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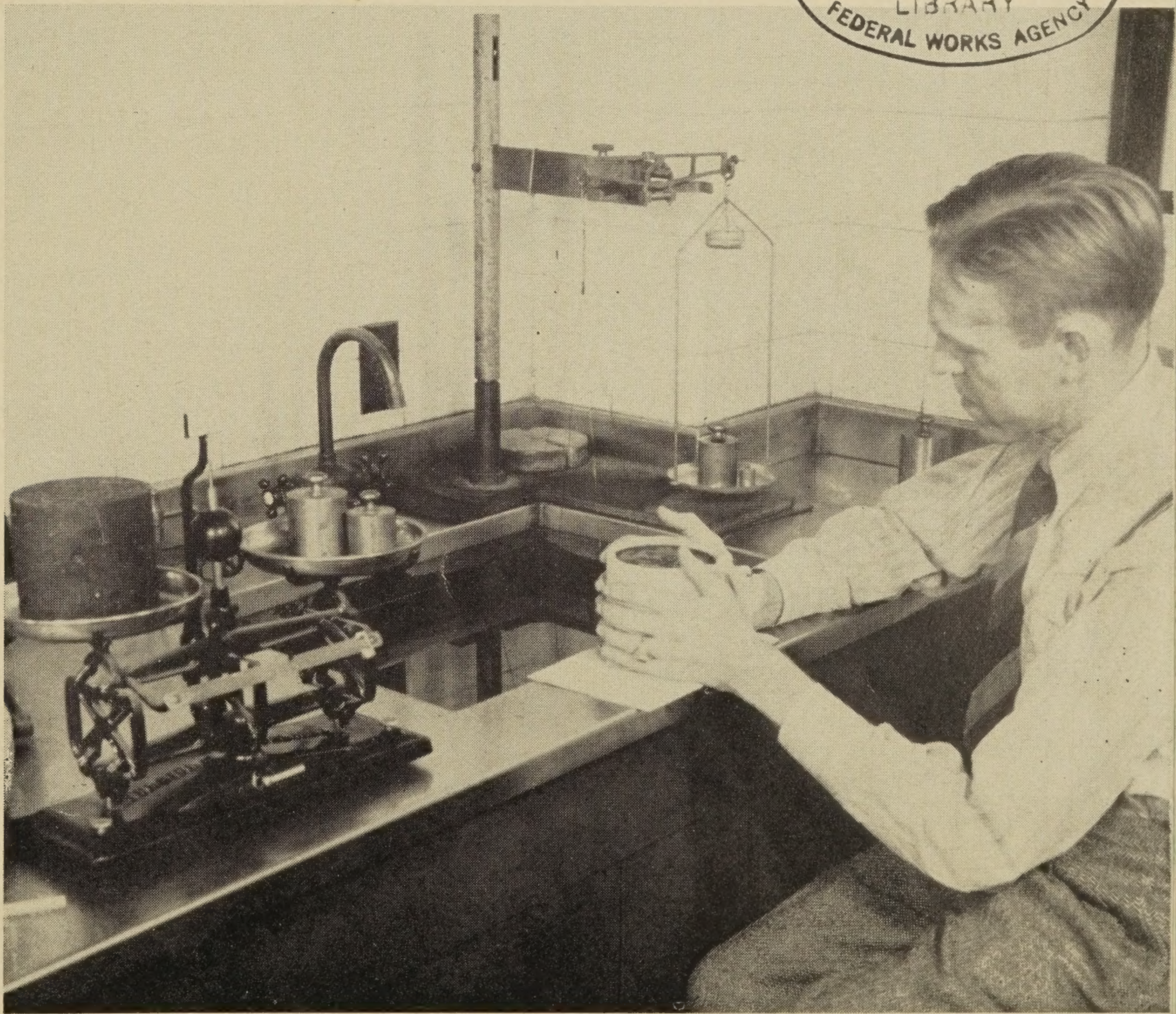

Determining bulk specific gravity of a bituminous test specimen

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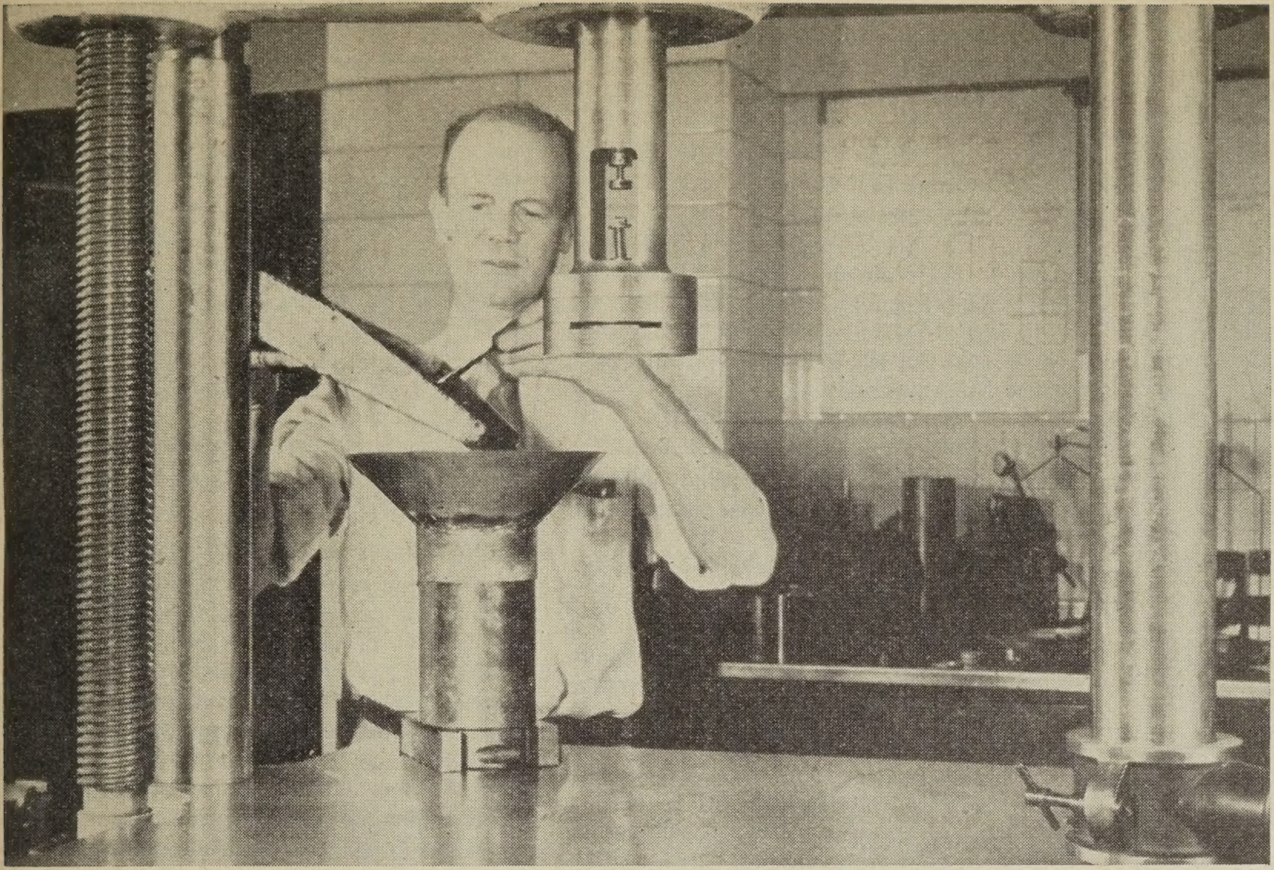

Figure 1.—Filling the 4-inch mold to produce a compressed specimen.

Further Developments and Application of the Immersion-Compression Test

BY THE DIVISION OF PHYSICAL RESEARCH PUBLIC ROADS ADMINISTRATION

Reported by J. T. PAULS, Principal Highway Engineer and J. F. GOODE, Highway Research Engineer

Loss of adhesion of bituminous film to the surface of mineral aggregate through the action of moisture, and the resultant stripping, is often a major cause of failure in bituminous pavements. The immersion-compression test was developed to provide a means of determining the resistance of bituminous mixtures to the detrimental action of moisture. This method has definite advantages over the various stripping tests now in use. It provides a quantitative index of the damage caused by moisture, and the test is made on the entire mixture as it might be prepared for use in the road rather than on the single-size fractions of aggregate that are used in stripping tests.

The immersion-compression test involves a comparison of the compressive strengths of molded cylindrical specimens of bituminous mixtures with the strengths of duplicate specimens that have undergone immersion in water for a definite period of time. The test gages the tendency of a mixture to strip by measuring the reduction in strength of the specimens caused by the loss in adhesion of the bituminous film to the aggregate particles. Reports of the initial development of this test have previously been published. This report describes the broader application of the test to mixtures containing a wide variety of bituminous materials, and presents definite procedures and test conditions that have been found appropriate to the various classes of materials.

THIS REPORT covers the recent developments in the procedure for conducting the immersion-compression test and gives additional results of its application to studies of a number of factors affecting the resistance of bituminous paving mixtures to the action of moisture. Earlier work was described in PUBLIC ROADS [1] in 1945. Subsequent studies showed the need for a number of changes in the procedure originally proposed and provided additional data that were presented in a progress report.[2] The essential information in that progress report and the results of more recent studies of mixtures made with cut-back asphalts and slow-curing liquid asphaltic materials are reported herein and make it possible at this time to establish a procedure that appears to be satisfactory.

The immersion-compression test involves a comparison of the compressive strengths of molded cylindrical specimens of bituminous mixtures with those of duplicate specimens which have undergone immersion in water for a definite period of time. All details of the test are controlled as described in the procedure. Essentially the test is intended to

[1] *A test for determining the effect of water on bituminous mixtures*, by J. T. Pauls and H. M. Rex; PUBLIC ROADS, vol. 24, No. 5, July-August-September 1945.

[2] *Application and present status of the immersion-compression test*, by J. T. Pauls and J. F. Goode; Proceedings of the Association of Asphalt Paving Technologists, vol. 16, February 1947.

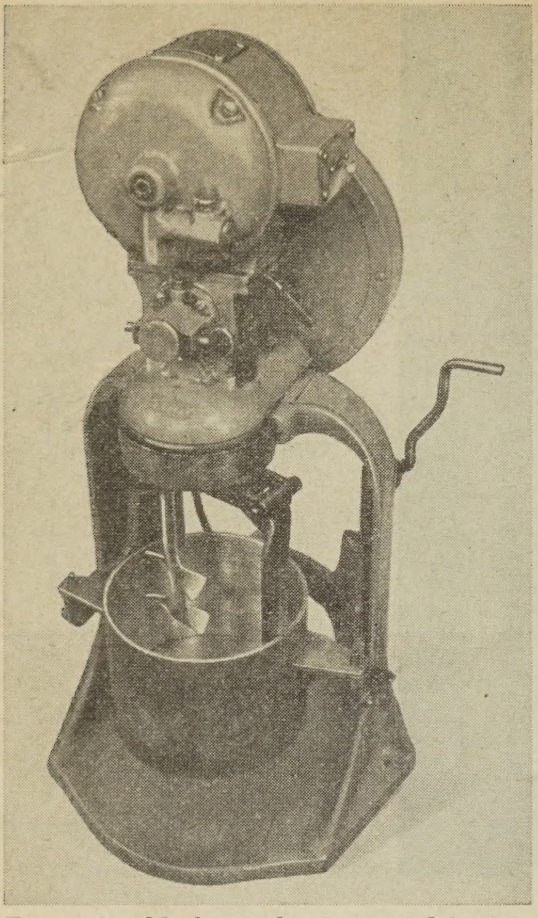

Figure 2.—Mechanical mixer used in preparing individual test batches.

simulate the action of water on a bituminous pavement, and it measures the tendency of a mixture to strip by indicating the loss in strength caused by loss in adhesion of the bituminous film to the aggregate particles. The compressive strength of a cylindrical specimen of a bituminous mixture without lateral support is dependent on the degree of cohesive strength of the mixture, as provided by the bituminous binder; and any damage to the mixture, such as stripping or reduction in the adhesion of the film to the surface of the aggregate particles, results in a measurable loss in the strength of the specimen. For these reasons a compression test of laterally unsupported cylinders is believed to provide a satisfactory means of evaluating the damage caused by water.

This method provides for the testing of bituminous mixtures as used in actual road construction. Thus the water-resistant property of a mixture as a whole is determined and it is also possible to determine the comparative value of each of the constituent parts of the mixture. This is not possible with the more common stripping tests, which are made only on a fraction of the coarse aggregate. It is well known that the sand and filler may have a greater effect on the behavior of a mixture than does the coarse-aggregate fraction. Often the bad effect of a less satisfactory coarse aggregate may be overcome by the use of a good sand or filler or by increasing the density or reducing the size of the pore spaces in the mixtures.

The following details of a procedure for the immersion-compression test as applied to bituminous mixtures are now believed to be satisfactory:

A. Preparation of Specimens

1. Mixing: Thorough mixing in batches just sufficient for one test cylinder.
2. Curing of mixtures before molding:
   (a) Hot mixtures: None.
   (b) Cold mixtures: 18 hours in oven at 140° F.
3. Test specimens:
   (a) Size: 4- by 4-inch cylinders.
   (b) Number: Six specimens for each test mixture, three of which are to be tested dry at the same time that the other three are put in water.
4. Molding temperatures:
   (a) Hot mixtures: 260° F.
   (b) Cold mixtures: 140° F.
5. Molding load (double plunger method) 3,000 pounds per square inch maintained for 2 minutes.
6. Curing after molding: 24 hours in oven at 140° F.
7. Selection of specimens for test groups: Each group of three specimens to have approximately the same average bulk specific gravity.

B. Immersion Conditions

1. Hot mixtures and asphalt emulsion mixtures: 4 days at 120° F. (For a quick test, 1 day at 140° F. may be used.)
2. Cold mixtures (except asphalt emulsion): 4 days at 100° F. (For a quick test, 1 day at 120° F. may be used.)

C. Compression Test Conditions

1. Temperature: 77° F.
2. Rate of deformation: 0.2 inch per minute.

D. Test Results

1. Retained strength of immersed specimens: Taken as the measure of the resistance of the mixture to the action of water, and expressed as a percentage of the compressive strength of the dry specimens.
2. Volumetric swell of immersed specimens: Determined and considered as corollary data to indicate the resistance of the mixture to the action of moisture.

PREPARATION OF SPECIMENS

Mixing.—Most satisfactory results were obtained by mixing in batches of just sufficient size for one test cylinder. In the original work the batches for each of the three specimens of a test group were sampled from a larger batch prepared in a 20-pound capacity laboratory pugmill. Because of segregation, especially in mixtures of the open-graded type, it was very difficult to obtain the desired uniformity among specimens of the same test group. The lack of uniformity that may result is demonstrated by the data given in table 1. These data were obtained from the first and third specimens molded from a 15-pound batch. As indicated by these tests, the first sample is apt to contain more fines than a later sample and therefore result in higher bitumen content and a lower percentage of air voids. Such nonuniformity between individual specimens is usually reflected in the final test results. The differences between the absorptions and between the final compressive strengths were rather wide for the two specimens shown in the table. Because of the difficulty in selecting uniform samples from a larger batch this method has been eliminated from the procedure in the testing of freshly prepared mixtures of the asphaltic concrete type. Instead, the one-batch-per-specimen method has been substituted. Although not necessary for sand-asphalt mixtures, the desirability of its use in the testing of mixtures that tend to segregate is obvious.

The individual batches may be prepared by hand mixing but the use of a small mixing

Table 1.—*Effect of molding from a large batch* [1]

	Specimen [2] First	Specimen [2] Third
Bulk specific gravity	2.56	2.52
Air voids ... percent	3.8	8.4
Absorption after immersion [3] ... percent	0.4	1.1
Compressive strength after immersion ... p. s. i.	271	248
Bitumen, CS_2 extraction ... percent	5.7	4.5
Aggregate passing—		
½ inch sieve ... percent	90	88
⅜ inch sieve ... do	76	70
No. 4 sieve ... do	54	47
No. 10 sieve ... do	33	28

[1] 85–100 penetration asphaltic concrete. Batch for three specimens, 15 pounds.
[2] Each specimen weighed about 4.6 pounds.
[3] 4-day immersion at 77° F.

Table 2.—*Effect of method of curing loose mixtures* [1]

Mixture grade and designation	Curing of loose mixture	Gross loss of original weight of volatile in cut-back— After curing loose mixture	Gross loss of original weight of volatile in cut-back— After curing [2] molded specimens	Compressive strength
RC–4 cut-back:		*Percent*	*Percent*	*P. s. i.*
1	None	[3] 14	26	105
2	24 hours at room temperature	32	39	138
3	18 hours in oven at 100° F	39	46	161
4	4 hours in oven at 140° F	44	49	162
5	18 hours in oven at 140° F	57	64	212
MC–3 cut-back:				
6	None	[3] 5	15	46
7	24 hours at room temperature	16	24	68
8	18 hours in oven at 100° F			81
9	4 hours in oven at 140° F	30	35	76
10	18 hours in oven at 140° F	53	56	105

[1] All of same blend. Aggregate retained on No. 10 sieve, trap rock; passing No. 10 sieve, river sand and limestone dust. Bitumen-aggregate ratio, 4.9 percent. Gradation: Passing ¾ inch sieve, 100 percent; passing No. 10 sieve, 21 percent; passing No. 200 sieve, 6 percent.
[2] 24 hours at 140° F.
[3] This loss took place during the mixing operation.

machine is much more desirable. A mixer now in use that has proven satisfactory is shown in figure 2. It is a kitchen-type mixer with a special mixing bowl, a stirrer for throwing the mix to the sides of the bowl, and a scraper for keeping the sides clean. It produces good mixtures with a minimum of crushing. Usually less than 2 minutes of mixing time is sufficient.

Because of differences in types of mixtures and in mixing machines, it is believed desirable to require that mixing be sufficient to obtain a satisfactory coating rather than to specify any particular time.

Curing of mixture before molding.—As stated in the outline of procedure, no curing is required for the hot mixtures (those containing asphalt cement as the binder). For these mixtures, a molding temperature of 260° F., as specified, makes it possible to mold immediately following the completion of mixing.

The cold mixtures containing cut-back asphalts, slow-curing liquid asphalts, and asphaltic emulsions require curing before molding. Both air and oven curing were studied and it was found that oven curing resulted in a loss of volatile material more nearly comparable with the loss that would be expected in good construction practice.

Table 2 gives the loss of solvent occurring in RC–4 and MC–3 cut-back mixtures when cured under a number of conditions.

The data show that curing of the loose mixture for 24 hours at room temperature is not a very effective method for a graded mixture. This curing condition, followed by the oven curing after molding, resulted in a final loss of volatile of 39 percent for the RC–4 mixture and of 24 percent for the MC–3 mixture. These losses were somewhat greater than those that occurred with no curing of the loose mixture, but the amount of curing offered by this method appears to be insufficient because it is believed that good construction procedure for medium-curing cut-back mixtures requires manipulation until the loss of volatile material has exceeded 40 or possibly 50 percent. Curing in an oven at 100° F. for 18 hours and at 140° F. for 4 hours, producing results with the RC–4 mixture that were almost identical, proved to be somewhat more effective in producing the desired amount of loss in volatile material. However, there is some doubt that the 35-percent loss of volatile matter, occurring in the MC–3 mixture under the 4-hour curing at 140° F., is of sufficient magnitude. Increasing the period of curing at 140° F. to 18 hours for this mixture increased the final loss of volatile material to 56 percent, which appears to be adequate.

Curing in an oven at 140° F. for 18 hours was found to be suitable for the RC–4 mixture in that a high amount of volatile material was removed without resulting in a mix too stiff for proper molding.

In tests made on an asphaltic-emulsion mixture it was found that this degree of curing was sufficient to remove all water and still provide a mixture not too difficult to handle. It appears, therefore, that an 18-hour period of curing at 140° F. is satisfactory for all cold mixtures.

Size of test specimens.—Cylindrical test specimens, 4 inches by 4 inches, have proven satisfactory. Specimens 4 inches in diameter by 8 inches in height have been used but these were difficult to handle, and the less stable specimens often failed in removing them from the water bath. Three- by three-inch cylindrical specimens have been used in some instances to conserve materials. Test specimens of this size were found satisfactory for the fine-grained types of mixtures but for the coarse mixtures the test results were not as uniform as for the larger specimens.

Molding temperatures.—Hot mixtures mold satisfactorily at 260° F. This is approximately the temperature of a hot mix after completion of mixing so that little or no delay need occur prior to molding in the case of these materials.

Table 3.—***Effect of vacuum and immersion temperature on test results for asphaltic concretes of same gradation*** [1]

Mix designation	¾ inch to No. 10 aggregate [2]	Air voids	Compressive strength, dry	Immersion temperature	After 4-day immersion—Compressive strength: No vacuum	After 4-day immersion—Compressive strength: Vacuum used	After 4-day immersion—Retained strength ratio: No vacuum	After 4-day immersion—Retained strength ratio: Vacuum used	Difference between percentages of retained strength ratios
Limestone dust mixtures:		*Percent*	*P. s. i.*	*°F.*	*P. s. i.*	*P. s. i.*	*Percent*	*Percent*	
11	Limestone	5.2	222	77	226	218	102	98	4
				100	213	200	96	90	6
				120	215	204	97	92	5
12	Trap	6.2	263	77	247	245	94	93	1
				100	237	216	90	82	8
				120	218	195	83	74	9
13	Granite	6.5	265	77	244	225	92	85	7
				100	215	178	81	67	14
				120	175	154	66	58	8
Silica dust mixtures:									
14	Limestone	5.8	218	77	211	203	97	93	4
				100	190	174	87	80	7
				120	161	129	74	59	15
15	Trap	6.9	251	77	251	241	100	96	4
				100	218	198	87	79	8
				120	136	80	54	32	22
16	Granite	7.2	272	77	241	220	89	81	8
				100	199	164	73	60	13
				120	113	103	42	38	4

[1] Gradation: passing ¾ inch sieve, 100 percent; passing No. 10 sieve, 21 percent; passing No. 200 sieve, 6 percent. Bitumen-aggregate ratio, 4.9 percent. Penetration of asphalt, 85–100.
[2] The sand fraction of the aggregate was river sand in each case.

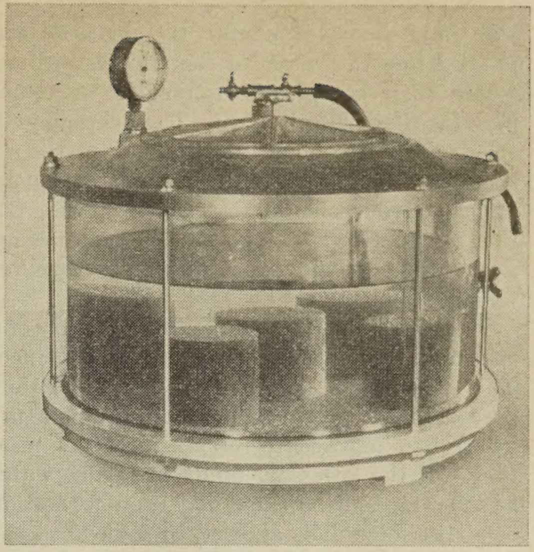

Figure 3.—Air bubbles on the specimens in the vacuum bath indicate the evacuation of voids.

The cold mixtures—those containing liquid bituminous materials—mold satisfactorily at 140° F., that is, at the temperature as removed from the curing oven. In this group the RC cut-back asphalts and the asphalt emulsions set the temperature for molding. While the MC cut-back asphalts and the SC liquid asphalts can be molded at lower temperature, the RC and emulsion mixtures require this higher temperature.

Molding load (double plunger method).—The double plunger method and a load of 3,000 pounds per square inch held for 2 minutes seem to be a satisfactory procedure for consolidation. Some crushing occurs with soft and fragile aggregate and it may be advisable to use a lower consolidating load for such conditions.

Loading the mold so as to prevent segregation of the aggregate particles is most important. It has been found that segregation may be largely eliminated by the exercise of care in loading and unloading the scoop used in filling the mold. The material should be spread into the loading scoop as uniformly as possible and then scraped into the mold, as illustrated in figure 1, rather than poured. After the mold is filled, the mixture should be leveled off on top and then spaded around the edge with a heated spatula to reduce the tendency for surface voids to form. Rodding is not required for it appears to be unnecessary.

Curing after molding.—After molding, curing at an elevated temperature seems to be desirable for both cold and hot mixtures. In the case of cold mixtures, it brings about more uniformity in the compressive strength by reducing the effect of variations in the degree of curing that occurs during laboratory storage. In the case of hot mixtures, although not losing an appreciable amount of volatile matter, they will increase in strength during storage even at room temperature.

Selection of specimens for test groups.—A small variation in density often results in a large variation in compressive strength. From tests on several specimens from one of the mixtures used in this study, it was found that a 1-percent change in density caused a 10-percent change in compressive strength.

The importance of having the densities approximately the same for the two test groups where their compressive strengths are compared is therefore obvious. The average bulk specific gravity should be obtained for the six specimens for a test, and the three specimens for each group should be selected so that the average specific gravity for each group is, as nearly as possible, equal to the average for all six specimens.

In determining the bulk specific gravity of a specimen three different weights are obtained. The specimen is first weighed in air and then placed in a water bath at room temperature where it is allowed to remain for a few minutes. If the specimen tends to absorb water at a fast rate, as indicated by a rapid formation of air bubbles, it is allowed to soak until the rate of absorption decreases to a small value. The specimen is then removed from the water and surface dried with a damp absorptive cloth, as illustrated on the cover of this issue. The specimen is next weighed in air and then in water. The difference between these two weights in grams is taken as the bulk volume of the specimen in cubic centimeters. Bulk specific gravity is calculated by dividing the initial weight of the unsoaked specimen in grams by the bulk volume of the specimen.

Table 4.—*Effect of vacuum on test results for asphalt*[1] *mixtures of various gradations*

Mix designation	Kinds of aggregate[2]: ¾ inch to No. 10 fraction	Kinds of aggregate[2]: Sand fraction	Gradation of aggregate: Maximum size	Gradation of aggregate: Amount passing— No. 10 sieve	Gradation of aggregate: Amount passing— No. 200 sieve	Bitumen-aggregate ratio	Air voids	Compressive strength, dry	After 4-day immersion at 120° F—: Compressive strength: No vacuum	After 4-day immersion at 120° F—: Compressive strength: Vacuum used	After 4-day immersion at 120° F—: Retained strength ratio: No vacuum	After 4-day immersion at 120° F—: Retained strength ratio: Vacuum used	Difference between percentages of retained strength ratios
				Percent	*Percent*	*Percent*	*Percent*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*Percent*	*Percent*	
11	Limestone	River sand	¾ in.	21	6	4.9	5.2	222	215	204	97	92	5
12	Trap	do	¾ in.	21	6	4.9	6.2	263	218	195	83	74	9
13	Granite	do	¾ in.	21	6	4.9	6.5	265	175	154	66	58	8
17	Granite	do	¾ in.	45	11	6.0	6.0	408	326	216	80	53	27
18	None	do	No. 10	100	15	12.0	8.2	322	228	116	71	36	35
19	None	Limestone	No. 10	100	22	7.0	6.3	518	429	224	83	43	40

[1] 85–100 penetration.
[2] The dust fraction was limestone dust in each case.

IMMERSION CONDITIONS

Considerable laboratory work was done to determine the most suitable immersion condition. Among the more important factors studied were the following:

1. Use of vacuum to bring about quick absorption of water into the specimen.
2. Temperature of the immersion storage bath.
3. Period of immersion.

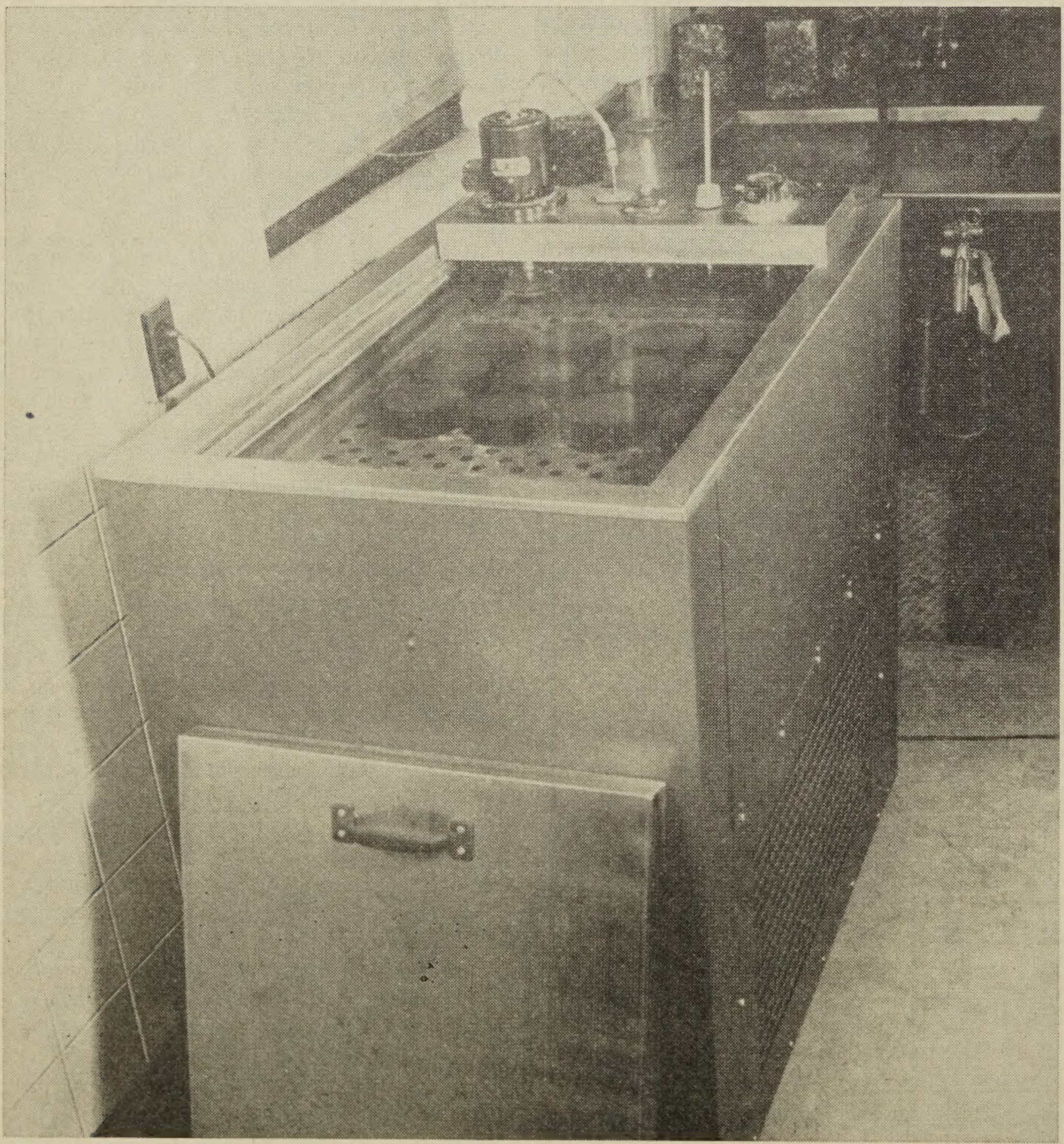

Figure 4.—The automatic water bath used in the immersion test.

Use of vacuum on hot mixtures found unsatisfactory.—Considerable work was done using vacuum as a preliminary treatment to produce saturation of the cylinders before immersion in the storage bath. The vacuum bath used in this investigation is shown in figure 3. In these tests the temperature of the water in the vacuum bath was 77° F. and the degree of vacuum was 27 inches of mercury. The specimens were kept in this bath until the air ceased to pass off from their surfaces.

The time required for this operation was about 20 minutes. Swell and strength tests were made immediately after saturation on a number of specimens covering several types of hot mixtures, and it was found that there was no swell or loss in strength. It was concluded, therefore, that the use of the vacuum had no initial detrimental effect on the specimens.

The effect of pre-saturation, by use of a vacuum, on the final results of several asphaltic mixtures of the hot type is given in tables 3 and 4. The six asphaltic concrete mixtures of the same gradation shown in table 3 were of the open-graded type that normally tend to absorb water quite readily. These mixtures were subjected to three different immersion temperatures. In all instances the use of a vacuum resulted in reducing the values of retained strength. Greater reductions occurred, in general, with the higher immersion temperatures. The reductions in percentage of retained strength averaged 4.7, 9.3, and 10.5 for immersion temperatures of 77°, 100°, and 120° F., respectively.

Table 4 shows the effect of using a vacuum on mixtures containing aggregate of various gradations. It includes three of the asphaltic concretes of the open-graded type given in table 3, an asphaltic concrete high in percentage of fine aggregate, a sheet asphalt, and an artificial sand mixture. All mixtures contained limestone dust as filler.

The latter three mixtures (Nos. 17, 18, and 19), containing large proportions of fine aggregate, were of types that under normal conditions absorb little or no water. These mixtures were not affected greatly when immersed at 120° F. without pre-saturation, but when subjected to the action of a vacuum prior to the immersion they were severely affected. The use of a vacuum on these mixtures resulted in decreasing the percentages of retained strength by values ranging from 27 to 40. These decreases are much greater than those ranging from 5 to 9 for the open-graded type mixtures Nos. 11, 12, and 13.

If the percentages of retained strength of the six mixtures are compared on the basis of the non-vacuum tests, it will be observed that the three mixtures high in fines (Nos. 17, 18, and 19) are not greatly inferior to the excellent open-graded limestone mixture (No. 11) and that they are somewhat better than the open-graded mixture which contains the granite (No. 13).

On the other hand, comparisons on the basis of tests in which a vacuum was used show that the three mixtures containing high percentages of fine aggregate are definitely inferior to the open-graded mixture containing limestone and somewhat inferior to the one containing granite. The sheet asphalt mixture (No. 18) and the all-limestone artificial sand mixture (No. 19) appear to be unsatisfactory for use as road materials. This does not seem to be in line with expected field behavior because fine-grained mixtures normally have a high resistance to the infiltration of moisture and for that reason usually show good service behavior.

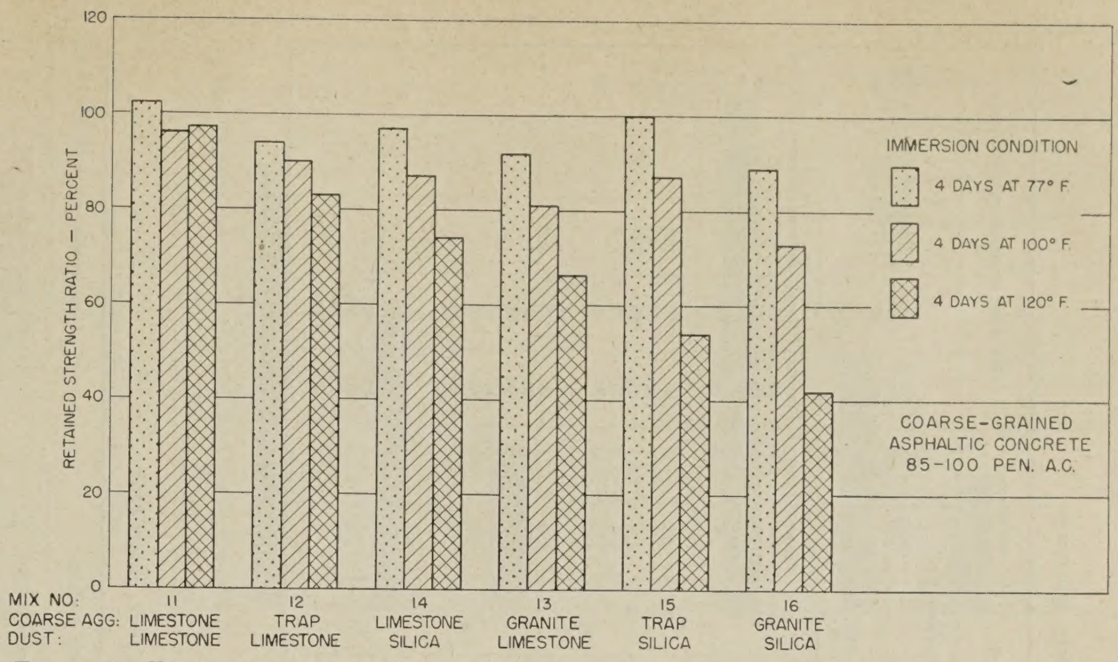

Figure 5.—Effect of immersion temperature on retained strength ratio of coarse-grained asphaltic concrete.

Hence, because of its failure to reflect expected service behavior, it must be concluded that the vacuum process does not belong in the test procedure.

Conditions of immersion for hot mixtures.—An extended study to determine the most suitable temperature and period of immersion for hot mixtures has been made. Temperatures of 77°, 100°, 120°, and 140° F., and periods of immersion of 1, 4, 14, and 35 days were investigated. Accurate control of temperatures was obtained by using the automatic water bath shown in figure 4.

In the early work 77° F. had been used, but it developed that this low temperature of bath did not show the significant differences between those mixtures that were known to have high resistance to the action of moisture and those known to have low resistance. The more recent study substantiates this conclusion and fully establishes that this temperature fails to differentiate between aggregates having satisfactory and those having unsatisfactory behavior in use. The effect of the various immersion temperatures on coarse-grained asphaltic concretes are shown in table 3 and figure 5.

It will be noted that a very narrow range in test values is obtained with an immersion temperature of 77° F. At 100° F. the results show a somewhat greater differentiation between the several mixtures, but do not provide as positive a one as is desirable. At 120° F., however, the strength retention of the mixtures containing aggregates that are known to be superior is consistently higher than that of the inferior mixtures.

Results of immersion tests at 140° F. for 1 day and at 120° F. for 4 days are compared in table 5. These results show a very close agreement between the test values for the two immersion conditions. It is believed, therefore, that where quick tentative results are

Table 5.—*Comparison of test results for 1-day immersion at 140° F. with those for 4-day immersion at 120° F. for asphaltic concrete mixtures*

Type of asphalt and mix designation	Coarse aggregate in mixture[1]	Gradation of aggregate:[2] Amount passing—		Bitumen-aggregate ratio	Air voids	Compressive strength, dry	Effect of immersion on—					
							Compressive strength		Volumetric swell		Retained strength ratio	
		No. 10 sieve	No. 200 sieve				1 day at 140° F.	4 days at 120° F.	1 day at 140° F.	4 days at 120° F.	1 day at 140° F.	4 days at 120° F.
50-60 penetration:		*Percent*	*Percent*	*Percent*	*Percent*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*Percent*	*Percent*	*Percent*	*Percent*
20	Limestone	21	6	4.9	4.1	399	354	332	0.6	0.4	89	83
21	Trap	21	6	4.9	5.6	478	352	354	.8	.9	74	74
22	Granite	45	11	6.0	5.7	679	536	529	.5	.7	79	78
23	do	21	6	4.9	6.5	472	292	298	1.3	1.3	62	63
85-100 penetration:												
11	Limestone	21	6	4.9	5.2	222	211	215	.3	0	95	97
12	Trap	21	6	4.9	6.2	263	216	218	.8	.2	82	83
17	Granite	45	11	6.0	6.0	408	307	326	.7	.8	75	80
13	do	21	6	4.9	6.5	265	187	175	1.1	1.0	71	66
120-150 penetration:												
24	Limestone	21	6	4.9	4.0	187	180	175	.1	0	96	94
25	Trap	21	6	4.9	5.3	234	197	195	.4	.5	84	83
26	Granite	45	11	6.0	5.7	342	280	279	.6	.5	82	82
27	do	21	6	4.9	5.9	224	176	169	.8	1.0	79	75

[1] Retained on No. 10 sieve. Aggregate passing No. 10 sieve was river sand and limestone dust.
[2] Maximum size, ¾ inch.

Table 6.—*Effect of period of immersion on test results for hot mixtures* [1]

Mix designation	¾ inch to No. 10 aggregate[2]	Gradation of aggregate			Bitumen-aggregate ratio	Air voids	Compressive strength, dry	Retained strength ratio after immersion at 120° F. for—			
		Maximum size	Amount passing—					1 day	4 days	14 days	35 days
			No. 10 sieve	No. 200 sieve							
Limestone dust mixtures:			*Percent*	*Percent*	*Percent*	*Percent*	*P. s. i.*	*Percent*	*Percent*	*Percent*	*Percent*
11	Limestone	¾ in.	21	6	4.9	5.2	222	95	97	----	----
12	Trap	¾ in.	21	6	4.9	6.2	263	94	83	----	----
17	Granite	¾ in.	45	11	6.0	6.0	408	85	80	71	63
18		No. 10	100	15	12.0	8.2	322	----	71	66	59
13	Granite	¾ in.	21	6	4.9	6.5	265	83	66	54	47
Silica dust mixtures:											
14	Limestone	¾ in.	21	2	4.9	5.8	218	89	74	----	----
15	Trap	¾ in.	21	6	4.9	6.9	251	87	54	----	----
16	Granite	¾ in.	21	6	4.9	7.2	272	68	42	----	----

[1] 85-100 penetration asphalt cement.
[2] The sand fraction of the aggregate was river sand in each case.

Table 7.—*Effect of immersion temperature on test results for liquid asphaltic mixtures*

Mix designation	Grade of bituminous material	Kinds of aggregate [1]: Retained on No. 10 sieve	Kinds of aggregate [1]: Dust fraction	Gradation of aggregate: Maximum size	Gradation of aggregate: Amount passing— No. 10 sieve	Gradation of aggregate: Amount passing— No. 200 sieve	Bitumen-aggregate ratio	Air voids	Compressive strength: Dry	Compressive strength: After 4-day immersion at— 77° F.	Compressive strength: After 4-day immersion at— 100° F.	Compressive strength: After 4-day immersion at— 120° F.	Volumetric swell after— 4 days at 77° F.	Volumetric swell after— 4 days at 100° F.	Retained strength ratio after— 4 days at 77° F.	Retained strength ratio after— 4 days at 100° F.	Retained strength ratio after— 4 days at 120° F.	Retained strength ratio after— 1 day at 100° F.	Retained strength ratio after— 1 day at 120° F.
				Inch	*Percent*	*Percent*	*Percent*	*Percent*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
RC cut-back:																			
28	RC-4	Limestone	Limestone	¾	21	6	4.9	6.1	166	151	134	124	0.3	0.8	91	81	75	91	86
29	RC-4	Trap	do	¾	21	6	4.9	6.3	215	172	135	108	.3	1.0	80	63	50	80	68
30	RC-4	Granite	do	¾	21	6	4.9	6.9	230	150	94	67	1.2	2.6	65	41	29	56	39
31	RC-4	Trap	Silica	¾	21	6	4.9	6.7	204	162	47	0	.6	5.2	79	23	0	----	----
MC cut-back:																			
32	MC-3	Limestone	Limestone	¾	21	6	4.9	6.6	84	79	78	75	.3	.3	94	93	89	----	----
33	MC-3	Trap	do	¾	21	6	4.9	6.5	104	83	61	45	.4	1.2	80	59	43	----	----
34	MC-3	Granite	do	¾	21	6	4.9	7.1	128	54	37	30	1.8	2.7	42	29	23	46	30
35	MC-3	Trap	Silica	¾	21	6	4.9	6.9	104	53	2	0	1.6	9.2	51	2	0	----	----
36	MC-3	Gravel	do	½	60	10	8.0	6.9	75	59	12	----	1.0	6.6	79	16	----	67	29
SC asphalt (high loss [2]):																			
37	SC-2	do	Limestone	½	60	10	8.0	4.8	32.9	29.6	25.5	----	.8	1.4	90	78	----	----	----
38	SC-3	do	do	½	60	10	8.0	5.0	49.6	38.8	32.6	----	.9	1.6	78	66	----	----	----
39	SC-4	do	do	½	60	10	8.0	4.6	78.9	63.3	44.6	----	.9	1.7	80	57	----	----	----
40	SC-2	do	Silica	½	60	10	8.0	5.6	28.4	27.5	15.7	----	.5	2.5	97	55	----	----	----
41	SC-3	do	do	½	60	10	8.0	6.4	33.3	29.2	20.9	----	.7	2.2	88	63	----	----	----
42	SC-4	do	do	½	60	10	8.0	5.4	66.0	54.9	32.2	----	.7	2.2	83	49	----	----	----
SC asphalt (low loss [3]):																			
43	SC-2	do	Limestone	½	60	10	8.0	4.9	26.1	23.2	22.3	----	.8	1.2	89	85	----	----	----
44	SC-3	do	do	½	60	10	8.0	4.9	38.8	34.2	27.3	----	.7	1.4	88	70	----	----	----
45	SC-4	do	do	½	60	10	8.0	4.7	71.4	57.9	41.6	----	.7	1.4	81	58	----	----	----
46	SC-2	do	Silica	½	60	10	8.0	6.0	21.1	18.9	11.3	----	.6	2.8	90	54	----	----	----
47	SC-3	do	do	½	60	10	8.0	6.7	26.9	24.1	13.9	----	.8	3.2	90	52	----	----	----
48	SC-4	do	do	½	60	10	8.0	5.2	60.7	48.7	24.7	----	.7	2.8	80	41	----	----	----

[1] The sand fraction of the aggregate was river sand in each case.
[2] Prepared in laboratory by blending a 279-penetration asphalt with a high-loss gas oil.
[3] Prepared in laboratory by blending a 279-penetration asphalt with a low-loss gas oil.

Table 8.—*Results of alternate drying and wetting tests on MC-2 mixtures (see text, page 116)*

Coarse aggregate,[1] and mix designation	Additive in cut-back	Compressive strength: Dry	Compressive strength: 4-day immersion at 77° F.	Compressive strength: 4-day immersion at 77° F. followed by alternate drying and wetting [4]: 5 cycles, 74 days	Compressive strength: 10 cycles, 144 days	Compressive strength: 20 cycles, 284 days	Compressive strength: 10-month exposure to weather [5]	Volumetric swell: 4-day immersion at 77° F.	Volumetric swell: 4-day immersion at 77° F. followed by alternate drying and wetting: 5 cycles, 74 days	Volumetric swell: 10 cycles, 144 days	Volumetric swell: 20 cycles, 284 days	Volumetric swell: 10-month exposure to weather	Final moisture content: 4-day immersion at 77° F.	Final moisture content: 4-day immersion at 77° F. followed by alternate drying and wetting: 5 cycles, 74 days	Final moisture content: 10 cycles, 144 days	Final moisture content: 20 cycles, 284 days	Final moisture content: 10-month exposure to weather	Retained strength [2] ratio	Compressive strength ratio [3]: 4-day immersion at 77° F. followed by alternate drying and wetting: 5 cycles, 74 days	Compressive strength ratio [3]: 10 cycles, 144 days	Compressive strength ratio [3]: 20 cycles, 284 days	Compressive strength ratio [3]: 10-month exposure to weather
		P. s. i.	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Pit gravel mix:																						
64	None	43	33	53	68	88	76	0.4	0.7	1.1	1.0	0.5	1.6	2.1	2.5	2.5	1.9	77	123	158	205	177
65	*A*	45	36	52	66	80	75	0	.3	.4	.4	.3	1.4	1.8	2.1	2.5	1.9	80	116	147	178	167
66	*D*	38	36	53	70	84	81	.3	1.2	1.4	1.4	.4	1.5	2.3	2.3	2.4	2.1	95	139	184	221	213
67	*B*	39	39	76	93	116	86	0	.1	.2	.2	.1	1.2	1.8	2.2	2.1	1.5	100	195	238	297	221
Granite mix:																						
68	None	64	35	41	44	67	60	1.3	2.9	3.0	2.6	1.2	1.5	2.6	2.7	2.4	2.1	55	64	69	105	94
69	*A*	61	50	67	74	93	83	.1	.3	.4	.4	.3	1.0	1.4	1.7	1.9	1.5	82	110	121	152	136
70	*D*	61	57	80	93	118	92	.5	1.3	1.2	1.1	.6	1.2	1.7	1.8	1.8	1.6	93	131	152	193	151
71	*B*	61	59	101	112	141	100	.1	.3	.4	.5	.4	1.0	1.4	1.5	1.6	1.3	97	166	184	231	164

[1] The sand fraction was river sand and the dust fraction silica in each case. Gradation of the aggregate was as follows: Passing ½ inch sieve, 100 percent; passing No. 10 sieve, 40 percent; passing No. 200 sieve, 10 percent. Bitumen-aggregate ratios: 6.0 percent for the pit gravel; 5.7 percent for the granite mixture.
[2] 4-day immersion at 77° F.
[3] Ratio of final strength to the original strength dry.
[4] A cycle of alternate drying and wetting consisted of 10 days' drying in laboratory air and 4 days' immersion in water at 77° F.
[5] The test cylinders were buried outdoors in a sand bed with top surfaces exposed. Before testing, they were immersed in water at 77° F. for 4 days.

desired, the results of tests made after 1-day immersion at 140° F. may be used with confidence.

Table 6 presents data on the effect of various periods of immersion at a temperature of 120° F. The 1-day results at 120° F. were approximately the same as the 4-day results at 100° F. (given in table 3 except for mixes Nos. 17 and 18) and failed to provide a sufficiently sharp evaluation. On three of the mixtures, the period of immersion was extended to 14 and 35 days in order to determine the reliability of results obtained in the shorter and more practical periods. The results of these tests indicate that, while some additional loss occurs, the immersions for 14 and 35 days show the same order of resistance by the various mixtures to water action as immersion for 4 days.

Conditions of immersion for cold mixtures.—A study was made to determine suitable immersion conditions for the liquid asphaltic mixtures. In table 7 are shown the data obtained on RC–4 and MC–3 mixtures with an immersion period of 4 days at temperatures of 77°, 100°, and 120° F. Also included are percentages of retained strength for a 1-day immersion period at temperatures of 100° and 120° F. The 4-day immersion results are shown in figure 6. If the results for mixtures Nos. 31 and 35 were not considered, it would appear that both temperatures 77° and 100° F. produce test values, for a 4-day immersion period, that permit satisfactory differentiation between the several mixtures. Mixtures Nos. 31 and 35 contained silica dust as a filler and the 4-day results at 100° F., for these two mixtures, are believed to be more nearly in agreement with expected field behavior than those in which the immersion temperature was 77° F. The results for a 4-day immersion period at 120° F. indicate that this condition is too severe, especially for mixtures containing silica dust. Specimens from mixtures Nos. 31 and 35 disintegrated in less than 4 days at this high temperature.

Several of the cut-back mixtures were tested after being immersed for a 1-day period at temperatures of 100° and 120° F. The results for the 1-day test at 100° F. compare very favorably with those for the 4-day test at 77° F. Also, there is very little difference between the results for a 1-day period at 120° F. and a 4-day period at 100° F. It therefore appears that when there is need for early test results, a 1-day immersion at 120° F. may be used for the cut-back mixtures.

Table 7 also gives data for mixtures containing SC–2, 3, and 4 materials. Both the slow and the more rapid curing SC materials were used in this study. Analyses of these materials are given in table 9. Again, as occurred with the cut-back mixtures, the use of silica dust as a filler resulted in mixtures which were affected to a much greater degree by the change in immersion temperature from 77° to 100° F. than were the mixtures in which limestone dust was used. Also, as with the cut-back mixtures, the results after 4-day immersion at 100° F. appear to conform more nearly to expected service behavior than do the results at the lower temperature.

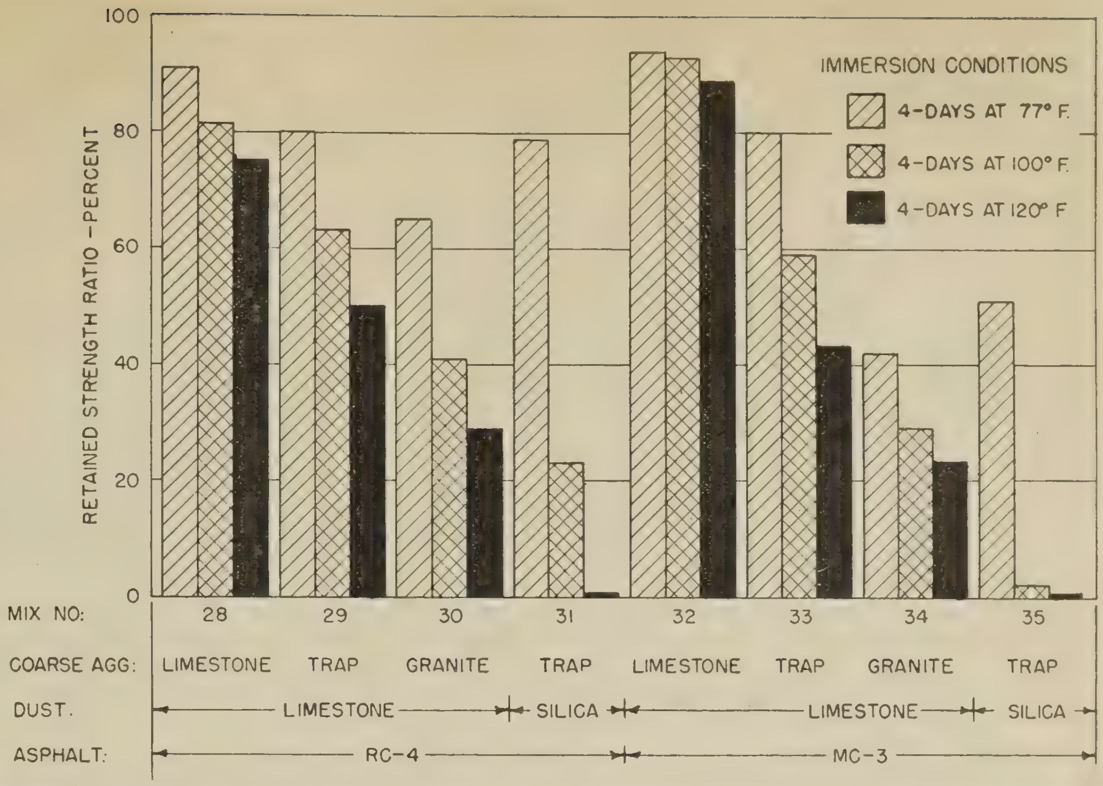

Figure 6.—Effect of immersion temperature on retained strength ratio of cut-back asphaltic mixtures.

Because of an insufficient quantity of these SC materials, specimens were not prepared for the 1-day immersion period. However, in view of the good correlations obtained for the other bituminous materials, it can be expected that the results after a 1-day immersion period at 120° F. would agree very closely with those after a 4-day immersion period at 100° F. and that it would be permissible to substitute this shorter period of test when necessary.

A limited amount of work with asphaltic-emulsion mixtures was included in this study to determine if any changes in procedure were necessary for handling this type of cold mixture. It was found that the same curing procedure used for the cut-back and slow-curing bituminous materials was also satisfactory for the emulsion mixtures. However, since the 18 hours of curing at 140° F. removed all the moisture from the mixtures, leaving a penetration grade of asphalt, it appeared desirable to make the immersion condition for this type of cold mix the same as for hot mixes; that is, 4 days at 120° F.

COMPRESSIVE TEST CONDITIONS

In bringing the dry specimens to the test temperature of 77° F., it was found that 2 hours in an air bath at this temperature was sufficient when the specimens, after curing, were permitted to cool overnight at room temperature. Specimens placed in the air bath directly from the oven require about 4 hours to bring them to the test temperature. Because of the higher conductivity of water, the immersed specimens required only 1 hour in water at 77° F. to bring them to the proper temperature for testing.

It is believed that the rate of deformation should vary with the height of the test specimen. A rate of 0.05 inch per minute per inch of height has been found to be suitable. This gives a rate of 0.2 inch per minute for the standard 4-inch specimen. A few tests have been made on 3- by 3-inch cylinders, and in these tests the rate was 0.15 inch per minute.

SOME APPLICATIONS OF THE TEST

The following summary describes some of the more important results that have been obtained with the immersion-compression test, and the nature of the problems that may be studied to advantage by this method of test.

From the work done so far, information has been obtained on a number of factors that affect the behavior of bituminous mixtures. In certain instances the results are conclusive while for some factors more work needs to be done.

Table 9.—*Analyses of slow-curing liquid asphaltic materials*

	Grade					
	SC–2	SC–2	SC–3	SC–3	SC–4	SC–4
Relative volatile content	High	Low	High	Low	High	Low
Furol viscosity at 140° F. .... seconds	170	145	373	412	------	------
Furol viscosity at 180° F. .... do	------	------	------	------	178	205
Flash point .... ° F.	255	360	270	365	280	380
Loss at 325° F., 50 gm., 5 hrs .... percent	8.8	2.6	7.1	2.1	5.3	1.9
Float test on residue at 122° F .... seconds	38	30	58	46	84	71
Asphalt content of 100 penetration .... percent	71	67	77	75	81	80
Ductility of residue at 77° F .... cm	165	165	195	200+	185	200+

Table 10.—*Results of a special filler investigation of 85–100 penetration asphaltic concretes*

Mix designation	Aggregate passing No. 200 sieve [1]	Bitumen-aggregate ratio	Air voids	Compressive strength, dry	After 4-day immersion at 120° F.	
					Compressive strength	Retained strength ratio
		Percent	*Percent*	*P. s. i.*	*P. s. i.*	*Percent*
50	Fly ash	6.5	3.6	212	212	100
51	do	5.5	6.5	224	226	101
52	Limestone dust	6.5	5.5	259	242	93
53	do	5.5	8.2	256	225	88
54	Trap-rock dust	6.5	5.7	264	246	93
55	do	5.5	8.5	245	227	93
56	Silica dust No. 2	6.5	3.9	251	213	85
57	do	5.5	6.9	281	189	67
58	Silicia dust [2]	6.5	4.5	252	198	79
59	do	5.5	8.4	261	144	55

[1] Aggregate retained on No. 200 sieve, trap rock and siliceous sand. Gradation: Passing ½ inch sieve, 100 percent; passing No. 10 sieve, 37 percent; passing No. 200 sieve, 7 percent.
[2] Same silica dust used as in the mixtures of tables 3 and 6.

Table 11.—*Effect of additives when used with rapid-curing cut-back bituminous mixtures* [1]

Mix designation	Grade of cut-back material	Additive used with—		Air voids	Compressive strength, dry	After 4-day immersion at 120° F.[2]			
		Bituminous material	Aggregate			Compressive strength, wet	Water absorbed	Volumetric swell	Retained strength ratio
				Percent	*P. s. i.*	*P. s. i.*	*Percent*	*Percent*	*Percent*
60	RC–2	None	None	11.5	184	19	9.6	10.2	10
61	do	None	*T*	10.2	201	159	4.3	1.1	79
62	do	*B*	None	11.4	176	106	5.9	2.7	60
63	RC–3	*F*	None	11.9	238	107	6.9	3.7	45

[1] Chert gravel mixtures using a bitumen-aggregate ratio of 9.3 percent.
[2] Normally a temperature of 100° F. is used for evaluating cut-back mixtures.

Results demonstrate the importance of the aggregates on the behavior of the bituminous mixtures. There is a wide range in the film-retention properties of the aggregates commonly used in bituminous road construction. This is not only true as between the general types but there is a wide range within a particular aggregate type. Limestone, in general, has high resistance to the stripping action of water, yet there may be a considerable range in quality between limestones from different sources because of impurities such as silica, clay, or iron. Similar differences in the quality of trap rock have been found, although, in general, this type of stone possesses high film-retention properties. Granite and quartzite are other types of stone that have been tested. As would be expected, granites range from good to bad and quartzite was found to be the least satisfactory of aggregates studied.

Absorption is a property of the aggregate causing increased adhesion of the bituminous film. Stone having high absorption, such as caliche (as found in New Mexico), soft limestone, and limerock, although they are hard to coat, retain the bituminous film very well.

The quality and quantity of the sand and filler used are shown to have considerable effect on the resistance of mixtures to the action of moisture. In many cases these materials may be more important than the coarser fractions. A high percentage of fines of good quality may often be used to overcome the adverse effect of a less satisfactory coarse aggregate.

Test results given in table 3 show the comparative effect of limestone dust and silica dust when used in asphaltic concrete mixtures. For the three conditions, covering coarse aggregates of limestone, trap, and granite, the substitution of silica dust for limestone dust as a filler resulted in reducing the percentage of retained strength (4-day immersion at 120° F.) by 23 or more. The silica dust used in these tests was analyzed and found to be 99.7 percent pure SiO_2. It is possible that this silica dust would have been more satisfactory if it had contained more impurities.

In table 10 are shown the results of tests on a series of bituminous concrete mixtures in which the variables were the type and the quantity of filler material. The superiority of the fly-ash, limestone, and trap-rock dust fillers over both of the two silica-dust fillers is clearly shown. Increasing the bitumen content is shown to be beneficial for the mixtures composed of the less satisfactory materials but to have little effect on the mixtures composed of the better materials.

The effectiveness of certain chemical additives on the adhesion of bituminous films to aggregates has been studied by means of this test. In this work, the test has proven effective in showing the comparative value of these additives and the amounts required.

Table 11 gives the results with and without the use of chemical additives on bituminous mixtures containing a chert gravel of unsatisfactory quality. An increase in retained strength together with a great reduction in swell is shown to result from use of these additives.

In table 8 (on page 114) are shown the results of alternate drying and wetting tests on two groups of bituminous mixtures containing treated and untreated MC–2 cut-back asphalts. Three anti-stripping additives were used as treatments, in concentrations of 2 percent for additive *A* and 1 percent for additives *B* and *D*. The coarse aggregate consisted of a pit gravel for the first group of mixtures and a granite for the second group. The table gives data for various periods up to 284 days during which the specimens, after having been initially immersed for 4 days, were subjected to repeated cycles of alternate drying and wetting. Each cycle consisted of a 10-day period of drying in laboratory air followed by a 4-day period of immersion in water at 77° F. Also included in table 8 are the results of tests made on specimens that had been buried outdoors with top surfaces exposed to the weather for a 10-month period. In addition to the above tests, compressive strength data were obtained for various periods of dry storage in laboratory air up to 284 days. These data are given in table 12. The compressive strength results for all conditions of storage and water exposure are summarized in the form of bar diagrams in figure 7.

A study of the data shows that additive *A*, when used in the cut-back for the gravel mixture, provided no improvement over the untreated cut-back. However, when used in the granite mixture this additive resulted in compressive strengths which, for all conditions of wet storage, were considerably higher than the corresponding strengths for the mixture containing no additive. Additive *D* did pro-

Table 12.—*Effect of length of storage in laboratory air on compressive strength of MC–2 mixtures*

Coarse aggregate,[1] and mix designation	Additive	Compressive strength after storage in air at room temperature				Compressive strength ratio [2] after storage in air at room temperature			
		0 days	74 days	144 days	284 days	0 days	74 days	144 days	284 days
Pit gravel mix:		*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*Percent*	*Percent*	*Percent*	*Percent*
64	None	43	82	93	121	100	191	216	281
65	*A*	45	78	92	126	100	173	204	280
66	*D*	38	82	92	116	100	216	242	305
67	*B*	39	80	91	115	100	205	233	295
Granite mix:									
68	None	64	98	115	140	100	153	180	219
69	*A*	61	91	105	129	100	149	172	211
70	*D*	61	95	114	134	100	156	187	220
71	*B*	61	95	111	134	100	156	182	220

[1] The sand fraction was river sand and the dust fraction was silica in each case.
[2] Ratio of final strength to original strength.

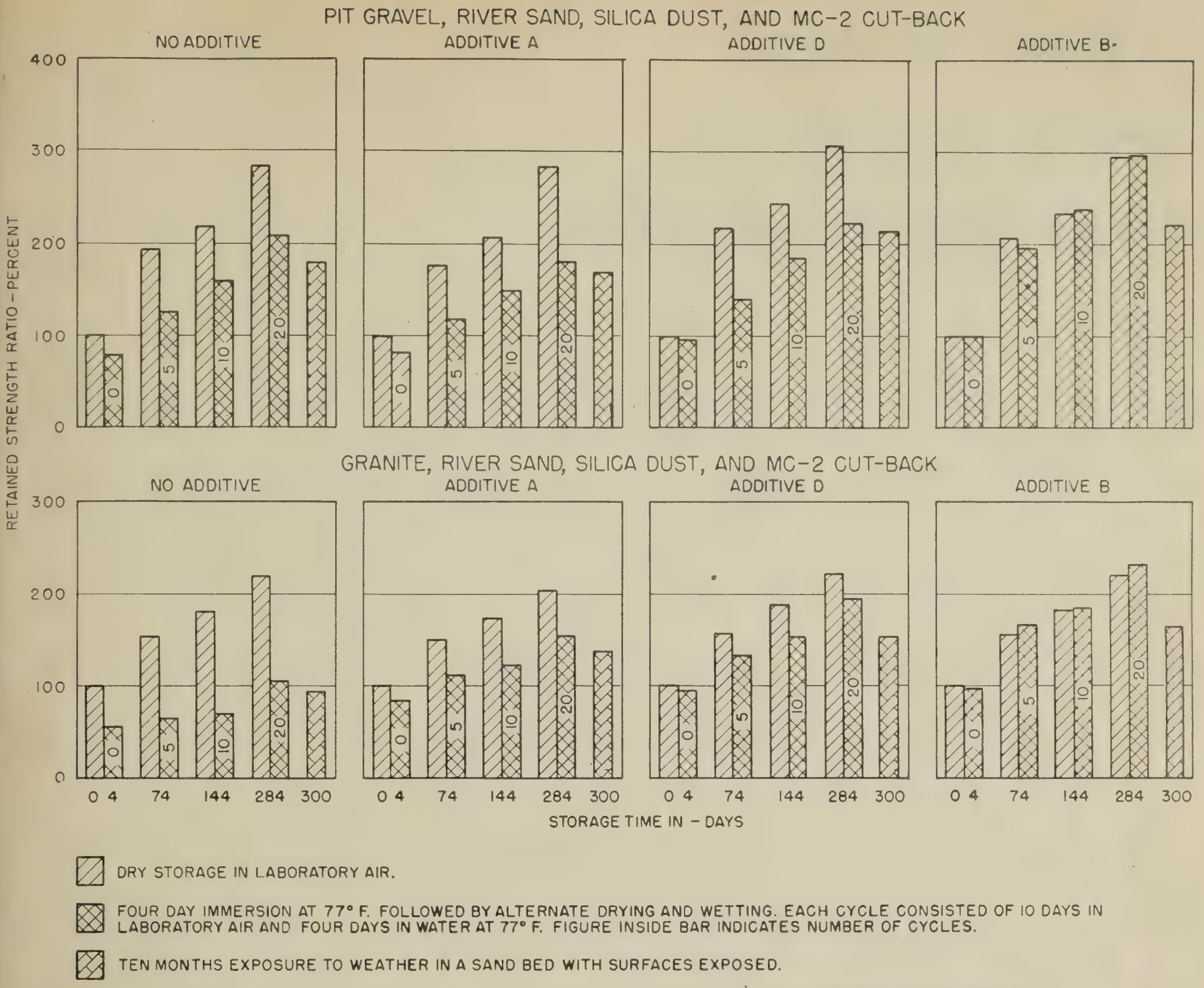

Figure 7.—Effect of additives on retained strength ratio in exposure tests over long periods of time.

vide improvement in the gravel mixture and was somewhat more effective than additive *A* in the granite mixture. Additive *B* was very effective for both mixtures. In the mixtures where this additive was used, the compressive strengths after the various periods of alternate drying and wetting were almost equal to or were greater than the compressive strengths for corresponding periods of dry storage. Considering the extended period over which these tests were made, the beneficial effect of certain additives, when used with medium-curing cut-back asphalt, would appear to be fairly permanent.

Table 13.—*Results of tests on mixtures* [1] ***containing asphalts from different sources***

Asphalts: Source	Asphalts: Penetration	Asphalts: Reaction to Oliensis test	Immersion-compression test: Mix designation	Air voids	Compressive strength, dry	After 4-day immersion at 120° F.: Compressive strength	Retained strength ratio	Volumetric swell
				Percent	*P. s. i.*	*P. s. i.*	*Percent*	*Percent*
A	89	Positive	72	7.0	325	246	76	0.5
B	93	Negative	13	6.5	265	175	66	1.0
C	78	do	73	6.5	287	153	53	2.0
D	85	do	74	6.5	297	148	50	2.1

[1] Mixtures contained granite, river sand, and limestone dust. Gradation: Passing ¾-inch sieve, 100 percent; passing No. 10 sieve, 21 percent; passing No. 200 sieve, 6 percent. Bitumen-aggregate ratio, 4.9 percent.

In connection with figure 7, it is of interest to note that during dry storage the pit-gravel mixtures increased in strength at a much greater rate than the granite mixtures. The pit gravel had an absorption of 1.7 percent as compared with an absorption of only 0.6 percent for the granite. Since absorption of bituminous material is gradual and continues over a long period of time, it is believed that the difference in absorption between the two aggregates is largely responsible for the difference in the rate at which their mixtures increase in strength.

The use of inferior aggregates is not necessarily the cause of all stripping failures. Many of the failures are probably caused by asphalts having inferior adhesive qualities. This is indicated by the data in table 13. The results given in this table show that asphalts of the same grade from different sources may vary widely in their resistance to the action of moisture.

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A Rapid Method of Testing Materials for Alkali-Aggregate Reaction

BY THE DIVISION OF PHYSICAL RESEARCH PUBLIC ROADS ADMINISTRATION

Reported by

D. O. WOOLF, Senior Materials Engineer and T. R. SMITH, Senior Scientific Aid

The generally accepted test method for determining the susceptibility of aggregates to reaction with the alkali in cement requires many months of observation and involves a number of difficulties. Accelerated tests heretofore proposed are found lacking in both simplicity and reliability.

The authors have developed a simple test method considered suitable for detection of reactive materials in a period of 1 month or less. Mortar or concrete, made with the aggregate under test and a high-alkali cement, is sealed in a pint mason jar, which is stored in an inverted position. The jar is examined daily until multiple cracking of the glass occurs, or until a reasonable age is reached without cracking. The method is suitable for field use since the test is simple and of relatively short duration, and the equipment is inexpensive and readily obtained.

THE GENERALLY accepted method of testing aggregate for reaction with the alkali in portland cement follows suggestions made by T. E. Stanton in 1941.[1] In this method, the aggregate is crushed, if necessary, to sand size, and used with high- or low-alkali cement to prepare a 1:2 mortar. The mortar is cast into bars about 11¼ inches long with a 1- or 2-inch square cross section. These bars are cast with gage plugs positioned in the ends to give a gage length of 10 inches in the same manner as in the cement bars for the standard method of test for autoclave expansion of portland cement, A. S. T. M. Method C 151.[2] The bars are measured for length, using a comparator reading to 0.001 inch or less, and stored in moist air. At periodic intervals, the bars are again measured to determine the amount of expansion, if any. With extremely reactive materials, a decision regarding the use of the material with a given type of cement may be made in a few months, but usually the tests must be continued for one or more years.

This method of test has several disadvantages. Possibly the most serious is the time usually required to test a material other than one which is extremely reactive with the alkali. When this time exceeds a month, as it usually does, the method becomes unsuitable for the routine or acceptance testing of aggregates. When samples of coarse aggregate are prepared for test by crushing the material to sand size, all of the crushed material customarily is used in preparing the mortar. In early studies of alkali-reactive aggregates, it was found that the size of the reactive aggregate had a marked effect on the amount of expansion. If the reactive aggregate was crushed fine enough, the mortar would develop little if any expansion. This same behavior may be found in tests of crushed aggregate when finely crushed fragments are included in the sample tested. Other disadvantages of the method include the fragility of the bars, especially when the smaller bar is used, and the difficulty of maintaining a saturated atmosphere for storing the bars. A method frequently used for storing the bars consists of placing the bars above a pool of water in a metal container, and sealing the cover. With a long storage period between consecutive inspections of the bars, and the use of galvanized containers, the loss of water and subsequent drying of the specimens because of the development of pin holes in the container has frequently been reported.

In an attempt to accelerate the reaction between the alkali in the cement and the aggregate, the addition to the mortar of 1 percent of sodium hydroxide, by weight of the cement, has been tried. In some instances mortar so treated has been found to have as much expansion at an age of 3 months as plain mortar developed in a year. Unfortunately, for some reason which has not been determined, a satisfactory correlation between the two methods has not been established. Too frequently, a treated mortar prepared with a high-alkali cement will show an excessive amount of expansion at an early age but plain mortar prepared with the same cement will develop only a small amount of expansion at considerably greater ages. In contrast to this, repeated tests have shown that when a low-alkali cement is used with a reactive aggregate, the addition to the mortar of 1 percent of sodium hydroxide, by weight of the cement, does not change materially the amount of expansion developed by the mortar. In spite of the lack of agreement between the results obtained with the long-time and accelerated tests, the use of mortar treated with sodium hydroxide has been found of value in furnishing at an age of 3 months information as to whether the material under test is unlikely to develop excessive expansion at some greater age when tested in plain mortar. If the treated mortar prepared with high-alkali cement fails to show excessive expansion at an age of 3 months, the aggregate used may be considered to be relatively nonreactive. However, if the mortar shows high expansion in the accelerated test, no definite conclusion can be drawn.

In further search for an accelerated test for the alkali-aggregate reaction, study was given to a method proposed by Alderman, Gaskin, and Vivian.[3] This method involves the warping of a slab of mortar due to differential expansion between the two mortars with which the specimen is prepared. Both mortars are prepared with the same cement. In one mortar, a nonreactive sand is used, and the aggregate under test is used in the second mortar. One mortar is used to form a layer 2- by 4- by ⅛-inch thick, and a layer of equal thickness of the second mortar is placed on and bonded to the first layer as rapidly as possible. When the specimen is removed from the mold at an age of 24 hours, it is placed in moist storage. If the mortar in one layer expands more than the other due to the chemical reaction, the slab will warp. The warping is stated to be preceded by the appearance of damp-looking spots on the surface of the reactive mortar, but the development of the spots is not always followed by warping. In the Australian tests, the specimens were stored at room temperature (58° to 59° F.) and at 110° F. Those prepared with cements containing 0.41 to 1.09 percent alkali, expressed as sodium oxide, and a very reactive aggregate, and stored at the higher temperature, developed prominent spots in 2 to 8 days, and prominent warping in 3 to 37 days. Bar specimens made with these same cements and stored at room temperature showed from 0.33 to 0.68 percent expansion at an age of 12 months.

A representative number of samples of both cements and aggregates were tested with the Australian slab method by the Public Roads Administration. Some of the aggregates for

[1] *Expansion of concrete through reaction between cement and aggregate (Discussion)*, by T. E. Stanton; Proceedings of the American Society of Civil Engineers, vol. 67, p. 1402, September 1941.

[2] *Standard method of test for autoclave expansion of portland cement*, 1946 Book of A. S. T. M. Standards, part II, p. 14.

[3] *A qualitative test for cement-aggregate reaction*, by A. R. Alderman, A. J. Gaskin, and H. E. Vivian; Journal of the Council for Scientific and Industrial Research, Commonwealth of Australia, vol. 18, No. 4, p. 433, Nov. 1945.

these tests were crushed to sand size and all were sieved to pass a No. 30 sieve and be retained on a No. 50 sieve. Ottawa sand graded to meet the requirements given in A. S. T. M. Method C 109 [4] was used as the inert aggregate. This material is graded between the No. 30 and No. 100 sieves. At first, a mold for a single 2- by 4-inch slab was used, as recommended by the originators of the method. The time required to prepare and clean these single molds was considered excessive, and new molds to prepare a slab 2 by 12 inches were made. After the mold had been filled, the specimen was cut into three pieces of equal length. Storage of the test specimens presented a problem, which was finally solved by the use of pint mason jars. The specimen was supported above a small pool of water in the jar by a block of brass or mortar, and the jars stored at 70° to 80° F., or at 120° F.

AUSTRALIAN METHOD TESTED

In the original report the authors state that the "bi-aggregate slabs have not failed to indicate expansive reaction in mortars which at atmospheric temperatures have developed significant expansions in twelve months," and that "in no instances have bi-aggregate slabs given indications of expansive reaction which were not fully substantiated by other observations." The mention of "atmospheric temperatures" is understood to refer to the room temperatures used in the Australian tests. The tests made by the Public Roads Administration do not support these statements fully. Many instances have been found where reactive mortars, as determined by other tests, have failed to develop the spotting or warping which are stated to be indicative in the bi-aggregate slab test. In other cases, very severe spotting has developed on slabs prepared with a mortar which, to an age of 3 years, has shown only an immaterial amount of expansion in bar tests. These slabs did not warp, and the question arises whether the spotting itself may be considered to indicate the alkali-aggregate reaction. When the indication of the reaction was limited to marked warping of the slab, the test was found to be no more rapid than the accelerated bar test, in which the mortar was treated with sodium hydroxide. Of these two tests, the latter was preferred due to its quantitative determinations. Attempts to obtain a quantitative measure of warping in the slab test through the use of feeler gages did not prove satisfactory, as the determinations could not be checked with sufficient accuracy.

An attempt to use bi-aggregate slabs 12 inches long has been made with the thought that warping of the slab could be detected at a much earlier age. Some difficulties in the preparation of a slab of uniform density throughout its length have been found, and the handling and storage of slabs 12 inches long and only one-fourth inch thick have presented some problems. No decision as to the suitability of these specimens has been reached.

SIMPLE TEST SOUGHT

The use of large numbers of aggregates obtained from noncommercial sources in the western part of the country caused some thought to be given to the development of a method of test for alkali-aggregate reaction which could be made with a minimum of testing equipment and with no requirements for delicate or expensive apparatus. The development of such a test would permit the examination of samples of aggregate in field offices and reduce the number of samples submitted to the laboratory for more thorough tests. In an effort to develop this test some study was given to the preparation of a hard and brittle container of mortar to enclose the mortar prepared with the aggregate under test. This was on the assumption that, should the mortar under test develop expansion due to chemical reaction, the expansion would crack the external mortar shell and reveal the reactive character of the aggregate at an early age. Difficulties were found in the preparation of the hard mortar containers. Although a hard mortar could be obtained with a 1:2 mix and a water-cement ratio of 0.25, the mortar had to be tamped in place and specimens of uniform density were seldom obtained.

MASON JARS AS CONTAINERS

To furnish a hard and brittle container for samples of mortar under test for alkali activity, the use of mason jars was suggested and tried initially by the junior author. These proved quite successful and a large program of tests to determine the suitability of the method was begun. The mortars included in the first series of tests were prepared with known reactive and nonreactive aggregates, and both high- and low-alkali cements. The jars used had various shapes of horizontal cross section. In some the cross section was circular; in others the cross section was square with rounded corners of short radius; and in a third type the radii of the rounded corners were so large that the cross section approached the circular. Some jars were filled to the neck; others were filled about half full. After the mortar had been placed in the jar, the jar cap was fastened and the jar set aside until the mortar hardened. A small amount of water, about 25 ml., was then added to each jar, and the cover replaced. Later in the study of the method, it was suggested that the jars be inverted after the water had been added to the jar, to insure that the alkali in the mortar would not be leached out. This practice was followed, not necessarily to prevent leaching of the alkali, but to permit more ready detection of cracking in the jar. Another suggestion, based on the behavior of specimens containing reactive aggregates and treated with acid solutions, involved the addition of about 5 ml. of concentrated hydrochloric acid to each jar of mortar. This was added to the mortar immediately after preparation of the test sample. No benefit to the test procedure or acceleration of the reaction was found with this treatment.

Some disappointing results were obtained in the first series of tests. A large number of jars containing mortar prepared with high-alkali cement and a nonreactive aggregate failed at an early age. Other jars containing reactive mortar did not fail within a reasonable length of time. Some jars cracked, while others containing the same mortar did not. The failure of the jars containing a high-alkali cement and an inert sand was traced to a reaction between the cement and the glass of the jar. This was subsequently prevented by coating the jar with an alkali-proof paint. A number of coating materials were tried but best results were obtained with pigmented materials having a chlorinated rubber or a vinyl resin base. Of these two, that with the chlorinated rubber base is preferred. The coating material with the vinyl resin base seems to be tougher and less liable to damage when the jar is being filled, but that with the rubber base has a less objectionable odor, is easy to handle, and dries very rapidly. Several different colors of the rubber base finish were tried, including white, black, and gray. The white coating is not desirable as it interferes with the detection of the dried gel—the end product of the alkali-aggregate reaction—when the mortar is removed for inspection from the jar after failure.

The mechanical strength of the jars undoubtedly has a considerable influence on the results of this test, and the variable results obtained in the first series of specimens possibly were due to the use of different types of jars of variable strength. From observation of the test results obtained, the least desirable type of jar appeared to be that with a square cross section and rounded corners of short radius. Many jars of this type failed only at the junction between the sides and the bottom, and it appears that this is the weakest point in the jar. Jars with a truly round cross section appeared to give the most uniform results but difficulty was found in obtaining these. Those most readily available seem to be the square jar with short radius corners and the jar with heavily rounded corners and a cross section approaching the circular.

OTHER JAR TYPES TESTED

Some jars used in these tests contained striations or mold marks in the glass, and an attempt was made to obtain a jar of more uniform quality. Consideration was given to glass jars used for packaging foodstuffs, such as mayonnaise, and these jars were compared for uniformity with the mason jars of nearly circular cross section. The results obtained with the two types of jar differed so little that it was decided to adopt the latter for use in the test, or the round mason jar when it could be obtained. These investigations indicated that it would be desirable to make more than one jar specimen for each mortar, and the practice of preparing three jars of a kind was adopted. The use of smaller containers such as shell vials was tried and found satisfactory in tests of mortars, but the pint mason jar is preferred as this can be used for testing concrete with a maximum size of aggregate of at least 1 inch.

[4] *Standard method of test for compressive strength of hydraulic cement mortars*, 1947 Supplement to Book of A. S. T. M. Standards. Part II, p. 43.

Storage of the jars at room temperature has been the general practice in most tests conducted to date as this condition probably would be followed in tests in the field. Some tests were made at 120° F. to determine the effect of the higher temperature. Included in this group were test specimens for a number of gravels composed essentially of quartz and chert. Many of these jars broke at a very early age, in several cases at only 2 or 3 days, and it was suspected that the cracking of the jar might have been caused either by increase in volume of the aggregate due to absorption of water, or to expansion of the mortar or concrete due to heat. To investigate these possibilities a series of tests was made on samples of concrete prepared with high-alkali cement, quartz sand, and a gravel containing chert or a crushed granite. Both coarse aggregates were graded between the 1-inch and No. 4 sieves. A batch of concrete, sufficient for six jars, of 1:2:3 mix was prepared, with each coarse aggregate in an oven-dry or saturated condition. Three of each group of six jars were stored at laboratory temperature until the concrete hardened. The other three were placed in a cabinet held at 120° F., as soon as they were prepared. At an age of 24 hours, water was added to all jars, and those which had been stored at laboratory temperature were placed in the warm storage cabinet. The jars were inspected daily for the development of cracking, and the test continued to an age of 71 days.

CAUSES OF FAILURE

The results obtained in this series of tests indicate that failure of the jars may be caused entirely by volume changes due to absorption of water by certain coarse aggregates, or by the thermal volume change of hardened concrete. Consequently, coarse aggregates used in this test should be saturated prior to mixing if they contain types of aggregate which have a large amount of absorption. Included in these types of aggregates are chert and some varieties of limestone. If the jars are stored at an elevated temperature, they should be placed in this storage before the contents of the jar have hardened. It should be stated that in tests of fine aggregate, that is, material passing the No. 4 sieve, no difficulty due to failure by thermal volume change has been found.

The mason jar test is being used in studies of the effect of an admixture of a finely divided siliceous material on the alkali-aggregate reaction. In a preliminary investigation, concerned primarily with the amount of this admixture to be used in a reactive mortar, the results shown in figure 1 were obtained. This figure is presented to show that quantitative results can be obtained with this method of test. All specimens were prepared with a high-alkali cement, and all contained opal except one group of reference specimens. Each point in figure 1 represents an average of three specimens. The tests were discontinued at an age of 30 days, when all except the reference specimens had failed. With few exceptions, the three jars in each group gave consistent results. In the entire set of 21 groups containing reactive aggregate, the three jars in each of 14 groups broke at the same age, or at an age of within 1 day of the average for the three jars. In only four groups did any of the jars break at an age more than 2 days from the average. As all jars developed the same type of cracking, it is believed that a marked difference between the ages of failure of specimens of a group may be due to differences in strength between the jars. Methods for classifying jars prior to use are being considered.

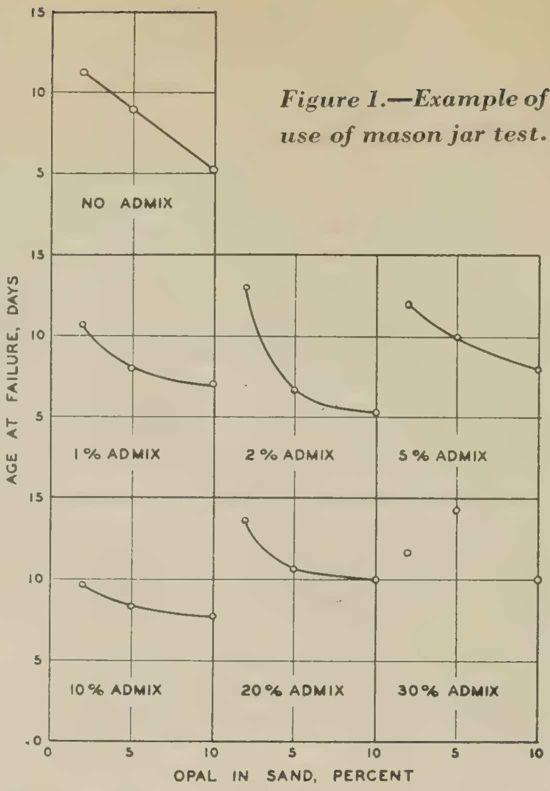

Figure 1.—Example of use of mason jar test.

The test is considered of most value for the rapid determination of the susceptibility of aggregates to alkali, and for the determination of the compatibility of given combinations of cement and aggregates. The test specimens are prepared readily with very simple equipment. Although a balance has been used to weigh the components of the mortar or concrete, proportioning of the mix by volume should be satisfactory. For 1:2 mortars, a water-cement ratio of 0.5 by weight, or 0.75 by volume, has been found suitable. When it is desired to test concrete mixtures, sufficient water to give a plastic mix should be used. The prepared mortar or concrete is placed in the coated jar, with care to prevent scratching the coating, and the bottom of the jar struck several times with the palm of the hand to settle the contents. The jar is then capped. At an age of 20 to 24 hours, about 25 ml. of water is added to the jar, the cap replaced firmly, and the jar placed in storage. Daily inspection of the jars is advisable. When the jars are stored in an inverted position, the cracking frequently can be seen without moving the jar if the bottom and an inch or so of the side of the jar can be inspected.

Various forms of cracking are found. Sometimes a simple vertical crack will appear. If this cracking does not develop further, it is considered to indicate an expansion of very low order, resulting possibly from an alkali-aggregate reaction which may be dismissed as immaterial. However, if simple cracking progresses into multiple cracking or, as is most often the case, the jar develops multiple cracking at the initial failure, the failure may be considered to indicate an aggregate of high susceptibility to reaction with the alkali in cement. Sketches of jars showing simple cracking, simple cracking which developed into multiple cracking, and initial multiple cracking are shown in figure 2.

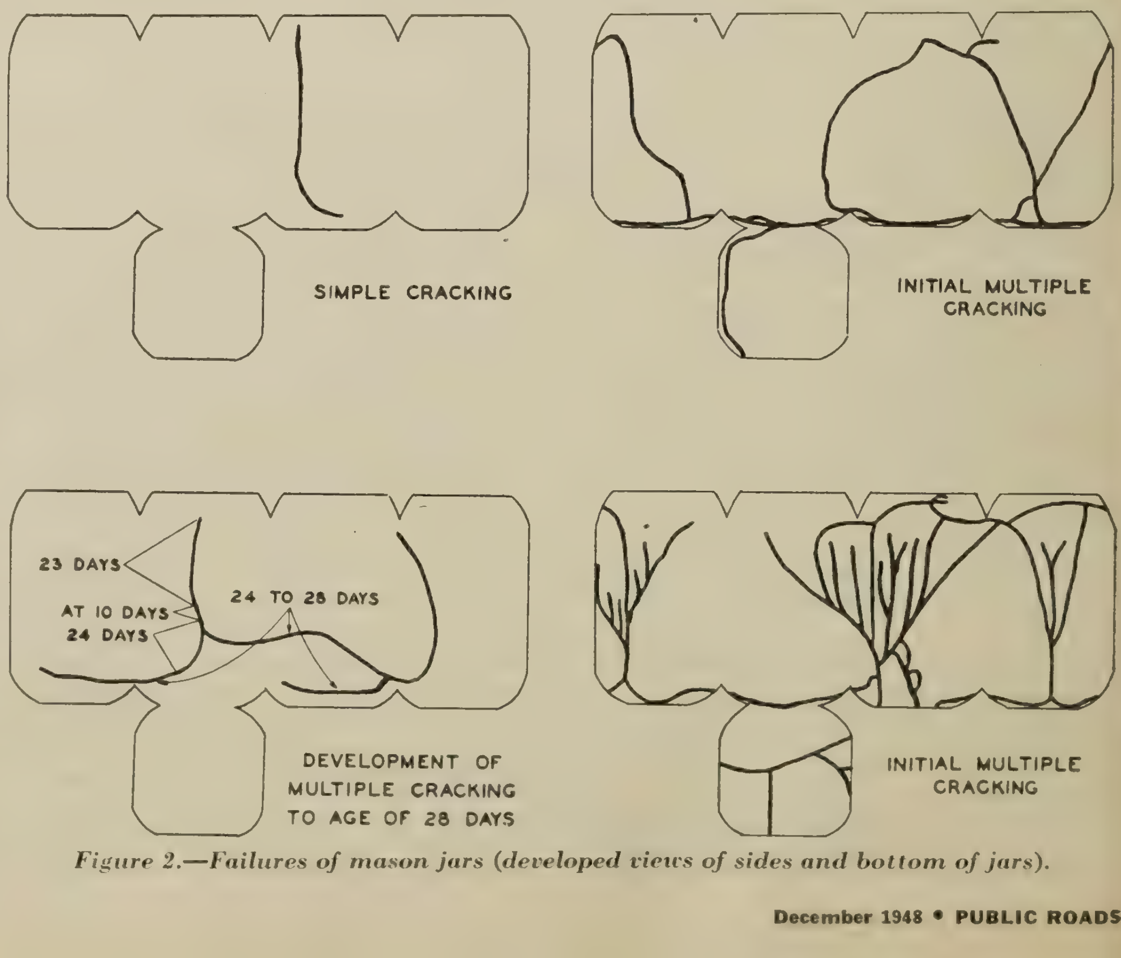

Figure 2.—Failures of mason jars (developed views of sides and bottom of jars).

Conducting Long-Range Highway Needs Studies

BY THE DIVISION OF FINANCIAL AND ADMINISTRATIVE RESEARCH
PUBLIC ROADS ADMINISTRATION

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The great upsurge of motor-vehicle travel since the end of the war has directed public attention to the rapidly mounting inadequacy of our road and street systems. It was realized years ago that complete appraisal of the entire field of highway transportation was needed, and the highway planning surveys, undertaken in the middle 1930's, were conceived for that purpose.

The preparation of all-inclusive, long-range plans is increasingly urgent. Several States have recently completed or have under way the preparation of comprehensive reports on the engineering, financial, and administrative phases of highway needs. These reports, which are actually the expected culmination of the State highway planning surveys, are usually prepared under the auspices of a specially created highway study committee of the State legislature, and are conducted in cooperation with each level of government having jurisdiction over highways, roads, or streets. The cooperation of other groups interested in the highway program, such as the motor-vehicle user groups, local civic associations, and farm and business interests, is likewise essential.

Preparation of the reports of highway needs requires consolidation of all available factual data, most of which is at hand in the planning surveys; collection of additional data needed to round out the portrayal of the current situation; and recommendation of the program for future action. Up to the present time, the engineering phases of the long-range reports have imposed the greatest demand upon the time, talent, and ingenuity of the highway engineers in those States that have undertaken them. Accordingly, the following discussion relates principally to the means by which the preparation of the engineering report is placed under way and some of the more unusual problems encountered in the conduct of this phase of a long-range highway needs study.

RAPID INCREASES in motor-vehicle ownership and travel that have developed since the end of World War II are bringing into focus the major weaknesses of the Nation's highways, roads, and streets—obsolescence, deterioration, and congestion, just to mention a few. New problems in providing adequate facilities have arisen from significant population shifts, from changes in land usage, and from other factors such as increased mechanization and development of new industries during and after the war. The backlog of deferred work that accumulated during the war; the present shortages of labor, equipment, materials, and transportation; and the current price situation have created additional problems.

It was realized, years ago, that a complete appraisal of the entire field of highway transportation, and its relation to other forms of transportation, was urgently needed. The highway planning surveys, undertaken in the middle 1930's by the State highway departments in cooperation with the Public Roads Administration, had that goal for their purpose. These surveys were conceived as instruments for the collection, analysis, interpretation, and summarization of complete factual data relating to the construction, maintenance, operation, and administration of all branches of the highway plant. Their ultimate aim, and the primary one, was the preparation of a documented, comprehensive report concerning adequate highway systems, and means for their fulfillment.

Over the past 3 years the need for such all-inclusive, long-range plans has become increasingly apparent. It has become evident that prewar policies, necessarily of restricted scope because they were largely dictated by the limited revenues available for expenditure, would be wholly inadequate to cope with mounting highway [1] needs. Demands for improvement come from all sides, for all classes of highways, from the lightly traveled country road to the heavily traveled city thoroughfare.

THREE PHASES OF STUDY

Out of the present situation has come an urgent interest, on the part of an increasing number of States, in the intensive studies of long-range needs—the studies which were, more than 10 years ago, set forth as the goal of the highway planning surveys.

These long-range needs studies, as proposed when the planning surveys were first initiated—proposals still valid and timely—can be divided logically into three phases:

The engineering study.—This involves an engineering appraisal of the improvements needed and costs involved in bringing the highways up to required standards.

The finance and taxation study.—This includes studies of methods of financing the program, determinations of responsibility of support for various classes of roads and streets, degree of support by classes of users, etc.

[1] Throughout this report all classes of highways, roads, and streets are frequently referred to simply as "highways."

The administrative study.—This includes appraisal of the administrative relationships and controls necessary for orderly prosecution of the program and of the organizational structures and operating practices of various governmental units having jurisdiction over highways.

Work in connection with these phases can be conducted simultaneously and completed at about the same time, but whether it is preferable to issue the reports separately or in combination is a matter to be decided in each particular instance.

There are, of course, considerable overlapping interests between each of the three categories. Up to the present time, the engineering phases of the long-range reports have imposed the greatest demand upon the time, services, talent, and ingenuity of the highway engineers in those States which have undertaken such comprehensive studies. Because of this circumstance, and because a sound engineering appraisal of highway needs is an absolute and basic essential to any study of financing and administration, the following discussions will relate principally to the engineering phases of the long-range highway needs reports.

FIRST REPORTS COMPLETED

The first of the comprehensive engineering reports as contemplated by the planning surveys, covering all highways in a State, were *Engineering Facts and a Future Program* prepared for the California legislature in 1946 and *Highway Needs in Michigan* prepared by the Michigan Good Roads Federation in 1947. Other States have recently completed or have under way the preparation of engineering reports following somewhat the same general pattern. It should be recognized that improvements in the method of approach and presentation of results are continually being made and that there is no fixed step-by-step procedure to follow. Each study must be conducted in a fashion that conforms most nearly to the requirements for the particular State.

The elements of a long-range program are such that they cannot be handled successfully when limited to a single class of roads or to a single jurisdictional unit. The long-range program must be developed on a broad base, and experience has demonstrated that it must embrace every mile of highway, road, and street in the State.

Further, the success of the long-range program depends upon the support given by various levels of government—State, counties, cities, towns, townships, and boroughs—and by various other groups interested in the highway program, such as the motor-vehicle user groups, municipal and county leagues or associations, farm and business interests, the construction industry, equipment manufacturers, and materials producers. Each of these must participate to the maximum extent to insure that the long-range program is soundly conceived and executed and in the best interests of all groups concerned and of the public at large.

SPONSORSHIP OF STUDY

There are various means of sponsoring the preparation of a long-range highway program. Among these are (1) a committee created by the State legislature, (2) a committee appointed by the Governor, and (3) a committee established by a citizens' association or a road-user group. In the latter two cases, it is desirable that the legislature be involved at least to the extent of creating an advisory committee to work with the sponsoring committee.

From an engineering standpoint, the State highway department is admittedly the organization best qualified to conduct the comprehensive study of highway needs in a particular State. Hence, in some instances, the legislature or the Governor may direct the State highway department to make the entire study. This procedure is not usually employed, however, because of the numerous public relations problems involved. By its very nature, a governmental agency is limited in the scope of the problems it can handle without suspicion of bias, especially in fields into which its activities and jurisdiction do not normally extend.

Because of the need for dealing with agencies at various levels of government, and with the many public and private groups having varying degrees of interest in the numerous problems affecting the highway program, it appears preferable in most instances to conduct a long-range highway needs study under a legislative mandate by means of a specially created committee composed of representatives of all parties concerned. If, because of the unwieldiness of a large group, it is not practicable to afford complete representation to all parties on the parent committee, advisory subcommittees can be created to enable their active participation.

For reasons similar to those previously cited, the parent committee will ordinarily prefer to have the study headed by an outside specialist in this field rather than by a member of the State highway department. In this connection, however, it may be pointed out that the talent currently available and qualified to organize and carry to satisfactory completion an engineering study of this magnitude is extremely limited.

There are various means of financing these studies, utilizing State and local funds, contributions by citizens' and user groups, and Federal funds. The Public Roads Administration has given full endorsement to the purposes of these surveys and has expressed its desire to render all possible aid in their execution. The conditions for obtaining Federal funds to aid in paying the expenses of studies of this kind are set forth in a statement of the Public Roads Administration dated September 24, 1947.

The fitting together of the complex and voluminous array of factual information into a long-range program is an undertaking of considerable magnitude. Intensive effort is required to organize a study, place it under way, and complete it within a reasonable period. Among the most valuable functions performed by the specialist, hereafter referred to as the engineer-director, is the step-by-step education of the committee membership. Committee members usually do not have an engineering background; but by holding periodic meetings at which the engineer-director can report progress and discuss the significance and interpretation of the findings which are being obtained, immeasurably beneficial results are realized. The county representatives become familiar with the problems in the cities and on the main highways, the cities acquire a greater appreciation of the rural road problem, and various special interests have the opportunity for round-table discussion of their own and each other's viewpoints. Broadening of outlook and a greater awareness and appreciation of the complex problems involved in highway administration, planning, programming, design, construction, and maintenance soon become evident. Artificial barriers are broken down in a mutual effort to solve highway problems equitably and in the best interests of all concerned.

PUBLIC HEARINGS

Public hearings conducted by the legislative committee are a customary and valuable phase of making a long-range highway study. At such hearings, usually held at a number of places through the State, the public has the opportunity to become informed about the study and to offer opinions on the nature of the road problems in their area. The timing of these public hearings is most important. There is a general tendency to hold such hearings too early, prior to starting the field work, when there is little in the nature of concrete proposals or procedures that can be presented to the public. As a result, public enthusiasm will gradually recede and the final report may come as an anticlimax.

On the other hand, public hearings can be of tremendous value in stimulating local cooperation when they are held at the time the counties, cities, and towns are contacted with respect to making an engineering appraisal of their own particular road and street needs. Since a high degree of public interest is desirable at the time of issuance of the report, public hearings are also of value when held just in advance of the date of issuance. Possibly the best arrangement is for some hearings to be held at both times.

The time schedule for undertaking a report should include allowances of about 3 months for completion of the preliminary arrangements and about 12 months additional for the active work program. If the study is scheduled for a shorter time, the work load is likely to become too heavy and certain desirable features may be sacrificed; if scheduled over a longer period, some of the phases become stale, interest in the work subsides, and very little practical improvement in the quality of the report is obtained. It is recognized, of course, that unusual complexity or magnitude of the work may make it desirable to conduct the study over a longer period in a particular State. In some instances, also, personnel shortages and cost considerations may be sufficient reason for scheduling the study over a longer period.

PRELIMINARY OPERATIONS

Essential preliminary operations in undertaking a highway needs study are the preparation of a report outline and a work program schedule. The complete report outline embraces the following:

1. Introduction.
2. History of highway development in the State.
3. Characteristics of road use.
4. Future development of the State and its subdivisions.
5. Classification of road systems.
6. Standards of road improvement.
7. Standards of road operational improvement (traffic control, lighting, parking, law enforcement, and maintenance standards).
8. Elements of road cost.
9. Future road improvement program.
10. Finance and taxation.
11. Highway administration.
12. Highway vehicle and traffic regulation.
13. Conclusions.

These items are the main headings included in a suggested outline set forth in a statement of the Public Roads Administration dated September 24, 1947, in which there is a listing of the detailed items included under each major heading. A similar discussion had been prepared for the highway planning surveys some years earlier. The main elements of the outline are applicable to practically all States. The emphasis to be placed upon various segments, however, will necessarily vary from State to State, depending upon the nature of the particular highway problems in each State.

PLANNING SURVEYS PROVIDE DATA

The major source of information will, in practically all States, be the State highway department's highway planning survey. Generally there is a wealth of information relating to the State highway system but only limited data relating to roads and streets beyond the jurisdiction of the State highway department. Actually, it had been anticipated that the planning surveys would collect data on all highways but many States, limited in their resources, spent their principal efforts on the State highway systems. This situation, which is quite common, requires careful planning of the techniques and procedures to be employed in evaluating the highway needs for all jurisdictions on a comparable, straightforward, and easily understandable basis.

Probably the greatest single problem in a study of highway needs, from the standpoint of planning, time, effort, and coverage, is the work that must be done by the various jurisdictional units in connection with determining the costs of their respective future road improvement programs. A work program for this phase of the study involves the following step-by-step operations:

1. Field classification of roads and streets into logical systems.
2. Field appraisal of needs for improvement.
3. Processing and analysis of collected data.

MANUALS OF INSTRUCTION

The work in connection with the first two items is usually conducted in conformity with procedures outlined in manuals of instruction prepared by the staff of the study committee specifically for each administrative jurisdiction. Thus, in a State where there are three jurisdictions (State highway department, counties, and cities) for the construction and maintenance of highways, there would be three corresponding series of manuals of instruction.

The principal variations in these manuals would be those occasioned by the differences in the character of the road systems and the differences in the caliber of the respective engineering organizations at the various governmental levels. For example, very rigid criteria can be applied to State highway needs, since the State highway departments have the background of experience and research as well as the engineering talent needed to undertake a comprehensive study. Most counties and cities, however, are not correspondingly situated, and accordingly the criteria employed must be on a broader base to enable a reasonably defensible appraisal of needs to be made by the county and city personnel.

PARTICIPATION NEEDED

The success in gaining acceptance of the final results of a long-range engineering study depends in large measure on the extent to which each governmental unit having any degree of responsibility for the construction and maintenance of roads or streets participates in the evaluation of the needs for those roads or streets under its jurisdiction. Thus, in those cases where no qualified engineering personnel are available at the county or city levels, it may be necessary to hire private consultants or to depend upon the State highway department personnel to do the work. In general, this latter procedure should be considered only when the counties or cities are wholly unable to do the work themselves. Even in such instances every effort should be made to have a local representative collaborate with the State highway department personnel at the time the classification and estimates of needs for a particular county or city are being appraised. In this fashion, each county and city will, to the maximum extent possible, be an active participant in the work.

SYSTEM CLASSIFICATION

With respect to the work program, the field classification of roads and streets into systems should be made independently and in advance of the field appraisal of needs. This classification is simply the sorting of existing roads and streets into groups on the basis of function, taking into account such factors as integration, predominant purpose of use, suitability from the standpoint of equitable financing, and adaptability to efficient and economical administration by various levels of government.

The classification, together with an inventory of existing mileages by types, is prepared by the State highway department, the counties, and the cities, in accordance with procedures prescribed in manuals of instruction, for the roads and streets currently under their respective jurisdictions, and is submitted to the engineer-director of the highway study committee. The work is then reviewed by the staff of the engineer-director, and errors and differences in interpretation are ironed out in each individual case.

NEEDS APPRAISAL

As this work nears completion, the field appraisal of needs can be undertaken. This involves determination of existing deficiencies and estimating of costs of needed improvements for every mile of highway. This phase of the work is also governed largely by manuals of instruction prepared for the purpose. In these manuals, it is the general practice to prescribe two sets of standards relating to widths, surface type, alinement, grades, etc.: (1) Tolerable standards, and (2) construction standards. Tolerable standards are those which are considered satisfactory or tolerable on highways now in service. Any road section or other facility that does not measure up to the tolerable standard is considered eligible for improvement. In some cases it may be questionable whether a single particular deficiency is of sufficient importance to warrant an improvement. In other cases this same deficiency, in combination with other deficiencies, would indicate the immediate need for improvement. If it is determined that an improvement is warranted, as measured by the tolerable standards, the improvement to be planned should be in accordance with the construction standards, and the costs estimated accordingly.

In a given situation it might be assumed, for example, that for surfaced width for a rural road carrying a certain traffic volume the tolerable standard is 18 feet and the construction standard is 20 feet (minimum) to 24 feet (maximum).

If the existing road in question has a surfaced width of 18 feet or more, it would not be considered deficient in width. If, however, the surface is only 16 feet wide, it would be considered deficient in this respect. When the manner of overcoming this particular deficiency is being considered, it would be necessary to take into account all service factors—terrain, volume and character of traffic, etc.—in order to determine whether the proposed improvement should be constructed to 20 feet in width, 24 feet in width, or somewhere in-between. Further, all deficiencies must be considered in total for any given road section when arriving at the final estimate of cost for a proposed improvement. This procedure summarizes, quite briefly, the method by which selected standards are applied to existing highways to determine the extent of existing deficiencies.

As the reports upon existing deficiencies and estimates of cost of needed improvements are prepared and submitted to the engineer-director by the State highway department, the counties, and the cities, they are reviewed for consistency, adherence to instructions, completeness, and accuracy. This is generally termed the "screening" process and is one of the most important phases of the entire engineering study. Most counties and cities can be depended upon to prepare their estimates in accordance with the instructions. Invariably, however, some will submit estimates that are obviously too low, too high, or incomplete. Such estimates must be taken up again with the reporting agency, and all errors, omissions, and differences in interpretation ironed out in each case.

Experience indicates that from 6 to 8 months should be allowed for the field work in connection with classification and appraisal of needs. The time required depends upon such factors as the number of governmental units having jurisdiction over highways; the caliber of the various engineering organizations in each jurisdiction; the degree of cooperation generated among the various levels of government; and the number and caliber of personnel made available as a staff to the engineer-director.

PROCESSING AND ANALYSIS

When the screening of the field reports of deficiencies and costs is completed, the work of processing and analyzing the reports is commenced. This phase of the work can be relatively simple or it can be complicated and confused, depending upon the amount of advance study given to each previous operation.

For example, it must be decided at the very outset of the engineering study whether manual or tabulating machine methods are to be employed in the final processing. On the basis of experience to date it can be stated that, almost without exception, tabulating machine methods are far superior to manual methods for this type of study. Accordingly, the following discussion relates almost entirely to tabulating machine methods, although the same principles will, of course, apply to manual methods.

It is essential, prior to starting any field work, that there be a clear understanding of what data are needed, how the data are to be obtained, and how the data are to be analyzed. This requires intensive preliminary study. To aid in this phase of the work, arrangements should be made for consultation with the State highway department personnel, county and city representatives, Public Roads Administration technicians, and experts in other public

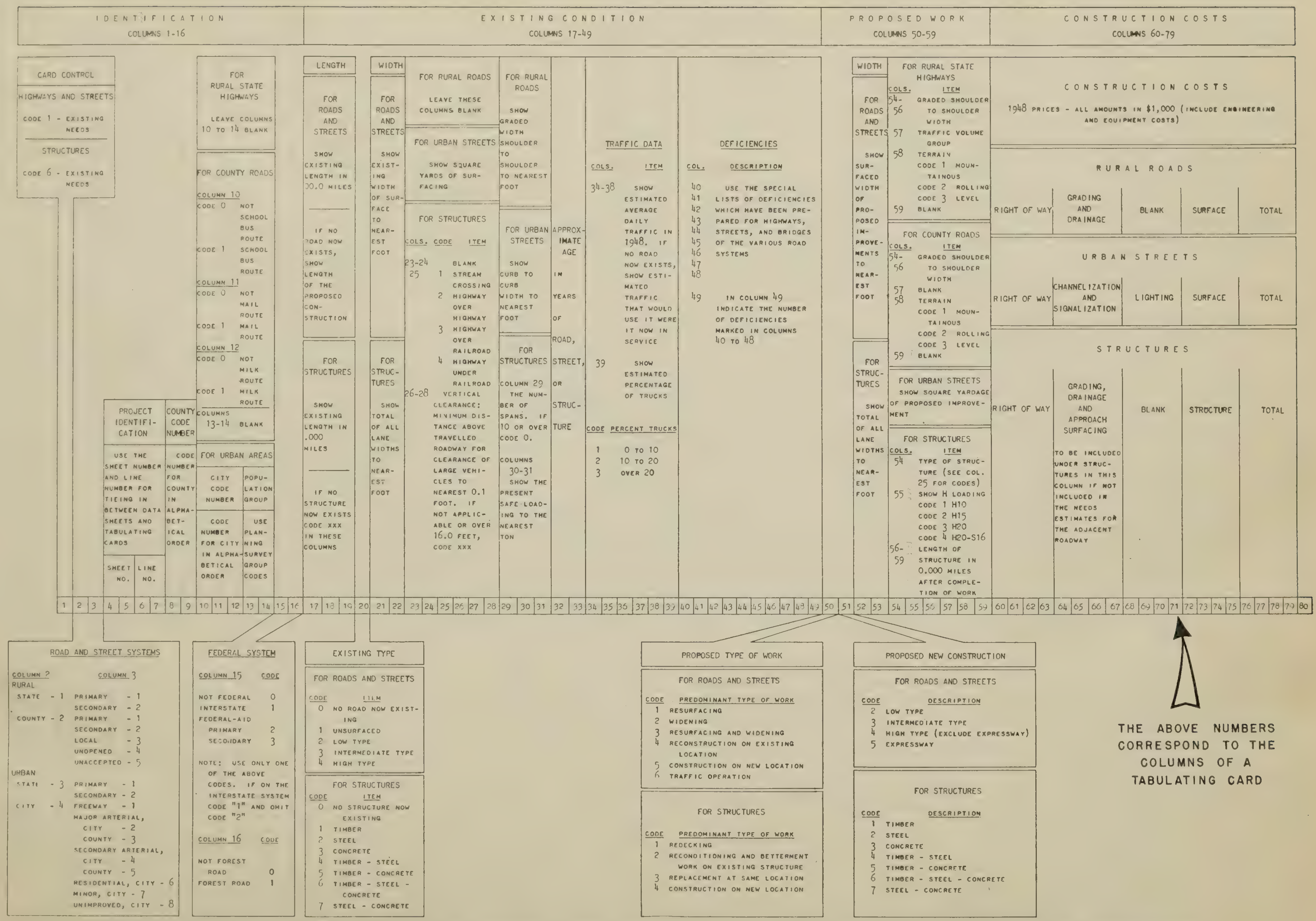

Figure 1.—Highway needs master control sheet for form set-up and machine tabulations of rural and urban road, street, and structure data.

or private organizations having prior experience or outstanding qualifications in studies of this type.

One major objective of such preliminary study and consultation is the development of a master control sheet showing the nature and sequence of the items to be processed in the various operations leading up to the final tabulations of the needs data. Figure 1 shows a sample lay-out for a master control sheet to which are geared the mechanics of the following phases of the engineering study: (1) Inventory of roads, streets, and bridges; (2) preparation of manuals of instruction for appraisal of needs; (3) design of record forms; (4) preparation of coding manual; (5) design of code sheets and tabulating cards; and (6) lay-out of final tabulations.

Once the engineer-director has approved the master control sheet, there should be no changes except with his specific approval. The insertion of additional data or a rearrangement of items for city streets, for example, could readily complicate the coding sequence and even require special attention to tabulating machine operations to an extent that would nullify the advantages inherent in the basic master control sheet set-up.

Figure 2 shows a work sheet used for recording deficiencies and cost of needed improvements for county roads. It will be noted that the sequence and arrangement of items on this form correspond to those on the master control sheet (fig. 1). Other forms, such as those for State highways and city streets, also correspond to the master control sheet. Figure 3 represents column headings for a series of six code sheets covering roads and structures on city streets, county roads, and State highways. These headings likewise follow the arrangement of the master control sheet.

TABULATION CONTROLS

The forms illustrated in figures 1, 2, and 3 are practically identical with those used in a State that has recently completed an engineering study, and represent a pioneer effort in setting up fixed controls for the mechanics of conducting the work. Since it is the primary purpose here to indicate the general nature of the master control sheet, there is no discussion included in this report of the many details of its design. Such details must be worked out for each individual State.

At every step of the processing operations for each county and for each city, care must be exercised to maintain frequent manual controls, especially for mileages and costs. Since there is such a mass of data being handled in a relatively short period of time, usually by inexperienced help, these manual controls are necessary in order that any missing or duplicated data can be readily spotted at any stage of the operations. These controls are prepared for the "as submitted" programs prior to screening, and for the "accepted" programs after completion of screening, and are also of tremendous value in proving the final tabulating machine listings.

After the tabulating cards have been proved, the final summaries of deficiencies, mileages, and costs in the needs program can be prepared. There is no particular problem involved in making the necessary tabulations, since they can be obtained readily from the punch cards. There are, however, certain tabulations relating to mileages and costs that are essential as matter of continuous reference and permanent record. These tabulations show the relationship between existing surface types, kind of work recommended, new type of surface, mileage, and costs. A typical arrangement of the form used for summarizing these data for rural roads is shown in figure 4. Similar tables should be prepared for structures and for city streets.

EXISTING CONDITION

5 LENGTH (IN 0.0 MILES)

6 TYPE SURFACE

7 SURFACE WIDTH

8 ROADWAY WIDTH

9 APPROXIMATE AGE OF SURFACE

10 ESTIMATED DAILY TRAFFIC

11 TRUCKS AND BUSSES
0-10% ... ☐ 10-20% ... ☐ OVER 20% .... ☐

IDENTIFICATION

1 COUNTY

2 ROAD NAME, NUMBER AND/OR LOCATION

3 PROJECT NUMBER

4 SYSTEM
C-1 .. ☐ SCHOOL BUS ROUTE ...... ☐ FEDERAL AID SECONDARY .. ☐
C-2 .. ☐ U.S. MAIL ROUTE ...... ☐ U.S. FOREST HIGHWAY .... ☐
C-3 .. ☐ MILK COLLECTION ROUTE .. ☐ OTHER FEDERAL .... ☐
C-4 .. ☐

DEFICIENCIES

	EXISTING	TOLERANCE	DEFICIENT		DEFICIENT
12 TYPE SURFACE	____	____	☐	17 ROADWAY BASE	☐
13 SURFACE WIDTH	____	____	☐	18 DRAINAGE	☐
14 ROADWAY WIDTH	____	____	☐	19 SURFACE CONDITION	☐
15 GRADIENT	____	____	☐	20 SPECIAL WARRANTS *	☐
16 CURVATURE	____	____	☐	* (EXPLAIN IN REMARKS)	

PROPOSED WORK

TYPE OF WORK

21 RESURFACING .... ☐
22 WIDENING SHOULDERS .... ☐
23 RESURFACING AND WIDENING .... ☐
24 RECONSTRUCTION ON SAME ALINEMENT .... ☐
25 CONSTRUCTION ON NEW LOCATION .... ☐

31 MOUNTAINOUS .. ☐ ROLLING .. ☐ LEVEL .. ☐

STANDARD OF CONSTRUCTION

26 TYPE SURFACE .... ________
27 SURFACE WIDTH .... ________
28 ROADWAY WIDTH .... ________
29 MAXIMUM GRADE .... ________
30 MAXIMUM CURVATURE .... ________

COSTS

32 RIGHT-OF-WAY .... $ ________
33 GRADE AND DRAIN .... $ ________
34 SURFACING .... $ ________
TOTAL .... $ ________

NOTE: INCLUDE PRORATED ENGINEERING COSTS IN ABOVE ESTIMATES. USE 1948 PRICES.

REMARKS
(EXPLAIN SPECIAL CONDITIONS SUCH AS TIME DELAYS, ACCIDENT HAZARDS, DUST, SNOW, ICE, ETC.)

Figure 2.—Project work sheet for county road needs.

SCHEDULING PROGRAMS

With the data at hand relating to the existing deficiencies and costs of needed improvements, the next step is the scheduling of the program. It cannot, within reason, be expected that existing deficiencies can be overcome in 1 or 2 years. Within the limits of foreseeable revenues, any period up to as much as 20 years may be required for bringing the highway systems up to a satisfactory standard of improvement. It has been the usual practice in earlier studies of this nature to present the total costs of the program for a given period of years, say for a 5- or a 10-year period.

Thus, the entire analysis and presentation of results are related to a preselected fixed span of years. There are serious disadvantages to this advance selection of the program period, the principal one being its inflexibility. After the report is completed, the legislature, for example, may desire to consider a shorter or a longer period, with the result that numerous

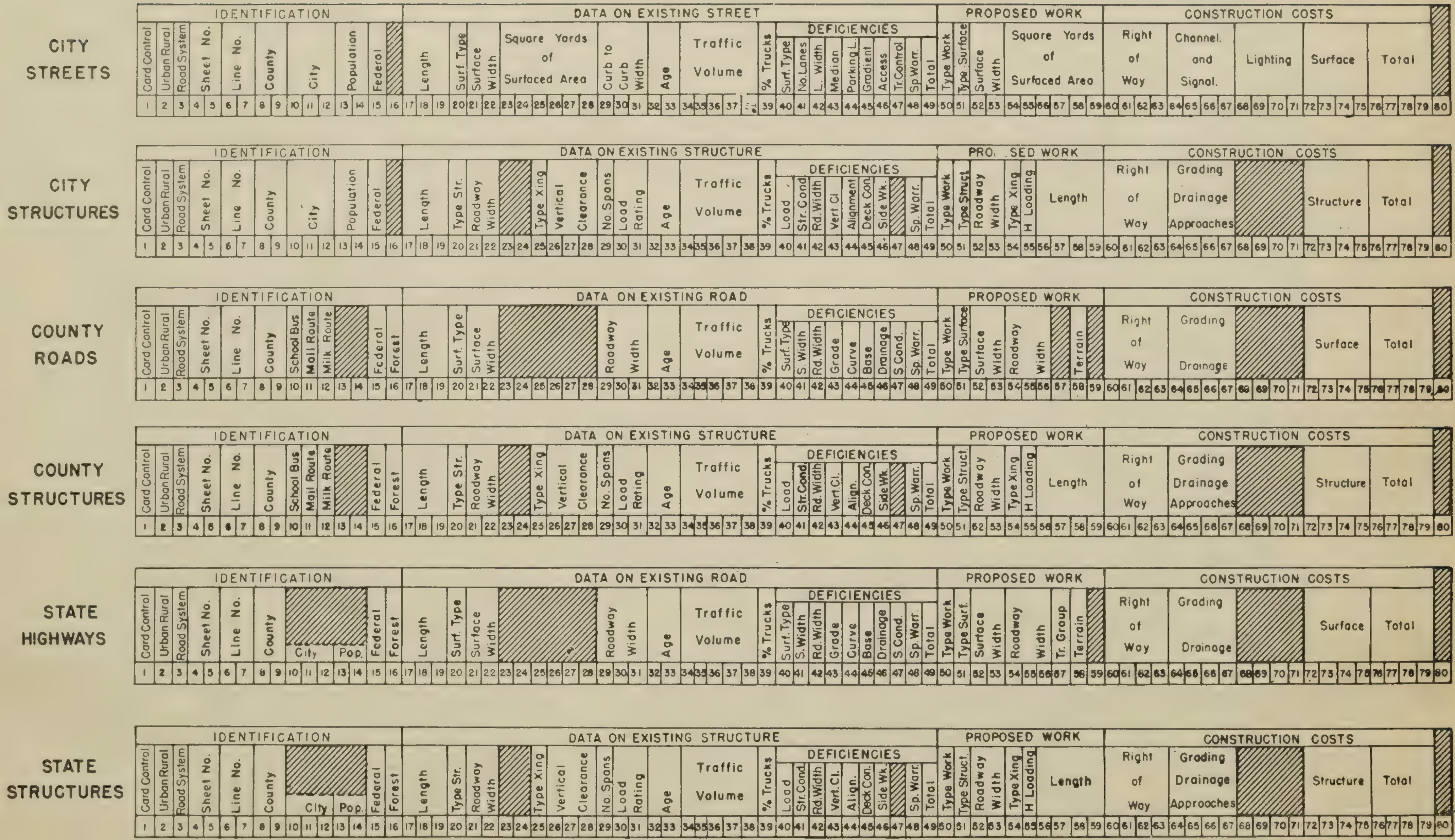

Figure 3.—Column headings for code sheets.

SERIES ________
SYSTEM ________

TYPE OF WORK	EXISTING TYPE IN SERVICE WHICH REQUIRES IMPROVEMENT	PROPOSED NEW CONSTRUCTION																								
		GRAVEL					LOW					HIGH (EXCLUDING EXPRESSWAYS)					EXPRESSWAY					TOTAL				
		MILES	COST				MILES	COST				MILES	COST				MILES	COST				MILES	COST			
			ROW	GRADING	SURFACE	TOTAL		ROW	GRADING	SURFACE	TOTAL		ROW	GRADING	SURFACE	TOTAL		ROW	GRADING	SURFACE	TOTAL		ROW	GRADING	SURFACE	TOTAL
RESURFACING	UNSURFACED																									
	GRAVEL																									
	LOW																									
	HIGH																									
	TOTAL																									
RESURFACING AND WIDENING	UNSURFACED																									
	GRAVEL																									
	LOW																									
	HIGH																									
	TOTAL																									
RECONSTRUCTION ON EXISTING LOCATION	UNSURFACED																									
	GRAVEL																									
	LOW																									
	HIGH																									
	TOTAL																									
CONSTRUCTION ON NEW LOCATION	NONE																									
	UNSURFACED																									
	GRAVEL																									
	LOW																									
	HIGH																									
	TOTAL																									
TOTAL EXCLUDING WIDENING	NONE																									
	UNSURFACED																									
	GRAVEL																									
	LOW																									
	HIGH																									
	TOTAL																									
WIDENING	GRAVEL																									
	LOW																									
	HIGH																									
	TOTAL																									
GRAND TOTAL																										

Figure 4.—Rural road replacements and costs summary sheet.

adjustments are made to the findings as presented in the report. Such adjustments are frequently arbitrary and sometimes involve serious errors when made by individuals not familiar with the nature or significance of the various elements of cost involved in an improvement program.

To overcome this situation, it has been the practice recently in engineering reports to present costs in a series of periods ranging from a 1-year program up to a 20-year or even a 25-year program. The very short periods and the very long periods have little significance other than to provide proper orientation and to answer certain questions which are logically raised when legislative consideration is being given to the period of years over which the highway systems should be brought up to standard.

OTHER COST ELEMENTS

In addition to the existing needs, as determined in accordance with the procedure which has been discussed, there are other cost elements involved in a long-range program that require careful analysis. These are the maintenance requirements and the additional needs accruing during the program period.

In estimating maintenance requirements, consideration is given to changes in the development of the road systems and to required improvements in standards of maintenance. Studies are made of the costs per mile or per square yard for maintaining various types of roads and streets at the stages of development which exist at the present time and which it is estimated will exist at the end of the program period. Such an analysis requires that a careful record be made of the probable mileages, by types and standard of construction, that will be constructed and retired during the period (see fig. 4). By such means, it is possible to arrive at reasonably reliable estimates of present and future maintenance requirements.

Certain additional costs, which may amount to as much as 20 percent of the total program, must be taken into account when the existing needs are spread out over any given program period in the future. For those needs that now exist but will come up for improvement in the later years of the program period, some kind of earlier stopgap improvement will be necessary to keep the affected roads in operation until the standard improvement can be made. Further, an existing road may currently be deficient only with respect to width, but by the time it comes up for improvement it may be deficient in several other respects and require complete reconstruction instead of widening alone.

It is also necessary to provide for the replacement, throughout the program period, of roads which are not in current need of improvement. In other words, it is possible that many of the existing roads are in tolerable condition as far as present-day traffic is concerned, and accordingly are not included in the program of existing needs. They will, nevertheless, deteriorate and become obsolete and require rebuilding in order to keep them up to satisfactory standards of service.

Some of the new construction built during the early years of the program period will also wear out and require rebuilding before the end of the period. The amount may be small but it must be taken into account in order that the full magnitude of the highway needs can be clearly and fully portrayed.

There is no list of factors that can be employed to evaluate these additional needs accruing during the program period. A procedure for making these estimates must be developed in each instance. Knowledge of construction practices and past experience with respect to the service life characteristics of various types of construction provides necessary background.

In addition to the needs which have already been discussed, there will be requirements for new roads, streets, and structures throughout the program period. Urban growth and redevelopment, industrial expansion, and other changes will generate new needs in the future years. Not all of these can, of course, be estimated with certainty.

ADJUSTMENT FOR PRICE LEVEL

A program of needs developed in accordance with the foregoing procedures will afford a positive index of the extent of the road and street requirements for varying program periods. At this stage of the analysis, however, the costs involved are at a price level existing at the time the estimates were made. If it is assumed, for example, that the costs of needed improvements were made at a 1948 price level, then there is the problem of adjusting the costs to the future price level that it is estimated will prevail throughout the various program periods.

It is not the purpose here to discuss or evaluate the various factors which are likely to influence the future trend of construction prices. However, a few comments concerning the nature of the problem can be made. Based upon a prewar index of 100 percent for the 1937–41 period, the present 1948 highway construction price level is in the neighborhood of 200 percent. Let it be assumed, for example, that the index will gradually recede and level off by the mid 1950's to a certain value. From the trend thus obtained, the index for each future year can be applied to the dollar volume of work which it is expected will be accomplished during that year. This adjustment is necessary to take into account the fact that income available for expenditure can normally be expected to increase gradually throughout the program period because of increases in gasoline consumption, etc.

SINGLE INDEX USED

From a weighted price index thus obtained for each program period, the program of needs at the 1948 or any other given price level can be readily adjusted for conditions which are expected to prevail in the future. Ordinarily an index based upon the projected trend of construction prices on State highway work is considered acceptable for the purpose of adjusting maintenance costs and the programs for counties and cities. Any refinements by virtue of computing special indexes for these latter elements are difficult to justify in view of the lack of data upon which to make individual projections. Further, the small degree of apparently additional accuracy which may be involved is of dubious value in view of the lack of exactness in projecting the future price trend in the first instance.

PRIORITIES OF IMPROVEMENT

Before closing, it might be well to comment upon priorities of improvement in relation to the long-range program. Priority listings are useful in making it possible to check readily upon each highway agency with respect to progress and adherence to a predetermined program. To a considerable extent these listings are likewise useful to the various highway agencies in dealing with pressure groups. With each passing year, however, increased knowledge and experience will inevitably dictate changes and adjustments to the priority listings that will defeat the purpose for which they were originally established.

Further, the reporting forms and estimating procedures which have been discussed in connection with the engineering study are not designed for use in setting up listings of improvements in order of priority. To accomplish this latter objective would require detailed study of each proposed improvement, both individually and in relation to other improvements, to such an extent that other important phases of the long-range study would be completely overshadowed.

There is, of course, a definite place for priorities in a current 2- to 5-year program in which strict adherence to a specific schedule of improvements can be justified on the basis of comprehensive detailed analyses of all the facts available at the time the program is set up. Such a fixed and definite but relatively short-term program covering certain designated improvements should be kept continuously at hand by each highway agency. Whether a particular highway agency is in a position to actually do this depends upon the status of its planning activities and the caliber of the analyses which are brought to bear upon the problems involved.

In addition, the short-term program should be consistent with the over-all long-range highway program, which in turn must be periodically reappraised and kept up to date. This reappraisal of the long-range program should be made at least once every 10 years.

REPORT IS A BEGINNING

The completion of the long-range report must also be viewed as the real beginning, rather than the end, of the period of usefulness of the various committees which have participated in the studies. The value received by continuing the life of these or similar committees is tremendous. Being composed of informed representative citizens, they are indispensible as deliberative bodies in the field of broad administrative and financial policy relating to highway matters.

By means of these long-range highway programs, great strides are being made in the orderly development of the vast mileage of highways, roads, and streets at a rate consistent with the increasing demands of motor-vehicle traffic, and at such locations and to such standards as will result in the most economical and efficient expenditure of funds.

Manual on Uniform Traffic Control Devices for Streets and Highways

A completely new edition of the Manual on Uniform Traffic Control Devices for Streets and Highways, prepared by the Joint Committee on Uniform Traffic Control Devices, has been published by the Public Roads Administration. The Manual is the generally recognized official national standard covering highway signs, pavement markings, traffic signals, islands, and similar devices used in the guidance, warning, and regulation of traffic. The importance of this work, and the need for its general adoption, will be obvious to any driver who has observed the lack of uniformity and the inadequacy of traffic control devices now used in many of our States and cities.

The Joint Committee includes representatives of the American Association of State Highway Officials, the Institute of Traffic Engineers, and the National Conference on Street and Highway Safety (now the National Committee on Uniform Traffic Laws and Ordinances). Its members are outstanding traffic engineers of State and city highway departments, the Public Roads Administration, and numerous national organizations concerned with highway use and safety. The Manual bears official approval of the three member organizations of the Joint Committee and of the American Standards Association.

Thomas H. MacDonald, Commissioner of Public Roads, has approved the Manual as implementation of section 12 of the Federal-aid Highway Act of 1944, which provides that signs, signals, and markings on roads and streets constructed with Federal aid shall be subject to approval of the State highway departments with concurrence of the Public Roads Administration. The law specifically directs the Commissioner of Public Roads to concur only in those installations that will promote the safe and efficient utilization of the highways.

Superseding editions of 1935 and 1942, the new Manual represents a 4-year effort of the Joint Committee and its staff. While basic standards generally remain unchanged, many refinements and some significant revisions have been made. The text has been entirely rewritten and expanded to provide a clearer, more complete exposition of each subject. Proper usage of traffic control devices is illustrated by 57 drawings and photographs. In addition, 85 signs are portrayed in the margins alongside the appropriate text. Many of the drawings are in color.

National standards for traffic control devices go back to 1925, when the American Association of State Highway Officials began to lay out a uniformly marked system of interstate highways. From this work grew the familiar U S numbered highway system and its well-known shield marker. In 1930 the National Conference on Street and Highway Safety adopted specifications for use of traffic control devices in cities. The two organizations created in 1931 the Joint Committee on Uniform Traffic Control Devices which produced, in 1935, the first Manual on Uniform Traffic Control Devices for Streets and Highways. The Joint Committee published a set of amendments in 1938; and in 1942 issued the War Emergency Edition, basically a condensed version of the original manual and its amendments with adaptations to wartime blackout conditions. Work on the new edition was begun in 1944 by the Joint Committee, enlarged to include the Institute of Traffic Engineers as well as the two founding organizations.

The Manual on Uniform Traffic Control Devices for Streets and Highways can be purchased from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 50 cents per copy (a discount of 25 percent is allowed on orders of 100 copies or more). Payment must accompany orders. Orders should not be sent to the Public Roads Administration. No free distribution will be made.

U. S. GOVERNMENT PRINTING OFFICE: 1948

A complete list of the publications of the Public Roads Administration, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Federal Works Building, Washington 25, D. C.

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The following publications are sold by the Superintendent of Documents, Government Printing Office, Washington 25, D. C. Please do not send orders to the Public Roads Administration.

ANNUAL REPORTS

(See also adjacent column)

Reports of the Chief of the Bureau of Public Roads:

1931, 10 cents.	1934, 10 cents.	1937, 10 cents.
1932, 5 cents.	1935, 5 cents.	1938, 10 cents.
1933, 5 cents.	1936, 10 cents.	1939, 10 cents.

Work of the Public Roads Administration:

1940, 10 cents.	1942, 10 cents.	1947, 20 cents.
1941, 15 cents.	1946, 20 cents.	

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 265T . . . . Electrical Equipment on Movable Bridges. 40 cents.

No. 191MP . . . Roadside Improvement. 10 cents.

No. 272MP . . . Construction of Private Driveways. 10 cents.

No. 1486D . . . Highway Bridge Location. 15 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. $1.25.

Highways of History. 25 cents.

Public Land Acquisition for Highway Purposes. 10 cents.

Public Control of Highway Access and Roadside Development (1947 revision). 35 cents.

Tire Wear and Tire Failures on Various Road Surfaces. 10 cents.

Legal Aspects of Controlling Highway Access. 15 cents.

House Document No. 379. Interregional Highways. 75 cents.

Highway Statistics, Summary to 1945. 40 cents.

Highway Statistics, 1945. 35 cents.

Highway Statistics, 1946. 50 cents.

Model Traffic Ordinance. 15 cents.

Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft. $1.50.

Federal Legislation and Regulations Relating to Highway Construction. 40 cents.

Manual on Uniform Traffic Control Devices for Streets and Highways. 50 cents.

Specifications for Construction of Roads and Bridges in National Forests and National Parks (FP-41). $1.25.

Single copies of the following publications may be obtained free upon request addressed to the Public Roads Administration. They are not sold by the Superintendent of Documents.

ANNUAL REPORTS

(See also adjacent column)

Public Roads Administration Annual Reports:

1943.	1944.	1945.

MISCELLANEOUS PUBLICATIONS

Road Work on Farm Outlets Needs Skill and Right Equipment.

Indexes to PUBLIC ROADS, volumes 17–23, inclusive.

Bibliography on Highway Lighting.

Bibliography on Highway Safety.

Bibliography on Automobile Parking in the United States.

Express Highways in the United States: a Bibliography.

Bibliography on Land Acquisition for Public Roads.

REPORTS IN COOPERATION WITH UNIVERSITY OF ILLINOIS

No. 313 . . . Tests of Plaster-Model Slabs Subjected to Concentrated Loads.

No. 314 . . . Tests of Reinforced Concrete Slabs Subjected to Concentrated Loads.

No. 315 . . . Moments in Simple Span Bridge Slabs With Stiffened Edges.

No. 345 . . . Ultimate Strength of Reinforced Concrete Beams as Related to the Plasticity Ratio of Concrete.

No. 346 . . . Highway Slab-Bridges with Curbs: Laboratory Tests and Proposed Design Method.

No. 363 . . . Study of Slab and Beam Highway Bridges. Part I.

No. 369 . . . Studies of Highway Skew Slab-Bridges with Curbs. Part I: Results of Analyses.

No. 375 . . . Studies of Slab and Beam Highway Bridges. Part II.

UNIFORM VEHICLE CODE

Act I.—Uniform Motor-Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor-Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor-Vehicle Civil Liability Act.

Act IV.—Uniform Motor-Vehicle Safety Responsibility Act.

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF SEPTEMBER 30, 1948

(Thousand Dollars)

Includes 1950 apportionment

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$15,784	$11,256	$5,572	228.3	$2,841	$1,435	132.5	$11,053	$6,009	243.4	$25,150	$13,016	604.2
Arizona	6,508	238	169	3.1	2,947	1,955	13.2	6,639	4,295	73.1	9,824	6,419	89.4
Arkansas	7,404	11,361	5,946	295.6	8,129	3,745	91.7	10,180	5,558	230.3	29,670	15,249	617.6
California	20,519	9,056	3,634	196.7	5,254	2,681	37.7	59,146	29,751	309.3	73,456	36,066	543.7
Colorado	9,666	4,189	2,603	61.4	3,998	2,451	50.1	12,508	7,201	186.2	20,695	12,255	297.7
Connecticut	4,521	11,985	5,813	24.1	3,859	2,164	2.9	5,240	2,437	25.7	21,084	10,414	52.7
Delaware	3,386	2,016	1,008	26.7	2,738	1,745	22.1	4,018	2,006	57.8	8,772	4,759	106.6
Florida	8,047	16,068	8,362	425.7	2,633	1,202	43.4	10,066	4,320	211.8	28,767	13,884	680.9
Georgia	10,495	10,072	4,963	296.1	20,654	9,292	226.7	33,217	17,124	855.4	63,943	31,379	1,378.2
Idaho	7,050	5,831	3,734	197.5	3,060	1,936	73.5	3,499	2,269	79.2	12,390	7,939	350.2
Illinois	29,320	47,018	24,023	603.0	17,878	8,914	196.5	44,935	22,142	662.3	109,831	55,079	1,461.8
Indiana	17,357	14,914	7,474	85.7	7,688	3,839	48.3	20,876	11,062	151.0	43,478	22,375	285.0
Iowa	12,290	14,814	6,095	590.3	8,857	3,993	355.8	22,407	10,957	912.6	46,078	21,045	1,858.7
Kansas	11,230	12,314	6,056	1,488.1	6,193	2,997	722.3	28,240	14,390	1,099.7	46,747	23,443	3,310.1
Kentucky	8,829	14,307	7,210	155.7	5,552	2,742	35.2	12,455	6,374	190.1	32,314	16,326	381.0
Louisiana	12,443	19,019	9,183	273.9	6,633	3,019	74.1	13,772	7,070	110.7	39,424	19,272	458.7
Maine	6,141	4,293	2,370	54.4	1,097	546	10.0	8,072	4,000	61.6	13,462	6,916	126.0
Maryland	4,706	8,718	4,028	46.7	7,693	3,418	22.6	9,576	4,840	49.7	25,987	12,286	119.0
Massachusetts	24,459	7,113	3,989	14.3	1,190	941	.4	20,963	10,813	41.2	29,266	15,743	55.9
Michigan	16,524	15,706	7,873	376.8	11,691	5,832	228.6	40,845	18,432	288.3	68,242	32,137	893.7
Minnesota	10,797	15,285	7,674	525.9	5,111	2,311	316.2	25,906	12,799	681.6	46,302	22,784	1,523.7
Mississippi	9,527	6,625	3,467	260.0	3,464	1,559	79.2	21,679	10,814	617.4	31,768	15,840	956.6
Missouri	15,965	32,235	15,600	906.6	11,497	5,678	169.3	24,150	13,206	559.7	67,882	34,484	1,635.6
Montana	10,786	12,328	7,581	483.2	5,475	3,280	127.6	10,783	6,459	422.9	28,586	17,320	1,033.7
Nebraska	10,534	13,330	6,947	487.2	5,007	2,691	132.6	12,775	6,116	557.6	31,112	15,754	1,177.4
Nevada	5,073	4,568	3,548	120.1	717	483	18.1	2,976	2,408	74.6	8,261	6,439	212.8
New Hampshire	3,303	3,214	1,578	23.7	1,508	992	8.0	4,415	2,229	30.8	9,137	4,799	62.5
New Jersey	9,648	7,733	3,592	23.1	6,372	3,130	9.4	24,642	12,743	43.9	38,747	19,465	76.4
New Mexico	7,023	7,227	4,614	311.9	2,794	1,817	65.6	5,360	3,422	86.3	15,381	9,853	463.8
New York	61,980	33,505	18,990	267.6	57,355	27,046	187.3	49,391	22,724	244.0	140,251	68,760	698.9
North Carolina	12,593	16,329	8,098	375.7	4,892	2,269	101.5	26,579	12,939	631.1	47,800	23,306	1,108.3
North Dakota	8,500	8,991	5,004	1,039.7	4,829	2,366	387.3	7,824	4,227	517.7	21,644	11,597	1,944.7
Ohio	22,741	49,147	23,842	345.6	13,055	6,558	115.5	35,826	18,425	113.6	98,028	48,825	574.7
Oklahoma	11,177	21,730	11,736	1,118.0	9,669	4,775	431.1	5,679	2,840	274.5	37,078	19,351	1,823.6
Oregon	6,515	7,021	3,649	70.4	1,669	855	24.6	9,969	5,105	205.7	18,659	9,609	300.7
Pennsylvania	23,940	24,882	12,825	32.6	14,840	7,074	52.5	74,119	37,673	237.9	113,841	57,572	323.0
Rhode Island	5,542	969	593	10.2	4,782	2,391	10.3	5,264	2,578	16.5	11,015	5,562	37.0
South Carolina	6,641	7,116	3,470	143.1	3,351	1,884	18.5	11,202	5,528	280.4	21,669	10,882	442.0
South Dakota	6,565	9,527	5,728	1,004.6	4,660	2,740	320.2	10,848	6,103	625.1	25,035	14,571	1,949.9
Tennessee	10,758	10,380	4,984	252.4	9,124	4,152	178.0	20,843	11,151	270.8	40,347	20,287	701.2
Texas	26,471	8,031	4,666	418.6	20,389	10,608	476.6	55,773	29,616	1,311.3	84,193	44,890	2,206.5
Utah	5,436	3,062	2,279	148.7	2,752	2,012	75.1	5,111	3,759	193.7	10,925	8,050	417.5
Vermont	2,222	3,242	1,618	51.0	1,100	549	18.9	4,703	2,354	67.7	9,045	4,521	137.6
Virginia	15,874	9,842	5,034	159.2	7,161	3,447	118.9	12,514	6,015	133.2	29,517	14,496	411.3
Washington	7,897	11,206	4,580	133.1	2,321	1,160	54.3	12,942	6,527	141.6	26,469	12,267	329.0
West Virginia	5,651	12,769	6,197	211.7	2,299	1,188	60.5	7,021	3,516	83.1	22,089	10,901	355.3
Wisconsin	15,459	21,243	10,289	341.3	7,047	3,107	141.9	28,899	12,953	460.2	57,189	26,349	943.4
Wyoming	4,470	2,263	1,470	60.4	1,992	1,301	57.8	6,046	3,972	199.5	10,301	6,743	317.7
Hawaii	2,832	6,841	3,412	48.3	3,014	1,254	.9	2,938	1,615	22.2	12,793	6,281	71.4
District of Columbia	2,701	270	135	.4	28	14		14,058	6,788	2.3	14,356	6,937	2.7
Puerto Rico	4,354	8,326	3,898	47.7	2,644	895	8.2	7,513	2,801	43.9	18,483	7,594	99.8
TOTAL	589,654	619,525	317,203	14,866.1	351,503	174,433	6,153.5	925,455	470,455	14,990.2	1,896,483	962,091	36,009.8