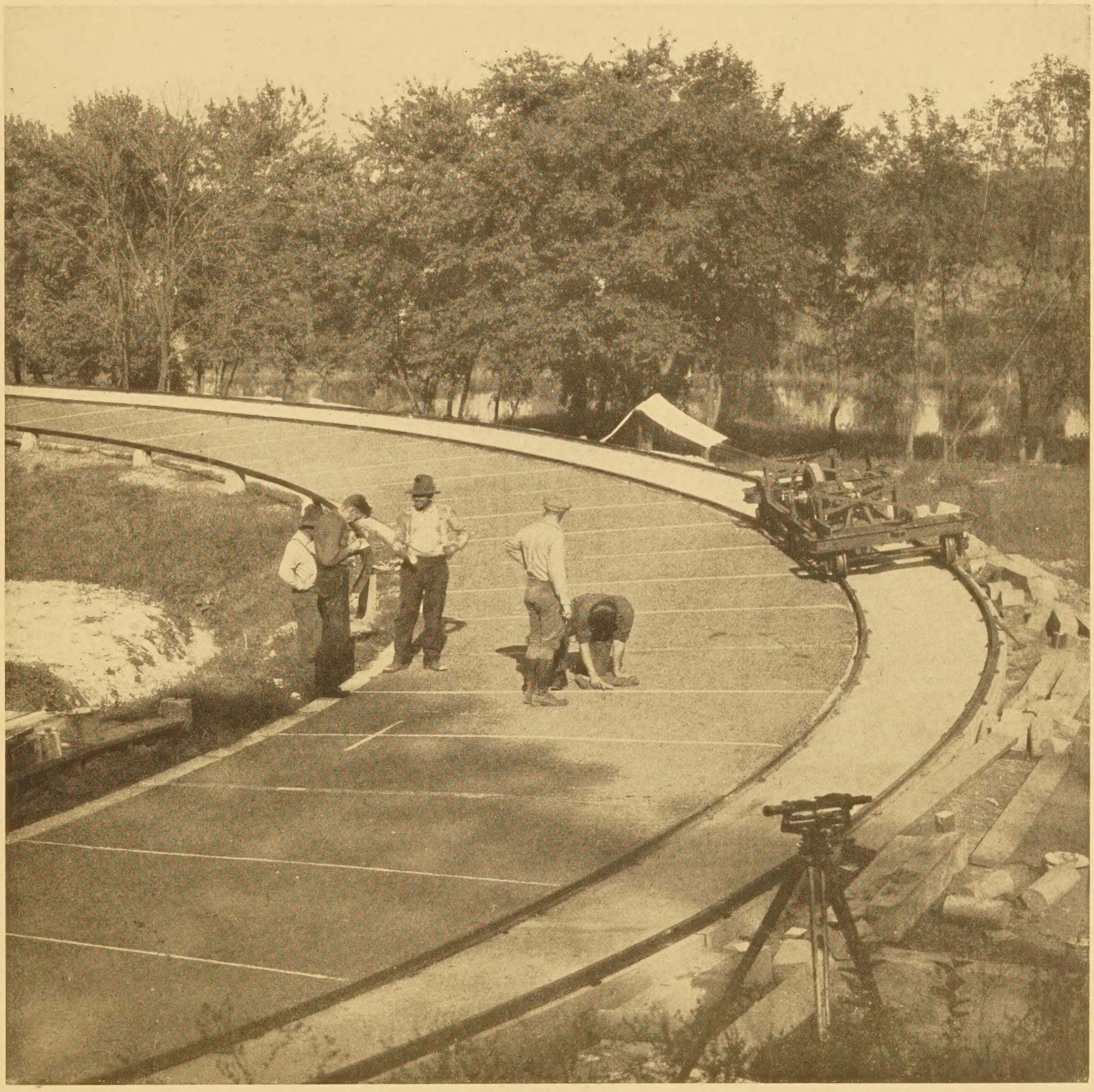
PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 5, NO. 3 ▽ MAY, 1924



General View of Experimental Pavements of the Bureau of Public Roads at Arlington, Va.

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1924

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH
U. S. DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

VOL. 5, NO. 3 MAY, 1924 H. S. FAIRBANK, Editor

TABLE OF CONTENTS

THE U. S. BUREAU OF PUBLIC ROADS

Willard Building, Washington, D. C.

REGIONAL HEADQUARTERS,
Bay Building, San Francisco, Calif.

DISTRICT OFFICES

DISTRICT No. 1, Oregon, Washington, Montana, and Alaska.
Box 3900, Portland, Oreg.

DISTRICT No. 2, California and Nevada.
Bay Building, San Francisco, Calif.

DISTRICT No. 3, Colorado and Wyoming.
301 Customhouse Building, Denver, Colo.

DISTRICT No. 4, Minnesota, North Dakota, South Dakota, and Wisconsin.
410 Hamm Building, St. Paul, Minn.

DISTRICT No. 5, Iowa, Kansas, Missouri, and Nebraska.
Webster-Sunderland Building, Omaha, Nebr.

DISTRICT No. 6, Arkansas, Louisiana, Oklahoma, and Texas.
1912 F. & M. Bank Building, Fort Worth, Tex.

DISTRICT No. 7, Illinois, Indiana, Kentucky, and Michigan.
South Chicago Station, Chicago, Ill.

DISTRICT No. 8, Alabama, Georgia, Florida, Mississippi, South Carolina, and Tennessee.
Box J, Montgomery, Ala.

DISTRICT No. 9, Connecticut, Maine, Massachusetts, New Hampshire, New Jersey, New York, Rhode Island, and Vermont.
Federal Building, Troy, N. Y.

DISTRICT No. 10, Delaware, Maryland, North Carolina, Ohio, Pennsylvania, Virginia, and West Virginia.
Willard Building, Washington, D. C.

DISTRICT No. 12, Idaho and Utah.
Fred J. Kiesel Building, Ogden, Utah.

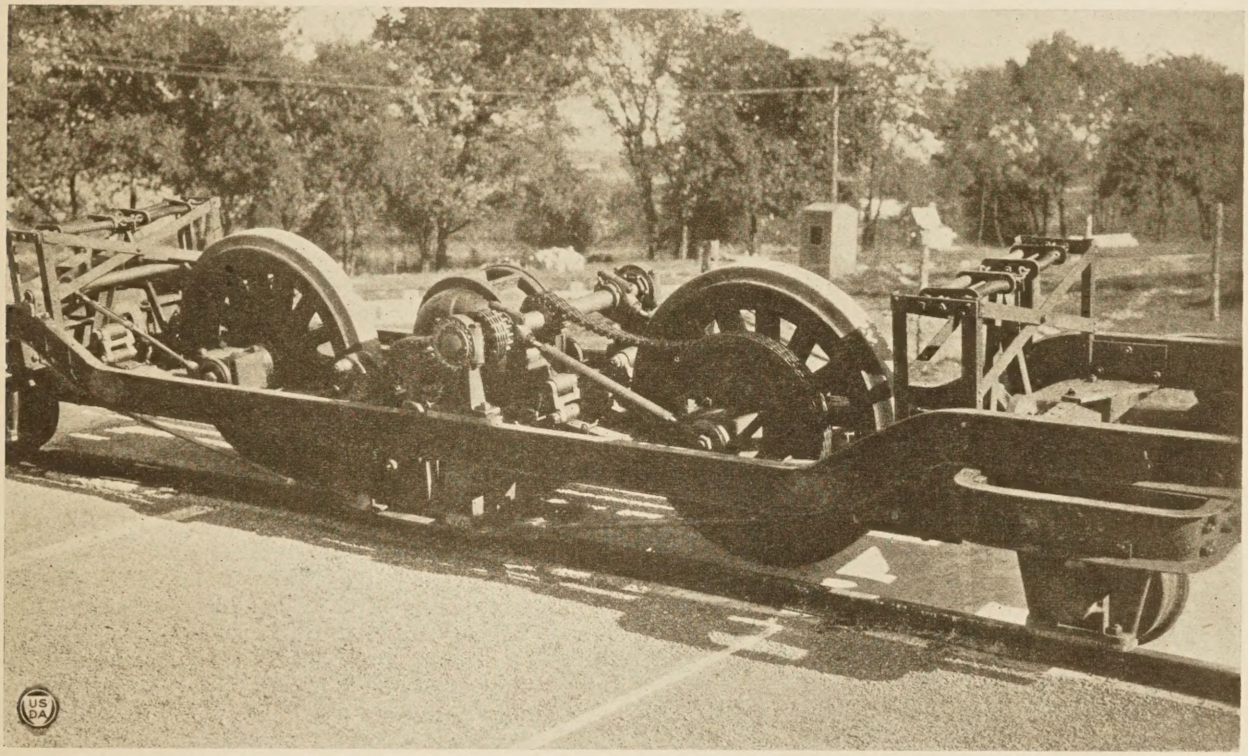
DISTRICT No. 13, Arizona and New Mexico.
242 West Washington St., Phoenix, Ariz.

Owing to the necessarily limited edition of this publication it will be impossible to distribute it free to any persons or institutions other than State and county officials actually engaged in planning or constructing public highways, instructors in highway engineering, periodicals upon an exchange basis, and Members of both Houses of Congress. Others desiring to obtain "Public Roads" can do so by sending 10 cents for a single number or $1 per year to the Superintendent of Documents, Government Printing Office, Washington, D. C.

WEAR OF CONCRETE PAVEMENTS TESTED

REPORT OF ACCELERATED TESTS BY U. S. BUREAU OF PUBLIC ROADS

By F. H. JACKSON, Senior Assistant Testing Engineer, and J. T. PAULS, Highway Engineer, U. S. Bureau of Public Roads



The loaded, rubber-tired wheels of the testing machine bear with their full weight on the concrete. The machine is guided by the rails at the side of the experimental sections

THE accelerated wear tests of concrete pavements conducted by the Bureau of Public Roads, at Arlington, Va., during the past 18 months, have reached the stage where it is possible to draw fairly definite conclusions from the data so far obtained. The 62 concrete test sections, each 4 feet wide by about 10 feet long, and constructed in the form of a circle approximately 625 feet in circumference, have been subjected to approximately 300,000 passages of a solid-rubber-tired truck wheel loaded to 3,000 pounds and traveling around the circle over the same path at a speed of 22 miles per hour. Careful observations of the lateral distribution of traffic across actual pavements indicate that about 10 per cent of the total passes over a 6-inch width at the point of greatest concentration. This is based on traffic passing in two directions over an 18-foot road. On this basis it is estimated that the experimental pavement sections have been subjected to a traffic equivalent to about 1,500,000 two-ton trucks operating at a speed of 22 miles per hour assuming the distribution indicated by the above observations. In addition, the test sections have been subjected to about 50,000 passages of a rubber-tired wheel loaded with the same weight and equipped with nonskid chains and the same number of passages of a plain rubber-tired wheel. The wheels in this part of the test traveled over a new path. It will be seen, therefore, that the equivalent of an enormous volume of traffic has been put upon the pavement—an amount probably equivalent to several years traffic on the average concrete road.

It was, of course, impossible to reproduce on the test sections in the short space of a year and a half the effect of freezing and thawing which, through a period of years, may weaken actual roads so that they become less resistant to traffic. It was likewise impossible to reproduce the effect of differential expansion and contraction of the thin, rich mortar top and the mass of the concrete, which, caused by wide variations in temperature, may also, in time, lower the resistance of a pavement. The essential time element is lacking. In these respects, therefore, the test did not duplicate actual conditions, and for this reason the differences in the behavior of the several sections under examination may not be so marked as they normally would have been.

THE PURPOSE AND RESULTS OF THE TEST

The primary object of the test was to determine what relations, if any, exist between the surface behavior of concrete roads under traffic and the

95633—24——1

various laboratory tests for quality of the aggregates. Incidentally it was also deemed desirable to determine the relation between the wear as produced by traffic, as well as by laboratory test, and the various physical properties of the concrete, such as crushing strength, transverse strength, modulus of elasticity and absorption. All of these tests were made on the concrete used in each of the 62 test sections.

Specifications for aggregates for concrete pavements are usually drawn to cover the gradation and the quality of the material furnished. The gradation, or proportion of the various sizes, especially as regards aggregates which are crushed and screened, depends largely on factors which can be controlled, so that the engineer may obtain almost any desired gradation of sizes provided he is willing to insist upon it. The quality of the material, however, can not be fixed in so arbitrary a fashion. Due to the high cost of transportation, the aggregates for any given improvement must generally be found within a relatively short distance of the site of the work. Therefore, as the quality and character of materials varies so greatly in different parts of the country, it follows that many different kinds and types of aggregates must be employed in the construction of concrete roads. All that can be done is to draw specifications in such a way as to insure the use of the best material available for the work. In so doing, however, there is always a minimum limit beyond which it is unsafe to go.

Placing the concrete

What are the safe test limits for the various kinds of aggregates used in concrete road construction? Do the present limits for wear, strength, etc., insure the selection of safe materials, or, on the other hand, do they unduly add to the cost of construction by setting laboratory requirements too high? These are some of the questions which the accelerated wear tests were designed to answer.

The detailed conclusions which it has been possible to draw from the test are set forth fully at the end of this article. Outstanding from these conclusions are two facts of major importance. These are:

1. That rubber-tired traffic alone does not appreciably abrade the surface of a concrete pavement.

2. That there is no consistent relation between the results of the "tensile-strength-ratio" test for sand, the compression or transverse strength tests or the Talbot-Jones wear test of concrete, and the wear-resisting properties of concrete pavements.

DESCRIPTION OF THE TEST SECTIONS

The test sections were constructed in the form of a circular concrete pavement approximately 625 feet in circumference and 4 feet wide. There were 62 sections, each about 10 feet long. A general view of the experiment, showing also the stability experiments on bituminous surfaces, is given in the cover illustration. Inasmuch as the only object of this test was to bring out differences in the amount and character of wear or surface disintegration due to traffic, the sections were constructed heavily enough to prevent the possibility of any structural failure. The test sections were laid in two courses, the lower 8 inches, laid directly on the subgrade, consisting of 1 : 1½ : 3 concrete, using Potomac River sand and gravel as aggregates. A high-grade Portland cement passing all of the American Society for Testing Materials requirements was used throughout the experiment. On one side of the circle it was found necessary to construct the test pavement on a fill. The base over this section was further strengthened by the use of a reinforced concrete T-beam section constructed with the beams running longitudinally. This design proved entirely adequate and there has been no evidence of failure due to settlement on any test section. The test concrete forming each section was cast directly upon the green concrete base and was 4 inches in depth. The total depth of pavement therefore was in every case at least 12 inches of monolithic concrete. No expansion joints were provided, the concrete in each day's run being finished to a vertical wooden bulkhead which was removed the next day and the fresh concrete deposited directly against that placed the day before. As a result of this procedure construction joint cracks have formed between several of the sections. Additional contraction cracks have also developed in a few places. None of these cracks, however, has affected the stability of the pavement in the slightest degree.

The test aggregates were proportioned by loose volume, each batch consisting of one bag of cement, 1½ cubic feet of sand, and 3 cubic feet of coarse aggregate, except on those sections where other proportions were tried for experimental purposes. The concrete was mixed in a one-bag, power-driven, batch mixer; and every effort was made to have the consistency as nearly uniform as possible except in the group where it was varied designedly. In view of the widely varying characteristics of the materials it was found impossible to control this factor absolutely. The consistency of the concrete going into each section was measured by means of the slump test and flow table. The average results of the consistency tests are given for each section in Table 1. The concrete in practically all cases was of medium consistency and rather inclined to be too dry than too wet. All the coarse aggregates, unless otherwise noted were graded from 1½ inches uniformly down to one-fourth inch in size. Potomac River concrete sand, used as the standard fine aggregate, was well graded and of known quality. The concrete in each section was struck off by means of a wooden strike board and was finished with a wooden hand float.

As the testing machine was expected to operate at a speed of approximately 20 miles per hour around a circle with a 100-foot radius, a superelevation of about 7 inches was found necessary. This was not as much as was theoretically desirable, but it was found impracticable to properly finish the concrete surface on any greater slope. Iron bolts were cast in the pavement at regular intervals for the purpose of holding the rails which were to guide the testing machine. The rails were laid 42 inches center to center. Various details of the construction of the test sections are shown in the accompanying illustrations.

TABLE 1.—*Consistency of concrete used in the test sections, showing relation between slump and flow*

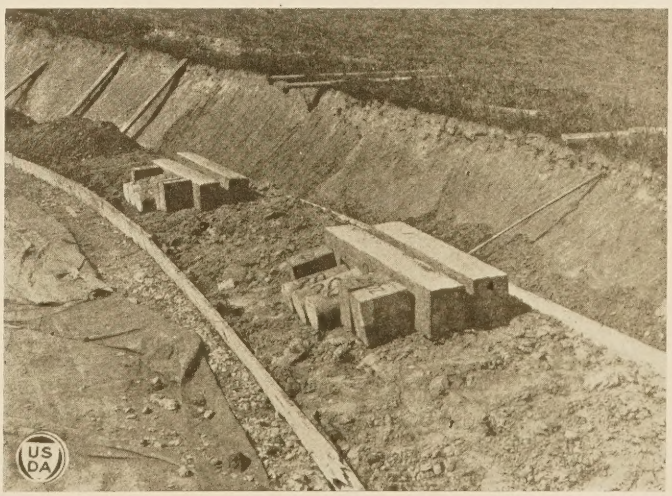
Section No.	Average slump	Average flow	Section No.	Average slump	Average flow
	Inches			*nches*	
1	3	165	32	1	135
2	3	165	33	4	180
3	2	155	34	1	140
4	3	160	35	2	175
5	3	165	36	4	170
6	2	140	37	3	150
7	2	160	38	2	150
8	1	135	39	2	150
9	2	150	40	3	160
10	1	140	41	2	145
11	3	170	42	2	150
12	3	170	43	4	150
13	2	145	44	1	130
14	1	140	45	4	175
15	2	155	46		
16	2	155	47	2	155
17	2	155	48	1	120
18	2	150	49	1	135
19	2	150	50	2	155
20	3	170	51	1	135
21	2	150	52	2	150
22	2	160	53	4	165
23	2	150	54	2	155
24	2	155	55		145
25	0	120	56	2	165
26	1	145	57	1	145
27	1	120	58	1	130
28	3	160	59	3	150
29	5	180	60	3	155
30	2	150	61	1	135
31	3	165	62	3	150

Each result is the average of 4 tests. The slump test was made with the 4 by 8 by 12-inch truncated cone. The flow test was made with the 30-inch flow table, using 15 drops from a height of one-half inch. The flow is the final diameter of a truncated cone whose original diameter was 100.

SECTIONS ARRANGED IN FIVE GROUPS FOR STUDY OF VARIOUS CHARACTERISTICS

The sections were divided for purposes of study into five major groups, as shown in Table 2. The source, character, and physical characteristics of the various coarse aggregates are given in Table 3 and the corresponding information for the fine aggregate in Table 4. The sections in Group I were constructed with the purpose of studying the effect of variations in quality of crushed stone as coarse aggregate. Ten samples of stone varying in quality from hard, tough traps to very soft limestones were used for this purpose. In Group II the effect of variations in the quality of gravel as a coarse aggregate was investigated, and for this purpose nine samples of gravel from various parts of the country were obtained. Group III comprised eight sections and was for the purpose of studying the behavior of slag as concrete road aggregate. Both the size and quality of the slag as well as the effect of slag sand used as fine aggregate were studied. Four sections were constructed with blast furnace slag, three of them using slag weighing approximately 75 pounds per cubic foot and one in which the slag weighed 56 pounds per cubic foot. Copper and lead smelter slags were used in the other three sections in this group. Group IV was composed of seven sections in which the type of sand was the object of study. The sands used ranged from a very coarse, glacial sand from Wisconsin to a fine, pure quartz sand from South Carolina. Strength ratios varied from 126 to 70 per cent and the fineness modulii from 2.2 to 3.5. The sections in Group V were constructed with the idea of noting the effect of various miscellaneous factors on the surface wear, such as consistency of the concrete, the time of mixing, proportions or cement content, grading of aggregates, and the effect of hydrated lime as an admixture. In addition, sections were constructed using mine chats from Joplin, Mo., and burnt clay as coarse aggregates and limestone screenings in place of sand as fine aggregates. Finally, two sections were built without coarse aggregate, one in the proportion of 1 : 1½ and the other a 1 :3 mortar.

Finishing the concrete



Experimental sections covered with earth for curing

TABLE 2.—*Layout of test sections*

GROUP I

[Sections 1 to 10, inclusive]

Factor studied.—Effect of quality of rock as coarse aggregate.
Fine aggregate.—Potomac River sand.
Coarse aggregate.—Various. (See Table 3.)
Proportions.—1: 1½: 3 by volume.

GROUP II

[Sections 11 to 19, inclusive]

Factor studied.—Effect of quality of gravel as coarse aggregate.
Fine aggregate.—Potomac River sand.
Coarse aggregate.—Various. (See Table 3.)
Proportions.—1: 1½: 3 by volume.

GROUP III

[Sections 35 to 42, inclusive]

Factors studied.—Effect of grading and quality of slag as fine and coarse aggregate.
Fine aggregate.—Potomac River and slag sand. (See Table 4.)
Coarse aggregate.—Various. (See Table 3.)
Proportions.—1 : 1½ : 3 by volume.

GROUP IV

[Sections 20 to 26, inclusive and Section 50]

Factors studied.—Effect of grading and quality of sand as fine aggregate.
Fine aggregate.—Various. (See Table 4.)
Coarse aggregate.—Dolomite. (See Table 3.)
Proportions.—1: 1½: 3 by volume.

GROUP V*a*

[Sections 27 to 34, inclusive and Section 46]

Factors studied.—Effect of consistency, time of mixing and cement content.
Fine aggregate.—Potomac River sand.
Coarse aggregate—
Sections 27 to 34: Dolomite.
Section 46: Potomac River gravel.
Section 27: Very dry consistency.
Section 28: Medium consistency.
Section 29: Wet consistency.
Section 30: Time of mixing, 0 minute.
Section 31: Time of mixing, 2 minutes.
Section 32: Proportions 1: 2: 4 by volume.
Section 33: Proportions 1: 2: 3 by volume.
Section 34: Proportions 1: 2½: 5 by volume.
Section 46: Proportions 1: 2½: 5 by volume.

GROUP V*b*

[Sections 47 to 54, inclusive]

Factors studied.—Effect of tolerance material in aggregates and one-size stone.
Fine aggregate.—Potomac River sand. (See Table 4.)
Coarse aggregate.—Dolomite graded 1½ to one-fourth inch unless otherwise noted.
Proportions.—
Sections 47 to 53: 1: 1½: 3.
Section 54: 1: 1.7: 3.2.
Section 47: Fine aggregate contains 15 per cent one-fourth to one-half inch gravel.
Section 48: Coarse aggregate contains 15 per cent stone screenings.
Section 49: Fine aggregate contains 15 per cent one-fourth to one-half inch gravel.
Coarse aggregate contains 15 per cent stone screenings.
Section 50: Fine aggregate, fine Potomac River sand. (See Table 4.)
Section 51: Coarse aggregate, one-fourth to three-fourths inch only.
Section 52: Coarse aggregate, three-fourths to 1½ inch only.
Section 53: Fine aggregate, fine Potomac River sand.
Coarse aggregate, three-fourths to 1½ inch only.
Section 54: Aggregates as in Section 53.
Proportions: 1: 1.7: 3.2.

GROUP V*c*

[Sections 56 to 62, inclusive]

Factors studied.—Effect of mortar top, hydrated lime and stone screenings.
Fine aggregate.—(See Table 4.)
Coarse aggregate.—(See Table 3.)
Proportions.—1: 1½: 3 unless otherwise noted.
Section 56: Mortar top 1: 1½ by volume.
Section 57: Mortar top 1: 3 by volume.
Section 58: 12 per cent hydrated lime by volume added, based on weight of cement.
Section 59: 12 per cent hydrated lime by volume added, based on weight of cement.
Section 60: 12 per cent hydrated lime by volume added, based on weight of cement.
Section 61: Limestone screenings free from dust used in place of sand.
Section 62: Limestone screenings containing dust of fracture used in place of sand.

GROUP V*d*

[Sections 43 to 45, inclusive]

Factors studied.—Effect of mine chats and burnt clay.
Fine aggregate.—(See Table 4.)
Coarse aggregate.—(See Table 3.)
Proportions.—
Section 43: 1: 1½: 3.
Section 44: 1: 1: 3.
Section 45: 1: 2: 2.

TABLE 3.—*Tests of coarse aggregates*

Section No.	Type of material and source	Wear	Hardness	Toughness	Weight per cubic foot		Absorption	Grading
					Solid	Crushed		
		P. ct.			*Lbs.*	*Lbs.*	*P. ct.*	*In.*
1	Argillaceous dolomite: Berkeley County, W. Va	4.0	17.0	9	176	------	0.15	¼-1½
2	Altered diabase: Somerset County, N. J	2.3	18.0	31	183	------	0.03	¼-1½
3	Quartzite: Minnehaha County, S. D	1.8	18.7	22	165	------	0.14	¼-1½
4	Granite porphyry: Llano County, Tex	2.4	18.7	20	164	------	0.15	¼-1½
5	Calcareous slate: Texas	4.0	18.3	22	163	------	0.51	¼-1½
6	Sandstone: Pine County, Minn	13.8	14.8	4	156	------	1.23	¼-1½
7	Sandstone: Wise County, Va	5.3	16.3	11	156	------	1.55	¼-1½
8	Argillaceous limestone: Huron County, Ohio	6.3	10.3	5	157	------	3.70	¼-1½
9	Limestone: Presque Isle County, Mich	9.5	13.3	5	162	------	0.80	¼-1½
10	Dolomite: Jones County, Iowa	14.5	0.0	4	132	------	8.96	¼-1¼
11	Gravel consisting of rounded fragments of quartz, schist, and sandstone: Potomac River	[1] 11.0	------	----	158	------	1.09	¼-1
12	Gravel consisting of rounded fragments of dolomite: Will County, Ill	[1] 9.3	------	----	161	------	1.63	¼-1½
13	Gravel consisting of rounded fragments of limestone, dolomite, chert and sandstone: Warren County, Ind	[1] 9.4	------	----	159	------	1.76	¼-1½
14	Gravel consisting of rounded fragments of quartz, sandstone, limestone and granite: St. Joseph County, Ind	(2)	------	----	160	------	1.48	¼-1½
15	Gravel consisting of rounded fragments of quartz, rhyolite, andesite and sandstone: Marathon County, Wis	(2)	------	----	164	------	1.49	¼-1½
16	Gravel consisting of rounded fragments of granite, quartz and diorite: Hampden County, Mass	[1] 14.3	------	----	149	------	0.48	¼-1½
17	Gravel consisting of rounded fragments of quartz: Richland County, S. C	[1] 29.0	------	----	144	------	0.33	¼-1½
18	Gravel consisting of rounded fragments of quartz, granite, sandstone, quartzite and gabbro: Vanderburg County, Ind	(2)	------	----	150	------	2.11	¼-1

[1] Test made in accordance with Method No. 2, Abrasion test for gravel, U. S. Dept. of Agric. Bulletin 949, p. 3.
[2] Pieces not large enough for this test.

TABLE 3.—*Tests of coarse aggregates*—Continued

Section No.	Type of material and source	Wear	Hardness	Toughness	Weight per cubic foot		Absorption	Grading
					Solid	Crushed		
19	Gravel consisting of rounded fragments of granite, sandstone shale and limestone: Hancock County, Iowa	[2]	------	----	149	------	4.45	¼–1½
20–34	Argillaceous dolomite: Berkeley County, W. Va	4.0	17.0	9	176	------	0.15	------
35	Blast furnace slag: Montgomery County, Pa	15.9	------	----	------	75	------	¼–2½
36	Blast furnace slag: Montgomery County, Pa	15.9	------	----	------	75	------	¼–1½
37	Blast furnace slag: Montgomery County, Pa	15.9	------	----	------	75	------	¼–1½
38	Blast furnace slag: Montgomery County, Pa	15.9	----	----	------	75	------	¼–1½
39	Blast furnace slag: Morris County, N. J	------	------	----	-----	56	------	¼–1½
40	Copper slag: Polk County, Tenn	4.1	18.0	14	220	------	0.58	¼–1½
41	Copper slag: Middlesex County, N. J	------	-----	----	------	------	------	¼–1½
42	Lead slag: Middlesex County, N. J	------	------	----	------	------	------	¼–1½
43	Chert chips: Jasper County, Mo	[2]	------	----	------	------	------	[3] 10–½
44	Chert chips: Jasper County, Mo	[2]	------	---	------	------	------	[3] 10–½
45	Burnt clay: St. Louis County, Mo	[2]						¼–¾
46	Gravel consisting of rounded fragments of quartz, schist and sandstone: Potomac River	[1] 11.0	------	----	158	------	1.09	¼–1½
47–55	Argillaceous dolomite: Berkeley County, W. Va	4.0	17.0	9	176	------	0.15	¼–1½
58	Altered diabase: Polk County, Wis	[2]	18.3	32	185	------	0.01	¼–1½
59–62	Argillaceous dolomite: Berkeley County, W. Va	4.0	17.0	9	176	------	0.15	¼–1½

[1] Test made in accordance with Method No. 2, abrasion test for gravel, U. S. Dept. of Agr. Bulletin 949, p. 3.
[2] Pieces not large enough for this test.
[3] Sieve No. 10.

TABLE 4.—*Tests of fine aggregates*

Sec. No.	Type of material and source	Mechanical analysis. Total per cent passing designated sieves							Silt	Color plate No.	Tensile strength ratio	
		¼ inch	10	20	30	50	100	200			7 days	28 days
	River sand:								*P. c.*		*P. c.*	*P. c.*
1–20	Potomac River	98	74	60	46	22	9	5	3.0	1	125	114
	Bank sand:											
21	Merrimack County, N. H	94	66	35	15	3	1	1	0.5	1	71	70
22	Fayette County, Pa	100	93	75	54	21	6	2	1.5	3–4	98	109
	Bank sand:											
23	Marathon County, Wis	99	86	66	34	7	1	1	0.4	1	127	123
	River sand:											
24	Charleston County, S. C	100	98	83	53	20	3	1	------	1	92	92
25	St. Clair County, Mich	100	68	55	47	18	2	1	0.5	1	145	126
26	Colorado River	100	89	55	16	2	1	1	0.4	1	126	108
27–37	Potomac River	98	74	60	46	22	9	5	3.0	1	125	114
38	Blast furnace slag sand: Montgomery County, Pa	94	51	28	19	12	9	7	5.0	---	55	94
	River sand:											
39	Potomac River	98	74	60	46	22	9	5	3.0	1	125	114
40	Copper slag sand: Polk County, Tenn	99	85	47	17	6	3	2	1.0	---	96	132
	River sand:											
41–42	Potomac River	98	74	60	46	22	9	5	3.0	1	125	114
	"Rolled chat sand":											
43	Jasper County, Mo	100	58	33	22	15	11	9	5.4	---	200	199
	"Jig Sand":											
44	Jasper County, Mo	100	100	94	60	28	12	5	1.4	---	82	95
	Burnt Clay:											
45	St. Louis County, Mo	99	76	51	33	19	12	8	3.8	---	108	127
	River sand:											
46	Potomac River	98	74	60	46	22	9	5	3.0	1	125	114
47	Do	83	63	51	39	19	8	4	3.0	1	------	------
48	Do	98	74	60	46	22	9	5	3.0	1	125	114
49	Do	83	63	51	39	19	8	4	3.0	1	------	------
50	Do	100	99	91	65	19	5	2	1.2	1	90	84
51–52	Do	98	74	60	46	72	9	5	3.0	1	125	114
53–54	Do	100	99	91	65	19	5	2	1.2	1	90	84
	Limestone screenings:											
55	Berkeley County, W. Va	95	50	5	0	0	0	0	------	---	------	------
	River sand:											
56–59	Potomac River	98	74	60	46	22	9	5	3.0	1	125	114
60	Do	100	99	91	65	19	5	2	1.2	1	90	84
	Limestone screenings:											
61	Berkeley County, W. Va	95	58	30	16	8	4	2	------	---	------	------
62	Do	94	69	47	35	28	22	18	------	---	202	162

DESCRIPTION OF THE TESTING MACHINE

A view of the testing machine is shown on page 1. It was so constructed as to have approximately the same effect as ordinary truck traffic. Since the width of the test section was only 4 feet the design of the machine included only one front and one rear truck wheel. The solid-rubber-tired wheels were mounted in a frame between two truck springs which served to carry the necessary loading of 600 pounds per inch of tire width on each of the truck wheels, the total load on each wheel being 3,000 pounds. The rear wheel was driven by means of an electric motor. The frame which carried the truck wheels and their loading was mounted in a flexible manner within another frame. The outer frame, equipped with four flanged car wheels, was supported and guided by railroad rails laid at each side of the test sections, and, in turn, guided the inner frame and truck wheels. The whole apparatus was propelled by the truck wheels, which acted freely with their whole weight on the concrete surface. Two complete machines were constructed and coupled together in the test, with each of the truck wheels so aligned that all followed in the same path, approximately 6 inches in width. The abrasive action, thus concentrated upon a narrow path, was greatly accelerated. The machines were operated and controlled through a third-rail system and were run at a speed of from 20 to 22 miles per hour.

TESTS OF CONTROL SPECIMENS

For each section of the pavement, control specimens were made for the purpose of determining the strength of the concrete in compression, cross bending, and resistance to wear. This group of specimens consisted of three 6 by 12 inch cylinders, two 6 by 8 by 48 inch beams, and three 8 by 8 by 5 inch Talbot-Jones wear blocks. A small amount of concrete was taken from each batch for the purpose of fabricating the control specimens, so that the concrete in these specimens may be considered to be fully representative of that in the test sections proper.

Twenty-four hours after molding, the forms were removed and the specimens placed on the section which they represented. An earth covering was then placed over the entire section and kept continuously wet for 14 days, after which the concrete was exposed to the weather. All of the specimens were in an air-dry condition when tested at the age of 90 days.

Compression tests were made in a 200,000-pound Universal testing machine, following in detail the standard American Society for Testing Materials procedure. Deformation readings for determining the modulus of elasticity in compression were taken on two cylinders from each section by means of an improved form of compressometer. This compressometer was of the two-ring, two-dial type and proved very sensitive, movements of the dials taking place under initial loadings of 40 pounds per square inch. It is shown mounted on a specimen in the illustration on page 7. The results of the compression tests, including modulus of elasticity in compression, are given in Table 5.

The transverse tests were made on 6 by 8 by 48 inch beams, using a span of 42 inches with center loading. The beams were supported at the ends by specially designed rockers having uniform bearing against the bottom surfaces of the beams and a rocker adjustment upon the supporting plates. These plates were in turn fastened to the flanges of the I-beams at such distance that the centers of the supporting rockers were 42 inches apart, center to center. The deflection-measuring device consisted essentially of three U-shaped brackets of strap iron fastened at the neutral axis on each side of the beam with pointed set screws. Two of the brackets were placed directly over the end supports and the third fastened at the center of the span. This center bracket supported two 0.001-inch Ames dials, one on each side of the beam. Supported horizontally by the set screws of the end brackets were two wooden bars, one on each side of the beam, against which the plungers of the Ames dials rested. The illustration on page 8 shows the details of this deflection measuring apparatus. The loads were applied through a three-fourths inch ball resting on a one-fourth inch steel bar about 7 inches long. They were applied very slowly in increments of 500 pounds until failure occurred. Between the 2,500-pound load and failure the dials were watched very closely to catch the maximum deflection at the point of rupture. Table 6 gives the results of the tests for modulus of rupture and modulus of elasticity in bending for the beams of the 62 test sections.

Table 5.—*Compression tests of concrete; age, 90 days*

Section No.	Crushing strength	Modulus of elasticity	Weight per cubic foot	Section No.	Crushing strength	Modulus of elasticity	Weight per cubic foot
	Pounds per square inch	*Pounds per square inch*	*Pounds*		*Pounds per square inch*	*Pounds per square inch*	*Pounds*
1	4,668	5,100,000	154	32	3,380	6,100,000	158
2	3,932	5,000,000	165	33	3,792	5,100,000	155
3	4,139	4,900,000	156	34	3,387	4,700,000	158
4	5,510	4,800,000	159	35	4,827	4,450,000	152
5	4,930	4,800,000	156	36	4,987	5,000,000	149
6	6,143	3,550,000	148	37	4,772	4,600,000	149
7	5,015	1,650,000	149	38	4,827	3,950,000	142
8	4,786	4,000,000	153	39	4,333	4,350,000	135
9	4,883	4,100,000	155	40	6,697	6,200,000	193
10	4,878	3,450,000	151	41	6,003	6,300,000	187
11	4,163	3,300,000	155	42	6,328	3,450,000	180
12	4,345	4,250,000	154	43	4,170	4,000,000	143
13	4,770	4,300,000	155	44	3,735	4,300,000	140
14	4,192	4,000,000	156	45	5,165	2,300,000	116
15	5,203	4,400,000	159	47	4,290	5,000,000	156
16	4,883	3,200,000	180	48	5,363	5,100,000	159
17	3,827	2,750,000	155	49	4,597	4,950,000	159
18	4,537	5,100,000	152	50	4,988	5,250,000	158
19	3,480	3,350,000	152	51	4,852	5,150,000	162
20	3,902	5,150,000	158	52	5,263	6,150,000	163
21	4,240	5,250,000	161	53	5,372	5,300,000	160
22	3,615	4,800,000	156	54	5,100	4,670,000	156
23	4,923	7,000,000	159	55	1,741	3,100,000	135
24	4,925	5,500,000	160	56	7,083	3,800,000	143
25	5,197	5,700,000	163	57	3,887	3,100,000	140
26	5,585	6,200,000	160	58	5,533	6,200,000	165
27	4,530	6,250,000	158	59	4,205	5,300,000	160
28	4,648	5,550,000	158	60	4,630	5,430,000	154
29	4,268	5,100,000	158	61	3,635	4,550,000	152
30	3,842	4,800,000	161	62	3,675	4,500,000	158
31	3,992	5,100,000	159				

Table 6.—*Transverse tests of concrete; age, 90 days*

Section No.	Modulus of rupture	Modulus of elasticity	Section No.	Modulus of rupture	Modulus of elasticity
	Lbs. sq. in.	*Lbs. sq. in.*		*Lbs. sq. in.*	*Lbs. sq. in.*
1	480	4,735,000	32	592	4,225,000
2	512	4,050,000	33	728	4,395,000
3	677	3,265,000	34	545	5,030,000
4	607	3,845,000	35	589	4,390,000
5	483	4,000,000	36	690	4,545,000
6	632	3,335,000	37	597	4,030,000
7	499	1,992,500	38	774	3,600,000
8	692	3,550,000	39	729	3,525,000
9	509	3,860,000	40	754	5,330,000
10	599	3,665,000	41	573	4,365,000
11	527	3,265,000	42	558	3,255,000
12	659	4,410,000	43	627	4,005,000
13	517	4,525,000	44	692	3,985,000
14	538	3,590,000	45	416	2,045,000
15	536	4,095,000	47	623	4,230,000
16	532	2,652,600	48	762	4,585,000
17	544	2,990,000	49	788	4,642,500
18	524	4,282,500	50	668	4,427,500
19	529	2,960,000	51	742	5,005,000
20	601	4,310,000	52	635	4,515,000
21	758	4,765,000	53	706	4,205,000
22	674	4,870,000	54	620	4,195,000
23	705	5,520,000	55	342	1,950,000
24	794	5,230,000	56	727	3,285,000
25	728	5,725,000	57	641	2,787,500
26	762	5,445,000	58	667	4,805,000
27	810	4,995,000	59	749	4,187,500
28	670	4,345,000	60	652	4,780,000
29	590	4,270,000	61	568	3,945,000
30	599	4,265,000	62	656	4,125,000
31	653	4,505,000			

Each result is the average of two tests on beams 48 inches long, 6 inches wide, and 8 inches deep, tested on a 42-inch span.

Table 7.—*Talbot-Jones wear tests of concrete; age, 90 days*

Section No.	Loss in weight	Depth of wear	Section No.	Loss in weight	Depth of wear
	Per cent	*Inches*		*Per cent*	*Inches*
1	3.48	0.174	32	6.42	0.321
2	7.55	.378	33	5.25	.261
3	7.57	.393	34	7.50	.375
4	5.76	.288	35	5.23	.262
5	3.91	.195	36	5.48	.274
6	5.42	.271	37	4.29	.215
7	4.06	.203	38	4.00	.200
8	5.60	.280	39	5.73	.287
9	6.51	.326	40	3.46	.173
10	6.73	.335	41	3.42	.171
11	7.73	.387	42	4.16	.208
12	6.71	.334	43	7.26	.363
13	5.82	.291	44	8.29	.415
14	6.16	.308	45	6.95	.347
15	6.05	.319	47	5.50	.276
16	4.98	.248	48	4.92	.246
17	5.05	.253	49	7.06	.353
18	5.97	.299	50	5.63	.282
19	6.39	.320	51	6.11	.305
20	6.52	.327	52	5.38	.269
21	6.31	.315	53	5.78	.289
22	6.93	.347	54	5.90	.295
23	6.00	.300	55	6.87	.344
24	4.68	.234	56	5.88	.294
25	5.72	.289	57	8.14	.407
26	5.52	.276	58	4.96	.248
27	7.08	.364	59	6.78	.286
28	5.69	.285	60	5.41	.271
29	6.28	.314	61	6.78	.339
30	7.15	.350	62	6.85	.343
31	5.28	.264	------	----------	----------

Each result is the average of three specimens.

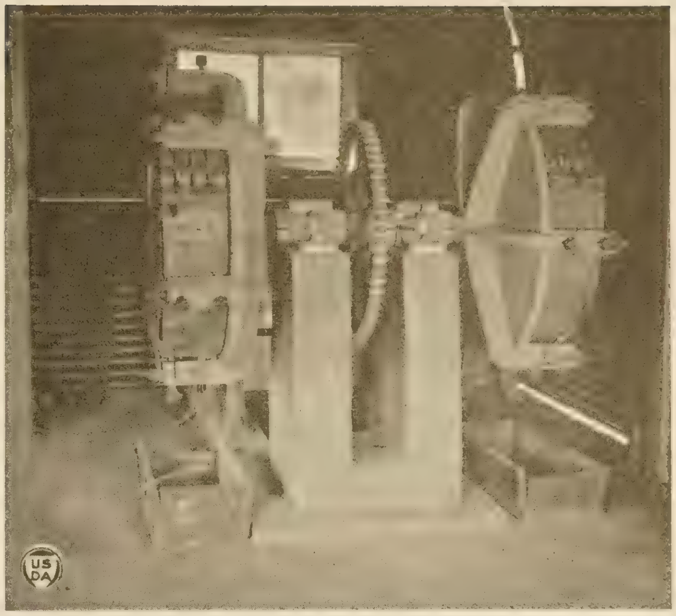
The laboratory wear tests were made in accordance with the methods employed by Abrams [1] using the Talbot-Jones rattler, and the results are given in Table 7. A view of the rattler is shown on page 7.

[1] Bulletin 10, Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill.

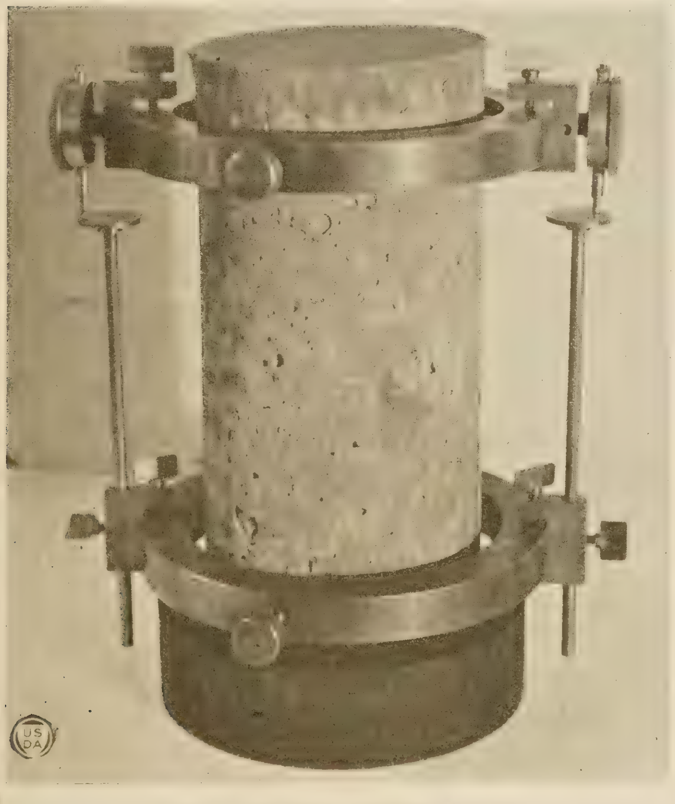
TESTS OF THE CIRCULAR PAVEMENT

Operation of the testing machine was begun in March, 1923, about six months after the completion of the test sections. As already noted the tests have so far been run in two stages, the first consisting of 75,000 trips of the machine equipped with solid rubber tires, and the second of 25,000 trips of the same machine, with the front wheel of each truck equipped with nonskid chains. The test with tire chains was started in July, 1923, and was continued intermittently until February, 1924, the wheels following an entirely different path from that followed during the first stage. Although a considerable portion of the second stage was run during December and January, weather conditions were so favorable that practically no snow or ice was encountered.

For the purpose of determining the actual amount of wear which took place under both rubber-tire and chain traffic, measurements were taken at four places on each test section by means of the special device shown on page 9. This device consists essentially of a rigid bar about 11 inches in length mounted on two flat, circular plates which rest upon the pavement and which are pivoted to the cross bar through universal joints. An Ames dial with a plunger capable of 1 inch extension is mounted rigidly at the center of the cross bar together with a cross bubble for leveling the apparatus. Measurements were taken at certain points on the section, the locations of which were fixed by black, circular spots of the same diameter as the bearing plates. With the instrument placed exactly on these spots, which were of course outside the line of travel, differences of reading on the dial between any two successive measurements represented the actual amount of wear which had taken place during the interval. An average of four readings on a section taken by this method is probably a fair indication of the wear on the section as long as it remains fairly uniform. Development of marked inequalities in wear, however, made this method very uncertain, so that for the final measurement under chain traffic another scheme was used. This consisted in taking a plaster cast of a portion of each section about 12 inches long, and somewhat wider than the width of the traffic groove. An average 12-inch length of each section was selected for this purpose. The casts were then sawed longitudinally through the center so that an exact profile of the surface of the pavement was obtained. Photographs were then made of these longitudinal sections. By blocking out everything except the area included between the horizontal line representing the original surface of the pavement and the final profile, an exact record of both the amount and character of wear was secured. The average depth of wear in inches over this 12-inch section was readily calculated by dividing the area by the length of the section.



The Talbot-Jones rattler



Two-ring, two-dial compressometer mounted

As already noted, measurements were taken with the Ames dial to determine the amount of wear taking place under the rubber-tired traffic. Table 8 gives the measurements of depth in inches for each of the 62 test sections at the end of 55,000 trips of the testing machine. Surface wear, under these conditions, it will be noted, is practically negligible, amounting at the point of maximum depth to only 0.018 inch. In fact a slight discoloration of the surface is about the only visible evidence of the traffic which has passed over it. This is true of every test section regardless of the quality of the aggregates composing the concrete. As a matter of fact, the various course aggregates did not come under test at all due to the fact that they were protected by a thin mortar top which formed the finished surface of each section. These observations lead to the conclusion that rubber-tired traffic, without impact, produces no material wear on the surface of a concrete pavement, at least as long as the protecting mortar top remains intact. It must be remembered, however, that traffic on our highways is not confined entirely to vehicles with rubber tires. There is still a certain percentage of steel-tired traffic, as well as an appreciable amount of traffic using nonskid chains, especially in the Northern States during the winter months. Moreover, the solid-rubber tires used in these experiments were new and therefore produced no impact or abrasion such as might result from the use of badly worn or broken tires, where the rim comes in contact with the pavement.

THE SIGNIFICANCE OF THE CHAIN TRAFFIC TESTS

It is realized, of course, that chain traffic on concrete highways is not dense enough, except in isolated instances, to cause any such wear of the surface of a pavement as was produced on the test sections. Wear, however, does take place at the unprotected edges of joints and cracks, eventually causing raveling or disintegration unless carefully maintained by the use of bituminous materials. It was believed that the relative resistance offered by the various sections to the chain traffic would be a fairly reliable index of the ability of the several materials to resist the combined destructive influences of traffic, whether such influences produce wear at joints or cracks or cause surface disintegration.

Taking the above facts into consideration, it was felt that the question was not definitely answered by simply determining that rubber tires rolling over the pavement produce no wear. It was decided therefore to continue the test, using the nonskid chains, this being not only the simplest but also the fairest method of securing the desired information.

The chains produced a noticeable effect on the surface from the beginning. Wear was rapid and progressive. Marked differences in the behavior of the different materials was also noted. In fact, at the completion of 25,000 runs, a groove almost an inch in depth had been cut in certain of the sections, whereas others showed as little as 0.2 inch wear, or only about one-fifth as much. It will be of interest to take up the several groups as outlined in Table 2 and discuss the behavior of the several sections with respect to the kind and quality of materials used in their construction.

DISCUSSION OF WEAR CAUSED BY NONSKID CHAINS

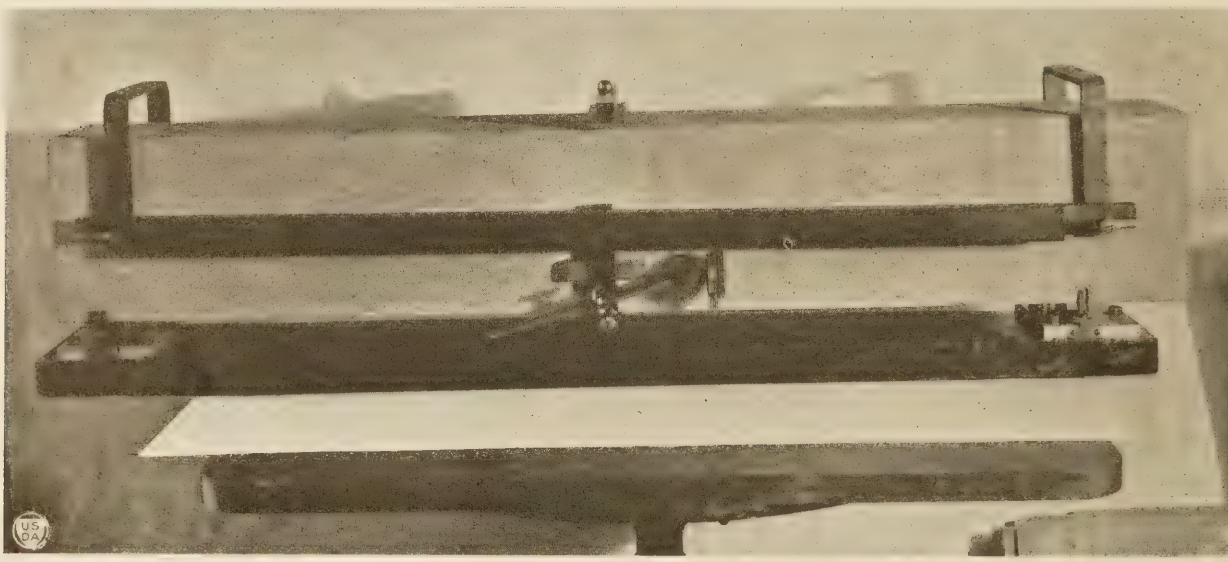
Ames-dial readings taken at various times during the test with chains are plotted in Figure 1. These results show the progress of wear from time to time. The first set of readings, for instance, was taken about the time the coarse aggregates became exposed so that it may be considered that only about 1,000 runs of the machine with chains were necessary to wear away the thin mortar top which resisted without appreciable damage over 75,000 runs of the same machine without chains.

The depth of wear at the conclusion of the second stage, together with the results of tests of the aggregates, as well as the control tests of the concrete are plotted together for purposes of comparison in Figure 2. The total depth of wear under chain traffic as well as the corresponding wear when the machine was operated without chains are also tabulated in Table 8. For the purpose of studying the relative uniformity as well as depth of wear, reproductions of the plaster-cast profiles are likewise given in Figures 3, 4, 5, and 6. A discussion of the results of the wear tests, group by group, follows.

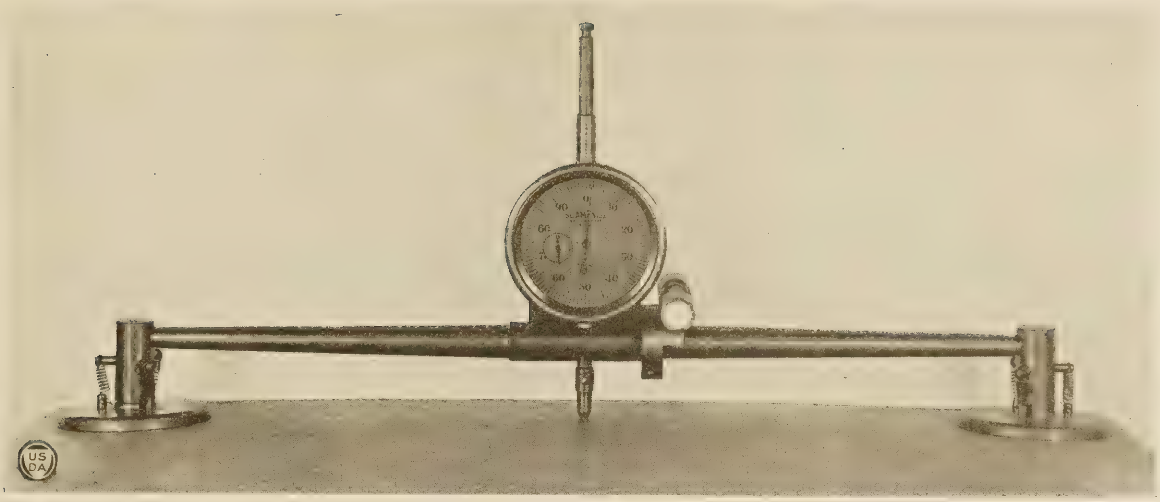
GROUP I.—SECTIONS 1 TO 10, INCLUSIVE: THE EFFECT OF QUALITY OF ROCK

Reference to Group I in Figures 2 and 3 reveals some very interesting facts. In Figure 2 the sections in Group I are arranged in the order of the resistance to wear of the various aggregates, as measured by the standard Deval abrasion test. It will be observed that the wear on the sections containing the soft limestones and sandstones was much greater than on the sections in which the harder materials were used. Sections 6, 9, and 10 in particular, the poorest materials from the standpoint of the laboratory test, show excessive wear in the pavement. On the other hand, sections 1 to 5, inclusive, all of which contain aggregate having a percentage of wear of 4 or less, show much less surface wear.

An interesting point in connection with this group is the relation between the wear on the mortar matrix



Apparatus for measuring the deflection of concrete



Instrument used for measuring the wear of the concrete

and the wear on the coarse aggregates. Reference to section 56, which is a 1:1½ mortar top, shows a wear of 0.33 inch, which is only a little greater than the wear on sections 2, 3, and 4 in which the hard and tough trap, quartzite, and granite were used. From the standpoint of surface wear it would seem, therefore, that it is only necessary that the coarse aggregate be at least as resistant to wear as the mortar matrix. Even a good mortar matrix, on the other hand, is apparently not able to protect a poor coarse aggregate, once it has been exposed, as witness the behavior of sections 9 and 10. Both of these sections contained extremely soft limestones which have been proposed for use in concrete roads. The results of this test, however, would indicate that stone so soft that it will show in the laboratory a percentage of wear over 7 should not be used in concrete road construction.

Table 8.—*Average depth of wear under rubber-tire and chain traffic*

Section No.	Depth of wear: Solid-rubber tires 55,000 runs	Depth of wear: Tire equipped with chains 25,000 runs	Section No.	Depth of wear: Solid-rubber tires 55,000 runs	Depth of wear: Tire equipped with chains 25,000 runs
	Inches	*Inches*		*Inches*	*Inches*
1	0.018	0.37	31	0.012	0.28
2	.006	.30	32	.015	.25
3	.012	.27	33	.005	.25
4	.008	.27	34	.010	.46
5	.005	.22	35	.009	.43
6	.010	.58	36	.012	.40
7	.007	.42	37	.007	.27
8	.006	.52	38	.014	.57
9	.012	.78	39	.005	.89
10	.013	.87	40	.008	.27
11	.010	.24	41	.014	.18
12	.013	.35	42	.013	.23
13	.008	.38	43	.010	.99
14	.009	.35	44	.006	.61
15	.008	.24	45	.014	.55
16	.008	.30	46	.012	.35
17	.007	.33	47	.013	.35
18	.012	.18	48	.015	.35
19	.009	.40	49	.014	.38
20	.008	.21	50	.009	.38
21	.016	.32	51	.011	.35
22	.011	.43	52	.005	.23
23	.013	.31	53	.004	.37
24	.005	.30	54	.011	.34
25	.009	.31	55	.008	----------
26	.008	.30	56	.009	.33
27	.012	.46	57	.009	.89
28	.008	.31	58	.012	.26
29	.006	.40	59	.006	.30
30	.011	.38	60	.014	.30
			61	.007	.44
			62	----------	.41

Another interesting point in connection with the first group is the behavior of the sandstone in section 6. This is the so-called Kettle River sandstone, which gave in the laboratory a percentage of wear of 13.0. Reference to Figure 3 will show that the wear on section 6, although deep, was exceptionally uniform. The depth of wear also is not so great in proportion to the percentage of wear in the Deval test, as in the case of the soft limestone sections. Whether this means that sandstones are better than limestones of equal hardness it is impossible to say, although as we shall see later, certain sections in the gravel group show the same relative behavior. With the regard to uniformity of wear, the moderately hard aggregates, i. e., those of about the same resistance to abrasion as the mortar matrix, seem to have all the best of it. Reference to Figure 3 and a visual examination of the sections themselves show that section 1 is somewhat smoother than either section 2, containing the trap, or section 3, containing the quartzite, and is much smoother than sections 9 and 10 in which the very soft limestones were used. There is not much choice between 1 and 4 or 5. Section 8, although worn rather deeply, is also smooth. This is an Ohio limestone with a percentage of wear of 6.0.

GROUP II.—SECTIONS 11 TO 19, INCLUSIVE: EFFECT OF QUALITY OF GRAVEL

In this group are 9 sections in which various types of gravel were used. The most noticeable fact about this group is that, although a very wide range in quality of gravel is represented, in no case was the depth of wear so great as in five of the sections in Group I. When tested by the Rea [2] abrasion test, the gravel samples ranged from 9.3 per cent to 29 per cent loss. Several of the samples consisted of pieces too small to make the abrasion test. Section 17 is a remarkable one. Although the South Carolina quartz gravel used in this section showed a percentage of wear of 29 (which is roughly equal to about 10 in the Deval abrasion test), the depth of wear is but little above the average for all

[2] U. S. Department of Agriculture Bulletin 949, p. 3.

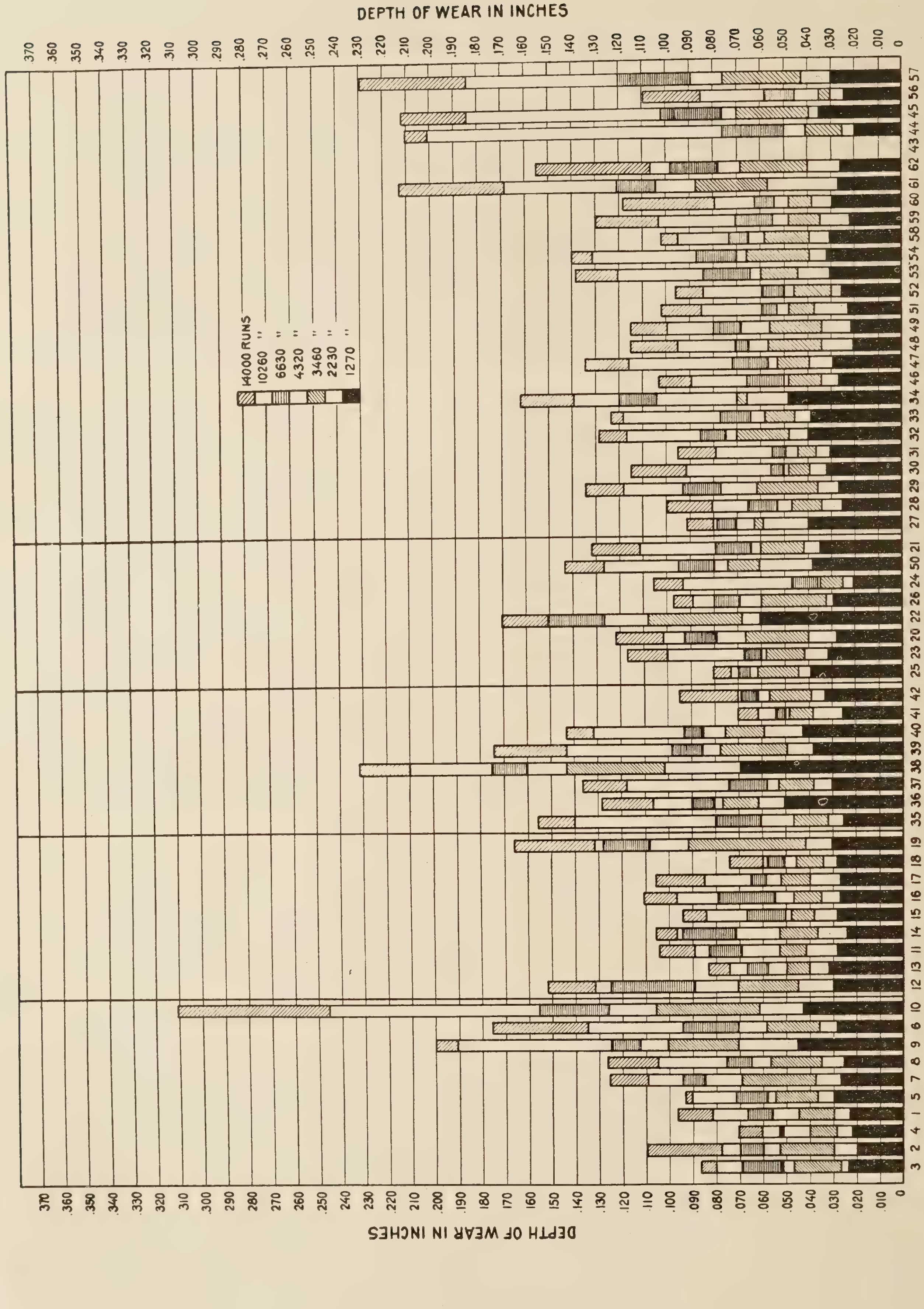


FIG. 1.—The depth of wear of all sections at several stages of the test

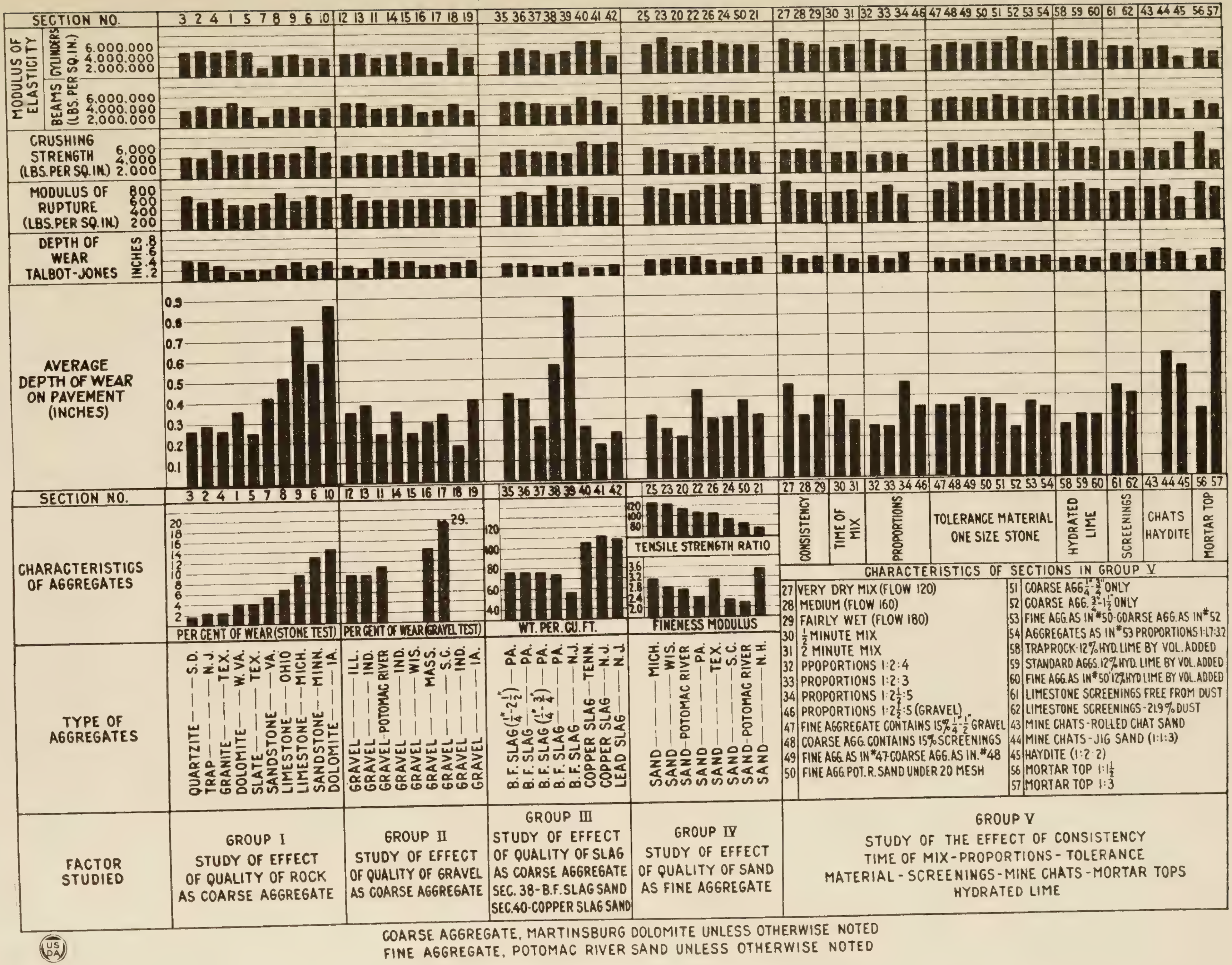


FIG. 2.—Showing the characteristics of the sections in Groups I to V

the gravel sections and is not so great as in many of the stone sections. A close examination of this gravel shows that it is made up entirely of quartz pebbles of varying hardness, on the inside, but all protected by a hard, exterior shell. A pebble when struck even a light blow will fly to bits, yet it is very difficult to scratch the surface with a knife. Apparently the quality of hardness was more important than toughness in resisting surface wear on these test sections, which leads to the conclusion that the modified Deval test for gravel may be too severe. Because of the presence of the large shot in the rattler, it is distinctly a test for toughness. The worst gravel from the standpoint of depth of wear is the one in section 19. This gravel contained about 5 per cent of shale particles and was submitted as an example of an aggregate of doubtful quality.

While the gravel sections appear to advantage in comparison with the stone sections from the standpoint of depth of wear, comparison as to uniformity of wear is not favorable to them. Sections 15 and 16, both containing glacial gravels, show up best in this respect. It is an interesting fact in this connection that the 4 sections in which limestone is a constituent material of the gravel are the poorest as regards uniformity of wear. In sections Nos. 13 and 14, the gravels contain quite an appreciable amount of soft sandstone pebbles which have been worn away by the action of the chains, leaving marked depressions in the surface. The result is well illustrated in Figure 3. On the other hand, the mere fact that the fragments are smooth and rounded appears to make little difference as far as wear is concerned. Section 18 is a good example. Before the chain traffic was well under way, the prediction was freely made that the smooth, rounded pebbles composing this sample would speedily kick out. As a matter of fact, this is one of the best of the gravel sections, especially as regards depth of wear. Summing up, it would appear from these tests that gravel as a class should prove as satisfactory for aggregate in concrete as crushed stone, provided it is free from an excess of soft or disintegrated pebbles. Moreover, the mere fact that the particles lack toughness is no drawback provided they are sufficiently hard to resist abrasion.

GROUP III.—SECTIONS 35 TO 42, INCLUSIVE: EFFECT OF QUALITY OF SLAG

The sections in Group III were arranged with the object of studying a number of factors which are involved in the selection of slag as concrete aggregate. In the first place it was of interest to determine the relative wear of slag concrete in which the maximum size of the coarse aggregate varied as follows: 2½ inches in section 35, 1½ inches in section 36, and three-fourths inch in section 37. The quality of the slag, i. e., weight per cubic foot, remained constant at 74 pounds in these sections. In section 38, the same slag, graded 1½ inches to one-fourth inch, but combined with slag sand in place of Potomac River sand was used. Section 39, however, contained slag weighing only 56 pounds per cubic foot. Copper and lead smelter slag, with both Potomac River and copper slag sand, were employed in the construction of sections 40 to 42, inclusive. Comparing, first, sections 35 to 37, it is observed that considerably less wear has taken place on the section containing the three-fourths-inch maximum size. In fact, the total wear on this section was about the same as on the sections containing the best quality of stone and gravel, which is interesting in view of the impression that slag is not suitable for concrete road construction under any circumstances.

Section 39, however, in which the 56-pound slag was used, tells a different story, the wear being over three times as great as that on section 37. A comparison of the behavior of these sections illustrates very clearly the value of keeping the light and porous material in blast furnace slag down to a minimum. In the light of these experiments, it would seem that the requirement of 70 pounds per cubic foot used in a good many States is about correct. Blast furnace slag sand as a substitute for natural sand was tried as the fine aggregate in section 38. The results were not very encouraging, due probably to the fact that the extreme harshness of the concrete made it very difficult to finish properly. The total wear on this section was 0.57 inch or twice as great as on the corresponding section using Potomac River sand. As might be expected, the copper and lead smelter slag used in sections Nos. 40 to 42 inclusive, showed very little wear. Section 41 showed less wear than any of the 62 test sections. There would seem to be no reason why such materials as these should not make excellent aggregates for concrete roads so far as durability is concerned. Unfortunately, however, the great weight of smelter slags, involving high transportation costs, limits their use to a great extent. In conclusion, it may be said that the slag experiments indicate: (1) That somewhat better results are secured by the use of the smaller sizes; (2) that the weight per cubic foot should be not less than 70 pounds; (3) that blast furnace slag sand does not make a satisfactory substitute for natural sand; and (4) that copper and lead smelter slag form excellent aggregates so far as resistance to wear is concerned.

GROUP IV.—SECTIONS 20 TO 26, INCLUSIVE, AND SECTION 50: QUALITY OF SAND

In sections 20 to 26, inclusive, as well as section 50, variations in wear, due to both quality and grading of natural sand as fine aggregates were studied. Sands were selected for this purpose varying in strength ratio from 126 per cent to 70 per cent of standard Ottawa sand and from 2.2 to 3.5 in fineness modulus. Of this group, section 22, in which a river sand from Fayette County, Pennsylvania, was used, developed the greatest depth of wear, a little over 0.4 inch. This sand contained an appreciable amount of carbonaceous shale which probably accounts for the increased wear. It was also a rather fine sand, only 25 per cent being retained on the No. 20 sieve. The next greatest wear took place on section 50, which contained the fine Potomac River sand. Reference to Table 4 will show that only 9 per cent of this sand was coarser than a No. 20 sieve, and that the strength ratio at 28 days was only 84 per cent. Four of the test sections, Nos. 21, 14, 25, and 26, all showed about the same reresistance to wear, about 0.3 inch. With the exception of No. 24, all of these may be classed as coarse sands, that is they contain more than 30 per cent of particles coarser than No. 20 mesh. The sand in No. 24, which was from South Carolina, though considerably finer, with only 17 per cent on the 20-mesh sieve, was of exceptionally good quality, being composed entirely of pure quartz grains. A comparison of this sand with that used in section 22 is of interest because they have approximately the same grading, indicating that the variation in wear is probably due to the difference in the quality of the material. Section 20, Potomac River sand, and section No. 23, contain-

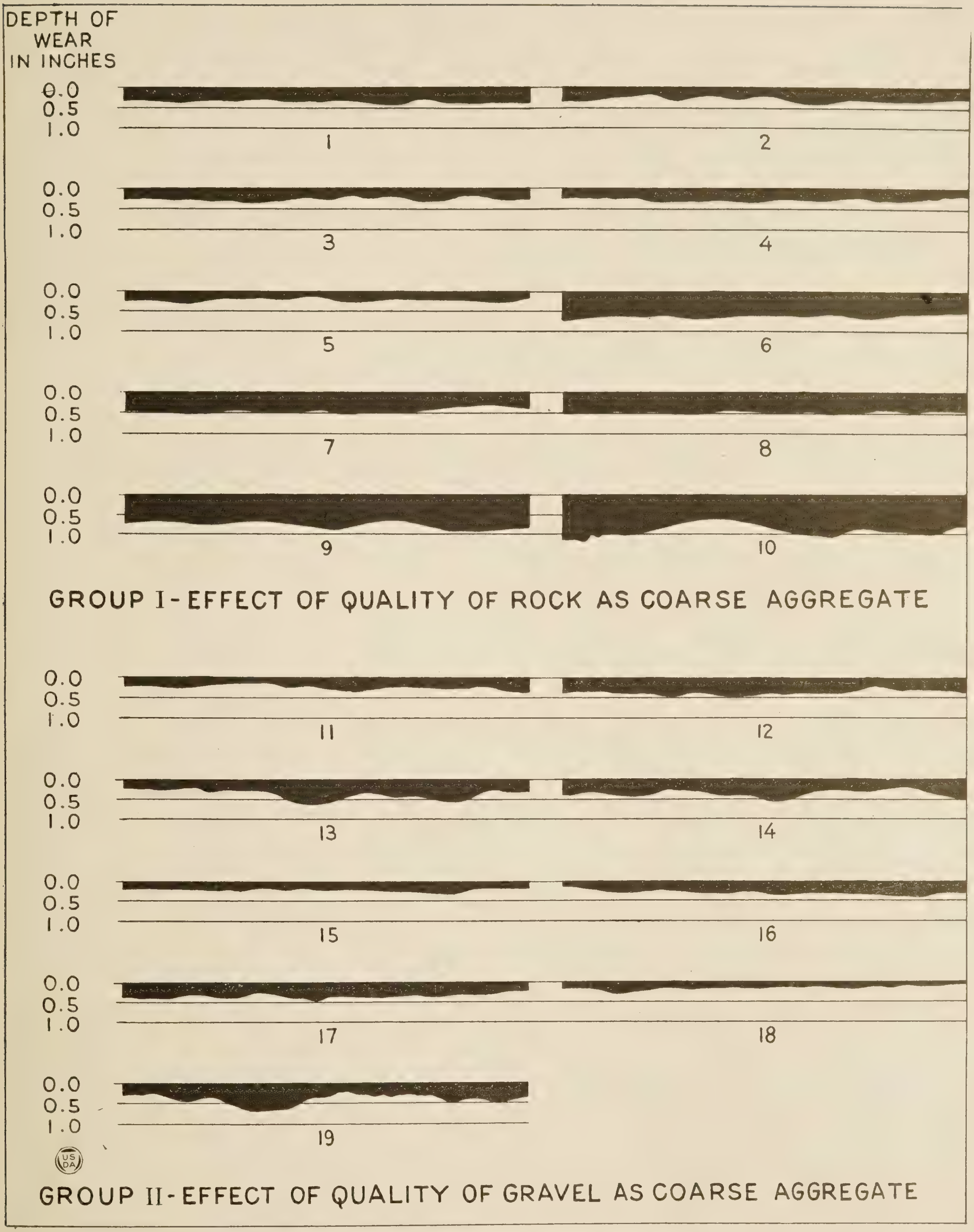


Fig. 3.—Cross sections of casts made from sections in Groups I and II

ing a glacial sand from Marathon county, Wisconsin, showed the smallest amount of wear. Both of these would be classed as coarse concrete sands.

The tests in general indicate the value of the coarse particles in reducing wear. For example, compare sections 20 and 50, in which Potomac River sand was used. The materials in these sections were identical except that the sand in section 50 was finer, the proportion of particles retained on the 20-mesh sieve being reduced from 40 per cent to 9 per cent. The effect on wear is quite marked.

When we come to consider the bearing which the strength-ratio test has on the pavement wear, we have difficulty in establishing any relation. The wide range in briquette strength is not reflected in the resultant wear any more than in the concrete strength of the materials. (Reference to Figure 2 will illustrate.) This is probably due to the fact that the factors which influence the strength, i. e., gradation and quality of sand grains, do not affect the surface wear to the same degree. For instance the mortar strength test is made on a 1 to 3 mix and compared with an Ottawa sand mortar in the same proportion, whereas the cement-to-sand ratio in the test sections was 1 to 1½. The discrepancy in results, insofar as this test is concerned, is most marked in the case of section 21, in which a glacial sand from New Hampshire, containing quite an appreciable percentage of feldspar, was used. A strength test of this sand developed only 70 per cent of the strength of Ottawa sand in spite of the fact that the sand was very coarsely graded. Although this section has worn somewhat more than the Potomac River sand sections, the difference is not nearly so great as might be supposed from the results of the mortar strength test. It would seem, therefore, that the strength-ratio test is of questionable value as an indication of the quality of sand for concrete pavement work, at least insofar as resistance to wear is concerned.

GROUP Va.—SECTIONS 27 TO 34, INCLUSIVE, AND SECTION 46: EFFECT OF CONSISTENCY, TIME OF MIX, AND CEMENT CONTENT

In Group V*a*, three very important factors having a bearing on the quality of concrete were investigated. The effects of variation in consistency show up in a very interesting way. Section 28, which contained concrete of what might be termed average consistency, i. e., slump about 3 inches, showed considerably less wear than either section 27, which was a very dry concrete, slump about 1 inch, or the very wet mix with a 5-inch slump used in section 29. The concrete comprising section 30 was mixed only one-half minute to compare with section 31, on which a two-minute mix was tried. The wear is somewhat less for the longer period. Moreover, section 31 checks fairly closely with section 28, in which the time of mixing was one minute, indicating that not much is gained, from the standpoint of wear, by mixing more than one minute.

As to the effect of cement content on wear, the tests seem to show that for the usual mixes employed in concrete road construction, i. e., 1:1½:3, 1:2:3, and 1:2:4, the actual cement content has very little influence on wear. This is illustrated by comparing sections 28, 32, and 33. This conclusion is not surprising in view of the fact that cement in itself has very little resistance to abrasion but acts simply as a binder, so that for any mix in which the cement content is high enough to produce a strong bond, the actual wear will be proportional to the hardness of the aggregates. This further emphasizes the point brought out in the discussion of the coarse aggregates to the effect that the destructive action producing wear does so more by abrasion than by impact, and that hardness rather than toughness is the essential property in the aggregate. When we come to section 34, however, we find a different condition. Here we have a 1:2½:5 mix, a cement-to-sand ratio of 1:2½. The wear on this section is much greater than on any other in this group, indicating that a 1:2 mortar is about as lean as should be used. The 1:2½:5 gravel section, No. 46, although not showing as high an average depth of wear as section 34, wore very unevenly, verifying the indications from the tests in Groups I and II with regard to the relative uniformity of wear shown by crushed stone versus gravel.

GROUP Vb.—SECTIONS 47 TO 54, INCLUSIVE: EFFECT OF TOLERANCE MATERIAL AND ONE-SIZE STONE

There has been considerable controversy from time to time regarding the allowable limit for the so-called tolerance material in fine and coarse aggregates proposed for use in concrete pavements. Many specifications limit this amount of over or under size material to not more than 5 per cent. The sections in this group were built for the purpose of noting the effect on surface wear of allowing a 15 per cent tolerance as well as the effect of one-size instead of graded stone. Section 47 is a nominal 1:1½:3 mix in which the sand carried 15 per cent of gravel passing a one-half-inch and retained on a one-fourth-inch screen. In section 48 the stone contained 15 per cent coarse limestone screenings while in section 49 both aggregates contained 15 per cent of tolerance material. The total wear is somewhat greater on all of these sections than on the corresponding sections in which accurately screened aggregates were used. They are also somewhat rougher as may be noted by referring to Figure 5 on page 16. As would be expected, section 49 shows the greatest wear, amounting to 0.38 inch. Sections 51 and 52 showed surprising results. Section 51 contained one-fourth-inch to three-fourths-inch stone only, while in section 52 only the three-fourths-inch to 1½-inch sizes were used. The wear is noticeably less on section 52. Section 53 is the same as section 52, except that the fine Potomac River sand was used in place of the regular concrete sand. The wear is very much greater, illustrating again the marked effect of the coarser sand grains in preventing wear. Section 54 contains the same aggregates as section 53, in the proportion of 1:1.7:3.2 which is the proportion recommended by the Portland Cement Association for this aggregate grading. There is practically no difference in wear.

The tests in group V*b*, in general indicate that variations in gradation of aggregates within the limits tried do not affect the wear to any great extent, other things being equal. It must be realized, however, that from the standpoint of uniformity in workability and consequently in strength, it is very important to have uniformly graded material in a concrete road job. A harsh-working mix due to poorly graded aggregate almost invariably results in poor concrete in the field. If an attempt is made to place the concrete at proper consistency it is very hard to secure a satisfactory finish. If, on the other hand, sufficient water is added to make the mix workable, concrete greatly deficient in strength will result. The answer is to have the mix of such workability that it can be placed and finished without using excess water, and the authors believe that

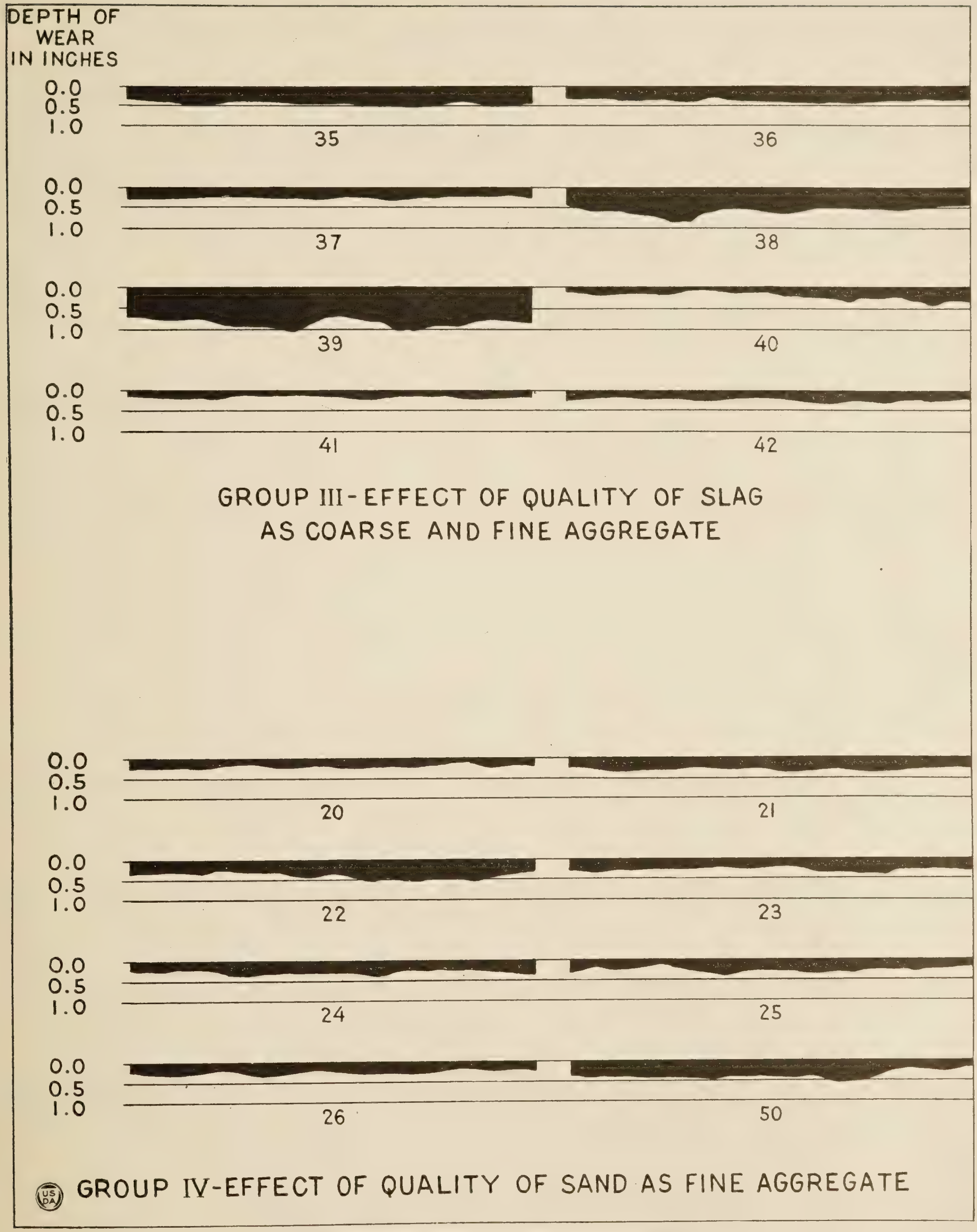


FIG. 4.—Cross sections of casts made from sections in Groups III and IV

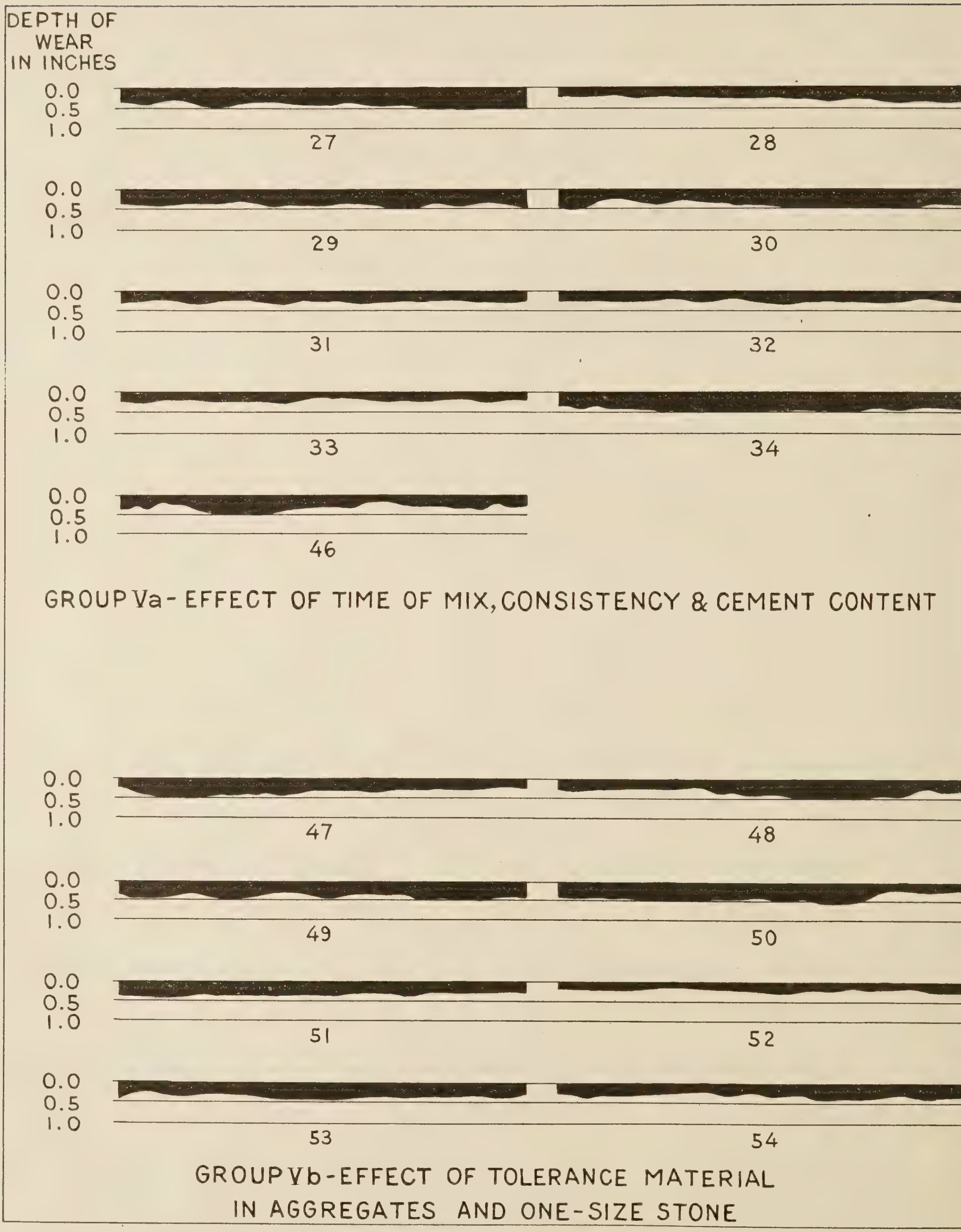


FIG. 5—Cross sections of casts made from sections in Groups V*a* and V*b*

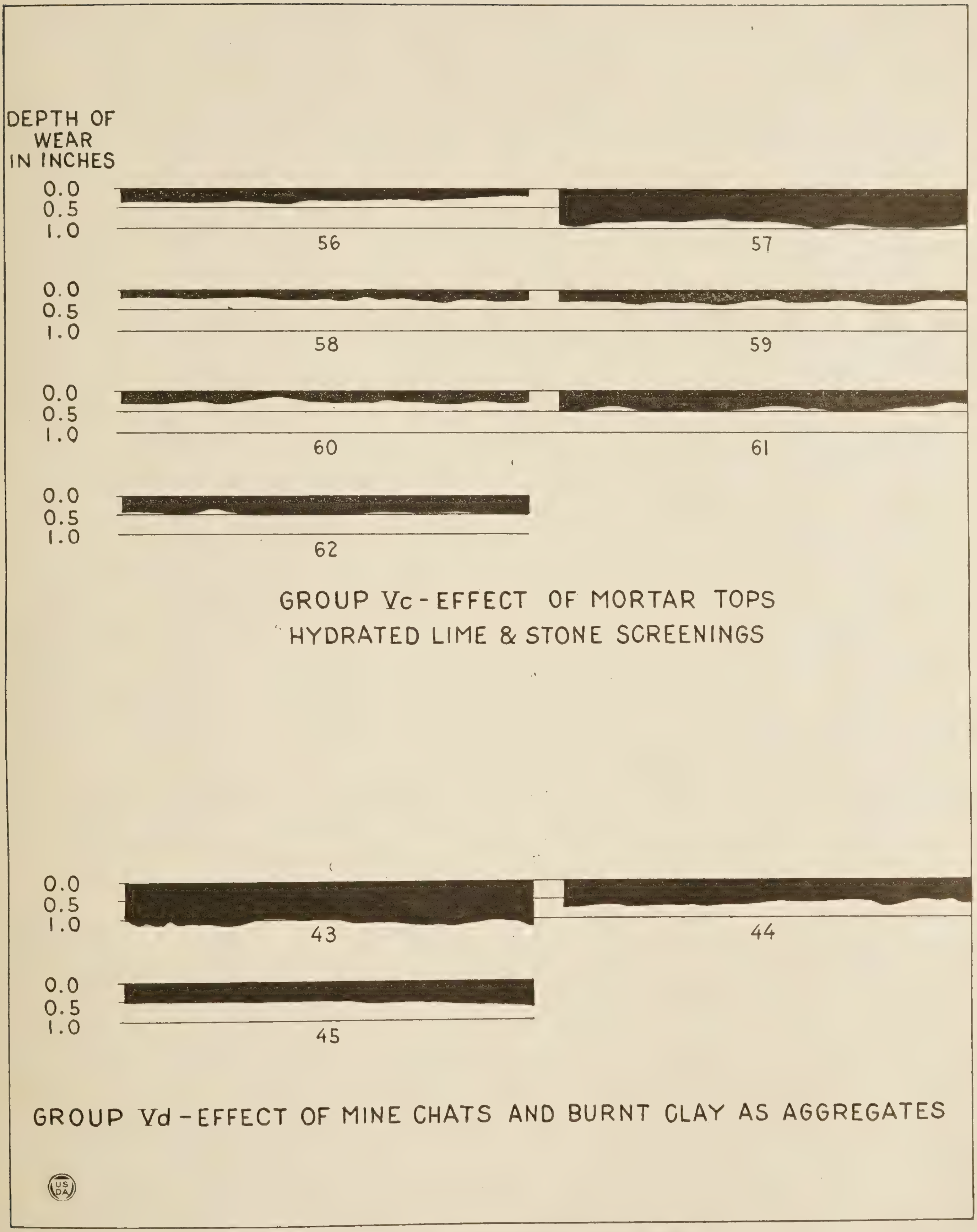


FIG. 6.—Cross sections of casts made from sections in Groups V*c* and V*d*

this can best be accomplished by having uniformly graded aggregates. This point is discussed here simply to emphasize a condition which is met with in practice, but which the tests did not cover, because, as has been stated, the concretes in all of these sections were of medium consistency, and not necessarily all of the same workability. These tests indicate what would probably result in practice provided the consistency were rigidly controlled irrespective of the harshness of the concrete mix.

GROUP Vc.—SECTIONS 56 TO 62, INCLUSIVE: EFFECT OF MORTAR TOP, HYDRATED LIME, AND STONE SCREENINGS

Reference has already been made to the condition of section 56 which is a 1:1½ mortar top compared with other sections in which the same mortar is combined as a matrix with several coarse aggregates. The wear of the mortar top is not much greater than that of the related concretes and is remarkably uniform. Section 57, however, which contains a 1:3 mortar has worn excessively, again emphasizing the fact that a 1:2 mortar is probably the leanest which should be used. Three sections, Nos. 58, 59, and 60, were constructed in which 12 per cent by volume of hydrated lime, based on the cement, was added to the regular 1:1½:3 mix. Section 58 contained a very hard, tough trap from Wisconsin, section 59, the Martinsburg limestone with Potomac River sand and section 60, the same stone with the fine Potomac River sand. About the only feature of interest in connection with these tests is the fact that section 60, although containing the fine sand, did not wear as much as corresponding sections in which no hydrated lime was used. Otherwise, the section behaved about the same as corresponding sections without lime. The two sections containing limestone screenings in place of sand, Nos. 61 and 62, did not behave very well. Due to the natural harshness of the concrete it was impossible to secure a proper finish, which has resulted in excessive wear. These two sections illustrate very well the danger of using screenings alone in place of sand unless the screenings are very carefully graded to give concrete which will not be excessively harsh, or unless a sufficient amount of sand is used to supply the lack of fines found usually in the commercial run of screenings.

GROUP Vd.—SECTIONS 43 TO 45, INCLUSIVE

Another illustration of the result of using harsh-working materials is furnished by sections 43 and 44 in which mine chats from Joplin, Mo., were used. Both of these sections have worn excessively, due not to the softness of the aggregates, but rather to the fact that the harsh mix prevented a proper finish from being obtained. The sections also furnish evidence indicating a lack of bond between the cement and the aggregates, due possibly to the very smooth surface of the latter which is peculiar to chert. Section 45 in which burnt clay, "Haydite," was used, likewise showed a large amount of wear, about double the average wear of those sections using standard aggregates.

THE RESULTS OF CONTROL TESTS

The control tests include determinations of the crushing strength, transverse strength, modulus of elasticity and resistance to wear as measured by the Talbot-Jones wear test. In this paper, however, the authors have been principally concerned with a discussion of the relations which exist between the wear on the test pavement sections and the physical properties of the aggregates used in them. No attempt will be made, therefore, to discuss in detail at this time the significance of the concrete tests. There are, however, a few salient points to which it may be well to call attention. By referring to Figure 2, it is immediately apparent that there is no consistent relation between the results of either the compression or the transverse strength tests and the depth of pavement wear. This is particularly noticeable in connection with the study of rock as coarse aggregate in Group I. Although the crushing strength of the concrete in this group varies considerably, there is no relation whatever between this property and the depth of wear. The average crushing strength of the concrete in sections 8, 9, and 10 is actually higher than in concrete in which the very hard aggregates were used.

Nor do we find any relation between wear and strength in the gravel group. While the crushing strength of the concrete in section 19 is low, its modulus of rupture is practically identical with the other concretes in this group. In the case of slag the same condition holds true. Section 39, containing the light slag, in fact, has a higher modulus of rupture than the corresponding sections containing the heavy slag. An interesting point in connection with slag is the high crushing strength developed by the smelter slag. These high values, however, are not reflected in the transverse tests. About the only feature of interest in connection with the sand Group IV, is the fact that the sand showing the greatest wear has likewise the lowest crushing strength. The results of the control tests so far as they reflect variations in consistency, time of mix and cement content, shown in Group V*a*, are fairly consistent. Variations in gradation, shown in Group V*b*, however, apparently have little effect on the strength, at least as far as is indicated by the limited number of tests made. The variation in both crushing and transverse strength is high but not consistent.

The results of the Talbot-Jones wear test likewise appear to give little indication of the actual resistance of the concrete to wear produced by traffic. The authors believe that this is due to the fact that the Talbot-Jones test is not strictly an abrasion test. The shot in the rattler have an appreciable shattering effect, especially at the edge of the block, which is not duplicated in actual traffic.

CONCLUSIONS

Before attempting to draw general conclusions from these tests, it may be well to restate certain premises regarding the volume and character of traffic to which the test pavement has been subjected. These premises follow:

1. That on the average 18-foot concrete road, approximately 10 per cent of the total traffic moving in both directions passes over a band 6 inches in width at the point of maximum concentration. Therefore, the traffic passing over the test road should be multiplied by at least 10 to obtain the equivalent volume of traffic on an actual pavement.

2. That the comparative resistance of each test section to the surface wear produced by the tire chains is an indication of the comparative resistance which would be offered by the concrete to wear or disintegration at the edges of exposed joints and cracks under service conditions.

3. That conditions which cause surface wear, such as steel tires, tire chains tracking through snow, etc., are present to such an extent as to make it necessary to give the question of wear consideration in the selection of concrete aggregates.

The authors believe that the data so far secured warrant the following conclusions:

1. That the rate of wear of stone concrete is, in general, not affected by the coarse aggregate, provided the coarse aggregate is equal or superior to the mortar matrix in resistance to wear.

2. That excessive wear will result from the use of very soft stone as coarse aggregate even though used in conjunction with a mortar of satisfactory quality. From the results of these comparative tests, it would appear that stone with a percentage of wear over 7 should not be used in concrete road construction.

3. That gravel concrete, in general, is at least as satisfactory from the standpoint of wear as stone concrete.

4. That gravels consisting essentially of siliceous materials are superior as regards both the amount and uniformity of wear to those containing a preponderance of calcareous fragments.

5. That gravels consisting of rounded particles are as satisfactory from the standpoint of wear as those consisting either wholly or in part of angular or crushed fragments.

6. That small amounts of shale occurring in the coarse aggregate will cause both excessive and uneven wear.

7. That the modified abrasion test for gravel in its present form is not an indication of the wear-resisting properties of coarse aggregates. It is suggested that if the severe impact action of the steel balls were decreased, much more indicative results would be secured.

8. That blast furnace slags should prove satisfactory for use in concrete pavements provided the proportion of light, porous slag is so controlled that the weight per cubic foot will be at least 70 pounds.

9. That the presence of large amounts of light, porous fragments in blast furnace slag will cause excessive wear.

10. That somewhat better results are secured by the use of the smaller sizes of slag.

11. That slag or stone screenings are, in general, unsatisfactory as substitutes for natural sand as fine aggregates in concrete road construction.

12. That the copper and lead smelter slags used in these tests would make satisfactory aggregates for concrete road construction from the standpoint of wear.

13. That coarse sands, other things being equal, show greater resistance to wear than fine sands.

14. That the so-called "tensile-strength-ratio" test is no indication of the wear-resisting properties of concrete made with these sands.

15. That the Talbot-Jones wear test is not, in general, an indication of the wear which takes place under traffic.

16. That neither the crushing nor the transverse strength of concrete is a measure of its wear-resisting properties.

17. That the addition of hydrated lime in the proportion used in these tests does not affect the wear-resisting properties of concrete.

18. That so far as resistance to wear alone is concerned, increasing the cement content beyond a cement-sand ratio of 1:2 does not materially affect the concrete. Leaner mixes on the other hand show marked increase in wear.

19. That unusual precautions should be taken in using mine chats or other similar harsh-working materials, so as to increase workability to a maximum and thus make possible a smoother surface finish.

20. That, other things being equal, either an excessively dry or an excessively wet mix will show less resistance to wear than concrete of medium consistency.

ROAD BOND ISSUES IN RELATION TO TOTAL DEBT

A COMPARISON OF THE HIGHWAY INDEBTEDNESS OF STATES AND LOCAL SUBDIVISIONS WITH THEIR TOTAL DEBT

By HENRY R. TRUMBOWER, Economist, U. S. Bureau of Public Roads

THE attention of the public has frequently been called to the amount of outstanding bonds which the States and their several subdivisions have issued from time to time to meet public expenditures. It has been intimated that a very substantial part of this public indebtedness consists of bonds issued to meet the costs of highway construction and improvements. An inquiry made by the United States Bureau of Public Roads into the financial status of our rural highways and the recent reports of the United States Census Bureau relative to State and local taxes and public indebtedness furnish the basis for a determination of the facts. An examination and analysis of these data prepared by governmental agencies show that for the country as a whole the indebtedness incurred on account of rural highway expenditures accounts for but a small part of the public indebtedness to which the States and their subdivisions have obligated themselves.

At the end of 1921 the total amount of highway bonds outstanding was $1,222,312,300. Of the whole issue 72 per cent are represented by local bonds, the obligations of counties, townships, and districts; 28 per cent are the obligations of State governments, as shown by the following table:

	Amount	Per cent
State Bonds	$345,574,100	28
Local Bonds	876,738,200	72
Total	1,222,312,300	100

There are only two States in the whole country which have no highway bonds of any kind outstanding, State or local. These are North Dakota and Vermont. All the other States have local bonds outstanding, with the exception of Colorado which has issued State highway bonds but has no local bonds issued for that purpose.

Over half of the States, 27 in number, have no State highway bonds outstanding. These States are:

Alabama	Kentucky	Ohio
Arizona	Louisiana	Oklahoma
Arkansas	Minnesota	South Carolina
Connecticut	Mississippi	Tennessee
Florida	Missouri	Texas
Georgia	Montana	Vermont
Indiana	Nebraska	Virginia
Iowa	New Jersey	Washington
Kansas	North Dakota	Wisconsin

New York has the largest issue outstanding, both State and local, amounting to $121,681,100; California is a close second with $101,258,000.

COMPARISON OF HIGHWAY BONDS AND TOTAL INDEBTEDNESS

The United States Census Bureau has reported the total amount of bonds issued for all purposes by States, counties, and all other subdivisions (including cities, villages, townships, and school districts). For the year 1922, which is the closest information which can be compared with the highway indebtedness, the total indebtedness, less the sinking fund assets set aside to meet such debts, was $8,695,906,000. This total indebtedness, which may be regarded as the net amount, is divided as follows:

	Amount	Per cent
State	$936, 414, 000	10. 5
Counties	1, 255, 226, 000	14. 5
All other subdivisions	6, 504, 266, 000	75. 0
Total	8, 695, 906, 000	100. 0

It will be observed that by far the major part of this total debt, or 75 per cent, is the obligation of cities, villages, townships, and school and other districts; the State and county share was but 25 per cent.

In order to make a closer comparison with the indebtedness for highway purposes, the debt of the counties and all other subdivisions, amounting to $7,759,492,000, may be called local. The direct comparison follows:

	Total indebtedness	Highway bonds: Amount	Highway bonds: Per cent of total debt
State	$936, 414, 000	$345, 574, 100	37
Local	7, 759, 492, 000	876, 738, 200	11
Total	8, 695, 906, 000	1, 222, 312, 300	14

According to this it is shown that the total highway bonds of the country amounted to 14 per cent of the total indebtedness of the States, counties, and other political subdivisions.

At the end of the fiscal year, June 30, 1923, the debt of the Federal Government was $22,525,773,000. Adding to this amount the State and local indebtedness we get $31,221,679,000 as the total indebtedness of the country. Of this amount $1,222,312,300 was represented by highway bonds, or 3.9 per cent.

A comparison of the amount of highway bonds outstanding and the total debts of States and their local subdivisions is presented below, according to geographic divisions and States:

	Total debt	Highway bonds: Amount	Highway bonds: Per cent of total debt
NEW ENGLAND STATES			
Maine	$42, 457, 000	$6, 439, 300	15. 2
New Hampshire	16, 604, 000	570, 100	3. 4
Vermont	11, 994, 000		
Massachusetts	326, 245, 000	26, 820, 800	8. 2
Rhode Island	49, 893, 000	2, 952, 000	5. 9
Connecticut	100, 954, 000	[1] 400, 000	. 4
Total	548, 147, 000	37, 182, 200	6. 8
MIDDLE ATLANTIC STATES			
New York	$1, 683, 820, 000	$121, 681, 100	7. 2
New Jersey	382, 001, 000	25, 833, 000	6. 7
Pennsylvania	550, 365, 000	[2] 50, 000, 000	9. 1
Total	2, 616, 186, 000	197, 514, 100	7. 5
EAST NORTH CENTRAL STATES			
Ohio	$670, 338, 000	$70, 936, 500	10. 4
Indiana	152, 840, 000	62, 415, 900	41. 0
Illinois	364, 019, 000	20, 617, 300	5. 6
Michigan	361, 779, 000	55, 234, 200	15. 3
Wisconsin	104, 523, 000	10, 201, 600	9. 8
Total	1, 653, 499, 000	219, 405, 500	13. 3
WEST NORTH CENTRAL STATES			
Minnesota	$269, 607, 000	$21, 402, 600	8. 0
Iowa	151, 911, 000	25, 677, 800	16. 8
Missouri	118, 276, 000	14, 942, 600	12. 6
North Dakota	40, 266, 000		
South Dakota	50, 554, 000	3, 649, 000	7. 2
Nebraska	97, 819, 000	2, 848, 000	2. 9
Kansas	123, 470, 000	9, 182, 400	7. 4
Total	851, 903, 000	77, 702, 400	9. 1
SOUTH ATLANTIC STATES			
Delaware	$22, 453, 000	$7, 144, 000	31. 8
Maryland	120, 954, 000	18, 212, 900	15. 2
Virginia	119, 494, 000	14, 936, 300	12. 5
West Virginia	70, 512, 000	37, 727, 900	53. 5
North Carolina	182, 711, 000	55, 808, 500	30. 6
South Carolina	65, 010, 000	13, 081, 000	20. 0
Georgia	64, 045, 000	15, 659, 000	24. 4
Florida	98, 243, 000	24, 130, 200	24. 6
Total	743, 422, 000	186, 699, 800	25. 1
EAST SOUTH CENTRAL STATES			
Kentucky	$50, 324, 000	$7, 135, 800	14. 2
Tennessee	142, 159, 000	28, 693, 500	20. 2
Alabama	75, 189, 000	8, 758, 500	11. 6
Mississippi	111, 499, 000	37, 912, 400	34. 0
Total	379, 171, 000	82, 500, 200	21. 8
WEST SOUTH CENTRAL STATES			
Arkansas	$91, 279, 000	$50, 955, 700	55. 7
Louisiana	126, 946, 000	28, 571, 400	22. 5
Oklahoma	129, 977, 000	11, 547, 600	8. 9
Texas	356, 342, 000	96, 517, 900	27. 1
Total	704, 544, 000	187, 592, 600	26. 6
MOUNTAIN STATES			
Montana	$65, 229, 000	$9, 775, 800	14. 9
Idaho	61, 693, 000	19, 772, 200	32. 0
Wyoming	19, 128, 000	3, 745, 000	19. 6
Colorado	99, 645, 000	2, 000, 000	2. 1
New Mexico	25, 010, 000	2, 001, 100	8. 0
Arizona	44, 973, 000	18, 501, 000	41. 2
Utah	50, 041, 000	11, 785, 200	23. 6
Nevada	7, 861, 000	1, 438, 000	18. 3
Total	373, 580, 000	69, 018, 300	18. 5
PACIFIC STATES			
Washington	$169, 063, 000	$20, 547, 900	12. 1
Oregon	137, 177, 000	42, 891, 300	31. 2
California	519, 214, 000	101, 258, 000	19. 5
Total	825, 454, 000	164, 697, 200	19. 8
RECAPITULATION			
New England States	$548, 147, 000	$37, 182, 200	6. 8
Middle Atlantic States	2, 616, 186, 000	197, 514, 100	7. 5
East North Central States	1, 653, 499, 000	219, 405, 500	13. 3
West North Central States	851, 903, 000	77, 702, 400	9. 1
South Atlantic States	743, 422, 000	186, 699, 800	25. 1
East South Central States	379, 171, 000	82, 500, 200	21. 8
West South Central States	704, 544, 000	187, 592, 600	26. 6
Mountain States	373, 580, 000	69, 018, 300	18. 5
Pacific States	825, 454, 000	164, 697, 200	19. 8
Total	8, 695, 906, 000	1, 222, 312, 300	14. 0

[1] Complete data not available. [2] No data for local bonds.

WHITE JOINT FILLER FOR CONCRETE ROADS

TWO-YEARS' TRIAL SHOWS NEW MATERIAL HAS DECIDED VALUE

By L. G. CARMICK, Chemist, U. S. Bureau of Public Roads

TWO years' trial of a light-colored material developed by the Bureau of Public Roads for use as a filler for joints and cracks in concrete roads has demonstrated that the material has qualities which make it decidedly valuable for the purpose. Considering the lasting qualities of the material it has also been demonstrated that it is certainly not much more costly than other fillers, and it is possible that it will be found to be no more expensive.

In the early part of 1922 the Bureau of Public Roads began a search for some material of about the same color as Portland cement concrete that could be used for filling cracks and expansion joints in concrete roads. The bituminous materials which have always been used for this purpose are considered fairly satisfactory in most respects, but they unquestionably disfigure the road. A large number of different substances were experimented with and abandoned, which it is not worth while to mention here. However, after numerous failures a plastic material was found which seems to have most of the desired qualities.

The light-colored material fills the joints well, does not bleed, and has outlasted two fillings with asphalt

The cracks filled with asphalt have been refilled once and are now badly in need of further attention

CHARACTER OF THE MATERIAL

It is an intimate mixture of crude unvulcanized rubber with rosin in the proportion of one part of rubber to about ten of rosin, to which mixture is added a considerable quantity of a neutral white pigment, preferably barytes or titanox. Some difficulty was experienced at first in securing a homogeneous mixture of these materials, as rubber does not dissolve readily in melted rosin. But, eventually it was found that if the rubber were softened by allowing it to absorb about twice its weight of a light petroleum lubricating oil, and the mixture effected in a steam-jacketed iron vessel provided with revolving arms which exerted a mixing and kneading action, a smooth product could be obtained. This material differs in many respects from tar and asphalt. In color it is almost white, but it can be readily tinted to any desired shade. It is extremely sticky, especially when hot, and has a high degree of ductility. In these respects it resembles bituminous materials, but it is far less affected by temperature changes. It is more difficult to melt and even in the coldest winter weather met with in the vicinity of Washington it does not become brittle.

To use this material successfully these facts must be borne in mind. It may be melted in open kettles at the roadside, but the melting takes longer than with bitumens and care should be taken to prevent the material from catching fire and to avoid overheating. At a temperature of about 110° to 125° C. it is fluid enough to pour, and it does not become much more fluid even though the temperature be considerably raised, but the quality of the material is injuriously affected if heated much above this temperature. It was also found that it did not flow to the bottom of the cracks as readily as bituminous fillers, but this was overcome by the use of a hot iron implement, with which it can be worked down into even very narrow cracks. The tool used at present somewhat resembles a laundry iron, but with a handle long enough to enable a man to use it without stooping uncomfortably. Two of these are necessary, one being heated while the other is in use. As soon as the material is poured into the crack it is gone over with the hot iron and in this way very good results are obtained.

BEHAVIOR AND COST OF FILLER

This crack filler has been used experimentally in several of the States and the reports have been generally favorable. It has also been used in the Columbia Pike, an experimental road constructed by the Bureau of Public Roads near Washington. Frequent inspections of the material as used in this road, and comparison with the several types of bituminous fillers also used provide the basis for the statements made in this article. On the Columbia Pike the cracks were filled with the new material in December, 1922. A recent inspection shows them to be in good condition still. The cracks are well filled and the material has shown no tendency to flow out or "bleed." Traffic has done it no harm. The cracks that were filled after the men had learned how to handle the material are particularly good. They are inconspicuous and, in some cases, almost invisible. Cracks that were filled at the same time with asphalt had to be filled again in September 1923 and are now badly in need of further attention. The asphalt flows out causing an unsightly black streak in many cases several inches wide on each side of the crack.

The new material is inconspicuous and, in some cases, almost invisible

The asphalt filler has flowed out of the joints, forming unsightly black streaks several inches wide

This light-colored crack filler costs at present about 10 cents per pound which is, of course, much more than asphalt or tar, and furthermore it is not likely that its cost can be very greatly reduced. Also the labor cost of applying it is somewhat more than for asphalt and it was accepted as a fact in the beginning that it could not compete in price with bituminous fillers. Its use was felt justified, however, because in any case it would be a very small item in the cost of a first-class concrete highway, and the finer appearance of the road, due to the absence of black streaks was considered to be worth the additional cost.

However, in view of recent inspections it seems that its cost, taken over a period of years may not be so much greater than asphalt after all. For instance, cost data on the road mentioned show that filling cracks with asphalt cost $0.0075 per square yard of road for labor and material. The cost of the light-colored filler was about 1 cent per square yard for material and 1¼ cents for labor, the total being about three times as much as for asphalt. It is, therefore, apparent that if the light-colored material lasts three times as long as asphalt it will entail no greater expense. Experience so far seems to indicate that this will be the case.

PROTECTION OF CONCRETE AGAINST ALKALI

PROGRESS REPORT OF TWO SERIES OF TESTS BY U. S. BUREAU OF PUBLIC ROADS

WATER-GAS AND COAL TARS AND PARAFFIN TREATMENTS INVESTIGATED

By Dr. E. C. E. LORD, Petrographer, U. S. Bureau of Public Roads

TWO series of investigations by the United States Bureau of Public Roads directed toward the development of treatments for concrete to protect it against attack by alkali have yielded certain results which are worthy of notice, although the investigations are still in progress, and it is not yet possible to report definitely with regard to the relative success of the several treatments studied.

Only one form of treatment was tried in the earlier experiments, that being immersion in thin, crude water-gas tar.

The specimens employed in the first experiments consisted chiefly of sections of well seasoned, 6-inch concrete drain tile containing very little coarse aggregate, and ordinary tension and compression strength specimens made up of one part cement and three parts sand. In addition to these samples, concrete and mortar specimens were prepared for tar penetration determinations.

The tile sections, after thorough drying in air, were immersed in thin, liquid water-gas tar at room temperature for a few minutes until all air had been expelled, when they were removed from the bath and allowed to drain and dry out in air, or they were heated in the air bath at 110° C. for four hours. In some cases two treatments of this kind were given in order to assure a complete penetration of the tar. It was found that the maximum amount of tar absorbed in this way was from 7 to 8 per cent after draining for 24 hours, and about 4½ per cent when dried in the air bath.

The mortar specimens for tension and compression tests were treated in the same way, but allowed to remain in the tar for six days before drying in the air bath.

PROPERTIES OF THE WATER-GAS TAR

Tests of the tar used showed the following properties:

1. General characteristics		Very thin liquid
2. Specific viscosity (Engler)		3. 17 at 25° C.
		1. 98 at 40° C.
3. Total bitumen soluble in CS_2	Per cent	96. 99
4. Free carbon	do	0. 30
5. Inorganic matter	do	0. 05
6. Water	do	2. 66
7. Distillation (170° C.–300° C.)	do	42. 2
8. Residuum (semi solid)	do	57. 8
9. Specific gravity (25° C.)		1. 053
10. Specific gravity of distillate (25° C.)		0. 956

In order to estimate the penetrating value of this tar, 4-inch by 2-inch sand-mortar cylinders were prepared which, after curing for seven days in damp air and drying in the laboratory for six days, were placed in glass stoppered jars containing 25 cubic centimeters of tar. After one month it was found that the tar had penetrated uniformly upward to a height of from 2¾ to 3¾ inches above the original level of the tar.

EXPOSURE TESTS

Prior investigation by the Department of Drainage and Waters of Minnesota[4] in connection with a study of the effect of alkali on concrete drain tile, had indicated in some cases a loss from the cement of about 30 per cent lime and the formation of gypsum (hydrous sulphate of lime) in large quantities. In view of these results it was decided to expose the test specimens to a solution containing 1½ per cent sodium sulphate and 1½ per cent magnesium sulphate, which approximates an average maximum concentration of these salts in alkali soil waters, and determine the amount of lime passing into solution as a measure of alkali attack.

The tests were carried out as follows: Sections of tile, both treated and untreated and weighing approximately 2,200 grams, were placed in porcelain-lined jars containing 10,000 cubic centimeters of the alkali solution and kept well covered. Determinations of lime were made from time to time and the solutions renewed at frequent intervals. Table 1 shows the loss of lime after various periods of exposure and the total lime dissolved at completion of the tests.

TABLE 1.—*Solubility of tar-treated and untreated concrete drain tile in 3 per cent alkali solution*

Sample No.	Weight of sample	Method of treatment	Time exposed	Loss of CaO	Remarks
	Grams		*Weeks*	*Grams*	
I	2,297	Immersed twice in tar and dried at 110° C.	6	0. 56	
			10	1. 31	
			14	1. 65	
			18	1. 52	
			22	2. 45	
			26	2. 10	
			30	2. 18	
			38	2. 10	
		Total		2. 45	
II	2,198	Immersed once in tar and dried at 110° C.	7	0. 61	
			12	1. 87	
			16	0. 82	Solution renewed.
			24	1. 52	
			28	2. 35	
			32	2. 04	
			36	0. 67	Solution renewed.
		Total		4. 89	
III	2,360	Immersed twice in tar and dried at 25° C.	10	1. 30	
			14	0. 56	Solution renewed.
			18	1. 58	
			22	1. 40	
			26	1. 44	
			30	3. 06	
			34	2. 61	
			38	1. 42	
		Total		4. 36	
IV	2,171	Untreated	5	4. 45	
			10	4. 38	Solution renewed.
			17	4. 02	Do.
			22	3. 96	Do.
			26	1. 79	Do.
			30	2. 29	
			38	5. 40	
			42	4. 04	
			46	1. 08	Solution renewed.
		Total		23. 29	
V	2,281	Untreated	2	2. 40	
			3	2. 19	Solution renewed.
			4	1. 25	Do.
			6	0. 50	2 weeks in tap water.
			10	4. 18	Solution renewed.
			14	2. 76	Do.
			18	1. 13	Do.
			22	1. 44	
			26	2. 40	
			30	2. 25	
			34	0. 67	Solution renewed
			38	1. 81	
		Total		17. 49	

[4] Report of Concrete-Alkali Investigations in Minnesota. State of Minnesota, Department of Drainage and Waters, E. V. Willard, Commissioner, 1919–20, pp. 13–16.

It will be seen that the lime dissolved in the untreated samples (Nos. IV and V) does not increase progressively with the time exposed, but after reaching a maximum (about $4\frac{1}{2}$ grams in 5 weeks) appears to decline, owing possibly to the formation of insoluble calcium sulphoaluminate compounds. Comparing the total loss of lime in the untreated samples (IV and V) with that of the treated (Nos. I, II, and III) for the same length of time (38 weeks) it will be observed that the former have lost from 7 to 9 times as much lime as No. I, which received a double tar treatment, and from 3 to 5 times as much as the samples given but one treatment (No. II), or two treatments and dried at 25° C. (No. III). In other words, it may be said that the foregoing results indicate a 7 to 9 fold protection in the case of thoroughly impregnated material (I) and about 4 fold when insufficient tar was absorbed (II), or where the fixation of the tar residuum was incomplete before exposure to the alkali solutions (III).

TENSION AND COMPRESSION TESTS

Tension and compression tests were made on mortar treated with water-gas tar and on untreated material after remaining in 3 and 10 per cent alkali solutions and in tap water for 28 and 182 days. The results are given in Table 2.

It will be observed that while the untreated samples failed completely after 6 months in 3 per cent alkali solution (2), the treated specimens (3) gave higher values for tension and about the same for compression as those stored in tap water for the same length of time (1). Treated samples stored in 10 per cent alkali solution (6) showed a loss in tension of about 50 pounds as compared with untreated samples after 6 months in tap water (4), but were 224 pounds stronger than untreated samples (5) stored in 10 per cent alkali solution for the same length of time.

TABLE 2.—*Strength of tar-treated and untreated mortar after one and six months in water and alkali solutions*

Sample No.	Treatment	Storage solution	Tensile strength: 28 days	Tensile strength: 182 days	Compressive strength: 28 days	Compressive strength: 182 days
			Lbs. per sq. in.	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
1	Untreated	Tap water	412 435 397	360 362 362	1,925 1,995 1,575	2,540 2,480 2,830
	Average		415	361	1,852	2,627
2	Untreated	3 per cent alkali	230 220 235	Disintegrated.	1,908 2,300 2,230	Disintegrated.
	Average		228		2,143	
3	6 days in water-gas tar.	3 per cent alkali	390 495 568	592 490 568	2,610 2,340 2,462	2,312 2,432 3,000
	Average		484	550	2,471	2,581
4	Untreated	Tap water	463 400 406 442	402 418 396		
	Average		427	405		
5	Untreated	10 per cent alkali	480 390 417	184 135 70		
	Average		429	130		
6	6 days in water-gas tar.	10 per cent alkali	515 502 465 463	363 330 370		
	Average		486	354		

From these results it was concluded that water-gas tar if thoroughly incorporated in concrete offers, for a considerable period at least, an effective protection against the action of saline solutions.

SECOND SERIES OF TESTS

These preliminary investigations of the treatment of concrete with water-gas tar as a protection against alkali attack gave such encouraging results that the work was continued on a somewhat larger scale with coal-gas tar and paraffin in addition to water-gas tar as waterproofing materials.

The test pieces consisted of 4 by 6 inch cylinders made up of Portland cement, Potomac River sand, and gravel in the proportions of $1:1\frac{1}{2}:3$, $1:2:4$, and $1:3:6$, which, after curing for 28 days in damp sand, were allowed to dry out for at least two weeks before being treated.

TABLE 3.—*Action of alkali solution on concrete treated with tar and paraffin*

Sample No.	Treatment	Mix	Weight of samples	Loss of CaO in grams by months: 1	2	3	4	5	6	7	8	9	10	11	12
			Kilograms												
1	Untreated	1:1½:3	10.684	7.7	9.2	9.2	9.2	9.4	9.4	10.4	11.2	12.2	12.2	----	----
2	do	1:2:4	10.688	10.8	11.6	12.8	13.1	----	13.9	13.9	13.9	13.9	13.9	14.5	14.5
3	do	1:3:6	10.504	7.9	9.6	10.6	10.9	11.5	11.6	12.2	12.9	13.7	13.7	----	----
4	6 coats of water-gas tar	1:1½:3	10.640	3.8	4.8	5.5	5.9	6.3	7.5	8.0	8.6	9.2	9.2	----	----
5	do	1:2:4	10.872	7.5	8.6	10.3	10.3	----	11.0	11.6	11.6	11.8	11.8	12.8	12.8
6	do	1:3:6	10.504	5.6	6.8	7.5	7.8	7.8	7.8	9.1	9.4	9.8	9.8	----	----
7	10 coats of water-gas tar	1:1½:3	11.120	4.3	5.7	6.2	6.7	6.7	7.1	7.1	7.7	7.7	7.7	----	----
8	do	1:2:4	11.084	3.4	3.4	3.4	3.4	----	4.1	4.3	4.3	4.5	4.5	4.5	4.5
9	do	1:3:6	10.852	3.2	3.9	4.9	4.9	5.2	5.2	5.7	5.7	6.7	6.7	----	----
10	10 coats of water-gas tar 1 coat of coal-gas tar	1:1½:3	11.324	0.3	1.4	1.1	0.4	0.4	1.1	1.4	1.4	1.6	1.8	----	----
11	do	1:2:4	11.052	.0	.0	0.2	0.3	----	0.3	0.3	0.3	0.7	0.7	0.7	0.7
12	do	1:3:6	10.888	0.2	0.7	1.4	2.1	1.9	1.8	1.8	2.0	2.0	2.7	----	----
14	24 hours in paraffin-kerosene	1:1½:3	11.853	----	5.0	5.9	5.9	6.0	6.4	6.4	6.4	----	----	----	----
15	24 hours in paraffin-kerosene, 4 coats paraffin	1:1½:3	11.411	0.6	0.6	0.7	1.1	1.1	1.1	----	----	----	----	----	----
16	do	1:2:4	11.596	.0	0.6	0.8	1.1	1.1	----	----	----	----	----	----	----
17	do	1:3:6	11.416	1.2	1.2	1.2	1.2	----	----	----	----	----	----	----	----

Sample No.	Treatment	Mix	Weight of samples	Gain in weight (grams) by months: 1	2	3	4	5	6	7	8	9	10	11	12
			Kilograms												
1	Untreated	1:1½:3	10.684	304	300	340	340	332	332	332	336	336	348	---	---
2	do	1:2:4	10.688	400	416	404	428	---	444	440	436	424	428	432	416
3	do	1:3:6	10.504	424	416	452	452	476	472	440	464	436	444	---	---
4	6 coats of water-gas tar	1:1½:3	10.640	176	156	180	180	172	172	180	180	186	184	---	---
5	do	1:2:4	10.872	276	284	260	288	---	300	292	292	288	288	288	276
6	do	1:3:6	10.504	288	296	324	324	336	332	328	332	320	320	---	---
7	10 coats of water-gas tar	1:1½:3	11.120	112	120	148	148	140	140	148	148	148	156	---	---
8	do	1:2:4	11.084	148	164	160	184	---	192	192	188	184	184	184	176
9	do	1:3:6	10.852	172	184	212	212	232	236	228	244	232	232	---	---
10	10 coats of water-gas tar, 1 coat of coal-gas tar	1:1½:3	11.324	80	84	96	96	88	88	92	100	88	92	---	---
11	do	1:2:4	11.052	132	136	124	126	---	144	136	132	128	132	124	116
12	do	1:3:6	10.888	124	126	160	160	172	176	168	180	172	172	---	---
14	24 hours in paraffin-kerosene	1:1½:3	11.853	64	100	108	104	100	108	104	108	---	---	---	---
15	24 hours in paraffin-kerosene, 4 coats of paraffin	1:1½:3	11.411	56	64	60	68	72	72	---	---	---	---	---	---
16	do	1:2:4	11.596	104	112	128	124	124	---	---	---	---	---	---	---
17	do	1:3:6	11.416	152	180	176	188	---	---	---	---	---	---	---	---

Four cylinders of each batch were treated as follows: One was painted with 6 coats of crude water-gas tar; one was painted with 10 coats of crude water-gas tar; one was painted with 10 coats of crude water-gas tar followed by 1 coat of coal-gas tar thinned with benzol; and one was immersed for 24 hours in a 20 per cent solution of paraffin and kerosene after which it was painted with 4 coats of the same solution. Water-gas and coal-gas tars were applied at normal room temperature; for paraffin treatment the concrete and solution were maintained at a temperature above 82° F.

After receiving the above treatment four cylinders of each batch, together with four untreated specimens, were stored in porcelain-lined, covered cans containing 1,600 cubic centimeters of a 3 per cent solution of sodium and magnesium suphate. At monthly intervals the specimens have been removed from the sulphate bath, allowed to dry out at room temperature for 48 hours, and weighed, and the gain in weight and loss in lime determined. This procedure will be continued until evidence of failure is noticeable in the untreated samples, when all cylinders will be tested for compressive strength and the broken material will be examined microscopically and chemically.

In Table 3 the results are recorded at monthly intervals extending over a period of 10 and 12 months' exposure to the action of the alkali. It will be observed that the effect of the tar treatment is similar for all concrete mixtures and that samples Nos. 10, 11, and 12 receiving 10 coats of water-gas tar and 1 coat of coal-gas tar indicate a minimum loss in lime and increase in weight. This is especially noticeable in the 1:2:4 mixtures (Nos. 2, 5, 8, and 11) where the loss in lime in 12 months is 14.5 grams for untreated samples (No. 2) and only 0.7 gram for samples treated with water-gas and coal-gas tars (No. 11), while the gain in weight, through absorbed moisture and secondary salts, is 416 grams for the untreated and only 116 grams for the sample treated with the two tars after 12 months' exposure to the alkali solution.

In regard to the paraffin treatment, it will be noted that samples from all batches receiving 4 coats of the paraffin solution after saturation (Nos. 15, 16, and 17) indicated about the same resistance to alkali attack as the tar-treated samples (Nos. 10, 11, and 12), whereas samples immersed for 24 hours in the paraffin bath but receiving no surface application of paraffin (No. 14) suffered an appreciable loss in lime after 8 months' exposure to the action of the alkali solution. The fact that gain in weight is not correspondingly large may indicate that the action which is taking place is confined largely to the surface of the specimens.

EFFECT OF ALKALI ON STRENGTH OF MORTAR

By CHAS. E. PROUDLEY, Junior Assistant Testing Engineer, U. S. Bureau of Public Roads

TESTS of mortar briquets made in the laboratory of the United States Bureau of Public Roads to determine to what extent the presence of alkali in mixing water is harmful indicate that—

1. The presence of alkaline sulphates in the mixing water in quantities of less than 1½ per cent has no serious effect on the strength or other physical properties of Portland cement mortar.
2. One per cent alkaline solutions in contact with cement mortar in mixes of 1:2 and leaner cause progressive decrease in the strength of the mortar.
3. The decrease in strength of mortar subjected to alkali action may be somewhat accelerated by the presence of considerable quantities of alkali in the mixing water, and for this reason the use of pure water in mixing is advisable under such conditions.

The specimens used in the tests were mortar briquets made with Ottawa sand and a good commercial brand of Portland cement. The alkali used had the following analysis:

	Per cent.
$CaCl_2$ (calcium chloride)	4. 50
$MgCl_2$ (magnesium chloride)	11. 20
$MgSO_4$ (magnesium sulphate)	3. 90
Na_2SO_4 (sodium sulphate)	80. 40
	100. 00

This represents the average analysis of a number of alkaline waters in which sulphates are the principal salts.

Figure 1 shows strength at various ages of specimens made with mixing water containing various percentages of alkali up to 5 per cent. The specimens were stored in water in the usual manner after the initial 24 hours in the moist closet. In a period of one year the use of the alkaline water has no objectionable effect; in fact, the specimens in which it is used have a slightly higher strength than the standard briquets.

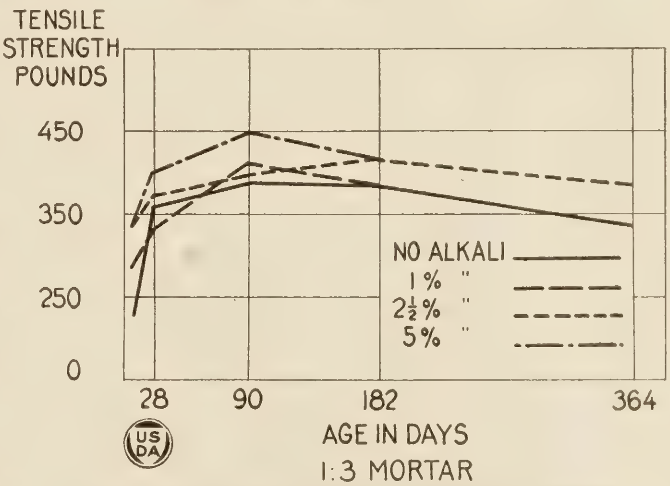


FIG. 1.—Effect of alkaline mixing water on tensile strength of mortar

It is noticeable that more than 2½ per cent alkali in the water does not have a correspondingly greater effect upon the strength and for this reason later tests were made using less alkali. Furthermore, analyses[3] of water from rivers, lakes, springs, and wells from every part of the country show that less than 1 per cent of the waters examined contain more than 1.5 per cent alkaline salts by weight. In fact, comparatively few contain more than 0.5 per cent of dissolved solids.

The results of the tests to determine the effect of alkali in contact with the specimens are shown in Figure 2. The curves shown at the top of the figure represent specimens stored under usual conditions. Below these are the curves of specimens cured in soil

[3] U. S. Geological Survey, Water Supply Paper 364.

(Continued on page 27)

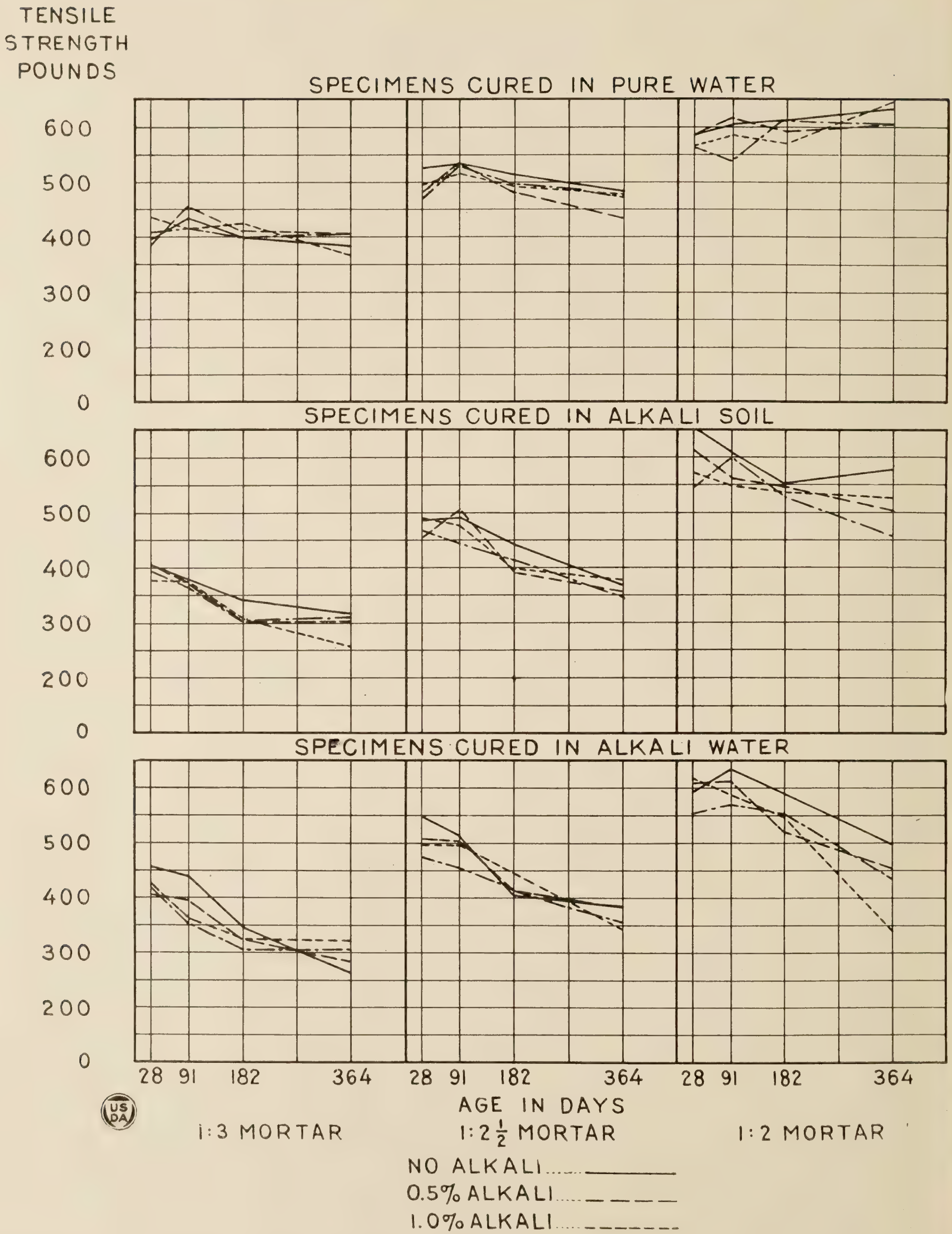


FIG. 2.—Effect of alkali in contact with specimens on their tensile strength

ROAD MATERIAL TESTS AND INSPECTION NEWS

UNIFORMITY OF CONCRETE

Information gathered recently in a number of States strikingly demonstrates the nonuniformity of concrete used in pavements. The information consisted of the results of compression tests on cores drilled from pavements and it can be safely stated that the situation disclosed is reflected in general by concrete pavements throughout the country.

For example, in one of the Eastern States the extreme range of strengths varied from 34 to 174 per cent of the average strength. This occurred with drilled cores which were five months old at the time of test. The minimum range for any particular age was from 86 to 121 per cent of the average strength. One of the Middle Western States reported on several projects an average strength of 4,417 pounds per square inch with an average minimum of 2,965 and an average maximum of 6,161 pounds per square inch.

Reflecting on the significance of this situation it may be pertinent to inquire which would be preferred, a pavement which has a range in strength from 2,000 to 4,000 pounds per square inch or one which has a range, say, from 2,700 to 3,300 pounds per square inch? The answer to this question is largely one of design. In the former case it is not practicable to take advantage of the higher or average strengths because there are parts of the pavement which yield only 2,000-pound concrete. With the more nearly constant strength, the design can be based with some degree of confidence on 3,000-pound concrete.

In other words, it is desirable from both economic and practical standpoints that the construction of concrete pavements be so controlled that the properties of the concrete will be uniform. If this uniformity is obtained, the design can be based on concrete of predetermined strength and the factor of safety required to cover the variations in the quality of the concrete can be materially reduced.

It is chiefly with the hope of controlling the construction in such a manner as to give this much-desired uniformity of the concrete that the United States Bureau of Public Roads proposes to try out an improved method of control on an actual paving project.

COMPRESSION TESTS OF CEMENT

The Bureau of Public Roads has begun a series of tests in cooperation with committee C1 on cement, of the American Society for Testing Materials for the purpose of studying further the possibility of substituting a compression test for the standard tension test of cement. The study will include comparative tests using 2-inch by 4-inch cylinders, 2-inch by 2-inch cylinders and 2-inch cubes in addition to the regulation tension briquettes as well as tests in which consistencies other than normal are employed. A novel feature of the work lies in the fact that the flow table will be used for measuring the consistency of the mortar. From the results it is hoped to determine whether or not the flow table may be used for the determination of consistency in routine testing. Full details including working drawings of the flow table will be sent to anyone interested on request.

COOPERATIVE TESTS OF PAVING BRICK

A study of the standard rattler test for paving brick has been undertaken by the Bureau of Public Roads in an effort to determine the effect of size of brick on the percentage of loss. The work is being done at the request of and in cooperation with the Committee on Brick of the American Society for Testing Materials. Other cooperating agencies are the Rensselaer Polytechnic Institute, Troy, N. Y., and the Department of Public Works of the city of Buffalo. Testing engineers have realized for some time that the so-called 3-inch brick may be somewhat penalized when tested under the specifications as to minimum allowable rattler loss which are used for the larger sizes. Although a charge of 3-inch brick weighs considerably less than a corresponding charge of 4-inch brick, the total length of edge exposed to wear is not proportionately smaller. Due to the fact that the loss is computed on the basis of percentage of original weight and also to the fact that by far the greatest amount of wear in the test comes on the edges of the brick, it will be seen that quite an error may possibly be introduced. Many specifications have recognized this fact by providing for a differential of 1 or 3 per cent loss in the case of the smaller size. These figures are, however, for the most part arbitrary and not based on test data. The tests which are to be made by these three laboratories should throw some light on the question as to just what this differential should be.

EFFECT OF ALKALI ON STRENGTH OF MORTAR

(Continued from page 25)

saturated with a saturated solution of alkali and the lower set is for specimens stored in a 1 per cent solution of alkali.

Referring to the curves in Figure 2, it will be noted that alkali in the mixing water in amounts of 1½ per cent or less has an immaterial effect upon the strength, although a tendency toward slightly lower strength in this series may be noticed. This may be due to a number of causes such as a slightly lower density resulting from the presence of the alkaline salts in some chemical combination with the cement or some of its constituents, or lower cementing qualities due to this chemical action alone. Where the specimens have been stored in alkali, it seems that the presence of alkali in the mixing water assists disintegration perceptibly. Inasmuch as it would seldom be necessary to use alkaline mixing water unless the concrete would later be subjected to contact with alkali, this point is worthy of note.

Time of set and soundness tests on cement pats mixed with alkaline waters of various strengths show that neither time of set nor soundness is altered to any appreciable extent by the presence of alkali in the mixing water.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

APRIL 30, 1924

STATES	FISCAL YEARS 1917-1923			FISCAL YEAR 1924									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1923			PROJECTS COMPLETED SINCE JUNE 30, 1923			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 3,789,011.74	$ 1,794,703.21	367.9	$ 528,287.87	$ 257,160.91	66.2	$ 13,920,566.31	$ 6,949,200.70	853.4	$ 652,035.75	$ 278,708.11	4.4	$ 431,137.51	Alabama
Arizona	6,518,025.52	3,195,689.68	372.1	1,194,453.52	727,736.97	104.8	2,262,520.85	1,323,211.35	173.4	825,736.07	504,607.28	45.0	691,452.16	Arizona
Arkansas	8,623,205.72	3,308,572.12	736.2	2,318,145.05	1,045,541.62	132.3	4,256,316.15	1,732,591.09	223.7	3,515,723.24	845,234.70	215.6	871,603.40	Arkansas
California	8,495,547.60	3,538,967.90	340.5	4,145,805.72	1,962,804.58	187.0	11,613,685.98	6,111,198.17	414.7	540,834.58	379,386.15	25.3	2,635,958.42	California
Colorado	6,470,695.56	3,136,236.67	426.6	1,189,833.78	651,192.83	58.7	4,400,763.35	2,398,531.33	169.7	250,933.76	140,822.34	11.5	1,871,613.77	Colorado
Connecticut	2,568,643.95	1,026,148.85	60.7	128,610.43	64,305.21	3.8	2,655,321.05	986,119.24	49.2	755,751.37	222,495.20	12.2	606,612.59	Connecticut
Delaware	2,196,610.22	621,154.83	45.2	718,598.97	317,815.00	22.7	988,468.80	389,767.50	27.5	76,668.90	31,961.10	0.2	13,206.40	Delaware
Florida	69,466.31	29,700.63	15.6	604,477.52	288,175.17	23.3	8,578,162.86	4,232,785.06	255.5	1,037,874.30	440,221.40	13.7	408,668.22	Florida
Georgia	13,960,499.47	6,363,703.60	925.5	2,965,428.24	1,472,390.06	274.9	7,865,396.47	3,809,568.64	672.1	1,537,997.01	692,146.45	102.8	129,065.26	Georgia
Idaho	6,880,673.73	3,272,111.01	426.1	1,141,068.96	727,522.09	74.2	1,404,050.80	826,531.30	96.2	727,191.21	408,930.73	54.0	505,918.86	Idaho
Illinois	25,211,526.23	11,414,291.96	743.5	768,512.90	379,214.16	29.2	14,394,340.81	7,160,290.35	485.9	1,250,965.78	616,070.33	36.9	662,757.21	Illinois
Indiana	5,513,487.76	2,668,277.55	157.0	1,945,251.03	928,993.26	64.6	12,486,232.40	6,121,359.53	411.3	2,109,233.75	1,032,191.18	81.4	1,621,667.16	Indiana
Iowa	16,040,079.62	6,056,136.48	1,059.4	6,952,470.02	3,097,340.58	600.5	6,061,281.68	2,832,857.08	549.8	2,703,571.60	1,228,200.00	101.7	43,354.53	Iowa
Kansas	12,216,426.90	4,246,436.34	349.7	4,392,131.87	1,602,266.68	129.9	16,048,072.99	5,925,834.08	548.1	2,511,965.23	1,058,678.20	115.1	382,841.66	Kansas
Kentucky	6,620,289.80	2,789,991.15	285.0	4,081,627.63	1,768,710.23	146.9	7,316,679.52	3,535,492.32	322.1	471,726.68	235,863.33	19.0	630,097.52	Kentucky
Louisiana	7,131,394.90	3,157,128.03	590.4	1,116,208.52	367,989.39	54.2	5,682,810.05	2,729,308.82	376.8	16,405.36	8,202.68	0.2	7,511.49	Louisiana
Maine	4,662,854.19	2,242,963.03	159.8	2,248,204.59	1,056,972.35	70.9	907,304.26	451,804.19	34.8	713,577.40	354,204.36	34.0	297,574.71	Maine
Maryland	5,309,394.94	2,524,843.64	185.0	1,427,449.75	676,878.38	57.3	1,601,912.79	669,855.76	52.0	234,966.08	10,000.00	6.2	131,427.32	Maryland
Massachusetts	6,841,975.16	2,802,957.88	163.7	1,967,316.05	767,062.11	40.7	5,120,287.44	1,849,601.03	96.5	782,886.11	257,589.00	12.0	1,132,763.76	Massachusetts
Michigan	9,528,041.53	4,343,817.14	363.4	3,051,384.48	1,305,637.89	106.5	10,448,363.51	5,102,961.38	402.9	3,200,506.31	1,554,684.84	84.7	1,345,846.02	Michigan
Minnesota	19,828,671.37	8,004,277.87	1,893.4	4,208,889.87	1,881,565.20	398.6	7,366,883.13	3,020,954.43	526.8	439,512.73	205,000.00	66.0	85,714.94	Minnesota
Mississippi	4,746,771.31	2,284,557.86	462.4	2,570,658.55	1,260,281.77	143.4	8,352,228.83	4,168,183.90	532.7	399,842.08	176,586.34	16.0	347,291.48	Mississippi
Missouri	5,272,757.08	2,358,848.60	365.6	5,562,471.34	2,673,920.07	383.5	14,148,945.38	6,813,519.37	618.1	1,772,165.49	821,039.81	103.3	2,849,374.40	Missouri
Montana	7,935,185.30	3,896,053.50	711.5	803,232.04	420,005.19	62.0	1,832,052.67	1,287,543.63	194.1	1,236,344.25	702,460.16	140.6	3,115,871.33	Montana
Nebraska	3,159,614.47	1,449,675.04	475.6	4,210,786.91	2,014,231.26	858.5	5,854,159.94	2,824,263.01	661.1	1,951,094.47	970,367.24	182.7	2,615,254.11	Nebraska
Nevada	3,049,109.29	1,562,111.35	203.6	411,136.23	291,513.63	22.0	3,994,660.18	3,436,013.46	343.5	446,060.81	341,009.43	10.7	312,049.88	Nevada
New Hampshire	2,336,997.00	1,129,004.69	140.5	739,753.19	358,862.89	30.8	591,313.98	279,768.77	21.5	265,029.38	119,644.07	7.4	182,058.57	New Hampshire
New Jersey	5,423,362.96	1,975,783.52	112.1	1,794,735.56	562,527.10	29.4	8,552,430.81	2,153,320.04	49.0	1,669,599.11	538,823.99	19.1	422,379.32	New Jersey
New Mexico	4,706,909.12	2,433,663.68	617.1	599,377.33	325,186.00	97.2	4,896,222.57	3,100,904.09	583.4	1,600,613.04	1,079,939.15	104.8	464,110.20	New Mexico
New York	9,826,633.82	4,356,659.70	290.6	6,978,203.72	2,944,367.53	222.8	22,547,526.96	8,929,161.47	526.1	5,505,000.00	2,005,425.00	119.3	4,807,428.44	New York
North Carolina	11,770,363.76	5,325,763.37	826.8	797,369.21	350,994.29	57.9	8,864,391.81	3,148,515.67	237.7	2,512,967.10	1,154,324.36	68.6	617,407.15	North Carolina
North Dakota	5,142,157.55	2,492,599.09	792.5	3,660,544.68	1,812,626.17	770.0	2,127,951.19	1,026,483.84	334.7	853,070.52	426,535.21	140.0	1,426,703.56	North Dakota
Ohio	22,840,050.09	7,966,325.17	662.8	9,618,128.44	3,666,542.82	281.0	11,612,728.70	4,400,022.84	315.7	2,549,123.62	862,626.05	80.5	428,842.43	Ohio
Oklahoma	9,795,634.54	4,417,788.46	371.5	3,130,150.61	1,440,648.45	124.3	7,822,145.92	3,787,990.79	352.1	1,571,115.56	733,123.45	74.5	403,962.14	Oklahoma
Oregon	10,391,362.48	4,782,736.38	534.1	1,691,510.69	1,036,357.41	121.5	2,662,216.36	1,416,202.87	144.0	138,208.95	65,675.96	17.5	28,356.23	Oregon
Pennsylvania	33,090,029.45	12,615,058.79	652.8	3,225,176.88	1,328,845.47	68.3	7,181,910.81	2,424,402.50	126.3	11,829,122.36	3,381,970.00	216.8	1,485,383.03	Pennsylvania
Rhode Island	1,484,075.35	647,634.96	36.6	290,321.90	131,593.00	7.4	1,087,787.06	411,759.49	22.7				376,428.68	Rhode Island
South Carolina	6,181,485.41	2,885,754.37	655.1	1,626,786.06	739,184.34	247.7	6,685,566.58	2,640,586.98	503.6	647,463.82	203,684.75	46.0	164,307.39	South Carolina
South Dakota	4,758,907.24	2,320,007.33	534.5	3,579,964.08	1,767,665.78	432.6	5,221,804.03	2,681,558.80	645.9	1,066,475.00	507,154.06	110.3	233,149.85	South Dakota
Tennessee	3,210,970.94	1,535,762.18	112.5	2,064,853.17	1,019,402.48	87.6	13,827,431.86	6,724,982.26	447.6	2,099,483.57	800,842.73	62.5	314,906.38	Tennessee
Texas	29,172,501.06	11,094,069.77	2,259.6	11,347,463.51	4,293,943.98	767.3	24,943,423.31	9,941,857.22	1,548.0	4,252,173.84	1,667,637.19	251.0	316,535.08	Texas
Utah	2,175,007.41	1,196,437.68	116.6	1,030,357.45	627,434.33	98.9	3,399,600.07	2,101,299.95	227.3	860,662.07	635,335.20	51.4	708,223.94	Utah
Vermont	1,370,715.63	669,433.75	53.1	551,398.53	273,336.37	21.3	1,144,973.83	551,274.77	34.6	363,118.12	173,388.64	8.9	500,921.20	Vermont
Virginia	6,823,928.36	3,279,029.56	402.5	2,690,547.72	1,262,704.51	134.7	8,860,281.55	4,078,223.14	325.3	551,161.37	130,773.51	18.3	393,659.73	Virginia
Washington	10,356,980.82	4,819,105.70	419.8	618,239.13	273,551.37	35.3	2,354,076.03	1,137,800.00	97.1	1,059,358.60	474,900.00	36.2	98,012.76	Washington
West Virginia	4,100,225.73	1,800,932.41	206.3	1,389,522.22	664,109.12	49.3	4,740,449.70	2,084,116.50	156.8	574,510.20	260,790.10	25.1	245,908.40	West Virginia
Wisconsin	15,504,691.39	5,996,989.24	1,077.6	2,671,162.12	1,177,011.47	208.9	3,204,764.79	1,526,918.42	150.2	109,324.13	54,662.06	9.3	3,045,269.49	Wisconsin
Wyoming	4,601,439.36	2,210,774.20	534.8	1,187,011.02	669,291.38	123.6	4,078,154.44	2,531,599.30	272.9	417,005.83	168,594.52	48.8	150,719.47	Wyoming
TOTALS	407,704,641.14	174,044,673.82	23,297.2	122,155,059.06	54,693,411.85	8,194.4	335,968,620.56	153,738,096.42	16,215.4	70,656,176.51	28,962,510.36	3,011.5	40,561,307.55	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 49,154,309.22 Federal aid $ 21,750,040.45 Miles 2,289.9

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

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

REPORTS

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

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
*347. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370. The Results of Physical Tests of Road-Building Rock. 15c.
386. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
*389. Public Road Mileage and Revenues in the Central, Mountain, and Pacific States, 1914. 15c.
390. Public Road Mileage in the United States, 1914. A Summary.
*393. Economic Surveys of County Highway Improvement. 35c.
407. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463. Earth, Sand-Clay, and Gravel Roads. 15c.
*532. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*555. Standard Forms for Specifications, Tests, Reports, and Methods of Sampling for Road Materials. 10c.
583. Reports on Experimental Convict Road Camp, Fulton County, Ga.
*586. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1916. 10c.
*660. Highway Cost Keeping. 10c.
670. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
*691. Typical Specifications for Bituminous Road Materials. 15c.
*704. Typical Specifications for Nonbituminous Road Materials. 5c.
*724. Drainage Methods and Foundations for County Roads. 20c.
*949. Standard and Tentative Methods of Sampling and Testing Highway Materials, Recommended by the Second Conference of State Highway Testing Engineers and Chemists, February 23 to 27, 1920, 25c.
*1077. Portland Cement Concrete Roads. 15c.
*1132. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.

DEPARTMENT CIRCULAR

No. 94. TNT as a Blasting Explosive.

FARMERS' BULLETINS

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

SEPARATE REPRINTS FROM THE YEARBOOK

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

OFFICE OF PUBLIC ROADS BULLETIN

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

OFFICE OF THE SECRETARY CIRCULARS

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

Public Roads Vol. III, No. 25. Automobile Registrations, Licenses, and Revenues in the United States, 1919.
Vol. III, No. 29. State Highway mileage, 1919.
Vol. III, No. 36. Automobile Registrations, Licenses, and Revenues in the United States, 1920.
Vol. IV, No. 5. Automobile Registrations, January 1 to July 1, 1921.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

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

---

*Department supply exhausted.

THE FEDERAL AID HIGHWAY SYSTEM

THE Federal highway act, approved November 9, 1921, provided for the selection by the State highway departments of a system of highways not to exceed 7 per cent of the total highway mileage of each State. Upon this system, all apportionments of Federal aid are to be expended. The system is to be divided into two classes, primary and secondary, of which the former are not to exceed three-sevenths of the total, the remainder to be of the latter class. The Secretary of Agriculture was given authority to approve in whole or in part the systems as designated by the State highway departments or to require modifications or revisions thereof.

THE total mileage of existing highways certified by the States was 2,866,061 miles. The States designated by maps and route description, systems of main roads totaling in mileage not more than 7 per cent of the certified mileage. The systems for groups of adjoining States were reviewed by representatives of the States and of the Bureau of Public Roads meeting in a series of conferences for the principal purpose of connecting the systems at State lines. In this way the entire system was coordinated and recommended to the Secretary of Agriculture by the Bureau of Public Roads. The system as approved by the Secretary of Agriculture and represented by the map published November 1, 1923, includes 168,881 miles which is 5.9 per cent of the certified mileage.

UP TO March 1 the Federal-aid highways which had been completed since the passage of the Federal-aid road act in 1916 totaled 33,036 miles, and 13,800 miles were under construction and reported as 59 per cent complete. The total of roads completed and under construction amounted therefore to 46,836 miles. Of the mileage reported as completed on February 29, 6,307 miles had been completed during the current fiscal year. All but a very small percentage of this mileage is on the Federal-aid highway system as now established.

IN ADDITION to the roads of the system improved with Federal aid, parts of it have been improved without Federal assistance. A careful study is being made of the improvement status of the system and an approximate estimate based upon these incomplete studies is that at the end of the year there were about 60,000 miles of surfaced roads and 8,700 miles graded, which leaves nearly 110,000 miles yet to be surfaced.

TO BRING this system up to serviceable standards, therefore, within the full decade ahead, will mean a surfacing program of about 11,000 miles for each of the 10 years; this in addition to the additions to the system, the separation of grade crossings, reconstruction, and much other work necessary.