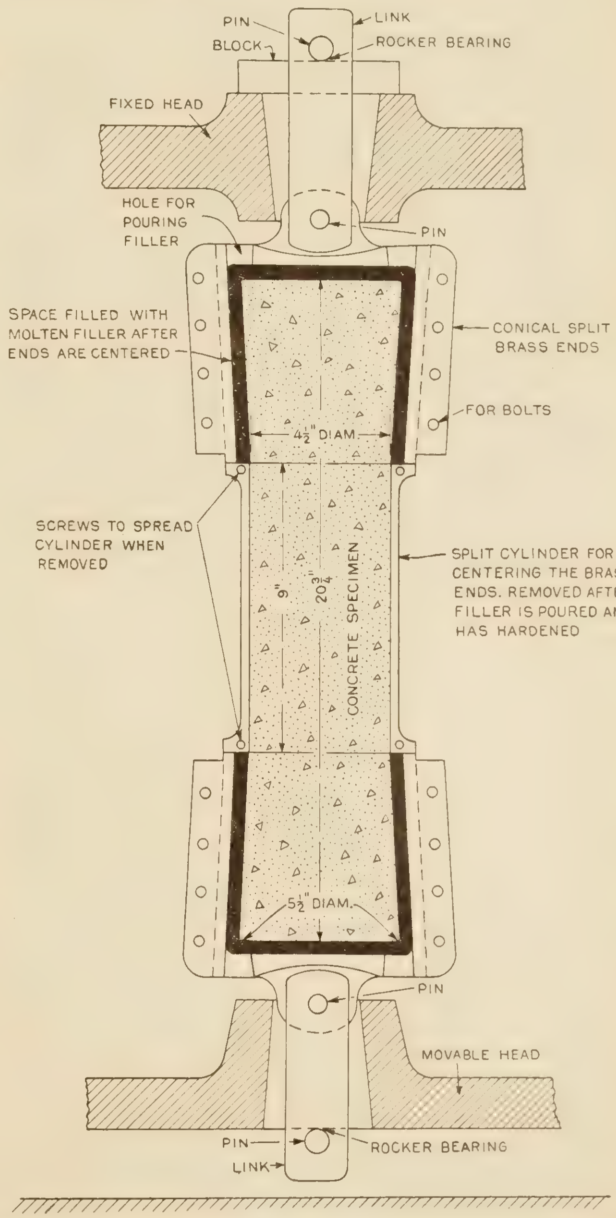


Figure 1.—Arrangement of Concrete Specimen for Tension tests

[1] Public Roads, vol. 9, Nos. 7, 8, and 9, 1928.

[2] These tables and full details of this series of tests were reported by the writer in the Proceedings of the A. S. T. M., vol. 26, Part II (1926), pp. 441-450.

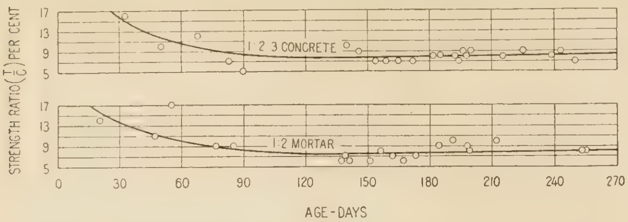


FIGURE 2.—RELATION BETWEEN RATIO OF TENSILE STRENGTH TO COMPRESSIVE STRENGTH AND AGE OF CONCRETE

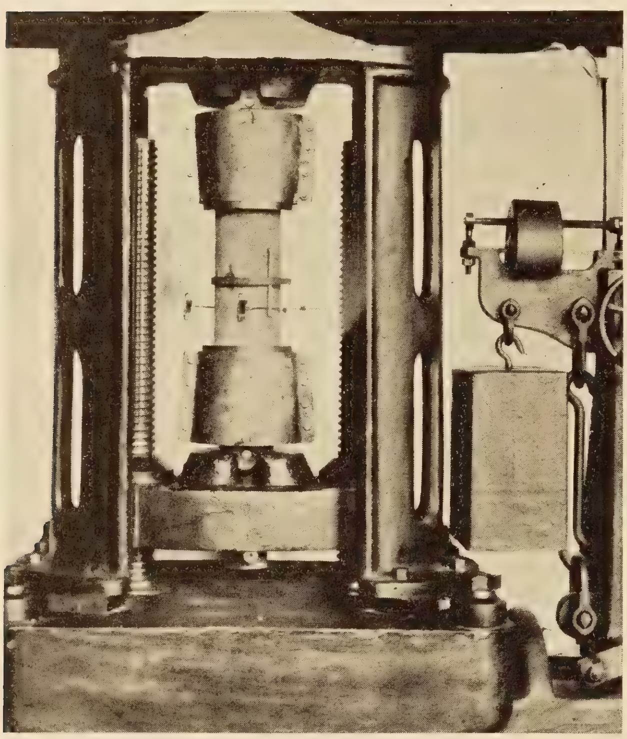


FIGURE 3.—A TENSION SPECIMEN IN THE TESTING MACHINE

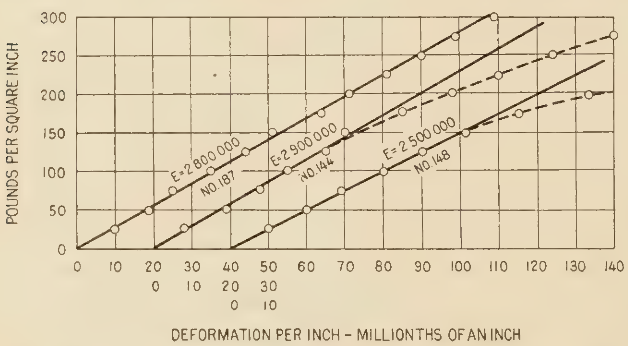


FIGURE 4.—TYPICAL STRESS-STRAIN CURVES FOR PORTLAND CEMENT MORTAR IN TENSION

This ratio, which is about 16 to 17 per cent at periods of about 30 days, gradually decreases to about 8 per cent at 90 days, and remains practically constant at this value thereafter. The tensile strength does not increase with age in the same proportion as does the compressive strength, although beyond a period of ninety-odd days the relative increase in tension and compressive strength is such as to keep this ratio almost constant at about one-twelfth.

TABLE 1.—*Results of compression tests and tensile tests on cement mortar (mix 1 : 2)*

[Cross section area 16 square inches (approximately). Each result average of 3 specimens]

Age	Compressive strength	Tensile strength	Tensile strength expressed as a percentage of compressive strength	Age	Compressive strength	Tensile strength	Tensile strength expressed as a percentage of compressive strength
	Lbs. per sq. in.	*Lbs. per sq. in.*			*Lbs. per sq. in.*	*Lbs. per sq. in.*	
20 days	2,510	349	14	162 days	6,123	399	7
47 days	3,827	421	11	167 days	6,750	431	6
55 days	2,640	442	17	173 days	7,133	466	7
76 days	4,990	434	9	184 days	5,790	525	9
85 days	4,200	370	9	191 days	6,340	630	10
137 days	6,230	347	6	198 days	5,420	494	9
139 days	7,140	489	7	199 days	5,873	496	8
141 days	7,183	434	6	212 days	5,613	541	10
151 days	7,577	488	6	253 days	6,663	547	8
156 days	7,223	583	8	255 days	6,183	505	8

TABLE 2.—*Results of compression tests and tensile tests of cement concrete (mix 1 : 2 : 3)*

[Cross section area 16 square inches (approximately). Each result average of 3 specimens]

Age	Compressive strength	Tensile strength	Tensile strength expressed as a percentage of compressive strength	Age	Compressive strength	Tensile strength	Tensile strength expressed as a percentage of compressive strength
	Lbs. per sq. in.	*Lbs. per sq. in.*			*Lbs. per sq. in.*	*Lbs. per sq. in.*	
33 days	2,160	349	16	185 days	4,280	326	8
50 days	3,197	335	10	192 days	3,773	301	8
69 days	3,043	360	12	194 days	4,667	307	7
83 days	3,007	220	7	196 days	2,130	192	9
90 days	2,503	126	5	197 days	2,863	235	8
140 days	3,353	334	10	200 days	2,503	236	9
146 days	4,160	369	9	215 days	4,260	330	8
154 days	4,175	293	7	225 days	3,337	298	9
159 days	3,310	244	7	239 days	3,647	284	8
165 days	3,473	257	7	243 days	2,797	247	9
172 days	3,023	221	7	249 days	3,373	248	7
181 days	1,847	155	8				

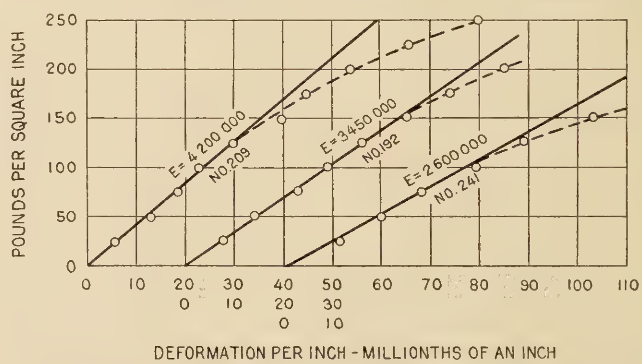


FIGURE 5.—TYPICAL STRESS-STRAIN CURVES FOR PORTLAND CEMENT CONCRETE IN TENSION

ELASTIC BEHAVIOR OF CONCRETE IN TENSION STUDIED

Another series of compression and tension tests, with both mortar and concrete specimens, was begun in 1926 and finished in 1928. This series was planned primarily to determine the elastic behavior of mortar and concrete in both tension and compression under comparable conditions. Compression and tension specimens were made from the same batches of material and cured under like conditions. Table 3, which shows the test results, is arranged to show which specimens were made from the same batch.

TABLE 3.—*Comparison of strength of 1:2 mortar in tension and compression*

Tension				Compression			
Specification No.	Age	Strength	Modulus of elasticity	Specification No.	Age	Strength	Modulus of elasticity
	Days	*Lbs. per sq. in.*	*Lbs. per sq. in.*		*Days*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
144	7	275	2, 900, 000	318	8	2, 370	2, 760, 000
145	7	275	2, 740, 000	319	8	2, 780	
146	7	225	2, 550, 000	320	8	2, 610	2, 860, 000
147	7	225	2, 660, 000	321	7	2, 180	2, 730, 000
148	7	200	2, 500, 000	322	7	2, 560	
149	7	225	2, 700, 000	323	7	2, 420	2, 760, 000
150	10	250	3, 350, 000	324	10	3, 140	
151	10	300	3, 350, 000	325	10	3, 000	3, 280, 000
152	10	325	3, 150, 000	326	10	3, 230	3, 280, 000
153	14	250		327	14	3, 175	3, 520, 000
154	14	245	3, 200, 000	328	14	3, 540	3, 520, 000
155	14	180	3, 100, 000	329	14	3, 020	
159	21	275	3, 120, 000	333	21	3, 300	3, 180, 000
160	21	285	3, 170, 000	334	21	3, 230	
161	21	310		335	21	3, 280	3, 320, 000
162	21	213		336	21	2, 500	3, 240, 000
163	21	140	2, 600, 000	337	21	2, 600	3, 140, 000
164	21	200	2, 650, 000	338	21	2, 260	
186	30	325	3, 050, 000	360	30	2, 760	3, 320, 000
187	30	310	2, 800, 000	361	30	3, 000	
188	30	283		362	30	2, 830	3, 400, 000
318	558	395		492	562	3, 940	
319	558	355		493	562	3, 970	
320	558	336		494	562	4, 150	
243	595	343		486	582	3, 470	
244	595	395		487	582	3, 860	
245	595	418		488	582	3, 680	
228	599	447		417	609	5, 870	
229	599	498		418	609	5, 610	
230	599	483		419	609	5, 450	
312	602	450	2, 750, 000	402	605	5, 850	
313	602	450	3, 200, 000	403	605	5, 980	
314	589	450		404	605	3, 850	

Elastic measurements in tension were made in the same manner as those previously reported for compression tests. The mirror extensometer was attached to the cylindrical or central portion of the tension specimen and deformations measured over a 4-inch gauge length, as in the case of the compression specimens. Figure 3 shows a specimen in the testing machine, with mirrors in position for testing. It is evident that when measuring tension deformations the mirrors revolve in opposite directions to those which take place when measuring compression deformations. The only change to be made is in the position of the scales—that is, the one which is read upward for compression is read downward for tension.

Considerable care was exercised in operating the testing machine, which was of 100,000 pounds capacity. It is probable that accurate results could be obtained more readily on a machine of smaller capacity, provided there was sufficient head room to carry a specimen.

The results of the elastic measurements on cement mortars and concrete in both tension and compression are presented in Tables 3 to 5. Table 3 gives a comparison between mortar in tension and compression and Table 4 gives similar data for concrete. Table 5 summarizes the data given in Tables 3 and 4. These data show that the modulus of elasticity in tension is approximately 90 per cent of the modulus of elasticity

TABLE 4.—*Comparison of strength of 1:2:3 concrete in tension and compression*

Tension				Compression			
Specification No.	Age	Strength	Modulus of elasticity	Specification No.	Age	Strength	Modulus of elasticity
	Days	*Lbs. per sq. in.*	*Lbs. per sq. in.*		*Days*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
255	7	150	3, 750, 000	429	7	1, 400	3, 720, 000
256	7	170	2, 950, 000	430	7	1, 445	3, 680, 000
257	7	160	3, 050, 000	431	7	1, 450	
192	8	200	3, 450, 000	366	8	1, 520	
193	8	220		367	8	1, 435	3, 400, 000
194	8	215	3, 200, 00	368	8	1, 470	3, 740, 000
231	14	175	3, 300, 000	405	14	1, 680	3, 800, 000
232	14	230	3, 150, 000	406	14	1, 805	3, 640, 000
233	14	193		407	14	1, 945	
240	14	175	3, 600, 000	414	14	1, 445	3, 920, 000
241	14	165	2, 600, 000	415	14	1, 420	3, 560, 000
242	14	185	3, 050, 000	416	14	1, 600	3, 920, 000
246	21	185	3, 350, 000	420	21	1, 600	3, 900, 000
247	21	175	2, 900, 000	421	21	1, 900	4, 400, 000
248	21	225	3, 750, 000	422	21	1, 830	4, 400, 000
258	29	225	3, 650, 000	432	29	1, 705	3, 680, 000
259	29	250	3, 800, 000	433	29	1, 900	3, 900, 000
260	29	225	4, 050, 000	434	29	1, 600	3, 560, 000
207	30	250	3, 600, 000	381	30	1, 825	4, 180, 000
208	30	183		382	30	1, 830	
209	30	265	4, 200, 000	383	30	1, 965	4, 280, 000
327	562	374		501	563	3, 450	
328	562	334		502	563	3, 530	
329	562	352		503	563	3, 580	
309	607	244		483	607	3, 105	
310	607	351		484	607	2, 990	
311	635	325	4, 000, 000	485	607	2, 810	
306	608	354		480	608	3, 690	
307	636	300	3, 750, 000	481	608	3, 650	
308	608	273		482	608	3, 235	
303	609	348		477	609	3, 610	
304	609	367		478	609	3, 590	
305	609	329		479	609	3, 550	
222	618	423		396	618	3, 580	
223	646	350	4, 000, 000	397	618	3, 750	
224	618	369		398	618	4, 220	

TABLE 5.—*Summary of tension and compression tests on cement mortars and concretes*

Mixtures	1:2 mortar		1:2:3 concrete	
Stress	Tension	Compression	Tension	Compression
Number of specimens	17	14	18	17
Average age, days	13	16	18	19
Ultimate strength, pounds per square inch	251	2, 842	282	1, 648
Ratio: Ultimate strength in tension / Ultimate strength in compression	9 per cent		17 per cent	
Modulus of elasticity, pounds per square inch	2, 920, 000	3, 200, 000	3, 410, 000	3, 840, 000
Ratio: Elasticity in tension / Elasticity in compression	91 per cent		89 per cent	

in compression for both mortar and concrete of the character tested.

The average value for modulus of elasticity of concrete in compression was found to be 3,840,000 pounds per square inch and for tension 3,410,000 pounds per square inch. Values for the mortar are somewhat less, for compression the modulus being 3,200,000 pounds per square inch, and for tension 2,920,000 pounds per square inch.

TABLE 6.—*Deformations of 1:2 mortar cylinders under tension in millionths inch per inch*

Specimen No.	144	145	146	147	148	149	150	151	152	154	155	159	160	163	164	186	187
Age, days	7	7	7	7	7	7	10	10	10	14	14	21	21	21	21	30	30
Load—pounds per square inch:																	
25	8	11	13	10	10	10	7	7	13	10	8	10	6	10	10	9	10
50	19	19	20	19	20	21	13	15	20	20	16	17	16	20	19	18	19
75	28	27	30	28	29	29	23	20	37	28	25	26	23	29	29	28	25
100	35	36	40	37	40	38	30	30	46	35	33	34	29	40	40	34	35
125	45	46	50	46	50	46	37	37	54	44	43	43	37	50	49	43	44
150	50	56	61	57	61	56	46	45	63	51	53	49	46	------	61	50	51
175	65	64	73	68	75	66	53	53	69	60	64	57	55	------	74	59	64
200	78	76	84	80	93	79	60	60	80	66	------	67	60	------	91	66	71
225	90	86	------	87	------	------	67	68	86	76	------	76	70	------	------	75	81
250	104	97	------	------	------	------	77	75	98	------	------	84	78	------	------	83	90
275	120	------	------	------	------	------	------	84	102	------	------	91	------	------	------	94	99
300	------	------	------	------	------	------	------	93	112	------	------	------	------	------	------	101	109
325	------	------	------	------	------	------	------	------	124	------	------	------	------	------	------	------	------
Ultimate strength—pounds per square inch	275	275	225	225	200	225	250	300	325	245	175	275	285	140	200	325	310

TABLE 7.—*Deformations of 1:2 mortar cylinders under compression in millionths inch per inch*

Specimen No.	318	320	321	323	325	326	327	328	333	335	336	337	360	362
Age, days	8	8	7	7	10	10	14	14	21	21	21	21	30	30
Load—pounds per square inch:														
100	36	39	36	36	33	31	35	30	34	33	34	33	29	29
200	70	70	77	71	64	60	60	56	66	61	60	63	60	58
300	107	106	106	93	93	89	86	84	96	91	91	95	91	88
400	145	141	143	143	124	133	116	111	129	121	123	125	121	118
500	184	177	179	180	158	150	143	141	157	153	154	158	151	146
600	230	210	220	218	188	185	174	170	190	184	188	191	173	179
700	269	256	263	262	229	218	205	200	224	216	221	225	216	213
800	316	297	309	306	263	255	235	229	254	249	258	259	249	244
900	370	343	350	356	299	284	270	238	291	283	294	298	281	278
1,000	421	387	396	390	341	323	313	293	325	319	340	335	318	310
1,200	546	485	500	506	424	404	381	359	396	391	413	430	393	381
1,400	------	------	635	630	502	484	446	425	469	465	503	504	471	456
1,600	------	------	------	------	608	575	526	499	554	534	600	604	560	534
1,800	------	------	------	------	724	678	613	575	644	640	713	720	670	628
2,000	------	------	------	------	838	791	708	656	744	744	------	858	795	------
2,200	------	------	------	------	971	913	814	750	856	860	------	1,023	989	------
2,400	------	------	------	------	1,150	1,063	------	------	988	997	------	------	------	------
2,600	------	------	------	------	1,256	1,250	------	------	------	------	------	------	------	------
2,800	------	------	------	------	1,575	1,556	------	------	------	------	------	------	------	------
3,000	------	------	------	------	------	1,963	------	------	------	------	------	------	------	------
Ultimate strength—pounds per square inch	2,370	2,620	2,180	2,420	3,000	3,230	3,175	3,540	3,300	3,280	2,500	2,600	2,760	2,830

TABLE 8.—*Deformations of 1:2:3 concrete cylinders under tension in millionths inch per inch*

Specimen No.	192	194	207	209	231	232	240	241	242	246	247	248	255	256	257	258	259	260
Age, days	8	8	30	30	14	14	14	14	14	21	21	21	7	7	7	29	29	29
Load—pounds per square inch:																		
25	8	8	9	6	6	9	6	11	10	9	9	8	10	5	10	9	8	9
50	14	14	15	13	16	16	14	20	19	16	17	15	16	14	15	14	13	13
75	23	19	21	19	23	24	21	28	28	24	24	24	23	23	25	20	20	19
100	29	25	29	28	31	35	49	39	35	31	34	29	30	35	33	28	25	23
125	36	31	35	30	40	40	35	49	43	41	44	36	39	45	44	34	34	31
150	45	37	41	40	49	51	46	63	55	50	55	44	50	61	51	39	39	36
175	54	43	49	45	56	63	------	------	63	68	88	49	------	------	------	49	45	44
200	65	50	56	54	------	71	------	------	------	------	------	58	------	------	------	56	53	50
225	------	------	68	66	------	88	------	------	------	------	------	68	------	------	------	------	65	------
250	------	------	84	80	------	------	------	------	------	------	------	------	------	------	------	------	------	------
Ultimate strength—pounds per square inch	200	215	250	265	175	230	175	165	185	185	175	225	150	170	160	225	250	225

TABLE 9.—*Deformations of 1:2:3 concrete under compression in millionths inch per inch*

Specimen No.	367	368	381	383	405	406	414	415	416	420	421	422	429	430	432	433	434
Age, days	8	8	30	30	14	14	14	14	14	21	21	21	7	7	29	29	29
Load—pounds per square inch:																	
100	33	28	28	25	25	29	28	26	31	28	24	25	28	30	31	30	30
200	59	51	48	45	49	53	50	69	49	46	46	48	53	66	54	49	53
300	87	80	71	69	78	66	75	81	75	74	74	74	79	80	79	75	80
400	117	109	95	94	106	108	106	110	104	96	96	98	109	109	109	101	110
500	150	140	118	121	136	136	135	144	133	121	124	125	138	140	140	131	139
600	184	175	143	148	166	169	168	178	164	149	151	154	173	174	171	163	170
700	220	209	168	164	203	204	201	218	200	181	179	181	209	213	205	194	205
800	250	248	196	208	240	239	240	258	239	210	210	213	249	256	249	230	243
900	309	299	226	240	280	285	289	304	279	244	240	241	290	313	290	265	270
1,000	361	353	260	278	328	331	334	351	339	280	276	278	338	374	336	307	333
1,200	501	489	331	363	438	441	460	475	450	378	355	365	448	551	445	403	443
1,400	975	800	423	478	600	588	719	881	651	513	455	464	725	963	593	529	625
1,600			603	615						881	590	623			875	706	1,087
1,800			844	856							831						
Ultimate strength—pounds per square inch	1,435	1,470	1,825	1,965	1,680	1,805	1,945	1,420	1,600	1,600	1,900	1,830	1,400	1,445	1,705	1,900	1,600

The stress-strain curves for concrete in tension possesses all the characteristics of the stress-strain curves for compression. Figure 4 shows curves for mortar specimens in tension. The total deformation over which measurements may be made is usually about one ten-thousandth inch per inch for tension, and about ten times that value for compression.

Figure 5 shows typical stress-strain curves for concrete specimens in tension. The specimens selected are those for a low, medium, and high value for the modulus of elasticity. As with the stress-strain curves in compression, there is for the lower loads an initial straight line relation, from which the stress-strain tension curve departs gradually.

Tables 6 to 9 show in detail the deformations for practically all the specimens in Tables 3 and 4. Specimens from the same batch of concrete can be identified by reference to the last-mentioned tables.

POISSON'S RATIO DETERMINED

A few measurements of Poisson's ratio were made in accordance with a method previously described by the writer.[3] The tests demonstrated that, for the lower loads, the lateral deformation of the specimens as well as vertical deformations are proportional to the applied loads. This straight line relation being a fact, then Poisson's ratio for a given specimen is a constant for the lower loadings up to the limit of proportionality of elastic limit.

The measurements of Poisson's ratio were made upon 1:2 mortar specimens and also upon 1:2:3 concrete cylinders. At the time of testing, the specimens were from 2 to 12 months in age. Poisson's ratio for 15 determinations varied from 0.108 to 0.180 with an average value of 0.147.

[3] Proceedings A. S. T. M., vol. 24, 1924. Part II, p. 1024.

(Continued from p. 236)

construction in North Carolina, where a mix containing considerably more coarse aggregate than we have been in the habit of permitting has been used by combining three separate sizes of coarse aggregate.

Another outstanding advantage of handling and proportioning coarse aggregate in this manner is that it makes possible much closer control of water at the mixer, through the elimination of a variable, which has caused more trouble than is commonly supposed. This is the fluctuation in the water requirements of individual batches due to changes in grading. Batch-to-batch variations in the quantity of the finer sizes of the coarse aggregate—that is, the material ranging from about three-fourths inch down—are quite common under the present practice and cause marked variation in the workability of the concrete. This leads to a tendency on the part of the mixer operator to control the workability by changing the amount of water. It will be admitted that uniform water content is essential to uniform concrete. Measurement of coarse aggregate in separate sizes will contribute much to this end.

With regard to hand finishing, the bureau feels that smoother riding surfaces can be produced with mechanical equipment, and also that it is possible by mechanical means to handle economically and efficiently a drier concrete and one containing a higher percentage of coarse aggregate than when hand-finishing methods are employed. The manner in which a finishing machine will handle a concrete which by all laboratory standards would be labeled unworkable has considerably altered our conception of what we term (for want of a better name) "workability" in concrete.

The bureau believes that the loose methods of control which have been the rule in the past have often led to the use of proportions capable of producing a concrete of considerably higher strength than that called for in the design, in order that we might be certain of obtaining the design strength in the field. We have been employing a factor of safety in the shape of richer mixtures to take care of inadequate control methods. The greater certainty of the methods proposed should enable us to design and produce concrete conforming more closely to whatever design requirements may be imposed with resulting benefits both physical and economical.

(continued from p. 233)

prior to 1915 the surface was not in condition for treatment, viewed from present-day knowledge. It was not stable and well bonded nor did it receive the priming coat or binder necessary to bind the surface treatment to the road structure. Consequently the bituminous surface mat was readily displaced and, due to movement of the stone particles directly underlying it, rapidly deteriorated.

In the case of the treatments since 1915, in which the same type of bituminous material was used as formerly, excellent results were obtained due to the penetration construction. This construction had stabilized the top portion of the road and furnished a surface to which the later surface treatments readily bonded.

TABLE 9.—*Distribution of subgrade groups with respect to pavement slab thickness* [1]

Total pavement thickness	Number of places examined	Number of places which exist on subgrade with laboratory group number—					
		1	2	3	4	5	6
12 inches and above	12	1	0	2	9	0	0
10 to 12 inches	10	0	1	0	9	0	0
Below 10 inches	10	0	0	0	8	0	2

[1] Only the north side and center thicknesses are considered, since the south side was raised to the level of the concrete shoulder. See Table 8 for typical analyses of the subgrade groups.

The following conclusions are presented as a result of consideration of the experimental data:

Stage construction furnishes an efficient means for eliminating variation in subgrade support due to difference in soil character and drainage conditions.

Old traffic-bound roads, although unsatisfactory when subjected to traffic action directly, may serve efficiently as foundations for waterproof, wear-resisting surface courses.

Increasing the width of the pavement, previously of inadequate width, has reduced the maintenance cost per average daily vehicle. It is obvious that the reduction in unit vehicle costs has been due to a large increase in traffic without a corresponding increase in maintenance costs.

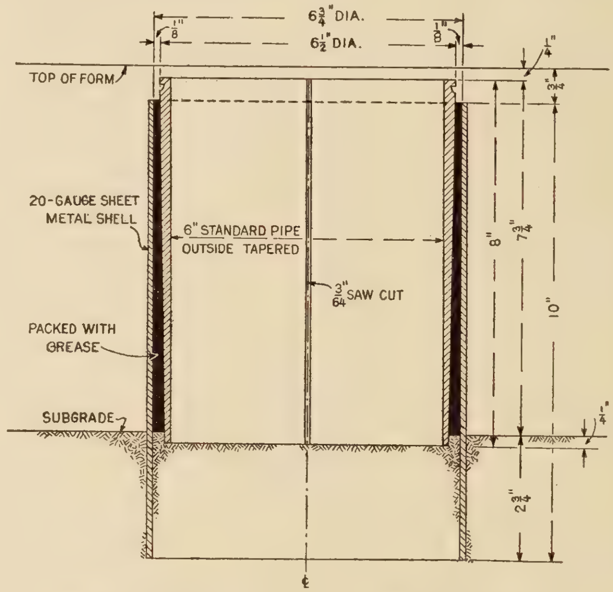
The success of a surface treatment depends to some extent upon the type of materials employed and the method of treatment, but to a greater extent upon the condition of the road to be treated.

A bituminous wearing course, which of itself does not furnish appreciable load support, will reduce the tendency of the macadam base to displace under the action of traffic. A comparatively thick bituminous wearing surface has the advantage in this respect over the thin mat treatment, although with the use of the priming coat, as at present, this difference is not so great as formerly.

FORM FOR CASTING CYLINDERS IN CONCRETE SLABS

The report on field experiments in curing concrete pavements in Maryland [1] described how concrete cylinders for testing purposes were cast and cured in a test slab adjacent to the road. The accompanying sketch shows the details of construction of the form used for this purpose.

The outer shell was driven about 2¾ inches into the subgrade so that it was firmly placed and the inner shell was adjusted so that its top was one-quarter inch below the top of the slab forms. A coating of heavy grease between the two molds facilitated removal of the iron cylinder and a layer of grease at the top of the outer mold sealed the space between the two, preventing any leakage of mortar which would have made removal difficult. Slots at the top of the iron mold made possible its removal by chiseling



DETAILS OF MOLDS USED IN CASTING CYLINDERS

through the ½-inch layer of concrete and then prying it out with two pinch bars bent at an angle of 45°. One person could remove these molds without difficulty.

[1] See Public Roads, vol. 9, No. 7, September, 1928.

HIGHWAY RESEARCH BOARD TO INVESTIGATE CURING

The highway research board of the National Research Council announces that arrangements had been made for conducting an investigation of methods of curing concrete pavement slabs. The work will consist first of making a correlation survey of all available existing data. These data will be analyzed and submitted to a special committee appointed by the board. The further program of the project will depend upon the findings of this committee.

The highway research board does not itself carry on research work, but from time to time special investigational projects are organized for correlating the work of other research agencies, for promoting additional research and for disseminating the results of research.

EARLY WRITER STRESSES IMPORTANCE OF WATER CONTENT OF CEMENT MORTAR[1]

[Extract from Niles Register, 1817]

Experiments showing the progress made in discovering the materials for a water cement, among our domestic resources, in a letter to Dr. Mitchill from David Meade Randolph, Esq., of Virginia, dated Richmond, June 26, 1817.

Dear Sir: Among the great variety of useful inventions connected with the arts, as it has been lately my object to pursue and to have observed in England, it was one of great importance in my mind, to discover in this country something that might answer an equally valuable purpose with the famous "Dutch Terias," or Parker's "Roman Cement," as it is now generally used in England and the West Indies for works under water and elsewhere situated.

In this pursuit it has been my good fortune to succeed so far as to indulge a confidence that upon the more satisfactory test of works upon a large scale, and a reasonable term of time for experience, there can be nothing discovered of more importance to the construction of durable foundations either under fresh waters or those of the sea than those certain minerals that I take the liberty of transmitting to you herewith. These will be found in two stone pots to your address. One of them contains a powder which I conceive to possess the properties of Puzzolana, or the cellular basalt of Doctor Rees; the other is a lime produced from certain concretions of lime, clay, and other matters found on the banks of York River, near to the town of Little York. These jagged and very various irregular sized (apparently) rocks seem to have been formed a little below the adjacent land, and to have tumbled from them as the washings of the tides have worn them down; for many fragments or distinct masses are seen pendant from their beds. The quantity is very extensive; and, from some parts of the same banks, the vertebra and other bones of some huge land or sea animal are found to have been dislodged likewise. This lime rock, upon being calcined, falls to an impalpable powder. It does not slack like other limes; on the contrary by the application of water as in slacking other limes, the powder forms itself into a mass, and coagulates by lying; and when made into a paste, forming a plate of it, suspecting it to dry, it assumes a stony or hard appearance, which being immersed in water before it is quite dry too, it does not dissolve like paste made of other lime.

One of those pots aforesaid (the other) contains a mixture of this lime powder and the powdered basalt, in the proportion of lime two, basalt three, which from my experiments seems to be the most perfect for terias mortar. They are to be reduced to a plastic state, by adding the smallest quantity of water possible, and that by little and little, to aid the beating in rendering it tough. Observe this rule: The more beating and the less water, the firmer the mortar. Hence you will perceive, sir, that my researches have been to the best chemical authorities, as far as my simple capacity has enabled me to understand from Doctor Rees and some others upon this subject. My acquirements and ability to investigate and to understand, are solely from exercising my practical knowledge and limited powers of mind; whilst I would most respectfully solicit your enlightened aid to mature my purpose, and to stamp a character upon my inventions.

Two bricks were cemented on the 1st of this month with a mortar far less perfect than the above and instantly (while the mortar was soft) they were placed in a basin of water, where they have remained ever since. The cement grows harder with time as is very perceptible; and from the crust that is evidently forming on the surface I am expecting a crop of stalactites.

You will readily perceive, sir, that if the invention shall prove effectual, I am fairly entitled to a reasonable compensation, to be secured by a patent or otherwise, and that your kind assistance in the promotion of my object would be gratifying in an eminent degree. In conclusion, sir, I pray you would have the goodness to favor me with a reply; for my apprehensions of having trespassed too far on your benignity can only be relieved by your favorable reception of this appeal to your liberality, and by such orders for a supply of the crude materials as you shall be pleased to give your most respectful and humble servant.

D. M. Randolph.

[1] This article has been submitted by Prof. R. S. Swinton of the University of Michigan. It was called to his attention by W. J. Worth, a student in the course on history of engineering.

REPORT ON TRACTIVE RESISTANCE BY IOWA ENGINEERING EXPERIMENT STATION

Conclusions as to the effect of tractive resistance are presented in Iowa Engineering Experiment Station Bulletin No. 88, Tractive Resistance of Automobiles and Coefficients of Friction of Pneumatic Tires, by T. R. Agg. The bulletin is the result of work done at the station in cooperation with the Bureau of Public Roads.

The more significant results of the investigation are summarized as follows:

1. There is no great difference in the tractive resistance of any particular vehicle on various road surfaces that are reasonably smooth and hard. In some instances the low or intermediate types give as low a tractive resistance as the high-type paved surfaces. A detailed inspection of the various tractive resistance diagrams will show that paved surfaces in good repair do not give as wide a range of variation in tractive resistance as do the intermediate and low types, nor does the tractive resistance on pavements reach as high maximum values as those determined for some of the low and intermediate types.

2. The high tractive resistance caused by mud is due in part to the necessity of squeezing the mud away from the tire as it rolls through the soft surface layer and in part to a certain springiness of the whole road crust. In deep mud the wheel tends to slide like a runner. This condition is not uncommon on earth roads and has come to be expected at certain seasons, but it adds greatly to the cost of transportation over the road.

3. The effect of a spongy subgrade under a thin road crust (a condition common on light gravel roads) is quite evident. The yielding of the foundation under the load, although the road crust does not break up, adds approximately 50 per cent to the tractive resistance at 20 miles per hour. This condition is perhaps

of no great consequence so far as light vehicles are concerned, but is a very important one from the standpoint of the truck and bus operator. On a road in this condition fuel consumption will be markedly greater than on the same road when the foundation is stable and unyielding, as it is when dry.

4. Rough surfaces of a given type generally have a higher tractive resistance than smooth surfaces of the same type. There is a tendency for the tractive resistance curves on certain varieties of roughness to drop at the higher speeds. In some instances they even cross those for smooth surfaces. In other instances the tractive resistance curves for rough surfaces are at all speeds higher than the corresponding curves for smooth surfaces.

The difference in behavior on different rough surfaces seems to be due to the effect of the resiliency of the tire and spring system. With a certain type of recurring roughness the wheel bounds in such a manner that the tire is partly off the road surface for appreciable time intervals and the distortion of the tire and the power loss therein reduced below that of a smooth surface. This apparent economy is doubtless much more than offset by the impact effects on the vehicle and the discomfort to the occupants. Roughness of the erratic type due to neglected maintenance, which leaves potholes, fissures, and irregular bumps in the road surfaces is an unmitigated nuisance.

5. The relation between the air pressure in the tires and the tractive resistance is well indicated by the data. As the pressure is lowered the tractive resistance increases, although some inconsistencies appear in the figures, due to unavoidable variations in road surface and wind during the time when the runs were made. The same type of observation could be made much more accurately in the laboratory. Nevertheless, these diagrams show that the motorist must pay in extra fuel consumption for neglecting to keep his tires at the proper pressure. Also, he loses in service life of the tire, if laboratory tests mean anything.

6. Laboratory tests indicate that rolling resistance decreases with an increase in the temperature of the tires.

7. In building so-called "traffic-bound" road surfaces, loose material is placed on the road to be packed by the traffic. A comparison of tractive resistances on both loose and compacted surfaces indicates that there is considerable cost in compacting a road surface by means of rubber-tired vehicles. Of course, this cost is concealed in the everyday operating costs to the owners of vehicles, but it is none the less real. It is unlikely that the public realizes the situation, and perhaps never will, but the practice is certainly open to serious question from the economic standpoint.

TYPOGRAPHICAL ERROR CORRECTED

In printing the article "Model Analysis of a Reinforced Concrete Arch" in the January issue of Public Roads a typographical error was made. On page 209 it was stated "It follows from the Maxwell theorem of reciprocal deflections that for a unit load, $I=\frac{d_2}{d_1}$." The formula should have been $T=\frac{d_2}{d_1}$.

U.S. GOVERNMENT PRINTING OFFICE: 1929

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

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

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.

DEPARTMENT BULLETINS

No. 105D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136D. Highway Bonds. 20c.
220D. Road Models.
257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370D. The Results of Physical Tests of Road-Building Rock. 15c.
386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387D. Public Road Mileage and Revenues in the Southern States, 1914.
388D. Public Road Mileage and Revenues in the New England States, 1914.
390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
463D. Earth, Sand-clay, and Gravel Roads.
*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537D. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660D. Highway Cost Keeping. 10c.
*670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
*691D. Typical Specifications for Bituminous Road Materials. 10c.
*724D. Drainage Methods and Foundations for County Roads. 20c.
*1077D. Portland Cement Concrete Roads. 15c.
1216D. 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.
1259D. 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.

DEPARTMENT BULLETINS—Continued

No. 1279D. Rural Highway Mileage, Income, and Expenditures, 1921 and 1922.
1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

No. 94C. T. N. T. as a Blasting Explosive.
331C. Standard Specifications for Corrugated Metal Pipe Culverts.

TECHNICAL BULLETIN

No. 55. Highway Bridge Surveys.

MISCELLANEOUS CIRCULARS

No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal Aid Highway Projects.
93M. Direct Production Costs of Broken Stone.
*109M. Federal Legislation and Regulations Relating to the Improvement of Federal-aid Roads and National-Forest Roads and Trails. 10c.

FARMERS' BULLETIN

No. *338F. Macadam Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. 914Y. Highways and Highway Transportation.
937Y. Miscellaneous Agricultural Statistics.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Connecticut.
Report of a Survey of Transportation on the State Highway System of Ohio.
Report of a Survey of Transportation on the State Highways of Vermont.
Report of a Survey of Transportation on the State Highways of New Hampshire.
Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio.
Report of a Survey of Transportation on the State Highways of Pennsylvania.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
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

CURRENT STATUS OF FEDERAL AID ROAD CONSTRUCTION

AS OF

JANUARY 31, 1929

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE			Estimated total cost	Federal aid allotted	MILEAGE				
				Initial	Stage[1]	Total			Initial	Stage[1]	Total		
Alabama	1,902.0	$ 3,688,826.44	$ 1,840,852.63	265.8	12.3	278.1	$ 492,879.53	$ 246,439.75	13.2	21.0	34.2	$ 2,519,324.79	Alabama
Arizona	874.5	1,614,742.54	1,269,926.61	70.8	12.3	83.1	8,807.81	6,631.39		.2	.2	3,710,470.71	Arizona
Arkansas	1,757.3	3,486,558.98	1,528,775.43	92.4	6.6	99.0						3,051,061.11	Arkansas
California	1,523.7	10,190,857.68	4,700,957.21	261.5	.6	262.1	1,025,787.98	573,804.18	44.5	5.7	50.2	2,954,588.64	California
Colorado	1,062.8	3,534,116.71	1,903,922.69	160.8	15.8	176.6	447,729.36	251,336.41	23.8		23.8	2,943,858.05	Colorado
Connecticut	220.1	1,893,291.22	419,866.53	21.7		21.7	667,713.84	279,338.73	3.6		3.6	837,708.90	Connecticut
Delaware	205.3	534,288.82	255,507.30	15.7		15.7	509,338.50	154,379.50	7.7		7.7	306,511.22	Delaware
Florida	429.6	2,783,853.47	1,232,677.16	110.7	5.4	116.1						2,078,896.61	Florida
Georgia	2,478.0	5,000,162.01	2,294,776.57	228.4	46.6	275.0	259,439.89	124,002.22	13.4	4.6	18.0	2,006,605.00	Georgia
Idaho	1,077.2	1,857,484.93	1,108,040.57	128.6	10.8	139.4	49,186.67	29,816.95	3.7		3.7	1,001,180.03	Idaho
Illinois	1,793.1	20,037,484.18	9,064,161.87	623.5		623.5	1,654,743.89	718,019.00	47.4		47.4	2,961,155.40	Illinois
Indiana	1,249.5	5,636,710.06	2,696,132.46	169.2	3.5	172.7	1,996,715.38	946,173.39	67.9		67.9	1,479,008.22	Indiana
Iowa	2,986.3	2,586,573.14	1,032,469.49	44.4	90.8	135.2	501,599.17	229,607.06	14.6	13.5	28.1	2,075,430.98	Iowa
Kansas	2,350.4	6,278,718.00	2,526,639.13	384.8	11.2	396.0	367,045.44	119,952.04	21.7		21.7	2,154,088.64	Kansas
Kentucky	1,279.2	4,546,996.33	2,051,831.59	211.5		211.5	1,231,047.65	546,223.80	80.9		80.9	1,103,569.62	Kentucky
Louisiana	1,281.3	4,681,926.40	2,332,944.70	193.1		193.1	103,055.30	25,000.00	.1		.1	1,209,402.36	Louisiana
Maine	480.5	1,779,653.03	596,458.57	40.9		40.9	192,121.39	93,601.55	8.9		8.9	1,439,377.47	Maine
Maryland	621.5	459,798.80	194,190.00	13.7		13.7						664,211.23	Maryland
Massachusetts	568.4	3,115,502.75	989,072.70	58.0		58.0	840,437.89	211,755.00	14.2		14.2	2,249,237.73	Massachusetts
Michigan	1,439.0	11,130,946.36	4,723,846.96	272.3		272.3	806,320.00	331,041.86	14.5	6.5	21.0	2,186,595.70	Michigan
Minnesota	4,089.7	1,249,220.75	350,618.27	96.1	11.2	107.3						2,147,297.96	Minnesota
Mississippi	1,590.9	5,359,683.26	2,462,797.14	234.3	41.5	275.8	275,179.91	137,590.43	24.9		24.9	1,422,774.43	Mississippi
Missouri	2,289.2	7,238,797.89	2,740,409.37	160.7	48.2	208.9	1,271,331.51	541,731.65	41.5	8.0	49.5	1,815,040.51	Missouri
Montana	1,534.1	2,947,610.99	1,829,328.05	225.1	13.4	238.5	1,414,142.75	922,368.47	80.0	.2	80.2	4,988,515.52	Montana
Nebraska	3,497.3	3,122,583.72	1,554,616.95	305.4	106.0	411.4	490,070.08	244,035.00	16.5	37.9	54.4	3,323,914.76	Nebraska
Nevada	1,024.1	1,201,547.37	1,049,898.13	138.3	71.0	209.3	107,712.85	95,702.86	14.5	7.5	22.0	1,032,584.65	Nevada
New Hampshire	331.7	279,253.01	108,613.31	7.5		7.5						403,144.24	New Hampshire
New Jersey	446.6	5,097,683.00	863,760.00	57.6		57.6	228,694.37	53,565.00	3.6		3.6	892,185.94	New Jersey
New Mexico	1,790.9	3,288,286.36	2,153,990.02	209.4	5.1	214.5	203,047.01	129,204.82	14.1		14.1	1,215,283.42	New Mexico
New York	2,114.4	23,298,900.00	5,143,000.55	342.7		342.7	2,303,191.76	504,555.00	33.7		33.7	6,846,530.12	New York
North Carolina	1,680.4	1,726,647.22	878,484.93	73.0	11.2	84.2	529,525.28	260,872.53	14.4	7.1	21.5	1,879,955.70	North Carolina
North Dakota	3,693.0	2,707,541.42	1,173,804.43	395.7	106.0	501.7	945,776.64	307,526.74	176.2	101.2	277.4	1,205,164.90	North Dakota
Ohio	1,965.9	10,825,810.39	3,829,003.15	237.1	.1	237.2	3,598,180.00	989,685.00	44.6	9.8	54.4	3,666,609.62	Ohio
Oklahoma	1,707.2	3,776,075.67	1,661,732.98	121.2	37.8	159.0	1,106,741.26	513,908.12	47.5	7.0	54.5	1,260,218.93	Oklahoma
Oregon	1,132.4	658,409.07	389,806.71	32.1		32.1						2,344,038.92	Oregon
Pennsylvania	2,032.1	11,542,846.32	3,120,917.63	188.6		188.6	1,755,364.18	567,923.88	35.2		35.2	3,633,166.51	Pennsylvania
Rhode Island	165.2	396,745.71	89,340.00	6.0		6.0	274,697.74	108,825.00	7.2		7.2	776,149.23	Rhode Island
South Carolina	1,716.7	6,807,124.53	1,489,452.17	160.5	88.1	248.6						1,077,107.90	South Carolina
South Dakota	3,264.4	2,647,945.91	1,441,883.36	422.2	39.4	461.6	198,198.47	109,009.13	39.0	3.0	42.0	1,168,118.57	South Dakota
Tennessee	1,075.2	4,342,813.37	1,920,198.10	104.4	60.6	165.0	1,130,696.23	457,036.26	4.1	38.1	42.2	1,912,629.21	Tennessee
Texas	6,065.4	12,074,592.00	5,106,575.21	445.1	195.5	640.6	5,689,626.21	2,335,394.40	210.8	104.6	315.4	4,505,811.58	Texas
Utah	902.5	1,545,997.03	1,033,731.53	71.5		71.5						846,865.01	Utah
Vermont	229.0	965,465.68	288,777.15	20.5		20.5						409,935.95	Vermont
Virginia	1,315.2	2,605,200.37	1,106,633.59	68.9	15.2	84.1	26,248.75	8,390.23	.2		.2	1,450,366.97	Virginia
Washington	833.3	3,872,438.32	1,302,775.25	75.8	18.1	93.9	215,457.07	116,100.00	14.5		14.5	1,421,537.83	Washington
West Virginia	688.6	1,146,917.91	538,131.65	39.9	12.4	52.3	341,958.11	145,997.43	11.2		11.2	1,058,298.21	West Virginia
Wisconsin	2,096.7	4,184,649.92	1,809,323.96	124.4	4.9	129.3	53,392.29	26,690.00	5.8		5.8	3,188,835.14	Wisconsin
Wyoming	1,679.1	929,507.06	596,220.16	100.9		100.9						1,009,600.91	Wyoming
Hawaii	39.4	175,931.99	57,501.20	1.8		1.8						1,432,123.59	Hawaii
TOTALS	76,570.4	220,904,713.09	88,854,375.12	7,834.5	1,101.6	8,936.1	33,313,202.16	13,463,234.78	1,219.6	375.9	1,595.5	97,266,088.64	TOTALS

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.