U.S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

Public Roads

VOL. 4, NO. 2 WASHINGTON, D. C. JUNE, 1921

PENNSYLVANIA FEDERAL-AID PROJECT NO. 29—AN ASPHALT CONCRETE ROAD

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1921

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 the planning or construction of 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 15 cents for a single number or $1.50 per year to the Superintendent of Documents, Government Printing Office, Washington, D. C.

U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

PUBLIC ROADS

TABLE OF CONTENTS

THOMAS H. MacDONALD . . . . Chief of Bureau
P. ST. J. WILSON . . . . Chief Engineer
H. S. FAIRBANK . . . . Editor

ADDITIONAL COPIES

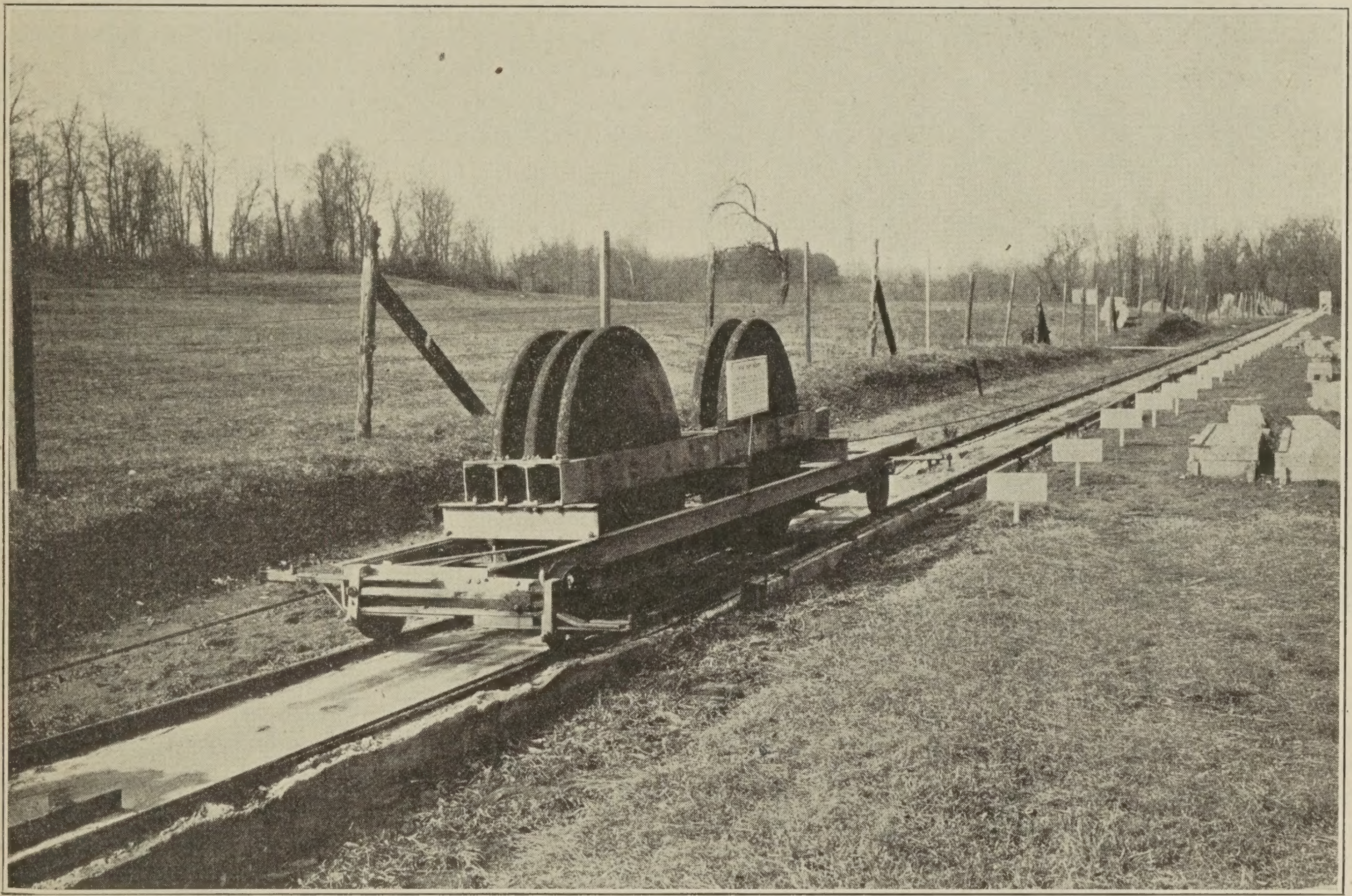
OF THIS PUBLICATION MAY BE PROCURED FROM
THE SUPERINTENDENT OF DOCUMENTS
GOVERNMENT PRINTING OFFICE
WASHINGTON, D. C.

AT

15 CENTS PER COPY
SUBSCRIPTION PRICE, $1.50 PER YEAR

ACCELERATED WEAR TESTS BY THE BUREAU OF PUBLIC ROADS.

By F. H. JACKSON, Senior Assistant Testing Engineer, and C. A. HOGENTOGLER, Highway Engineer.



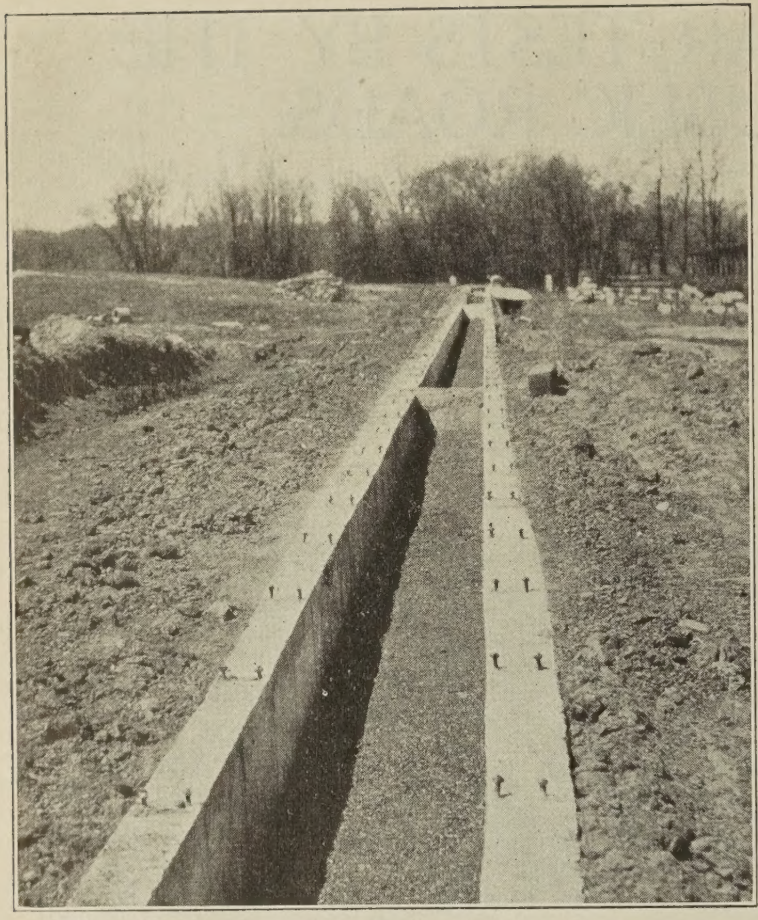
THE TESTING MACHINE ON THE RUNWAY.

THE Bureau of Public Roads has recently completed at the Arlington (Va.) Experiment Station a series of accelerated wear tests upon granite-block, vitrified-brick, and concrete pavement surfaces, the results of which should prove of interest to engineers and others engaged in the use, selection, or manufacture of materials for these types of pavement.

The twofold purposes of the tests were: First, to compare the behavior of various forms of the several types when subjected to exceptionally heavy steel-tired traffic; and, second, to ascertain whether the resistance to wear of the constituent parts of the several pavement types, as determined by laboratory tests, may be considered as a reliable index of the wearing value of these materials when combined in a pavement.

For the purpose of regulating the quality of the various materials used in granite-block, brick, and concrete pavements it has long been customary to specify certain controlling requirements based upon laboratory tests. For granite block the tests selected were the usual tests for toughness and abrasion of rock developed many years ago as an aid to the selection of stone for use in macadam road construction. For brick roads the controlling test is the rattler test, and for concrete the coarse aggregate has been tested for abrasion and the mortar has been tested for tensile strength, and, lately, there have been added compression tests on samples of the concrete.

Because the tests for abrasion and toughness, when applied to granite block, were not sufficiently sensitive for the purpose engineers have been inclined to ignore the test results and rely on their experience with certain grades of block with which they happen to be familiar. Needless to say this practice has led to discrimination against certain quarries, merely for the reason that material from them had never been used by the particular engineer controlling the selection. In a previous



CONCRETE CURBS WITH CINDER FILL IN PLACE.

paper by one of the authors[1] there is described a modified abrasion test which seemed to meet the requirements in so far as a satisfactory laboratory test is concerned, but the results obtained are of no value to the specification writer unless they are correlated with service tests.

Although the tests in use for brick and concrete have given greater satisfaction, the fact remains that they fail to take into account a number of factors involved in the use of the materials in the road surface. Their value depends entirely upon the accurate correlation of the test results with the behavior of the material under the test of actual service. To obtain preliminary information along this line in a brief space of time was the principal purpose of the Arlington experiments.

DESCRIPTION OF THE TEST SECTIONS.

In conducting the tests the aim has been to simulate as closely as possible the conditions of actual pavement use of the materials. The various materials were incorporated into a number of pavement sections laid in the form of a runway approximately 400 feet long by 2 feet wide. In all there were 48 sections, 21 of which were brick, 19 granite block, and 8 concrete. The sections were constructed in the following manner: A trench 36 inches wide and 30 inches deep was excavated the full length of the proposed runway. Two 6 by 30 inch reinforced concrete curbs were then constructed the full length of the trench to form the edges of the pavement sections and to support the rails which were laid to guide the testing machine. Between these curbs, which were spaced 2 feet apart, cinders were placed in layers and thoroughly compacted by hand tamping to a depth of 16 inches under the granite-block sections, about 17½ inches under the brick, and 24 inches under the concrete. A 4-inch clay drain-tile was placed longitudinally down the center of the trench under the cinder fill, with laterals at various points, for the purpose of preventing the accumulation of water in the foundation. The base for the granite-block and brick sections was 8 inches of 1:3:6 concrete laid upon the compacted cinder fill. The concrete sections were laid immediately upon the cinders. Over the base in the granite-block sections were laid four different 1-inch bedding courses: (*a*) 1:4 dry cement mortar, (*b*) sand, (*c*) asphalt-sand, and (*d*) tar-sand. Upon these beds the blocks were laid by an expert paver in strict accordance with the best practice and in such manner that after thorough ramming their heads were flush with the top of the curbs. The brick were similarly laid on sand and sand-cement cushions. All brick were acceptable from the standpoint of visual inspection, as were also all the granite block except the samples of Georgia block used in sections 7 and 8. These block were rough when received, which made it impossible to pave them to as smooth a surface as the other sections. Both granite block and brick were filled in the various sections severally with 1:1 cement grout, asphalt, tar, and asphalt and tar mastics.

Mechanical analyses of the sands used are given in Table 1. Analyses of the asphalt and tar fillers are shown in Table 2. Cement grout was mixed in a grout box similar to those used in practice. Bituminous fillers were heated in a portable kettle and were poured by means of an ordinary pouring pot. The asphalt was heated to a temperature of approximately 350° F. The mastics were prepared by mixing the separately heated sand and bituminous materials in a wheelbarrow with hoes in a manner similar to the method which has been used in New York City. It was found that the maximum amount of sand which could be carried in the mastic and still insure proper application was about 40 per cent by volume.

TABLE 1.—*Mechanical analysis of sand used in wear test.*

Total retained on—	No. 1 sand.	No. 2 sand.
	Per cent.	*Per cent.*
¼-inch sieve	2	
No. 10 sieve	19	
No. 20 sieve	35	7
No. 30 sieve	50	23
No. 40 sieve	67	51
No. 50 sieve	78	68
No. 80 sieve	92	91
No. 100 sieve	94	95
No. 200 sieve	99	99

No. 1 sand was used as fine aggregate in concrete base and in sand-cement bed.
No. 2 sand was used in cement grout, bituminous-sand fillers and bituminous-sand bed.

[1] The Standard DeVal Abrasion Test for Rock, by F. H. Jackson, Eng. Cont., Mar. 3, 1920.

PLATE 1. WEAR TESTS OF GRANITE BLOCK—1,650 RUNS.

TABLE 2.—*Analysis of refined coal-tar filler.*

General characteristics: Semisolid.		
Water		1.5
Specific gravity, 25° C./25° C.		1.2389
Float test, 50° C.	seconds	260
Total bitumen soluble in carbon disulphide	per cent	81.5
Free carbon, organic matter insoluble	do	18.2
Inorganic matter insoluble	do	.3

Distillation.

Fractions.	Character.	Per cent by volume.	Per cent by weight.
170° C.	Liquid	2.0	1.02
170° C. to 235° C.	do	1.5	.97
235° C. to 270° C.	do	4.0	2.62
270° C. to 300° C.	¾ solid	4.0	2.65
Residue	Soft pitch	88.5	92.58
		100.0	99.84

ANALYSIS OF OIL-ASPHALT FILLER.

General characteristics: Semisolid.		
Specific gravity, 25° C./25° C.		1.026
Flash point (° C.)		245
Penetration, 0° C., 200 grams, 60 seconds		22
Penetration, 25° C., 100 grams, 5 seconds		35
Melting point (° C.)		69
Loss, 163° C., 5 hours	per cent	.042
Characteristics of residue: Smooth.		
Consistency of residue: Penetration, 25° C., 100 grams, 5 seconds		25
Total bitumen (soluble in carbon disulphide)	per cent	99.8
Organic matter insoluble	do	.2
Inorganic matter insoluble	do	.0

THE TESTING MACHINE.

The wear-testing machine consists of five cast-iron wheels, 48 inches in diameter by 2 inches wide, and each weighing 1,000 pounds. This gives a unit wheel load of 500 pounds per inch width of tire. The wheels are mounted inside a channel-iron frame in such a way that they roll over the center 12 inches of the 24-inch test strips. Each wheel is mounted independently of the others, so as to be free to move up and down and thus adjust itself to any inequalities and depressions in the pavement along the line over which it travels. It will therefore be seen that the effect approximates very closely the action produced by heavily loaded steel-tired trailers or horse-drawn vehicles. The machine is pulled back and forth over the test sections by means of an endless steel cable driven by a 30-horsepower gasoline engine and travels at the rate of approximately 5 miles per hour.

THE PROGRESS OF THE TEST.

The actual testing was begun November 13, 1919, and continued intermittently until August 1, 1920, when, because of the complete failure of some of the sections, further running of the apparatus was impracticable. By this intermittent running an opportunity was afforded for study of the behavior of sections under test in winter and summer as well as in wet and dry weather.

During the entire course of the test detailed observations and notes, as well as photographic records, were made of the behavior of the various sections. The progress of the test is shown by the following table giving the dates and number of runs of the car.

THE CONCRETE BASE IN PLACE.

Date.	Number of runs.	Date.	Number of runs.
Nov. 13, 1919	125	May 20, 1920	3,225
Dec. 4, 1919	625	May 27, 1920	3,750
Feb. 14, 1920	1,175	June 15, 1920	4,650
Mar. 26, 1920	1,650	June 22, 1920	5,000
Apr. 14, 1920	2,150	July 15, 1920	5,730
May 17, 1920	2,650		

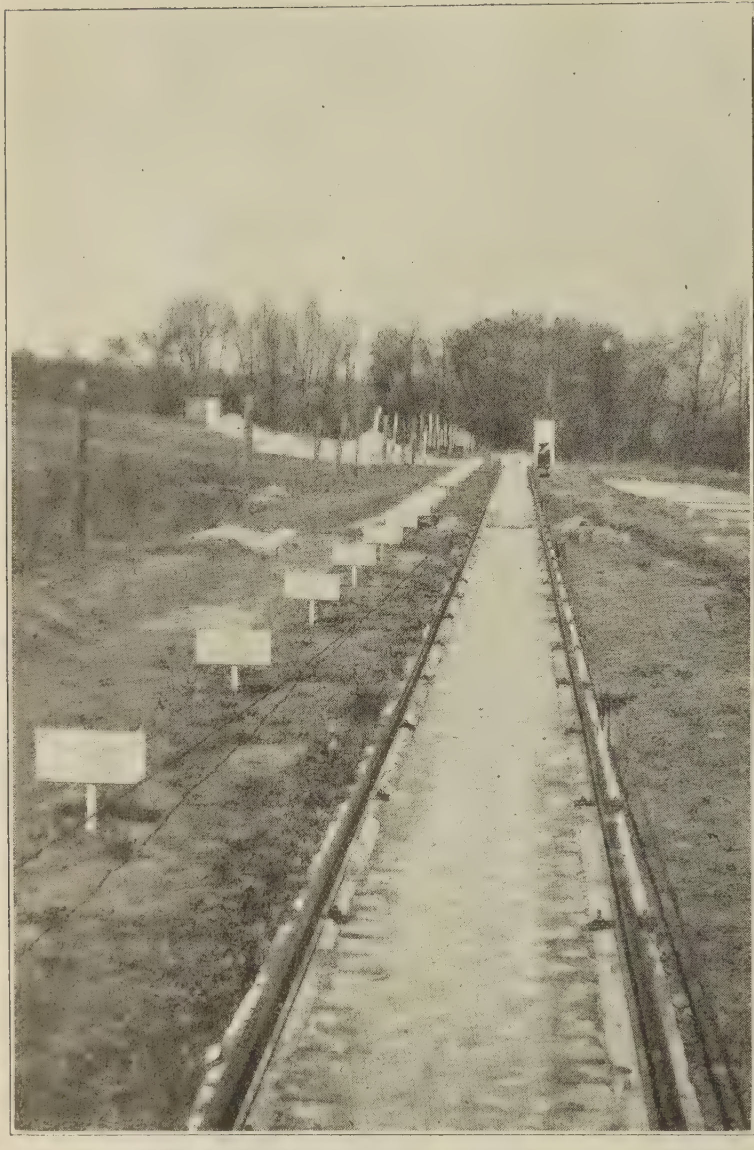
COMPARISON OF GRANITE BLOCK SECTIONS.

A detailed layout of the various sections of granite block, showing variations in the quality of the block used and difference in the filler and bedding courses, is given in Table 3. The figures given under "Per cent of wear" in each case were determined by the use of the modified abrasion test previously referred to and bear no definite relation to the per cent of wear determined by the standard De Val abrasion test.

TABLE 3.—*Granite block sections.*

Section.	Block from—	Type of filler.	Type of bedding course.	Per cent of wear.
1	Vinalhaven, Me.	Cement grout	Sand-cement	4.0
2	Rockport, Mass.	do	do	4.2
3	Mount Airy, N. C.	do	do	6.0
4	Chelmsford, Mass.	do	do	4.4
5	Concord, N. H.	do	do	5.0
6	Cape Ann, Mass.	do	do	4.4
7	Lithonia, Ga.	do	do	6.2
8	do	Asphalt	do	6.2
9	Vinalhaven, Me.	do	do	4.0
10	Rockport, Mass.	do	do	4.2
11	Mount Airy, N. C.	do	do	6.0
12	Vinalhaven, Me.	Asphalt mastic	do	4.2
13	Chelmsford, Mass.	do	do	4.4
14	Concord, N. H.	do	do	5.6
15	Somes Sound, Me.	Asphalt	Sand	4.2
16	Long Cove, Me.	Asphalt mastic	Asphalt-sand	4.4
17	Vinalhaven, Me.	Tar mastic	Sand-cement	4.2
18	Somes Sound, Me.	Tar	Sand	4.2
19	Long Cove, Me.	Tar mastic	Tar-sand	4.4

PLATE 2.—WEAR TESTS OF GRANITE BLOCK—6,000 RUNS



GENERAL VIEW OF GRANITE BLOCK SECTIONS Nos. 10 to 19 AFTER TEST.

The first section to show indications of failure was No. 7, the grouted section of Georgia granite. As previously noted, the block with which this section was paved were very roughly cut at the quarry, so that it was impossible to lay them as smoothly as the other block. It is probable, then, that the impact produced by the heavy wheels rolling over the rough heads of these block started the breaking up of this section, and the result illustrates very forcibly the value of using accurately cut block on pavements subjected to heavy steel-tired traffic. The first noticeable effect of the action of the testing machine on section 7 was the failure of the cement grout, which started to scale off the block and break out of the joints shortly after the test was started. The grout failure was accompanied by an appreciable settlement of the entire section. This settlement increased as the test progressed and amounts at the present time to about 1½ inches. There has been considerable wear on the unprotected edges of these block and several have been broken. The portion of this section adjoining section No. 6 has likewise been subjected to additional impact caused by the presence between the two sections of a concrete bulkhead constructed across the trench at this point to the full depth of the concrete base. Under the action of the wheels this bulkhead was worn down more than the adjoining block, and the resulting difference in elevation caused considerable impact. That the excessive wear which has taken place on this section is due to the additional impact caused by this bulkhead and the rough block rather than to the fact that the granite itself was rather soft is evidenced by the fact that section No. 3, containing nearly the same quality of granite, as measured by the abrasion test, is still in very good condition.

The next evidences of failure were observed on sections 1, 2, and 6. As in the case of No. 7, the action was started by the cement grout scaling off the surface and breaking out of the joints of the block, accompanied by settlement of the sections as a whole. The appearance of these sections just after initial failure of the grout may be noted by reference to figures 1, 2, and 6 of plate 1. This plate shows the condition of the various sections at the end of 1,650 runs of the testing machine. Figure 7 on the same plate shows the condition of section No. 7 at the same time. It will be noted that the action has progressed considerably farther on this section.

Settlement on all four of these sections has been accompanied by considerable transverse bending. On section No. 1 particularly, the block along the center of the runway directly under the wheels of the testing machine are about one-half inch lower than along the sides. This is probably due to the concentration of the load on the center 12-inch strip of the experimental section. This load working against the friction between the ends of the block and concrete curbs probably caused the bending which has been observed. The transverse bending has likewise produced a pinching action at the surface between the ends of the center blocks. The compressive stress thus developed would undoubtedly be great enough to crush the grout in the joints along these lines and even spall off the joint itself. This may account to some extent for the rapid failure of the cement grout on these sections and also for the fact that considerable spalling has taken place. An interesting point in connection with this bending is the fact that the concrete base has not cracked longitudinally and is perfectly flat, although it has been pushed down about 2 inches. The sand-cement bed, furthermore, is well set up, though it has not adhered to any extent either to the base or the block. The only explanation of this fact would appear to lie in the assumption that the sand-cement bed has compressed sufficiently under the center blocks to take care of the bending without cracking the concrete base. The remaining three grouted sections, Nos. 3, 4, and 5, are still in very good condition. The grout bond is intact and flush with the joints. While there is some evidence of wear on these sections it is uniform, the block and the grout wearing down together in the ideal way. Recently the grout has begun to break out of the joints of the first 3 feet of section No. 3, but the action here is progressive and was begun by impact due to the somewhat rough surface of section No. 2. Settlement not exceeding one-fourth inch has taken place.

BITUMINOUS FILLED SECTIONS SHOW LITTLE WEAR

The most noticeable feature of the sections containing bituminous fillers is the fact that although some settlement has taken place at various points there is very little evidence of wear and almost no breaking up of individual block. Even section No. 8, which, from the beginning, was much rougher than the others, has suffered less than the corresponding section No. 7 with cement grout filler. A slight amount of wear has taken place on the sections in which the softer granites were used. The wear has been uniform, however, and the sections are practically as smooth as they were in the beginning. On the sections using asphalt-mastic filler a considerable ironing out of the filler over the surface of the block has taken place. This characteristic action is clearly shown in plate 1. The excess filler has remained in place in certain portions of the section up to the present time. On the two sections containing tar-mastic filler, however, the mastic has not adhered quite so well to the surface of the block although the joints are as well filled as in the case of the asphalt-mastic filler. The single straight tar-filled section No. 18 is not in very good condition. The filler has chipped out to some extent, and there is evidence that water has found its way through the joints of the block to the sand bed. The surface is somewhat rougher than the corresponding section 15 on sand bed in which a straight asphalt filler was used. Considerable settlement has taken place, however, on both sections, No. 15 and No. 18, laid on the sand bed, and the surfaces are slightly more uneven than in the cases where a sand-cement bed was used. Section No. 16, built on an asphalt-sand bed prepared by mixing about 10 per cent asphalt with 90 per cent sand, has likewise settled somewhat unevenly. Section No. 19 constructed on a tar-sand bed prepared in the same manner is, on the other hand, in fairly good condition, very little settlement being noted. The present condition of section 19 is especially interesting in view of the fact that it adjoins a concrete bulkhead similar to the bulkhead separating sections Nos. 6 and 7, which were cement grouted. However, the impact produced by this bulkhead has not caused the damage observed on the grouted sections. In general, the amount of settlement on the bituminous-filled sections varies from approximately 1 inch, on sections laid on sand bed, to zero, on certain sections laid on sand-cement bed. Bituminous-filled sections have, in general, not settled as much as grout-filled sections Likewise settlement on sections laid on sand-cement bed is not as great, in general, as on sections laid on straight sand or bituminous-sand beds.

DISCUSSION OF THE GRANITE-BLOCK TESTS.

It will be of interest to review briefly the reasons underlying the behavior of the various sections under test. Reference has been made to the fact that the first six cement-grouted sections constituted originally a continuous beam about 50 feet long, 2 feet wide, and 5 inches deep. This beam rested upon a 1-inch 1:4 sand-cement bed, which, in turn, was placed on a concrete base 8 inches in depth. The concrete base rested upon a compacted cinder fill, which was rammed by hand, every effort being made to make it uniform throughout. In other words, the construction was what is ordinarily known as "semimonolithic." Section 1 of this beam joined the last section of the vitrified-brick portion of the runway. Section 6 was placed next to the concrete bulkhead previously mentioned.

Both the brick sections and the concrete bulkhead started to wear somewhat before the granite. Observations showed that the brick section adjoining granite section 1 developed considerable unevenness at about

DETAILED VIEW OF SECTION 1, SHOWING SETTLEMENT AND PINCHING ACTION ON UPPER EDGE OF BLOCK.

1,650 runs. It is probable, therefore, that the increased impact caused by the worn brick and concrete started the disintegration of the cement grout on stone-block sections 1 and 6. The action once begun progressed slowly toward the center from both ends, the breaking of the grouted joints being accompanied by gradual settlement of the sections. This action up to the present time has progressed in one direction through sections 1 and 2 and the first 3 feet of section 3 and in the other direction entirely through section 6. The amount of settlement varies with the distance from the ends. The maximum side settlement is about 1 inch at the ends of sections 1 and 6, whereas sections 4 and 5 show very little settlement at all. The condition of all of these grouted sections illustrates very forcibly the necessity for reducing impact from any cause on granite-block pavements. In these experiments where the impact has been excessive, as on sections 1, 2, and 6, the grout has failed, and the sections have settled. Where it has not been excessive, as on those portions of sections 3, 4, and 5, not yet subject to increased impact, no damage has been done. Another point of interest is the fact that the three center sections are all in equally good condition in spite of the fact that the percentage of wear of the three granites represented varies from 4.4 to 6. This leads naturally to the conclusion that the so-called softer granites, if well cut and properly laid, are practically as resistant to steel-tired wheels as the harder varieties. If the blocks are not well cut, however, so that they can not be laid to a smooth surface, as was the case in section 7, impact of heavy traffic will soon break out the grout, after which the soft block will undoubtedly wear faster than the harder varieties. This is brought out by comparing sections 1 and 2 with section 7. Sections 1 and 2, constructed with hard granites, although subjected to considerable impact, due to the grout having failed and also because of the poor brick section adjoining section 1, have not worn or broken up nearly as badly as section 7, which contains the soft granite.

The most interesting feature in connection with the bituminous-filled sections is the fact that although considerable settlement has taken place, there is very little evidence of wear or breaking up of individual block. Even the sections which have settled unevenly show but little wear, indicating that the soft filler has served as a cushion against the impact of the iron wheels, which has been lacking in the rigid type. The soft filler likewise is capable of readjusting itself to slight inequalities in the surface, due to uneven settlement of the block—an advantage not possessed by the rigid type. It would appear, then, that under the practical conditions approximated by this test, bituminous fillers are, in general, as satisfactory as cement grout filler, especially when there is any tendency of the block to settle unevenly, thereby increasing the impact force exerted by moving wheels. Again, as has been noted, there is practically no difference in the behavior of bituminous and bituminous mastic-filled sections, except that in cases where the mastic is used a very noticeable ironing out of the excess filler has taken place. This would indicate that the latter is just as satisfactory as the straight bituminous filler for street pavement work. The fact that both sections laid on sand bed show considerable settlement and unevenness would tend to indicate that this type of bedding course is not as satisfactory as the sand-cement bed, even when bituminous fillers are used. The fact that the tar-sand bed section, No. 19, has not settled as much as the asphalt-sand bed section, No. 16, indicates that the tar may have provided a somewhat more rigid bedding course than the asphalt. This single comparison would, however, hardly justify a conclusion to this effect.

CONCLUSIONS FROM THE GRANITE BLOCK TESTS.

Although it is realized that certain limiting conditions, such as the small size of the test sections and the fact that settlement occurred at certain points must be taken into account when interpreting the results of these tests, it is felt that a number of general conclusions may be drawn. These conclusions may be summed up as follows:

1. Bituminous-filled granite block pavements will resist the impact produced by heavily loaded steel-tired traffic as well as cement-grouted pavements.
2. Bituminous mastic fillers are as satisfactory for this type of traffic as straight bituminous fillers.
3. The effect of impact is tremendously increased by irregularities produced by poorly cut block.
4. Irregularities of surface or other factors producing impact are more serious with grouted than with bituminous-filled pavements.
5. Slight variations in resistance to wear, such as occur among the commercial granite block from the Atlantic coast quarries, are of much less importance in judging the probable resistance of the block to the action of traffic than has commonly been supposed.
6. Cement-sand bedding courses are more satisfactory than sand or bituminous-sand bedding courses.

COMPARISON OF BRICK SECTIONS.

The types of brick pavements tested and their arrangement in the runway are shown in Table 4. The per cent of wear was determined by the manufacturer, using the standard rattler test.

In general, the progress of wear in all sections was similar. First the excess filler was broken and pulled off the surface of the bricks; then a uniform wearing of the bricks occurred over the entire length of the section. This uniform wear was followed by excessive wearing in spots, causing a very rough and uneven surface. Complete failure of the section rapidly followed this uneven condition.

PLATE 3.—REPRESSED AND WIRE-CUT LUG BRICK—16 PER CENT WEAR, LAID ON SAND-CEMENT CUSHION.

TABLE 4.—*Brick sections.*

Section No.	Brick.		Cushion.		Length section.	Joint filler material.
	Type.	Wear.	Material.	Thick.		
		Per cent.		*In.*	*Ft.*	
1	Repressed	16	Sand-cement 4:1	$\frac{3}{16}$	8	Grout 1:1.
2	Wire-cut lug	16	do	$\frac{3}{16}$	8	Do.
3	Repressed	19	do	$\frac{3}{16}$	8	Do.
4	Wire-cut lug	19	do	$\frac{3}{16}$	8	Do.
5	do	24	do	$\frac{3}{16}$	8	Do.
6	Repressed	24	do	$\frac{3}{16}$	8	Do.
7	do	16	do	$\frac{3}{16}$	8	Asphalt.
8	Wire-cut lug	16	do	$\frac{3}{16}$	8	Do.
9	Repressed	19	do	$\frac{3}{16}$	8	Do.
10	Wire-cut lug	19	do	$\frac{3}{16}$	8	Do.
11	Repressed	24	do	$\frac{3}{16}$	8	Do.
12	Wire-cut lug	24	do	$\frac{3}{16}$	8	Do.
13	Vertical fiber	18.8	do	$\frac{11}{16}$	8	Do.
14	do	18.8	Sand	1	8	Do.
15	3-inch wire-cut lug.	17.7	do	1	8	Do.
16	do	17.7	do	1	8	Asphalt mastic 1:1.
17	do	17.7	do	1	8	Tar.
18	do	17.7	do	1	8	Tar mastic 1:1
19	do	17.7	do	1	8	Grout 1:1.
20	Wire-cut lug	16	Sand-cement 4:1	$\frac{1}{8}$	8	Tar.
21	Vertical fiber	18.8	do	$\frac{1}{2}$	8	Grout 1:1.

In sections with elastic fillers the wear was confined to the areas which came in contact with the cast-iron wheels, causing ruts or grooves to develop as the test continued. In sections having nonelastic fillers the wear was of a crushing or shattering kind, causing the bricks to shear and break in areas adjacent to as well as in the path of the wheels.

For convenience the stages of wear on the various sections are noted and referred to in the following discussion as (1) the first indication, (2) the beginning of nonuniform wear, and (3) total failure.

Plates 3 to 7, inclusive, show photographs of the different sections arranged with respect to their kind and hardness. Five views of each section, taken at different stages of the wear, are shown. For any particular section, the same brick is indicated in successive views by a black dot. By this means the study of the progress of wear is facilitated. The following comparisons are made from notes taken during the test as well as from the photographs.

BRICK SHOWING 16 PER CENT WEAR COMPARED.

On plate 3 are grouped the sections made of 4-inch brick, which showed 16 per cent of wear in the rattler test. All the sections shown are laid on a sand-cement cushion. Sections 2, 8, and 20 are wire-cut lug bricks, with cement grout, asphalt, and tar fillers, respectively, while sections 1 and 7 are repressed bricks, with cement grout and asphalt fillers, respectively.

Of these sections Nos. 2, 8, and 20, made of wire-cut lug brick, are all in good condition after 5,730 runs. The first indication of wear on sections 2 and 8 appeared after 2,150 runs; on section 20 after 1,650 runs. However, the wear increased on Nos. 20 and 2 faster than on No. 8, so that after 5,730 runs No. 20 is not as good as No. 2, and No. 2 is slightly worse than No. 8.

The repressed brick in sections 1 and 7 showed no indications of wear until after 2,150 runs, after which the wear on the asphalt-filled section developed more rapidly than on the section filled with cement grout. The former developed nonuniform wear after 2,650 runs. After 5,730 runs the cement-grouted section is very good, and the asphalt-filled bricks are very much worn.

Arranging the five sections in the order of their resistance to wear in the test, they rank 8, 2, 1, 20, 7. It should be explained, however, that sections 1 and 2 were possibly subjected to less abrasive action on account of their position at the end of the runway, and No. 20, being between two badly worn sections, was subjected to exceptionally heavy abrasive action. Taking these conditions into account, a fairer statement of their relative resistance would rate them 8, 20, 2, 1, and 7, the difference between the first four being very slight. This would indicate that in resistance to wear the wire-cut lug brick rank above the repressed. A study of plate 3 shows also that neither cement grout nor bituminous fillers protect or support the edges of repressed brick as well as those of wire-cut lug. It also appears that in resistance to wear the wire-cut lug bricks with bituminous fillers are better than with cement grout, while with repressed bricks cement grout gives the better results. A shattering of sections with cement fillers and rutting of sections with bituminous fillers is indicated.

OBSERVATIONS OF BRICK SHOWING 19 PER CENT WEAR.

On plate 4 are grouped the 4-inch bricks that showed 19 per cent wear in the rattler test. Each of the four sections was laid on a sand-cement cushion, sections 4 and 10 being wire-cut lug brick and sections 3 and 9 repressed. Nos. 4 and 3 were laid with cement-grout filler and Nos. 10 and 9 with asphalt filler.

The two sections of wire-cut lug brick showed first indications of wear after 1,175 runs, after which on the section filled with cement grout the wear increased uniformly to the end of the test, the surface remaining good. On the section filled with asphalt the wear was faster, but uniform, until 3,750 runs, after which nonuniform rutting occurred

The sections of repressed brick showed first indications of wear after 1,175 runs. From that point the cement-filled section increased uniformly in wear to 2,650 runs, after which nonuniform breaking and shearing occurred. By 3,750 runs the nonuniform wear was well developed, and total failure came at the end of 4,650 runs. Up to that point the asphalt-filled section had shown only uniform wear, but subsequently shearing of brick caused a very uneven surface.

Comparison of these 4-inch brick of the same hardness (19 per cent wear) laid on a sand-cement cushion indicates that in resistance to wear they rank 4, 10, 9, and 3, there being appreciable differences between them, with the wire-cut lug showing better than the

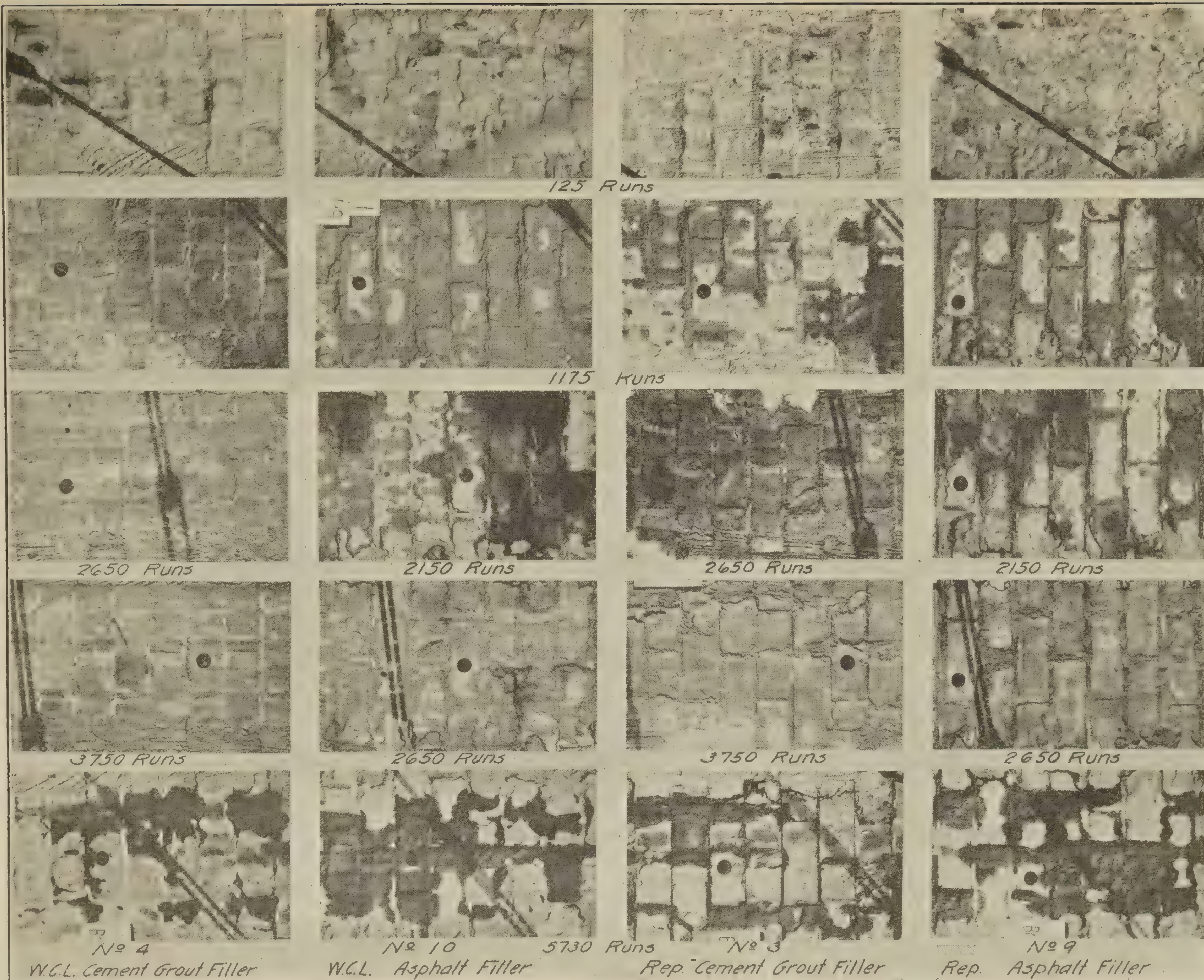


PLATE 4.—REPRESSED AND WIRE-CUT LUG BRICK—19 PER CENT WEAR, LAID ON SAND-CEMENT CUSHION.

repressed bricks. The wire-cut lug brick show better with cement grout than with asphalt filler, while the repressed show better with asphalt. As in the sections of 16 per cent brick, those filled with cement grout show a tendency toward shattering, and the asphalt-filled sections develop grooves. The 19 per cent brick show less resistance to abrasion than the 16 per cent brick. Again, also the fillers, both asphalt and cement grout, offer considerably more protection and support to the edges of the wire-cut lug than to those of the repressed brick.

WEARING QUALITIES OF THE 24 PER CENT BRICK.

Plate 5 shows the 24 per cent brick of 4-inch thickness laid on a sand-cement cushion. Sections 5 and 12 are wire-cut lug with cement grout and asphalt fillers, respectively, and sections 6 and 11 are repressed brick with cement grout and asphalt filler, respectively.

All four sections showed first indications of wear after 625 runs, after which the wire-cut lug with cement filler showed uniform wear to 2,150 runs, when nonuniform shattering and shearing began, resulting in total failure after 4,650 runs. Filler afforded little protection to edges after 3,250 runs. On the asphalt-filled, wire-cut lug section the wear was uniform to 2,650 runs, after which nonuniform grooving action developed deep ruts by 5,500 runs. The asphalt filler offered better protection to the edges of the brick in this section at 5,730 than at 2,150 runs, due to the ironing out of the filler in warm weather. On the repressed section with cement-grout filler, the wear became nonuniform after 1,175 runs, resulting in sheared and shattered brick after 3,750 runs. The filler offered no protection to the edges after 1,175 runs. The repressed, asphalt-filled brick showed uniform wear to 2,150 runs, after which nonuniform grooving resulted in deep ruts by 5,730 runs.

Comparison of the four sections shows that in resistance to wear they rank 12, 11, 5, and 6. Again it is indicated that both types of filler offer more protection to the edges of the wire-cut lug than to those of the repressed brick, and again the test results are favorable to the wire-cut lug. For both kinds of brick the asphalt filler shows better than cement grout.

THE BEHAVIOR OF THE 17.7 PER CENT BRICK.

Plate 6 shows the group of 3-inch brick sections. All these brick are wire-cut lug, and all are laid on sand cushions. All the brick showed 17.7 per cent wear in the rattler test, consequently the only variable in the five sections is the filler, which was different in each section. As shown in the plate, the fillers used were asphalt, asphalt mastic, tar, tar mastic, and cement grout.

The first indication of wear appeared in the cement-filled section after 125 runs; in all the others there was no indication of wear until after 1,175 runs. Nonuniform wear, in the several sections, began as follows:

	Runs.
Asphalt filler	4,650
Asphalt-mastic filler	3,750
Tar filler	2,150
Tar-mastic filler	1,650
Cement-grout filler	625

In general resistance to wear, the sections rank in the same order, indicating that as the elasticity of the filler decreases the crushing and shattering of the brick increase. The cement-filled section in this group showed complete failure after 2,150 runs. This was the worst of the 21 sections.

THE 3½-INCH VERTICAL-FIBER BRICK WITH 18.8 PER CENT WEAR.

The vertical fiber brick which tested 18.8 per cent wear in the rattler are shown in plate 7. In sections 13, 14, and 21 they are laid with asphalt, asphalt, and cement-grout fillers, respectively, and on sand-cement, sand, and sand-cement beds.

The two asphalt-filled sections showed first indications of wear after 1,175 runs, the wear increasing slowly and uniformly, leaving a fairly good surface after 5,730 runs. The filler afforded excellent protection to the edges. The cement-filled section, on the other hand, showed traces of wear after 625 runs, the wear became nonuniform after 1,650 runs, and the section was a total failure after 3,750 runs.

In general, the sections rank 13, 14, and 21, but there is almost no appreciable difference between 13 and 14. Section 21, with the cement-grout filler, is far inferior to the other two. The elasticity of the asphalt and the protection it afforded the edges of the brick is no doubt responsible for this difference.

ALL BRICK SECTIONS COMPARED.

Generalizing from the results of the tests, the various sections can be divided into three classes, according to their condition at the end of the test. In the first class are the sections which show uniform wear, but have not developed marked unevenness of surface. In the second class are those sections in which nonuniform wear has developed resulting in a very uneven condition covering not more than 50 per cent of the length of the section. In the third class are those sections which are considered as total failures, in which more than 50 per cent of the lengths are shattered and rutted nonuniformly.

The various sections grouped in accordance with this mode of classification and arranged within the classes

PLATE 5—REPRESSED AND WIRE-CUT LUG BRICK—24 PER CENT WEAR, LAID ON SAND-CEMENT CUSHION.

PLATE 6.—3-INCH WIRE-CUT LUG BRICK—17.7 PER CENT WEAR, LAID ON SAND CUSHION WITH DIFFERENT FILLERS.

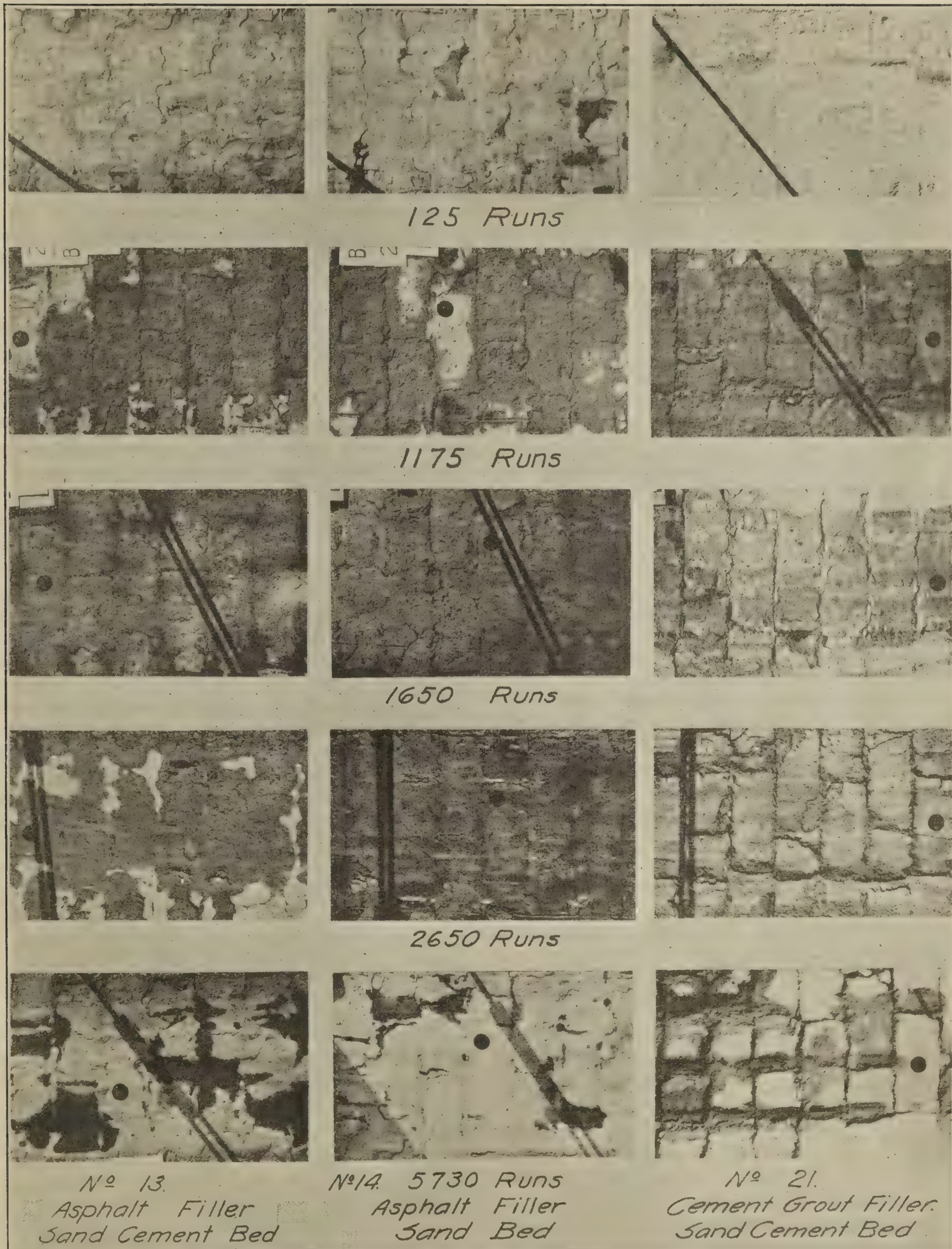


PLATE 7.—3½-INCH VERTICAL FIBER BRICK—18.8 PER CENT WEAR, LAID ON DIFFERENT BEDS AND FILLED WITH DIFFERENT FILLERS.

as well as possible with regard to their resistance to wear, are as follows:

CLASS 1.

Section No.	Description.
8	16 per cent, 4-inch wire-cut lug, sand-cement cushion, asphalt filler.
2	16 per cent, 4-inch wire-cut lug, sand-cement cushion, cement filler.
4	19 per cent, 4-inch wire-cut lug, sand-cement cushion, cement filler.
1	16 per cent, 4-inch repressed, sand-cement cushion, cement filler.
20	16 per cent, 4-inch wire-cut lug, sand-cement cushion, tar filler.
13	18.8 per cent, 3½-inch vertical fiber, sand-cement cushion, asphalt filler.
14	18.8 per cent, 3½-inch vertical fiber, sand cushion, asphalt filler.

CLASS 2.

Section No.	Description.
10	19 per cent, 4-inch wire-cut lug, sand-cement cushion, asphalt filler.
16	17.7 per cent, 3-inch wire-cut lug, sand cushion, asphalt-mastic filler.
7	16 per cent, 4-inch repressed, sand-cement cushion, asphalt filler.
9	19 per cent, 4-inch repressed, sand-cement cushion, asphalt filler.
15	17.7 per cent, 3-inch wire-cut lug, sand cushion, asphalt filler.
12	24 per cent, 4-inch wire-cut lug, sand-cement cushion, asphalt filler.
11	24 per cent, 4-inch repressed, sand-cement cushion, asphalt filler.
17	17.7 per cent, 3-inch wire-cut lug, sand cushion, tar filler.
18	17.7 per cent, 3-inch wire-cut lug, sand cushion, tar-mastic filler.

CLASS 3.

Section No.	Description.
6	24 per cent, 4-inch repressed, sand-cement cushion, cement filler.
3	19 per cent, 4-inch repressed, sand-cement cushion, cement filler.
5	24 per cent, 4-inch wire-cut lug, sand-cement cushion, cement filler.
21	18.8 per cent, 3½-inch vertical fiber, sand-cement cushion, cement filler.
19	17.7 per cent, 3-inch wire-cut lug, sand cushion, cement filler.

Table 5 shows the progress of wear on the various sections during the test.

GENERAL COMMENTS.

In general, the sections filled with cement grout showed settlement along the sides as well as in the center, while the bituminous-filled sections showed more settlement in the center than along the edges. The maximum settlement of the grout-filled sections on sand-cement cushion ranged from $\frac{1}{2}$ to $1\frac{1}{2}$ inches, with an average of $\frac{15}{16}$ inch. The maximum settlement of bituminous-filled sections on sand-cement cushion ranged from $\frac{1}{4}$ to 1 inch, with an average of $\frac{1}{2}$ inch. The average longitudinal variation of the above two types was $\frac{1}{8}$ inch. The maximum settlement of bituminous-filled sections on sand cushion ranged from $\frac{1}{2}$ to $\frac{7}{8}$ inch, with an average of $\frac{11}{16}$ inch. The average longitudinal variation was $\frac{3}{16}$ inch. The one cement-grout-filled section on sand cushion showed $1\frac{1}{4}$ inches settlement. Considering the uniform base and foundations under these sections, the additional settlement of the grout-filled sections over the bituminous-filled sections can not but indicate that the effect of the impact caused by the same agency (steel-tired traffic) is much greater on a rigid surface than it is on an elastic surface. It is also indicated that a 1-inch sand cushion allows more compression and consequently more unevenness of surface than a $\frac{3}{16}$-inch sand-cement bed. The difference in compression or, possibly, movement of the sand cushion would account for the difference in the settlement found between sections laid on sand and sand-cement cushions.

Table 5 shows that there is conformity between the first indications of wear on the various sections and the results of tests made with the standard brick rattler. This first indication of wear seems to be more or less dependent upon the degree of hardness of the brick. Several exceptions show that the method of laying and the thickness of the brick are also factors to be considered. When laid on a sand cushion the 3-inch, grout-filled, wire-cut lug brick showed first wear 1,050 runs before the bituminous-filled brick. Also the $3\frac{1}{2}$-inch vertical fiber brick on sand-cement cushion showed first wear 650 runs before the same brick with bituminous fillers. The fact that first indications of wear on section 20 occurred 500 runs before the other four 16-per-cent sections can be attributed, at least in part, to the fact that it received extra heavy impact because of the very bad condition of section 19. In general, then, the 24 per cent wear bricks showed first indications of wear after 625 runs, the 17.7 per cent, 18.8 per cent, and 19 per cent brick after 1,175, and the 16 per cent brick after 2,150 runs. After this first stage, however, the thickness of the bricks and the type

TABLE 5.—*Showing progress of wear—all brick sections.*

Runs.	Sections.																				
	16 per cent wear.					19 per cent wear.				24 per cent wear.				18.8 per cent wear.			17.7 per cent wear.				
	1	7	2	8	20	3	9	4	10	5	12	6	11	13	14	21	15	16	17	18	19
125																S					S W
625					S					W	W	W	W		S	W	S	S	S	S	N
1,175						W	W	W	W			N		W	W		W	W	W	W	
1,650					W			S		S		S				N				N	
2,150	W	S W	S W	S W		S	S		S	N	S		S N	S					N		F
2,650	S	N				N					N										
3,250																					
3,750									N			F				F		N			
4,650						F	N			F							N				
5,730																					
Max. settlement, inches.	$\frac{1}{2}$	$\frac{1}{4}$	$\frac{5}{8}$	$\frac{1}{4}$	1	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{3}{4}$	$\frac{1}{4}$	$1\frac{1}{4}$	$\frac{3}{4}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{3}{8}$	$\frac{5}{8}$	$1\frac{1}{2}$	$\frac{7}{8}$	$\frac{7}{8}$	$\frac{1}{2}$	$\frac{1}{2}$	$1\frac{1}{4}$

W=first indication of wear on bricks.
N=beginning of nonuniform wear.
F=total failure.
S=first indication of settlement.

of filler seemed to control the rate of disintegration. Of the 7 sections remaining in good condition at the end of the test, 3 are 4-inch grout-filled brick, 2 are 4-inch bituminous-filled, and 2 are 3½-inch bituminous-filled brick. Of the 4-inch brick all are wire-cut lug except section 1, and, as noted before, this section probably received less wearing action from the apparatus than any other in the runway.

The 9 sections in which nonuniform wear was registered are all bituminous-filled, the best being 4-inch wire-cut lug showing 19 per cent. The 5 sections which showed total failure are all cement-grout-filled sections.

CONCLUSIONS FROM THE BRICK TESTS.

Making allowance for the smaller amount of wearing action occasioned on section 1 and the exceptional amount of wear on section 20, the following conclusions are obvious:

1. The edge protection offered by bituminous and cement-grout fillers is considerably greater for vertical fiber and wire-cut lug than for repressed brick.

2. The adhesion of bituminous fillers to wire-cut lug and vertical fiber brick tends to protect the surface and to reduce the wear.

3. With cement-grout fillers the surface becomes a rigid slab and failure occurs because of the breaking of this slab under load and consequent loosening and shattering of the brick.

4. With cement-grout fillers and sand-cushion construction the brick must be so thick as to make a slab which will resist without excessive distortion the impact produced by the load moving over it. In such cases the cement-grout filler offers excellent support to the wire-cut lug and vertical fiber brick. The above results indicate that for sand and sand-cement cushions and for such loads as were had at Arlington thicknesses under 4 inches are impractical.

5. For brick of sufficient thickness to form a beam which will not be broken under impact, cement grout offers better support to the edges than bituminous fillers. The bituminous fillers cushion the edges, but under certain kinds of traffic (steel tires) allow the edge to be crushed even while the filler remains intact.

6. Tar and tar-mastic fillers form more rigid slabs in cold weather than asphalt and asphalt mastic, and consequently brick filled with the former tend more toward shattering than those filled with the latter.

7. For the same conditions of brick and type of construction, brick with rounded edges offer less resistance to wear than those with square edges.

8. Sand cushions are subject to more compression than sand-cement cushions, and the greater compression results in a more uneven surface.

9. Elastic fillers considerably reduce the effects of impact occasioned by steel-tired traffic, and the destructive effect increases with increased rigidity of the fillers, the maximum destructive effect occurring with such fillers as have greatest rigidity. The shattering of the brick and the additional settlement noted on the cement grout sections warrant this conclusion.

10. The resistance to wear of the various pavement sections, as shown from the wear test, is the same as that indicated by the standard rattler test for the brick comprising the above sections.

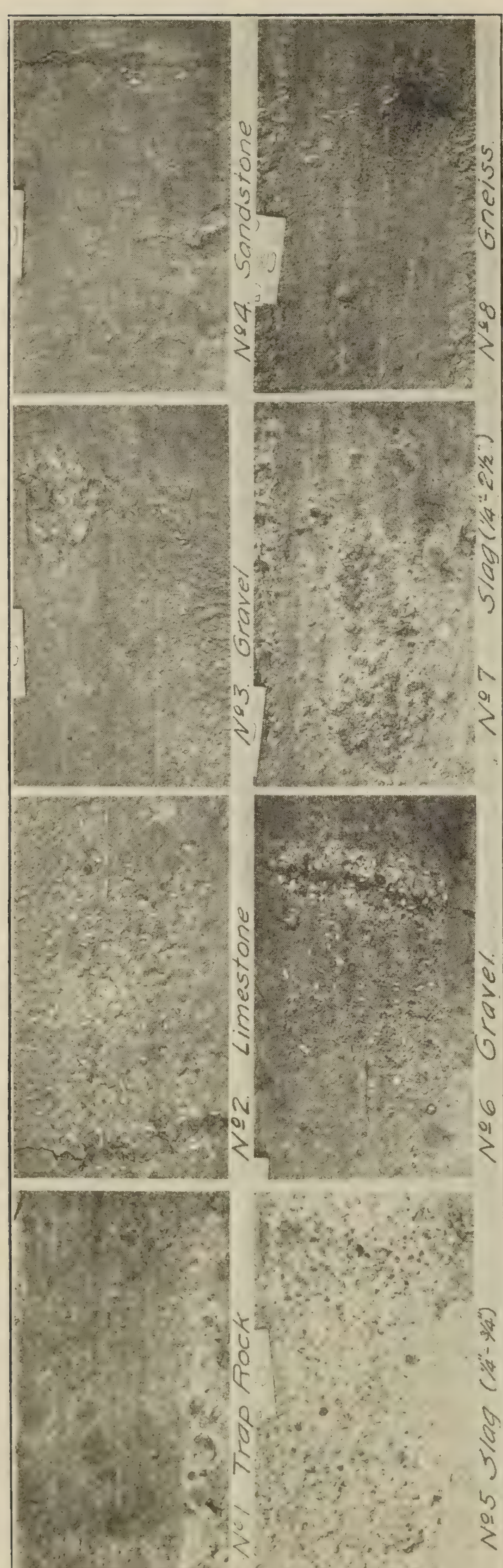
CONCRETE PAVEMENT SECTIONS.

From the behavior of the concrete sections under test it was intended to secure comparisons of the resistance to wear offered by concretes made from different aggregates. Eight sections were constructed, each 6 inches thick and each of 1:1½:3 concrete. Views of these sections after 2650 and 5730 runs, respectively, are shown in plate 8. The concrete was hand-mixed and finished with a wooden float. For the purpose of determining any possible relation between the resistance to wear and compressive strength afforded by use of the different aggregates, 6 by 12 inch test cylinders were made from the same batches of concrete used in the different sections. These cylinders were buried beside the runway August 8, 1919, and tested in the laboratory April 7, 1920. The kinds of aggregates used for the different sections, with their arrangement in the runway, together with the breaking strength of the test cylinders, is given in Table 6. The slump indicated was determined from 6 by 12 inch cylinders.

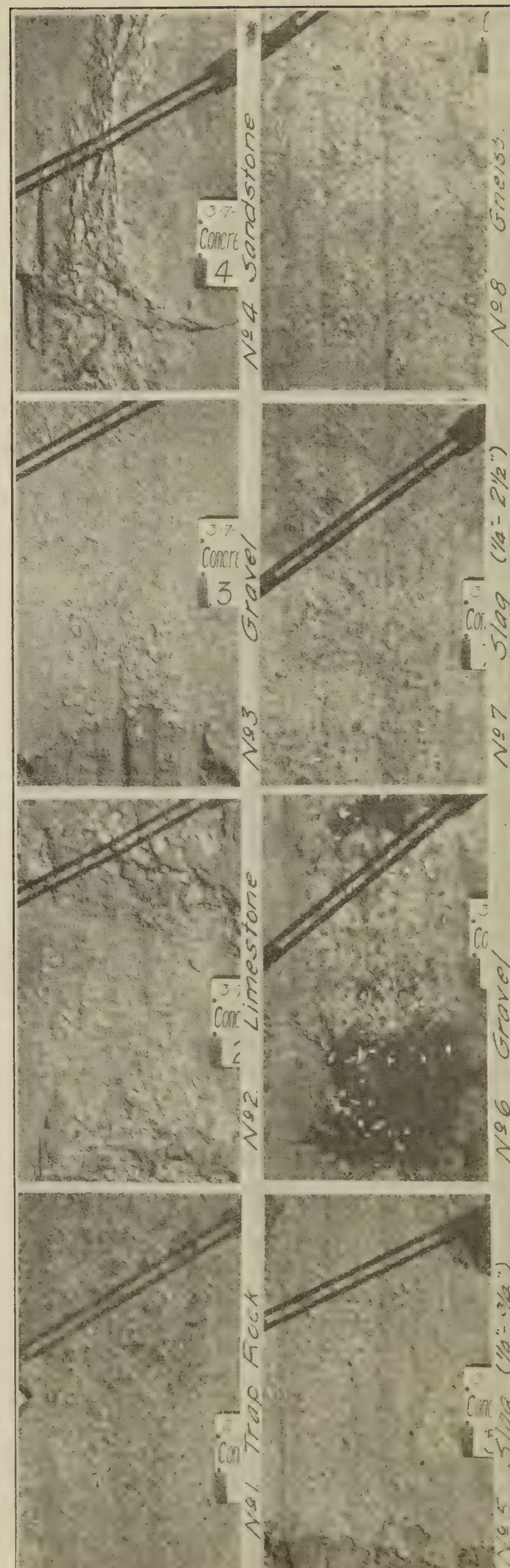
TABLE 6.

Section No.	Aggregate.		Length of section.	Cylinders.			
	Kind.	Size.		Slump.	Breaking strength.		
					1	2	Average.
		In.	Ft.	In.	Lbs.	Lbs.	Lbs.
1	Trap rock	¾–2	16.5	5	2,445	1,942	2,194
2	Limestone	¼–2½	9.0	5	2,578	2,815	2,698
3	Gravel	¼–1½	11.2	5	2,620	1,930	2,275
4	Sandstone	¼–2½	10.5	5	3,170	3,150	3,160
5	Slag	¼–¾	11.1	6–7	3,260	2,420	2,840
6	Gravel	¼–1½	12.0	7	3,000	2,800	2,910
7a	Slag	¼–2½	5.2	8			3,290
7b	do	¼–2½	2.0	2			
8a	Gneiss	¼–1½	5.7	7			3,180
8b	do	¼–1½	6.8	6			

During the progress of the test these sections, in addition to wear on their surfaces, developed a considerable number of transverse cracks followed by settlement, which makes comparisons of resistance to wear somewhat difficult. The following observations for wear were noted, however, on parts of the sections other than those adjacent to the cracks. In general the wear progressed the same on all sections beginning with a slight grooving action which gradually uncovered the large aggregate after which the depth of wear as well as the uniformity of the resulting



CONCRETE SECTIONS AFTER 2.650 RUNS.



CONCRETE SECTIONS AFTER 5.730 RUNS.

PLATE 8.—SHOWING WEAR OF THE CONCRETE SECTIONS AFTER 2.650 AND 5.730 RUNS.

surface depended more or less upon the hardness and size of the aggregate.

Section 1 (trap) showed gradual and uniform grooving action through the test and at the end of 5,730 runs these grooves which are fairly well defined show a maximum depth of $\frac{1}{4}$ inch, with a longitudinal variation not exceeding $\frac{1}{8}$ inch.

Section 2 (limestone) showed wear faster than No. 1 and a tendency toward unevenness over the worn area rather than distinct grooves. After 5,730 runs the surface shows a nonuniform wear $\frac{3}{8}$ inch in depth with a longitudinal variation of from $\frac{3}{16}$ inch to $\frac{1}{4}$ inch.

Section 3 (gravel) after 5,730 runs is very uniform, showing $\frac{1}{8}$ inch wear and $\frac{1}{16}$ inch longitudinal variation.

Section 4 (sandstone) after 5,730 runs is not so uniform as No. 3 and shows wear to a depth of $\frac{3}{8}$ inch, with a longitudinal variation of $\frac{1}{4}$ inch.

Section 5 (slag, $\frac{1}{4}$ to $\frac{3}{4}$ inch) shows wear to a depth of $\frac{1}{4}$ to $\frac{1}{2}$ inch with a longitudinal variation of $\frac{1}{8}$ inch.

Section 6 (gravel) is very uniform showing $\frac{1}{8}$ to $\frac{3}{16}$ inch wear and an average $\frac{1}{8}$ inch longitudinal variation.

Section 7*a* (slag, $\frac{1}{4}$ to $2\frac{1}{2}$ inches, 8-inch slump) shows nonuniform wear $\frac{1}{2}$ to $\frac{5}{8}$ inch deep, with a longitudinal variation of $\frac{1}{4}$ to $\frac{1}{2}$ inch.

Section 7*b* (slag, $\frac{1}{4}$ to $2\frac{1}{2}$ inches, 2-inch slump), with the exception of one small spot $\frac{1}{2}$ inch deep, shows more uniformity than 7*a* with an average of $\frac{3}{8}$ inch wear and $\frac{1}{8}$ inch longitudinal variation.

Section 8*a* (gneiss, 7-inch slump) shows grooving action to the depth of $\frac{1}{4}$ inch with a longitudinal variation of $\frac{1}{8}$ inch.

Section 8*b* (gneiss, 6-inch slump) shows slightly less wear than 8*a*, but this could be at least in part due to the fact that, being at the end of the runway, it did not receive as much wearing action as the other sections.

As can be noted, the trap rock and gneiss sections show the greatest resistance to wear, presenting very uniform surfaces. The gravel sections rank next, showing slightly more wear, but about the same uniformity. The limestone and sandstone sections compare favorably with the gravel as concerns depth of wear, but tend more toward developing nonuniformity of top. The slag sections show the least resistance to wear and compare favorably with each other except that 7*a* shows less uniformity. These sections, then, as regards resistance to wear should rank 1 and 8, 3 and 6, 2 and 4, 5, 7. From Table 6 it will be noted that in resistance to compression, as determined from the test cylinders, they should rank 7, 8, 4, 6, 5, 2, 3, 1. Comparisons of the above would indicate that this test shows no relation between compressive strength and resistance to wear afforded by the several concretes.

Comparison of sections 3 and 6, 7*a* and 7*b*, and 8*a* and 8*b* would indicate that the dry concretes offer more resistance to wear than the wet ones. Sections 3 and 6 as well as 8*a* and 8*b* give a slight indication of this while sections 7*a* and 7*b* furnish a very marked indication.

SEVERAL SECTIONS FAILED BY CRACKING.

The failure of several of the concrete sections was due to transverse cracking and settlement, accompanied by increased impact. It is possible that this cracking might indicate the resistance of the various aggregates to impact. It should be noted that in placing the cinder foundation every precaution was taken to have it compacted uniformly so that it would afford the same support under all slabs. The first crack occurred between sections 4 and 5 after 2,300 runs, the second and third occurred 3 feet each side of the first after, respectively, 2,600 and 2,650 runs. The effect of these cracks was to reduce the area over which the load was distributed, so that the intensity of the pressure was increased on the cinder foundation, with the result that the surface settled about $2\frac{1}{2}$ inches. In a similar manner a second series of cracks, resulting in the same condition of settlement, occurred around the junction of sections 3 and 4 after 3,900 runs. Further cracking allowed section 4 to settle to the elevation of the ends. Cracking in each direction from this settled section resulted in settlement of the surface, which extended from the joint between Nos. 6 and 7 to a point 4 feet from the joint between sections 1 and 2. Longitudinal cracking developed only in section 4. These observations would indicate that slag, sandstone, and limestone do not offer as much resistance to cracking as do gravel, trap rock, and gneiss. This conclusion, however, is questionable in the absence of further experiments.

From the tests on the concrete sections there are indications that neither the resistance to wear nor the resistance to cracking are dependent upon the compressive strength of the concrete as determined by 6 by 12 inch test cylinders, that more resistance to wear is afforded by dry concretes than by wet, and that the harder aggregates offer more resistance to wear than the softer. Whether this greater resistance offered by the harder materials would warrant securing them for a road at much additional expense when softer materials are at hand must be left to the judgment of the engineer with full knowledge of the traffic conditions to be accommodated.

NOTE.—Acknowledgment is made to Mr. L. I. Teller for able assistance in the conduct of the work. The test was under the general direction of Mr. Thomas H. MacDonald, Chief of the Bureau of Public Roads, and Mr. A. T. Goldbeck, engineer of tests, and under the direct supervision of Messrs. Earl B. Smith and F. H. Jackson, senior assistant testing engineers.

ON BITUMINOUS MACADAM AND BITUMINOUS CONCRETE ROADS

By E. J. WULFF, Senior Highway Engineer, Bureau of Public Roads

IT IS customary to differentiate the types of bituminous pavement into two classes, namely, bituminous macadam and bituminous concrete. In the former the cold stone is coated, after being placed in the road, with heated bituminous material; in the latter the heated mineral aggregate composed of coarse and fine aggregate is mixed with heated bituminous material in a machine mixer and then placed on the road in a hot condition. The two materials that enter into the construction of the types of roads under construction are bituminous materials and the mineral aggregate. These materials must be adapted for the various purposes, and most specifications describe the materials by certain requirements that they must have for the specific purpose intended.

The term "bituminous material" applies in common to asphalt as well as tar products. Both materials are used in penetration macadam; tars are employed only to a limited extent in bituminous concrete. When the term "asphaltic macadam" or "asphaltic concrete" is used the bituminous material is an asphalt.

The functions of the bituminous material in bituminous macadam and bituminous concrete are not quite analogous, although the adhesive quality of the bituminous material is the essential quality in both cases. In bituminous macadam the small stone and bituminous material serve as filler or binder, similar to the screenings and fine dust of a waterbound macadam. The interlocking action of the mineral aggregate is the main dependence against lateral displacement. As screenings and dust are omitted in bituminous macadam, these are replaced by stone chips and adhesive asphalt or tar, a combination which is not affected by water or excessive dryness nor by the sucking action of rubber tires.

In an asphaltic concrete the interlocking action of the stone is supplemented by a mortar formed of the fine mineral aggregate, which includes dust, and an asphaltic cement. The function of this mortar is essentially analogous to that of the mortar in cement concrete.

CONSISTENCY THE IMPORTANT QUALITY OF BITUMINOUS CEMENT.

On account of the different functions to be performed by the asphalt in the two different types, as well as the different working conditions under which it is employed when the pavement is laid, and the varying effect of climate, the consistency of the asphalt to be employed is of considerable importance.

The suitable consistency differs materially for different types of pavement as well as for different climatic conditions. Thus for penetration macadam the consistency of the bituminous material must be such that when in a melted condition it is fluid enough under proper working conditions to flow freely and coat the coarse stone aggregate with a thin film. When cooled to summer temperatures it must be stiff enough not to flow. When still further cooled to winter conditions it must remain plastic enough to contract without cracking.

The range of summer and winter temperatures differs materially for the United States. While high and prolonged summer temperatures may be expected throughout the country, the winter conditions are dissimilar. In the North, prolonged periods may occur when the temperature is at and below zero. In the Southern States, the winter temperature may but rarely reach the freezing point and then for short periods only. It is apparent that these extreme cases, as well as the variety of intermediate conditions, require bituminous materials of appropriate consistencies for local requirements.

The same principle applies as well to asphaltic concretes. In the latter the asphalt bodies, while distributed more uniformly through the fine mineral aggregate, are exceedingly small, and contraction is distributed on a large number of small bodies instead of concentrated on a few but large bodies. That alone would be a sufficient reason to employ a stiffer asphalt; however, other reasons and circumstances prompt the use of a stiffer asphalt. The principal reason is the function of the asphalt as a cement in uniting the mineral aggregate into a dense, stonelike mass. In addition there is the circumstance that the mineral aggregate is heated to substantially the same temperature as the asphalt before the materials are mixed, and this justifies the use of a stiffer asphalt. As the asphalt is exceedingly fluid at the proper working temperatures, it is possible to coat the mineral aggregate with a very thin film of asphalt instead of the comparatively heavy one which results in penetration macadam.

The bituminous materials used in penetration macadam are fluxed native asphalts, oil-asphalts (derived from the asphaltic-base oils), and tar products.

The suitability of a bituminous material for penetration work is measured by the test requirement for penetration in asphalts and the float test for tar products. Both tests determine the consistency of the bituminous material. For the northern section of the country, where very low temperatures may prevail for prolonged periods, an asphalt of a penetration of from 120 to 150 is ordinarily employed. That means, while the asphalt is not unreasonably soft during summer conditions, it retains a certain amount of plasticity during extremely cold weather. Furthermore, on account of its pliability it will reunite again when warm weather comes if there has been any separation of the asphalt bodies during the winter months.

Where extremely low temperatures do not occur or are of comparatively short duration the penetration requirement is placed at 90 to 120, and in the southern part of the United States, where freezing occurs but rarely, the penetration is from 80 to 90.

The corresponding float test requirements for tar products are as follows:

	Seconds.
For the northern section	90 to 120
For the middle belt	120 to 150
For the southern section	150 to 180

These test requirements should not be arbitrarily followed. The present tendency in the northern section is to select a somewhat stiffer material than that which is provided by the penetration test of 120 to 150 for asphalt or a corresponding float test of 90 to 120 seconds for tar products. The weight and amount of traffic should be one of the determining factors to be considered in selecting an appropriate consistency. Heavy and continuous traffic will prompt the use of a stiffer material than light traffic.

In specifications for the construction of penetration macadam, asphalts and tars are usually placed on an equal footing and the selection of the appropriate material is determined by local conditions. With the exception of one-size-stone bituminous concrete, where a tar product is occasionally employed, the bituminous material for the concretes is usually an asphalt.

QUALITY OF STONE ALSO IMPORTANT FACTOR.

Of equal importance with the asphalt is the quality and size of the stone aggregate from the viewpoint of service as well as workability when crushed. The adaptability of a rock is usually measured by its hardness and toughness, the latter quality being of greater importance than hardness alone. A very hard stone may also be friable under impact and may shatter under rolling or the subsequent impact of traffic much more easily than a softer stone of relatively greater toughness. The particular quality desired is frequently expressed by a test standard based upon the French coefficient of wear in which is expressed the results of a test which combines both hardness and toughness tests.

In a general way it may be stated that a rock composed of small crystals is better adapted for use than a rock composed of comparatively large crystals. The trap rocks and other related volcanic rocks are composed of comparatively small crystals. A number of granites and metamorphic gneisses derived from granites are usually not so well adapted on account of their large crystals. Where a choice of materials is to be had, economically, the best material should be employed.

However, there are large sections of the United States where the best rock material is unobtainable economically and where rock material of an inferior quality must be used. The rocks mentioned above show more specifically the desirable and undesirable structural features to be noted by a merely visual observation.

PENETRATION MACADAM.

In the construction of bituminous macadam roads the lower limits of the French coefficient are usually placed at 7 or 8. This requirement includes all the trap rocks and many granites, gneisses, sandstones, and quartzites. Limestones of various origins range from extremely high to extremely low. Where a rock with the minimum French coefficient of 7 is unobtainable, as happens in many of our States, the lower limit is sometimes placed as low as 4½. Where such a stone between 4½ and 7 must of necessity be employed, it is advisable to use a coarser stone aggregate than where a harder stone is employed.

Next in importance to the suitability of the rock as such is the size of the stone fragments. The latter must be large enough so that the voids contained in the stone bed after compaction are still large enough to permit the penetration of the bituminous material and the coating of the stone. The same principle is also carried out in the stone chips which take the place of screenings as used in water-bound macadam. The stone chips must be free from dust.

The effect of using an excessively friable rock for the stone aggregate manifests itself in the further crushing of the stone under rolling with a heavy roller and the consequent closing of the necessary passages. Hence the requirement to use a larger sized coarse aggregate where a soft stone is the only kind available is needed to mitigate somewhat the effect of the further crushing under the rolling.

As the chief reliance is placed upon the interlocking action of the stone to obtain stability, rolling must be done until full compression has been obtained and all lateral movement has ceased before the bituminous binder is applied.

The importance of adequate rolling can not be emphasized too much, as a large number of failures or, at least, very undesirable features in the finished pavement can only be assigned to insufficient rolling before applying the binder. Unless the coarse stone is rigidly keyed together to prevent lateral displacement the following application of bituminous material will have the effect of surrounding the stone with a lubricant which, under summer conditions and under traffic, favors the readjustment and constant displacement of the stone aggregate. This will produce ruts and other depressions and corresponding ridges over large parts of the road surface, which will not only make travel unpleasant but will also be destructive to vehicles. The condition of such a pavement becomes worse and worse in time as vehicular impact increases and the depressions increase in depth and the ridges in height.

After the first application of the bituminous binder stone chips are spread, and these stone chips when rolled tend still further to key the coarse stones together and prevent lateral displacement.

After rolling this course, and after sweeping off all unabsorbed stone fragments, a further application of bituminous material is made in an amount just sufficient to provide a thin coating of the tar or asphalt, which is immediately covered with stone chips and rolled. It is of the utmost importance that the stone should be reasonably clean and free from dust in order that the stone, whether large or small, may be properly coated with the binder. Dust, if in the form of a permanent coating on the stone, will prevent the adhesion of binder to the stone; if present as part of the stone due to crushing under the rolling or as bodies of fine material present in the coarse aggregate, it will prevent the uniform penetration of the binder. Such conditions if permitted will in time develop patches of unabsorbed bituminous material on the surface. An excessive amount of the tar or asphalt in the seal coat has the effect of forming a mat of stone ships and bituminous material which, being more or less pliable, develops alternate ridges and depressions at short intervals, giving the road surface a corrugated appearance. It frequently happens that the stone is placed some time in advance of the spreading of the bituminous binder and the stone becomes covered with fine roadside dust. The latter condition prevents the adhesion of the bituminous binder and only a liberal flushing with water will restore a condition that will permit the adhesion of the binder to the stone. This flushing should be done some time in advance of the spreading of the binder. A careful inspection of the stone bed is necessary to detect and remove pockets of fine stone or excessively crushed material and replace them with new stone properly compacted. Good workmanship and care in the various processes while simple is absolutely essential to obtain satisfactory results. While it is possible to spread the bituminous material by hand from pots it has become customary to make the application by means of specially constructed distributing wagons. The latter are tanks mounted, usually, on motor trucks and the distribution of the hot liquid is effected in a thin sheet under a constant air pressure. The latter insures a uniform application in a predetermined amount.

ASPHALTIC CONCRETES.

The true asphaltic concretes—that is, mixtures in which the coarse aggregate is united into a dense stone-like mass by a mortar composed of fine aggregate and asphaltic cement—are quite numerous. They include mixtures in which the coarse aggregate is comparatively large and constitutes about two-thirds of the entire mass, and mixtures in which the coarse aggregate is of a much smaller size and constitutes about one-third of the entire mass. These two represent the two extreme conditions, but there are other types, as well, in which the coarse aggregate constitutes intermediate percentages of the total aggregate with corresponding variations in the size of the coarse aggregate.

Asphaltic concretes occupy a middle ground between the asphalt macadams on the one side and sheet asphalt on the other, and are related to both.

Sheet asphalt is, par excellence, the recognized pavement in most of our large cities and sustains, with the exception of specialized heavy trucking, the comprehensive city traffic successfully. This success is due to the fact that a rational formula for the pavement has been developed and is understandingly enforced in most cases. As sheet asphalt is a pavement expensive in first cost, early attempts were made to reduce the cost by decreasing the depth of the pavement, hoping thereby to make it applicable to highways where the initial cost had to be considered to make them economically possible.

Without going into analytical detail the results of such changes were principally the addition of crushed stone to the surface course. In sheet asphalt, the mineral aggregate of the binder course is crushed stone; the mineral aggregate of the surface course is sand and a filler composed of a very finely ground mineral dust. The chief function of the binder course, as the name implies, is to bind the surface course to the base course and to prevent the horizontal displacement of the surface course. The interlocking quality of the coarse aggregate of the binder course has prevented such displacement measurably even where the sheet asphalt surface, as such has not been of the best possible composition.

By incorporating the coarse stone into the wearing surface of a sheet asphalt mixture, it was found to be possible to decrease the depth of the pavement and, consequently, its first cost. The asphaltic concrete type of pavement is the outcome of these modifications and when the fine grading of the mixture approximates the rational formula of sheet asphalt the results have been satisfactory. These results have, however, been more or less accidental and examples of poor asphaltic concretes of both the coarse and small-aggregate types are quite common. Such failures are principally of two kinds. In the case of coarse-aggregate asphaltic concrete, the life of the pavement is much shortened by a poor grading of the fine aggregate which, being porous, admits water and causes the early oxidation of the asphaltic cement. It also permits displacement of the aggregate and causes uneven surfaces. In the small-aggregate asphaltic concrete, the first condition also exists and the displacement and readjustment of the mineral aggregate is much more pronounced and produces typical surface irregularities manifesting themselves in depressions and adjoining elevations of some extent as well as in narrow alternating ridges of comparatively small height.

The underlying causes of these defects were early recognized in the small aggregate or Topeka type of pavements, and the grading of the material passing the 10-mesh sieve was modified in such a manner that the fine grading is now the same as in a standard sheet-asphalt pavement. This type is now known as modified Topeka pavement.

USE OF COARSE-AGGREGATE TYPE NOW UNRESTRICTED BY PATENTS.

In the coarse-aggregate type the chief reliance for stability has always been placed on the coarse stone, and, as the specifications providing for the grading of the aggregate were protected by a patent, not much could be done heretofore to improve the specifications in regard to the grading of the fine aggregate. With the expiration of these patents, the course-aggregate type of pavement is open to unrestricted use, and the engineer has it within his power to modify the specifications along rational lines.

The asphalts used in asphaltic concretes are substantially the same as those used in the macadams with the exception that the consistency is considerably stiffer. However, the climatic difference of the various sections is recognized in this type also, and the asphalts used in the northern section are softer than the ones used in the middle or southern belts. For the Topeka type of pavements the consistency of the asphalts is somewhat stiffer than for the coarse aggregate types in the corresponding climatic belts.

The mineral aggregate for asphaltic concrete is divided into coarse and fine. The coarse aggregate is composed of crushed stone, and the physical characteristics are measured by the same test standards as for macadam stone. It should be the best obtainable, economically, as the life of the pavement, apart from the grading, depends on the abrasive quality and toughness of the coarse material. As this type of pavement is usually laid to meet exacting traffic conditions, a stone as soft as is sometimes admitted in bituminous macadam without serious detriment should be rigidly excluded. The fine aggregate, for the purpose of proportioning, may be considered as all the material passing the 10-mesh sieve, and may to advantage include both screenings and sand as well as a fine mineral dust usually designated as "filler."

In combination with asphaltic cement and provided that the fine aggregate and dust are properly proportioned, it forms the mortar that fills the interstitial voids of the coarse aggregate in the completed pavement. In asphaltic concrete considerable dependence is placed upon the interlocking action of the coarse aggregate against displacement, and where the coarse aggregate constitutes nearly two-thirds of the entire mass of the pavement very little displacement need be apprehended; but the quality of the mortar determines whether the pavement is short-lived or whether its usefulness shall extend over many years.

In the Topeka type of pavement, where the coarse aggregate constitutes barely one-third of the mass of the pavement, this amount is not entirely sufficient to place the stones in close enough contact to prevent displacement, and the angular fragments of crushed rock passing the 10-mesh sieve supplement that essential quality measurably. While, therefore, screenings are not so essential in coarse-stone concrete, they should by all means be included in the fine aggregate of the Topeka types, and sand should be employed only to supplement the deficiency of fine aggregate.

It occasionally happens that a stone in crushing will produce only a small amount of material passing the 10-mesh sieve, in which event the deficiency would have to be made up with sand. In other cases that have come under the writer's observation the prevailing limestone rock was so soft that all the material passing the 10-mesh sieve was rejected as unsuitable. Such pavements are liable to distortion much more than where the screenings constitute a substantial part of the fine aggregate.

TOPEKA WITH BINDER COURSE UNECONOMICAL.

The expedient in such cases has been frequently to place the Topeka pavement on a binder course, resulting in a form of construction similar to sheet asphalt pavement. While this binder course may measurably prevent excessive displacement, the method appears scientifically and economically unsound. In this case the cost of the Topeka type of pavement would be substantially the same as that of a sheet asphalt pavement, while its period of usefulness must of necessity be much shorter owing to the greater abrasion of the soft stone in the Topeka than of the fine sand grains in the sheet asphalt, all other conditions being equal.

With the expiration of the patent rights above referred to, which determined the amount of coarse aggregate in the Topeka types at not exceeding 32 per cent by a decision of the United States courts, the percentage of coarse stone may now be safely increased and this would tend to produce greater stability. In fact, the coarse aggregate may now be proportioned to any intermediate percentage between 32 per cent of the modified Topeka and the maximum of about 67 per cent of the large aggregate types. With an increase in the percentage of stone the size of the stone should be correspondingly increased.

Another essential requirement for the fine aggregate is a finely ground mineral, usually consisting of ground limestone or Portland cement, although other ground minerals, if commercially available and of the required fineness may be used. The essential quality of this dust, usually designated as filler, in addition to fineness, is angularity, such as would be produced by grinding. Experience has shown that an equally fine material, produced by water or wind action, as occurs occasionally in fine sand, has not the essential qualities and should not, therefore, be considered as an equivalent or a substitute for ground dust. For that reason a limit, usually about 6 per cent, is placed upon sand particles passing the 200-mesh sieve, although as high as 12 and 15 per cent of ground dust, including fine sand passing the same mesh opening, may be required in the fine aggregate when properly combined. The reason for this requirement as well as a definite grading of the aggregate for asphaltic concretes is deserving of a more detailed explanation, as the success of these pavements depends essentially upon the quality of the asphaltic mortar in the concrete.

If we first consider the physical qualities of most sands, we find that they are composed of grains that may be more or less angular, but are in any event more or less rounded at the corners, due to the abrasive action of grain upon grain in the process of formation. Consequently, the grains move more or less freely against each other under pressure. This applies to sands of uniformly sized grains and sands that are composed of differently sized grains, not, however, with equal force. If these sand grains are bound together with asphalt cement they are held in position if no pressure is exerted. If pressure is exerted, particularly when the asphaltic cement becomes more or less plastic under summer temperatures, the sand granules are displaced and readjust themselves in new positions. If the sand is coarse, or of a too uniform size, the asphalt bodies are large and the readjustment of particles under pressure is excessive.

CHARACTER OF SAND GRADING RECOMMENDED.

If we conceive a sand grading in which the voids formed by adjacent sand particles are successively occupied by other sand particles decreasing correspondingly in size, and the smallest voids remaining are occupied by an angular dust, we have then a grading in which the voids have been eliminated as far as it is practicable to do so. Such a grading is used in the present standard sheet asphalt, in the material passing the 10-mesh sieve of a modified Topeka pavement, and in the fine mineral aggregate of a coarse-aggregate asphaltic concrete as required by the Bureau of Public Roads in its new typical specifications as well as in the specifications of the Asphalt Association.

If we further conceive the voids filled with asphaltic cement, the angular dust will largely prevent an excessive displacement or readjustment of the sand particles, even if the summer temperatures make the asphaltic cement more or less plastic.

Another very important effect of the density of this grading lies in the fact that it produces an essentially waterproof mortar which is, therefore, not affected by percolating water or air and the consequent weathering action on the asphaltic cement. The latter, in turn, retains its plastic and ductile qualities for much longer periods than it would in a porous mixture.

In essence the permanency of an asphaltic pavement and its resistance to displacement depends chiefly upon the rational grading of the mortar, and this applies with equal force to sheet asphalt where the entire paving surface is composed of this asphaltic mortar as well as to the asphaltic concretes where the mortar unites the coarse stone into a monolithic mass.

It follows that the grading requirement of specifications must be followed closely if success is to be obtained, and that disregard of such requirements either through ignorance or carelessness can only have disastrous effect, as is evidenced only too frequently and more particularly on country highways. In our larger cities the practice of building asphalt pavements appears to be much better understood. Their greater success is due to competent inspection and enforcement of specification requirements. This inspection is comprehensive in cities and covers every phase of manipulation. On country roads, on the other hand, inspections are frequently performed by men who have no understanding of the subject, and still more frequently the only attempt at control consists in forwarding a daily sample of the mixture to some testing laboratory. Constant inspection and control, both at the mixing plant and on the road, are essential and the inspection should be done by men experienced in this type of construction and who have a rational understanding of the requirements and are capable of making the comparatively simple tests required in the field.

The laboratory equipment at the plant to be used by the inspector should consist of the necessary laboratory sieves and scales so that the composition of the sand may be quickly determined and controlled. A penetrometer and an extractor may be desirable under certain conditions, and a thermometer is an essential part of the inspector's equipment.

CLOSE AND INTELLIGENT INSPECTION NECESSARY.

It is the function of the plant inspector to control the grading of the mineral aggregate, the heating of the materials, and the process of mixing. The various processes are usually described in considerable detail in most specifications and must be followed closely.

It is unusual that a sand from one source has the grading requirements for the fine aggregate of an asphaltic concrete, and for that reason two sands or even three sands of different mesh compositions must often be combined to obtain the desired grading.

As the physical work of shoveling sand into the heater is usually done by unskilled labor, this part of the work should be placed in charge of a competent foreman so that the sand from various stock piles may be fed into the heater in the correct proportions. The inspector will take samples from the hot sandbox at frequent intervals and sift the sand to control the grading and keep it within the limiting requirement.

Lack of care in this respect is probably the most prolific cause of early failure and lack of permanency of our asphaltic concrete pavements.

The temperature limits placed on the heating of the mineral aggregate and asphaltic cements have a number of reasons. If we consider first the asphalt cements, we find that the maximum temperature limit for native asphalts is placed at 350° F., for oil asphalts at 250° F. The requirement of a higher temperature for the native asphalts is for the reason that they are somewhat stiffer, at the same temperature, than the oil asphalts. However, if maintained at the higher temperature for prolonged periods or if the temperature be materially increased they harden much more rapidly than oil asphalts. It is, therefore, essential that the maximum temperature for the native asphalts be not exceeded. The oil asphalts are sufficiently fluid at the lower temperatures but an increase in temperature is not so detrimental. However, if a higher temperature is used it delays the rolling after the material is placed in the road as it must cool off sufficiently before effective rolling can be commenced.

With reference to the heating of the mineral aggregate, the same reasons apply both as to the workability of the mix in the mixer and rolling on the road. Overheating of the stone will have the same detrimental effect on the asphalt as if the asphalt itself had been overheated. Particular care is required where the coarse aggregate is composed of dolomitic limestone. This rock occasionally calcines at comparatively low temperatures that might conceivably have no serious effect if used in connection with an oil asphalt. However, cases have come under the writer's observation in which, after a comparatively short period, the coarse limestone disintegrated and slaked out of the pavement. While such occurrences are rare they will have to be considered.

ACCURATE PROPORTIONING A NECESSITY.

In combining the materials in the mixer care in weighing the materials is essential and this applies with particular force to asphaltic cement. Once the amount necessary to coat the mineral aggregate has been determined by computation and final experiment that amount should be correct for succeeding batches, provided the quality of the mineral aggregate also remains uniform, and it follows that the amount of asphalt should be weighed out carefully for each batch. Even a comparatively small increase or decrease has its detrimental effect and will manifest itself sooner or later, even if no very noticeable differences are apparent during the operation. In order to obtain a well-coordinated mixture, the materials should be combined by first mixing the mineral aggregate consisting of the coarse stone, the fine aggregate, and the mineral filler for a number of revolutions until thoroughly mixed before adding the asphaltic cement, after which the mixing should continue long enough to be sure that each particle is thoroughly and uniformly coated. A very useful field test is the so-called stain test in which a small amount of the mixture is compressed in a folded sheet of heavy manila wrapping paper. The resulting stain on the paper is an indication of the amount of asphalt in the mixture. This test while most useful in sheet asphalt is also applicable with some modifications to asphaltic concretes.

It is assumed that the mixing is done in a modern mixing plant designed to separate the screened stone and fine aggregate into separate compartments, whence they are combined by weight into the mixing box in which the combined materials are thoroughly mixed by rapidly revolving blades.

Cases have come under observation where a definite mixture was specified, but the contractor was permitted to use concrete mixers and rough field methods for combining the materials. The result of such methods can only produce uneven mixtures and lack of uniformity. Such pavement can not possibly conform to essential grading requirements and can not, therefore, have the permanency that should be expected.

It is customary and necessary, although frequently omitted, to have a daily sample of the finished mixture tested as to its composition. Such a test should be confirmatory of the predetermined composition of the mixture. This test is in a great many cases the only attempt made at control of the mixture, and if so used, it is of no particular value unless supplemented by intelligent inspection. The sample taken represents a daily output of from 800 to 1,800 square yards of pavement. The report of the test, which under the most favorable condition takes a number of hours if made at the plant itself and as many days perhaps if made at a testing laboratory at some distance from the work, ordinarily arrives too late to be of much immediate benefit. The plant inspector should be qualified and should have the necessary apparatus to make an extraction of asphalt from the finished mixture and make the sieve analysis of the composition of the sample. Only in such a case has the test any value, as a remedy can then be applied immediately. The subsequent test made at the testing laboratory would, in such a case, be a check of the test at the field laboratory.

It is customary particularly in the construction of country roads to obtain the asphaltic cement ready fluxed and in that case the penetration requirement can be checked a sufficient time in advance of the beginning of operations. In the event that the asphalt and flux are combined at the plant, the laboratory equipment should also include a penetrometer in order that an asphaltic cement of the required penetration may be prepared. The penetrometer is also useful if it is found that the ready-fluxed asphalt cement differs in the penetration from the specification requirement. In such a case the correction can be controlled by the addition of asphalt or flux as the case may be.

THE ROAD INSPECTOR'S FUNCTIONS.

The inspection on the road while of relatively less importance than at the plant is, however, usually done in some perfunctory manner. Even if the road inspection is competent, it is then too late to correct any serious errors made at the plant. The road inspector's function is primarily to see that the mixture is carefully placed and rolled—an important function, because the manipulation in the road determines very largely the behavior of the pavement under traffic. Of primary importance in placing the pavement is the requirement that the base course shall be clean and free of loose dirt or dust. As no binder course is required in asphaltic concrete the rolling is made difficult when loose dirt or dust favors the so-called shoving or pushing of the hot asphalt mixture under rolling. The presence of such foreign material often leads to checking and cracking of the surface under rolling, particularly in the Topeka types of pavement. When the pavement is to be laid on a broken stone or macadam foundation, it is particularly necessary not only that the stone course shall be consolidated well, but also that all free sand or dust shall be swept off in order that the mixture may come in immediate contact with the stone. Where the foundation is a cement concrete the latter should be slightly rough and if, as happens occasionally, a surface shows excessively shiny or smooth a thin paint coat of asphalt should be applied before placing the hot mixture.

When the mixture is spread, the load should be dumped sufficiently in advance of the face of the material already laid so that it will be necessary to shovel the whole load into position for raking. Only if the material is uniformly loose and of uniform depth is it possible for the rakers to produce a surface that will remain free from lumps due to uneven compression.

In order to have an effective control of the depth of the pavement the inspector should have the necessary information regarding the specific gravity or density of the pavement.

The density varies with different types of mixtures as well as with the varying density of the mineral aggregate and asphalt composing the mixture. The theoretical density is proportional to the volumetric percentage of the mineral aggregate and asphalt and their specific gravities. This density which should be determined by computation and test, ranges from about 2.25 for a sheet asphalt mixture to 2.60 for a coarse-aggregate type of asphaltic concrete, if made of trap rock.

While in practice these theoretical densities are not obtainable absolutely, the actual densities should not differ materially from the theoretical; otherwise it is a certain indication of a defective grading and implies generally an excessive porosity. If the mixture is delivered on the road in definite weights, as is customary, it is practicable to assign a definite space for each load, thus insuring uniform depth of pavement when ultimate compression has been obtained.

RAKING REQUIRES CONSIDERABLE SKILL.

Raking is work requiring considerable skill, and the function of the inspector is principally to control the depth of the pavement and the correction of defects that may occur under the subsequent rolling. The latter are principally checking (i. e. the opening of fine cracks), irregularity of compression, and irregularities of surface at points. These defects can ordinarily be remedied if attended to immediately. If hair cracks occur, which are usually due to creeping of the soft surface mixture on loose material under rolling, loosening and reraking will ordinarily be sufficient. In more severe cases the defective portion must be cut out and replaced after removing the responsible foundation defect.

The rolling of a Topeka type of pavement requires a tandem roller, and the rolling should be commenced as soon as it is practicable to do so. If the material is too hot when placed in the road there is too much delay, aside from possible defects due to burning; if too cold, the necessary compression and the maximum density can not be obtained. If the latter is obtained subsequently under road traffic it is at the expense of uniformity of surface.

After the first rolling, which is generally parallel to the axis of the road, the second rolling is diagonal on the more narrow country roads and cross rolling on city streets. This has the effect of ironing out ridges formed during the first rolling; it also introduces a kneading action necessary to produce the interlocking of the coarse aggregate. After the cross rolling and after all surface corrections have been made Portland cement is swept over the surface and the rolling continued. It should be pointed out here that the surface finish depends entirely upon rolling, also that one roller will only roll satisfactorily not much over 1,000 square yards of surface. If the plant capacity exceeds that amount per day, as is frequently the case, at least two rollers should be employed. Even where the smaller amount is laid each day, a second but smaller roller, would be of advantage, as the first rolling can be done somewhat sooner than if only one heavy roller is employed.

The requirement for efficient rolling is not enforced in many cases, and the requirement might reasonably be made plainer in many specifications.

(Continued on page 36.)

TESTS OF ROAD-BUILDING ROCK IN 1920.

THE following tables give the results of the physical tests of road-building rock made in the laboratories of the Bureau of Public Roads from January 1, 1920, to January 1, 1921. Samples of material from 33 States, the District of Columbia, Canada, and the West Indies are included in the list.

Taken with the preceding reports of tests made in other years, these annual reports of material tests constitute a record of the characteristics of road materials which grows more and more valuable year by year.

The results of all tests made up to January 1, 1916, were published in Department of Agriculture Bulletin No. 370, entitled "The Results of Physical Tests of Road-Building Rock"; those made during 1916 and 1917 were reported in Bulletin No. 670; the 1918 tests were recorded in Public Roads, volume 1, No. 11, issued March, 1919; the 1919 report was printed in Public Roads, volume 2, No. 23, issued in March, 1920; and the following are the results for 1920:

Results of physical tests of road-building rock from the United States, Canada, and the West Indies, Jan. 1, 1920, to Jan. 1, 1921.

Serial No.	Town or city.	County.	Name of material.	Crushing strength, pounds per square inch.	Weight per cubic foot.	Absorption, pounds per cubic foot.	Per cent of wear.	French coefficient of wear.	Hardness.	Toughness.
	ALABAMA.									
16112	(1)	Jackson	Crystalline limestone	(2)	174	0.57	3.8	10.5	15.3	8
16113	(1)	do	Argillaceous limestone	(2)	166	.34	6.2	6.4	18.0	6
16186	Athens	Limestone	Chert	(2)	146	4.10	10.7	3.7	(2)	(2)
16188	do	do	Dolomitic marble	(2)	168	.60	4.4	9.1	14.7	7
16189	do	do	Crystalline limestone	(2)	167	.79	5.3	7.6	13.3	5
16072	(1)	Morgan	Limestone	(2)	167	.50	3.4	11.8	15.3	8
16073	(1)	do	do	(2)	168	.45	3.5	11.4	15.3	9
16074	(1)	do	Crystalline limestone	(2)	165	.81	5.0	8.0	16.3	6
	ARKANSAS.									
17475	Heber Springs	Cleburne	Argillaceous sandstone	(2)	(2)	(2)	4.0	10.0	(2)	(2)
17310	Morrilton	Conway	Feldspathic sandstone	(2)	156	3.18	3.9	10.3	13.0	11
16383	Springfield	do	do	(2)	164	.80	3.4	11.8	(2)	(2)
16777	Morrilton	do	Sandstone	(2)	155	2.10	3.5	11.4	17.7	9
15386	(1)	Franklin	do	(2)	158	2.26	5.1	7.8	17.0	11
15143	Lamar	Johnson	Feldspathic sandstone	(2)	155	3.27	5.0	8.0	17.0	11
16510	do	do	do	(2)	156	2.57	3.8	10.5	13.0	(2)
17258	do	do	do	(2)	155	2.69	3.4	11.8	17.3	11
16657	Williford	Sharp	Dolomite	(2)	169	2.71	3.6	11.1	(2)	(2)
16488	Fayetteville	Washington	Weathered chert	(2)	(2)	(2)	2.2	14.0	(2)	(2)
15466	Plainview	Yell	Feldspathic sandstone	(2)	152	4.01	4.7	8.5	15.3	6
	COLORADO.									
15782	Boulder	Boulder	Sandstone	10,625	151	3.92	8.6	4.7	16.7	7
15783	do	do	do	19,110	150	3.53	6.7	6.0	16.7	7
15784	do	do	do	17,510	148	2.89	5.1	7.8	16.7	8
	DELAWARE.									
15401	(1)	Kent	Pyroxene quartzite	(2)	177	.18	3.3	12.1	18.7	13
15367	Wilmington	Newcastle	do	(2)	175	.69	3.3	12.1	18.7	12
	DISTRICT OF COLUMBIA.									
15795	Washington		Biotite granite	27,400	(2)	(2)	(2)	(2)	(2)	10
16314	do		Limestone	(2)	168	.45	4.4	9.1	(2)	(2)
	GEORGIA.									
17641	Elberton	Elbert	Biotite granite	28,360	165	.38	3.6	11.1	18.7	12
16655	(1)	Lawrens	Chert	(2)	150	4.03	8.2	4.9	(2)	6
15757	Tate	Pickens	Marble	(2)	168	.47	12.5	3.2	11.3	3
15378	Talbotton	Talbot	Diabase	(2)	187	.32	1.7	23.5	18.7	27
	IDAHO.									
17329	Filer	Twin Falls	Olivine basalt	(2)	175	1.34	4.6	8.7	16.3	12
	IOWA.									
15961	(1)	Story	Dolomite	(2)	(2)	(2)	6.4	6.3	(2)	(2)
15962	(1)	do	Limestone	(2)	(2)	(2)	3.4	11.8	(2)	(2)
	ILLINOIS.									
15938	Olive Branch	Alexander	Chert	(2)	148	1.97	9.7	4.1	(2)	(2)
16039	(1)	Cook	Dolomite	(2)	159	3.12	4.5	8.9	14.0	7
	INDIANA.									
16723	New Paris	Elkhart	Argillaceous limestone	(2)	166	.12	4.0	10.0	12.0	(2)
15374	Floyds Knobs	Floyd	Limestone	(2)	164	2.43	4.7	8.5	13.3	6
15376	do	do	Feldspathic sandstone	(2)	134	9.75	10.9	3.7	.0	3
15375	do	do	do	(2)	132	9.64	11.8	3.4	3.7	4
	KANSAS.									
15939	Sedan	Chautauqua	Argillaceous limestone	(2)	155	3.64	9.2	4.4	8.7	

[1] Exact locality not known.

[2] Test not made.

Results of physical tests of road-building rock from the United States, Canada, and the West Indies, Jan. 1, 1920, to Jan. 1, 1921—Continued

Serial No.	Town or city.	County.	Name of material.	Crushing strength, pounds per square inch.	Weight per cubic foot.	Absorption, pounds per cubic foot.	Per cent of wear.	French coefficient of wear.	Hardness.	Toughness.
	KENTUCKY.									
16746	Jackson	Breathitt	Feldspathic sandstone	(2)	153	4.62	8.6	4.7	(2)	(2)
16192	Princeton	Caldwell	Limestone	(2)	167	.47	6.0	6.7	(2)	(2)
17075	Olive Hill	Carter	Ferruginous sandstone	(2)	(2)	(2)	17.6	2.3	.7	(2)
17093	do	do	Limestone	(2)	167	.61	4.5	8.9	15.3	8
17158	do	do	do	(2)	167	.65	4.9	8.2	14.0	6
17060	do	do	do	(2)	169	.15	5.1	7.8	15.3	6
16637	Nicholasville	Jessamine	Argillaceous limestone	(2)	171	1.14	7.0	5.7	(2)	6
17257	do	do	Limestone	(2)	167	1.15	5.3	7.6	14.0	5
17061	East Bernstadt	Laurel	Feldspathic sandstone	(2)	143	4.62	9.9	4.0	.7	4
16778	Brackenburg	Meade	Argillaceous limestone	(2)	143	7.34	9.5	4.2	(2)	5
17011	Harrodsburg	Mercer	Crystalline limestone	(2)	169	.28	8.1	4.9	8.0	4
17658	do	do	Limestone	(2)	169	.20	6.0	6.7	16.7	5
17073	Livingston	Rockcastle	Siliceous dolomite	(2)	170	2.20	3.4	11.8	13.0	13
16313	(1)	Rowan	Calcareous sandstone	(2)	147	5.09	5.8	6.9	(2)	11
16416	Elkton	Todd	Limestone	(2)	163	.63	5.2	7.7	(2)	7
	MAINE.									
16304	Mount Desert	Hancock	Granite	(2)	164	.51	3.3	12.1	17.3	12
17459	Swans Island	do	Biotite granite	36,530	163	.55	3.4	11.8	(2)	(2)
17022	Clark Island	Knox	do	24,915	168	.63	2.6	15.4	19.0	11
17023	do	do	do	22,315	168	.65	2.6	15.4	18.7	12
17024	do	do	do	30,825	169	.53	2.5	16.0	18.0	12
	MARYLAND.									
15387	(1)	Baltimore	Marble	(2)	172	.30	5.4	7.4	(2)	(2)
16850	Ashland	do	Blast furnace slag	(2)	162	2.04	6.7	6.0	(2)	(2)
16851	do	do	do	(2)	138	9.91	19.0	2.1	15.3	6
16852	do	do	do	(2)	166	3.52	6.5	6.2	(2)	6
16853	do	do	do	(2)	175	.57	4.6	8.7	(2)	(2)
16196	Frederick	Frederick	Siliceous limestone	(2)	169	.25	2.9	13.8	16.7	11
17458	(1)	Howard	Hornblende gneiss	(2)	167	.78	4.6	8.7	17.3	9
	MASSACHUSETTS.									
15457	Pittsfield	Berkshire	Marble	(2)	172	.65	5.1	7.8	13.3	7
17198	Winchester	Middlesex	Hornblende gabbro	(2)	189	.20	3.0	13.3	18.0	13
17503	Southbridge	Worcester	Biotite schist	(2)	172	.62	4.3	9.3	16.3	11
	MICHIGAN.									
15771	Scofield	Monroe	Dolomite	(2)	163	2.26	5.2	7.7	13.3	7
	MISSISSIPPI.									
16131	Chunkey	Newton	Sandstone	(2)	124	8.96	9.8	4.1	3.3	5
	MISSOURI.									
15444	(1)	Cape Girardeau	Limestone	(2)	168	.37	4.3	9.3	17.0	5
	MONTANA.									
15898	(1)	Deer Lodge	Quartzite	(2)	162	1.10	5.6	7.1	(2)	(2)
16155	(1)	do	do	(2)	163	.34	4.3	9.3	(2)	(2)
15955	East Helena	Lewis and Clark	Smelter slag	(2)	212	1.03	5.9	6.8	17.3	12
15899	(1)	Wibaux	Burnt shale	(2)	122	13.77	23.8	1.7	(2)	(2)
	NEBRASKA.									
16080	Louisville	Cass	Argillaceous limestone	(2)	163	2.44	5.8	6.9	14.7	6
16081	do	do	Ferruginous clay	(2)	130	23.50	18.2	2.2	(2)	(2)
	NEW JERSEY.									
16027	(1)	Hunterdon	Dolomitic marble	(2)	177	.38	3.5	11.4	15.6	11
16029	(1)	do	do	(2)	177	.48	3.4	11.8	16.0	14
	NEW MEXICO.									
16202	(1)	San Miguel	Siliceous clay limestone	(2)	166	.42	4.6	8.7	16.7	8
17292	North Laguna	Valencia	Calcareous sandstone	(2)	152	3.92	6.1	6.6	11.3	7
	NEW YORK.									
16543	Akron	Erie	Limestone	(2)	168	.19	5.4	7.4	(2)	(2)
16544	(1)	do	do	(2)	164	.85	6.2	6.5	16.0	11
16154	Jay	Essex	Gabbro	(2)	179	.31	2.7	14.8	17.6	10
17327	Albion	Orleans	Sandstone	(2)	152	3.07	3.9	10.3	14.7	10
17328	do	do	do	(2)	150	3.56	4.0	10.0	15.0	10
16032	(1)	Rockland	Argillaceous dolomite	(2)	173	.50	7.8	5.1	(2)	(2)
	NORTH CAROLINA.									
15877	Haw River	Alamance	Diabase	(2)	181	.53	3.5	11.4	18.0	16
17048	Jefferson	Ashe	Hornblende gneiss	(2)	185	.89	5.2	7.7	12.0	6
16071	Granite Falls	Caldwell	Biotite gneiss	(2)	166	.76	3.1	12.9	16.7	11
16514	Stokesland	Caswell	Granite gneiss	(2)	162	.76	3.7	10.8	(2)	12
17466	(1)	Chatham	Altered andesite	(2)	180	.20	1.7	23.5	17.7	45
16896	Winston-Salem	Forsyth	Biotite gneiss	(2)	170	.43	4.3	9.3	16.7	8
15775	Greensboro	Guilford	Quartz and quartzite	(2)	164	.18	8.6	4.7	(2)	(2)
15878	(1)	do	Amphibolite	(2)	190	.42	2.8	14.3	18.7	18
16536	Littleton	Halifax	Hornblende quartzite	(2)	170	.35	2.7	14.6	(2)	17
17480	Hendersonville	Henderson	Gneissoid granite	(2)	(2)	(2)	3.4	11.8	(2)	(2)
16869	(1)	Mitchell	Granite gneiss	(2)	168	.37	3.7	10.8	18.3	9
17465	Swift Island Ferry	Montgomery	Amphibolite	(2)	185	.98	3.8	10.5	18.0	17
15442	Monroe	Union	Siliceous slate	(2)	173	.60	3.0	13.3	12.3	10
15474	(1)	Vance	Gneissoid granite	(2)	167	.51	4.0	10.0	17.7	6
16004	Raleigh	Wake	Hornblende granite	(2)	168	.65	4.9	8.2	18.7	9
16256	Manson	Warren	Gneissoid granite	(2)	164	.76	4.2	9.5	17.0	6

[1] Exact locality not known.

[2] Test not made.

Results of physical tests of road-building rock from the United States, Canada, and the West Indies, Jan. 1, 1920, to Jan. 1, 1921—Continued.

Serial No.	Town or city.	County.	Name of material.	Crushing strength, pounds per square inch.	Weight per cubic foot.	Absorption, pounds per cubic foot.	Per cent of wear.	French coefficient of wear.	Hardness.	Toughness.
	OHIO.									
16722	(1)	Adams	Limestone	(2)	173	0.60	5.5	7.3	11.0	(2)
16660	Springfield	Clark	Micaceous sandstone	(2)	141	7.30	19.6	2.0	(2)	3
16728	(1)	Clermont	Limestone	(2)	168	.78	10.0	4.0	11.7	6
17062	Miamiville	do	do	18,575	171	.17	5.5	7.3	13.0	5
15464	(1)	Defiance	Shale	(2)	157	.45	10.0	4.0	(2)	6
15888	Marble Cliff	Franklin	Argillaceous limestone	25,900	166	1.52	4.7	8.5	15.3	7
16340	Franklin	do	do	(2)	167	1.39	4.3	9.3	(2)	7
16347	Columbus	do	Blast furnace slag	(2)	139	5.44	12.1	3.3	(2)	4
16418	Franklin	do	Argillaceous limestone	(2)	162	1.14	5.4	7.4	(2)	(2)
17079	Columbus	do	Limestone	(2)	165	1.44	4.0	10.0	11.7	6
17411	Marble Cliff	do	do	(2)	166	1.50	4.5	8.9	(2)	(2)
16417	Kenton	Hardin	Dolomite	(2)	168	2.12	2.9	13.9	(2)	(2)
16939	(1)	Highland	Argillaceous dolomite	(2)	163	3.91	5.3	7.5	3.7	5
15996	Ironton	Lawrence	Blast furnace slag	(2)	148	4.08	7.2	5.6	18.0	9
15997	do	do	do	(2)	153	3.47	7.1	5.6	(2)	(2)
16747	Waterville	Lucas	Magnesian limestone	(2)	163	2.07	6.9	5.8	4.3	5
16290	Youngstown	Mahoning	Blast furnace slag	(2)	201	3.07	12.8	3.1	(2)	(2)
17505	do	do	do	(2)	(2)	(2)	26.0	1.6	(2)	(2)
17504	do	do	do	(2)	(2)	(2)	20.1	2.0	(2)	(2)
16171	(1)	Morgan	Feldspathic sandstone	(2)	143	6.78	7.7	5.2	0.0	4
16173	(1)	do	do	(2)	138	7.61	10.5	3.8	0.0	4
16922	Malta	do	Limestone	(2)	168	.26	6.5	6.2	12.7	6
17389	do	do	Argillaceous limestone	(2)	164	1.32	4.7	8.5	15.3	7
16547	Woodville	Sandusky	Dolomite	(2)	162	3.53	5.8	6.9	(2)	6
	OKLAHOMA.									
16134	(1)	Bryan	Limestone	(2)	166	1.46	4.9	8.2	15.7	6
16135	(1)	do	Fossiliferous sandstone	(2)	167	1.20	4.9	8.2	14.7	7
16136	Durant	do	Calcareous sandstone	(2)	154	5.32	6.8	5.9	11.3	7
16170	(1)	do	Argillaceous limestone	(2)	158	3.69	6.1	6.6	14.7	6
17057	Gowen	Latimer	Sandstone	(2)	143	5.02	15.8	2.5	(2)	(2)
16050	(1)	Le Flore	do	(2)	156	1.72	2.3	17.4	18.3	13
16060	Wister	do	do	(2)	159	1.88	1.7	23.5	18.3	12
16062	do	do	do	(2)	155	2.28	4.9	8.2	(2)	(2)
15957	Hartshorne	Pittsburg	Limestone	(2)	168	.49	4.1	9.8	15.7	5
15510	Garnett	Tulsa	do	(2)	(2)	(2)	4.2	9.5	(2)	(2)
16586	Sand Springs	do	do	(2)	159	2.63	8.2	4.9	11.0	5
15788	Tulsa	do	Argillaceous limestone	(2)	161	1.93	5.8	6.9	13.0	5
15789	do	do	do	(2)	164	1.07	5.7	7.0	12.7	5
15790	Lost City	do	Limestone	(2)	163	3.03	5.7	7.0	(2)	(2)
15791	do	do	do	(2)	161	2.50	7.4	5.4	13.3	4
16825	(1)	do	Argillaceous limestone	(2)	163	1.59	6.0	6.7	12.0	6
16826	Sand Springs	do	Limestone	(2)	166	.66	5.1	7.8	13.3	7
	PENNSYLVANIA.									
16012	(1)	Adams	Diabase	(2)	189	.53	1.8	22.2	18.7	34
16077	(1)	Armstrong	Limestone	19,470	161	.88	4.2	9.5	15.3	6
15989	Tyrone	Blair	Chert	(2)	(2)	(2)	3.0	13.3	(2)	(2)
15987	Rock Hill	Bucks	Diabase	(2)	(2)	(2)	2.3	17.4	(2)	(2)
15440	Donaghmore	Lebanon	Blast furnace slag	(2)	123	8.50	8.8	4.5	(2)	(2)
15441	Hokendaqua	Lehigh	do	(2)	130	7.70	17.1	2.3	12.3	3
16587	Green Lane	Montgomery	Siliceous slate	(2)	172	.38	4.2	9.5	18.0	10
15439	Bethlehem	Northampton	Blast furnace slag	(2)	139	5.19	20.2	2.0	12.3	3
	SOUTH CAROLINA.									
15453	Trenton	Edgefield	Biotite granite	(2)	164	.74	3.2	12.5	(2)	(2)
	SOUTH DAKOTA.									
16017	(1)	Tripp	Sandstone	(2)	149	1.06	9.3	4.3	16.3	6
16018	(1)	do	do	(2)	148	1.52	8.6	4.7	(2)	(2)
16019	Colome	do	Siliceous limestone	(2)	144	4.19	24.8	1.6	5.3	5
16465	do	do	Tuffaceous limestone	(2)	138	6.12	10.9	3.7	(2)	7
	TENNESSEE.									
15986	Erin	Houston	Siliceous sinter	(2)	90	21.32	43.1	.9	(2)	(2)
16275	Iron City	Lawrence	Argillaceous limestone	(2)	168	.47	5.0	8.0	15.0	8
17507	Clarksville	Montgomery	Limestone	(2)	158	2.05	4.9	8.2	9.7	5
15897	Copper Hill	Polk	Smelter slag	(2)	(2)	(2)	5.5	7.3	(2)	(2)
	TEXAS.									
16633	Blanket	Brown	Argillaceous limestone	(2)	162	1.42	6.7	6.0	(2)	5
17633	Fredericksburg	Gillespie	Limestone	(2)	(2)	(2)	6.9	5.8	(2)	(2)
16590	(1)	Harrison	Ferruginous sandstone	(2)	165	4.72	21.3	1.9	(2)	11
16828	Grosbeck	Limestone	Siliceous limestone	(2)	158	1.32	6.0	6.7	14.0	8
17497	Llano	Llano	Rhyolite	(2)	164	.38	2.3	17.4	19.3	21
17442	Madisonville	Madison	Sandstone	(2)	135	6.62	5.8	6.9	13.0	9
17074	Starville	Smith	Ferruginous sandstone	(2)	177	.41	5.7	7.0	17.3	11
17080	Friendship	do	do	(2)	166	8.47	27.3	1.5	12.0	4
17020	San Angelo	Tom Green	Limestone conglomerate	(2)	165	1.21	5.6	7.1	17.6	5
	VERMONT.									
16352	Brandon	Rutland	Marble	(2)	(2)	(2)	(2)	(2)	12.6	(2)
	VIRGINIA.									
15887	Charlottesville	Albemarle	Amphibolite	(2)	182	1.47	6.8	5.9	15.0	7
16454	(1)	do	Gneiss	(2)	169	.77	5.8	6.9	(2)	12
17332	Langdale	Alleghany	Slag	(2)	152	1.86	12.0	3.3	(2)	(2)
16641	(1)	Augusta	Limestone	(2)	172	.48	5.5	7.3	15.3	10
16776	(1)	do	Argillaceous dolomite	(2)	174	.34	2.8	14.3	16.7	19
15227	(1)	Bedford	Hornblende gneiss	(2)	187	.73	10.0	3.9	16.7	4

[1] Exact locality not known.

[2] Test not made.

Results of physical tests of road-building rock from the United States, Canada, and the West Indies, Jan. 1, 1920, to Jan. 1, 1921—Continued.

Serial No.	Town or city.	County.	Name of material.	Crushing strength, pounds per square inch.	Weight per cubic foot.	Absorption, pounds per square foot.	Per cent of wear.	French coefficient of wear.	Hardness.	Toughness.
	VIRGINIA—Continued.									
16228	(1)	Bedford	Hornblende schist	(2)	192	0.82	5.6	7.2	16.0	14
16440	(1)	do	(3)	(2)	187	.68	10.3	3.9	(2)	8
16686	(1)	do	Siliceous dolomite	(2)	176	.32	4.4	9.1	(2)	12
16648	Buchanan	Botetourt	Calcareous slate	(2)	170	.23	5.5	7.3	(2)	(2)
15452	Saltpetre Cave	do	Dolomite	(2)	179	.50	6.7	6.0	16.7	7
16307	Lynchburg	Campbell	Dolomitic marble	(2)	172	.24	5.0	8.0	(2)	7
16783	do	do	Hornblende schist	(2)	186	.52	6.9	5.8	17.0	9
17474	(1)	Caswell	Granite gneiss	(2)	162	.75	4.5	8.9	18.0	10
16190	Clintwood	Dickenson	Calcareous sandstone	(2)	158	2.47	5.0	8.0	17.3	7
16191	do	do	Feldspathic sandstone	(2)	158	3.03	8.2	4.9	11.3	6
16729	Fairfax	Fairfax	Mica gneiss	(2)	172	1.85	9.8	4.1	(2)	6
17231	do	do	Amphibolite	(2)	174	1.32	9.1	4.4	6.3	6
17699	do	do	do	(2)	185	.72	4.1	9.8	14.0	7
17700	do	do	do	(2)	187	.51	3.4	11.8	17.7	18
15767	(1)	Fauquier	Chlorite Talc. schist	(2)	180	.81	3.9	10.3	16.7	12
15768	(1)	do	do	(2)	177	1.52	10.7	3.7	(2)	(2)
15769	(1)	do	Chlorite schist	(2)	181	.72	11.7	3.4	(2)	(2)
15770	(1)	do	Chlorite epidote schist	(2)	184	.52	3.2	12.5	17.3	24
15896	(1)	do	do	(2)	187	.48	4.8	8.3	17.0	10
16030	Marshall	do	Altered granite	(2)	164	.65	3.8	10.5	18.7	10
16226	(1)	do	Altered basalt	(2)	175	.81	3.0	13.3	17.3	8
16308	The Plains	do	Hornblende epidote schist	(2)	185	.40	9.9	4.1	(2)	(2)
16309	do	do	Chlorite epidote schist	(2)	184	.72	5.2	7.7	(2)	(2)
16310	do	do	Hornblende epidote schist	(2)	190	.42	6.2	6.5	(2)	22
16312	do	do	(3)	(2)	187	.44	6.4	6.3	(2)	(2)
16626	do	do	Chlorite epidote schist	(2)	183	.39	4.4	9.1	(2)	31
17086	do	do	do	(2)	187	.24	5.3	7.5	15.7	8
15485	Strathmore	Fluvanna	Chlorite schist	(2)	185	.33	3.3	12.1	15.0	10
16519	Richmond	Henrico	Limestone	(2)	167	.94	6.9	5.8	(2)	5
16988	(1)	do	Biotite granite	(2)	164	.47	3.4	11.8	18.0	10
15487	Lucketts	Loudoun	Limestone conglomerate	(2)	175	.39	3.8	10.5	17.0	10
15488	do	do	Calcareous sandstone	(2)	151	2.82	3.4	11.8	15.7	17
15489	Leesburg	do	Chlorite epidote schist	(2)	188	1.20	12.3	3.3	(2)	8
15490	do	do	Crystalline limestone	(2)	172	.92	6.3	6.4	13.7	6
16031	Lucketts	do	Conglomerate	(2)	187	.63	5.8	6.9	(2)	(2)
16047	Belmont Park	do	Gabbroitic diabase	(2)	(2)	(2)	2.2	18.2	(2)	(2)
16048	do	do	(3)	(2)	(2)	(2)	2.3	17.4	(2)	(2)
16257	Lucketts	do	Limestone conglomerate	(2)	175	.28	4.1	9.8	(2)	(2)
16414	do	do	do	(2)	174	.33	4.3	9.3	(2)	9
17087	do	do	do	(2)	175	.35	5.9	6.8	(2)	(2)
15994	Ryans Mill	Nelson	Altered gneiss	(2)	171	1.60	9.3	4.3	18.3	11
15995	do	do	Amphibolite	(2)	192	.61	2.7	14.8	16.3	13
17944	Compton	Page	Feldspathic sandstone	(2)	148	5.55	18.0	2.2	8.3	4
15786	(1)	Pulaski	Limestone	(2)	167	.71	6.8	5.9	15.0	7
15919	Draper	do	Siliceous limestone	(2)	168	.35	5.3	7.5	16.3	(2)
16025	do	do	Argillaceous limestone	(2)	170	.24	5.4	7.4	15.3	7
16026	do	do	Siliceous clay limestone	(2)	165	.20	4.6	8.7	14.7	11
16223	(1)	Rockbridge	Siliceous limestone	(2)	169	.26	5.0	8.0	13.3	17
17092	(1)	do	Limestone	(2)	169	2.10	6.0	6.7	12.6	5
16513	Harrisonburg	Rockingham	Argillaceous limestone	(2)	169	.23	5.7	7.0	(2)	5
16110	Atkins	Smyth	do	(2)	173	.38	2.8	14.3	16.3	7
16011	do	do	Dolomite	(2)	173	.81	6.7	6.0	18.0	7
16262	do	do	Siliceous dolomite	(2)	174	.67	4.7	8.6	(2)	10
16287	(1)	do	Dolomite	(2)	172	.79	7.0	5.7	(2)	5
16289	(1)	do	Quartzite	(2)	162	1.28	4.5	8.9	(2)	(2)
16507	Atkins	do	Limestone	(2)	167	9.02	5.6	7.1	(2)	9
16508	Groseclose	do	Argillaceous dolomite	(2)	173	1.41	9.0	4.4	(2)	5
16509	Atkins	do	Dolomite	(2)	174	.74	5.8	6.9	(2)	7
16589	(1)	do	Limestone	(2)	169	.58	5.4	7.4	13.7	5
16021	Holston	Washington	Argillaceous limestone	(2)	168	.15	4.0	10.0	16.9	10
16075	do	do	Siliceous limestone	(2)	168	.58	5.7	7.0	17.3	8
16178	do	do	Argillaceous limestone	(2)	169	.55	6.4	6.3	10.0	6
16421	(1)	Wise	do	(2)	170	.17	4.9	8.1	(2)	5
16111	(1)	Wythe	do	(2)	169	.23	4.7	8.5	16.7	7
	WEST VIRGINIA.									
15486	Clarksburg	Harrison	Feldspathic sandstone	(2)	154	4.34	4.7	8.5	10.0	8
15988	Kearneysville	Jefferson	Limestone	(2)	(2)	(2)	4.8	8.3	(2)	(2)
16209	do	do	Argillaceous limestone	(2)	169	.12	4.6	8.7	14.7	6
16384	Princeton	Mercer	Sandstone	(2)	156	2.80	3.9	10.3	(2)	8
15812	Union	Monroe	Siliceous limestone	(2)	170	.20	5.3	7.5	16.3	8
15813	do	do	do	(2)	171	.27	4.6	8.7	16.0	9
15814	do	do	do	(2)	170	.19	2.9	13.8	17.3	13
16146	Surveyor	Raleigh	Sandstone	(2)	156	2.64	3.4	11.8	17.7	10
16203	Cranberry	do	Feldspathic sandstone	(2)	159	2.19	8.0	5.0	11.3	5
16326	Beckley	do	do	(2)	151	3.47	5.1	7.8	(2)	7
16464	Price Hill	do	Sandstone	(2)	152	2.74	6.4	6.3	(2)	7
16487	Prosperity	do	do	(2)	156	1.96	4.4	9.1	16.3	10
	CANADA.									
16923	Dundas	Ontario Province	Dolomite	(2)	(2)	(2)	4.1	9.8	15.0	13
16924	do	do	do	(2)	(2)	(2)	4.3	9.3	14.0	13
	DOMINICAN REPUBLIC.									
15864	Dominican Republic	Santo Domingo Province	Crystalline limestone	(2)	143	5.01	24.0	1.7	(2)	(2)
15865	do	do	Fragmental basalt	(2)	177	.97	3.5	11.4	18.7	11
15866	do	do	Tuffaceous limestone	(2)	155	3.01	11.2	3.6	(2)	(2)
15867	do	do	Crystalline limestone	(2)	170	.27	(2)	(2)	8.5	6
16101	do	do	Argillaceous limestone	(2)	178	4.13	6.3	6.4	10.7	5
16103	do	do	Altered basalt	(2)	179	1.14	3.5	11.4	16.7	8
16104	do	do	Altered andesite	(2)	177	1.26	3.9	10.3	13.7	10

[1] Exact locality not known. [2] Test not made. [3] Classification not known.

STATUS OF FEDERAL AID, APRIL 30.

UNDER construction and completed on April 30 there were 22,210 miles of Federal aid roads, almost enough if connected into a single road to girdle the earth. About a quarter of this mileage was in projects which had been entirely completed and the balance in projects still under construction. A fair idea of the progress which had been made on the uncompleted roads is to be gained from the fact that the reports of the district engineers indicated that 50 per cent of the Federal money set aside for them had been earned by the States.

Minnesota with 597 miles leads all the States in respect to length of completed projects with Texas a close second. In amount of work under contract Texas leads with 1,622 miles, followed by Nebraska with 1,171 and Minnesota with 1,147.

Of the total apportionment to all the States amounting to $266,750,000 for the five fiscal years 1917 to 1921, inclusive, almost two-thirds had been placed under contract on April 30; and there was a balance of slightly more than $100,000,000 available for new contracts. If contracts are let during the present season at the rate at which they were handled last year, the last project to participate in the present appropriation will be contracted for by the end of May, 1922. It is probable, however, that the average rate of 7¾ millions

TABLE 1.—*Financial statement as of April 30, 1921.*

State.	Total apportionment of Federal aid.	Federal aid in work under construction. and completed.[1]	Federal aid available for new contracts.	Federal aid in completed work.[2]	Federal aid in uncompleted work on projects under construction.	Amounts Federal aid paid States.	Balance Federal aid earned by States.	State.
Alabama	$5,776,552	$1,834,410	$3,942,143	$1,535,465	$298,945	$1,286,318	$249,147	Alabama.
Arizona	3,771,351	2,800,686	970,666	2,159,902	640,784	1,074,920	1,084,982	Arizona.
Arkansas	4,619,929	2,913,773	1,706,157	1,732,309	1,181,464	836,624	895,685	Arkansas.
California	8,384,354	3,445,208	4,939,147	2,554,064	891,144	1,855,995	698,069	California.
Colorado	4,780,064	2,070,746	2,709,318	1,453,028	617,718	1,251,301	201,727	Colorado.
Connecticut	1,689,324	981,823	707,502	303,782	678,041	168,647	135,135	Connecticut.
Delaware	447,655	447,655	....	399,055	48,600	308,035	91,020	Delaware.
Florida	3,150,112	2,605,601	544,511	1,081,878	1,523,723	363,137	718,741	Florida.
Georgia	7,407,579	6,777,174	630,405	4,554,152	2,223,022	3,521,735	1,032,417	Georgia.
Idaho	3,360,389	3,043,750	316,639	2,008,016	1,035,734	1,345,682	662,334	Idaho.
Illinois	12,024,267	11,603,774	420,493	8,220,428	3,383,346	6,498,117	1,722,311	Illinois.
Indiana	7,415,293	2,695,235	4,720,058	1,275,815	1,419,420	978,058	297,757	Indiana.
Iowa	7,939,343	6,340,334	1,599,009	3,713,051	2,627,283	1,505,000	2,208,051	Iowa.
Kansas	7,895,309	4,888,166	3,007,143	2,521,162	2,367,004	1,398,772	1,122,390	Kansas.
Kentucky	5,370,065	2,775,497	2,594,568	1,408,326	1,367,171	1,085,455	322,871	Kentucky.
Louisiana	3,742,525	3,730,264	12,261	1,957,762	1,772,502	1,576,785	380,977	Louisiana.
Maine	2,645,964	1,506,822	1,139,142	793,828	712,994	480,795	313,033	Maine.
Maryland	2,390,749	2,030,730	360,019	1,701,360	329,370	1,255,185	446,175	Maryland.
Massachusetts	4,052,565	1,941,713	2,110,852	1,236,512	705,201	953,095	283,417	Massachusetts.
Michigan	7,961,296	4,374,294	3,587,002	2,206,712	2,167,582	1,411,716	794,996	Michigan.
Minnesota	7,815,383	6,813,159	1,002,224	4,881,404	1,931,755	4,158,337	723,067	Minnesota.
Mississippi	4,951,542	2,099,391	2,852,151	1,225,077	874,314	827,275	397,802	Mississippi.
Missouri	9,322,076	4,536,874	4,785,202	1,935,830	2,601,044	607,947	1,327,883	Missouri.
Montana	5,498,827	3,311,458	2,187,359	2,207,418	1,104,050	1,625,606	581,812	Montana.
Nebraska	5,866,762	3,256,565	2,610,197	2,956,501	300,064	2,078,253	878,248	Nebraska.
Nevada	3,527,276	1,258,384	2,268,892	1,010,040	248,344	773,123	236,917	Nevada.
New Hampshire	1,143,089	894,290	248,799	798,488	95,802	780,379	18,109	New Hampshire.
New Jersey	3,265,299	1,421,024	1,844,275	1,143,795	277,229	856,626	287,169	New Jersey.
New Mexico	4,389,795	2,044,490	2,345,305	1,313,619	730,871	994,124	319,495	New Mexico.
New York	13,688,802	2,066,838	11,621,964	277,394	1,789,444	195,916	81,478	New York.
North Carolina	6,270,691	5,771,437	499,254	3,574,083	2,197,354	2,342,898	1,231,185	North Carolina.
North Dakota	4,222,488	1,707,908	2,514,580	1,216,871	491,037	726,032	490,839	North Dakota.
Ohio	10,202,948	5,361,739	4,841,209	3,108,805	2,252,934	2,703,181	405,624	Ohio.
Oklahoma	6,338,246	3,167,629	3,170,617	1,328,696	1,838,933	417,221	911,475	Oklahoma.
Oregon	4,332,178	4,130,684	201,494	2,935,649	1,195,035	2,324,433	611,216	Oregon.
Pennsylvania	12,632,644	10,767,327	1,865,317	7,564,709	3,202,618	5,951,688	1,613,021	Pennsylvania.
Rhode Island	641,166	450,081	191,085	265,050	185,031	265,050	....	Rhode Island.
South Carolina	3,946,618	2,087,615	1,859,003	1,470,111	617,504	1,022,318	447,793	South Carolina.
South Dakota	4,452,883	2,968,993	1,483,890	1,444,285	1,524,708	570,211	874,074	South Dakota.
Tennessee	6,228,138	4,193,581	2,034,557	1,234,554	2,959,027	407,826	826,728	Tennessee.
Texas	16,100,405	9,783,444	6,316,961	5,651,567	4,131,877	2,873,170	2,778,397	Texas.
Utah	3,117,206	2,505,621	611,585	797,905	1,707,716	430,519	367,386	Utah.
Vermont	1,242,104	500,379	741,725	184,823	315,556	164,435	20,388	Vermont.
Virginia	5,451,730	2,946,337	2,505,393	2,167,904	778,433	1,120,335	1,047,569	Virginia.
Washington	3,971,676	3,885,517	86,159	3,661,833	223,684	3,324,632	337,201	Washington.
West Virginia	2,922,504	2,902,504	20,000	1,762,865	1,139,639	1,282,609	480,256	West Virginia.
Wisconsin	7,004,281	3,515,455	3,488,826	2,177,617	1,337,838	1,800,100	377,517	Wisconsin.
Wyoming	3,378,558	2,695,270	683,283	1,808,156	887,114	1,423,054	385,102	Wyoming.
	266,750,000	165,851,659	100,898,341	102,921,656	62,930,003	71,202,670	31,718,986	

[1] Includes projects entirely completed and paid for.

[2] Includes completed portions of projects under construction.

NOTE.—Work was put under contract by the States during 1920 at the approximate average rate of 7¾ million dollars a month; during month of April, 1921, approximately $6,452,389 was put under contract by the States.

TABLE 2.—*Status of construction work, April 30, 1921.*

State.	Projects under construction.					Projects on which construction is completed.			State.
	Total estimated cost.	Federal aid.	Miles.	Per cent complete.	Federal aid earned.	Total estimated cost.	Federal aid.	Miles.	
Alabama	$1,293,257	$622,803	98	52	$323,858	$2,471,168	$1,211,607	285	Alabama.
Arizona	3,813,405	1,686,275	192	62	1,045,491	2,282,569	1,114,411	117	Arizona.
Arkansas	9,142,991	2,747,590	798	57	1,566,126	413,096	166,183	67	Arkansas.
California	3,535,687	1,767,837	164	50	876,693	3,388,146	1,677,371	187	California.
Colorado	2,990,627	1,464,613	203	58	846,895	1,234,787	606,133	89	Colorado.
Connecticut	2,433,932	928,823	51	27	250,782	120,232	53,000	5	Connecticut.
Delaware	210,143	54,000	6	10	5,400	1,719,668	393,655	28	Delaware.
Florida	5,275,301	2,497,906	138	39	974,183	215,389	107,695	27	Florida.
Georgia	10,618,384	4,729,833	863	53	2,506,811	4,400,785	2,047,341	222	Georgia.
Idaho	5,042,490	2,353,940	287	56	1,318,206	1,397,135	689,810	149	Idaho.
Illinois	16,264,716	7,048,638	456	52	3,665,292	9,292,755	4,555,136	286	Illinois.
Indiana	4,971,694	2,447,276	137	42	1,027,856	505,752	247,959	13	Indiana.
Iowa	14,290,668	5,711,484	807	54	3,084,201	1,538,376	628,850	152	Iowa.
Kansas	15,118,736	4,383,341	395	46	2,016,337	1,470,659	504,825	32	Kansas.
Kentucky	5,075,234	2,441,376	233	44	1,074,205	711,405	334,121	28	Kentucky.
Louisiana	7,828,166	3,292,856	592	46	1,520,354	935,146	437,408	112	Louisiana.
Maine	2,687,892	1,296,352	74	45	583,358	420,942	210,470	19	Maine.
Maryland	1,539,192	686,187	54	52	356,817	2,748,448	1,344,543	100	Maryland.
Massachusetts	3,088,375	1,410,402	84	50	705,201	1,105,636	531,311	43	Massachusetts.
Michigan	7,652,529	3,612,637	277	40	1,445,055	1,567,284	761,657	94	Michigan.
Minnesota	12,774,236	5,043,776	1,147	62	3,112,021	4,127,630	1,769,383	597	Minnesota.
Mississippi	3,867,816	1,714,341	322	49	840,027	792,012	385,050	106	Mississippi.
Missouri	9,879,849	4,408,550	544	41	1,807,506	333,167	128,324	30	Missouri.
Montana	4,666,138	2,253,164	486	51	1,149,114	2,190,997	1,058,304	149	Montana.
Nebraska	6,047,639	3,000,643	1,171	90	2,700,579	717,046	255,922	73	Nebraska.
Nevada	1,989,340	993,375	112	75	745,031	540,559	265,009	79	Nevada.
New Hampshire	547,440	273,719	35	65	177,917	1,246,104	620,571	85	New Hampshire
New Jersey	2,123,030	717,740	42	61	440,511	1,551,636	703,284	41	New Jersey.
New Mexico	2,798,538	1,399,269	327	48	668,398	1,290,443	645,221	130	New Mexico.
New York	4,548,978	1,926,219	102	7	136,775	281,237	140,619	10	New York.
North Carolina	9,953,106	4,776,856	726	54	2,579,502	2,344,796	994,581	189	North Carolina.
North Dakota	3,123,734	1,561,733	569	69	1,070,696	292,353	146,175	122	North Dakota.
Ohio	12,263,122	4,023,097	332	44	1,770,163	3,827,287	1,338,642	[illegible]	Ohio.
Oklahoma	6,157,083	2,966,021	210	38	1,127,088	412,863	201,608	7	Oklahoma.
Oregon	7,178,867	3,198,190	337	61	2,003,155	1,447,574	932,494	138	Oregon.
Pennsylvania	20,199,571	7,278,677	360	56	4,076,059	7,772,444	3,488,650	192	Pennsylvania.
Rhode Island	723,477	296,995	17	38	111,964	308,404	153,086	10	Rhode Island.
South Carolina	3,789,056	1,543,761	347	60	926,257	1,162,697	543,854	103	South Carolina.
South Dakota	5,569,072	2,772,196	628	45	1,247,488	393,598	196,797	45	South Dakota.
Tennessee	8,337,148	4,167,644	355	29	1,208,617	54,738	25,937	2	Tennessee.
Texas	21,345,499	8,263,755	1,622	50	4,131,878	3,855,768	1,519,689	545	Texas.
Utah	5,111,465	2,474,951	310	31	767,235	61,340	30,670	9	Utah.
Vermont	819,627	409,813	33	23	94,257	181,131	90,566	10	Vermont.
Virginia	4,905,117	2,432,602	242	68	1,654,169	1,028,429	513,735	92	Virginia.
Washington	2,457,478	1,186,630	101	81	962,946	5,697,351	2,698,887	259	Washington.
West Virginia	5,696,355	2,500,955	280	54	1,361,316	858,579	401,549	37	West Virginia.
Wisconsin	7,603,206	2,675,676	471	50	1,337,838	2,535,322	839,779	221	Wisconsin.
Wyoming	4,493,846	2,146,849	392	59	1,259,735	1,097,995	548,421	207	Wyoming.
Total	301,843,252	127,591,366	17,529	50	64,661,363	84,342,878	38,260,293	5,681	

NOTE.—Ratio of Federal aid to total cost in projects under construction equals 42 : 100.

a month which was reached last year will be exceeded this season, and in that event the last of the money will be contracted for at an earlier date.

That the States are meeting the Federal Government more than halfway is indicated by the fact that the ratio of Federal aid to total cost in the projects under construction is as 42 to 100.

One State (Delaware) had placed its entire apportionment under contract and will not be able to initiate any more new projects unless there is another appropriation. Louisiana and West Virginia had contracted for work which will leave them only enough Federal aid to apply to about 2 miles more, and several other States have only a small proportion of their total apportionment available for new contracts. The more advanced are Arizona, Florida, Georgia, Idaho, Illinois, Iowa, Maryland, Minnesota, New Hampshire, North Carolina, Oregon, Pennsylvania, and Washington.

Illinois leads all the States in the amount of Federal aid in completed work, with $8,220,428. New York with a larger apportionment has earned only $277,394 of its apportionment by completing work.

The above facts are revealed by two tables prepared by the Bureau of Public Roads and reproduced herein as Tables 1 and 2. Table 1 presents a financial statement as of April 30, and Table 2 shows the status of construction work on the same date. Similar information will be published monthly hereafter, replacing the customary statement of project agreements executed during the month, which was discontinued in the last issue. The usual statement of project statements approved will be continued for the information of contractors and others interested in prospective highway work.

FEDERAL AID ALLOWANCES.

PROJECT STATEMENTS APPROVED IN APRIL, 1921.

State.	Project No.	County.	Length in miles.	Type of construction.	Project statement approved.	Estimated cost.	Federal aid.
Alabama	24	Walker	[1] 9.59	Chert	Apr. 28	[1] $21,205.66	[1] $10,602.83
Arizona	46	Maricopa	32.00	Concrete	Apr. 12	1,109,128.11	554,564.05
	47	do	4.11	do	Apr. 5	133,478.06	66,739.03
California	68	Del Norte	3.180	Gravel	Apr. 20	110,715.00	55,357.50
	70	Mendocino	13.060	Earth	do	285,615.00	142,807.50
	71	Modoc	17.110	Gravel	do	219,670.00	109,835.00
	72	Los Angeles		Bridge	Apr. 15	231,000.00	115,500.00
	73	do		do	Apr. 12	145,200.00	72,600.00
	74	San Luis Obispo	11.250	Earth	Apr. 5	259,475.41	129,737.70
	75	Humbolt	10.640	Gravel	Apr. 20	280,566.00	140,283.00
	78	Mendocino	11.970	do	Apr. 25	256,850.00	128,425.00
Colorado	111	Lincoln	9.875	Sand-clay	Apr. 21	95,118.51	47,559.25
	146	Weld	2.075	Concrete	Apr. 20	88,466.40	41,500.00
	174	San Juan-Ouray	3.150	Earth	Apr. 4	119,807.33	59,903.66
Georgia	221	Bibb-Houston		Reinforced concrete bridge	Apr. 12	54,907.54	21,224.62
Kansas	63	Neosha	[1] 4.982	Gravel	Apr. 25	[1] 93,654.71	[1] 46,827.35
	64	do	[1] 5.432	do	do	[1] 116,311.34	[1] 58,155.67
Kentucky	51	Henderson	29.400	Gravel, Kentucky rock asphalt	Apr. 8	678,810.00	339,405.00
Massachusetts	55	Norfolk	.549	Reinforced concrete	Apr. 25	40,843.71	10,980.00
Minnesota	203	Winona	5.490	Concrete slab, macadam shoulder	Apr. 14	156,320.78	75,000.00
Mississippi	23	Pontotoc	[2] 12.050	Gravel	Apr. 4	[2] 395,170.77	[2] 199,439.25
Missouri	184	Wright	2.250	do	Apr. 15	28,435.00	14,217.50
Montana	151	Teton	15.000	do	Apr. 12	110,000.00	55,000.00
	154	Toole	16.000	do	Apr. 14	107,800.00	53,900.00
	156	Custer	36.000	do	Apr. 4	258,395.50	129,197.75
	157	do	22.800	do	do	159,997.75	79,998.87
	158	do	9.000	do	do	50,068.42	25,034.21
Nebraska	45	Sioux	[1] 8.250	Earth	Apr. 26	[1] 45,804.00	[1] 22,902.00
	63	Hayes-Perkins	[1] 41.500	do	do	[1] 77,000.00	[1] 38,500.00
	65	Greeley	[1] 18.500	do	do	[1] 45,320.00	[1] 22,660.00
	69	Banner	[1] 25.850	do	do	[1] 73,370.00	[1] 36,685.00
	85	Greeley	[3] 28.600	Earth and sand-clay	Apr. 15	[3] 75,460.00	[1] 37,730.20
Nebraska	90	Merrick	[1] 22.200	Earth	Apr. 26	[1] 55,825.00	[1] 27,912.50
	94	Arthur-Keith	[1] 27.800	Earth and sand-clay	do	[1] 134,349.60	[1] 67,174.80
	101	Scottsbluff	[1] 6.250	Gravel	do	[1] 46,156.00	[1] 23,078.00
	102	Dawes	[3] 22.330	Sand-clay	Apr. 15	[3] 99,318.56	[3] 49,659.28
	114	Hitchcock-Dundy	[1] 20.800	Earth and sand-clay	Apr. 26	[1] 85,558.00	[1] 42,779.00
	117	Garden	[1] 13.100	Earth	do	[1] 40,777.00	[1] 20,388.50
	124	Sheridan-Dawes	[1] 34.400	Gravel, earth, sand-clay	do	[1] 140,043.20	[1] 70,021.60
	125	Brown	[3] 11.740	Earth	Apr. 15	[3] 36,067.80	[3] 18,033.90
	127	Custer	[1] 23.400	do	Apr. 26	[1] 77,198.00	[1] 38,599.00
	131	Rawnee-Richardson	[1] 24.100	do	do	[1] 82,137.00	[1] 41,068.50
	136	Pierce	[3] 31.000	Earth and sand-clay	Apr. 9	[3] 108,043.76	[3] 54,021.88
	140	Custer	[1] 11.800	Earth	Apr. 26	[1] 37,345.00	[1] 18,672.50
	144	Morrill-Scottsbluff	[1] 16.400	Gravel	do	[1] 78,936.00	[1] 39,468.00
	148	Gosher-Frontier	[1] 36.800	Earth	do	[1] 141,724.00	[1] 70,862.00
	149	Cheyenne	[1] 36.300	do	do	[1] 137,929.00	[1] 68,964.50
	157	Seward-Butler	[1] 24.800	do	do	[1] 95,964.00	[1] 47,982.00
	159	Fillmore	[1] 26.807	do	Apr. 29	[1] 26,807.00	[1] 13,403.50
Nevada	39	Clark	11.640	Gravel	Apr. 27	38,244.58	19,122.29
	40	do	12.850	do	Apr. 29	55,255.20	27,627.60
	41	do	15.430	do	Apr. 25	50,696.30	25,348.15
	44	White Pine	21.700	Earth and gravel	Apr. 20	263,219.00	131,609.50
New Jersey	31	Morris	5.420	Concrete	Apr. 14	581,240.00	108,400.00
New Mexico	27	McKinley	23.000	Crushed stone	Apr. 9	188,100.00	94,050.00
	66	Dona Ana	7.000	Concrete	Apr. 12	191,400.00	95,700.00
New York	66	Cayuga	5.900	Reinforced concrete	Apr. 30	324,000.00	113,400.00
Ohio	119	Hardin	5.830	Bituminous or asphaltic top—macadam base	Apr. 21	287,100.00	97,500.00
	121	Miami	3.875	Brick, asphalt, or concrete	do	199,000.00	14,150.00
	122	do	4.235	do	do	203,000.00	14,150.00
	135	Montgomery	2.623	Concrete or bituminous macadam	Apr. 16	110,000.00	32,000.00
Ohio	139	Hamilton	2.530	Concrete	Apr. 21	162,200.00	43,400.00
	145	Allen	3.043	Asphalt, Kentucky rock asphalt, bituminous macadam	do	145,000.00	59,600.00
	152	Coshocton	4.043	Brick or concrete	Mar. 22	192,000.00	57,000.00
	158	Hancock	3.372	Asphaltic concrete	Apr. 21	145,000.00	52,500.00
	162	Seneca	2.576	Brick, sand cushion on clay base	Apr. 15	133,500.00	22,000.00
	165	Brown	4.497	W. B. macadam	do	130,000.00	48,000.00
	173	Scioto	2.000	Monolithic brick on clay base	Apr. 21	95,000.00	33,500.00
	177	Ottawa	3.922	Concrete	Apr. 15	143,500.00	66,000.00
	182	Hamilton	2.007	Brick, bitulithic or Kentucky rock asphalt on concrete base	Apr. 21	146,000.00	23,000.00
	183	do	4.075	Bitulithic or Bitoslag	do	369,000.00	62,000.00
	185	Mahoning	.860	W. B. macadam	Apr. 4	30,000.00	15,000.00
	188	Stark	1.415	Brick on concrete base	Apr. 15	110,000.00	25,000.00
	193	Hamilton	4.677	Concrete	Apr. 23	327,000.00	69,000.00
	194	do	2.991	Concrete, brick, bituminous macadam	Apr. 21	245,000.00	50,000.00
	195	do	5.240	Concrete, brick, Bitulithic, or bituminous macadam	Apr. 27	471,000.00	97,000.00
	201	Ashland	5.374	Grading	Apr. 25	98,760.00	40,000.00
	202	Franklin	3.640	Concrete	Apr. 15	188,000.00	25,000.00
Oklahoma	51	Muskogee-Wagoner		Bridge	Apr. 12	735,000.00	367,500.00
	53	Okmulgee	12.000	Concrete	Apr. 20	480,000.00	240,000.00
Oregon	51	Yamhill	7.680	do	Apr. 4	306,514.17	153,257.08
	52	do	5.900	do	do	185,057.95	92,528.97
Pennsylvania	98	Bucks	2.154	Reinforced concrete	Apr. 2	189,122.72	43,080.00
South Carolina	70	Bamberg	4.242	Sand-clay	Apr. 25	20,910.96	10,455.48
South Dakota	74	Douglas	12.380	Gravel	Apr. 22	81,528.26	40,764.13
	76	Davidson		Bridge	Apr. 21	41,114.04	20,557.02
Tennessee	52	Dyer	8.280	Bituminous macadam	Apr. 12	345,631.66	172,815.83
	57	Madison	6.655	do	do	258,157.42	129,078.71
	74	Davidson	2.340	do	Apr. 5	80,893.35	40,446.67
Texas	233	Burnet	10.000	Gravel	Apr. 4	55,000.00	27,500.00
	238	Jasper	11.100	do	Apr. 2	134,156.00	42,000.00
	241	Fayette	7.290	do	Apr. 20	123,458.61	30,864.65
Vermont	25	Windham	2.000	Bituminous macadam	Apr. 4	63,690.00	31,845.00

[1] Withdrawn.
[2] Revised statement. Amounts given are increases over those in the original statement.
[3] Revised statement. Amounts given are decreases over those in the original statement.

PROJECT STATEMENTS APPROVED IN APRIL, 1921—Continued.

State.	Project No.	County.	Length in miles.	Type of construction.	Project statement approved.	Estimated cost.	Federal aid.
Virginia	44	Drayson	7.250	Earth	Apr. 16	$115,500.00	$57,750.00
	75	Prince Edward	3.580	Top soil	Apr. 4	36,790.93	18,395.46
	78	Goochland	8.550	do	Apr. 21	79,090.00	39,545.00
	89	Fluvanna	4.750	do	Apr. 8	55,999.90	27,999.95
	92	Orange, Spottsylvania	5.400	Gravel	Apr. 5	54,914.82	29,457.41
	100	Wythe	9.000	W. B. macadam	Apr. 21	184,339.04	92,169.52
	102	Shenandoah	.644	Bituminous macadam	Apr. 11	60,725.87	16,408.69
West Virginia	104	Mineral	[1] 4.250	Earth	Apr. 21	[1] 55,246.40	[1] 27,623.20
Wisconsin	141	Grant	4.240	do	Apr. 25	55,869.00	24,000.00
	142	Burnett	7.290	Gravel	Apr. 22	90,493.79	27,006.50
	154	Fond du Lac	[3] 2.440	Concrete	Apr. 21	[3] 135,860.64	[3] 39,742.91
	168	Lincoln	7.050	Earth	do	41,824.56	18,000.00
	183	Ashland	2.140	Gravel	Apr. 4	15,582.32	6,000.00
	184	do	1.410	do	Apr. 14	12,763.25	5,000.00
	186	Barron	5.400	do	Apr. 11	50,053.03	20,000.00
	196	Dane	7.810	Earth	Apr. 25	46,519.00	17,000.00
	215	Jefferson	1.500	Concrete	Apr. 14	59,345.50	19,314.50
	216	do	4.040	Earth	do	59,010.05	23,000.00
	218	Kewaunee	1.870	Gravel	Apr. 4	24,316.60	9,000.00
	219	do	1.700	do	do	19,784.22	7,000.00
	220	Lafayette	4.810	Earth	do	53,797.15	25,000.00
	225	Marathon	5.060	Disintegrated granite or gravel	Apr. 14	75,217.61	30,000.00
	230	Oneida	5.720	Gravel	Apr. 2	88,548.64	35,000.00
	235	Polk	1.860	do	Apr. 16	14,752.58	6,000.00
	236	Portage	3.670	Concrete	Apr. 30	127,640.59	43,325.57
	237	Price	5.210	Gravel	Apr. 2	71,189.94	30,000.00
	238	Racine	4.500	Concrete	do	179,703.70	65,314.98
	247	Shawano	6.580	Gravel	do	74,060.36	35,000.00
	258	Price	3.050	do	do	31,112.58	9,333.33

[1] Withdrawn.
[3] Revised statement. Amount given are decreases over those in the original statement.

(Continued from page 28.)

Reference is made above to the sweeping of Portland cement over the pavement while it is still warm and pliable. The principal purpose of the cement is to fill the fine superficial pores, and it is taken up to a limited extent by the asphalt. It takes the place of what was called a seal coat in the old Topeka type before it was modified in its finer aggregate.

The seal coat in the now obsolete Topeka specifications as well as the still valid specifications for some coarse-aggregate asphaltic concretes served a twofold purpose.

In both these mixtures the grading requirements of the fine aggregate, even if properly enforced, could only produce porous mixtures. The seal coat consisted of a squeegee application of fluid asphalt cement that penetrated to some slight depth into the surface mixture and provided at least a temporary waterproofing. This seal coat with the following covering of stone chips also filled the superficial pores caused by the depression of the coarse stone into wearing surface.

A seal coat is still employed in the coarse-aggregate types and is required for the reason that the superficial pores are of much greater extent than in the Topeka type. As the coarse aggregate is much larger and constitutes nearly two-thirds of the entire mass of the pavement it follows that, in rolling, the larger stone, being depressed into the mass, will leave surface pores of comparatively large size. The only object of the seal coat at the present time is to fill these pores. While a thin coat of fluid asphalt covered with stone chips is frequently employed, a more rational seal coat consists of a thin coating of a mixture of fine aggregate and asphalt, as required in the body of the asphaltic concrete—in substance a sheet-asphalt mixture. This is thinly spread and raked over the pavement after the first rolling and after all surface corrections have been made and then rolled into the surface. The final finish for this pavement, as well as the other types, is Portland cement or limestone dust.

What has been said about field operations and inspection heretofore applies with equal force to the coarse-aggregate types. The latter can be made, the same as the small aggregate types, a pavement of permanent value, not by a lucky accident, as has been the case many times in the past, but by design and conscientious insistence on the observance of essential requirements.

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 Office of Public Roads for 1917. 6c.
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.

DEPARTMENT BULLETINS.

Dept. Bul. *105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913. 5c,
*136. Highway Bonds. 25c.
220. Road Models.
*230. Oil Mixed Portland Cement Concrete. 10c.
*249. Portland Cement Concrete Pavements for Country Roads. 15c.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
314. Methods for the Examination of Bituminous Road Materials.
347. Methods for the Determination of the Physical Properties of Road-Building Rock.
*348. Relation of Mineral Composition and Rock Structure to the Physical Properties of Road Materials. 10c.
*370. The Results of Physical Tests of Road-Building Rock. 15c.
*373. Brick Roads. 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.
414. Convict Labor for Road Work.
*463. Earth, Sand-Clay, and Gravel Roads. 15c.
532. The Expansion and Contraction of Concrete and Concrete Roads.
*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.
583. Reports on Experimental Convict Road Camp, Fulton County, Ga.
586. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1916.
*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.
*724. Drainage Methods and Foundations for County Roads. 20c.
*Public Roads, Vol. I, No. 11. Tests of Road-Building Rock in 1918.
*Public Roads, Vol. II, No. 23. Tests of Road-Building Rock in 1919. 15c.

DEPARTMENT CIRCULAR.

No. 94. TNT as a Blasting Explosive.

FARMERS' BULLETINS.

F. B. *338. Macadam Roads. 5c.
505. Benefits of Improved Roads.
597. The Road Drag.

SEPARATE REPRINTS FROM THE YEARBOOK.

Y. B. Sep. 727. Design of Public Roads.
739. Federal Aid to Highways, 1917.

OFFICE OF PUBLIC ROADS BULLETINS.

Bul. *45. Data for Use in Designing Culverts and Short-Span Bridges. (1913.) 15c.

OFFICE OF PUBLIC ROADS CIRCULARS.

Cir. *89. Progress Report of Experiments with Dust Preventatives 1907. 5c.
*90. Progress Report of Experiments in Dust Prevention, Road Preservation, and Road Construction, 1908. 5c.
*92. Progress Report of Experiments in Dust Prevention and Road Preservation, 1909. 5c.
*94. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1910. 5c.
*99. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1912. 5c.
*100. Typical Specifications for Fabrication and Erection of Steel Highway Bridges, (1913.) 5c.

OFFICE OF THE SECRETARY CIRCULARS.

Sec. Cir. 49. Motor Vehicle Registrations and Revenues, 1914.
*52. State Highway Mileage and Expenditures to January 1, 1915. 5c.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*65. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Aid Road Act. 5c.
*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.
*77. Experimental Roads in the Vicinity of Washington, D. C. 5c.
Public Roads Vol. I, No. 1. Automobile Registrations, Licenses, and Revenues in the United States. 1917.
Vol. I, No. 3. State Highway Mileage and Expenditures in the United States, 1917.
*Vol. I, No. 11. Automobile Registrations, Licenses, and Revenues in the United States, 1918. 15c.
*Vol. II, No. 15. State Highway Mileage and Expenditures in the United States, 1918. 15c.
Vol. III, No. 25. Automobile Registrations, Licenses, and Revenues in the United States, 1919.
Vol. III, No. 29. State Highway Mileage, 1919.

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. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
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.
Vol. 17, No. 4, D-16. Ultra-Microscopic Examination of Disperse Colloids Present in Bituminous Road Materials.

* Department supply exhausted.