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BUREAU OF PUBLIC ROADS

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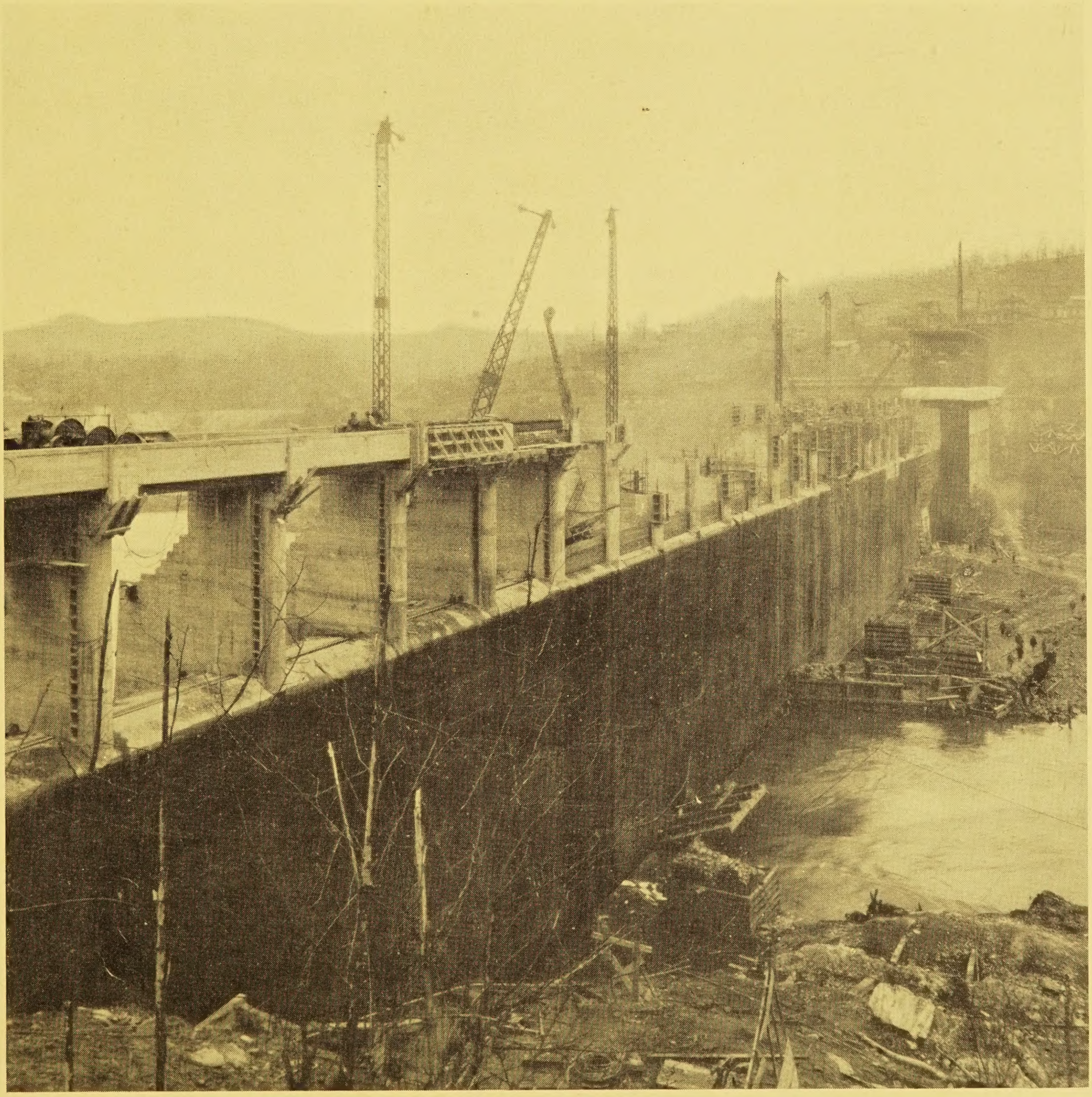

Cheat Haven Dam, Pennsylvania, Treated with Coal Tar

U. S. GOVERNMENT PRINTING OFFICE : 1927

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PROTECTION OF CONCRETE AGAINST ALKALI

FURTHER TESTS BY BUREAU OF PUBLIC ROADS ON TREATMENT OF CONCRETE WITH TAR AND PARAFFIN

Reported by E. C. E. LORD, Petrographer, United States Bureau of Public Roads

THE investigations carried on for some years by the Bureau of Public Roads to determine the degree of protection against alkali attack afforded by treatment of concrete with tar and paraffin have been continued, and additional information has been gained regarding the physical properties of the preservatives and their behavior when applied to various types of concrete.

SCOPE OF TESTS OUTLINED

The earlier tests, results of which have already been published,[1] were carried out on Portland cement concrete cylinders (4 by 6 inches) made of Potomac River sand and gravel graded in the proportions of 1:1½:3, 1:2:4, and 1:3:6, and cured for 28 days in damp sand and about four weeks in dry air. These cylinders were treated with 6 and 10 coats of water-gas tar alone and with 10 coats of water-gas tar and a seal coat of heavy coal tar cut back with solvent naphtha to a working consistency. In later tests a lighter tar requiring no dilution has been used in place of this heavy tar.

In addition to the tar treatment samples of each mix were immersed for 24 hours in a 20 per cent paraffin-kersosene solution at about 80° F. and given four coats of this solution after the cylinders had dried out for one week.

Two batches from each mix were left untreated, one to be stored in alkali solution and the other in tar water.

These earlier samples were weighed before and after treatment and stored, four each, in porcelain-lined, covered cans containing 6,000 cubic centimeters of a 3 per cent sodium-magnesium sulphate solution.[2] For the first two years the solution was frequently renewed and the specimens removed at weekly, monthly, and trimonthly periods and allowed to dry out for 48 hours before returning to the alkali bath. Following the two-year period the samples were allowed to remain undisturbed in the alkali solution for the final twelve months, but with the cans partly uncovered. During this time a considerable loss by evaporation took place, resulting in an appreciable concentration of the solution and crystallization of sulphate salts in the surfaces of the cylinders. (See fig. 1.)

At the end of each year all samples were removed from the solution, weighed, and allowed to dry out for 10 days in the laboratory, when approximately constant weight was obtained. The difference between this weight and that immediately after removal from the bath indicated capillary or absorbed moisture, while the original dry weight deducted from that after immersion and drying represented essentially the secondary salts formed during the test period. These salts together with the lime dissolved from the test specimens, as well as insoluble residues found in the cans, served as a measure of alkali attack.

During the early stages of the investigation it was noted that the lime dissolved from the untreated specimens gradually decreased in quantity until after a period of about three months it was entirely removed from solution. This loss in lime was accompanied by an equivalent loss of magnesia and by the formation of lime and magnesia carbonates and calcic sulphoaluminate. It may be a matter of interest to state that the latter salt was found only as a precipitate in the cans and not in the concrete itself, indicating that the failure of concrete under the present test conditions can not be attributed to this salt. The destructive agencies have been found to consist chiefly of magnesium hydrate and gypsum which are formed immediately when free lime developed in the cement on setting, is brought in contact with solutions containing magnesium sulphate. This was especially noticeable in imperfectly seasoned concrete, where colloidal magnesium hydrate and gypsum crystals forming in the body of the concrete caused sufficient volume change to rupture the specimens within three months. In well-seasoned concrete, on the other hand, the formation of calcium carbonate during the curing period increased the density of the mass, thereby retarding the entrance of alkali solutions and rendering the concrete very resistant to alkali attack.

Upon completion of the three-year period of exposure the cylinders were photographed to show the surface condition of the concrete. Figure 1 shows the condition of the samples of 1:2:4 concrete at this time.

It may be stated, in general, that no evidence of serious injury by alkali was observed on any of the specimens except those of 1:3:6 mix, and that poor fabrication was seemingly responsible for the injury of these specimens.

RESULTS OF TESTS

The tests thus far described are essentially of a chemical nature and serve to indicate, through gain in weight by accumulation of secondary salts, and absorbed moisture, and loss of soluble material, the effect of alkali attack, and the degree of protection offered by the various treatments.

Table 1 gives in condensed form, the results of the tests covering a period of three years, together with the quantities of tar and paraffin absorbed on a percent-by-weight basis. It will be noted that the amount of protectives taken up increases with the leanness of the mix and is proportionate to the number of coats applied. Considering the quantitative effect of alkali attack, indicated in columns 18 and 21 of the table, it will be observed that the total percentage of gain and loss increases with the leanness of the mix and decreases with the quantity of protectives applied.[3]

An interesting point is the very small percentage of dissolved material (column 21) as compared with the

[1] Protection of Concrete against Alkali, Public Roads, vol. 5, No. 3, May 1924 and another article under the same title in Public Roads, vol. 6, No. 11, January, 1926.

[2] The solution contained 7.78 grams $MgSO_4$ and 6.61 grams Na_2SO_4 per liter.

[3] An apparent exception is indicated in batch 17, column 18, where an excessive gain in weight resulted from partial failure of two of the cylinders.

large increase in secondary salts, chiefly carbonates of lime and magnesia, which exceed 4½ per cent in untreated 1:3:6 specimens stored in alkali (batch 14, column 15). It will be noted that, during the third year, when the samples were partly immersed in the alkali solution (column 12), appreciably lower values for absorbed moisture were obtained than in previous years (columns 10 and 11), while the average quantity of secondary salts remained about constant (column 15).

Regarding the indicated protection afforded by the various treatments, as shown by the gain in weight (column 18) and loss by chemical and mechanical action (column 21), it will be observed that batches receiving a maximum quantity of both tar and paraffin are about equally protected (batches No. 5, 7, 12, 13, 18, and 19), but where the seal coat was omitted (batches No. 4, 6, 11, and 17) there was a decrease in protection of from one-third to two-thirds in the case of tar-treated samples and about two-fifths for those treated with paraffin.

As stated in a previous report the effect of alkali attack on samples immersed in paraffin solution (batch No. 6) appeared to be mainly superficial. This is further substantiated by a relatively high loss by abrasion and chemical action (0.07 per cent) and low percentage of absorbed moisture (1.12 per cent).

Comparing the chemical effect of alkali attack on the untreated samples after three years' exposure (batches Nos. 1, 8, and 14, columns 18 and 21) with that of cylinders receiving the maximum protective treatment (batches Nos. 5, 7, 12, 13, 18, and 19, columns 18 and 21) the results of the tests indicate for the latter at least a fourfold protection.

Fig. 1.—Surface Condition of 1:2:4 Concrete Cylinders With Various Treatments After Three Years in Alkali Solution and Tap Water

Table 1.—*Effect of alkali on treated and untreated concrete*

Batch No.	Mix	Treatment	Weight after treatment	Total tar absorbed	Paraffin absorbed	Weight after—			Moisture absorbed after—			Secondary salts formed after—			Total gain in weight after—			Total loss after [1]—		
						1 year	2 years	3 years	1 year	2 years	3 years	1 year	2 years	3 years	1 year	2 years	3 years	1 year	2 years	3 years
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
			Grams	*Per cent*	*Per cent*	*Grams*	*Grams*	*Grams*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	1:1½:3	Untreated, stored in alkali	10,683	------	------	11,074	11,105	11,096	0.72	0.77	0.41	2.96	3.18	3.45	3.68	3.95	3.86	0.14	0.16	0.17
2	1:1½:3	Untreated, stored in tap water	11,252	------	------	------	------	11,625	------	------	.82	------	------	2.48	------	------	3.30	------	------	------
3	1:1½:3	6 coats water-gas tar	10,641	0.49	------	10,848	10,859	10,819	.56	.58	.12	1.38	1.48	1.55	1.94	2.06	1.67	.08	.08	.10
4	1:1½:3	10 coats water-gas tar	11,129	.82	------	11,315	11,322	11,216	.48	.47	.09	1.18	1.27	.70	1.66	1.74	.79	.08	.09	.11
5	1:1½:3	10 coats water-gas tar, 1 coat coal tar	11,326	1.12	------	11,422	11,429	11,381	.18	.27	.01	.68	.64	.47	.86	.91	.48	.01	.02	.04
6	1:1½:3	24 hours in paraffin solution	11,463	------	1.84	11,592	11,463	11,591	.21	.34	.20	.92	.86	.92	1.13	1.26	1.12	.06	.07	.07
7	1:1½:3	24 hours in paraffin solution, 4 coats paraffin	11,411	------	2.32	11,493	11,524	11,491	.15	.32	.11	.57	.68	.59	.72	1.00	.70	.03	.03	.03
8	1:2:4	Untreated, stored in alkali	10,687	------	------	11,172	11,185	11,164	.92	.60	.54	3.62	4.06	3.93	4.54	4.66	4.47	.17	.19	.27
9	1:2:4	Untreated, stored in tap water	11,304	------	------	------	------	11,738	------	------	1.00	------	------	2.85	------	------	3.85	------	------	------
10	1:2:4	6 coats water-gas tar	10,872	.46	------	11,119	11,178	11,153	.50	.31	.22	2.27	2.50	2.36	2.77	2.81	2.58	.13	.17	.17
[2] 11	1:2:4	10 coats water-gas tar	8,293	1.29	------	8,445	8,428	8,405	.36	.30	.13	1.47	1.33	1.23	1.83	1.63	1.36	.04	.07	.07
12	1:2:4	10 coats water-gas tar, 1 coat coal tar	11,053	1.46	------	11,196	11,176	11,158	.36	.23	.09	.93	.90	.86	1.29	1.13	.95	.01	.04	.04
13	1:2:4	24 hours in paraffin solution, 4 coats paraffin	11,598	------	2.62	11,737	11,724	11,693	.25	.26	.16	.95	.83	.66	1.20	1.09	.82	.03	.03	.03
14	1:3:6	Untreated, stored in alkali	10,410	------	------	10,902	10,967	10,988	.99	1.15	.96	3.74	4.20	4.59	4.73	5.35	5.55	.24	.26	.33
15	1:3:6	Untreated, stored in tap water	11,099	------	------	------	------	11,623	------	------	1.61	------	------	3.11	------	------	4.72	------	------	------
16	1:3:6	6 coats water-gas tar	10,505	.64	------	10,831	10,847	10,846	.45	.63	.53	2.66	2.63	2.72	3.11	3.26	3.25	.12	.15	.18
17	1:3:6	10 coats water-gas tar	10,852	2.26	------	11,099	11,147	11,239	.42	.75	.98	1.85	1.97	2.58	2.27	2.72	3.56	.07	.11	.12
18	1:3:6	10 coats water-gas tar, 1 coat coal tar	10,887	2.94	------	11,051	11,045	11,015	.32	.43	.10	1.20	1.03	1.08	1.52	1.46	1.18	.04	.07	.08
19	1:3:6	24 hours in paraffin solution, 4 coats of paraffin	11,415	------	2.77	11,612	11,600	11,591	.29	.58	.44	1.39	1.04	1.10	1.68	1.62	1.54	.07	.03	.04

[1] Material at bottom of container.

[2] Three cylinders in batch. All other batches contained 4 cylinders.

An examination of the test specimens after breaking (see fig. 2) indicates a very unequal grading of the coarse aggregate in the tar-treated samples resulting in high porosity and frequently in honeycomb structure. This imperfect grading is especially noticeable in the leaner mixes where partial failure of the test pieces occurred during the third year of exposure to alkali attack (batches No. 16 and 17, columns 18 and 21).

No evidence of alkali attack is noticeable in any of the specimens except possibly in the 1:3:6 mix showing a disproportionate amount of coarse aggregate. It will be observed that the depth of tar penetration is dependent upon the porosity of the concrete and varies from about one-fourth inch in the denser mixes to almost complete saturation in the 1:3:6 mixes receiving 10 coats of tar.

QUALITY OF TARS

Upon completion of the foregoing investigations preliminary tests were made with various types of water-gas and coal tars in regard to their penetration and waterproofing values in order to qualify the material found best suitable for further work. As a result of these tests the following tentative specifications were drawn up.

PROVISIONAL SPECIFICATION FOR TARS FOR USE IN PROTECTION OF CEMENT CONCRETE

I. *Grades.*—The materials covered by this specification shall be supplied in one or both of the following grades, as ordered by ____________________ (the purchaser).

Grade	Material
TW-1-X	Water-gas tar for absorptive treatment.
TR-1-25	Fluxed refined tar for seal application.

II. *Materials.*—Materials supplied under this specification as—

1. Grade TW-1-X shall be crude water-gas tar, which may be treated for the removal of excess water if necessary to meet the detail requirements of this specification.

2. Grade TR-1-25 shall be prepared from refined gas-house, coke oven, and/or water-gas tars fluxed with suitable distillates.

III. *General requirements.*—The tars shall be homogeneous.

IV. *Detail requirements.*—The respective grades of tar shall meet the following requirements:

	Grade TW-1-X	Grade TR-1-25
1. Specific gravity, 25°/25° C. (77°/77° F.)	1.030 to 1.100	1.090 to 1.190.
2. Specific viscosity at 40° C. (104° F.)	Not more than 3.0.	35.0 to 60.0.
3. Total distillate by weight:		
To 170° C. (338° F.)		2.0 to 8.0 per cent.
To 235° C. (455° F.)		10.0 to 20.0 per cent.
To 270° C. (518° F.)		18.0 to 30.0 per cent.
To 300° C. (572° F.) not more than	50.0 per cent	38.0 per cent.
(a) Softening point of residue, not more than.		65° C. (149° F.).
4. Bitumen (soluble in carbon disulphide), not less than.	98.0 per cent	80.0 per cent.
5. Water, not more than	3.0 per cent	2.0 per cent.

V. *Methods of testing.*—Tests of the physical and chemical properties of the tars shall be made in accordance with the following methods:

1. Specific gravity: A. S. T. M. tentative test D70-20T; Proc. A. S. T. M., 1920, Part I, p. 764.

2. Specific viscosity (on first 50 c. c.): United States Department of Agriculture Bulletin 314, p. 7.

3. Distillation test: A. S. T. M. standard method D20-18 (A. S. T. M. Standards, 1924, p. 951).

The following modifications in procedure of the A. S. T. M. method shall be applied:

The distillation test may be made on the sample as received without dehydration, if water is present not to exceed 2 per cent, but the results shall be reported on a dry basis. (Par. 2 of A. S. T. M. method D20-18.)

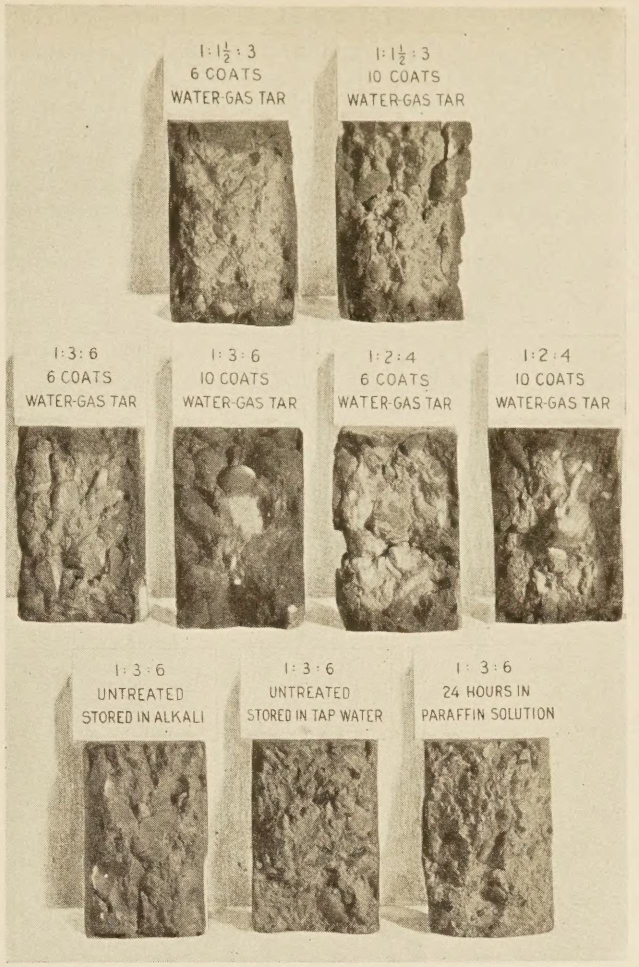

FIG. 2.—CONDITION OF THE INTERIOR OF CONCRETE SPECIMENS AFTER THREE YEARS IN ALKALI SOLUTION AND TAP WATER

The thermometer shall be placed in the flask so that the top of the bulb is level with the lowest point of junction between tubulature and neck of the flask. (Par. 4 of A. S. T. M. method D20-18.)

For use in the distillation test the thermometer shall be calibrated by distilling (*a*) water, (*c*) chemically pure naphthalene, and (*c*) chemically pure diphenylamine in the apparatus as assembled for a test. When the pure material is distilling at the specified rate, and not less than 15 cubic centimeters have been condensed, the thermometer reading and a barometric reading are taken. The actual temperatures are calculated from the equations:

For water, temperature = 100 + 0.037 (mm. barometer − 760);

For naphthalene, temperature = 218 + 0.058 (mm. barometer − 760);

For diphenylamine, temperature = 302 + 0.055 (mm. barometer − 760).

From the observed thermometer readings and corresponding actual temperatures as thus calculated, thermometer readings corresponding to specified fractionating temperatures shall be calculated and used in distillation tests of bituminous materials. (Par. 5 of A. S. T. M. method D20-18.)

The rate of distillation shall be so regulated that 50 to 70 drops pass over every minute. (Par. of A. S. T. M. method D20-18.)

3 (*a*). Softening point: A. S. T. M. standard method D36-24 (A. S. T. M. Standards, 1924, p. 955).

4. Bitumen soluble in carbon disulphide: A. S. T. M. tentative method D4-23T (Proc. A. S. T. M., 1923, Part I, p. 751).

5. Water: A. S. T. M. standard method D95-24 (A. S. T. M. Standards, 1924, p. 901).

VI. *Notes.*—1. The requirements of this specification for grade TR-1-25 are based on United States Government Master Specification No. 281, Tar for use in repair work.

ABSORPTIVE AND WATERPROOFING PROPERTIES OF TAR

In addition to the experiments thus far recorded it seemed desirable to determine the penetrating and waterproofing properties of water-gas tar when applied to cement mortars and concrete of varying consistencies both by surface application and immersion. The tests were made on 1:1½ and 1:2 mortar and 1:1½:3, 1:2:3, and 1:2:4 concrete cylinders of both wet and dry consistency, with water-gas and coal tar conforming to provisional specifications TW-1-X and TR-1-25.

The immersion treatment was carried out on cylinders, four to a batch, which, after weighing, were submerged successively in water-gas tar, the first for one-half hour, the second for 1 hour, the third for 1½ hours, and the fourth for 2 hours. After each treatment the samples were allowed to dry out thoroughly

TABLE 2.—*Absorption of water-gas tar and moisture by mortar and concrete cylinders stored seven days in water and seven days in air*

1	2	3	4	5	6	7	8	9	10	11	12	13
Group No.	Sample No.	Mix	Consistency [1]	Tar treatment	Weight before treatment	Tar absorbed		Penetration	Dried	Water absorbed [2]		
										First 10 days in water	Second 10 days in water	Third 28 days in water
					Grams	*Grams*	*Per cent*	*Inch*	*Hours*	*Per cent*	*Per cent*	*Per cent*
1	1	1:1½	Dry	Immersion, ½ hour	456	3	0.66	⅛	½	1.97	0.43	0.64
	2	1:1½	do	Immersion, 1 hour	455	5	1.10	¼	1	.87	.43	.65
	3	1:1½	do	Immersion, 1½ hours	455	6	1.32	¼	1½	.87	0	.43
	4	1:1½	do	Immersion, 2 hours	454	8	1.76	⅜	2	.87	0	0
2	5	1:1½	do	Painting, 1 coat	459	2	.44		½			
	6	1:1½	do	Painting, 2 coats	456	3	.66		¾			
	7	1:1½	do	Painting, 3 coats	459	4	.87	1/16	1	2.39	0	0
	8	1:1½	do	Paining, 4 coats	456	5	1.10	⅛	1¼	1.74	0	0
3	9	1:1½	Wet	Immersion, ½ hour	430	6	1.39	¼	½	3.25	.22	.45
	10	1:1½	do	Immersion, 1 hour	430	7	1.63	⅜	¾	1.62	.22	.45
	11	1:1½	do	Immersion, 1½ hours	439	9	2.05	⅜	1	.90	0	.22
	12	1:1½	do	Immersion, 2 hours	437	11	2.52	½	1½	.91	0	.22
4	13	1:1½	do	Painting, 1 coat	432	2	.46		¼			
	14	1:1½	do	Painting, 2 coats	433	4	.92		½			
	15	1:1½	do	Painting, 3 coats	432	5	1.16	⅛	¾	3.71	0	.45
	16	1:1½	do	Painting, 4 coats	433	6	1.39	¼	1	3.45	0	.22
5	17	1:2	Dry	Immersion, ½ hour	459	4	.87	⅛	½	1.74	.21	.43
	18	1:2	do	Immersion, 1 hour	462	5	1.08	¼	1	.86	.21	.22
	19	1:2	do	Immersion, 1½ hours	453	6	1.32	¼	1½	.88	.44	.65
	20	1:2	do	Immersion, 2 hours	452	7	1.55	⅜	2	.88	.44	.44
6	21	1:2	do	Painting, 1 coat	456	2	.44		½			
	22	1:2	do	Painting, 2 coats	464	3	.65		¾			
	23	1:2	do	Painting, 3 coats	456	4	.88	1/16	1	2.40	.21	.44
	24	1:2	do	Painting, 4 coats	464	5	1.08	⅛	1¼	.86	0	.21
7	25	1:2	Wet	Immersion, ½ hour	441	4	.90	¼	½	2.72	.22	.22
	26	1:2	do	Immersion, 1 hour	433	7	1.62	¼	¾	.84	.45	.45
	27	1:2	do	Immersion, 1½ hours	441	8	1.81	⅜	1	1.12	.22	.22
	28	1:2	do	Immersion, 2 hours	442	9	2.04	⅜	1½	1.04	.22	.22
8	29	1:2	do	Painting, 1 coat	440	2	.45		¼			
	30	1:2	do	Painting, 2 coats	440	4	.91		½			
	31	1:2	do	Painting, 3 coats	440	5	1.14	⅛	¾	3.86	.44	.88
	32	1:2	do	Painting, 4 coats	440	6	1.37	¼	1	2.26	.22	.22
9	33	1:1½:3	Dry	Immersion, ½ hour	1,589	18	1.13	¼	½	2.00	.12	.43
	34	1:1½:3	do	Immersion, 1 hour	1,558	21	1.35	⅜	1	1.78	.19	.62
	35	1:1½:3	do	Immersion, 1½ hours	1,575	23	1.46	⅜	1½	1.32	.31	.50
	36	1:1½:3	do	Immersion, 2 hours	1,580	24	1.52	⅜	2	1.06	.31	.44
10	37	1:1½:3	do	Painting, 1 coat	1,625	5	.31		½			
	38	1:1½:3	do	Painting, 2 coats	1,656	8	.48		¾			
	39	1:1½:3	do	Painting, 3 coats	1,625	10	.62	⅛	1	1.90	.06	.54
	40	1:1½:3	do	Painting, 4 coats	1,656	12	.72	¼	1¼	.96	.06	.30
11	41	1:1½:3	Wet	Immersion, ½ hour	1,566	21	1.34	¼	½	2.54	.18	.44
	42	1:1½:3	do	Immersion, 1 hour	1,648	23	1.40	⅜	¾	1.62	.30	.47
	43	1:1½:3	do	Immersion, 1½ hours	1,585	24	1.51	⅜	1	1.20	.12	.44
	44	1:1½:3	do	Immersion, 2 hours	1,585	25	1.58	⅜	1½	1.06	.12	.24
12	45	1:1½:3	do	Painting, 1 coat	1,579	5	.31		¼			
	46	1:1½:3	do	Painting, 2 coats	1,635	7	.43		½			
	47	1:1½:3	do	Painting, 3 coats	1,579	9	.57	⅛	¾	2.53	.25	.62
	48	1:1½:3	do	Painting, 4 coats	1,635	12	.73	¼	1	2.07	.06	.12
13	49	1:2:3	Dry	Immersion, ½ hour	1,632	17	1.04	⅛	½	2.01	.12	.46
	50	1:2:3	do	Immersion, 1 hour	1,632	21	1.29	⅜	1	1.52	.24	42
	51	1:2:1	do	Immersion, 1½ hours	1,644	22	1.34	¼	1½	1.10	.18	.42
	52	1:2:3	do	Immersion, 2 hours	1,640	23	1.41	⅝	2	1.20	.18	.42
14	53	1:2:3	do	Painting, 1 coat	1,575	8	.51		½			
	54	1:2:3	do	Painting, 2 coats	1,581	12	.76		¾			
	55	1:2:3	do	Painting, 3 coats	1,575	13	.82	⅛	1	2.02	.18	.50
	56	1:2:3	do	Painting, 4 coats	1,581	14	.90	⅛	1¼	1.69	.25	.43
15	57	1:2:3	Wet	Immersion, ½ hour	1,559	18	1.16	⅛	½	2.42	.12	.36
	58	1:2:3	do	Immersion, 1 hour	1,575	23	1.46	¼	¾	1.44	.18	.43
	59	1:2:3	do	Immersion, 1½ hours	1,578	26	1.65	⅝	1	1.32	.06	.43
	60	1:2:3	do	Immersion, 2 hours	1,625	28	1.72	½	1½	1.28	.18	.36
16	61	1:2:3	do	Painting, 1 coat	1,578	8	.51		¼			
	62	1:2:3	do	Painting, 2 coats	1,557	10	.64		½			
	63	1:2:3	do	Painting, 3 coats	1,578	12	.76	1/16	¾	2.85	.06	.31
	64	1:2:3	do	Painting, 4 coats	1,557	14	.89	⅛	1	2.56	.06	.25
17	65	1:2:4	Dry	Immersion, ½ hour	1,577	22	1.39	⅜	½	2.26	.62	1.16
	66	1:2:4	do	Immersion, 1 hour	1,576	26	1.65	½	1	1.06	.50	1.12
	67	1:2:4	do	Immersion, 1½ hours	1,652	27	1.63	½	1½	1.01	.60	1.07
	68	1:2:4	do	Immersion, 2 hours	1,567	28	1.78	½	2	.76	.20	.40
18	69	1:3:4	do	Painting, 1 coat	1,590	8	.50		½			
	70	1:2:4	do	Painting, 2 coats	1,578	11	.70		¾			
	71	1:2:4	do	Painting, 3 coats	1,590	12	.75	⅛	1	2.57	.74	1.23
	72	1:2:4	do	Painting, 4 coats	1,578	14	.88	¼	1¼	1.82	.50	.74
19	73	1:2:4	Wet	Immersion, ½ hour	1,574	24	1.52	¼	½	2.48	.77	1.41
	74	1:2:4	do	Immersion, 1 hour	1,578	35	2.22	½	¾	1.74	.74	1.35
	75	1:2:4	do	Immersion, 1½ hours	1,537	37	2.41	½	1	1.28	.57	1.20
	76	1:2:4	do	Immersion, 2 hours	1,554	39	2.51	½	1½	1.52	.13	1.00
20	77	1:2:4	do	Painting, 1 coat	1,564	8	.51		¼			
	78	1:2:4	do	Painting, 2 coats	1,619	10	.62		½			
	79	1:2:4	do	Painting, 3 coats	1,564	12	.77	1/16	¾	3.18	1.06	2.06
	80	1:2:4	do	Painting, 4 coats	1,619	14	.87	¼	1	2.26	.60	1.02

[1] A mix of dry consistency gave approximately 1-inch slump, a mix of wet consistency gave approximately a 6-inch slump.

[2] See text.

and the depth of tar penetration and time required for the drying of each specimen recorded. (Table 2, columns 9 and 10.) Gain in weight after each treatment indicated the quantity of tar absorbed. (Table 2, columns 7 and 8.) The same procedure was followed in the surface treatment, except that the samples were painted with 1, 2, 3, and 4 coats of tar.

The results of the tests, given in Table 2, indicate clearly that dry-mixed mortar and concrete (groups Nos. 1, 5, 9, 13, and 17) are less tar absorbent than similar mixes of wet consistency (groups Nos. 3, 7, 11, 15, and 19), and that the rate of penetration as indicated by time of drying, is slower in the dry mixes than in the wet. This may be attributed to less complete hydration in the former with correspondingly lower colloidal content than where a normal quantity of mixing water was used. It will be observed also that the tar absorbed by concrete (groups Nos. 9, 11, 13, 15, 17, and 19) increases in general with the leanness of the mix, as previously noted, evidently due to increased porosity. Comparing the quantity of tar absorbed, after one-half hour immersion by concrete of all mixes (samples Nos. 33, 41, 49, 57, 65, and 73) with samples of similar mixes receiving four coats of water-gas tar (sample Nos. 40, 48, 56, 64, 72, and 80), the tests indicate that appreciably more tar has been taken up by the immersed specimens. This may be explained in part by a relatively rapid evaporation of the lighter tar constituents when painted in hot weather over a large exposed surface area.

Considering the mortar specimens it appears from the test results that the dry mixes (groups Nos. 1, 2, 5, and 6) are about equally absorbent, but when of wet consistency (groups Nos. 3, 4, 7, and 8) the richer mixes (group Nos. 3 and 4) are the more absorbent.

After receiving the water-gas tar treatment all samples were immersed in water for 10 days and their gain in weight determined (column 11). They were then allowed to dry out for 10 days and given one coat of coal tar (TR-1-25) and again immersed in water and reweighed after 10 and 28 day periods. The moisture absorbed is indicated in columns 12 and 13. These results point to the fact that four coats of water-gas tar or immersion for any period up to two hours without an additional seal coat of coal tar are insufficient to greatly retard the ingress of water into concrete under the above test conditions. With the seal coat of coal tar, however, the quantity of moisture absorbed after 28 days averages less than one-half per cent in the richer mixes (group Nos. 1 to 16) and somewhat over 1 per cent in the leaner mixes (group Nos. 17-20).

The results indicate in general that concrete as well as mortar of dry consistency, although less tar absorbent, is fully as moisture resistant as similar mixes of wet consistency. It will be noticed also that the penetration (column 9) is about the same for samples immersed for one-half hour and those receiving four coats of tar while the water absorption (column 13) is somewhat less in the painted samples. It appears from the foregoing that concrete exposed to the action of alkali solutions should not be leaner than a 1:2:4 mix, and should be protected by at least four coats of water-gas tar (equivalent to one-half hour immersion) and one seal coat of heavy coal tar.

FIELD TESTS

Supplementing the laboratory tests it was decided to obtain information regarding the quantity of tar absorbed per square yard of surface (covering capacity) when applied under conditions approximating those met with on structures situated above water level. For this purpose concrete slabs (36 by 36 by 4 inches) were made with Portland cement, and Potomac River sand and gravel in the proportions of 1:1½:3, 1:2:4, and 1:3:6, and cured in the forms for two and seven days under damp burlap. Some of the slabs were painted immediately after stripping the forms while others were allowed to dry out for 7, 21, and 83 days before painting. All slabs were given four coats of water-gas tar at intervals of about one hour, and one coat of coal tar, in some cases immediately following the final coat of water-gas tar and in others 60 days thereafter. The quantity, by weight, of tar absorbed was determined after each application, and the covering capacity calculated from the weight per gallon of the tars employed (water-gas tar 4,087 grams and coal tar 4,738 grams).

TABLE 3.—*Water-gas tar and coal tar absorbed by concrete slabs 36 by 36 by 4 inches*

1	2	3	4	5	6	7	8	9	10	11
Slab No.	Mix	Curing	Water-gas tar absorbed by coats					Coal tar absorbed (1 coat)	Rate of application (gallons per square yards)	
			1	2	3	4	Total		Water-gas tar (4 coats)	Coal tar (1 coat)
			Gms.	*Gms.*	*Gms.*	*Gms.*	*Gms.*	*Gms.*		
1	1:1½:3	48 hours in form	87	75	60	59		125	0.069	0.026
2	1:2:4	do	97	64	60	63	909	130	.069	.028
3	1:3:6	do	105	91	78	70		165	.084	.035
4	1:1½:3	48 hours in form, 7 days in air	143	105	92	70		136	.101	.029
5	1:2:4	do	160	107	86	65	1,302	139	.102	.029
6	1:3:6	do	165	108	104	97		153	.116	.032
7	1:1½:3	7 days in form	73	58	52	50		118	.057	.025
8	1:2:4	do	100	107	60	50	924	120	.079	.025
9	1:3:6	do	111	109	83	71		137	.092	.029
10	1:1½:3	7 days in form, 7 days in air	75	86	63	53		[1] 270	.068	.057
11	1:2:4	do	147	109	82	64	1,163	[1] 290	.098	.061
12	1:3:6	do	168	125	100	91		[1] 288	.119	.061
13	1:1½:3	7 days in form, 21 days in air	80	72	57	38		[1] 233	.061	.049
14	1:2:4	do	100	80	73	72	905	[1] 225	.079	.048
15	1:3:6	do	91	88	78	76		[1] 245	.083	.053
16	1:1½:3	7 days in form, 83 days in air	100	65	52	30		189	.061	.040
17	1:2:4	do	115	75	62	55	925	195	.075	.041
18	1:3:6	do	150	86	70	65		230	.091	.049

[1] Coal tar applied 60 days after water-gas tar.

The results given in Table 3 indicate that the quantity of tar absorbed increases with the leanness of the mix regardless of the method of curing and is greatest in the slabs cured for 7 days in air after remaining in the forms either 48 hours or 7 days. (Slabs Nos. 4, 5, 6, and 10, 11 and 12, columns 8 and 10.)

The tests indicate also that approximately equal amounts of tar are absorbed by samples painted immediately after removal of the forms (slabs Nos. 1, 2, and 3 and 7, 8, and 9, column 8), and after curing for 21 and 83 days in air (slabs Nos. 13 to 18, column 8). Estimated on a per-cent-by-weight basis it will be seen that these samples (tar absorbed taken as 100 per cent) have taken up on an average 34.6 per cent less water-gas tar and 20.8 per cent less coal tar than those cured for 7 days in air. (Slabs Nos. 4, 5, 6, and 10, 11 and 12, column 8.)

The tests indicate a very appreciable increase in coal tar absorption where the slabs are allowed to dry out for 60 days after the water-gas tar treatment. (Slabs Nos. 10 to 15, column 9.) Comparing the figures given in columns 10 and 11 it will be seen that the maximum absorption for coal tar was at the rate of about one-sixteenth gallon per square yard for one coat on 1:2:4 concrete (slab No. 11, column 11), while that for water-gas tar was about one-eighth gallon for four coats on 1:3:6 concrete cured for seven days in the forms and seven days in air (slab No. 12, column 10).

Figure 3 shows slabs painted with four coats of water-gas tar and one coat of coal tar after one year in the open. The marked penetration of the seal coats in the leaner mixes is shown by the faded tones. The leaner mixes received the heaviest seal coat applications but the surface condition indicates that a still heavier application might have been used.

IMMERSION TREATMENT

1. *For precast concrete units.*—The forms shall be removed and concrete cured as may be directed by the engineer, or as may be required by the specifications governing the work. After the required curing period the concrete shall be allowed to dry out for at least 10 days before the application of the protective coatings. The precast units shall then be completely immersed in unheated water-gas tar, grade TW–1–X, and allowed to remain in the bath for a period of at least thirty minutes. They shall then be removed from the bath and allowed to drain and dry thoroughly, after which a seal coat of coal tar, grade TW–1–25, shall be applied at a temperature of about 80° F. and thoroughly brushed into all surfaces. The seal coat shall be allowed to dry for at least four days or as long as necessary to harden properly before handling.

STRUCTURES TREATED WITH WATER-GAS AND COAL TARS

Pier of bridge over Arkansas River, Cowley County, Kans.—While the treatment described in the foregoing pages is still in an experimental stage, numerous structures exposed to the action of alkali of sea water have been successfully treated both above and below water line. As a typical example of under-water

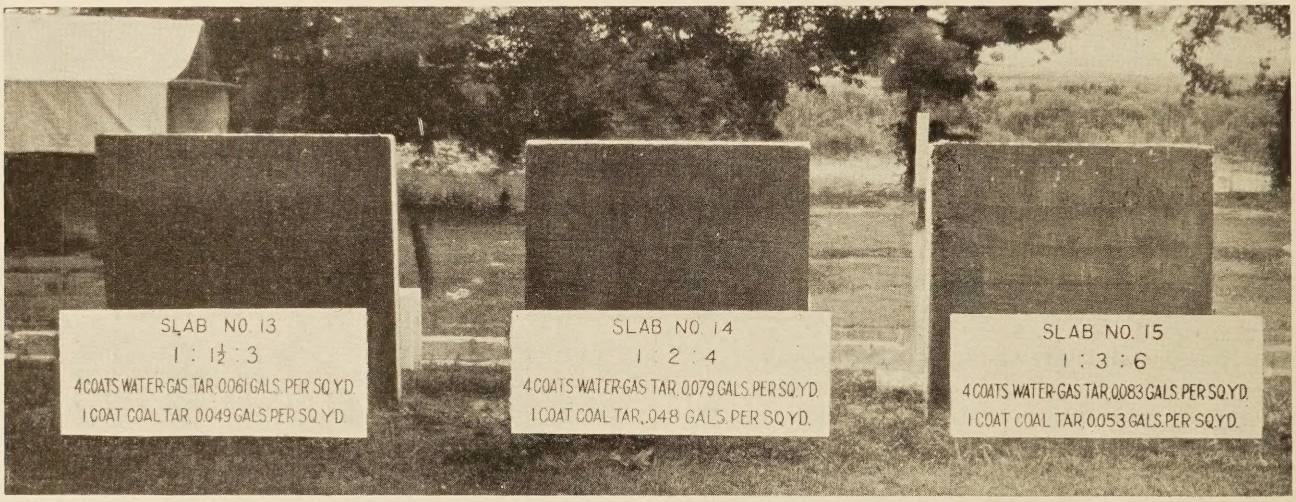

FIG. 3.—SLABS PAINTED WITH FOUR COATS OF WATER-GAS TAR AND ONE COAT OF COAL TAR AFTER ONE YEAR IN THE OPEN. THE MARKED PENETRATION OF THE SEAL COATS IN THE LEANER MIXES IS SHOWN BY THE FADED TONES

APPLICATION OF TAR

Owing to the varying conditions under which the tar may be applied to structures both above and below water level the following directions for application are recommended:

BRUSH TREATMENT

1. *For cast-in-place concrete above water level and for precast concrete units.*—The forms shall be removed and concrete cured as may be directed by the engineer or as may be required by the specifications governing the work. After removal of the forms and curing, the concrete shall be allowed to dry for at least 10 days before the application of the protective coatings. The concrete shall then be thoroughly coated with at least four coats of water-gas tar, grade TW–1–X, applied cold with a brush or spray and each coat shall be absorbed before the succeeding coat is applied. After the absorption of the final coat, a seal coat of coal tar, grade TR–1–25, shall be applied, preferably at a temperature of about 80° F. and thoroughly brushed into all surfaces. The seal coat shall be allowed to dry for at least four days, or as long as necessary to harden properly before handling.

2. *For cast-in-place concrete below water level.*—The concrete shall be cured in the forms for at least 48 hours and allowed to dry for at least five days after stripping the forms. It shall then be thoroughly coated with at least four coats of water-gas tar, grade TW–1–X, applied cold with a brush or spray, allowing each coat to be absorbed before the succeeding one is applied. After the absorption of the final coat a seal coat of coal tar, grade TR–1–25, shall be applied at a temperature of about 80° F. and thoroughly brushed into all surfaces. The seal coat shall be allowed to dry for at least 24 hours before water is permitted to come in contact with it.

treatment may be cited the case of a bridge pier in Cowley County, Kans., reported by T. J. Lough, materials engineer, Bureau of Public Roads, District No. 5. The schedule of treatment as compiled from Mr. Lough's report is given in Table 4.

TABLE 4.—*Schedule of treatment of bridge over Arkansas River, Kans.*

Date	Treatment	Time required for application	Number of men required for application	Man-hours per 100 square feet		Time allowed for drying	Tar applied on 49.5 square yards	Rate of application per square yard
1926		*Hours*		*H.*	*m.*	*Hours*	*Gallons*	*Gallon*
June 28	Cofferdam unwatered.					51		
June 30	First coat water-gas tar.	2½	1	0	34	3½	4.8	0.097
Do.	Second coat water-gas tar.	1¼	2	0	34	13¾	3.0	.060
July 1	Third coat water-gas tar.	1¼	2	0	34	8	2.2	.044
July 2	Fourth coat water-gas tar.	1¼	1	0	17	13⅚	1.6	.032
July 3	One coat coal tar	1⅙	2	0	31	24	2.24	.045
Total	Five coats	7 5/12		2	30	114 1/12	13.84	(1)

[1] Water-gas tar 0.233 gallon, coal tar 0.045 gallon.

The purpose of the treatment was to protect the concrete from the action of alkali water of the Arkansas River. (See fig. 4.) The treatment consisted in applying successive coats of water-gas tar (grade TW–1–X) and one coat of coal tar (grade TW–1–25) to pier No. 1 of the bridge, 2.75 feet above and below water level, the treated band having an area of 49.5 square yards.

It was carried out under very favorable weather conditions, temperature ranging from 68° to 96° F. and a bright sun and light breeze persisting throughout the days of treatment.

The concrete was fabricated of one part cement, two parts of fine Arkansas River sand, and approximately three and one-half parts of clean, washed flint gravel, and was two months old when treated.

DESCRIPTION OF WORK

Pumping units.—Two pumping units were employed to unwater the cofferdam. This was to preclude the possibility of water reaching the concrete under treatment should one pump break down. No difficulty was experienced in holding the cofferdam dry.

Condition of concrete.—At the time of applying the initial coat of water-gas tar the concrete appeared to be dry and in a satisfactory condition to receive the tar. The concrete was wire-brushed immediately prior to the application to remove a film of water-deposited impurities.

Rate of absorption.—Where exposed to the sun's rays and the breeze the initial coat penetrated rapidly. On shaded areas the absorption proceeded more slowly. The penetration was not uniform, indicating variations in the density of the concrete. On the web of the pier, the concrete of which had received the most spading, absorption proceeded at the slowest rate.

A decreasing rate of absorption of the successive coats was noted, requiring increasing time intervals between the successive applications. The priming coat had been fully absorbed when the second coat was applied. Small areas had not been fully penetrated when the third coat was applied and slightly larger areas had not been completely penetrated at the time of placing the fourth coat. Considerable areas of the fourth coat were sticky and not entirely absorbed when the coal-tar seal coat was applied, indicating that the saturation point was approached. Postponement of the seal coat application to permit possible additional penetration of the water-gas tar was considered unwarranted.

Application.—No difficulty was experienced in applying the tars with ordinary calcimine brushes. The seal coat was spread out as thin as practicable and was applied cold, heating not being necessary. The proximity of the cofferdam to the concrete with resultant restricted working space somewhat handicapped the workmen in appyling the tar, but a workman of average ability should be able to apply a coat of either kind of tar at the rate of 25 square yards per hour.

Penetration.—Immediately prior to the application of the seal coat a number of holes were drilled through the water-gas tar coats and the depth of penetration was found to average one-eighth inch.

Cost.—The cost of the entire operation was as follows: (1) Moving pumps, $15.83; (2) pumping, $177.94; (3) cleaning surface, $5.35; (4) cost of tars, $5.60; (5) applying tars, $10.81; total, $215.53, or $0.484 per square foot of surface.

Summary and conclusions.—The following conclusions may be considered as applying to treatment under conditions similar to those described:

1. Duplicate pumping units are essential.
2. Given favorable weather a 48-hour initial drying period for the concrete is sufficient.
3. Rate of absorption is affected both by density of concrete and exposure to sun and wind.
4. As each coat of water-gas tar is less rapidly absorbed than the preceding one, the time intervals between applications should be correspondingly increased, so far as the working day and a reasonable total period of treatment will permit.
5. A workman of average ability should be able to apply a coat of either kind of tar at the rate of 25 square yards per hour.
6. Approximately 2.5 gallons of water-gas tar per 100 square feet of surface are required for four applications; and one application of coal tar in the amount of 0.50 gallon per 100 square feet.
7. The average penetration of the water-gas tar at the time of applying the seal coat will be approximately one-eighth inch.
8. Absorption of more than four coats of water-gas tar is not obtainable without extending the treating period.
9. The concrete must be held in the dry for five days to permit satisfactory treatment.
10. The efficacy of the treatment can only be determined after a considerable lapse of time, and then only by comparing the condition of untreated concrete masonry exposed to the same stream.

Fig. 4.—Application of Tar to Pier of Arkansas River Bridge, Cowley County, Kans.

Comparing the tar treatment described in the foregoing report with that of the slabs, Table 4 shows that the four coats of water-gas tar were applied to the bridge pier at the rate of about one-fourth gallon per square yard of surface, whereas the slabs of a similar mix cured for one and three months in air absorbed the tar at the rate of about one-thirteenth of a gallon per square yard. (Table 3, slab Nos. 14 and 17.) The coal tar was applied at approximately the same rate on both slabs and pier (about one-twentieth of a gallon per square yard). From these results it is evident that the quantity of tar absorbed by concrete of the same age and consistency may vary considerably, depending upon local weather conditions and manner of application.

Other structures receiving the tar treatment have been reported as follows:

Abutments and piers of bridge over the Arkansas River, near Ford, Kansas (F. A. P. No. 236–A. Dist. No. 5).—The area treated was 2,489 square feet on two abutments and six piers, 3 feet above and 3 feet below low water line. After unwatering the cofferdam the masonry was allowed to dry out for four or five days and the area to be treated was thoroughly cleaned with a stiff wire brush. The treatment consisted of six coats of water-gas tar and one coat of coal tar conforming to the provisional specifications of the Bureau of Public Roads. A coat of water-gas tar was applied by going completely around the pier or abutment and ending at the point of beginning. At the completion of the first coat the tar that had first been applied was dry enough to receive the second coat. About two hours elapsed between the application of the second and third coats. The third coat was allowed to dry for four or five hours before placing the fourth coat. After drying out over night the fifth coat was applied and in the afternoon of the second day the sixth coat was put on.

The seal coat of coal tar was usually placed later in the afternoon of the second day, a swab being used instead of the brush used to apply the water-gas tar. It was found that two men could finish a pier or abutment in two days.

Fig. 5.—Treated Concrete Piles, Santa Maria River Bridge Near Santa Maria, Calif.

The rate of application was about one-third gallon of water-gas tar per square yard of surface and slightly over one-tenth gallon of coal tar. The approximate cost of treatment including pumping, labor, fuel, oil and tars was $540, or about 22 cents per square foot.

Coast Highway Bridge over North Channel, Santa Maria River, about 2 miles north of Santa Maria, San Luis Obispo Co., Calif. (F. A. P. 25–C).—The treatment of piles supporting the bridge girders consisted of two coats of water-gas tar applied before driving and two additional coats after driving covering the exposed portion and extending two feet below the normal ground line. The bed of the stream was dry and there was no difficulty in excavating around the piles to a depth of two feet. The application was made with calcimine brushes. A stream of tar from the spout of a watering pot was run along the top surface of the pile as it lay on one of its sides and this was brushed into the top and sides, giving an especially thick coating on the top. The pile was later turned through 180 degrees and the fourth side was treated, as well as the two vertical sides where additional tar could be absorbed. The piles had dried out after curing for more than a month so the tar absorption was rapid. The tar was used at a rate of 1⅓ gallons per 100 square feet. The depth of penetration was about one-eighth inch, or as much as might be expected from three or four applications within a few days.

Reinforced concrete viaduct between Wilmington and Long Beach, Calif., over part of the west basin of Los Angeles harbor.—The treatment was here given to concrete footings and extended from the ground surface to two feet below permanent low water level. It consisted in applying about six coats of water-gas tar followed by a seal coat of coal-tar pitch, at a cost, including labor, of 43 cents per 100 square feet for the water-gas tar and 96 cents for the coal tar. The concrete was a 1:3:5 mix, and although the penetration of the tar did not exceed one-eighth inch it was felt that an excellent waterproof condition had been obtained.

St. Marys River Bridge, Camden-Nassau Cos., Fla.-Ga. (F. A. P. 421).—The parts treated were the backs of abutments from top of footing to top of embankment, the faces of abutments from top of footing to an elevation 3 feet above high tide, the reinforced piers from top of footing to 3 feet above high tide, the undersides of slabs and girders and the sides of girders of the concrete approaches unexposed to view.

The treatment consisted of ten coats of water-gas tar (grade TW–1–X) uniformly applied with a brush in ten separate applications at intervals of 2 hours each, the first application being made immediately upon the removal of the forms, or as soon thereafter as the concrete became sufficiently dry to receive and hold it.

Morehead City-Beaufort Bridge, Carteret County, N. C. (F. A. P. 64–A Reo.).—The treatment consisted of four coats of water-gas tar and one seal coat of coal tar on bascule pier shafts and concrete piles.

The concrete of the four pier shafts consisted of one part cement, two parts of clean bank sand and four parts of washed flint gravel, graded ¼ to 1½ inches. The piles were fabricated of one part cement, one and one-half parts sand and three parts gravel, graded ¼ to ¾ inch in size.

The treatment of the shafts extended from 6 feet above to 6 feet below mean water level. The tars were applied very effectively with calcimine brushes with very long handles as the cofferdams were too close to the shafts to permit free working conditions around the pier.

The concrete was dry and clean when the tar was applied, and the rate of absorption was reasonably uniform except where the concrete had taken up resin from the boards. These places stood out prominently and the absorption was very slow.

Each successive coat showed decreasing absorption and thus required longer intervals between applications, but in no case was the time required for the penetration of a coat more than one hour and 15 minutes. Small areas were not completely dry when the fourth and fifth coats were applied.

Under conditions similar to the above one man should apply 270 square feet per hour of the first coat, 315 square feet per hour of the second, third, and fourth coats, and 210 square feet per hour of the seal coat.

The quantity of tar used on this project per 100 square feet of surface was 2.95 gallons of water-gas tar and 1.10 gallons of coal tar. The method of treatment appears satisfactory and is recommended.

(Continued on page 119)

STATIC AND IMPACT LOADS TRANSMITTED TO CULVERTS[1]

DIGEST OF REPORT ON EXPERIMENTAL DETERMINATIONS AT IOWA EXPERIMENT STATION

Experimental determinations of static and impact loads on highway culverts have resulted in data which can be directly applied in culvert design. The experiments have been carried on at the Iowa Engineering Experiment Station in cooperation with the United States Bureau of Public Roads. They have shown that a formula to determine the pressure on a culvert resulting from a concentrated load on the fill above it can be used to determine the maximum obtainable transmitted load. For fills of 5 feet and less the effect of concentrated superimposed loads is appreciable, but for fills greater than 5 feet and widths of culverts not greater than 3½ feet the effect of ordinary truck loadings may be ignored. Moving loads may result in a pressure on the culvert much greater than similar static loads. It is believed that an increase of 50 to 100 per cent over the calculated static load effect is sufficient to take care of the impact produced by moving trucks up to speeds of 10 miles per hour.

The general program of culvert investigations has included work with earth embankments and the theory of loads on culverts which has been or will be reported in experiment station bulletins.[2]

LAW GOVERNING PRESSURE ON CULVERT FROM CONCENTRATED LOAD AT TOP OF FILL INVESTIGATED

The work with static loads was an attempt to discover the law governing the transmission of loads to culvert tops from static concentrated loads, such as wheel loads when the wheels are applied on the surface of the fill at a distance, H, above the top surface of the culvert. So far as known J. H. Griffith was the first to suggest[3] that the formula, $P=\frac{3}{2\pi}\frac{H^3}{H^5_s}T$ (see below for notation) would apply to masses of earth as in this case. This formula was derived long ago by Boussinesq[4] in his theoretical work on stresses in elastic solids. In the case of an infinite elastic solid bounded by a plane he worked out the effect of a concentrated normal force applied at a point on the surface, calculating the stress distribution and displacements at different points in the solid.

The formula shown is a part of the solution of the general problem of finding the stresses due to a force T applied normal to an infinite plane bounding an elastic solid. The general formula is:

$$W_t=\frac{C_tT}{l}, \text{ in which } C_t=\frac{\Sigma Pa}{T} \text{ and } P=\frac{3}{2\pi}\frac{H^3}{H^5_s}T, \text{ where}$$

T = Total concentrated load applied at one point on the surface of the embankment.

W_t = The total corresponding load, per unit of length, transmitted to the top area of the test section.

l = Length of the test section of culvert.

C_t = A calculated coefficient of transmitted load.

Σ = Summation of pressure transmitted to each small unit area of the top of the test section of culvert.

P = The intensity of vertical pressure per unit of area due to T at the center of any small unit area, a of the top area of the test section.

a = Area of each of the small equal units into which the horizontal top area of the test section of the culvert must be divided for summation purposes.

H = Height of embankment above top of the culvert.

H_s = Slant height direct from the point at which T is applied to the center of the particular unit area a for which calculation is being made.

The work with static concentrated loads was con-confined to determining the applicability of these formulås which were found to apply with a satisfactory degree of closeness.

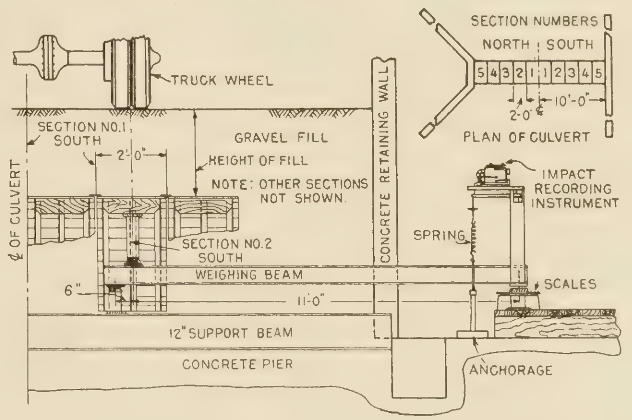

Fig. 1.—Showing Arrangement of Apparatus for Measuring Pressure on Culvert. The Impact Recording Instrument and the Spring Attached to Balance Beam Was Used Only in the Impact Tests

Figure 1 shows the general arrangement of apparatus used in the test. Culvert pipe sections 2 feet in length and 3½ feet outside diameter were placed beneath a gravel fill weighing from 125 to 140 pounds per cubic foot. This gravel was used as taken from the pit and contained a high percentage of clay. A sieve analysis showed about 30 per cent retained on a ¼-inch sieve. Pipe sections No. 2 north and No. 2 south (see fig. 1) were selected for measurement of transmitted wheel loads and each equipped with a lever and platform scale weighing device. The weight of a culvert section is transmitted to the weighing beam 6 inches from the fixed-end knife-edge support. The other end of the weighing beam rests on platform scales 11 feet from the fixed end. Differences in scale readings due to applied wheel loads were therefore multiplied by 23 to give the difference in load on the culvert section. A 3-ton truck loaded with various amounts of gravel was used for loading. The axle spacing of this truck was 13½ feet and the wheel gage 5 feet 10½ inches. The tire equipment consisted of 40 by 6 inch single-tread, solid tires on the front wheels and 40 by 6 inch dual-tread solid tires on the rear wheels.

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[1] The full report is contained in Iowa Engineering Experiment Station Bul. 79, Experimental Determinations of Static and Impact Loads Transmitted to Culverts, by M. G. Spangler, Clyde Mason and Robley Winfrey.

[2] Buls. 31, 36, 47, 57 and 76, Iowa Engr. Exper. Sta.

[3] Report of subcommittee on bearing values of soils, Proc. Am. S. of C. E. Aug., 1920, vol. 49, p. 931.

[4] "Application des Potentiels," by M. J. Boussinesq, Paris, 1885, pp. 65–66.

FIVE SERIES OF STATIC TESTS MADE

The field work with static loads is classified into series A, B, C, D, and E. Each series represents data taken under different conditions of fill, apparatus, and time intervals. Series, A, B, and C were run with the embankment material wholly in contact with the pipe, so that the arrangement was similar to a field installation of a circular culvert having the same projection condition. In series D and E this arrangement was modified, as explained later.

Static load series A.—Series A is a set of readings taken with heights of fill on the culvert of 24, 18, 12, and 6 inches with the truck body empty and consequently a relatively light load. The truck was run onto the culvert and stopped with one rear wheel directly over section 2, south, standing in this position for a few minutes only. This placed the other rear wheel within 1.5 inches of being directly over the center of section 2, north. The scale readings for both sections were noted while the truck stood over the culvert. These scales were read again after the truck was removed and the additional load on the 2-foot section calculated.

Static load series B.—The readings for series B were obtained in exactly the same manner as those of series A, except that a heavier load was used on the truck and the heights of fill ranged upward in the order of 6, 12, 18, and 24 inches.

Static load series C.—For each of the runs in series A and B the truck was allowed to stand over the culvert for only the few minutes necessary to take the scale readings. In order that the effect of time on the transmitted load might be observed, series C was conducted, in which the truck was placed over the culvert and allowed to stand over night, or about 15 hours. This series was run in connection with the impact work; that is, impact runs were made during the day and the truck allowed to stand on the culvert at night for observation of static load distribution.

Static load series D.—As indicated above, Series A, B, and C were conducted, using a circular culvert section. After these were completed it seemed desirable to study the load on a section having a flat top which would fulfill the conditions encountered in the case of a rectangular box culvert where the fill is of equal depth at all points. Accordingly, a platform of the same area as the projection of the circular culvert section on a horizontal plane was constructed and made to rest upon the circular section.

Static load series E.—Following series D the gravel fill was removed down to the top of the platform and an attempt was made to calibrate the apparatus by placing weights on the flat top and noting the readings on the scales. It was found during this work that when the gravel fill was in contact with the sides of the platform placed on top of the culvert it was not possible to check the calculated mechanical advantage of the lever system. When the gravel was removed to a depth equal to nine-tenths of the vertical diameter, however, this mechanical advantage ratio could be checked very closely. This fact indicated that the gravel in contact with the culvert section actually helped to support it when a superimposed load was applied, and that the total load transmitted to the flat platform was not recorded on the scale.

To eliminate this probable friction support, a crib was designed and built which prevented the gravel from coming in contact with the sides of the culvert. This crib consisted of 2 by 6 inch planking placed parallel to the longitudinal axis of the culvert and achored back into the fill in such manner as to be free to move vertically and still not come in contact with the culvert. The planks were spaced vertically with about 1 inch between them and these openings flashed with tin. Figue 2 shows a sketch of this arrangement.

Series E was run, using the above apparatus and was carried up to the 6-foot level beginning at the 6-inch level. The truck was allowed to stand on the section for a few minutes only. In the earlier series considerable trouble was experienced in maintaining the exact height from the truck tire to the culvert surface which was desired. That is, if readings were being taken at the 12-inch level, the tires would cut into the gravel to such an extent that this distance might be reduced as much as 25 per cent. Since the vertical distance from tire to top of culvert is an important factor affecting the distribution of the load, a reading was taken by means of an engineer's level and rod on the truck wheel for each run of series E, so that this vertical distance was measured exactly.

During the progress of this series, difficulty was encountered at about the 3-foot level, which necessitated removing gravel down to the platform and then replacing it before continuing the tests. The opening

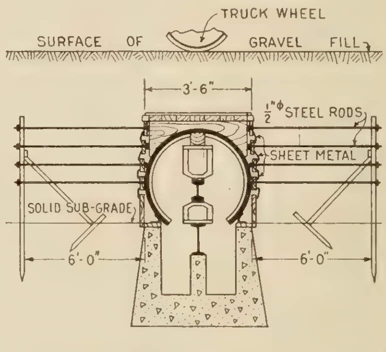

Fig. 2.—Showing Apparatus Installed to Prevent Friction Between Gravel Fill and Sides of Culvert

between the bottom of the platform and the top of the cribbing had been flashed with canvas instead of tin. As the fill increased and the crib planks gradually settled downward, the horizontal pressure of the gravel forced the canvas to bulge in and to afford some support to the platform. With this condition the total load being received by the platform was not being indicated on the scales, so the rib flashings were changed to tin before readings were continued. This condition undoubtedly existed to a much less degree at the levels of 2 feet and below, but the effect of load transmitted fell off rapidly at the 3-foot level so that an examination of the apparatus was made and the above trouble discovered.

Table 1 shows the distribution of truck weights for the static load series.

Table 1.—*Distribution of truck weights for static load series*

Static series	Height of fill	Gross weight	Weight on rear wheels	Weight on front wheels
	Feet	*Pounds*	*Pounds*	*Pounds*
A	0. 5–2. 0	11, 010	6, 590	4, 420
B	. 5–2. 0	13, 380	8, 700	4, 680
C	2. 0–6. 0	16, 160	11, 290	4, 870
D	. 5–2. 0	10, 570	6, 320	4, 250
E	3. 0–6. 0	11, 010	6, 590	4, 420

Table 2.—*Results of static load tests*

Series	Height of fill	Weight on one rear wheel	Section 2, south: Load on culvert section [1]	Section 2, south: Percentage of wheel load	Section 2, north: Load on culvert section [1]	Section 2, north: Percentage of wheel load
	Feet	*Pounds*	*Pounds*	*Per cent*	*Pounds*	*Per cent*
A	2	3, 295	695	21. 06	734	22. 24
	1½		1, 029	31. 12	922	27. 97
	1		1, 465	44. 47	1, 504	45. 62
	½		1, 976	60. 08	1, 798	54. 55
B	½	4, 350	2, 721	62. 55	2, 762	63. 53
	1		2. 045	47. 02	1, 923	44. 23
	1½		1, 309	30. 09	1, 398	32. 15
	2		975	22. 44	925	21. 24
C [2]	2	5, 645	1, 127	20. 0	1, 449	25. 7
	3		805	14. 3	713	12. 6
	4		736	13. 1	----------	----------
	5		529	9. 8	368	6. 5
	6		644	11. 4	506	9. 0
D	½	3, 295	2, 339	71. 0	----------	----------
	1		1, 579	48. 3	----------	----------
	1½		1, 044	31. 7	----------	----------
	2		561	17. 1	----------	----------
E	½	3, 160	2, 926	92. 7	----------	----------
	1		2, 491	78. 8	----------	----------
	1½		1, 748	55. 3	----------	----------
	2		1, 217	38. 5	----------	----------
	3	3, 295	590	17. 9	----------	----------
	4		432	13. 2	----------	----------
	5		209	6. 4	----------	----------
	6		200	6. 1	----------	----------

[1] The values given represent the average of 10 or more measurements in every case except series C, where only one measurement is represented.

[2] Load applied for about 15 hours in every case except for section 2, north, with a 6-foot fill, which was for 45 hours.

EXPERIMENTAL RESULTS COMPARED WITH THEORETICAL LOADS

Figure 3 shows curves plotted according to the formula, $C_t\ (\%) = a\ \Sigma \frac{3\ H^3}{2\ \pi H^5_8} \times 100$ (giving the values obtained for C_t expressed as a percentage of the applied load). The load is figured for a top area of 3.5 by 2 feet, which equals the horizontal projection of the test section. The various heights of fill are plotted as ordinates and the percentages of applied load transmitted to the culvert section as abscissæ. The drawings also show the relation between the percentage of the load applied to the culvert and height of fill as indicated by test results. In series A, B, and D, it will be seen that the actual observed percentages of the applied load distributed to the culvert section fall materially short and in series C generally somewhat short of the theoretical curve, especially below the 3-foot level. In series E, however, when an attempt was made to eliminate side support of the section from the surrounding gravel, the observed percentages come very close to the theoretical curve, sometimes reaching, but practically never exceeding it. The conditions existing in series E were such that the observed loads may be said to be identical with those obtained under field conditions, where the top slab of the culvert receives the total load imparted to the structure by a concentrated load on top of the fill.

CONCLUSIONS REGARDING STATIC LOAD RESULTS

The theoretical formula seems to give a locus showing the maximum possible percentage of load transmitted through any thickness of fill. In the experimental work, however, this maximum load generally was not reached, but when conditions were most favorable, as in series E, the experimental results came very close to the theoretical.

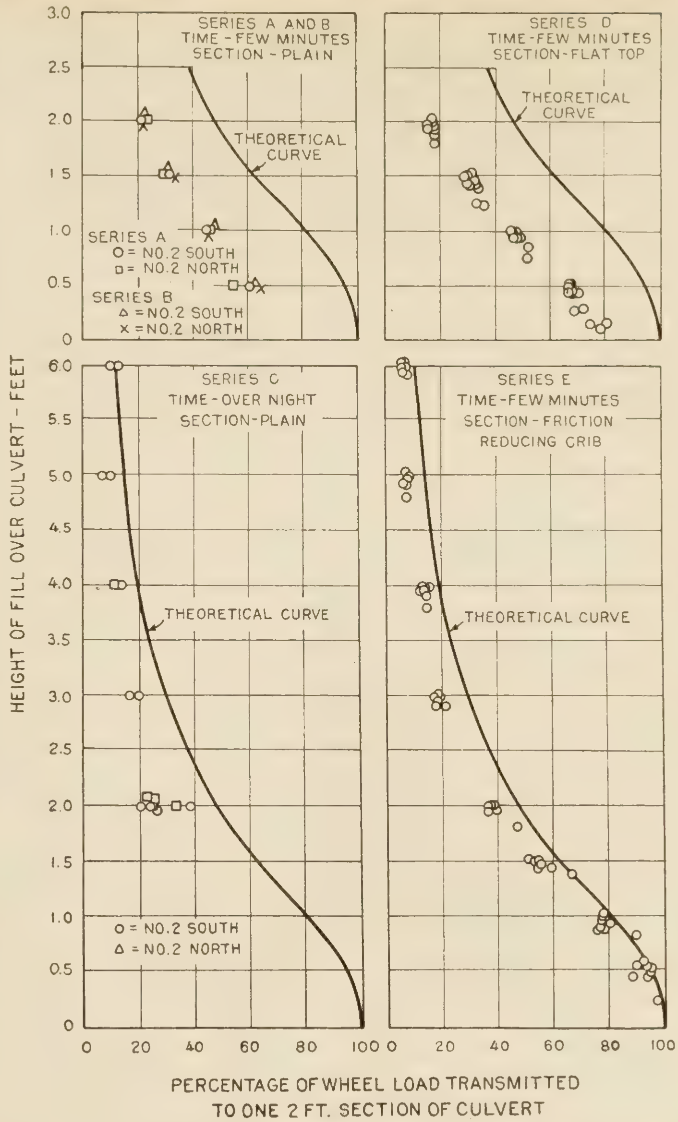

Fig. 3.—Comparison of Experimental Results from Static Load Tests With Theoretical Curve

It is rather remarkable that the theory developed by Boussinesq for elastic solids fits so closely when applied to a granular material such as gravel. The observed points on the curves in Figure 3 were taken under track conditions varying from the loose freshly filled embankment to a wheel track worn fairly hard and smooth by the truck. Under certain conditions, as in series E, the load on the culvert section approaches close to the theoretical curve.

The formula $W_t = \frac{C_t T}{l}$ should be used in designing culverts or conduits through embankments when it is necessary to design for superimposed concentrated loads of a static nature. For convenience in using the formula the values of C_t have been computed for the values of H of 0.5, 1.0, 1.5, 2.0, 3.0, 4.0, 5.0, 7.0, 10.0, 15.0, and 20.0 feet for any size structure up to 10 by 20 feet and are to be published later.

Table 3 gives such values for 2-foot section of a culvert 3½ feet wide.

TABLE 3. —*Values of the coefficient C_t; for a section 3.5 feet wide and 2 feet long*

Height of fill in feet	Values of C_t		
	Rear wheel directly over center of area	Other rear wheel (5.875 feet away)	Total [1]
0.5	0.95262	0.00005	0.95267
1	.79758	.00042	.79800
1.5	.61295	.00131	.61426
2	.46271	.00549	.46820
3	.27283	.00679	.27962
4	.17351	.01107	.18458
5	.11822	.01435	.13257
6	.08510	.01628	.10138
7	.06393	.01700	.08093
8	.04968	.01689	.06657
9	.03965	.01623	.05589
10	.03236	.01530	.04766
11			
12			
15	.01464	.01015	.02479
20	.00829	.00670	.01499

[1] The effect of the front wheels is negligible in this case.

For culverts up to 3½ feet in width and under 5 feet or more of embankment the effect of a concentrated load at the surface is negligible compared with the weight of the fill. For greater widths of culvert the depth of embankment at which concentrated loads may be neglected is greater than 5 feet, but will be moderate.

APPARATUS FOR IMPACT MEASUREMENTS

The same general arrangement of apparatus and material as used for the static tests was used for the impact measurements. These measurements were made on section 2, south as illustrated in Figure 1. This section was chosen because when the truck passed through the center of the culvert, one wheel passed over the center of the section.

Additional apparatus was required for measuring the rapid movement of the balance beam under impact. A specially designed instrument for this purpose was mounted on the wooden guard above the balance beam. This instrument consisted of a pointer equipped with a pen which traced its movement on a continuous sheet of recording paper. The pointer was actuated by a system of levers attached to a vertical rod which rested on top of the balance beam of the scale near its end. The continuous recording paper was moved by a 6-volt motor. In addition to the pointer which recorded movement of the balance beam, two other pointers with pens attached, were used, one to record half seconds of time at the bottom of the recording paper and the other to indicate, at the top of the paper, revolutions of a drum of known diameter which unreeled as the truck moved forward. From these last two records the speed of the truck as it passed over the culvert could be calculated.

The movement of the balance beam was retarded by means of a spiral spring attached to the bottom of the scale pan and anchored to the floor of the scale house. Springs of various sizes and strengths were used according to the amount of impact so that the movement of the pointer arm could be held down within the limits of the recording paper. Weights on the scale pan and on the scale platform were also used to retard the movement of the pointer. These various combinations were calibrated and treated as separate apparatus.

The operation of the above described apparatus was as follows: As the truck moved toward the culvert it unwound a small brass wire from a drum placed at the starting point whose diameter was exactly 4 inches. Each revolution of the drum made an electrical contact which actuated the pointer at the top of the recording sheet and thus a record was obtained of the distance the truck moved for any half second. As the truck moved across the culvert, the effect of the rear wheel was transmitted through the gravel fill to the culvert section. This transmitted effect was indicated by movement of the beam of the scale on which the weighing lever for this particular section rested. The movement of the scale beam was recorded by the pointer of the impact instrument and the highest peak of the ink trace was called the impact reading. The truck was then backed onto the culvert and the position of the pointer noted while the truck was standing still. This was called the static reading. The difference between these two readings formed the basis of calculating the increase in effect on the culvert due to a moving load over the effect of an equal static load.

IMPACT MEASURING DEVICE CALIBRATED

In setting up an apparatus to determine calibration curves for the impact data, resort was made to the well-known physical law that a weight suddenly applied to a surface, but without a blow, produces a maximum pressure on the receiving surface equal to twice the applied weight, but continuing only an infinitely short time. An instantly applied load, then, momentarily produces twice the ultimate continuous effect of a slowly applied load. With this law in mind, an apparatus was planned by which loads of various magnitudes could be suddenly applied to the culvert section over which the original impact runs were made.

To make the calibration tests, the gravel fill was removed from the culvert section and a wooden framework built over and supported by the culvert section. This framework was so arranged that it could be loaded to give a pressure on the culvert equal to that of any desired height of fill. At the center of this framework and directly over the culvert a cage containing any desired weight was suspended so as to just touch the platform or framework. By means of a trip and rope this load could be suddenly applied without any appreciable fall. A record was made of the resulting deflection of the system of levers at the instant of application and also as a static load. Data were thus secured for the preparation of calibration curves representing various heights of fill and other test conditions. Figure 4 shows a typical calibration curve.

These curves were used to determine the suddenly applied load which would produce the same deflection of the recording apparatus as caused by any particular truck run. Knowing the deflection caused by truck impact the proper calibration curve gives the suddenly applied load which causes the same effect. This suddenly applied load is multiplied by two to give the static load which would cause the same effect continuously. This value is then compared with the calculated theoretical static load effect and the percentage of momentary impact determined.

Take for example, a typical truck run over a 3-foot fill. The initial reading on the recording paper was 73, the static reading was 97, and the impact reading as the wheel hit the bottom of the hole in the track was 118. This gives an impact difference of 45 and a static difference of 24. From the proper calibration curve (fig. 4) 850 pounds is found as the load which if applied suddenly produces the same throw of the scale beam. Doubling this to get an equivalent static load gives 1,700 pounds. The actual static load is 960 pounds as shown by the lower or static load calibration curve.

Seventeen hundred pounds is 108 per cent of 1,581 pounds, the theoretical transmitted static load under the same conditions. The moving truck, therefore, produced a momentary effect on the culvert equal to 108 per cent of the theoretical effect of the truck when standing over the culvert.

McCOLLUM-PETERS STRAIN TELEMETER USED

One group of the impact tests was conducted using the McCollum-Peters strain telemeter. This instrument is designed to measure stresses in structural members by means of a recording device which registers the strain of the member under stress. There are 12 gauges in connection with the instrument so that it is capable of registering strains at 12 points in a member simultaneously. Each gauge consists of two hardened points exactly 8 inches apart. These points are clamped to the structural member at the place where observations are desired. As the member is lengthened

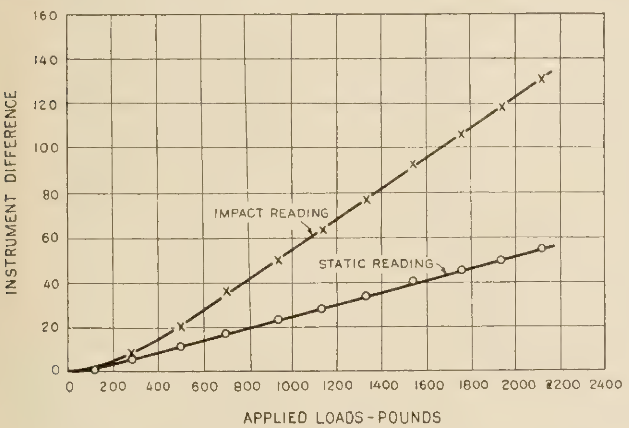

Fig. 4.—Sample Calibration Curve Used in Impact Test

or shortened under stress, depending on whether the member is in tension or compression, the points cause a change in the electrical carrying capacity of a carbon resistance unit in the gauge and the amount of current passing through the carbon resistance unit is thereby varied. This variation in current causes a small mirror at the main recording unit of the instrument to revolve slightly and throw a beam of light so that it describes an arc which is proportional to the strain in the structural member. The movement of this beam of light is recorded on a roll of sensitized paper which moves at a constant speed and a photographic record is thereby made of the change in strain of the member at each point where a gauge is attached. Since strain is proportional to the load applied to a beam, other conditions being the same, a record showing changes in strain will also be a record of change in load applied to the beam.

In applying the above instrument to the culvert section on which impact observations were made, the platform scale on which the end of the weighing lever rested was removed and an unyielding support substituted therefor. This produced a beam simply supported at each end with a concentrated load applied eccentrically. Ten of the gauges were clamped to the lower flange of this beam, arranged in pairs. Then a photographic record of the change in stress in the beam was made by taking an exposure first, with no load on the culvert (except the gravel fill), second, at the time of impact, and third, with the truck standing over the culvert. Although 10 gauges were used in the impact runs, only the two which were nearest the knife edge supporting the culvert were properly calibrated. The calibration was by the same general method as used with the other impact measuring apparatus supplemented by laboratory calibrations by independent methods. The results given are based on the strains recorded by only two gauges. Due to the short time which this instrument was available, observations were made only on the 2 and 3-foot levels.

IMPACT TESTS MADE UNDER DIFFERENT CONDITIONS

The impact tests runs were made with a 3-ton truck loaded with various amounts of gravel and with tire equipment as previously described. Table 4 shows the heights of fill and wheel loads for the four groups of impact tests.

Table 4.—*Truck loads for impact groups*

Group No.	Heights of fill	Gross load	Weight on rear wheels	Weight on front wheels
	Feet	*Pounds*	*Pounds*	*Pounds*
1	2, 3, 4	16, 180	11, 400	4, 780
	5	19, 030	13, 660	5, 370
	6, 8, 10	31, 940	25, 700	6, 240
2	2, 3, 4, 5	16, 160	11, 290	4, 870
3	2, 3, 4, 5	17, 750	12, 600	5, 150
4	2, 3	15, 960	11, 020	4, 940

It was found that with a smooth track over the culvert, the truck produced only a quite small percentage of impact. Since in actual service, the roadway over a culvert is quite often rough and full of ruts, it was deemed desirable to place obstructions in the track so that this naturally rough condition might be simulated. The truck was allowed to pass over these obstructions in such a manner that after leaving the obstruction the wheel struck the track directly over the center of the culvert section. In the case where a hole was dug in the track to produce impact the hole was centered directly over the section. The obstructions used in these impact runs were a 2 by 4 inch block of wood, a 4 by 6 inch block, a 2-inch hole dug in the wheel track, a 4-inch hole and a 6-inch hole.

FOUR GROUPS OF TESTS MADE

The prosecution of the impact phase of load determination was spread out over a long period of time and conducted with different men in charge of the work. These conditions produced data of such a nature that it is convenient to divide the whole study into four distinct phases, or groups, which have been designated as groups 1, 2, 3, and 4.

Impact Group 1.—Group 1 includes the data taken prior to September 1, 1923, by Clyde Mason. The early work included in this group was, of necessity, of a pioneer nature. So far as could be ascertained, no similar work had ever been performed and it was necessary to do a great deal of experimenting with apparatus before a satisfactory method of recording impact was developed.

Impact Group 2.—Group 2 includes the work done in the spring of 1924 up to about the 1st of July. Impact runs were made on levels of fill from 2 to 6 feet high under the same conditions as those existing when the data for Group 1 was taken; Group 2 constitutes a check series for Group 1.

Impact Group 3.—Group 3 is a set of readings taken on the same levels and under the same conditions as Group 2, except that the culvert was surrounded by the friction reducing crib mentioned under the discussion of static loads.

Impact Group 4.—Group 4 covers a series of impact observations taken on the 2 and 3 foot levels, in which impact was measured by means of the McCollum-Peters strain telemeter. The method of recording impact by this instrument is of such a different nature from that employed in the other three groups, that these results form a valuable check on the first work.

Table 5 gives the results from two runs selected at random from a large number of tests made. With regard to the complete data included in the report it is noted that the percentages of impact are far from being uniform; that is, two runs apparently made under the same conditions of track, of speed, and apparatus, quite often show radically different results. There are undoubtedly many reasons why these inconsistencies exist. One contributing factor was the difficulty of always guiding the truck so the rear wheel would strike the center of the section. The shifting character of the gravel fill and the steep downward approach to the culvert gave conditions which probably affected the results somewhat. It was also difficult to place the plank obstruction so that the truck wheel would strike the track in the same place each time. In some cases the wheel would merely roll off the edge of the planking and deliver a comparatively light blow to the gravel, while at other times the wheel would follow the path of a projectile and strike the track several inches in front of its customary place, but with a much more severe impact. There seemed to be no way of controlling this, as speed did not seem to cause the difference. In fact, some of the lowest impact percentages were obtained at the highest speeds. In general, the impact seemed to increase as the speed of the truck increased, though the range of speeds was too small to warrant definite conclusions on this point.

TABLE 5.—*Results of impact test runs selected at random from each group of tests*

GROUP 1

Height of fill (feet)	Truck speed (m. p. h.)	Obstacle	Calculated transmitted static load (pounds)	Loads from calibration curve (pounds)		Impact effect, percentage of calculated transmitted static load [1]
				Static	Impact effect	
3	2.3	Smooth track	1,596	970	1,050	66
	2.0	do	1,596	970	1,050	66
	8.1	2 by 4 inch block	1,596	970	1,860	117
	7.4	do	1,596	970	1,720	108
	4.1	4 by 6 inch block	1,596	400	3,160	198
	5.0	do	1,596	970	3,040	190
	7.7	4-inch hole in track	1,596	1,160	3,440	216
	7.0	do	1,596	1,350	3,580	224
	4.1	6-inch hole in track	1,596	1,040	3,080	193
	4.4	do	1,596	620	3,080	193
4	6.1	Smooth track	1,049	1,570	2,100	208
	5.1	do	1,049	1,420	2,040	114
	6.5	4 by 6 inch block	1,049	1,500	2,520	240
	7.2	do	1,049	1,460	2,580	246
	5.8	4-inch hole in track	1,049	1,570	3,840	366
	5.1	do	1,049	1,460	3,580	341
	5.7	6-inch hole in track	1,049	720	2,020	193
	4.3	do	1,049	720	2,160	206
5	9.3	Smooth track	908	1,250	1,600	176
	8.6	do	908	1,120	1,360	150
	8.6	4 by 6 inch block	908	1,120	1,800	198
	9.1	do	908	1,250	1,600	176
	5.0	4-inch hole in track	908	1,000	2,140	236
	6.0	do	908	1,000	2,480	273

[1] Total effect transmitted to culvert top from below on roadway surface.

TABLE 5.—*Results of impact test runs selected at random from each group of tests*—Continued

GROUP 2

Height of fill (feet)	Truck speed (m. p. h.)	Obstacle	Calculated transmitted static load (pounds)	Loads from calibration curve (pounds)		Impact effect, percentage of calculated transmitted static load
				Static	Impact effect	
2	2.0	Smooth track	2,642	1,200	1,420	54
	1.7	do	2,642	1,200	1,260	48
	2.5	2 by 4 inch block	2,642	1,200	2,080	79
	2.8	do	2,642	1,320	2,240	85
	1.9	4 by 6 inch block	2,642	1,400	2,560	97
	1.9	do	2,642	1,460	2,340	89
	1.9	2-inch hole in track	2,642	1,540	2,340	89
	3.6	do	2,642	1,590	2,620	99
3	2.7	Smooth track	1,581	860	940	59
	2.0	do	1,581	860	1,060	67
	------	2 by 4 inch block	1,581	800	1,580	100
	2.1	do	1,581	800	1,400	89
	1.1	4 by 6 inch block	1,581	850	1,420	90
	2.5	do	1,581	880	1,800	114
	2.1	2-inch hole in track	1,581	880	1,880	119
	5.7	do	1,581	1,000	2,040	129
	2.1	4-inch hole in track	1,581	960	1,700	108
	2.2	do	1,581	960	1,820	115
	3.2	6-inch hole in track	1,581	1,150	2,720	172
	2.0	do	1,581	1,040	2,140	135
4	1.4	Smooth track	1,039	670	860	83
	2.1	do	1,039	720	900	87
	3.8	4 by 6 inch block	1,039	720	1,420	137
	4.3	do	1,039	720	1,340	129
	1.4	4-inch hole in track	1,039	720	1,140	101
	1.4	do	1,039	720	1,320	127
	2.0	6-inch hole in track	1,039	720	1,860	179
	2.4	do	1,039	800	1,820	175
5	3.4	Smooth track	751	300	600	80
	3.7	do	751	320	640	85
	3.3	4 by 6 inch block	751	360	980	131
	3.3	do	751	340	920	123
	3.1	4-inch hole in track	751	340	860	115
	2.1	do	751	340	780	104
	3.7	6-inch hole in track	751	360	1,200	160
	3.7	do	751	410	1,180	157

GROUP 3

Height of fill (feet)	Truck speed (m. p. h.)	Obstacle	Calculated transmitted static load (pounds)	Static	Impact effect	Impact effect, percentage of calculated transmitted static load
2	------	Smooth track	2,948	2,160	2,380	81
	3.7	do	2,948	2,160	3,260	111
	2.2	2 by 4 inch block	2,948	2,070	3,200	108
	1.9	do	2,948	1,980	2,880	98
	6.0	4 by 6 inch block	2,948	2,160	4,240	144
	7.0	do	2,948	2,160	4,220	143
3	1.4	Smooth track	1,764	680	780	44
	3.4	do	1,764	680	800	45
	2.7	2 by 6 inch block	1,764	600	980	56
	2.4	do	1,764	550	940	53
	3.5	4 by 6 inch block	1,764	770	2,160	122
	4.3	do	1,764	770	3,720	211
	3.7	4-inch hole in track	1,764	930	3,720	210
	2.8	do	1,764	960	2,140	121
4	1.8	Smooth track	1,159	870	1,140	98
	2.1	do	1,159	890	1,220	105
	6.0	4 by 6 inch block	1,159	980	1,820	157
	7.1	do	1,159	900	1,320	114
	7.1	4-inch hole in track	1,159	1,050	1,920	166
	7.6	do	1,159	1,100	2,140	185
5	3.0	Smooth track	838	400	660	79
	5.5	do	838	380	640	76
	9.9	4 by 6 inch block	838	480	780	93
	10.0	do	838	410	760	91
	4.7	4-inch hole in track	838	550	1,180	141
	3.5	do	838	470	820	98

GROUP 4

Height of fill (feet)	Truck speed (m. p. h.)	Obstacle	Calculated transmitted static load (pounds)	Static	Impact effect	Impact effect, percentage of calculated transmitted static load
2	3.3	Smooth track	2,580	2,500	3,500	136
	4.4	do	2,580	2,650	3,750	145
	3.2	4 by 6 inch block	2,580	3,150	8,600	334
	4.1	do	2,580	2,950	8,700	338
	---	2-inch hole in track	2,580	3,000	7,500	291
	7.4	do	2,580	2,850	6,350	246
3	8.5	Smooth track	1,540	1,100	1,750	114
	7.2	do	1,540	1,100	1,300	85
	5.5	4 by 6 inch block	1,540	950	3,550	296
	4.8	do	1,540	850	3,900	253
	5.7	4-inch hole in track	1,540	1,000	4,250	276
	4.2	do	1,540	900	2,750	178

CONCLUSIONS REGARDING IMPACT LOADS TRANSMITTED TO CULVERTS

Viewing the results obtained in the impact determinations from the standpoint of the design of culverts, which is, of course the ultimate reason for conducting the tests, it may be stated for the range of conditions covered in these experiments that culverts should be designed to carry loads equal to the dead loads upon them from the embankment materials plus from 150 to 200 per cent of $C_t \frac{T}{l}$ (see pp. 113) to allow for the effect of moving concentrated traffic loads.

The values used for C_t should be those calculated by the formulæ given on page 113.

The actual impact blows reaching the culvert tops vary greatly with accidental conditions accompanying the exigencies of actual traffic, weather, and soil conditions. The above allowances are believed to safely provide for the impact effects which are reasonably expected to occur occasionally, at least for trucks running at speeds up to 10 miles an hour.

As with static superloads, the effects of moving superloads are negligible for heights of embankment exceeding 5 feet for culverts up to 3.5 feet in width and above moderate heights for wider culverts.

(Continued from page 112)

The treatment of the piles extended from 2 feet below mean low water to one foot from the end of the pile, the treated length varying from six to sixteen feet. The pile was always dry when painting was started and was allowed to dry thoroughly after each successive coat. The painting was done with ordinary calcimine brushes and a penetration of from one-eighth to one-sixteenth inch was obtained.

The cost of painting the piles, if so stored that they are accessible, is very small. It would be advisable to arrange the storage so that as few piles as possible have to be moved for painting, by so spacing them that they can be reached on all sides for treatment.

The length of time between the application of the seal coat and driving should be not less than seven days.

Cheat Haven Dam, Cheat Haven, Pa.—This treatment, reported by H. H. Haggard, of Sanderson & Porter, Engineers, New York, was applied to the upstream face of the dam and to the interior of penstocks and scroll cases. Approximately 7,000 square yards of the dam face was given two coats of water-gas tar at the rate of about 0.285 gallons per square yard, followed by one coat of coal tar at a cost for both applications of about 30 cents per square yard. The work was all submerged in December, 1925, and has not since been exposed for observation.

The penstocks and scroll cases received two coats of water-gas and two coats of coal tar. The two coats of coal tar proved to be too heavy and showed a tendency to run before water was admitted to the penstocks. After the plant had been in operation a few months an examination showed that the coating in the penstocks was still intact, but in the scrolls, where velocity of water was relatively high, the coal tar was stripped clean in patches exposing the concrete surface, stained dark brown by the absorbed water-gas tar.

Observations in the tunnels of the dam indicated that extremely little water had penetrated the concrete. Water had entered the tunnels at some contraction joints, but it was not expected that they would be tight. The penstocks and scrolls leaked somewhat in spots, which might have been avoided if the waterproofing could have been applied under more nearly perfect conditions.

In connection with the waterproofing of the dam, concrete test cylinders were made up of a 1:3:5½ mix of wet and dry consistency, cured in damp sand and air for 14 and 3 days, respectively, and given two coats of water-gas tar and one coat of coal tar. On completion of the treatment the test pieces were immersed in water for 7 days and the gain in weight after this period was found to be 1.4 per cent for the wet and 0.9 per cent for the dry-mixed concrete.

Abutment of Hancock-Sullivan Bridge over Taunton River, Me.—It is known that a treatment of four coats of water-gas tar and a seal coat of coal tar has been applied below water level; but no details of the treatment have been obtained.

GENERAL SUMMARY AND CONCLUSIONS

From the results of the tests both in the field and laboratory the following conclusions seem warranted.

1. That water-gas tar of the proper quality is readily absorbed by cement mortar and concrete, the rate of absorption varying with the manner of curing, age and density of the mix. Concrete of a 1:3:6 mix cured, respectively, 48 hours and 7 days under moist conditions in the forms followed by 7 days' exposure to dry air, was found to be the most absorptive, while a 1:1½:3 mix, cured 7 days in forms and 83 days in air, was the least absorptive.

2. That the absorption of coal tar by concrete is similar to that of water-gas tar except that the quantity absorbed increases with the time of exposure after treatment with water-gas tar.

3. That a treatment consisting of 4 coats of water-gas tar applied at the rate of about one-fourth gallon per square yard of surface, followed by one coat of coal tar appears to afford adequate protection against alkali attack, provided the concrete is of good quality, has been properly fabricated and not leaner than a 1:2:4 mix.

PARAFFIN SOLUTION USED ON MOLDS FOR CEMENT MORTAR BRIQUETTES

Paraffin solution has been found to be much better than oil for preventing cement mortar from adhering to briquette molds, glass plates, or any other laboratory apparatus, according to R. B. Dayton, materials engineer of the State Road Commission of West Virginia.

A 6 to 7 per cent solution of paraffin in carbon tetrachloride is used instead of the customary heavy mineral oil. It may be applied with either a brush or rag. The surfaces of the molds are very easy to clean, requiring but slight brushing with a stiff fiber brush. The top and bottom of briquette molds require a slight scraping with a small trowel to remove the mortar, as the process of forming removes the paraffin coating, but this also happens when oil is used.

One advantage of the paraffin solution over oil is that the paraffin solution is quick drying and does not become incorporated with the mortar as sometimes happens with excess oil on the molds and plates.

EFFECT OF QUALITY OF PORTLAND CEMENT UPON THE STRENGTH OF CONCRETE

A REPORT ON TESTS CONDUCTED JOINTLY BY THE MICHIGAN STATE HIGHWAY DEPARTMENT AND THE BUREAU OF PUBLIC ROADS [1]

Reported by F. H. JACKSON, Engineer of Tests, Bureau of Public Roads

During the last several years Portland cement concrete has been the subject of a great deal of study. Many tests have been made to determine the effect of such factors as quantity of water, quantity of cement, character and gradation of aggregates, etc., on the quality of the product. However, the effect of the quality of cement upon the quality of the concrete has received very little attention, the tacit assumption being, apparently, that any variations in the quality of the cement would be of relatively small importance, in so far as the quality of the resulting concrete is concerned, provided the cement passed the minimum requirements of the American Society for Testing Materials.

The results of tests made jointly by the Michigan State Highway Department and the Bureau of Public Roads at the Ann Arbor laboratory of the former during the summer of 1926 and presented in this report show that such an assumption may not hold under all conditions, and call attention to the desirability of studying those factors, including the cement, which affect the rate of hardening of paving concrete, because of the economic importance of this feature in influencing the time necessary to keep the completed pavement out of service.

Although the number of specimens subjected to tests was not large enough to justify general conclusions, their remarkable consistency points strongly to the probability that the strength of concrete in tension, flexure, and compression varies directly with the tensile strength of standard 1:3 mortar briquettes made of the cement and Ottawa sand. They indicate also that the strength of a concrete pavement at the end of the customary curing period may or may not be as great as the assumed strength, depending upon the character of the cement used, all other elements being the same.

It is now quite generally recognized that the tensile and flexural strength of concrete are of more significance than the crushing strength in determining the value of the product for use in pavement construction. Other things being equal, the distance apart at which transverse cracks will form in a plain concrete pavement will vary directly with the tensile strength of the concrete. Moreover, modern methods of design utilize the flexural strength of the concrete as a basis for calculating the thickness of pavement necessary to carry the maximum loads which will be allowed upon it. Unless traffic is restricted the maximum load is just as apt to come upon the pavement the day the road is opened as at some later period. Therefore, the critical strength of the concrete is the strength which it has attained at the time traffic is allowed upon the pavement. Any increase in strength which the concrete may attain subsequently should, in general, be considered only in the light of an additional factor of safety and should not be utilized in design. This, of course, assumes that there will be no retrogression in strength at any later period. The rate of hardening, therefore, becomes an important item and should control the time necessary to keep the pavement closed to traffic.

It was for the purpose of determining the effect of variations in the quality of certain Portland cements used in the State of Michigan upon the strengths of paving concrete at the end of the curing period, as well as the relative rate of increase in strength up to and subsequent to this critical stage of the pavement's history, that the cooperative investigation was undertaken.

CHARACTER OF THE CEMENT THE ONLY VARIABLE IN THE TESTS

All concrete specimens used in the study were of the standard paving mix used by the Michigan State Highway Department, which is 1:2:3½ by volume. The only variable was the quality of the Portland cement, of which three grades were used—a slow-hardening grade with a briquette strength averaging about 200 pounds per square inch, a grade of medium strength running about 250 pounds per square inch, and a cement of fairly high early strength testing approximately 300 pounds per square inch, all at seven days.

A single consistency, corresponding to good average paving practice, was used throughout. Typical aggregates, a sand and gravel conforming in all respects to the requirements of the Michigan State highway specifications were employed; and all proportioning, mixing, molding, curing, and testing were strictly in accordance with the practice recommended by the American Society for Testing Materials. The results of routine tests on the three Portland cements used are given in Table 1, and the gradation and physical properties of the aggregates in Table 2. Thus far the specimens have been tested at the age of 7, 14, 21, 28, 90, and 180 days. Specimens for one and two year tests have been made and will be tested at the proper time.

TABLE 1.—*Results of tests on three Portland cements used in Michigan concrete tests*

Type of cement	Fineness—retained on No. 200 sieve	Setting time				Tensile strength	
		Initial		Final		7 days	28 days
	Per cent	*Hrs.*	*Min.*	*Hrs.*	*Min.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
High strength	18.5	4	10	5	40	325	400
Medium strength	18.7	3	55	5	55	255	380
Low strength	20.0	4	10	6	00	180	315

Three types of tests have been made—direct tension, flexure, and compression. For the tension tests, cylinders 6 inches in diameter by 21 inches long were cast and tested in accordance with the method first

[1] These tests were conducted at the Ann Arbor laboratory of the Michigan State Highway Department, under the joint supervision of R. L. Morrison, director of the laboratory, and C. E. Proudley, formerly assistant engineer of tests, Bureau of Public Roads.

TABLE 2.—*Results of tests of aggregates used in Michigan concrete tests*

SAND		GRAVEL	
Size:		Size:	
Retained on No. 10 sieve per cent	28	Retained on 1½-inch screen per cent	0
Retained on No. 30 sieve per cent	75	Retained on 1-inch screen per cent	30
Retained on No. 50 sieve per cent	93	Retained on ¾-inch screen per cent	60
Retained on No. 100 sieve per cent	98	Retained on ½-inch screen per cent	85
Color	O. K.	Retained on ¼-inch screen per cent	100
Weight per cubic foot pounds	112	Per cent of wear (gravel test)	5
Strength ratio, 7 days	138	Weight per cubic foot pounds	109
Strength ratio, 28 days	135		

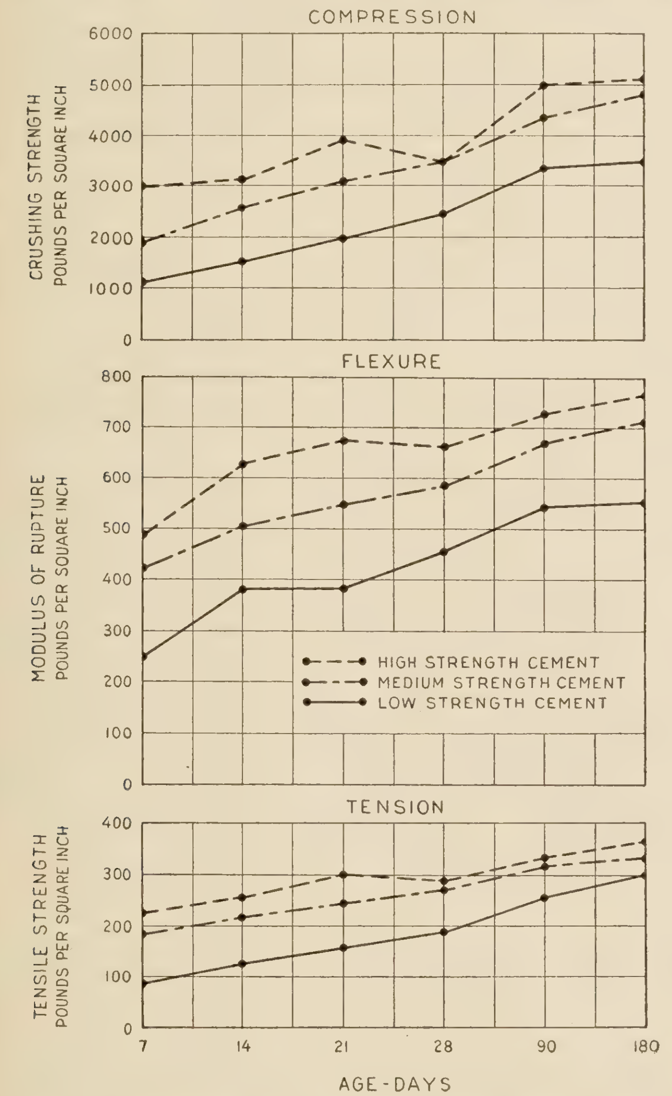

FIG. 1.—STRENGTH OF CONCRETE MADE FROM THREE GRADES OF PORTLAND CEMENT

used in the structural materials research laboratory at Lewis Institute, Chicago (now the research laboratory of the Portland Cement Association), and described in the Proceedings of the American Society for Testing Materials for 1926.[2] By this method the cylinders are broken in a universal testing machine. The grips consist of two pieces of 6-inch steel pipe lined with leather and split part way along four elements so as to slip over the ends of a cylindrical concrete specimen. The segments are then drawn tight by means of tangential bolts to prevent slipping during the application of the load. Ball and socket joints in each grip make the set-up self-aligning. In general, two breaks were secured from each specimen.

The specimens for the flexure tests were beams 6 by 6 by 36 inches in size, tested as cantilevers by the method first proposed by Clemmer.[3] The beams are supported as cantilevers and an extension arm with a container at the extreme end placed over the free end of the beam. Load is applied by an even flow of shot or water from a separate container equipped with a quick-acting valve. Calculations to determine the flexural strength involve the overhang of the specimen, the extension arm, and the weight of shot or water required to cause failure. Two breaks were secured from each beam.

The compressive tests were made in the conventional manner on 6 by 12 inch cylinders; and in these, as in the other tests, the specimens were tested immediately on removal from moist storage.

The schedule of tests called for three specimens of each type for each age and for each cement. As it was possible in almost all cases to obtain two breaks from each of the tension and flexure specimens, six values for tensile and flexual strength were obtained and reported for each combination and three values in compression. The results of all tests up to and including the six-month period are given in Tables 3 to 5, inclusive. The average strengths shown in these tables for each of the three cements tested are plotted against age in Figure 1, and in Figure 2 the results of the three types of concrete tests at the age of 7 and 28 days are plotted against the tensile strengths of the cements as shown by 1:3 Ottawa sand briquette tests at the same periods.

TABLE 3.—*Results of tests of 1:2:3½ gravel concrete specimens made with high-strength cement*

Type of test	Age in days					
	7	14	21	28	90	180
Compressive strength in pounds per square inch of 6 by 12 inch cylinders. $\frac{W}{C}=0.76$.	3,073	3,400	3,840	3,060	4,900	5,115
	2,890	2,865	3,630	3,995	5,200	5,100
	2,970	3,168	4,220	3,310	4,810	5,020
Average	2,978	3,144	3,897	3,455	4,970	5,078
Tensile strength in pounds per square inch of 6 by 21 inch cylinders. $\frac{W}{C}=0.78$.	203	------	297	276	334	371
	------	232	294	282	314	------
	224	262	312	303	316	374
	------	279	300	294	337	363
	237	246	282	274	352	352
	235	273	310	263	361	------
Average	225	258	299	282	336	365
Modulus of rupture in pounds per square inch of 6 by 6 by 36 inch beams, tested as cantilevers. $\frac{W}{C}=0.75$.	483	678	542	625	702	781
	465	715	615	668	708	780
	532	664	818	678	745	729
	498	654	858	703	790	684
	462	495	672	665	706	834
	497	562	548	620	682	786
Average	489	628	676	660	722	766

BRIQUETTE STRENGTH OF CEMENT AND STRENGTH OF CONCRETE RELATED

The effect of variations in the quality of cement upon the strength of the concrete, as shown by Figure 1, appears to be quite marked. Observing the relative values for each of the three grades at the end of the conventional curing period of 21 days, it will be seen that the strength of the concrete containing the high-test cement is approximately double that of the concrete

[2] Proc. A. S. T. M., 1926, pt. 2, p. 447.

[3] Proc. A. C. I., 1926, p. 304.

containing low-test cement, and that the concrete in which the medium-strength cement was used runs about midway between. The significance of such wide variations in strength from the design standpoint will be readily apparent.

TABLE 4.—*Results of tests of 1 : 2 : 3½ gravel concrete specimens made with medium-strength cement*

Type of test	Age in days					
	7	14	21	28	90	180
Compressive strength in pounds per square inch of 6 by 12 inch cylinders.	1,881	2,670	3,435	3,383	4,320	4,762
$\frac{W}{C}=0.76$.	1,816	2,540	2,880	3,610	4,390	4,895
	2,000	2,535	2,992	3,500	4,200	4,685
Average	1,899	2,582	3,102	3,497	4,303	4,781
Tensile strength in pounds per square inch of 6 by 21 inch cylinders.	194	-------	273	297	309	344
$\frac{W}{C}=0.78$.	-------	207	259	299	303	-------
	-------	225	215	-------	291	315
	-------	211	-------	246	346	-------
	178	222	259	273	329	332
	177	210	206	235	333	-------
Average	183	215	242	270	319	330
Modulus of rupture in pounds per square inch of 6 by 6 by 36-inch beams tested as cantilevers.	456	525	487	576	694	743
$\frac{W}{C}=0.75$.	430	500	531	554	686	749
	447	544	627	592	666	669
	442	469	568	625	703	664
	376	487	551	591	619	702
	393	495	512	588	639	716
Average	424	503	549	588	668	707

TABLE 5.—*Results of tests of 1 : 2 : 3½ gravel concrete specimens made with low-strength cement*

Type of test	Age in days					
	7	14	21	28	90	180
Compressive strength in pounds per square inch of 6 by 12 inch cylinders.	1,051	1,230	2,000	2,482	3,260	4,055
$\frac{W}{C}=0.76$.	1,158	1,760	-------	2,290	3,412	3,195
	1,150	1,710	1,970	2,470	3,250	3,250
Average	1,120	1,567	1,985	2,414	3,307	3,500
Tensile strength in pounds per square inch of 6 by 21 inch cylinders.	-------	-------	143	158	240	305
$\frac{W}{C}=0.78$.	-------	112	174	210	285	-------
	86	-------	155	196	253	314
	-------	124	166	196	-------	309
	89	145	162	189	262	275
	88	-------	150	174	225	-------
Average	88	127	158	187	253	301
Modulus of rupture in pounds per square inch of 6 by 6 by 36 inch beams tested as cantilevers.	-------	442	337	428	570	558
$\frac{W}{C}=0.75$.	255	450	341	524	599	587
	222	332	435	423	519	529
	244	359	449	456	571	537
	252	377	361	486	486	585
	268	345	372	447	410	520
Average	248	384	383	461	543	553

If it be assumed that the edge thickness of a concrete pavement has been determined by the corner formula on the assumption that the concrete will have a modulus of rupture of 600 pounds per square inch at the time the road is opened to traffic, it will be found from Figure 1 that, with the aggregates employed and for the conditions obtaining in these tests, the required modulus of rupture was attained in approximately 12 days by the use of the high-strength cement, in somewhat more than 28 days when the medium-strength cement was employed, and that it had not been reached up to six months when the low-strength cement was used. The low-strength cement did not pass the requirements for strength of the American Society for Testing Materials, but if, for this reason, we ignore the results of the tests on the concrete in which it was used, we still find the time required to produce concrete of the required quality to be markedly longer when the medium-strength cement (which passed all A. S. T. M. requirements) was used than when the high-strength cement was employed.

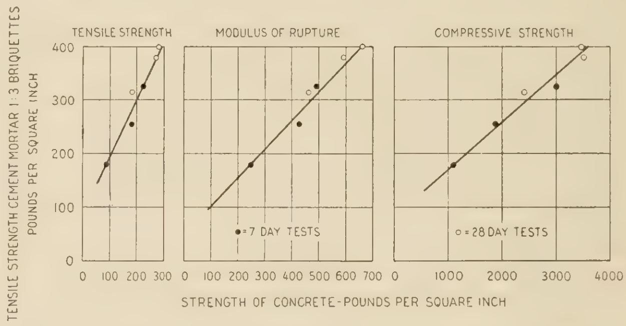

FIG. 2.—RELATION BETWEEN QUALITY OF CEMENT AS SHOWN BY ROUTINE TENSILE TESTS OF BRIQUETTES AND THE QUALITY OF CONCRETE (1 : 2 : 3½ BY VOLUME) FOR THREE GRADES OF PORTLAND CEMENT

Referring now to Figure 2, it will be observed that there is a very interesting relation between the results of briquette tests on the three cements at 7 and 28 days and the results of the various concrete tests at the same periods. There appears to be a very definite relation between briquette strength and concrete strength at corresponding ages. This relation, moreover, applies to all three types of concrete tests. It will not be asserted, of course, that these results are sufficient to warrant any conclusions of a general nature. It is believed, however, that the meagerness of the data presented is to a certain extent balanced by the remarkable concordance of the results obtained, so that they may be said to be at least indicative and, therefore, of value in suggesting a field for additional and more extensive research.

REPORT AVAILABLE ON RUN-OFF FROM SMALL AGRICULTURAL AREAS

An article entitled "Run-off from small agricultural areas," by C. E. Ramser, drainage engineer, reprinted from the Journal of Agricultural Research, volume 34, No. 9, will be of interest to highway engineers. The report deals with rainfall and run-off measurements made on six watersheds ranging in area from 1¼ to 112 acres in Madison County, Tenn. The results are applicable to the design of all types of drainage structures where similar conditions exist. Copies of this report may be obtained without charge from the Office of Information, United States Department of Agriculture.

CAPPING SQUARE FOR CONCRETE COMPRESSIVE STRENGTH SPECIMENS

By F. V. REAGEL, Engineer of Materials, Missouri State Highway Department

The road materials testing laboratory of the Missouri State Highway Department has for some time been using a device for capping concrete cylinders for compression tests which gives the specimens a uniform bearing and at the same time permits the work to be done rapidly. This device was developed by A. D. Conrow, formerly assistant testing engineer and now employed in the same capacity by the Kansas Department of Highways. Comments of visiting engineers have indicated that the method which has been developed will be of general interest to testing engineers.

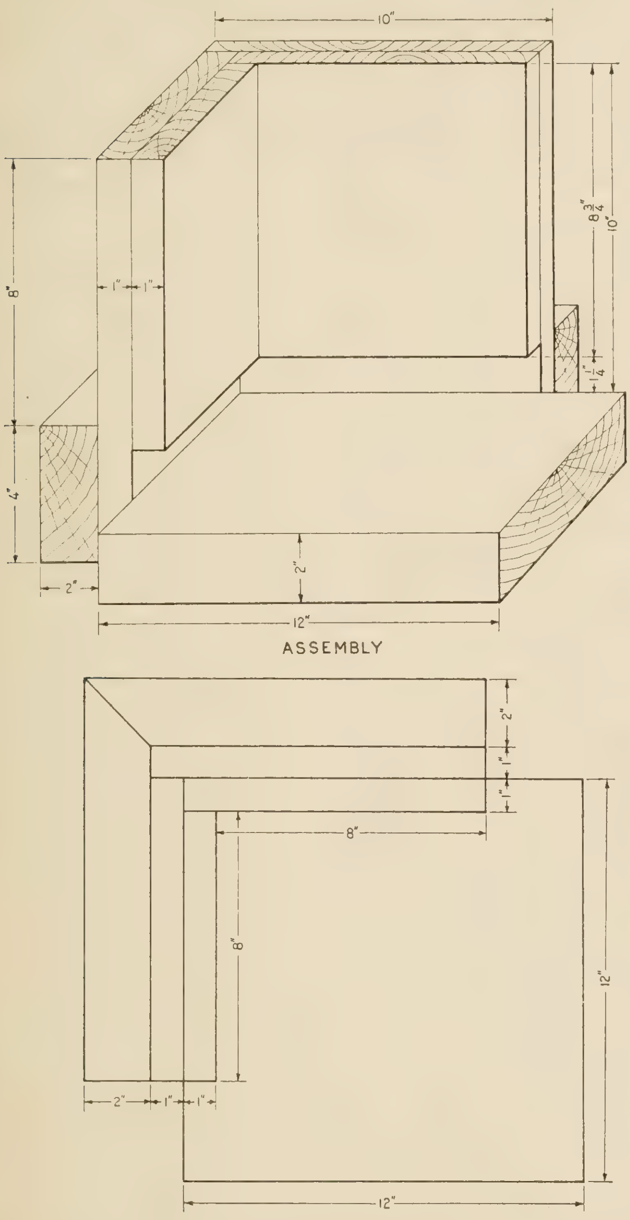

Fig. 1.—Showing Details of Construction of Capping Square

Figure 1 shows the details of design of the apparatus, and with this guide it can be constructed at small expense. Well-seasoned wood should be used for constructing the framework to prevent warping, and the inside faces are lined with 16-gauge metal. The slot shown just above the base is to receive a square metal plate, from three-eighths to one-half inch in thickness, on which the specimens are capped. For convenience in handling, three rivet or bolt heads are inserted in the bottom of the plate.

CYLINDERS CAPPED ACCURATELY AND RAPIDLY

The plate upon which the specimen is to be capped is first given the usual coating of oil to prevent sticking. Sufficient capping material for one cap plus a slight excess is then deposited upon the plate. The plate is shoved into the corner snugly, the groove at the bottom allowing the specimen to rest against the corner and still remain near the center of the plate. The specimen is lowered upon the capping material with a rotating motion under considerable pressure, the motion finally ending with the specimen resting snugly against each side of the corner of the capping square. The rotating motion serves to squeeze out the excess capping material which is rapidly worked up around the outside of the specimen. A large trowel is then inserted under the plate and the plate and specimen set aside until the cap has hardened.

Capping Square in Use and Capped Specimens

As soon as the cap has sufficiently hardened the plate is loosened by tapping lightly against the edge, and the process is repeated with the other end of the specimen upon the plate. Using this method, one operator can cap about 80 specimens on each end per day and the resulting specimens will have uniformly parallel ends.

The capping materials in use at present are a mixture of plaster of Paris and Portland cement for hand-cast specimens and a 1 to 3 mortar of Lumnite cement and sand for the more irregular field-drilled specimens.

The illustration shows the capping device in use. On account of the extreme irregularity of some specimens drilled from pavements, this process is not always suitable for use in capping each end of the specimen. In such cases we use the metal collar shown on the specimen in the extreme right of the picture. This collar is of 18-gage metal and is provided with two wing-nut bolts by means of which the collar may be drawn tightly around the specimen. Capping material is then placed within the collar, building up the entire specimen to the height of the longest element plus the desired thickness of cap.

NEW RESEARCH PROJECTS INITIATED BY BUREAU OF PUBLIC ROADS

Six new highway research projects have been initiated by the United States Bureau of Public Roads during the current year. Announcement has been in previous issues of PUBLIC ROADS of the Yadkin River bridge tests and of tests of anchorage and bond of pile heads in concrete. The more recent projects include investigations of motor-truck impact as influenced by road type, motor-truck impact as influenced by wheel type, statistical analysis of highway grade crossing accidents in 1926, and the effect of type and gradation of coarse aggregate upon the flexural strength of plain concrete.

MOTOR-TRUCK IMPACT INVESTIGATIONS

The investigations of motor-truck impact as influenced by road type and as influenced by wheel type constitute two separate but related investigations. The first of these is for the purpose of determining the cushioning properties of the various road types under controlled conditions of motor-truck impact. A number of road types such as plain concrete, sheet asphalt on concrete base, bituminous concrete on concrete base, and penetration macadam will be selected in the vicinity of Washington, and on these sections motor-truck impacts will be produced identical as to conditions of drop, load, and tire equipment. Comparison between pavement types will then be made on the basis of the magnitude of the impact developed on each under exactly comparable test conditions. Any cushioning properties inherent in the pavement types should tend to decrease the magnitude of the impact.

Two methods are possible for producing the impact forces. The impact machine which has been in use for some years on the slab impact tests can be made portable. This machine is designed to give a wide range of impact blows, and measurement would be made of the height of drop of a wheel representing various conditions of truck load and equipment and the impact reaction. It is also planned to have a truck used in other impact investigations pass over artificial obstructions and to compute the impact forces produced from the record of an accelerometer attached to the truck. The first tests will probably be made under conditions representing a worn tire and heavy truck; and if the differences in impact reactions warrant, the program will be amplified to include other conditions. The tests on bituminous types will include as wide a range as possible of temperature conditions.

The investigation to determine the relative cushioning effect of various types of motor-truck wheels will be similar to but less extensive than the recently completed tests with various tire equipments. Wheels to be tested will include steel disk, wood spoke, and cushion types.

A 2-ton truck will be loaded so as to have 2,500 pounds on each rear wheel and later 5,000 pounds on each rear wheel, and equipped with dual pneumatic, dual new cushion, and dual worn solid tires, and will be operated at various speeds over different types of pavement and artificial obstacles. Impact forces will be measured by the coil-spring accelerometer and comparisons made where all conditions are constant except the type of wheel.

GRADE-CROSSING ACCIDENTS TO BE STUDIED

A statistical analysis is to be made of highway-railroad grade-crossing accidents in 1926, as reported by the Class I steam railroads to the Interstate Commerce Commission. The object is to determine the relative frequency of grade-crossing accidents in rural and urban areas. The data may be used later in an attempt to establish the correct ratios which grade-crossing accidents in the rural areas bear to the total of all the highway accidents in those areas. It is also desired to discover any other signficant evidence as to causes and conditions of accidents which may be revealed in a mass analysis of the 5,890 accidents reported in 1926.

The Bureau of Statistics of the Interstate Commerce Commission has on file a complete set of individual accident reports from all railroads under its jurisdiction. Pertinent data will be transcribed from those involving highway-railroad grade crossings and analyzed by means of tabulating machines.

EFFECT OF GRADING AND TYPE OF CONCRETE AGGREGATE TO BE STUDIED

This project is to determine how quality and economy in concrete construction are effected by variations in type and grading of coarse aggregate. It is planned to obtain a number of representative coarse aggregates such as trap, granite, limestone, sandstone, and dolomite rock, and glacial, siliceous, and calcareous gravels, and blast-furnace slag, selecting a total of 18 aggregates in all. These are to be prepared in six different gradings, as follows:

Coarse aggregate gradations

Grading No.	Total passing, square opening				
	1/8-inch	1/4-inch	3/4-inch	1 1/4-inch	2-inch
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	0	0	15	40	100
2	0	0	30	55	100
3	0	5	45	70	100
4	0	5	45	100	100
5	0	10	65	100	100
6	0	10	100	100	100

Flexure tests are to be made on cantilever beams made of concrete with the above coarse aggregate types and gradings as the only variables. Four mixes are to be used ranging from 1:1½:3 to 1:2:4, making a total of 432 combinations.

Accurate measurements of yield will be made on all combinations. Compression tests will also be made, using the fractured prisms resulting from the flexure tests. All tests will be made at 28 days.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

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

ANNUAL REPORTS

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

DEPARTMENT BULLETINS

No. 105D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136D. Highway Bonds. 20c.
220D. Road Models.
257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370D. The Results of Physical Tests of Road-Building Rock. 15c.
386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387D. Public Road Mileage and Revenues in the Southern States, 1914.
388D. Public Road Mileage and Revenues in the New England States, 1914.
390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463D. Earth, Sand-Clay, and Gravel Roads. 15c.
*532D. The Expansion and Contraction of Concrete and Concrete Roads, 10c.
*537D. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests, 5c.
*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660D. Highway Cost Keeping. 10c.
*670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
*691D. Typical Specifications for Bituminous Road Materials. 10c.
*724D. Drainage Methods and Foundations for County Roads. 20c.
*1077D. Portland Cement Concrete Roads. 15c.
*1132D. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.

DEPARTMENT BULLETINS—Continued

No. 1259D. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279D. Rural Highway Mileage, Income, and Expenditures, 1921 and 1922.
1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

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

MISCELLANEOUS CIRCULARS

No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal Aid Highway Projects.
93M. Direct Production Costs of Broken Stone.
105M. Federal Legislation Providing for Federal Aid in Highway Construction and the Construcion of National Forest Roads and Trails.

FARMERS' BULLETINS

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

SEPARATE REPRINTS FROM THE YEARBOOK

No. *739Y. Federal Aid to Highways, 1917. 5c,
*849Y. Roads. 5c.
914Y. Highways and Highway Transportation.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

JULY 31, 1927

STATES	FISCAL YEARS 1917-1927			FISCAL YEAR 1928									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1927			PROJECTS COMPLETED SINCE JUNE 30, 1927			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 20,061,371.68	$ 9,615,099.94	1,400.2	$	$		$ 6,830,945.98	$ 3,259,611.99	395.6	$ 121,788.96	$ 60,894.48	2.8	$ 2,961,068.59	Alabama
Arizona	11,809,950.70	6,447,169.27	800.8				1,398,735.50	961,236.28	62.7	364,876.07	347,611.63	19.1	2,918,225.82	Arizona
Arkansas	22,337,014.63	9,525,192.75	1,560.6	24,788.75	12,394.37	0.1	2,949,771.43	1,431,836.82	211.3	977,873.98	284,001.03	19.5	1,630,275.03	Arkansas
California	35,128,269.04	16,967,026.82	1,306.3	384,361.20	224,871.31	15.9	7,088,452.73	3,205,702.36	136.3	103,357.10	62,014.26	4.5	4,096,637.25	California
Colorado	15,497,121.91	7,934,298.91	829.0	91,935.74	48,486.64	0.9	6,300,104.29	3,080,024.71	278.1	35,740.10	20,057.34	2.9	2,619,464.40	Colorado
Connecticut	6,397,392.29	2,444,000.54	137.3				5,692,535.10	1,568,064.22	69.9	901,980.80	216,991.88	7.1	577,309.36	Connecticut
Delaware	6,237,026.55	2,345,572.42	159.5				796,771.28	312,891.57	32.9	517,941.52	176,900.72	15.5	4,318.29	Delaware
Florida	7,476,856.31	3,627,912.60	245.1	191,191.56	94,260.00	6.3	8,901,733.22	3,917,573.09	190.1	1,190,401.52	398,091.78	24.1	946,567.53	Florida
Georgia	31,951,436.50	15,101,232.40	2,173.6	407,778.81	193,943.67	18.8	8,622,836.80	4,253,317.42	346.0	406,647.72	203,323.84	8.4	659,344.67	Georgia
Idaho	13,225,515.45	7,075,527.16	835.5				2,460,812.75	1,540,701.94	183.4	450,158.47	270,883.89	25.0	607,707.11	Idaho
Illinois	48,538,982.16	22,781,516.60	1,530.8				10,718,399.76	5,129,488.63	375.7	4,787,716.61	2,377,164.01	168.9	2,698,457.76	Illinois
Indiana	23,372,717.74	11,238,568.20	732.5	205,222.85	99,273.14	6.4	16,069,406.17	7,675,346.44	485.8	1,475,249.55	737,574.77	53.5	380,364.45	Indiana
Iowa	34,306,138.86	14,395,603.75	2,484.4				15,541,892.11	6,684,816.47	597.1	557,728.91	300,298.89	11.6	149,842.89	Iowa
Kansas	37,442,061.61	14,730,829.48	1,495.2	210,334.61	101,233.29	11.4	14,472,997.54	5,702,839.06	741.3	1,212,637.15	490,379.63	93.1	507,661.54	Kansas
Kentucky	23,216,600.53	9,510,694.75	874.9	274,075.75	128,687.99	9.4	9,498,493.81	4,480,432.48	435.4	847,013.75	412,849.17	29.9	98,091.61	Kentucky
Louisiana	15,877,552.20	7,093,892.21	1,178.7	140,406.65	70,203.32	10.7	3,964,364.82	1,858,179.84	122.6	1,143,779.00	309,000.00	0.9	954,440.63	Louisiana
Maine	10,564,800.06	4,858,452.67	357.6	94,229.38	47,040.48	5.2	2,788,314.32	943,327.06	69.6	225,491.03	112,521.00	10.1	1,184,280.79	Maine
Maryland	11,790,203.93	5,524,938.27	477.8				814,475.47	373,185.64	33.9	1,454,726.22	653,700.00	70.7	8,352.09	Maryland
Massachusetts	20,670,246.02	7,425,928.15	410.4	199,749.40	50,955.00	3.4	5,769,172.44	1,489,290.69	89.5	576,808.76	162,286.57	10.9	2,069,365.59	Massachusetts
Michigan	31,977,248.37	14,328,484.99	1,084.2				12,008,915.33	5,451,826.35	369.9	1,861,617.73	780,125.00	40.9	1,996,619.66	Michigan
Minnesota	45,099,648.47	19,046,145.67	3,643.5				7,222,329.30	2,114,898.90	378.3	374,153.82	1,000.00	11.1	550,476.43	Minnesota
Mississippi	18,331,230.75	9,004,294.62	1,314.1	42,332.96	21,166.48	3.8	6,933,805.73	3,357,827.47	376.6	517,585.37	258,792.68	48.4	793,815.75	Mississippi
Missouri	42,389,290.41	19,681,026.48	1,944.8	531,938.13	221,824.30	22.9	8,699,304.87	3,603,728.88	247.9	1,582,685.80	747,158.77	74.7	937,872.57	Missouri
Montana	12,854,995.72	7,287,288.69	1,151.6	104,903.93	92,806.71	18.1	1,881,824.50	1,484,103.69	144.5	2,874,779.44	1,715,075.80	273.1	4,397,109.11	Montana
Nebraska	16,157,040.25	7,739,386.39	2,246.6	634,885.62	314,090.57	80.3	12,795,400.84	6,311,218.98	1,298.1	2,064,620.07	1,029,504.31	125.1	826,172.75	Nebraska
Nevada	10,421,349.31	7,589,168.68	853.6	21,814.74	19,135.88	4.9	1,497,195.87	1,294,256.63	177.8	114,490.21	100,430.81	17.8	740,733.00	Nevada
New Hampshire	5,868,897.76	2,778,928.05	264.8	17,021.81	7,936.70	0.7	1,[illegible],196.40	601,625.45	38.9	232,267.02	92,280.00	6.2	54,346.80	New Hampshire
New Jersey	22,228,240.08	7,495,364.48	316.3	499,831.00	84,960.00	5.7	7,398,011.37	1,704,058.56	111.5	170,592.65	44,520.00	3.0	73,127.96	New Jersey
New Mexico	13,336,250.94	7,937,598.06	1,505.2	29,250.21	18,319.32		3,310,023.00	2,568,095.90	233.6				1,635,135.72	New Mexico
New York	54,183,085.44	21,693,955.65	1,439.3	547,830.55	187,931.34	13.7	37,978,153.00	9,540,513.95	586.9	11,975,433.00	2,514,467.50	160.8	3,743,543.56	New York
North Carolina	35,295,849.21	14,518,903.16	1,460.1				3,172,750.26	1,528,044.97	100.4	1,339,644.20	596,443.50	34.0	787,170.37	North Carolina
North Dakota	15,881,558.55	7,746,293.68	2,715.6				6,017,957.11	3,241,283.89	862.3	1,223,410.74	591,250.17	210.2	364,782.26	North Dakota
Ohio	52,621,391.49	19,331,376.76	1,515.0	193,318.52	83,800.00	5.0	11,521,025.69	4,479,620.07	329.3	1,507,340.00	522,195.00	33.8	4,077,013.17	Ohio
Oklahoma	30,381,957.08	14,117,589.21	1,268.1	18,411.76	8,846.09	5.0	4,160,473.45	1,764,346.57	246.2	1,228,252.00	598,674.70	99.0	1,322,221.43	Oklahoma
Oregon	19,583,584.76	10,041,452.94	1,055.0				2,499,956.26	1,307,528.26	62.4	50,108.85	25,696.80	9.9	686,871.00	Oregon
Pennsylvania	77,726,174.22	26,317,620.32	1,534.3				17,371,715.38	5,421,084.81	351.8	3,628,879.47	1,131,808.81	81.7	1,804,002.06	Pennsylvania
Rhode Island	5,233,413.38	1,998,479.06	116.0				1,291,855.67	338,055.00	22.5	594,256.61	165,263.74	10.2	531,396.20	Rhode Island
South Carolina	17,002,039.93	7,526,988.80	1,568.4				7,035,602.59	2,928,901.40	247.1	351,178.21	52,000.00	15.5	348,621.80	South Carolina
South Dakota	19,262,063.24	9,507,525.54	2,502.9	109,739.28	57,825.99	29.1	3,851,889.87	2,090,331.29	655.2	452,819.57	249,050.74	101.0	482,120.44	South Dakota
Tennessee	24,283,035.03	11,551,457.55	868.7				8,792,828.20	3,614,687.07	225.7	714,113.59	357,056.79	31.7	1,372,155.59	Tennessee
Texas[1]	78,190,246.37	31,566,960.45	5,466.4	1,043,207.81	449,844.91	30.8	14,572,980.38	6,503,161.05	497.5	2,262,108.71	1,007,930.61	62.6	5,575,805.98	Texas
Utah	9,154,377.33	5,767,079.95	628.9	164,797.18	129,266.22	13.4	2,199,742.27	1,658,316.58	165.0	1,041,683.82	759,598.84	69.0	351,423.41	Utah
Vermont	5,037,118.23	2,348,856.01	152.7				3,045,803.81	1,138,098.13	61.2	473,355.09	147,004.12	10.3	173.74	Vermont
Virginia	26,844,025.24	12,537,143.25	1,168.9	161,065.10	72,129.90	5.5	4,853,387.70	1,970,948.28	109.2	873,879.24	345,874.43	15.6	18,132.14	Virginia
Washington	18,184,505.97	8,246,551.95	711.1	153,789.41	75,000.00	4.8	3,527,890.78	1,675,600.00	63.4	237,489.59	65,000.00	10.2	1,215,156.05	Washington
West Virginia	10,424,847.32	4,573,748.01	419.4				7,132,862.00	2,934,191.85	238.2	1,450,131.53	598,675.47	44.1	39,531.67	West Virginia
Wisconsin	27,891,502.16	11,847,858.90	1,729.5	77,656.94	38,828.47	1.3	10,505,040.99	4,885,058.80	394.5	372,065.04	159,950.00	25.0	2,377,573.83	Wisconsin
Wyoming	12,650,712.15	7,139,267.05	1,315.9	37,658.13	22,215.00	4.2	2,673,721.58	1,729,838.73	203.0	71,771.58	46,077.34		563,244.88	Wyoming
Hawaii	343,664.15	97,440.00	6.5				1,786,323.89	562,362.64	29.7				805,975.36	Hawaii
TOTALS	1,154,740,601.48	510,007,691.24	60,957.6	6,613,527.78	2,977,277.03	337.7	347,772,183.61	145,671,480.76	13,685.8	56,992,630.17	22,299,450.82	2,203.4	63,544,100.09	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 88,969,279.12 Federal aid $ 39,575,452.01 Miles 3,550.6