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G. P. St. CLAIR, *Editor*

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions.

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SURFACE TREATMENT OF TOPSOIL ROADS[1]

Reported by J. S. WILLIAMSON, Assistant to the State Highway Engineer, South Carolina State Highway Department, and PAUL F. CRITZ, Associate Highway Engineer, United States Bureau of Public Roads

ABSTRACT.—This project, approximately 8 miles in length, was constructed in May and June, 1925, on United States Route 176, between the south limits of Inman and the North Carolina State line. The surface treatment consisted of a prime coat of one-quarter gallon per square yard, a second application of one-third gallon per square yard, and 45 pounds of cover material. Cut-back asphaltic oil and 8–13 viscosity (Engler) tar were used as primers. The materials used for second application were asphalt of 150–200 penetration and tar, both applied hot. One-fourth to one inch crushed granite was used as cover material on all sections except a part of section 9, on which ¼-inch chats were used, and parts of sections 9 and 10, on which ¼–1 inch slag was used.

Seal coats were applied to a portion of the experiment in November, 1925, and the remainder was sealed in March, 1926. The seal consisted of one-fifth gallon of bituminous material, and cover. The following materials were used on different sections and parts of sections: 18–25 viscosity tar, 8–13 viscosity tar, quick-drying asphaltic oil, and slow-drying asphaltic oil. Sand and ¼–¾ inch crushed granite were used as cover.

Portions of sections 7, 8, 10, and 11 required re-treatment in 1928, and part of section 11 in 1929. In 1930 portions of sections 8 and 11 were given treatments for the purpose of providing a nonskid surface. Maintenance was continued until July 1931, when the road was abandoned for a new location. The average costs for maintenance and re-treatments over the 6-year period amounted to $324 per mile, or $270 per mile exclusive of the cost of nonskid treatments applied to sections 8 and 11. The initial cost of surface treatment, including seal, varied from 17.67 to 24.43 cents per square yard.

Two materials failed to prove satisfactory, the cut-back asphaltic oil which was used as a primer and the slow-drying asphaltic oil used in the seal treatment. The former was a combination of a heavy asphaltic base and a highly volatile distillate. Because of its high original viscosity, increased by the loss of distillate on application, there was little penetration. The slow-drying oil developed a surface that was slippery in wet weather, and its use has been largely discontinued in favor of tars and quick-drying asphalts.

At the termination of the project all the sections were in good condition and indicated that they would have continued to give excellent service. Average traffic on this road was in the neighborhood of 600 vehicles per day in 1924 prior to treatment, and about 950 in 1931.

Beginning in the summer of 1923 the South Carolina State Highway Department undertook to develop an inexpensive bituminous treatment for use on the better class of topsoil and sand-clay roads, included in the State system. It was hoped to provide an all-weather surface which would eliminate the dust nuisance and carry comparatively dense motor traffic without excessive maintenance either for an indefinite period or, for certain roads, until such a time as funds might become available for providing a higher type of pavement.

A considerable mileage of topsoil and sand-clay roads had already been constructed which, under favorable conditions, were proving entirely adequate. However, on some roads, because of the amount of traffic or the character of the soil or both, it was impossible to provide a satisfactory riding surface free from objectionable dust in spite of excessive maintenance. It was to design a surface for such roads that a study of bituminous treatment was carried on by the State in 1923 and 1924.

The first series of experimental surfaces was constructed in 1923 and embraced six short sections. Five of these were constructed by the penetration method, the sixth by surface treatment. The conclusions drawn from this group of experiments were that the penetration type was less practical than the thin mat type of surface treatment, and that the degree of success obtained with the latter was dependent upon the quality of its supporting base.

The second series, built during 1924, covered six sections surface treated with heavy oils and tars as binding materials, with and without light oil and tar priming coats. The binding materials were applied at the rate of approximately a half gallon and the priming materials, when used, at about one-fourth gallon per square yard. Within a year after construction almost half of the areas on which the priming coat had been omitted required scarifying and re-treating, as did approximately one-third the area of the section on which a light, asphaltic oil was used as a primer. The two sections on which a light tar prime coat was used apparently remained in better condition as the percentage of failure recorded was much less.

In this early experimenting it was noted that some combinations of bituminous materials were satisfactory for some soil conditions but were not so satisfactory under other conditions. The form of treatment, however, gave such promising results in general that the highway department in cooperation with the Bureau of Public Roads constructed an experimental project to study the feasibility of surface treating topsoil or sand-clay roads using various types and grades of bituminous materials. The preliminary studies here briefly touched upon are described in detail elsewhere,[2] and are not included in this report, which covers only the cooperative experimental project referred to above.

The cooperative project was built in May and June, 1925, on State Road 19, now designated as United States Route 176, between the south city limits of Inman and the North Carolina State line. It was approximately 8 miles in length and was divided into five sections. Bituminous treatment was applied over a width of 19 feet, except for a distance of 3,800 feet through the town of Inman, where its width was 25 feet. A report describing the construction and early behavior of the project was published in PUBLIC ROADS, volume 8, No. 9, November, 1927. The project was maintained jointly by the State and the bureau until July, 1931, when the road was abandoned for a new location and the construction of a high-type pavement. A record was kept of the cost and character of maintenance and of the service behavior of each section. Table 1 gives the location, description, and maintenance cost of the various sections.

As previous experience had taught that the surface treatments remained more satisfactory on some types of soil than on others, a study of the topsoil and subgrade was made prior to applying the bituminous surfaces, in order to determine if possible the properties which a satisfactory soil base should possess. It was observed that on a major portion of each section the road surface was smooth and well bonded but on the remaining areas it was rough, pot-holed, or in a loose condition. On that portion of section 11 between stations 729+00 and 791+25 the surface scaled badly during treatment work. This condition was caused, it was believed, by the finely crushed granite which had been spread over the surface to a depth of about 2 inches but which had not worked into the surfacing material and as a result was only partially bonded.

[1] Final report on a cooperative experiment in bituminous surface treatment by the South Carolina State Highway Department and the Bureau of Public Roads.

[2] Surface Treatment of Roads, by N. S. Anderson. Proceedings of the Fifth Annual Asphalt Paving Conference.

TABLE 1.—*Construction details and maintenance costs*

ORIGINAL CONSTRUCTION

Section	Station	Area	Surface treatment, May and June, 1925			Seal				Cost per square yard
			Prime coat (¼ gallon per square yard)	Second application (⅓ gallon per square yard)	Cover (45 pounds per square yard)	Date applied	Stations sealed	Bituminous material (⅙ gallon per square yard)	Cover	
		Sq. yds.								*Cents*
7 [1]	0+00 to 82+50	19,950	8-13 viscosity tar	Tar, hot	¼ in.-1 in. granite.	November, 1925. March, 1926	38+00-82+50 0+00-38+00	18-25 viscosity tar. 8-13 viscosity tar	Sand do	21.39
8	436 to 512	16,044	Cut-back asphaltic oil.	150-200 penetration asphalt, hot.	do	do	436-512	Quick-drying asphaltic oil.	do	17.67
9	512 to 517 517 to 533 533 to 536	1,056 3,378 633	do	Tar, hot	do ¼ in. chats ¼ in.-1 in. slag.	do	512-536	8-13 viscosity tar	do	24.43
10	536 to 553 553 to 622+50	3,589 14,672	8-13 viscosity tar	do	do ¼ in.-1 in. granite.	November, 1925. March, 1926	574+50-622+50. 536-553 553-574+50	18-25 viscosity tar. 8-13 viscosity tar	do ¼ in.-¾ in. granite. Sand	22.84
11 [2]	622+50 to 791+25	35,414	do	150-200 penetration asphalt, hot.	do	November, 1925.	622+50-720 720-735 735-750 750-780 780-791+25	Slow-drying asphaltic oil.	do ¼ in.-¾ in. granite. Sand ¼ in.-¾ in. granite. Sand	18.04

MAINTENANCE AND RETREATMENT COSTS

[Cents per square yard]

Section	1925-26	1926-27	1927-28		1928-29		1929-30		1930-31		Total	Average annual	Annual surface cost per mile 20 feet wide
	Maintenance	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance			
7	3.08	2.11	[3] 0.99	1.30		1.22		1.57		1.33	11.60	1.93	$226
8	2.63	1.99	[3] 1.39	1.20		1.51	[3 4] 4.84	1.14	[3 4] 3.96	.74	19.40 [5] 10.60	3.23 [5] 1.77	379 [5] 208
9	2.37	.78		.62		.97		.73		1.42	6.89	1.15	135
10	4.00	2.07	[3] 3.07	.69		1.33		1.64		.66	13.46	2.24	263
11	5.56	3.22	[3] 1.31	2.01	[3] 2.41	1.41		1.36	[3 4] 5.86	.37	23.51 [5] 17.65	3.92 [5] 2.94	460 [5] 345
Average													324 [5] 270

[1] Treatment was made 25 feet wide from station 0+00 to 38+00; remainder of project was 19 feet wide.
[2] Station equation: Sta. 721+00=722+00.
[3] Section treated in part only but cost is proportioned over entire section.
[4] Treatment applied mainly to provide a nonskid surface.
[5] Costs exclusive of nonskid treatment.

TABLE 2.—*Analysis of topsoil and subgrade samples* [1]

SAMPLES TAKEN FROM THE TOPSOIL SURFACE

Laboratory No.	Section No.	Sample station	Mechanical analysis of granular material: Retained ¼ inch	Passing ¼ inch, retained No. 10	Passing No. 10, retained No. 20	Passing No. 20, retained No. 60	Passing No. 60, retained No. 100	Passing No. 100, retained No. 200	Passing No. 200, retained No. 270	Total	Silt [2] 0.05 to 0.005 mm	Clay [2] less than 0.005 mm	Dye adsorption number	Moisture equivalent	Lower liquid limit	Plastic index	Shrinkage: Limit	Shrinkage: Ratio	Depth of topsoil surface at center line	Condition of the topsoil surface before treatment
1,562	7	5	4	7	8	26	9	12	4	70	17	13	7	8					7	Smooth, well bonded.
1,563	7	28	14	9	8	22	8	10	3	74	18	8	6	8					6	Loose, poorly bonded.
1,545	8	446	8	3	5	34	12	9	2	73	14	13	5	8	17	0	15	1.8	5	Rough, well bonded.
1,546	8	472	5	3	6	37	13	11	3	78	14	8	3	6	17	0	15	1.7	6	Loose, poorly bonded.
1,547	8	497	8	4	4	29	9	10	4	68	21	11	7	13	31	6	21	1.7	6	Good, well bonded.
1,549	9	525	11	5	5	38	14	11	0	84	12	4	2	4	15	0			6	Loose, poorly bonded.
1,551	10	552	3	4	6	33	10	11	4	71	15	14	9	10	23	0	19	1.7	5	Good, well bonded.
1,553	10	585	3	2	4	37	13	10	2	71	15	14	5	8	19	0	15	1.8	4	Good, fairly well bonded.
1,554	10	620	1	4	6	36	11	11	2	71	15	14	5	9	18	0	15	1.8	6	Do.
1,555	11	661	4	5	7	34	11	8	5	74	13	13	8	8			16	1.7	8	Good, well bonded.
1,556	11	693	8	10	7	30	11	9	2	77	15	8	4	7	18	0	14	1.7		Smooth, poorly bonded.
1,558	11	726	2	7	7	26	10	12	2	66	18	16	7	12	23	0	19	1.6	5	Fairly smooth, fairly well bonded.
1,559	11	757	8	9	6	25	12	11	5	76	20	4	5	7	17	0	17	1.7	5	Loose, very poorly bonded.
1,560	11	785	2	5	6	30	16	13	4	76	16	8	5	8	20	0	18	1.7		Do.

SAMPLES TAKEN FROM THE SUBGRADE

Laboratory No.	Section No.	Sample station	Retained ¼ inch	Passing ¼ inch, retained No. 10	Passing No. 10, retained No. 20	Passing No. 20, retained No. 60	Passing No. 60, retained No. 100	Passing No. 100, retained No. 200	Passing No. 200, retained No. 270	Total	Silt	Clay	Dye adsorption number	Moisture equivalent	Lower liquid limit	Plastic index	Shrinkage: Limit	Shrinkage: Ratio	Depth of topsoil surface at center line	Condition
1,565	7	49	0	1	1	26	9	8	4	49	16	35	10	21	44	22	27	1.5		
1,548	8	497	0	2	4	26	8	9	3	52	14	34	11	21	37	12	26	1.6		
1,552	10	552	5	4	6	29	10	12	4	70	17	13	7	13	30	0	23	1.6		
1,557	11	693	1	5	5	23	7	8	4	53	19	28	8	14	33	15	21	1.7		

[1] According to the present laboratory method of analysis, the topsoil materials would most probably fall in the friable varieties of the A-2 soil group, while the subgrade materials would be classified as follows: No. 1,552, friable A-2; No. 1,557, plastic A-2; Nos. 1,548 and 1,565, A-7.
[2] Although originally determined by methods not now in use, the values given herein are corrected to conform to present laboratory practice of defining silt and clay.

TABLE 3.—*Analyses of the tars used in the original construction*

Laboratory No.	27,153	27,154	27,198	27,199	27,201	27,706	27,769
Section	7	7	9 and 10	10 and 11	11	7 and 10	7,9 and 10
Location of use (stations)	0+00 to 82+50	0+00 to 82+50	512 to 536, 536 to 622 +50	536 to 630, 729 to 791 +25	630 to 729	38+00 to 82+50, 574 +00 to 622 +00	0+00 to 38+00, 512 to 574
Purpose	Prime coat	Second application	Second application	Prime coat	Prime coat	Seal coat	Seal coat
Specific gravity, 25°/25° C	1.186	1.23	1.231	1.169	1.158	1.176	1.145
Specific viscosity, Engler, at 25° C	------	------	------	48.7	36.7	------	53.8
Specific viscosity, Engler, at 40° C	12.4	------	------	12	11.3	18.9	11.4
Float test at 32° C ... seconds	------	167	182	------	------	------	------
Float test at 50° C ... do	------	69	------	------	------	------	------
Bitumen soluble in CS_2 ... per cent	92.2	88.5	81.8	88	89.6	------	94.75
Free carbon ... do	7.7	11.4	18.1	11.8	10.3	8.8	5.19
Inorganic matter insoluble ... per cent	.1	.1	.1	.2	.1	.1	------
Distillation—							
Water ... do	1.2	0	0	2	2.4	1.36	2.57
Distilled to 170° C ... do	2.29	0	0	2.79	4.21	------	------
170° to 235° C ... do	7.07	2.25	2.47	9.33	6.26	4.83	8.80
235° to 270° C ... do	10.83	7.10	6.54	10.90	9.98	12.51	12.76
270° to 300° C ... do	6.49	3.66	4.73	5.10	5.83	6.24	6.76
Residue ... do	73.28	87.15	86.60	71.84	73.79	75.33	69.04
Softening point of residue, ° C	34	42	49	40	38	30	38

TABLE 4.—*Analyses of the asphaltic materials used in the original construction*

Laboratory number	27,197	27,200	27,202	27,203	27,705	27,760
Section	8 and 9	8	11	11	11	8
Location of use (stations)	436 to 512, 512 to 536	436+00 to 512	700 to 791+25	622+50 to 700	622+50 to 791+25	436+00 to 512+00
Purpose	Prime coat	Second application	Second application	Second application	Seal coat	Seal coat
Specific gravity 25°/25° C	0.950	1.031	1.032	1.031	0.9473	0.9439
Flash point ... ° C	28	235	235	235	90	30
Burning point ... ° C	31	------	------	------	150	55
Specific viscosity, Engler, at 25° C	329	------	------	------	169.3	84.7
Specific viscosity, Engler, at 40° C	117	------	------	------	44.19	31.8
Specific viscosity, Engler, at 100° C	------	336	327	332	------	------
Penetration, 25° C., 100 grams, 5 seconds	------	165	161	164	------	------
Softening point ... ° C	------	40	40	40	------	------
Loss, 163° C., 5 hours, 20 grams ... per cent	23	------	------	------	9.6	32.2
Residue, float test, 32° C ... seconds	4,122	------	------	------	------	------
Residue, float test, 50° C ... do	360	------	------	------	------	------
Loss, 163° C., 5 hours, 50 grams ... per cent	21.3	.17	.14	.16	6.1	27.4
Residue, float test, 32° C ... seconds	852	------	------	------	------	------
Residue, float test, 50° C ... do	237	------	------	------	------	------
Residue, penetration, 25° C., 100 grams, 5 sec	------	139	147	146	------	------
Bitumen soluble in CS_2 ... per cent	99.8	99.8	99.8	99.8	99.9	100
Organic matter insoluble ... do	.2	.2	.2	.2	.1	------
Inorganic matter insoluble ... do	0	0	0	0	0	0
Bitumen insoluble in 86° Baumé naphtha ... per cent	20	23.6	24.2	23.5	13.3	17.6

The depth of the old topsoil wearing surface varied throughout the sections, ranging from 4 to 8 inches. Samples of the topsoil surfacing and of the subgrade were analyzed and the results are given in Table 2, together with a statement of the general appearance of the surface at the time the treatments were applied.

The bituminous materials were selected with a view to forming a stable mat of bitumen and stone which would adhere to the topsoil surface. In the early experiments it had been found that a bituminous material sufficiently viscous to hold a stone cover did not adhere satisfactorily to an untreated surface and that a priming application was highly desirable. The materials used on this project for priming were fluid products designed to penetrate well and to dry rapidly to permit early application of the second coat. The analyses of the various bituminous materials used are given in Tables 3 and 4.

The general procedure in constructing the bituminous surfaces was to sweep the topsoil surface clean of all loose and foreign material and then to apply about one-fourth gallon per square yard of the selected priming material. Holes or breaks occurring in the topsoil surface were repaired after priming, ordinarily by filling the depression with stone but in the case of very bad breaks a bituminous cold patch mixture was used. A considerable amount of such repair work was required on the north third, approximately, of section 11.

The prime coat was permitted to penetrate and set up for a day or two after which a more viscous bituminous material was applied at the rate of about one-third gallon per square yard. The surface was immediately covered by hand with about 45 pounds per square yard of the mineral cover selected and then rolled, after which the section was opened to traffic. During the first two or three weeks following construction it was necessary to respread the cover material thrown to the sides by traffic over those areas where bleeding and picking up indicated a deficiency of covering. Chats, which were waste concentrates from zinc mines and were mainly dolomite passing a ⅝-inch and retained on a No. 10 sieve, as well as slag and crushed granite, were used as cover materials on the location shown in Table 1.

During the fall and spring following construction all of the sections received a light re-treatment which served as a seal. The bituminous materials were of the same types as those used in the original construction but were less viscous. Sand was used as the cover material on all sections except a portion of section 10, on which crushed slag had been used originally, and two short lengths of section 11. On these portions three-fourths to one-fourth inch crushed granite was used. The nature and extent of these treatments as well as those applied subsequently to the various sections are shown in Figure 1.

HISTORY AND SERVICE BEHAVIOR OF THE SECTIONS

Section 7, stations 0+00 to 82+50.—Construction: Prime coat, one-fourth gallon 8 to 13 viscosity[3] tar. Second application, one-third gallon hot tar. Cover, 45 pounds ¼ to 1 inch crushed granite. Seal application: Bituminous material, one-fifth gallon, 8 to 13 viscosity tar on stations 0 to 38, one-fifth gallon 18 to 25 viscosity tar on stations 38 to 82+50; cover, sand.

Cost, including seal coat, 21.39 cents per square yard.

The 3,800-foot portion of this section through Inman, because of its excellent condition in the fall following construction, was permitted to go through the first winter without a seal. On the remaining portion of the section, however, numerous surface breaks developed, and a seal was applied which consisted of one-fifth gallon of an 18 to 25 viscosity tar, applied

[3] Unless otherwise noted all references to viscosity are expressed in terms of specific viscosity (Engler) at 40° C.

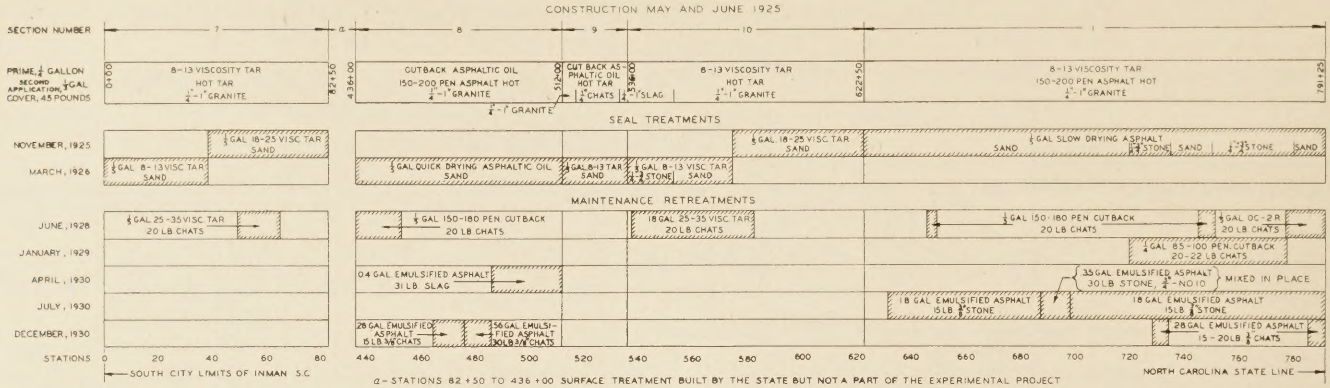


Figure 1.—Chart Showing Nature and Extent of Treatments Applied to Experimental Sections

cold, and a cover of sand. The area thus sealed remained in excellent condition during the winter but the portion which was not sealed in the fall required, by March, 1926, considerable patching; and it was then sealed with one-fifth gallon of an 8 to 13 viscosity tar and a cover of sand. The only re-treatment which this section received was applied in June, 1928, to that portion of the section between stations 49 + 25 and 64 + 50 and consisted of one-fifth gallon of 25 to 35 viscosity tar and 20 pounds of chats per square yard. Frequent inspections showed the section to have remained in excellent condition except for some roughness due partly to the original surface roughness of the base at the time of treatment and to a certain extent to the shoving resulting from the crushing of the fragile granite used. The section at the close of the experiment was probably the roughest of the group but a light drag or mixed-in-place re-treatment would have remedied this condition and, judging from its past history, would have created a satisfactory riding surface which could have been economically maintained indefinitely. The surface condition typical of the section is illustrated n Figure 2.

Figure 2.—Detailed View of the Surface of Section 7 Taken in March, 1931. Surface is Uncracked and Shows Complete Freedom from Undue Hardening of the Bituminous Material

Section 8, stations 436 + 00 to 512 + 00.—Construction: Prime coat, one-fourth gallon cut-back asphaltic oil. Second application, one-third gallon 150 to 200 penetration asphalt, hot. Cover, 45 pounds ¼ to 1 nch crushed granite. Seal application: Bituminous material, one-fifth gallon quick-drying asphaltic oil; cover, sand.

Cost, including seal coat, 17.67 cents per square yard.

This section was allowed to go through the winter following construction without a seal and as a result a large number of small patches were required during the winter and early spring. The entire section was sealed in March, 1926, with one-fifth gallon of quick-drying oil and a covering of sand. The only maintenance re-treatment required after that time was that given to the portion between stations 436 and 452 + 80. It was applied in June, 1928, and consisted of one-fifth gallon of cut-back asphalt and 20 pounds of chats per square yard. In 1930 the portion of the section between stations 464 + 50 and 512 + 00 was treated experimentally to eliminate slipperiness in wet weather and to provide a nonskid surface.

Figure 3.—Failure of the Heavy Nonskid Treatment on Section 8

Several methods of applying the treatment were tried out. One consisted of the application of a quick-breaking emulsion, spreading the cover material and rolling. This method was varied by applying a tack coat, then spreading and smoothing the aggregate and spreading another application of emulsion, depending upon the second application to penetrate through the cover stone to the tack coat. This treatment was generally not successful, because of the use of an amount of bituminous material insufficient to penetrate the stone cover. The result was that the surface peeled and raveled as shown in Figure 3. By another method the stone was spread, the emulsion applied, the two mixed, and the mixture spread and rolled. This method proved successful where sufficient bituminous material was used.

While a nonskid surface texture was obtained by each method, the first one seemed to serve the purpose most satisfactorily.

The portion of the section between stations 452 + 80 and 464 + 50 remained in excellent condition and received no treatment after March, 1926. Its condition in April, 1930, is illustrated in Figure 4.

FIGURE 4.—PORTION OF SECTION 8 NEVER RE-TREATED AFTER CONSTRUCTION, PHOTOGRAPHED IN APRIL, 1930

Except for the failure of the nonskid treatment mentioned the section as a whole remained in very good condition. Its cost was somewhat higher than that of section 9, primarily because of the necessity of providing the nonskid surface in 1930. The annual maintenance cost indicates that the section could have been continued in service at a reasonable cost.

Section 9, stations 512 + 00 to 536 + 00.—Construction: Prime coat, one-fourth gallon cut-back asphaltic oil. Second application, one-third gallon hot tar. Cover: 45 pounds ¼ to 1 inch crushed granite on stations 512 to 517; 45 pounds ¼ to 1 inch chats on stations 517 to 533; 45 pounds ¼ to 1 inch slag on stations 533 to 536. Seal application: Bituminous material, one-fifth gallon 8 to 13 viscosity tar; cover, sand.

Cost, including seal coat, 24.43 cents per square yard.

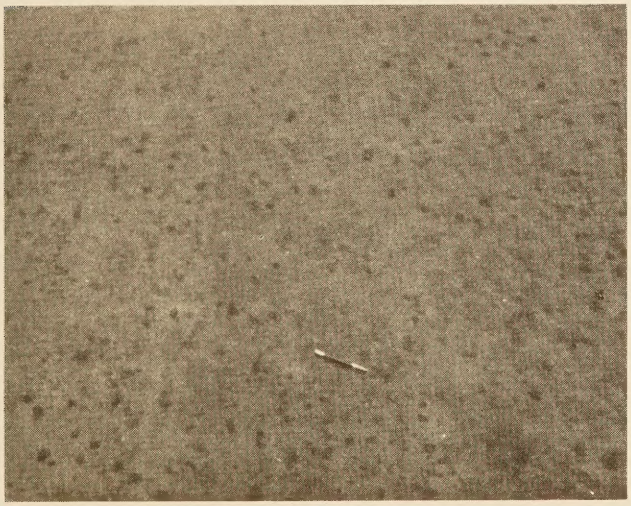


FIGURE 5.—DETAILED VIEW OF SECTION 9, WHICH WAS NEVER RE-TREATED AFTER CONSTRUCTION, PHOTOGRAPHED IN APRIL, 1930

Section 9 also was permitted to go through the first winter without a seal as its surface was smooth and showed no defects. The portion on which chats were used resembled a Topeka pavement, while the slag-covered portion had a rough and granular texture. The former required some little patching during the winter but its condition in general was much better than the latter.

The entire section was given a seal coat in March, 1926, consisting of one-fifth gallon of 8 to 13 viscosity tar and a cover of sand. After that date the section received no further treatments. This fact, as well as the low maintenance cost, indicates its continued excellent behavior. It continued in the best condition of any of the sections, the chat-covered portion appearing somewhat better than the slag, which was more open. Figures 5 and 6 are typical illustrations of this section.

FIGURE 6.—GENERAL VIEW OF SECTION 9, COVER MATERIAL ON AREA SHOWN IS CHARTS

Section 10, stations 536 + 00 to 622 + 50.—Construction: Prime coat, one-fourth gallon 8 to 13 viscosity tar. Second application, one-third gallon hot tar. Cover: 45 pounds ¼ to 1 inch slag on stations 536 to 553; 45 pounds ¼ to 1 inch crushed granite on stations 553 to 622 + 50. Seal application: Bituminous material, one-fifth gallon 8 to 13 viscosity tar on stations 536 to 574 + 50, one-fifth gallon 18 to 25 viscosity tar on stations 574 + 50 to 622 + 50; cover, ¼ to ¾ inch crushed granite on stations 536 to 553, sand on stations 553 to 622 + 50.

Cost, including seal, 22.84 cents per square yard.

The portion of this section between stations 574 + 50 and 622 + 50 was sealed in the fall after construction with an 18.9 viscosity tar and covered with sand. The remainder of the section was not sealed until the following spring as it was desired to retain the rough granular texture through the winter and also to obtain information as to the necessity for early sealing. During the winter many breaks occurred in the unsealed surface and these were repaired prior to applying the treatment in March, 1926. The treatment of this portion consisted of one-fifth gallon of 8 to 13 viscosity tar. Crushed granite ¼ to ¾ inch in size was used as the cover material on the portion between stations 536 + 00 and 553 + 00, originally covered with ¼ to 1 inch hard slag, while on the portion between stations 553 + 00 and 574 + 50, originally covered with crushed granite, sand was used.

After March, 1926, only one re-treatment was given and that was applied in June, 1928, to the portion of the section between stations 537 + 59 and 582 + 14, i. e., approximately the same portion as was sealed in March, 1926. The treatment consisted of an application of 0.18 gallon of 25 to 35 viscosity tar and a cover of 20 pounds of chats per square yard. The remainder of the section was not treated after the fall following its construction.

The section in general remained in excellent condition. The portion which was re-treated in 1928 had developed some cracks and showed indications of brittleness in the area where slag cover had originally been used. This may have been due to the fact that no allowance was made for the absorptive property of the slag and as a result the amount of second application material used developed a mat that was leaner than it should have been. Had the project been continued it is believed that a light re-treatment would have been desirable on this area. The portion never re-treated showed no need of immediate re-treatment. Figure 7 is a typical view showing its excellent condition.

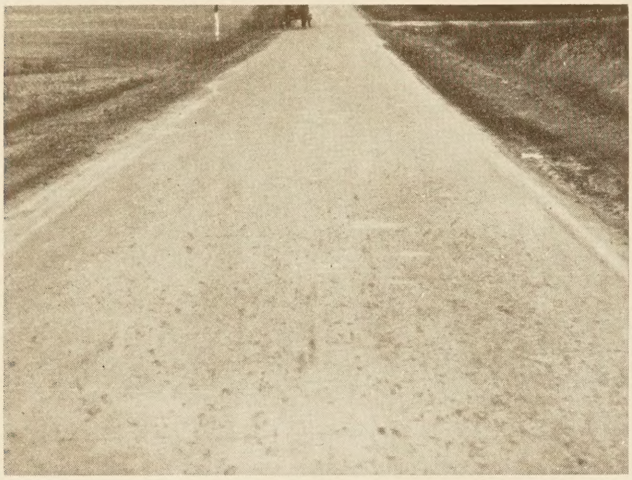


FIGURE 7.—TYPICAL CONDITION OF THE PORTION OF SECTION 10 WHICH WAS NEVER RE-TREATED

Section 11, stations [4] *622+50 to 791+25.*—Construction: Prime coat, one-fourth gallon 8 to 13 viscosity tar. Second application, one-third gallon 150 to 200 penetration asphalt, hot. Cover, 45 pounds one-fourth to 1 inch granite. Seal application: Bituminous material, one-fifth gallon slow-drying asphaltic oil; cover, sand on stations 622+50 to 720, 735 to 750, and 780 to 791+25, ¼ to ¾ inch crushed granite on stations 720 to 735 and 750 to 780.

Cost, including seal, 18.04 cents per square yard.

The north portion of the section from about station 720 to the end of the project at station 791+25, which lay on a steep grade, was covered with a 2-inch course of fine crushed granite prior to the surface treatment. It was expected that this material would stabilize the topsoil for later surface treatment and at the same time provide a more nonskid surface in wet weather. However, at the time of treatment, the primed surface was not well bonded, as indicated by the fact that it scaled and raveled under traffic. The prime was inadequate both as to the amount used and the method of application to overcome this condition and as a result considerable failure developed in the bituminous mat shortly after construction, necessitating a large amount of patching and numerous partial re-treatments.

The remainder of the section had a well-bonded topsoil surface comparable with that of the other sections and it remained in good condition, requiring no such extensive maintenance and re-treatments as did the north portion.

[4] Station equation, 721+00=722+00.

FIGURE 8.—VIEW OF THE SOUTH END OF SECTION 11, SHOWING THE EXCELLENT CONDITION OF THE PORTION NEVER RE-TREATED

In the November following construction the entire section was given a seal consisting of an application of one-fifth gallon of slow-drying asphaltic oil. Sand was used as cover material except for two short sections on which ¼ to ¾ inch granite was used. In June, 1928, re-treatments were applied to three short sections as follows: Stations 645+64 to 649+11, 744+71 to 751+05, and 776+75 to 791+25. The bituminous materials used were a 150 to 180 penetration asphalt cut-back and an asphaltic oil and were applied at the rate of one-fifth gallon per square yard. The cover material was chats spread at the rate of 20 pounds per square yard. In January, 1929, the area between stations 719+50 and 777+25 was re-treated using a quarter of a gallon of 85 to 100 penetration asphalt cut-back and 20 to 22 pounds of limestone chats. After this application all portions of the section were given one or two re-treatments, with the exception of the area between stations 622+50 and 631+25, which has never been re-treated. The re-treatments were applied primarily to develop a nonskid surface as the earlier seal treatment with road oil and sand cover had produced a surface that was very slippery in wet weather. An asphalt emulsion and ⅜-inch stone were applied by methods similar to those employed on section 8. These treatments were applied in July and December,

FIGURE 9.—TYPICAL APPEARANCE OF SECTION 11 AFTER APPLICATION OF THE NONSKID TREATMENT

1930. The July treatment, except for the area between stations 687+25 and 698+00, upon which mixed-in-place seal coat was applied, was of the light seal type consisting of 0.18 gallon of emulsion and 15 pounds of ⅜-inch stone. The mixed-in-place seal was composed of 0.35 gallon of emulsion and 30 pounds of stone passing the ¾-inch and retained on the No. 10 sieve. Between stations 727+75 to 733+75 and 785+00 to 791+25 the surface was given an additional treatment in December of the same year. In this treatment 0.28 gallon of emulsion and 15 to 20 pounds of ⅜-inch chats were used. These treatments in general proved satisfactory, and the mixed-in-place seal produced considerable improvement in the surface smoothness.

Figure 10.—General View of North End of Project, Which was Located in Fairly Rough Country

The section as a whole was in better condition in July, 1931, than at any previous inspection. The portion which was never re-treated remained in excellent condition as illustrated in Figure 8. Figures 9 and 10 show other views of the section.

DISCUSSION

These experiments illustrate rather strikingly the fact that under average conditions surface treatments can be applied at moderate cost which will give excellent service over a period of years. They also offer an interesting comparison of methods of treatment in vogue six years ago with those of the present time. Although traffic had increased over 50 per cent during the period of the experiment its effect was not reflected in the cost of maintenance, which continued very uniform, as shown by the accumulated cost curves in Figure 11. At the termination of the project the sections were all in good condition and gave evidence that their record of excellent service would have continued.

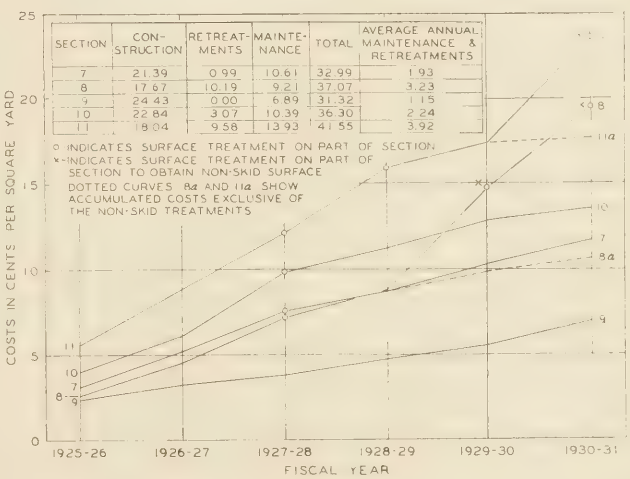
The subgrade and topsoil on the project were, in general, well suited to surface treatment as shown by their analyses, given in Table 2. One of the subgrades, sample No. 1552, section 10, was of the friable A–2 variety. No. 1557, from section 11, was of the plastic A–2 variety. The remaining two would be classed as the better variety of the A–7 group because of their higher liquid limits and lower sand contents. They are so close to the line separating the A–2 from the A–7 group that they also would be considered satisfactory subgrade materials under the favorable moisture conditions existing on this project. The topsoil surfacing, which averaged about 5 inches in depth, had the characteristics of the friable varieties of the A–2 group, material which provides an excellent base for surface treatment.

At the time the surface treatments were applied it was believed that a thoroughly bonded base was necessary and it was thought that the ideal base would be one which, untreated, would remain stable and would not ravel under traffic. This idea indirectly implied the use of a natural binder, such as clay, to maintain the bond, the amount varying with the characteristics of the binder and with the moisture conditions encountered.

It is known that capillary moisture stabilizes the cohesionless sands and other granular materials required for satisfactory road surfacing and bases for surface treatment. In the untreated road the surface portion is deficient in moisture because of evaporation, with the result that a clay binder is required to bind or cement the particles of granular material into a stable and wear-resisting surface. In the surface-treated road evaporation is largely prevented and the moisture content of the soil is increased by capillary action, with the result that the bond is sufficient to furnish the required stability not only in the top portion but throughout the depth of the base. Thus the need for clay binder is largely eliminated.

This does not mean that bond is not required in a base to be surface treated. As a matter of fact bond is required for stability in all types of soil roads whether they serve as bases or as surfaces. Thus the necessity for having a bonded surface upon which to apply the treatment is still recognized, but ideas have changed as to the method of obtaining this condition.

The binding properties of clay reach a maximum value at a certain moisture content and decrease with further increase in moisture. Consequently, the increase in moisture content due to surface treatment may not only reduce the binding property of the clay but may soften it to such an extent as to cause loss of stability in the base resulting in failure of the surface treatment. For this reason the quantity of clay required for an untreated clay-bound road, is often excessive when the road is to serve as a base for bituminous surface treatment.



SECTION	CON-STRUCTION	RETREAT-MENTS	MAINTE-NANCE	TOTAL	AVERAGE ANNUAL MAINTENANCE & RETREATMENTS
7	21.39	0.99	10.61	32.99	1.93
8	17.67	10.19	9.21	37.07	3.23
9	24.43	0.00	6.89	31.32	1.15
10	22.84	3.07	10.39	36.30	2.24
11	18.04	9.58	13.93	41.55	3.92

Figure 11.—Accumulated Costs of Surface Treatments and Maintenance

The early unsatisfactory behavior of the north portion of section 11 could have been avoided by a more suitable priming treatment of the loosely bound top course. A greater amount of priming material with light surface mixing would have stabilized the base course to a greater depth than the light application of prime used and would have largely eliminated the failures resulting from the displacement of the base materials.

The bituminous materials used in the original construction, analyses of which are given in Tables 3 and 4, were, with two exceptions, of the same types that are used to-day for similar work. The two exceptions are the quick-drying asphaltic material used as a prime and the slow-drying material used in the seal treatment.

The asphaltic primer was a combination of a heavy asphaltic base and a highly volatile distillate and had a specific viscosity at 40° C. of 117 which was approximately ten times that of the tar prime. Because of its high original viscosity, which was proportionately increased by the loss of distillate upon application, there was little penetration, and the heavy material remained on the surface as a mat. Had the prime been made up with a less volatile distillate and reduced to the consistency satisfactory for priming purposes, much better penetration would undoubtedly have resulted, without the deposition of heavy material on the surface.

The 8 to 13 viscosity tar used is still considered highly satisfactory as a priming material. Asphaltic primers also are used and the type favored is a material of low viscosity composed of a base and distillate such as to insure penetration and the development of a residue after penetration which will harden and develop some cementing value. The residue is not required, however, to serve as a binder to hold a cover of gravel or stone chips.

The other material which did not prove satisfactory was the slow-drying asphaltic oil used in the seal treatment on section 11. This material, according to its analysis, showed a low loss at 163° C. and an apparently soft residue. In combination with the sand cover it developed a surface that was slippery in wet weather. Because of this tendency its use has been largely discontinued as a seal material in favor of tars or quick drying asphalts.

There has been a tendency in recent years to use for re-treatment cold surface treatment material of viscosity higher than that of those formerly used. The greater viscosity produces stiffening qualities more nearly approaching those of materials applied hot, while at the same time permitting more manipulation during construction for the purpose of improving the riding qualities of the surface.

Cold application materials, compared with those applied hot, seem to insure in greater degree and for a longer period of time the maintenance of a nonskid surface. In this respect, however, it must be noted that the tar-treated sections in this experiment retained an excellent nonskid surface throughout the entire period, and did not require any treatment on this account.

To study the physical changes which might have occurred in the bituminous materials after six years of exposure to traffic and atmospheric conditions, samples of the surface mat above the primed base were taken from areas of sections 10 and 11 which had received no treatment subsequent to the seal following construction. A summary of the materials originally composing the samples and their analyses are given in Table 5.

With reference to the sample taken from section 10 which contained only tar, the percentage of bitumen extracted does not indicate the amount of the bituminous material contained in the sample, as the free carbon remained with the aggregate upon extraction. The original analyses of the tars used for the second application and seal coat show them to have had an average solubility in carbon disulphide of only about 85 per cent. Disregarding the increase in carbon content due to the loss of volatile matter and to weathering during the period of exposure, which would further decrease the percentage soluble in carbon disulphide, the actual amount of bituminous material present would be somewhat above 5.3 per cent, which agrees reasonably well with that contained in sample 34371 taken from section 11.

TABLE 5.—*Analyses of surface mats after six years' service*

Section No.	10	11	11
Location sampled (station)	600	Center of road	625.
Original construction:			
Prime	¼ gallon 8–13 viscosity tar.	¼ gallon 8–13 viscosity tar.	¼ gallon 8–13 viscosity tar.
Second application	⅓ gallon hot tar.	⅓ gallon 150–200 penetration asphalt.	⅓ gallon 150–200 penetration asphalt.
Cover	¼–1 inch stone, 45 pounds.	¼–1 inch stone, 45 pounds.	¼–1 inch stone, 45 pounds.
Seal application—			
Bituminous material.	⅓ gallon 18–25 viscosity tar.	⅓ gallon slow-drying oil.	⅓ gallon slow-drying oil.
Cover	Sand	Sand	Sand.
Thickness of surface mat	⅜ inch	⅜ inch	⅜ inch.
Laboratory number of sample.	34370	34371	34372.

ANALYSIS

Section No.	10	11	11
	Per cent	*Per cent*	*Per cent*
Bitumen extracted	4.5	5.9	8.4
Passing ¾ inch, retained ½ inch	8.5	10.5	3.5
Passing ½ inch, retained ¼ inch	16.9	30.2	26.4
Passing ¼ inch, retained No. 10 sieve	17.8	10.4	11.4
Passing No. 10 sieve, retained No. 20 sieve	9.9	5.3	7.0
Passing No. 20 sieve, retained No. 30 sieve	4.9	4.3	4.4
Passing No. 30 sieve, retained No. 40 sieve	7.1	5.3	6.4
Passing No. 40 sieve, retained No 50 sieve	5.9	4.5	5.4
Passing No. 50 sieve, retained No. 80 sieve	8.0	7.0	8.2
Passing No. 80 sieve, retained No. 100 sieve	2.8	2.9	3.2
Passing No. 100 sieve, retained No. 200 sieve	5.3	5.3	5.8
Passing No. 200 sieve	8.4	8.4	9.9
	100.0	100.0	100.0
Consistency of extracted bitumen:			
Float test at 32° C. ... seconds	255		
Float test at 50° C. ... do	50		
Penetration at 25° C.		76	72
Softening point ... °C		55.2	55

Sample 34372 was taken from an area on section 11 on which successive applications of bituminous material overlapped. In spite of its high bitumen content the area represented was in excellent condition and showed no evidence of rutting or shoving.

The tar extracted was composed of approximately five parts of a tar having an original float of 182 seconds at 32° C. and three parts of one having an original specific viscosity of 18.9 at 40° C. The resultant material after six years of service had, as noted, a float of 255 and 50 seconds at 32° C. and 50° C., respectively. Its final consistency therefore approximates that of a tar intermediate between materials suitable for hot surface treatment and bituminous macadam and would not indicate a tendency to harden unduly and become brittle although this tendency has been an assumed characteristic for this type of material. Apparently pronounced hardening is limited largely to the exposed surface and does not seem to affect greatly the major portion of the bituminous material in the mat even though the total thickness of the latter is relatively small.

The asphaltic material extracted was composed of approximately five parts of an asphalt cement having

(Continued on p. 184)

EFFECT OF SIZE OF SPECIMEN, SIZE OF AGGREGATE AND METHOD OF LOADING UPON THE UNIFORMITY OF FLEXURAL STRENGTH TESTS

Reported by W. F. KELLERMANN, Associate Materials Engineer, United States Bureau of Public Roads

SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

A. Relative flexural strength.—1. The flexural strength of the specimens was influenced by the method of loading and computing the results. The three methods which were investigated gave values of modulus of rupture in the following descending order of magnitude:

(*a*) Center loading, bending moment computed at center of span;

(*b*) Center loading, bending moment computed at plane of fracture;

(*c*) Third-point loading.

2. The flexural strength of the concrete increased as the maximum size of the coarse aggregate became smaller.

3. With a constant ratio of span length to depth, higher flexural strengths were obtained on specimens having the smaller cross section.

4. With a constant cross section, higher flexural strengths were obtained on specimens having the shorter span.

B. Uniformity.—1. The uniformity of flexure test results was affected by the method of loading the specimen and by the method of computing the bending moment. The three methods used in these tests are given below in order of decreasing uniformity.

(*a*) Third-point loading;

(*b*) Center loading, bending moment computed at center of span;

(*c*) Center loading, bending moment computed at plane of fracture.

2. The uniformity of flexure test results was affected by the maximum size of the coarse aggregate employed, smaller aggregates producing somewhat more uniform results.

3. With a given ratio of span length to depth the specimens having the smaller cross section gave the more consistent results.

4. In tests of specimens of constant cross section tested on 18 and 27 inch spans, the shorter span produced the more uniform results in the case of the third-point loading and the center loading, with moment computed at center. In the case of the center loading with moment computed at plane of fracture, the reverse is generally true.

C. Recommendations.—It is recommended that the third-point method of loading and a cross section of 6 by 6 inches be standardized for laboratory work. In regard to span length no recommendation is made.

Although the compression test has long been used as a measure of the strength of Portland cement concrete and the procedure for making the test has been standardized, the results do not in all cases supply data for the correct design of a concrete pavement slab. Concrete for this purpose is usually investigated to determine its resistance to bending stresses and to do this it is necessary to test the concrete in flexure. While this test has been used regularly by a number of the State highway epartments for several years it is only quite recently that there has been any standardized procedure for making it. As a result, it has been very difficult to correlate the large amount of test data which have been obtained by the States. The problem has been further complicated by the fact that, for purposes of control testing, it has become customary to make flexure tests directly on the job rather than in the laboratory. This has resulted in the development of a number of different field testing machines which differ considerably in design and, consequently, give different results. A report showing the extent to which variation in the type of testing machine may affect the results of flexure tests was published in Public Roads, volume 12, No. 12, February, 1932.

Variations in flexural strength may be due to many causes, ranging from differences in the quality of the concrete itself to differences in the procedure followed in making and testing the specimens and in computing the results. Variations due to methods of conducting the test may be caused by variations in any one or more of the following: (1) Method of loading; (2) rate of application of load; (3) cross-sectional dimension of specimen; (4) length of span; (5) method of computing bending moment.

Recently the committee on materials of the American Association of State Highway Officials, realizing the need for a standardized procedure, sponsored a series of cooperative tests to determine the variation in strength which different laboratories would obtain between their laboratory and field methods of testing. The results of these cooperative tests indicated clearly that it would be necessary to standardize the laboratory method before progress could be made with field standardization. A second series of cooperative tests were therefore conducted, the results of which were published in Public Roads, volume 12, No. 2, April, 1931. In this series all specimens were made and tested as simple beams in the laboratory, the load being applied at the third points by means of special apparatus designed for the purpose.

The need for a standard method of making and testing concrete beam specimens has been recognized by Committee C–9 of the American Society for Testing Materials, and as a result a tentative standard for this test was published in 1930 (A.S.T.M. designation C 78–30 T). This method proposes that the specimens be tested as simple beams with the load applied at the center of the span and may, therefore, give quite different results from the so-called third-point loading used by the American Association of State Highway Officials.

The tests herein reported were undertaken for the purpose of determining what effect this variation in method, as well as certain other variables, would have upon the results of flexure tests. The following variables were investigated: (*a*) Method of loading; (*b*) Size of specimen; (*c*) Span length; (*d*) Maximum size of coarse aggregate.

While the primary purpose of this investigation was to study the uniformity of the results, information was obtained showing how the above-mentioned variables affected the results quantitatively.

MATERIALS AND PROPORTIONS

Fine aggregate used in these tests was a bank-run sand having the physical properties shown in Table 1. This table also gives the results of the physical tests on the cement. The coarse aggregate was a limestone, which tends to produce concrete of relatively high flexural strength. It was screened into separate sizes and recombined into four definite gradings, as follows:

Percentage passing (square openings)

Grading	¼-inch	¾-inch	1¼-inch	2-inch	2½-inch
A	0	20	45	80	100
B	0	30	60	100	--------
C	0	40	100	--------	--------
D	0	100	--------	--------	--------

It will be observed from these grading combinations that there is a rather wide range in maximum size with a corresponding change in the void content of the material in a dry-rodded condition. Table 2 gives the results of physical tests on the coarse aggregate.

In designing the proportions for the various grading combinations the cement factor and consistency were held practically constant and the percentage of sand in the mix varied in order to compensate for a change in void content in the coarse aggregate. Data pertaining to the proportions and consistency will be found in Table 3.

FABRICATION AND STORAGE

All concrete was mixed in pans with shovels, one batch being of sufficient size to make one specimen. Three sizes of beam specimens were made—6 by 6 by 21 inches, 6 by 6 by 30 inches, and 8 by 8 by 27 inches. The 6 by 6 inch beams were molded in two layers, each layer rodded with a ⅝-inch rod, and spaded on the sides and ends. After the top layer had been poured the concrete was struck off and finished with a cork float. The 8 by 8 inch beams were molded in three layers. Each layer was rodded sixty times per square foot of area for all specimens. This required rodding the 6 by 6 by 21 inch beams fifty-four times per layer, the 6 by 6 by 30 inch beams seventy-five times per layer and the 8 by 8 by 27 inch beams ninety times per layer.

All specimens were covered with wet burlap immediately after fabrication and kept covered for approximately 24 hours, at which time they were removed from the molds and placed in a moist room for 27 days storage prior to test.

The procedure followed was to make 24 beams each working day. This constituted one round of tests and furnished one specimen for each combination of grading of aggregate, size of specimen, and method of test.

At the 28-day period, the specimens were tested in a 100,000-pound Universal testing machine as simple beams, the load being applied by a handwheel at a speed which would produce a stress of 150 pounds per square inch per minute in the extreme fibers (A. S. T. M. tentative standard).

Half of the beams were tested with the load applied at the third points and half with the load applied at the center of the span. Figure 1 shows the method of applying the load to the specimen while Figures 2 and 3 are photographs of a specimen set up in the testing machine. In computing the modulus of rupture in the case of specimens loaded at mid span, the bending moment was computed at the plane of fracture and also under the load at the center of the span. Two sets of results for modulus of rupture were therefore obtained on all specimens loaded at mid span. In addition to the variables in testing previously mentioned one set of 6 by 6 by 30 inch beams using ¾-inch aggregate was tested with the side instead of the bottom in tension as was the case with all other specimens.

TABLE 1.—*Physical properties of cement and fine aggregate*

CEMENT	
Fineness, percentage retained on 200-mesh sieve	7.8
Time of set (Gillmore):	
Initial	2 h. 52 m.
Final	4 h. 40 m.
Steam test for soundness	(1)
Normal consistency, per cent	24.2
Tensile strength (1:3 Ottawa sand mortar):	
7 days ... pounds per square inch	380
28 days ... do	420
FINE AGGREGATE	
Sieve analysis:	
Total retained on No. 4 sieve ... per cent	0
Total retained on No. 8 sieve ... do	11
Total retained on No. 16 sieve ... do	34
Total retained on No. 30 sieve ... do	61
Total retained on No. 50 sieve ... do	85
Total retained on No. 100 sieve ... do	94
Fineness modulus	2.85
Silt and clay ... per cent	2.8
Apparent specific gravity	2.66
Weight in pounds per cubic foot (dry-rodded)	102
Organic matter (color test)	(1)
Strength ratio:	
7 days	132
28 days	113

Description: Sand consists essentially of angular grains of quartz, chert, gneiss, and sandstone.

[1] Satisfactory.

TABLE 2.—*Physical properties of coarse aggregate*

Specific gravity	2.72
Absorption ... per cent	.12
Wear ... do	[1] .44

Grading	Weight per cubic foot, dry-rodded	Voids
	Pounds	*Per cent*
A	103	39.1
B	102	39.7
C	101	40.3
D	98	42

[1] Not standard test; made with crushed rock.

TABLE 3.—*Proportions and consistency of concrete*

Grading	Maximum size aggregate	Proportions by dry-rodded volumes	W/C [1]	Cement factor	Mortar voids ratio	b/b_0
	Inches			*Bags per cubic yard*	*Per cent*	
A	2½	1:2.0:3.30	0.75	6.01	192	0.735
B	2	1:2.03:2.27	.76	6.03	193	.730
C	1¼	1:2.09:3.21	.77	6.04	198	.718
D	¾	1:2.21:3.05	.84	6.04	211	.682

[1] *W/C* corrected for absorption. Consistency approximately 2½-inch slump.

ANALYSIS OF DATA

Detailed results of the strength tests, giving values for the individual specimens, were prepared in mimeograph form, but are not included in the report because of lack of space. They give the individual strength values for all 20 rounds of tests and the percentage variation of each value from the average. The average strengths and the average variations are also shown. While some values reported show a rather wide variation from the average for the group, they were not excluded because of the fact that it was the primary object of these tests to determine the extent of just such variations.

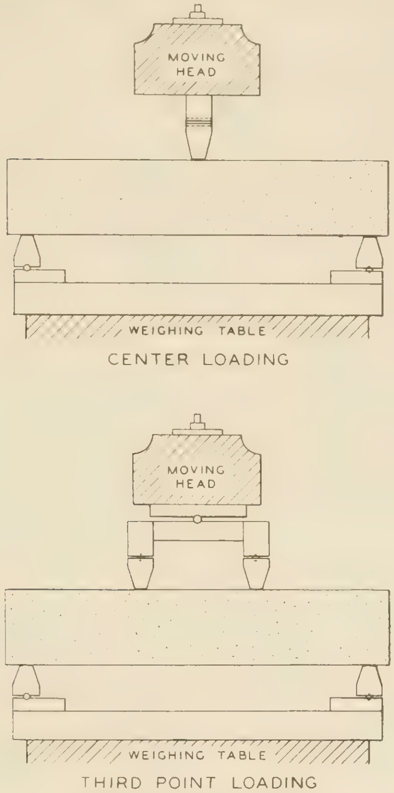


FIGURE 1.—APPARATUS FOR TESTING SPECIMENS IN 100,000-POUND TESTING MACHINE

Persons desiring to make a study of these detailed tables may obtain them upon request addressed to the bureau.

The average strengths and average deviations are also given in summary form in Table 4, and, for purposes of study, are shown in graphic form in Figures 4, 5, and 6.

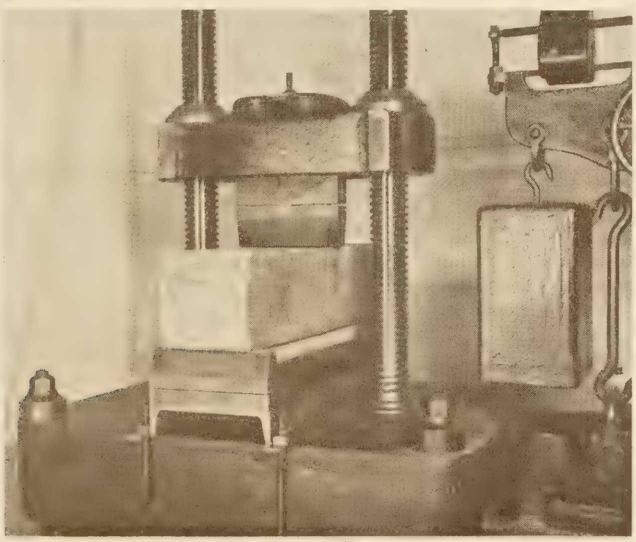


FIGURE 2.—SPECIMEN SET UP IN TESTING MACHINE, LOAD APPLIED AT CENTER OF SPAN

The discussion deals with the effect which each of the variables studied has on (1) the relative average strength of the concrete and (2) the uniformity of strength within each group of individual specimens from which the average values were calculated. Figures 4, 5, and 6 have been plotted from the data shown in the tables in order to bring out certain relationship which

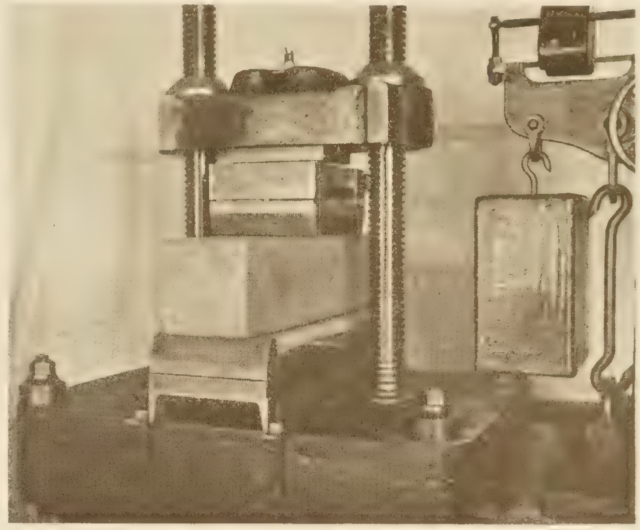


FIGURE 3.—SPECIMEN SET UP IN TESTING MACHINE, LOAD APPLIED AT THIRD POINTS

TABLE 4.—*Average values of flexural strength at 28 days and average percentage variations of individual values from the average*

Maximum size aggregate (inches)	Third-point loading		Center loading, moment at plane of fracture		Center loading, moment at center of span	
	Modulus of rupture	Variation	Modulus of rupture	Variation	Modulus of rupture	Variation
	Lbs. per sq. in.	*Per cent*	*Lbs. per sq. in.*	*Per cent*	*Lbs. per sq. in.*	*Per cent*
6 BY 6 BY 21 INCH BEAMS—18-INCH SPAN						
2½	589	5.1	616	7.1	684	5.7
2	621	4.3	655	7.8	704	4.4
1¼	637	3.4	677	7.5	721	4.4
¾	672	4.1	705	6.0	753	3.6
Average		4.2		7.1		4.5
6 BY 6 BY 30 INCH BEAMS—27-INCH SPAN						
2½	572	5.6	608	7.9	646	7.4
2	573	7.3	632	6.2	663	5.0
1¼	605	5.7	646	6.3	684	5.1
¾	622	6.2	665	4.9	696	4.5
¾ [1]	641	5.2	652	6.3	693	4.2
Average		6.0		6.3		5.2
8 BY 8 BY 27 INCH BEAMS—24-INCH SPAN						
2½	534	5.2	573	10.3	620	8.5
2	562	4.7	590	7.0	646	5.5
1¼	576	4.2	628	6.8	669	5.3
Average		4.7		8.0		6.4
Grand average		5.1		7.0		5.3

[1] These specimens tested with side in tension, all others tested with bottom in tension.

appear to exist. These indications are discussed in the following order:

1. The effect of variations in the method of loading and in the method of computing the bending moment. (Fig. 4.)
2. The effect of changing maximum size of aggregate. (Fig. 4.)
3. The effect of changing the cross-sectional dimensions of the specimens when the ratio of depth of beam is to span constant at 1 to 3. (Fig. 5.)
4. The effect of changing the ratio of depth of beam to span length when the cross section is constant. (Fig. 6.)

The data plotted in the upper portion of each of the figures referred to above are the average strengths of the group of specimens representing each test condition. In most cases each point represents the average of 20 individual tests made on different days in accordance with the schedule which has been described. The data plotted in the lower part of each figure gives, for each test condition, the average per cent variation from the average of the group.

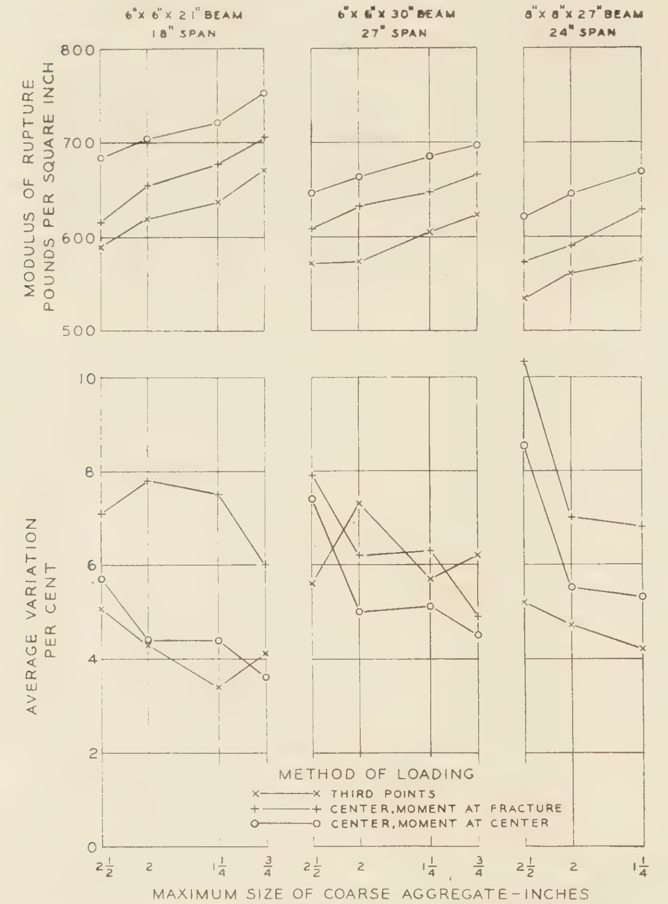


FIGURE 4.—EFFECT OF METHOD OF LOADING AND MAXIMUM SIZE OF COARSE AGGREGATE ON AVERAGE VALUES OF MODULUS OF RUPTURE AND AVERAGE PERCENTAGE VARIATION OF TEST RESULTS

Effect of method of loading and method of computing bending moment.—From Figure 4 it is evident that both the method of loading the specimen and the method of computing the bending moment have a marked effect upon the modulus of rupture. An examination of the curves in the upper portion of this chart shows that the method of loading the specimens at the third points produces the lowest results in flexure while the highest results are obtained by applying the load at the center of the span and computing the moment at the same place. Intermediate results are obtained by loading the specimen at the mid span and computing the moment at the plane of fracture. This relationship holds for all sizes of specimen and all maximum sizes of coarse aggregate without exception. If the specimens were fabricated from a material absolutely homogeneous in character and loaded at center of span, all of them would break at the center (plane of maximum moment). Concrete is no such material and very frequently when tested with the load applied at the center of the span the specimen will break at a plane somewhere between the point of application of the load and one of the reaction points. In such cases if the bending moment is computed at the plane of fracture, a lower flexural strength will be obtained than if the moment is computed at the center of span. For instance, in Figure 4, with 8 by 8 inch specimens, 2½-inch aggregate, the average value for the group was 573 pounds per square inch when the bending moment was computed at the plane of fracture, while the same specimens gave an average value of 620 pounds per square inch when the bending moment was computed at the center, a difference of 8 per cent. If individual cases are examined, greater differences will be noted. Thus, on round No. 1 of this series of tests a value of 508 pounds per square inch was obtained when the moment was computed at the plane of fracture while the corresponding value obtained when the moment was computed at the center of the span was 609 pounds per square inch, a difference of 20 per cent. This difference was caused by the specimen breaking 2 inches off center.

The uniformity of test results as affected by the method of loading is shown in the lower portion of Figure 4. It is interesting to note, in the two cases where the span length is three times the depth (left and right hand groups in the figure) that loading at the third points generally gives the most consistent results, with the center-loading method, moment computed at plane of fracture, always giving the least consistent results. For the span ratio of 1:4½ (6 by 6 by 30 inch beams) loading at center with moment computed at center gives in most cases the greatest uniformity.

Table 5, A shows the strengths obtained with center loading expressed as a percentage of the strength when tested at the third points. These percentages are shown for moments computed at plane of fracture and at the center of beam, as well as for each size of beam. It will be observed that the average strength of the concrete when tested with center loading exceeds that obtained from tests at the third points by about 7 per cent when the bending moment is computed at the plane of fracture; and by about 14 per cent when the moment is computed at the center of the beam.

Effect of maximum size of coarse aggregate.—These relationships are shown in Figure 4 for each size of beam and for each method of loading. In all cases, decreasing the maximum size of the aggregate increases the modulus of rupture. In this connection it is of interest to refer to Table 3 and to note that practically identical cement factors were used in the concretes containing the various maximum size aggregate and, moreover, that because of this fact the net water-cement ratio increased as the maximum size became smaller. This fact would lead one to assume that the actual strength would decrease instead of increase. However, the results are consistent and they would appear to indicate either that the relation between water-cement ratio and flexural strength varies with the maximum size, or that the cross sections of the test specimens were too small to indicate the true relation.

In considering the effect of size of aggregate upon the flexural strength it should be remembered that the proportions for the smaller aggregate contained a greater percentage of sand than those for the larger aggregate combinations. The percentage of sand in the mixes for gradings A, B, and C is in proportion to the voids in the coarse aggregate. Previous tests with an aggregate similar to the one used in this investigation where two gradings corresponding to gradings B and C were employed showed slightly higher strengths

TABLE 5.—*Strength ratios*

A.—EFFECT OF METHOD OF LOADING

Maximum size aggregate (inches)	Flexural strength expressed as percentage of that given by third-point loading					
	Center loading, moment at fracture			Center loading, moment at center		
	6 by 6 by 21 inch beams	6 by 6 by 30 inch beams	8 by 8 by 27 inch beams	6 by 6 by 21 inch beams	6 by 6 by 30 inch beams	8 by 8 by 27 inch beams
2½	105	106	107	116	113	116
2	105	110	105	113	116	115
1¼	106	107	109	113	113	116
¾	105	107		112	112	
Average	105	108	107	114	114	116

B.—EFFECT OF MAXIMUM SIZE OF AGGREGATE

Method of loading	Flexural strength expressed as percentage of strength of concrete containing 2½-inch maximum size aggregate		
	2-inch	1¼-inch	¾-inch
6 by 6 by 21 inch beams, 18-inch span:			
Third point	105	108	114
Center, moment at fracture	106	110	114
Center, moment at center	103	105	110
6 by 6 by 30 inch beams, 27-inch span:			
Third point	100	106	109
Center, moment at fracture	104	106	109
Center, moment at center	103	106	108
8 by 8 by 27 inch beams, 24-inch span:			
Third point	105	108	
Center, moment at fracture	103	110	
Center, moment at center	104	108	
Average	104	107	111

C.—EFFECT OF SIZE OF BEAM

Maximum size aggregate (inches)	Flexural strength expressed as percentage of strength of 8 by 8 by 27 inch beams					
	6 by 6 by 21 inch beams			6 by 6 by 30 inch beams		
	Third-point loading	Center loading		Third-point loading	Center loading	
		Moment at fracture	Moment at center		Moment at fracture	Moment at center
2½	110	108	110	107	106	104
2	110	111	109	102	107	103
1¼	111	108	108	105	103	102
Average	110	109	109	105	105	103

D.—EFFECT OF LENGTH OF SPAN

Maximum size aggregate (inches)	Flexural strength of 6 by 6 by 21 inch beams, 18-inch span, expressed as percentage of strength of 6 by 6 by 30 inch beams, 27-inch span		
	Third-point loading	Center loading	
		Moment at fracture	Moment at center
2½	103	101	106
2	108	104	106
1¼	105	105	105
¾	108	108	108
Average	106	104	106

for the finer grading, even though there was a difference of 0.15 of a sack of cement per cubic yard of concrete in favor of the coarser grading. This present series indicates a possible change in flexural strength of as much as 14 per cent where the consistency and cement factor were held constant. (Table 5, B.)

As to uniformity, it will be observed that there is a distinct tendency for greater uniformity for the smaller maximum sizes. These relationships are shown in the lower part of Figure 4.

Table 5, B shows the effect of size of aggregate expressed as a percentage of the strength obtained with the 2½-inch material. It is noted that a decrease in maximum size to three-fourths inch increases the strength about 10 per cent; also that the effect of size is about the same for the two cross sections and for the two ratios of depth to span, except that in the latter case the shorter span shows a somewhat greater increase for the ¾-inch size. The method of loading, likewise, appears in general to exert little influence on this relation.

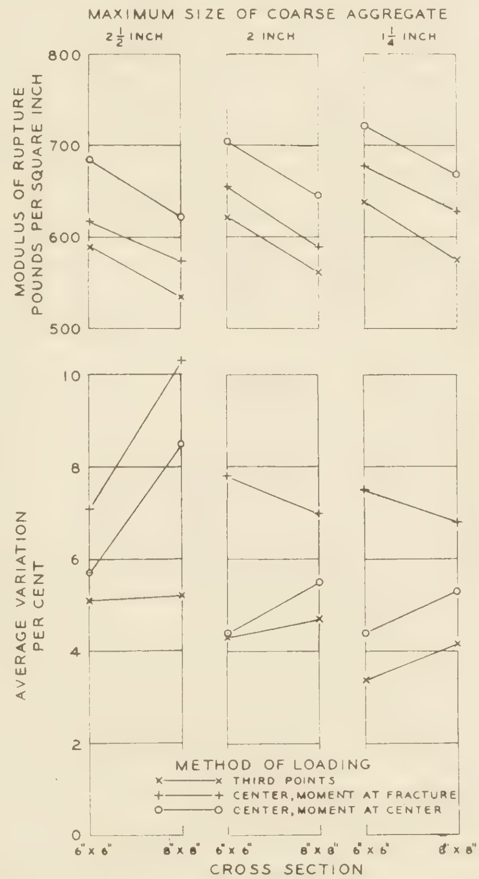


FIGURE 5.—EFFECT OF SIZE OF BEAM ON AVERAGE VALUES OF MODULUS OF RUPTURE AND AVERAGE PERCENTAGE VARIATION OF TEST RESULTS. SPAN IS THREE TIMES DEPTH

Effect of variations in cross section with ratio of depth to span constant.— The effect of increasing the cross-sectional dimensions of the specimen while maintaining the ratio of depth to span length constant at 1:3 is shown in Figure 5. It will be observed that for all three methods of loading, as well as for all maximum sizes of coarse aggregate, the smaller cross section gives the higher results. These observations are in line with the results of tests reported by a subcommittee of the committee on materials of the American Association of State Highway Officials, and published in the April, 1931, issue of PUBLIC ROADS. In this series various depths and widths of beam specimens ranging from 4 by 4 inches to 10 by 10 inches were tested. It was observed that the modulus of rupture of the concrete decreased in direct proportion as the depth of the beam increased and it was pointed out that this dimension will have to be standardized if comparable results are to be expected.

From the standpoint of uniformity (lower portion of fig. 5) the results are not so consistent, although, in general, it may be said that the 6 by 6 inch beams give somewhat more uniform results than the 8 by 8 inch beams. This is true in all cases of third-point loading and also for center loading, moment at center.

The percentage differences are shown in Table 5, C, where the strength obtained on the 6 by 6 by 21 inch beams are shown as percentages of the strength of the 8 by 8 by 27 inch beams. The 6 by 6 inch cross section gave results about 10 per cent higher than the 8 by 8 inch section.

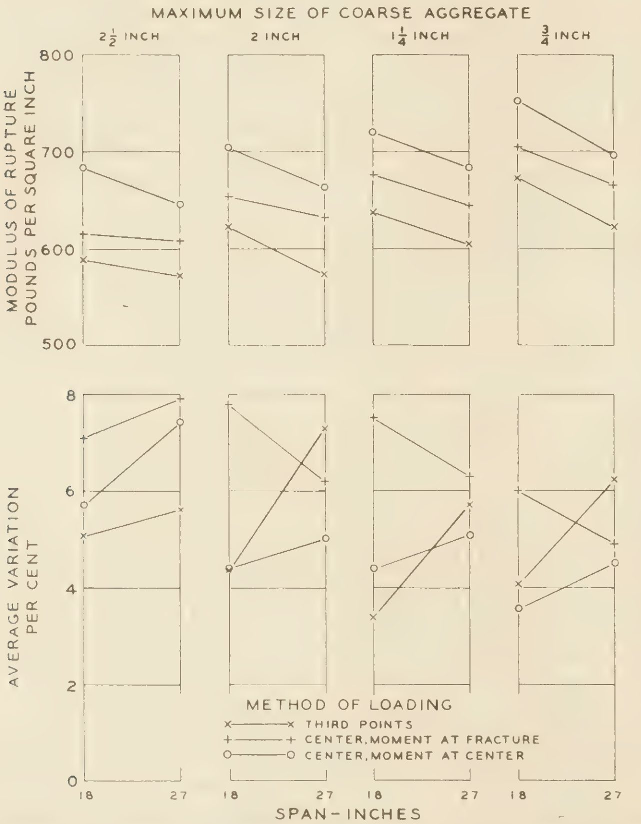


FIGURE 6.—EFFECT OF LENGTH OF SPAN ON AVERAGE VALUES OF MODULUS OF RUPTURE AND AVERAGE PERCENTAGE VARIATION OF TEST RESULTS, FOR 6 BY 6 INCH BEAMS

The strength obtained in tests of 6 by 6 by 30 inch beams are also shown in Table 5, C as percentages of the strengths of 8 by 8 by 27 inch beams. While the ratio of depth to span length is not the same in these two cases, it is of some significance that the 6 by 6 inch beams give somewhat higher strengths (approximately 5 per cent) than the 8 by 8 inch beams.

Effect of variations in ratio of depth of beam to span length for constant cross section.—This relationship, for each method of loading and for each maximum size of coarse aggregate, is shown in Figure 6. In all cases the specimens tested on the shorter span are somewhat higher in strength. In this respect the data do not check the results secured either by the committee on materials as reported in PUBLIC ROADS, volume 12, No. 2, April, 1931, or the work of the Portland Cement Association as reported by F. R. McMillan, director of research in 1928. McMillan, using beams 7 inches in depth by 10 inches in width, found the effect of span to be unimportant in the case of third-point loading. In the case of center loading, however, he found that the modulus of rupture decreased as the span increased.

When the data are analyzed with respect to uniformity, it will be observed that the shorter span length gives the most consistent results (lower average percentage variations) in all but three cases, all of which are for beams loaded at the center with moment calculated at plane of fracture. Each group of beams loaded at the third points show the most consistent results for the shorter span. This also holds for center loading with moment computed at the center of the beam.

Reference to Table 5, D will show that, if strengths are expressed on a percentage basis, the specimens tested on the shorter span (ratio of depth of beam to span length, 1:3) gave results about 5 per cent higher than those tested on the longer span (ratio 1:4½). The average percentage of increase is about the same for all methods of loading. For any given method of loading the percentage of increase, as affected by maximum size of aggregate, varies from 1 to 8 per cent, with a tendency for the concrete containing the 2½-inch aggregate to show somewhat lower ratios than the concrete containing the ¾-inch material. This will be discussed further in the next section.

DISCUSSION

The primary purpose of this series of tests was to determine which combination of the several variations in test procedure gave the most satisfactory results from the standpoint of uniformity. Reference to Table 4 will show that the grand average deviation for all specimens (236) tested at the third points was 5.1 per cent. The corresponding average for 239 specimens tested at the center of the span was 5.3 per cent when the moment was computed at the center but rose to 7 per cent for the same specimens when the moment was computed at the plane of fracture. Of the group tested at the third points, the 6 by 6 inch beams tested on 18-inch span gave the lowest average deviation, 4.2 per cent, as compared to 5.1 per cent for the entire group and 4.5 per cent for similar specimens tested with center loading, moment at center.

A possible reason for the higher variations obtained with the center-loading method where the moment was computed at the plane of fracture is the inability to determine just where this plane is with any degree of accuracy. With 2½-inch aggregate it is not hard to see that an error of one-half inch could easily be made in measuring the distance from the reaction to the plane of fracture. An illustration will show just what effect such an error would have upon the modulus of rupture. For instance, assume a beam specimen 8 by 8 inches tested on a 24-inch span, with the load applied at the center of the span. Assume also that the load required to break this specimen was 9,000 pounds and that the specimen broke 1 inch off center or 11 inches from one reaction.

Using the ordinary formula for computing the modulus of rupture and neglecting the dead weight of the beam itself, we obtain a value of 580 pounds per square inch when the moment arm is taken at 11 inches. An error in measurement of one-half inch, giving, say, a distance from the reaction of 10½ inches, would give a modulus of 554 pounds per square inch, a variation of 4.5 per cent due to this cause alone.

The simplest method is of course to assume the moment arm as one-half the span, regardless of where the beam breaks. This is the method ordinarily employed in practice. The value so computed is theoretically correct only for cases where failure occurs exactly at the center. For all other cases the true modulus of rupture at the section at which failure occurred will always be less than the apparent modulus computed at the center. For such cases the value reported as the modulus of rupture is not the stress which caused failure at all but simply the theoretical maximum fiber stress existing in the concrete at the center of the span at the moment of failure.

From the above discussion it would appear that both methods of computing the modulus of rupture where the specimen is loaded at the center are open to objection, the first because of the difficulty of accurately measuring the true moment arm, and the second because the value obtained does not represent the stress in the section of failure unless the specimen breaks exactly in the center, a condition not always observed.

By loading at the third points both of these objections are overcome. Provided the specimen breaks within the middle third of the span, it is not necessary to measure the moment arm because the moment is constant over the middle third and the value obtained will represent the true modulus of rupture of the section at failure.

In a further effort to throw light on the matter of uniformity the bar diagrams shown in Figure 7 have

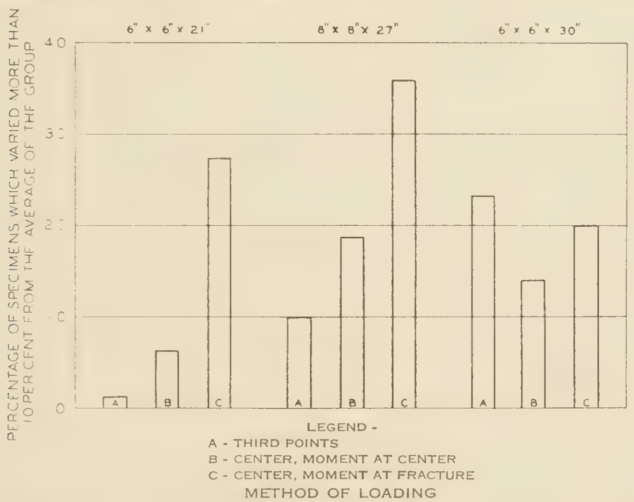


FIGURE 7.—UNIFORMITY OF RESULTS FOR DIFFERENT TEST CONDITIONS AS INDICATED BY PERCENTAGE OF SPECIMENS WHICH VARIED MORE THAN 10 PER CENT FROM AVERAGE OF GROUP.

been prepared. These figures show the percentage of the total number of specimens in each group indicated which gave results varying more than 10 per cent from the average of the group. It is evident that, from this standpoint, the third-point method, using 6 by 6 inch beams tested on 18-inch span, gave the most consistent results. It should be remembered when studying these charts that each point represents a group of approximately 80 specimens in the case of the two 6 by 6 inch series and approximately 60 specimens in the case of the 8 by 8 inch series, and that the ¾-inch aggregate was not included in the fabrication of the 8 by 8 by 27 inch specimens.

The bar diagrams shown in Figure 8 afford a comparison between the results obtained from 6 by 6 by 30

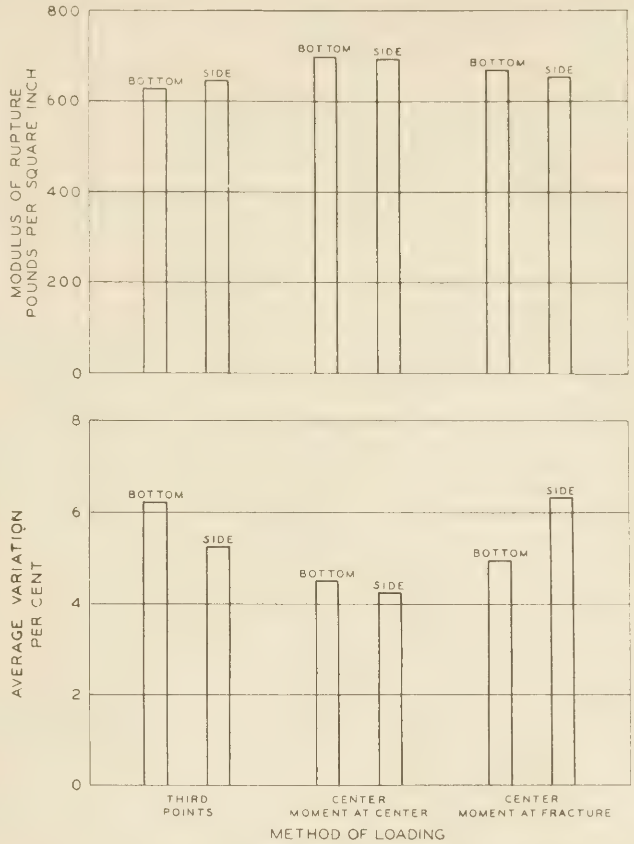


FIGURE 8.—COMPARISON OF RESULTS GIVEN BY SPECIMENS TESTED WITH BOTTOM, AS MOLDED, IN TENSION, WITH THOSE GIVEN BY SPECIMENS TESTED WITH SIDE IN TENSION

inch specimens tested with the bottom (as molded) in tension, and those obtained from similar specimens tested with the side in tension. The differences in average modulus of rupture are slight. In the case of third-point loading the specimens tested with the side in tension gave somewhat the higher values. The reverse is true in the case of center loading, by both methods of computation.

The diagrams showing average variation indicate, in two cases out of three, a slight advantage in favor of the specimens tested with the side in tension. However, these tests are not conclusive enough to warrant a statement as to whether specimens should be tested with the side or the bottom in tension.

A short series of tests in which a rounded coarse aggregate was used in place of crushed stone was conducted for the purpose of giving some information regarding the effect of type of aggregate on strength and uniformity. Tests were made on one size of specimen only (6 by 6 by 30 inch) and on one maximum size of coarse aggregate (1¼-inch). Comparison of the method of loading at the third points and center loading shows the same trend as with the crushed aggregate. Loading at the third points gave the lowest strengths while center loading, moment taken at the center of the beam, gave the highest.

SUMMARY

From the data obtained in these tests the following indications have been summarized.

A. Relative flexural strength.—1. The flexural strength of the specimens was influenced by the method of loading and of computing the results. The three methods which were investigated gave values of the modulus of rupture in the following descending order of magnitude:

(*a*) Center loading, bending moment computed at center of span;

(*b*) Center loading, bending moment computed at plane of fracture;

(*c*) Third-point loading.

2. The flexural strength of the concrete increased as the maximum size of the coarse aggregate became smaller.

3. With a constant ratio of span length to depth, higher flexural strengths were obtained on specimens having the smaller cross section.

4. With a constant cross section, higher flexural strengths were obtained on specimens having the shorter span.

B. Uniformity.—1. The uniformity of flexure test results was affected by the method of loading the specimen and by the method of computing the bending moment. The three methods used in these tests are given below in order of decreasing uniformity.

(*a*) Third-point loading;

(*b*) Center loading, bending moment computed at center of span;

(*c*) Center loading, bending moment computed at plane of fracture.

2. The uniformity of flexure test results was affected by the maximum size of the coarse aggregate employed, smaller aggregates producing somewhat more uniform results.

3. With a given ratio of span length to depth the specimens having the smaller cross section gave the more consistent results.

4. In tests of specimens of constant cross section on 18 and 27 inch spans, the shorter span produced the more uniform results in the case of the third-point loading and the center loading with moment computed at center. In the case of the center loading with moment computed at plane of fracture, the reverse is generally true.

RECOMMENDATIONS

1. *Method of loading.*—From the standpoint of uniformity of test results there would appear to be but little choice between the third-point method of loading and a center loading with the moment computed at the center of the span. However, it is felt that the theoretical objections raised against the latter method, which have been discussed in this report, coupled with the fact that the uniformity of the results obtained by the third-point method of loading is as good as with the center loading, warrant a recommendation that the third-point method of loading be standardized for laboratory work.

2. *Cross section of specimen.*—The data indicate that specimens having cross-sectional dimensions of 6 by 6 inches give fully as uniform results as specimens of 8 by 8 inch cross section. Moreover, the decrease in uniformity due to increasing the maximum size of the coarse aggregate up to 2½ inches is so small as to be negligible from a practical point of view. It is recommended, therefore, that a cross section of 6 by 6 inches be standardized for laboratory testing.

3. *Span length.*—The results indicate a slight advantage from the standpoint of uniformity in favor of the shorter span. The data also indicate that the modulus of rupture is decreased by increasing the span. As far as the third-point method of loading is concerned, these results are in conflict with the results reported by both the committee on materials, American Association of State Highway Officials and the Portland Cement Association. In view of this fact, and also because only two span lengths were investigated, no recommendations regarding this dimension are made.

(Continued from p. 176)

originally a penetration of 164 and a softening point of 40° C., and three parts of an asphaltic oil having an original specific viscosity of 169 at 40° C. and 44 at 50° C. The resultant residue had a penetration of 76 and a softening point of 55° C., approximating a normal asphalt cement, and was fairly soft and plastic, indicating its ability to continue to hold the cover stone without excessive cracking in cold weather or bleeding in hot weather.

The cost of maintaining the wearing surface of the project through the 6-year period, while varying among the sections, is considerably less than that of an untreated topsoil carrying much less traffic. Data obtained in a survey of low-cost roads by C. N. Connor [5] indicate that the corresponding maintenance cost of untreated topsoil and sand-clay roads averages between $300 and $600 per mile annually and that such roads, ordinarily, serve economically and satisfactorily a maximum of about 400 vehicles daily, while under heavier traffic the maintenance cost increases rapidly.

Excluding the cost of the nonskid treatments which, while they obviously added somewhat to the betterment of the areas affected, were applied principally as a safety factor, the maintenance ranged from $135 to $345 per mile annually and averaged $270. Including the nonskid treatment the average annual cost was $324 per mile. While this cost may be higher than present-day maintenance of similar type, it should be recalled that little preliminary work was done on the road prior to applying the surface treatments and also that the project was made up of relatively short experimental sections. The crown, while satisfactory for the old sand-clay type, was excessive for a surface-treated road and is believed to have added somewhat to the cost of maintenance.

Traffic records show that in 1924 the road was carrying an average of 636 and a maximum of 800 vehicles daily based on a count through the summer months only. For the fiscal year 1930–31 a monthly count showed the sections to be carrying an average of 956 vehicles daily with a maximum of 1,402 which is an increase of over 50 per cent. As shown in Table 1 the maintenance cost of the sections, with the exception of the first year after construction which usually is relatively high, continued very uniform in spite of the great increase in traffic.

In addition to the decided economy of maintenance and the providing of a surface of the required carrying capacity, other advantages which can not be estimated in dollars and cents, such as reduced operating expense, elimination of dust, all-weather surfaces, and increased surface smoothness were obtained and could be retained indefinitely at a reasonable cost.

[5] Pt. 2 of the Proceedings of the Seventh Annual Meeting of the Highway Research Board.

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

DECEMBER 31,1932

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION Estimated total cost	UNDER CONSTRUCTION Federal-aid allotted	UNDER CONSTRUCTION Percentage completed	UNDER CONSTRUCTION MILEAGE Initial	UNDER CONSTRUCTION MILEAGE Stage[1]	UNDER CONSTRUCTION MILEAGE Total	APPROVED FOR CONSTRUCTION Estimated total cost	APPROVED FOR CONSTRUCTION Federal-aid allotted	APPROVED FOR CONSTRUCTION MILEAGE Initial	APPROVED FOR CONSTRUCTION MILEAGE Stage[1]	APPROVED FOR CONSTRUCTION MILEAGE Total	BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
Alabama	2,356.8	$ 4,556,923.95	$ 2,278,461.89	22	149.5	71.0	220.5	$ 55,684.41	$ 27,842.20	0.2		0.2	$ 3,780,068.45	Alabama
Arizona	1,199.2	2,949,843.07	1,413,090.53	68	89.6	167.9	257.5	87,905.53	17,581.10	15.6	1.2	16.8	204,794.94	Arizona
Arkansas	1,951.4	2,813,712.52	1,309,563.67	76	82.2	38.4	120.6	1,051,861.97	525,899.92	24.9		24.9	1,789,683.07	Arkansas
California	2,476.4	8,388,931.59	2,647,321.90	64	150.1	33.2	183.3	1,151,372.82	110,397.61	34.0		34.0	8,200.71	California
Colorado	1,726.3	2,895,098.75	1,383,452.79	77	120.8	51.0	171.8	1,030,442.59	463,699.06	53.4		53.4	829,661.17	Colorado
Connecticut	289.8	4,202,606.92	1,600,288.57	71	36.8		36.8	293,364.98	136,961.58	6.9		6.9	235,454.15	Connecticut
Delaware	374.5	591,801.95	148,953.90	74	16.2	11.2	27.4	217,621.10	40,800.00	10.9	6.0	16.9	43,564.26	Delaware
Florida	649.0	5,975,361.00	2,849,920.47	49	140.7	11.2	151.9	259,631.22	129,815.59	17.1		17.1	1,269,128.83	Florida
Georgia	3,184.0	5,667,220.89	2,292,790.32	50	163.6	161.9	325.5	605,806.82	259,140.89	23.3	38.7	62.0	136,800.60	Georgia
Idaho	1,483.7	2,659,799.54	1,135,270.07	81	136.1	156.3	292.4	552,631.35	176,247.73	14.1	19.3	33.4	88,957.93	Idaho
Illinois	3,049.2	20,367,431.33	8,175,776.16	80	630.5	33.0	663.5	2,339,991.15	672,202.48	88.2		88.2		Illinois
Indiana	1,992.4	8,646,426.35	4,023,788.55	91	311.7	3.0	314.7	1,123,467.35	111,500.00	59.2		59.2	215,980.36	Indiana
Iowa	3,501.8	5,628,149.98	1,066,185.38	92	278.4	40.0	318.4	359,559.81	23,060.00	25.0	1.2	26.2	11,498.31	Iowa
Kansas	3,826.8	3,878,723.94	1,792,236.37	61	265.2	68.6	333.8	1,777,322.69	146,560.84	111.1	42.9	154.0	209,093.30	Kansas
Kentucky	1,865.3	4,923,364.48	1,769,101.76	54	221.0	128.0	349.0	297,079.27	79,800.00	10.4		10.4	147,325.49	Kentucky
Louisiana	1,617.8	6,155,353.72	2,766,064.79	52	58.4	14.1	72.5	1,119,578.58	421,437.65	8.7		8.7	23,644.29	Louisiana
Maine	819.1	1,631,574.45	450,859.77	71	52.7	.3	53.0	824,783.83	85,446.97	17.8	1.9	19.7	67,851.99	Maine
Maryland	823.0	1,154,092.52	458,441.25	86	47.8	5.6	53.4	667,376.80	30,160.00	24.4		24.4	34,050.11	Maryland
Massachusetts	862.5	4,625,191.90	1,486,801.48	71	51.9	4.9	56.8	809,885.04	90,000.00	17.0		17.0	45,594.16	Massachusetts
Michigan	2,273.9	7,418,458.62	3,064,832.45	69	294.2	101.2	395.4	997,000.00	364,117.50	77.6		77.6	232,367.49	Michigan
Minnesota	4,338.0	4,636,721.39	131,628.91	84	169.1	120.0	289.1	114,649.27	100.00	1.3	.4	1.7	27,912.79	Minnesota
Mississippi	1,832.8	5,240,861.56	2,580,805.16	63	195.5	70.1	265.6	647,288.94	323,644.43	26.6	4.8	31.4	3,836,022.85	Mississippi
Missouri	3,177.6	2,892,126.89	694,216.47	72	111.6	14.3	125.9	613,660.53	166,419.71	43.7		43.7	389,002.63	Missouri
Montana	2,657.2	7,698,670.86	4,321,375.04	57	609.8	253.2	863.0	1,117,847.85	628,152.16	78.8	37.1	115.9	85,122.27	Montana
Nebraska	4,265.6	5,642,108.55	2,647,662.53	39	160.7	130.5	291.2	16,436.22	8,218.11	.5		.5	307,133.27	Nebraska
Nevada	1,333.9	1,683,271.03	925,075.93	85	29.6	128.0	157.6	236,318.92	26,384.56	9.1	20.3	29.4	69,842.56	Nevada
New Hampshire	427.1	950,750.66	416,351.61	77	27.8	3.5	31.3						170,907.95	New Hampshire
New Jersey	624.2	7,724,869.29	2,708,060.69	64	68.8	.5	69.3	79,106.70	19,718.17	2.7		2.7	6,811.52	New Jersey
New Mexico	2,229.0	2,782,515.32	1,289,139.72	52	164.7	78.6	243.3	403,963.67	177,474.14	4.4	26.8	31.2	305,581.88	New Mexico
New York	3,448.9	18,886,994.40	5,883,985.00	55	486.5	31.2	517.7	2,447,343.00	1,061,798.55	47.7	3.6	51.3	43,750.00	New York
North Carolina	2,259.2	2,693,213.57	1,348,145.58	40	312.9	16.9	329.8	1,752,055.40	877,865.13	216.9	.4	217.3	1,859,234.71	North Carolina
North Dakota	5,138.8	4,781,076.77	2,080,296.59	51	399.2	481.0	880.2	916,881.06	295,644.97	64.4	254.0	318.4	124,145.97	North Dakota
Ohio	2,991.8	7,413,241.34	2,227,149.23	82	191.7	74.2	265.9	3,597,828.00	734,522.60	43.4	6.4	49.8	42,544.41	Ohio
Oklahoma	2,383.8	4,106,670.40	2,042,943.66	77	203.9	48.4	252.3	2,109,809.89	440,091.91	114.8	30.0	144.8	562,753.00	Oklahoma
Oregon	1,602.5	3,541,954.56	1,503,216.02	56	102.7	84.3	187.0	922,864.43	303,995.15	25.1	22.4	47.5	127,260.94	Oregon
Pennsylvania	3,135.2	10,120,469.97	3,406,989.18	65	379.4	9.8	389.2	3,634,638.17	847,799.27	118.3	4.2	122.5	1,675.85	Pennsylvania
Rhode Island	263.8	1,226,843.60	432,058.05	54	25.0	4.3	29.3	285,171.25	103,490.86	5.6	.2	5.8		Rhode Island
South Carolina	1,996.5	2,968,789.31	1,142,092.73	54	160.1	119.2	279.3	609,448.09	192,953.34	12.4	31.5	43.9	2,200.00	South Carolina
South Dakota	4,076.2	4,353,209.43	1,975,343.14	76	330.0	280.9	610.9	68,178.30	30,549.67	14.6		14.6	87,786.58	South Dakota
Tennessee	1,690.3	4,184,392.57	2,091,178.22	33	132.5	49.9	182.4	1,234,074.82	617,037.39	40.8	14.8	55.6	705,399.78	Tennessee
Texas	7,782.6	17,146,700.12	7,214,864.71	62	724.0	463.0	1,187.0	6,081,700.24	1,697,301.02	297.4	143.4	440.8		Texas
Utah	1,205.4	1,927,206.25	918,241.76	78	115.0	89.2	204.2	360,887.34	108,477.74	34.5	20.3	54.8	247,576.96	Utah
Vermont	364.7	994,586.22	328,925.86	89	45.1		45.1	56,067.00	6,000.00	2.4		2.4	15,804.25	Vermont
Virginia	1,941.5	3,943,384.27	1,730,745.60	50	196.4	38.1	234.5	832,320.68	227,710.29	22.5	13.9	36.4	757,623.59	Virginia
Washington	1,318.8	2,306,174.10	812,112.10	52	95.3	1.1	96.4	567,014.18	174,000.00	20.3		20.3	197,723.59	Washington
West Virginia	913.0	3,375,497.98	1,307,832.99	61	116.8	8.5	125.3	274,081.75	64,622.41	8.3		8.3	188,766.75	West Virginia
Wisconsin	2,714.1	7,335,076.72	1,605,535.94	81	184.8	112.5	297.3	52,061.50	21,500.00		.9	.9	2,443.67	Wisconsin
Wyoming	2,048.1	3,114,484.77	1,199,108.82	79	256.6	183.7	440.3	115,762.14	2,104.28	9.1	21.8	30.9	5,328.32	Wyoming
Hawaii	78.4	2,176,281.06	1,179,014.06	31	60.1		60.1	314,782.03	243,992.92	5.9	1.9	7.8	749,436.92	Hawaii
TOTALS	104,561.9	250,978,210.43	98,257,297.37	63	9,353.0	3,995.7	13,348.7	45,084,608.68	13,314,239.90	1,911.3	770.3	2,681.6	20,293,512.62	TOTALS
Construction completed		145,427,000	62,296,000											
Balance uncompleted		105,551,000	35,961,000											

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