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

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A STUDY OF BITUMINOUS-COATED CORRUGATED SHEET METAL CULVERTS

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by J. Y. Welborn, Associate Highway Engineer, and P. J. Serafin, Assistant Chemical Engineer

IN THE CONSTRUCTION of certain sections on the Blue Ridge parkway during 1936–41, bituminous-coated corrugated sheet metal culverts were installed for the drainage of surface water. Preliminary inspection of these pipe was made by United States naval inspectors at the plants of the fabricators, at which time the erosion test was made. Samples of the pipe were then sent to the Public Roads laboratory, where the asphalt coating and galvanized metal were tested for conformity with the governing specifications.

Since these culvert pipe have been in service from 4 to 9 years it seemed advisable to determine their service behavior. This report covers the field inspection of the pipe in the fall of 1945 and laboratory analyses of the samples taken.

The types of bituminous-coated metal culverts and the sections of road on which they are used are given in table 1.

In order that the reader may distinguish between the types of culvert pipe used on the various sections and have a better understanding of the conditions found during the inspection, brief descriptions of the three general types of bituminous-coated sheet metal pipe are given below.

TABLE 1.—*Types of culvert pipe installed on the Blue Ridge parkway*

Type of culvert pipe	Section	Fabricator	Number of pipe culverts installed	Number of culverts inspected	Date of construction
Galvanized metal coated with asphalt and invert paved with asphalt	2M–1	A	79	26	1938–39
	2U–2	B	67	11	1939–40
Asbestos-bonded metal coated with asphalt and invert paved with asphalt	2M–2	A	38	11	1936–38
	2N–1	A	87	26	1936–38
	2P–2	A	110	31	1937–38
Galvanized metal coated with asphalt and invert paved with metal reinforced asphalt	2N–2	C	69	17	1940–41
	2P–1	C	(1)	7	(1)
	2V–1	C	45	11	1939–40

[1] Section not completed. Construction started in 1940, stopped in 1941.

TYPES OF BITUMINOUS-COATED CORRUGATED SHEET METAL CULVERT PIPE DESCRIBED

Galvanized metal pipe coated with asphalt and with an asphalt pavement.—Galvanized corrugated sheet metal pipe is uniformly coated with asphalt on the outside and the inside. One-fourth of the inside circumference, which forms the bottom of the pipe when installed, is coated with an additional thickness of asphalt such that the corrugations in the pipe are filled and a smooth pavement formed. The specifications required that the thickness of asphalt coating on the inside three-fourths of the circumference should be not less than three-hundredths inch and on the bottom quarter that the thickness should be sufficient to meet the requirements of the erosion test.

Asbestos-bonded metal pipe coated with asphalt and with an asphalt pavement.—In the process of galvanizing the metal sheet for use in the fabrication of this type of culvert pipe a layer of asbestos felt is rolled into the molten spelter coating on one or both sides of the sheet. In the pipe covered by this report the asbestos felt was on only one side of the sheet. On cooling, a portion of the asbestos felt becomes securely embedded in the coating, leaving a mass of fiber on the surface of the sheet. This surface is then primed with a bituminous material after which the sheets are fabricated into corrugated pipe. The asbestos-bonded surface formed the inside of the pipe inspected. As a final step in manufacture the pipe is coated with asphalt and the asphalt pavement formed as for the plain galvanized metal pipe.

It is claimed that the asbestos treatment assures permanent tight adhesion of the asphalt coating, increases the resistance of the asphalt coating to fracture on impact, and tends to cushion pressure caused by water freezing within the pipe.

Galvanized metal pipe coated with asphalt and with a metal reinforced asphalt pavement.—Galvanized corrugated sheet metal is used in the fabrication of this kind of pipe. The outside of the pipe and three-fourths of the inside circumference are uniformly coated with asphalt but the remaining fourth, which forms the bottom of the pipe when installed, is paved with asphalt reinforced with metal.

In fabricating the pavement a layer of asphalt-impregnated felt is placed inside the pipe over the corrugations and over this a sheet of metal, curved to fit the circumference of the pipe, is placed. The felt and the metal plate cover approximately one-fourth of the circumference of the pipe and are the same length as each corrugated section used in fabricating the pipe. The metal reinforcing plate is galvanized sheet iron, perforated on 2-inch centers across the width of the plate and the rows of perforations are spaced so that a row is directly over each valley of the corrugations when the plate is placed in the pipe. The plate is fastened to the pipe by rivets placed on approximately 4-inch centers across each end and one rivet on each edge near the center of the plate. The ends of sheets are lapped approximately 2 inches at each section junction. With the metal reinforcement in place each length of pipe is coated with asphalt. The hot asphalt fills the valleys of the corrugations between the layer of felt and the pipe and an additional thickness of bituminous material is deposited on the surface of the metal plate, thus forming the metal reinforced asphalt pavement. The reinforcing plate in the pipe used on the Blue Ridge parkway had a spelter coating of 0.68 ounce per square foot and was approximately 26 gage.

In November 1939 a sample of bituminous-coated corrugated sheet metal pipe with reinforced asphalt pavement was submitted from section 2V–1 for examination of the paved invert and conformity with the specifications. On examining the invert it was observed that the crests of the corrugations, where the felt made

TABLE 2.—*Inspection data for asphalt-coated galvanized metal culvert pipe with asphalt paved invert*

Section and station No.	Diameter of pipe	Length of pipe	Gage of metal	End Protection[1] Left	End Protection[1] Right	Grade of invert	Soil condition around culvert	Present flow of water through culvert	Condition of paved invert	Condition of coating inside pipe above invert: Appearance	Loss	Adhesion	Condition of coating on outside of pipe	Condition of exposed metal
Section 2M-1:	*Inches*	*Feet*				*Percent*								
18+68	24	126	14	HW	HW	55.8	Damp	None	Cracked, low loss	Checked	Low	Fair		Slightly rusted.
44+00	18	99	16		HW		do	do	Badly cracked	Good		do	(2)	
55+00	18	50	16	DI		2.0	do	do	Checked	Good, some flow		do	(2)	Slightly rusted.
63+73	18	44	16	DI		4.9	do	do	do	Good		do	(2)	
73+96	18	42	16	DI		.2	do	do	Cracked	do		do	(2)	
84+85	24	149	14	HW		36.0	do	do	Good	Good, some flow		do	(2)	
98+84	18	176	14	HW		37.1	do	do	Cracked	Good		do	(2)	
112+05	18	113	16	DI	HW	47.1	do	do	Good	do		do		
124+67	18	39	16	DI	HW	5.7	do	do	Slightly cracked	Good, some flow		do		
141+94	18	50	16	DI		15.0	do	do	Badly cracked, loose	Good		do	(2)	
152+23	18	60	16		DI	6.3	do	Slight	Badly cracked	do		do	(2)	
168+11	18	52	16	HW	DI	17.4	do	None	Good	do		do		
185+73	18	162	14	HW	HW	21.8	do	Slight	Checked	do		do		
204+50	24	83	14	HW	HW	7.1	do	None	Good	Good, some flow		do		
221+50	18	43	16	HW	DI	10.3	do	do	do		Low	do		
243+64	18	42	16		DI	3.3	do	do	Badly cracked, loose	Good		do	(2)	
255+00	18	53	16	DI		1.7	do	do	Cracked		Medium[3]		(2)	Good.
266+00	18	38	16	DI	HW	6.7	do	do	Slightly checked	Good		Fair		
293+78	18	112	16		DI	35.0	do	do	Badly cracked, some loose		Low	do	(2)	
306+47	18	58	16	DI		6.2	do	do	Badly cracked	Good		do	(2)	
328+35	18	78	16	HW	HW	26.9	do	do	Slightly cracked	Checked		do		
351+00	30	218	12	HW		48.9	do	Slight	do	Good, some flow		do	(2)	
356+57	30	182	12	HW		32.5	do	do	Slightly cracked, low loss	do		do	(2)	
375+57	18	144	14	HW		52.0	do	None	Good	Good		do	(2)	
385+28	24	128	14	DI	HW	48.9	do	do	do	do		do		
399+80	18	45	16	DI		15.1	do	do	Badly cracked, some loose	do		do	(2)	
Section 2U-2:														
287+00	18	65	16		HW	39.0	do	Slight	Cracked, loose, high loss[3]	do		Poor	(2)	Slightly rusted.
306+50	24	32	14	HW	DI	12.2	do	do	Cracked, loose, loss[3]	do		do		
326+50	18	40	16		DI	.2	do	do	do	Checked	Low	do	(2)	
343+75	18	56	16		DI	36.6	do	None	Badly cracked, loose	do		do	(2)	
346+90	24	114	14		HW	38.0	do	do	Cracked, loss[3]	Good		do	(2)	
361+00	18	54	16		DI	17.0	do	do	Cracked, loss		Low	do	(2)	
380+00	18	41	16	HW	DI	9.1	do	do	Cracked		do	do		
396+00	18	34	16	HW	DI	4.4	do	Slight	Badly cracked, low loss	Good		do		
419+75	18	46	16	DI	HW	13.6	do	Small	Cracked	do		do		
450+00	18	91	16		HW	46.6	do	None	Cracked, high loss[3]		High[3]	do	(2)	Rusted.
537+95	18	49	16	DI	HW	20.4	do	do	Checked, low loss	Good		do		

[1] HW = Head wall. DI = Drop inlet.
[2] Where exposed, the outside coating was badly checked and brittle. Where covered by fill, coating was badly checked and had very poor adhesion to metal.
[3] Loss was throughout entire length.

TABLE 3.—*Inspection data for asphalt-coated asbestos-bonded metal culvert pipe with asphalt paved invert*

Section and station No.	Diameter of pipe	Length of pipe	Gage of metal	End protection [1]		Grade of invert	Soil condition around culvert	Present flow of water through culvert	Condition of paved invert	Condition of coating inside pipe above invert			Condition of coating on outside of pipe	Condition of exposed metal
				Left	Right					Appearance	Loss	Adhesion		
Section 2M-2:	*Inches*	*Feet*				*Percent*								
425+94	18	67	16		DI	32±	Damp	None	Cracked	Good		Good	[2]	
440+06	18	47	16		HW	1±	do	do	do	Checked		Fair	[2]	
464+00	18	60	16	DI		5±	do	do	do	Good		Good	[2]	
495+44	24	94	14	HW			do	do	do		Low		[2]	Asbestos intact.
505+19	18	89	16	DI		47±	do	do	Transverse cracks entire length	Good		Good	[2]	
514+50	18	43	14	DI		5±	do	do	Slightly cracked	do		do	[2]	
526+65	18	57	16		DI	5±	do	do	Cracked, loose	Checked	Low		[2]	Do.
535+80	18	50	16		HW	8±	do	do	Cracked	Good		Good	[2]	
550+84	18	68	16		HW	9±	do	do	do	do		do	[2]	
578+25	24	44	14	HW		10±	do	do	Cracked entire length	Checked	Low		[2]	Do.
594+45	24	154	14	HW		35±	do	Medium	Cracked	do	do		[2]	
Section 2N-1:														
9+99	18	122	14	HW		19±	do	None	[3]		do		[2]	Do.
20+79	18	49	16	HW		6±	Dry	do	Cracked	Good		Good	[2]	
43+50	18	48	16	DI		3.0	Wet	Slight	Checked, some loose	do		do	[2]	
56+50	18	107	14		HW	34±	Damp	None	Checked	do		do	[2]	
69+36	18	78	16		DI	48±	do	do	do	do		do	[2]	
79+75	18	89	16		HW	38.0	do	do	Badly cracked	do		do	[2]	
90+25	18	52	16		HW	24.1	do	do	Cracked	do		do	[2]	
114+50	18	85	16		DI	44.1	do	do	do	do		do	[2]	
142+29	18	176	16		HW	53.0	Wet	Medium	Cracked at inlet end, some loose	Checked		Poor	[2]	
151+30	24	225	14		HW		Damp	None	Good	Good		Good	[2]	
166+74	18	45	16		DI	7.5	do	do	Cracked, loose	do		do	[2]	
176+45	18	63	16		DI	3.9	do	do	Cracked	do		do	[2]	
194+00	18	85	16		HW		do	do	Badly cracked	do		do	[2]	
207+35	24	81	14		DI	5.4	do	do	Cracked, loose	do		do	[2]	
219+50	18	108	16		HW		do	Slight	do	do		do	[2]	
298+80	18	64	16	DI			do	None	Checked	do		do	[2]	
309+09	18	153	16	DI		57.7–59.5	do	do	Good	do		do	[2]	
317+92	18	67	16	HW		4.1	do	Small	Checked	do		do	[2]	
331+00	18	96	16	HW			do	None	do	do		do	[2]	
341+79	30	140	12	HW			do	do	Checked entire length	do		do	[2]	
354+51	30	74	12	HW		26.4	do	Small	do	Good, some flow		do	[2]	
382+30	18	50	16	DI		17.0	do	None	Checked	Good		do	[2]	
391+24	18	56	16	DI		3.5	do	do	do	do		do	[2]	
401+50	24	46	14	DI		10.5	do	do	do	Good, some flow		do	[2]	
412+49	24	69	14	HW		29.1	do	do	do	Good		do	[2]	
440+75	18	80	16	DI		39.3	do	do	Low loss, spalling	do		do	[2]	Do.

[1] HW=Head wall. DI=Drop inlet.
[2] Where exposed, the outside coating was badly checked and brittle. Where covered by fill, coating was badly checked and had very poor adhesion to metal.
[3] Culvert filled with debris; invert could not be inspected.

TABLE 4.—*Inspection data for asphalt-coated asbestos-bonded metal culvert pipe with asphalt paved invert*

Section and station No.	Diameter of pipe	Length of pipe	Gage of metal	End Protection [1]		Grade of invert	Soil condition around culvert	Present flow of water through culvert	Condition of paved invert	Condition of coating inside pipe above invert			Condition of coating on outside of pipe	Condition of exposed metal
				Left	Right					Appearance	Loss	Adhesion		
Section 2P-2:	*Inches*	*Feet*				*Percent*								
406+39	18	62	16		DI	3.0	Damp	None	Good		Medium	Good	(2)	Asbestos intact.
408+50	18	98	16		DI		do	Small	Badly cracked	Checked	High		(2)	Do.
417+50	18	48	16		HW	3.0	do	None	Cracked	Good		Good	(2)	
428+99	18	55	16		DI	28.1	do	do	Good	do		do	(2)	
449+63	18	42	16		HW	3.0	do	do	do		Low		(2)	
466+50	24	52	14		HW	17.7	do	Slight	Badly cracked, loose [3]	Good		Good	(2)	
477+37	24	46	14		HW	12.3	do	Small	Cracked	do		do	(2)	
501+99	18	58	16		HW	20.4	do	None	do	do		do	(2)	
517+50	18	48	16		HW	2.9	do	do	Good	do		do	(2)	
530+54	18	48	16		HW	3.1	do	do	Checked	do		do	(2)	
544+00	18	78	16		DI	40.0	do	do	Good	do		do	(2)	
567+50	18	46	16		DI	23.7	do	Slight	Badly cracked, loose	do		do	(2)	
587+22	18	46	16		HW	4.2	do	None	Cracked, loose	do		Poor	(2)	
602+50	18	38	16		HW	4.1	do	do	Good [4]	Good [4]			(2)	Badly rusted in invert.
633+95	18	46	16		HW	21.4	do	Medium	Good	Good		Poor	(2)	
642+82	18	36	16		HW	6.1	do	None	Cracked	do		Good	(2)	
654+75	18	75	16		DI	32.6	do	do	Good	do		do	(2)	
664+27	24	72	14		HW	34.7	do	do	Good [4]	Good [4]			(2)	Good.
686+50	30	82	12		HW	30.7	do	do	Good	do	Low		(2)	
696+39	24	40	14		HW	3.4	do	do	Badly cracked	Good		Good	(2)	
710+00	18	76	16		DI	2.8	do	do	do	do		do	(2)	
719+66	30	118	12		HW	31.2	do	do	Good	Good, some flow		do	(2)	
728+90	18	56	16		DI	3.0	do	do	Checked	Good		do	(2)	
739+03	18	36	16		DI	5.2	do	do	Good	do		Poor	(2)	
759+97	30	84	12		HW	19.2	do	Slight	do	Good, some flow		Good	(2)	
767+90	18	78	14		DI	32.5	do	None	do	Good		do	(2)	
796+95	18	103	16		HW	10.0	do	do	Checked	do		do	(2)	
807+25	18	67	16		DI	3.0	do	do	Good	Good, some flow		do	(2)	
816+67	18	89	16		HW		do	do	Checked	Checked		do	(2)	
828+00	18	49	16		DI	28.9	do	do	Cracked	Good		do	(2)	
839+00	18	82	16		DI	43.8	do	do	do	do		do	(2)	

[1] HW=Head wall. DI=Drop inlet.
[2] Where exposed, the outside coating was badly checked and brittle. Where covered by fill, coating was badly checked and had very poor adhesion to metal.
[3] At left end one length of asphalt-coated galvanized metal pipe was used; remainder was asbestos-bonded pipe.
[4] At left end one length of uncoated galvanized pipe was used; remainder was asbestos-bonded pipe and appeared to be in good condition.

Table 5.—*Inspection data for asphalt-coated galvanized metal culvert pipe with reinforced asphalt invert*

Section and station No.	Diameter of pipe	Length of pipe	Gage of metal	End protection[1]		Grade of invert	Soil condition around culvert	Present flow of water through culvert	Condition of asphalt coating in invert			Condition of exposed metal reinforcing plate	Condition of coating inside pipe above invert			Condition of coating on outside of pipe
				Left	Right				Appearance	Loss	Adhesion		Appearance	Loss	Adhesion	
Section 2N-2:	*Inches*	*Feet*				*Percent*										
447+75	18	115	16	HW		37.7	Damp	None	Badly broken	Medium	Poor	Loose at ends, some rusting	Good	None	Fair	(2)
462+52	18	72	16	DI		11.1	Wet	Small	do	do	do	Some rusting	do	do	do	(2)
466+53	18	60	16	HW		27.9	Damp	do	Good	None	Fair		do	do	do	(2)
474+50	18	46	16	HW	DI	16.3	do	None	Badly broken	Medium	Poor	Intact, no rusting	Checked	do	do	
485+04	18	43	16	HW	DI	10.0	do	do	Checked	None	Fair		Good	do	do	
490+89	18	100	16		DI	45.0	do	Small	Badly broken	High	Poor	Loose at ends	do	do	do	(2)
504+28	24	79	14		HW	38.9	do	do	do	do	do	Intact, no rusting	do	do	do	(2)
514+63	24	50	14		DI	3.6	do	None	Good	None	do		do	do	do	(2)
533+88	18	61	16	HW	DI	35.6	Wet	do	Cracked	do			do	do	do	
535+75	24	77	14	HW	DI	24.8	do	Medium	Badly broken	Very high	Poor	Loose at ends, slight rusting	do	do	Poor	
537+71	30	70	12	HW	DI	20.4	Damp	Slight	Cracked	Medium	do	Loose at ends, pipe ¼ blocked	do	do	Fair	
544+32	18	57	16		DI	23.5	Wet	Small	Badly broken	do		Loose, badly rusted	do	do	do	(2)
576+28	30	48	12	HW	DI	7.9	Damp	do	Cracked	do		Intact, no rusting	do	do	do	
596+18	30	70	12	HW	HW	28.6	do	do	Good	None			do	do	do	(2)
617+64	30	80	12		DI	50.8	do	do	Badly broken	High	Poor	Loose, turned up at end	do	do	do	(2)
643+57	24	92	14		DI	36.6	do	do	Good	None			do	do	do	(2)
676+50	24	54	14	HW	DI	27.2	do	Large	do	do			do	do	do	
Section 2P-1:																
201+00	18	42	16		HW		Damp	None	Badly broken	Medium		Intact, no rusting	do	do	do	(2)
216+00	18	42	16		HW		do	do	Good	None			do	do	do	(2)
341+20	18	59	16				do	Medium	do	do	Poor		do	do	do	(2)
354+00	18	105					do	Small	do	do			do	do	do	(2)
381+00	30	134	12				do	Medium	Badly broken	High		Intact, no rusting	do	do	do	(2)
392+55	18	123			HW		do	None	(3)				do	do	do	(2)
399+45	18	48	16		HW		do	Slight	Badly broken	Medium		Intact, no rusting	do	do	do	(2)
Section 2V-1:																
140+17	24	104	14		HW	35.1	do	None		Low	Good	Intact, some rusting	do	do	Good	(2)
143+00	18	37	16		DI	4.0	do	do	Cracked	None	do		do	do	do	(2)
153+70	18	95	14		HW	21.7–33.0	do	do		Slight	do	Intact, no rusting	do	do	do	(2)
166+65	18	96	14		DI	66.6	do	do		High		do	do	do	do	(2)
174+35	24	73	14		HW	30.8	do	do		do		Loose, turned up at ends	do	do	do	(2)
191+38	18	49	16		DI	10.0	do	Slight	(3)				do	do	do	(2)
204+50	18	48	16		DI	5.0	do	None	Broken	Medium		Intact, badly rusted	do	do	do	(2)
223+46	24	36	14	DI		7.5	Wet	Medium		High			do	do	Poor	
245+25	18	71	16	HW		14.1–22.0	do	do		do			do	do	do	
262+70	18	93	14	HW		37.5	Damp	None	Checked	None	Poor		do	do	do	
274+38	18	99	14		HW	17.0–19.1	do	Medium		Medium	do		do	do	do	(2)

[1] HW=Head wall. DI=Drop inlet.
[2] Where exposed, the outside coating was badly checked and brittle. Where covered by fill, coating was badly checked and had very poor adhesion to metal.
[3] Outlet end of culvert partially filled and condition of paved invert could not be examined.

FIGURE 1.—TYPICAL CONDITION OF FLOW FROM CRESTS OF CORRUGATIONS IN THE TOP QUARTER INSIDE THE CULVERT PIPE.

contact, were not coated with asphalt. Also that the underside of the reinforcing plate, which was in contact with the felt, was not uniformly coated. Although the metal plate was perforated the space between the felt and the plate was too small to permit the hot asphalt to completely fill it.

LARGE NUMBER OF FIELD INSTALLATIONS INSPECTED

Because of the large number of culvert pipes it was not considered necessary to inspect all of them. To get a general idea of the condition of the pipes it was planned to inspect every third installation. This plan was not adhered to strictly because of inaccessibility of some of the outlets or because some of the culverts were filled with debris. The plan of inspection was altered in some instances to include culverts of special interest.

A separate report form was used for each culvert inspected to record data pertaining to the type and condition of the installation.

The date of installation, diameter and length of culvert pipe, gage of metal, type of end protection, and grade of the pipe invert were obtained from the records for each section of road. The soil surrounding the culvert was observed only as to its moisture content. The quantity of water flowing through the pipe was noted. Since inspection was made at a relatively dry time of the year the flow was considered continuous if there was water flowing through the pipe when inspected, and the flow was estimated on a comparative basis.

The condition of the paved invert and of the coating above the pavement and on the outside of the pipe was determined. Where there had been a loss of coating either in the paved invert or on the walls of the pipe, the condition of the exposed metal was observed. Drop inlets on many of the culverts made it difficult to make a thorough examination of the coating and pavement of the upper ends. Where possible, inspections were made from both ends or by crawling through the pipe.

DESCRIPTIVE TERM DEFINED

The data from the inspection report for each culvert were tabulated for each project and combined for each of the three general types of bituminous-coated pipe used. The inspection data are given in tables 2, 3, 4, and 5. Explanation of the terms used in these tables is given below.

Good.—Very little or no checking, no cracking, and no apparent loss of asphalt coating.

TABLE 6.—*Summary of conditions of paved inverts and coating on inside walls of all culverts inspected*

Section No.	Total culverts inspected	Number of culverts with paved invert: Good	Checked, no loss	Cracked or broken, no loss	Loss of coating	Number of culverts with inside walls: Good	Checked, no loss	Loss of coating
Asphalt-coated galvanized metal pipe with asphalt paved invert:								
2M-1	26	7	4	13	2	21	1	4
2U-2	11	0	0	3	8	6	1	4
Asphalt-coated asbestos-bonded pipe with asphalt paved invert:								
2M-2	11	0	0	11	0	6	1	4
2N-1	[1] 26	2	12	10	1	24	1	1
2P-2	31	15	4	12	0	26	1	4
Asphalt-coated galvanized metal pipe with reinforced asphalt invert:								
2N-2	17	5	1	1	10	16	1	0
2P-1	[1] 7	3	0	0	3	7	0	0
2V-1	[1] 11	0	1	1	8	11	0	0

[1] Outlet end of one culvert was filled; invert was not inspected.

Checked.—Fine hair cracks of insufficient width to permit large infiltration of water and silt. In the asphalt pavement checking usually occurred directly over the crests of the corrugations and often extended the full width of the invert. On the inside walls above the pavement and on the outside of the pipe, checking generally had the pattern of alligator skin.

Cracked.—This term is used to describe the condition of the asphalt pavement and denotes cracks of sufficient width to permit rapid infiltration of water and silt. In some culverts cracks often exceeded one-half inch in width for the full depth of the asphalt pavement. Cracking generally occurred above the crests of the corrugations, although in some pipes there were large longitudinal cracks. In some pipes where the pavement was severely cracked, the infiltration of water and silt had caused a loss of adhesion between the asphalt and the metal with subsequent loosening of pieces of the pavement.

Broken.—This term is used to describe the condition of the asphalt coating on the reinforcing plate (where used) and signifies severe cracking and breaking of the coating into small pieces.

Loss.—In the invert, loss was usually caused by pieces of the asphalt pavement first cracking and then being loosened and dislodged by flowing water.

On the inside walls above the pavement, loss appeared to be due to either erosion, impact, or stripping of the coating from the metal by the action of water.

Adhesion.—Adhesion of the asphalt to the metal was not determined accurately but a comparative indication was obtained by noting the ease with which the coating could be removed with a putty knife.

Flow.—In some culverts a displacement of the asphalt coating by flow was noted. This condition occurred at the top of the pipe and was in the form of strings of asphalt hanging from the crests of the corrugations. Flow is illustrated in figure 1, which shows the coating cut from the corrugations, reassembled, and photographed.

In the two types of pipes where the pavement consisted wholly of asphalt, the most deterioration of the

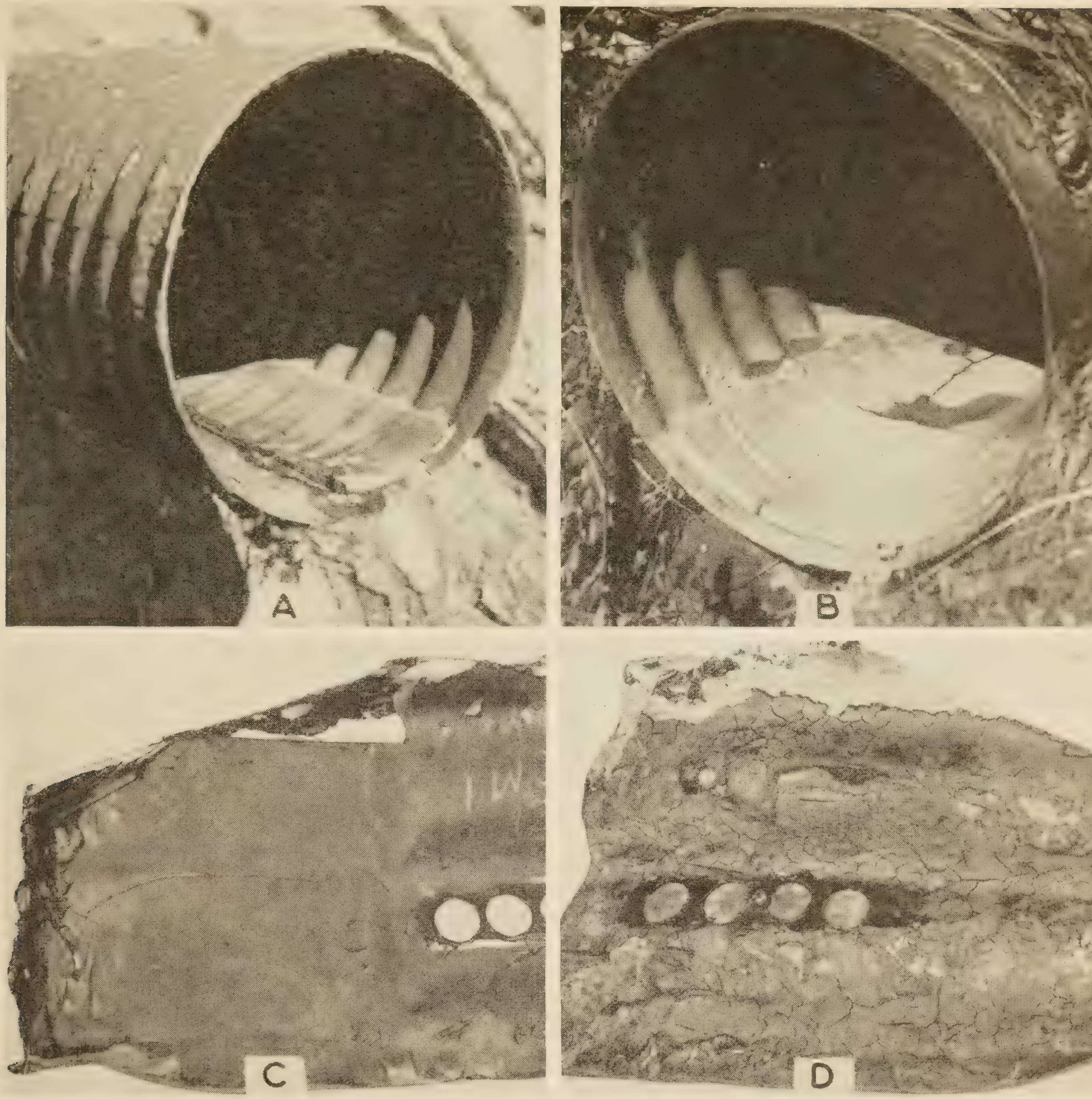

FIGURE 2.—BITUMINOUS-COATED GALVANIZED METAL PIPE: (A) SEVERE LONGITUDINAL CRACKING; (B) CRACKING OF ASPHALT PAVEMENT; (C) INSIDE VIEW OF SAMPLE CUT FROM OUTLET END OF PIPE. CONDITION APPEARS TO BE GOOD BUT ADHESION TEST (ROUND SPOTS) SHOWS LOW ADHESION; AND (D) OUTSIDE OF SAMPLE SHOWN IN (C). THIS PORTION OF THE PIPE WAS IN CONTACT WITH SOIL. TESTS SHOWED POOR ADHESION.

pavement and coating on the inside walls was found to be at the outlet end of the culverts which was more exposed to sunlight and changes in temperature. The condition of the invert, inside and outside coating, and exposed metal given in tables 2, 3, and 4 are for the outlet end except where otherwise indicated. In the culvert pipe with the reinforced asphalt pavement the deterioration of the asphalt coating and metal in the invert was, in general, more uniform throughout the entire length of the pipe. Therefore, these conditions as shown in table 5 apply to the full length of the culvert pipe.

Table 6 gives a summary of the condition of the paved inverts and inside walls by road sections. There follows a discussion of the data on the performance of each type of pipe on the different sections of road.

PLAIN ASPHALT-COATED PIPE

Section 2M–1.—At the time of the inspection the asphalt coating and pavement were considered to be giving adequate protection. No culvert pavements or coatings had failed but conditions were found that may be indicative of future failures. Twenty-six culverts

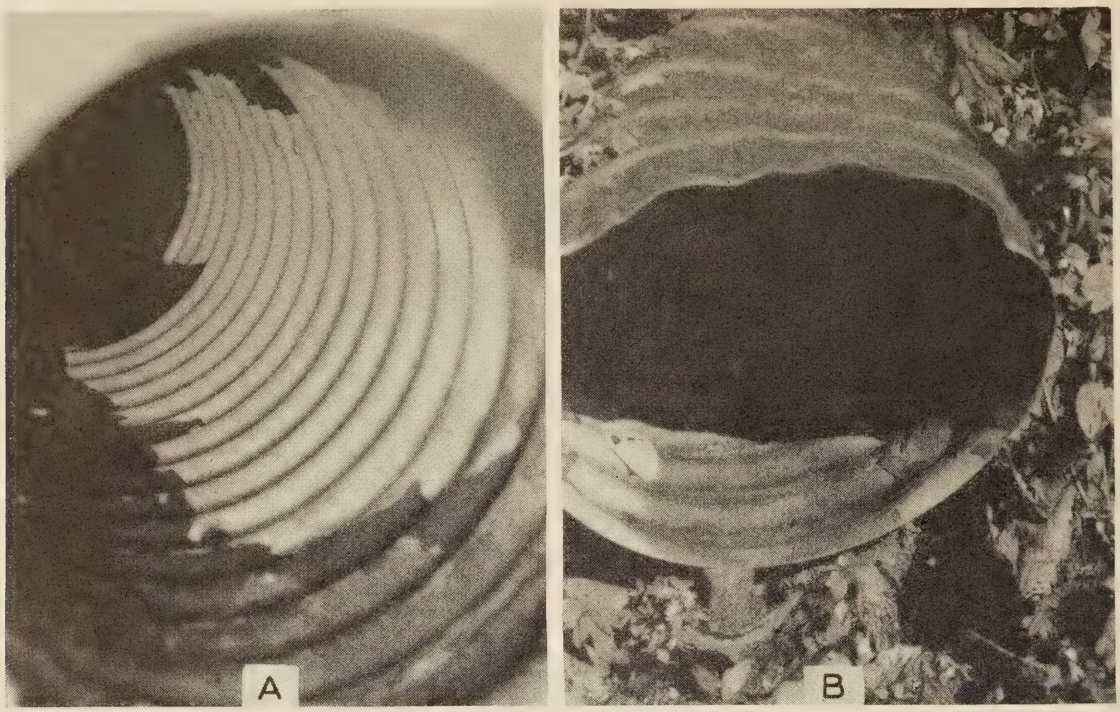

Figure 3.—A, Asphalt Coating Stripped From Wall by Water Surging Through the Culvert on 47 Percent Grade; B, Loss of Pavement From Same Culvert With Rusting of Exposed Metal.

were inspected. The pavements in 7 culverts were rated good, 4 were checked, 13 were cracked only, and 2 showed sufficient loss of coating to expose the metal. Typical conditions are shown in figure 2.

The coatings on the inside of the pipes above the inverts were generally in good condition but appeared to have only fair adhesion to the metal. In this respect, 21 culverts were rated good, 1 was checked only, and 4 showed some loss of coating.

Section 2U–2.—In general the condition of the culvert pipe installed on section 2U–2 was much worse than on section 2M–1 and for some culverts the coating was not considered to be giving adequate protection to the metal. The adhesion of the asphalt to the metal appeared to be poor. Of the 11 culverts inspected none of the paved inverts was rated as good, 3 were cracked, and 8 had lost some of the coating.

The inside coatings above the inverts also showed more deterioration than on section 2M–1. For 6 of the 11 culverts inspected the coatings were good, 1 was checked only, and 4 showed some loss of coating. One culvert, which had an extremely high loss of coating on the inside walls, is of special interest. This culvert was installed on a 25° skew on a grade of 47 percent with a head wall at the inlet end. It appeared that during heavy rainfall water ran down the gutter and into the pipe at high velocity. It struck the side of the pipe above the pavement at an angle and had stripped a large area of the asphalt coating from the side wall. This condition was repeated on alternate right and left sides of the pipe for a considerable distance down the pipe. Figure 3, A shows the high loss of coating from the inside wall near the inlet. The loss of asphalt coating from the pavement is illustrated in figure 3, B.

BITUMINOUS-COATED ASBESTOS-BONDED PIPE

Bituminous-coated asbestos-bonded metal culvert pipe was used on sections 2M–2, 2N–1, and 2P–2 Although there was considerable checking and cracking of the asphalt pavements near the outlet, all the inverts were giving good service. In comparison with sections 2M–1 and 2U–2 there were fewer cracked areas that were loose or easily removed. Where there was a loss of asphalt the asbestos sheet was generally intact and appeared to give some additional protection to the metal. The coatings on the inside walls above the pavements seemed to have better adhesion to the asbestos-bonded metal than did the asphalt on the plain galvanized metal on section 2M–1. A summary of the condition of the pavements and inside coatings in the culverts where the asbestos-bonded metal was used follows.

Section 2M–2.—Eleven culverts were inspected and all the inverts were found to be cracked but there was no loss of coating. The coating on the inside walls of 6 culverts was rated good, 1 was checked only, and on 4 there was some loss.

Section 2N–1.—Twenty-six culverts were inspected. The paved inverts of 2 were regarded as good, 12 were checked, 10 were cracked, 1 showed some loss of asphalt, and the condition of 1 invert could not be ascertained. The coating on the inside walls of 24 culverts was considered good, 1 showed some checking and 1 some loss of asphalt.

Section 2P–2.—Thirty-one culverts were inspected. The paved inverts in 15 were good, 4 were checked, and 12 were cracked. There was no loss of pavement in any of these pipes. In 26 of the culverts the coating on the inside walls was considered good, 1 was checked only, and 4 had some loss.

Typical conditions of the culvert pipe installed on sections 2M–2 and 2N–1 are shown in figures 4 and 5.

PIPE WITH REINFORCED PAVEMENT

Bituminous-coated corrugated sheet metal culverts with metal reinforced asphalt pavement were installed on sections 2N–2, 2P–1, and 2V–1. Before the inspec-

Figure 4.—Bituminous-Coated Asbestos-Bonded Pipe Showing Severe Cracking of Pavement at Outlet End.

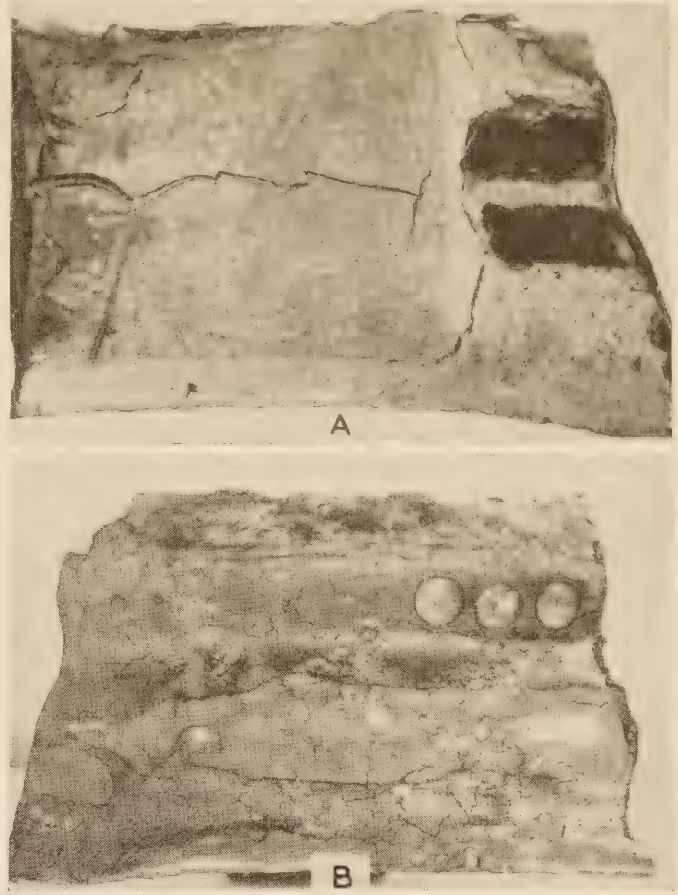

Figure 5.—Sample Taken From Outlet End of Bituminous-Coated Asbestos-Bonded Metal Pipe. (A) Inside Shows Cracked Pavement; (B) Outside Shows General Condition of Coating Which has Been in Contact With Soil Surrounding Culvert.

tion some difficulty had been experienced by the maintenance men in keeping some of the culverts on section 2N–2 in serviceable condition due to failure of the inverts. Inspection of the culverts revealed that in some culverts there had been a high loss of coating from the paved invert, exposing the metal reinforcing plate to the action of water and erosion. Subsequent action of water and erosion through the pipes caused the exposed reinforcing plates to loosen and pull away from the rivets holding them to the pipe. Because of the light weight of the reinforcement the loosened ends were turned up and eventually formed blocks within the pipes. This condition usually occurred at the upper end of the section lengths where they were connected by bands.

A summary of the conditions found in culverts where the metal reinforced pavement was used follows.

Section 2N–2.—Of the 17 culverts examined there were 5 in which the pavement was considered good, 2 were checked or cracked only, and in 10 culverts the asphalt was cracked or broken and showed considerable loss. In 6 culverts the exposed metal reinforcing plates were partially turned up, and in 4 culverts they were rusted. The coatings on the insides of the pipes above the pavement were in good condition and appeared to have fair adhesion to the metal.

Section 2P–1.—Since construction of this section had not been completed only 7 culverts were inspected. The paved inverts of 3 were in good condition, 3 were badly broken with some loss of coating, and 1 invert could not be examined. The metal reinforcement in all culverts was in place. The inside walls were in good condition and the coatings appeared to have fair adhesion to the pipe.

Section 2V–1.—Eleven culverts were inspected. Eight had lost coating from the inverts and the exposed plates in 2 of these were rusted. The asphalt coating in the inverts of 2 culverts was checked or cracked. The metal reinforcing plates appeared to be in place. The condition of the invert of one culvert could not be ascertained. The inside walls of all culverts inspected were in good condition but in some culverts the coating appeared to have better adhesion to the metal than in others.

The conditions of some of the culvert pipe installed on sections 2N–2, 2P–1, and 2V–1 are illustrated by figure 6.

OUTSIDE COATINGS SEVERELY CHECKED

The asphalt coating on the outside of the culvert pipes was found to be in the same general condition on all the sections inspected. Where the outlet ends of the culverts were exposed, the asphalt coating on the outside was hard and severely checked but on most culverts there was good adhesion to the metal. On some culverts there was loss of coating, which appeared to be due to impact of rocks from the slopes above the outlets.

Where it was possible to dig away the cover material and examine the asphalt coating under the fill, it was found to be severely checked and to have very low or no adhesion to the metal. At the time of the inspection the soil surrounding the culverts usually was damp or wet and it is probable that it remains in this condition most of the time.

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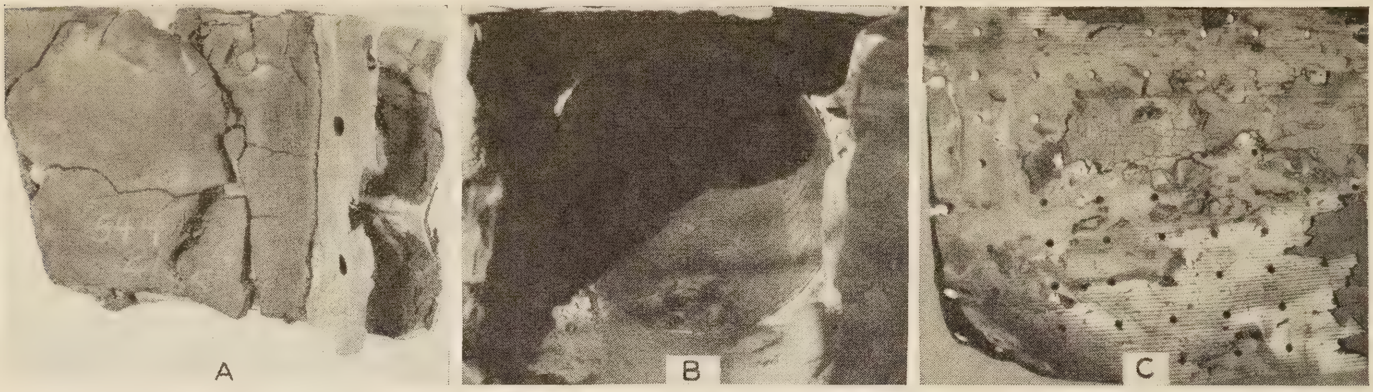

Figure 6.—Metal-Reinforced Asphalt Paving. (A) Sample From Outlet End Showing Loosening of Reinforcement; (B) Coating on Reinforcing Plate Badly Broken With Some Loss; and (C) Section of Reinforcing Plate Showing Loss of Coating and Rusting of Exposed Metal.

LABORATORY TESTS MADE ON FIELD SAMPLES

Samples, representing the three general types of bituminous-coated metal pipe, were cut from the culverts for analysis in the laboratory. Samples of asphalt were also taken from the paved inverts or from the inside walls.

During the construction of Blue Ridge parkway, samples of culvert pipe had been submitted to the laboratory to be tested for conformity with the specifications. Since the only test of asphalt required by these specifications was determination of solubility in carbon disulphide, other characteristics of the asphalt at the time of installing the pipe are not known. Because of this lack of information, a satisfactory comparison between the characteristics of the original asphalt coating and the samples taken at this time cannot be made.

A complete analysis of all asphalt samples was made and the results are shown in table 7. Considering the effects of variations in exposure of the different samples, the average results of tests on samples from the same fabricator of the coated pipe, as shown in table 8, indicate that different asphalts were used by the three fabricators. Analyses of seven typical blown asphalts of the type used or intended for use as pipe coating materials were also made and the test values are given in table 9. These asphalts were submitted either by the fabricator of the bituminous-coated pipe or by the producer of the asphalt. Although the analyses of the field samples given in tables 7 and 8 do not correspond exactly to those of any of the seven typical asphalts, both groups of asphalts are of the same general type.

In 1945 the American Railway Engineering Association [1] issued a specification for bituminous-coated corrugated metal pipe and arches. In this specification a test was proposed for the determination of the adhesion of bituminous coating to metal, the determination to be made on samples taken from the coated pipe, delivered or about to be delivered to the purchaser. To make

[1] Committee Report, American Railway Engineering Association, Bul. 451, February 1945, vol. 46, No. 451, p. 493.

Table 7.—*Analyses of samples of asphalt coating taken from culvert pipes*

Section and station No.	Penetration, 100 gm, 5 sec.			Ductility 5 cm. per min. 25° C.	Softening point	Specific gravity, 25°/25° C.	Solubility in CS_2			Bitumen insoluble in 86° B. naphtha	Oliensis test	
	At 15° C.	At 25° C.	At 35° C.				Bitumen	Organic insoluble	Inorganic insoluble		Character of spot	Naphtha-xylene equivalent
Section 2M–1:				*Cm.*	*° C.*		*Percent*	*Percent*	*Percent*	*Percent*		*Percent*
18+68	29	44	64	2.0	98.5	0.999	99.71	0.10	0.19	32.15	Positive	5–10
98+84	31	47	71	2.0	95.5	.999	99.48	.12	.40	30.82	do	10–15
141+94	27	42	64	1.75	96.5	.999	99.67	.10	.23	31.20	do	5–10
152+23	27	41	62	1.75	100.0	1.000	99.73	.21	.06	32.51	do	5–10
204+50	23	35	55	2.0	99.0	1.007	99.08	.19	.73	34.17	do	10–15
Section 2U–2, 306+50	23	36	51	1.75	99.0	.988	99.80	.16	.04	31.82	Negative	
Section 2M–2, 526+65	26	41	61	2.0	97.3	1.003	99.41	.08	.51	32.77	Positive	5–10
Section 2N–1:												
79+75	27	40	60	1.75	98.3	.999	99.82	.13	.05	32.29	do	0–5
166+74	30	45	68	1.75	98.5	1.000	99.72	.12	.16	30.74	do	15–20
207+35	26	40	60	1.75	101.5	1.001	99.35	.16	.49	31.84	do	5–10
Section 2P–2:												
719+66	28	43	63	2.0	99.5	.998	99.68	.15	.17	32.82	do	5–10
466+50	25	40	63	1.75	98.5	1.000	99.66	.11	.23	34.83	do	5–10
587+22	26	39	59	1.75	100.8	.998	99.75	.10	.15	33.36	do	0–5
Section 2N–2:												
447+75	18	26	38	[1].5	114.0	1.020	99.12	.37	.51	34.73	do	45–50
474+50	20	34	48	[1] 1.0	101.0	1.011	99.64	.17	.19	33.94	do	45–50
504+28	17	27	42	2.25	100.0	1.010	99.55	.21	.24	35.07	do	5–10
535+75	21	29	40	[1].5	119.5	1.009	99.30	.19	.51	35.15	do	50–55
544+32	20	31	49	2.25	98.3	1.012	99.73	.11	.16	34.11	do	15–20
617+64	22	33	49	2.5	96.0	1.012	99.72	.15	.13	34.86	do	10–15
Section 2V–1:												
140+17	19	29	44	2.25	101.8	1.012	99.31	.20	.49	34.81	do	15–20
166+65	24	35	51	[1].5	109.3	1.013	99.66	.16	.18	32.91	do	45–50
245+25	29	43	62	1.75	101.3	1.009	99.77	.12	.11	31.34	do	40–45
262+70	23	33	47	[1].5	109.0	1.011	99.30	.25	.45	33.28	do	45–50

[1] Test specimen broke at smallest cross section.

TABLE 8.—*Average analysis of asphalt coatings used by the fabricators of the coated pipe*

Fabricator	Sections where used	Penetration, 100 gm. 5 sec. at—			Ductility at 25° C.	Softening point	Specific gravity, 25°/25° C.	Bitumen insoluble in 86° B. naphtha	Xylene equivalent
		15° C.	25° C.	35° C.					
					Cm.	° *C.*		*Percent*	*Percent*
A	2M–1, 2M–2, 2N–1, 2P–2	27	41	63	1.9	98.7	1.000	32.46	[1] 8.3
B	2U–2	23	36	51	1.8	99.0	.988	31.82	0
C	2N–2, 2V–1	21	32	47	1.4	105.0	1.012	34.02	[1] 34.0

[1] Average value of the mean naphtha-xylene equivalent.

TABLE 9.—*Analyses of typical blown asphalts from various sources*

Identification No.	Penetration 100 gm., 5 sec. at—			Ductility 5 cm. per min. 25° C.	Softening point	Specific gravity, 25°/25° C.	Solubility in CS_2			Bitumen insoluble in 86° B. naphtha	Oliensis test	
	15° C.	25° C.	35° C.				Bitumen	Organic insoluble	Inorganic insoluble		Character of spot	Naphtha-xylene equivalent
				Cm.	° C.		*Percent*	*Percent*	*Percent*	*Percent*		*Percent*
1	21	31	45	2.0	110.0	1.011	99.86	0.14	0	37.16	Positive	12–16
2	32	49	72	2.0	107.2	.981	99.64	.20	.16	33.24	do	0– 4
3	29	45	69	2.3	100.0	1.007	99.86	.14	0	32.32	do	24–28
4	21	35	59	4.0	83.3	1.014	99.74	.26	0	32.62	Negative	
5	19	30	46	2.5	95.0	1.003	99.84	.16	0	33.34	do	
6	23	38	62	3.3	90.6	1.013	99.83	.17	0	33.11	Positive	8–12
7	29	42	61	2.0	97.8	.983	99.89	.11	0	30.34	Negative	

this test, a round brass disk one-half square inch in area is embedded in the asphalt coating and pulled off by applying a load at a uniform rate. The ability of the coating to retain its adherence to the pipe is indicated by the amount of coating remaining on the metal after pulling off the disk. This amount of coating retained by the metal is expressed as a percentage of the total area under the disk. For 100-percent retained coating the asphalt breaks in cohesion and denotes good adhesion, zero percent denotes poor adhesion and values between 100 and zero percent coating retained are proportional between good and poor adhesion.

For some time the Public Roads Administration has been studying the adhesion of bituminous coatings to metal and a test similar to that proposed by the American Railway Engineering Association has been used. In this study not only the amount of coating removed by the adhesion disk is noted but also the load required to pull the disk off is determined. It has been found that the adhesion load may be as significant as the amount of coating retained in evaluating the adhesion of coating materials.

The samples of bituminous-coated metal culverts from the Blue Ridge parkway were tested for adhesion of the asphalt coating to the metal. The results are given in table 10, and include adhesion values for the coating on the outside of the pipe, the inside walls above the pavement, and the coating in the invert. With the exception of the asbestos-bonded metal the adhesion values, as measured by the percentage of coating retained, indicate the asphalt to have little adhesion to the metal for all the samples and for the different areas tested on the same sample. In the tests on asbestos-bonded metal, failures occurred in the asbestos sheet and the coating can be considered as having good adhesion. The adhesion values as measured by the load required to pull the disk off indicate the variable adhesiveness of the asphalt coating not only on different pipes but also on different areas of the same pipe.

As indicated by the values of the adhesion load, the adhesion of the asphalts to the metal was, in general, best in the paved inverts and poorest on the outside walls of the pipe. The adhesion load values for the sample from section 2N–1 are nearly the same for the outside, inside, and the invert, but the latter two values were determined on asbestos-bonded metal and should not be compared with those made on plain galvanized metal. The adhesion values for the sample from section 2N–2 show the asphalt coating to have lower adhesion to the reinforcing plate than to either the outside of the pipe or to the inside of the pipe directly under the reinforcement. The adhesion of the coating to the reinforcing plate from section 2N–2 also was very low. These low values may account for the high loss of coating from the inverts in those culverts having the reinforcing plates.

The asphalt coating from the samples was heated and used to coat various kinds of metal plates. Adhesion tests were made on these coated plates to get some indication of the initial adhesion of the various asphalts to the kind of metal on which they were used in the pipe. The results of these tests are given in table 10.

The adhesion load values for the metal plates coated with the reheated asphalts are in most cases higher than the highest value obtained on the corresponding samples taken from the culverts. The amount of coating retained by the metal also is higher on the recoated galvanized and reinforcing plates than on the corresponding pipe samples. Assuming that the adhesion values for the recoated plates are indicative of the adhesion of the original coating to the pipe, there has been a loss of adhesion while the culverts were in service.

There is no correlation between any of the laboratory tests made on the asphalt coatings from the culvert pipes and the adhesion of these coatings. In the study of the adhesion of pipe coating materials now in progress it has been found that certain characteristics of asphalts may be indicative of their adhesion and should be considered in selecting asphalt to insure high initial adhesion and the retention of that adhesion to the metal after exposure in service.

TABLE 10.—*Results of adhesion tests on pipe samples and on laboratory-coated metal plates using asphalts taken from the pipe samples*

Section number	Station number	Coated pipe samples from culverts				Laboratory-coated plates using asphalts from pipe samples		
		Area tested for adhesion	Type of coated metal	Adhesion load	Coating retained	Type of surface coated	Adhesion load	Coating retained
				Pounds	*Percent*		*Pounds*	*Percent*
2M-1	98+84	Outside	Galvanized	11.4	None	Galvanized	31.4	100
		Inside wall	do	13.8	None			
		Paved invert	do	28.9	None			
2M-1	152+23	Outside	Galvanized	14.9	None	Galvanized	39.2	100
		Paved invert	do	39.5	50			
2M-2	526+65	Outside	Galvanized	9.9	None	Galvanized	42.7	100
		Inside wall	Asbestos-bonded	12.8	(1)	Asbestos-bonded	33.8	(1)
		Paved invert	do	24.3	(1)			
2N-1	207+35	Outside	Galvanized	18.0	None	Galvanized	36.1	100
		Inside wall	Asbestos-bonded	18.9	(1)	Asbestos-bonded	39.8	(1)
		Paved invert	do	18.0	(1)			
2N-2	544+32	Outside	Galvanized	26.8	None	Galvanized	51.6	100
		Paved invert [2]	Reinforcing plate	15.8	10	Reinforcing plate	32.0	20
		Paved invert [3]	Galvanized	38.0	35			
2N-2	535+75	Paved invert [2]	Reinforcing plate	12.0	20			

[1] Asbestos sheet was split by test.
[2] Coating on surface of reinforcing plate.
[3] Coating on corrugated metal under reinforcing plate.

SUMMARY

The data obtained in this study seem to warrant the following summary:

1. In general, the most severe deterioration of the asphalt coating and pavement was found at the outlet ends of the pipes.

2. The amount of flow of water did not seem to bear any consistent relation to the degree of deterioration occurring in the coating and pavement.

3. In general, the slope of the pipe had little influence upon the deterioration of the asphalt coating and pavement.

4. The asphalt coating on the outside of the culvert pipe was usually checked and had very little adhesion to the metal.

5. The metal-reinforced asphalt-paved inverts showed the greatest amount of deterioration under service conditions.

6. The culvert pipes in which asbestos-bonded metal was used appeared to have the greatest resistance to deterioration. Even though there was some loss of asphalt coating the asbestos sheet appeared to give added protection to the metal.

7. The better condition of the asphalt coating on section 2M–1 as compared with that on section 2U–2 seems to indicate that the durability of the coating depends to a considerable degree upon the type of asphalt used.

Because of the limited information available on the durability of bituminous-coated sheet metal culvert pipe and because of the short time the culverts inspected had been in service, it seems advisable to continue periodic inspections of these and other installations.

A STUDY OF COARSE-GRADED BASE-COURSE MATERIALS

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by Edward A. Willis, Senior Highway Engineer

THIS REPORT [1] is the seventh in a series describing investigations of granular materials for surface and base-course construction. Previous reports described laboratory and circular track tests on sand-clay and sand-clay-gravel materials, nonplastic granular materials with admixtures of water-retentive chemicals, the chemical treatment of chert-gravels, lignin binder in the treatment of crusher-run materials, and volcanic cinders.

In the report on the tests of sand-clay-gravel materials,[2] it was indicated that the plasticity index of the binder soil is less critical in coarse-graded mixtures having less than 40 percent passing the No. 10 sieve than in similar mixtures containing larger amounts of soil mortar. The purpose of this investigation was to study the effect of variations in plasticity index, ranging from nonplastic to well above the generally accepted A. A. S. H. O. specification limit of 6, on the stability of base-course mixtures having essentially the same grading and all with less than 40 percent passing the No. 10 sieve.

Five mixtures were prepared by combining crushed limestone, sand, silica dust (ground quartz), and clay. The mixtures were designed to have a grading meeting A. A. S. H. O. specification for a type B–1 base-course material. The plasticity index was varied by varying the amount of silica dust and clay in the fraction of the mixture passing the No. 200 sieve. One mixture was nonplastic while the other four mixtures were designed to have plasticity indexes of approximately 5, 10, 15, and 20. The gradings, liquid limits, and plasticity indexes of the five mixtures and the A. A. S. H. O. specification limits for type B–1 base-course materials are shown in table 1.

TABLE 1.—*Grading and physical constants of base-course materials*

Grading	Type B–1 base course, A. A. S. H. O. specification	As constructed: Sec. 1	As constructed: Sec. 2	As constructed: Sec. 3	As constructed: Sec. 4	As constructed: Sec. 5	At end of test: Sec. 1	At end of test: Sec. 2	At end of test: Sec. 3	At end of test: Sec. 4	At end of test: Sec. 5
Passing, sieve:	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1-inch	100	100	100	100	100	100	100	100	100	100	100
¾-inch	70–100	90	88	89	90	93	94	89	91	89	93
⅜-inch	50–80	56	59	60	58	61	63	60	63	60	59
No. 4	35–65	39	42	42	44	43	48	45	46	40	41
No. 10	25–50	32	33	33	38	34	36	34	33	28	30
No. 40	15–30	18	19	18	21	19	19	20	19	16	18
No. 200	5–15	10	10	10	11	10	10	10	10	9	9
Dust ratio [1]	Less than 66⅔	56	53	56	52	53	53	50	53	56	50
Liquid limit	Not over 25	(2)	19	23	29	34	(2)	18	25	31	32
Plasticity index	Not over 6	(2)	4	9	17	21	(2)	3	12	18	18

[1] Dust ratio $=100\times\frac{\text{percentage passing No. 200 sieve}}{\text{percentage passing No. 40 sieve}}$

[2] Nonplastic.

TRAFFIC TESTS MADE ON OUTDOOR CIRCULAR TRACK

The mixtures were subjected to traffic tests on the circular track used in previous studies of water-retentive chemicals as admixtures with nonplastic road-building materials.[3] Wheel loads were applied by 30- by 5-inch high-pressure tires requiring an inflation pressure of 80 pounds per square inch. The load imposed by each wheel was 800 pounds until near the end of the test when it was increased to 1,000 pounds.

Distributed traffic was used for compacting the mixtures and was obtained by gradually shifting the rotating beam longitudinally with respect to its axis of rotation causing the wheels to pursue alternately expanding and contracting spiral courses covering the entire track area.

Concentrated traffic was applied in the actual tests by locking the sliding pivot of the beam in such position that the wheels pursued two concentric circular courses whose center lines were about 2½ inches on either side of the center line of the track.

Since the mixtures used in this investigation contained a large percentage of coarse angular particles, the standard A.A.S.H.O. compaction test could not be used to determine the optimum moisture content for compaction. The materials for each section, after thorough dry mixing, were wetted with just enough water to make them workable and again thoroughly mixed. Representative samples were taken from the mixture as prepared for each section. The moisture contents were determined and vibratory compaction tests were made. Data from these tests as well as on moisture content and density obtained at the end of the traffic tests are shown in table 2.

After the materials for each section had been prepared by wetting and mixing, they were placed in the track in two approximately equal layers to obtain a total base thickness of 6 inches. The bottom lift was hand-tamped in place and further compacted by 400 wheel-trips of distributed traffic at slow speed, obtained by rotating the beam at 9.6 revolutions per minute. The

TABLE 2.—*Moisture-density relations*

Section No.	Vibrator compaction test: Dry density	Vibrator compaction test: Volume aggregate	Vibrator compaction test: Volume voids	Vibrator compaction test: Moisture content if voids are filled	Moisture content when placed	Data obtained at end of test: Dry density	Data obtained at end of test: Moisture content	Volumetric composition: Solids	Volumetric composition: Water	Volumetric composition: Air
	Pounds per cubic foot	*Percent*	*Percent*	*Percent by weight*	*Percent by weight*	*Pounds per cubic foot*	*Percent by weight*	*Percent*	*Percent*	*Percent*
1	146	85.9	14.1	6.0	5.3	148.2	3.3	86.7	7.8	5.5
2	141	82.6	17.4	7.7	5.2	146.3	4.0	85.5	9.4	5.1
3	141	82.5	17.5	7.7	5.0	146.5	4.7	85.6	11.0	3.4
4	136	79.7	20.3	9.3	5.8	144.9	5.1	84.7	11.8	3.5
5	139	81.3	18.7	8.4	4.8	142.6	6.3	83.4	14.4	2.2

[1] This report is based on data collected, under the supervision of the author, by Mr. Walter T. Hamilton, junior highway engineer, prior to his entrance into the armed services.

[2] A Study of Sand-Clay-Gravel Materials for Base-Course Construction, by C. A. Carpenter and E. A. Willis, PUBLIC ROADS, vol. 20, No. 1, March 1939.

[3] Studies of Water-Retentive Chemicals as Admixtures with Nonplastic Road-Building Materials, by E. A. Willis and C. A. Carpenter, PUBLIC ROADS, vol. 20, No. 9, November 1939.

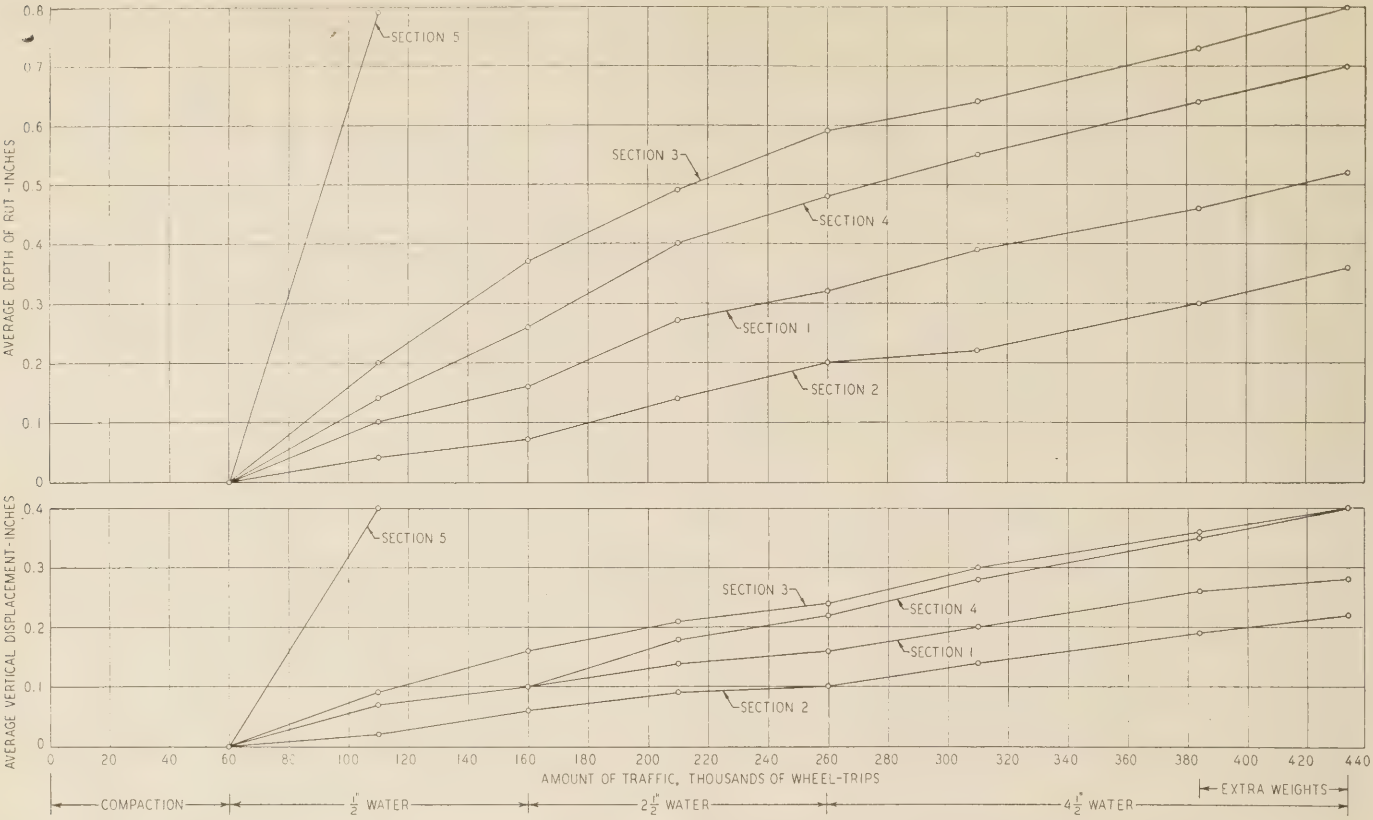

FIGURE 1.—SURFACE DISPLACEMENT OF SECTIONS OF THE TRACK DURING THE COURSE OF THE TEST.

top lift then was placed and tamped, and distributed traffic continued. After 270 wheel-trips, the speed was increased to medium or 14.0 revolutions per minute.

The compaction operations were started during damp weather. Under this condition the track did not become smooth and firm. Two soft spots developed in section 4, which showed pronounced movement under traffic, and operations were suspended temporarily. With the advent of good drying weather traffic was resumed. Compaction proceeded rapidly and the base was soon ready for surface treatment. The total amount of distributed traffic required to compact the base was 40,800 wheel-trips.

After compaction, the base was allowed to dry for several days. Then a prime coat consisting of approximately 0.3 gallon per square yard of light tar was applied and allowed to cure. Finally, a surface treatment consisting of approximately 0.4 gallon of hot-application bituminous material and a cover of 50 pounds per square yard of aggregate of ¾-inch maximum size was applied. Distributed traffic was used to compact the surface treatment. After 400 wheel-trips at slow speed, the speed was increased to medium and continued until the entire surface was smooth, well sealed, and showed no movement.

Compaction of the surface treatment required 19,200 wheel-trips bringing the total distributed traffic to 60,000 wheel-trips.

TESTING WITH CONCENTRATED TRAFFIC

After construction and compaction had been completed, initial profile measurements were made, water

TABLE 3.—*Schedule of operations and behavior of test sections*

Operation	Water level above top of subbase	Traffic	Behavior				
			Sec. 1 ([1])	Sec. 2 (P. I.—4)	Sec. 3 (P. I.—9)	Sec. 4 (P. I.—17)	Sec. 5 (P. I.—21)
	Inches	*Wheel-trips*					
Distributed traffic at slow speed, compacting bottom lift		0–400	Stable	Stable	Stable	Stable	Stable.
Distributed traffic at slow speed, compacting top lift		400–670	do	do	do	Unstable	Do.
Distributed traffic at medium speed, compacting top lift		670–40,800	do	do	do	Stable	Do.
Distributed traffic at slow speed, compacting surface treatment		40,800–41,200	Good	Good	Good	Good	Good.
Distributed traffic at medium speed, compacting surface treatment		41,200–60,000	do	do	do	do	Do.
Testing with concentrated traffic	½	60,000–110,000	do	do	do	Unstable	Failed.
Do	½	110,000–160,000	do	do	do	Good	
Do	2½	160,000–210,000	do	do	Fair	Unstable	
Do	2½	210,000–260,000	do	do	do	Fair	
Do	4½	260,000–310,000	Fair	do	Failed	Failed	
Do	4½	310,000–384,000	Unstable	do			
Testing with concentrated traffic and extra weights	4½	384,000–434,000	Failed	Slightly unstable			

[1] Plasticity index, nonplastic

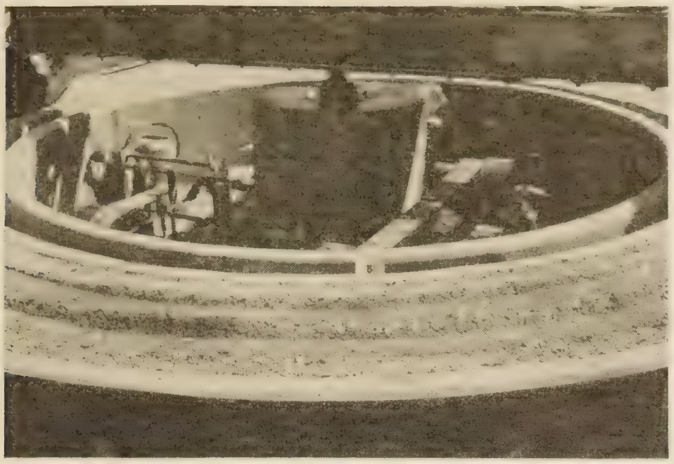

Figure 2.—Appearance of Section 5 After 110,000 Wheel-Trips.

was introduced into the subbase and allowed to rise to a height of one-half inch above the top of the subbase, and testing with concentrated traffic was started. As shown in the schedule of operations, table 3, the water elevation was maintained at one-half inch above the top of the subbase during the first 100,000 wheel-trips of concentrated traffic. The water was then raised to 2½ inches for a similar period and finally was raised to 4½ inches above the top of the subbase. This high water elevation was maintained throughout the remainder of the test period. After 384,000 wheel-trips, the load on each wheel was increased from 800 to 1,000 pounds. Testing was discontinued after a total of 434,000 wheel-trips had been applied.

The behavior of the materials was judged by the appearance of the sections at various stages of the tests supplemented by measurements of vertical displacement of the surface. Previous reports have described the transverse [4] and longitudinal [5] profilometers with which the measurements were made.

Transverse profiles were plotted and the area between the initial and each succeeding profile was measured with a planimeter. The measured area divided by the track width of 18 inches gave the vertical displacement. The average displacement for the two stations on a section gave the average vertical displacement for that section.

The area between the initial and each succeeding longitudinal profile made for each wheel lane was measured for each section. That area divided by the length of the wheel lane gave the depth of rutting, and the average for the two-wheel lanes in a section gave the average depth of rutting for that section.

Surface displacements of the sections at various stages of the test are shown in figure 1. The schedule of traffic applications and changes in water elevations with notations on the behavior of the five test sections are given in table 3.

In reports on previous investigations on the same circular track, it has been stated that an average vertical displacement of 0.25 inch is sufficient to cause marked damage to the bituminous wearing course. It was also found that, with the ground water elevation one-half inch above the top of the subbase, concentrated traffic provided a condition sufficiently severe to enable the identification of the definitely unsatisfactory base-course materials.

Section 5 clearly failed to give satisfactory performance as measured by these standards. This section, which had a plasticity index of 21, began to show movement under traffic soon after water was admitted to the subbase. At 110,000 wheel-trips, the average vertical displacement had reached 0.4 inch and the section was considered as "failed" (see table 3). Figure 2 shows the condition of the section at this time.

It was necessary to repair section 5 before the test on the other sections could be continued. The surface was scarified and crushed limestone and tar added. The section was then hand-tamped until firm. Profilometer measurements on the section were discontinued.

BASES WITH HIGH PLASTICITY INDEXES INFERIOR

The nonplastic base course in section 1 remained in good condition until exceptionally severe conditions were imposed near the end of the test. When the water was raised to 4½ inches above the top of the subbase after 260,000 wheel-trips, this section began to show slight signs of distress although the average vertical displacement (see fig. 1) did not reach 0.25 inch until about 370,000 wheel-trips. Damp spots began to appear on the surface and slight movement under traffic was noted. Increasing the wheel load to 1,000 pounds had no appreciable effect as far as could be observed and the average displacement continued at a uniform rate.

At 434,000 wheel-trips, when traffic testing was discontinued, section 1 was rated as failed primarily because the average vertical displacement was in excess of 0.25 inch. Figure 3 shows the condition of sections 1, 2, and 3 at the end of the test.

Section 2, which had a plasticity index of 4, behaved slightly better than section 1 at all stages of the test. Introduction of water, even at the maximum elevation of 4½ inches above the top of the subbase, had little effect on the stability. The only noticeable change was a gradual consolidation in the wheel tracks. No upward movement of material was observed between the wheel tracks or at the edges of the track. As shown in figure 1, the displacements measured on section 2 were less than those measured on any other section. At the end of the test, the average depth of rut was approximately 0.36 inch and the average vertical displacement was approximately 0.22 inch. After 434,000 wheel-trips, the conclusion of traffic, the section was still in serviceable condition.

Section 3, with a plasticity index of 9, was stable throughout the compaction period (60,000 wheel-trips). During the testing with concentrated traffic with water one-half inch above the top of the subbase, the section remained in good condition although there was a gradual development of ruts as shown by the displacement measurements plotted in figure 1.

After 160,000 wheel-trips, the water level was raised to 2½ inches and concentrated traffic continued. During this phase of the testing, displacement and rutting continued at a uniform rate.

At about 200,000 wheel-trips slight movement of the surface and base could be observed with the passage of each wheel. This movement continued throughout the remainder of the test although it was never of sufficient magnitude to disrupt the surface.

[4] Circular Track Tests on Low-Cost Bituminous Mixtures, by C. A. Carpenter and J. F. Goode, PUBLIC ROADS, vol. 17, No. 4, June 1936.
[5] See footnote 2, page 239.

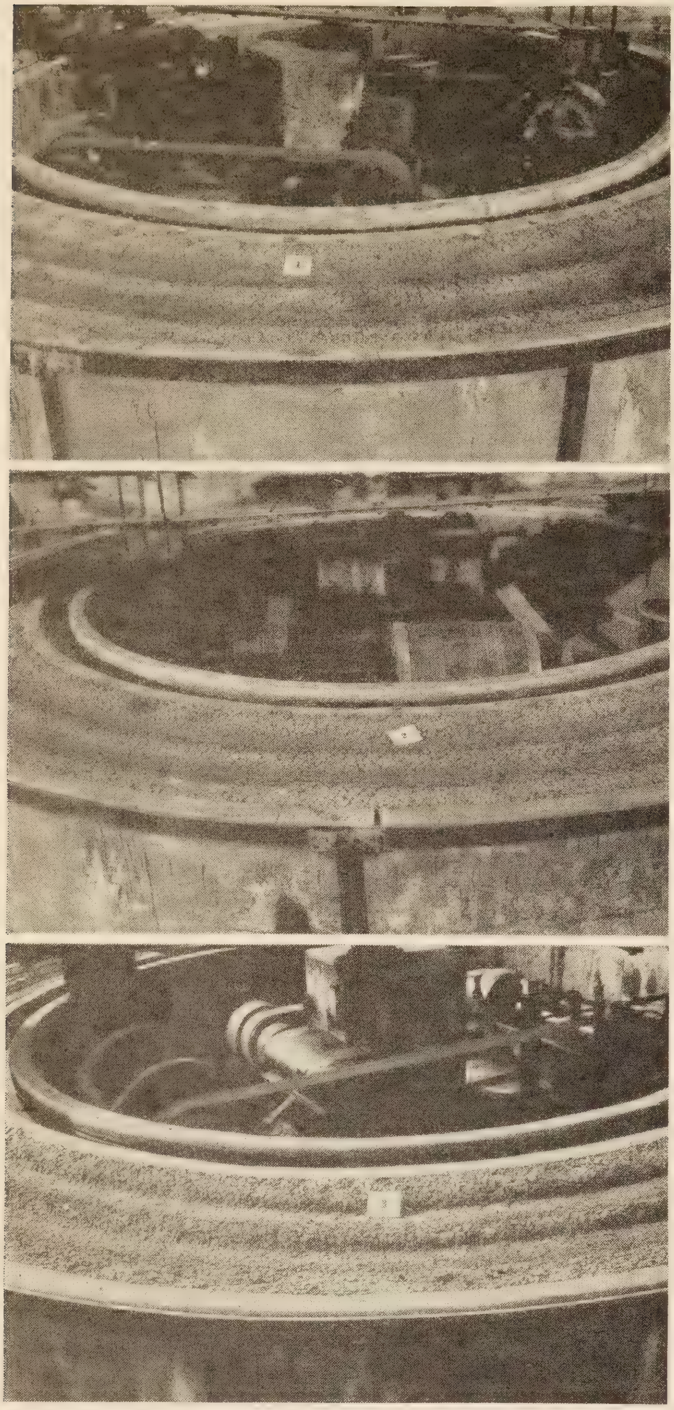

Figure 3.—Condition of Sections 1, 2, and 3 at the End of the Tests.

The average vertical displacement of section 3 was approximately 0.24 inch at 260,000 wheel-trips, the conclusion of testing with water at the 2½-inch elevation. This was more than for any other section except section 5 which had failed at about 100,000 wheel-trips. Displacements continued uniformly under concentrated traffic with the water level raised to 4½ inches above the top of the subbase. By the time traffic had reached 300,000 wheel-trips, the average vertical displacement was well over 0.25 inch, mud and water began appearing through cracks in the surface, and the section was rated failed although traffic was continued over the section to the end of the test without difficulty.

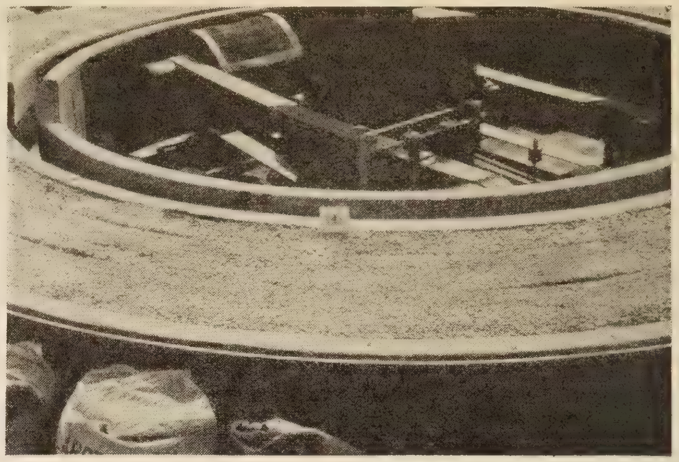

Figure 4.—Section 4 After 110,000 Wheel-Trips. Note Signs of Local Failure While the Section as a Whole Shows Very Little Displacement.

The behavior of section 4, which had a plasticity index of 17, was erratic. As previously noted, two soft spots developed in this section early in the compaction operation. Although these spots became firm before the surface treatment was constructed, they persisted as areas of local weakness throughout the early phases of testing with concentrated traffic. Immediately after the introduction of water into the subbase the passage of the wheels caused these spots to move visibly and to become depressed to such an extent that it appeared the section would fail.

As traffic continued, these areas became more stable. By the time 110,000 wheel-trips had been completed, the movement of the base was barely perceptible. At this time, the surface generally was in good condition with local rutting at the two weak points as shown in figure 4. No further change was observed until the water level was raised to 2½ inches above the top of the subbase at 160,000 wheel-trips.

The average vertical displacement in section 4 at 160,000 wheel-trips was less than that in section 3 at the same time. However, when testing was resumed with the water level raised to 2½ inches, the rate of average vertical displacement in section 4 was greater than that in section 3, and by the end of the test the average vertical displacements of both sections were equal and well in excess of 0.25 inch.

The two weak spots originally observed in section 4 again became unstable under traffic when the water was raised to 2½ inches above the top of the subbase. The movement in these areas gradually decreased. At 200,000 wheel-trips slight movement of the base was observed in the entire section. This general movement throughout section 4 continued until 260,000 wheel-trips, at which point the water level was raised to 4½ inches above the top of the subbase.

Section 4 failed as soon as traffic was resumed after increasing the water elevation. A mud-boil appeared at one of the soft spots at this time and the surface of the entire section moved under traffic. At 310,000 wheel-trips, the average vertical displacement of section 4 was almost 0.3 inch and the entire section was rated failed although traffic was continued to 434,000 wheel-trips in order to evaluate other sections. The appearance of section 4 at the end of traffic testing is shown in figure 5.

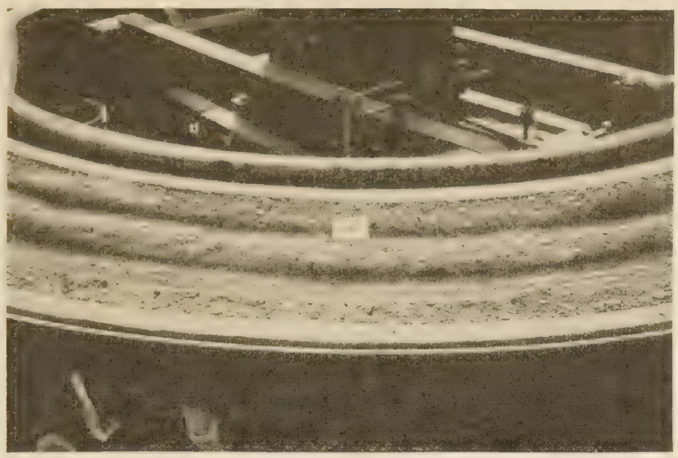

FIGURE 5.—SECTION 4 AT THE END OF THE TEST.

PLASTICITY INDEX IS A GUIDE TO PERFORMANCE

All five of the mixtures tested met the requirements for grading of the A. A. S. H. O. specification for type B–1 base-course material, but, as shown in table 1, only those placed in sections 1 and 2 met the plasticity index requirement.

The criteria used in evaluating these mixtures in the circular track tests were that an average vertical displacement of 0.25 inch was sufficient to cause marked damage to the bituminous wearing course and that concentrated traffic with water one-half inch above the top of the subbase provided a condition sufficiently severe to enable the identification of definitely unsatisfactory base-course materials.

Based on these criteria, the sections can be rated in order of superiority, as follows:

Section 2—Plasticity index 4	Excellent
Section 1—Nonplastic	Satisfactory
Section 3—Plasticity index 9	Border line
Section 4—Plasticity index 17	Unsatisfactory
Section 5—Plasticity index 21	Unsatisfactory

The behavior of section 2 was excellent throughout the tests. After 434,000 wheel-trips, including 174,000 wheel-trips with water 4½ inches above the top of the subbase, the average vertical displacement was still less than 0.25 inch. The dry density at the conclusion of the test (table 2) was 146.3 pounds per cubic foot and the moisture content was 4 percent. Section 1 behaved almost as well as section 2. The nonplastic material compacted under traffic somewhat more than did the mixture with a plasticity index of 4 used in section 2. As a result, the dry density at the end of the test was 148.2 pounds per cubic foot.

It was not until near the end of the test when extremely severe conditions were imposed that the average vertical displacement measured on section 1 exceeded 0.25 inch.

Base-course materials having gradings and plasticity indexes similar to those used in sections 1 and 2 would, therefore, be expected to give excellent service under all moisture conditions with those having low but definite plasticity indexes being slightly superior under the most adverse conditions.

Section 3 (plasticity index 9) was readily compacted and, under concentrated traffic, with water one-half inch above the top of the subbase, the average vertical displacement of this section was less than 0.2 inch. Therefore, the material cannot be rated as definitely unsatisfactory. However, displacement of the surface continued under continued concentrated traffic and exceeded 0.25 inch shortly after 260,000 wheel-trips when the water elevation was raised to 4½ inches above the top of the subbase. Although the final dry density of section 3 was 146.5 pounds per cubic foot, slightly higher than that of section 2, evidence of lateral movement was observed and mud and water were being forced through cracks in the surface near the end of the test. The section was therefore rated as border line. Materials having similar gradings and plasticity indexes would be expected to perform satisfactorily in well-drained areas, but would probably undergo sufficient displacement under adverse moisture conditions to cause failure of the surface.

Section 4 (plasticity index 17) gave trouble during compaction as well as periodically during the testing with concentrated traffic. Although the average vertical displacements measured on this section were less than those measured on section 3, except for the readings at the end of the test, the pronounced tendency of the material to become unstable each time the water elevation was raised is indicative of unsatisfactory performance under any but dry conditions.

Rutting started in section 5 (plasticity index 21) as soon as testing with concentrated traffic commenced with water one-half inch above the top of the subbase and progressed rapidly. The average vertical displacement was 0.4 inch at 110,000 wheel-trips at which time it was necessary to repair the section. Mixtures meeting A. A. S. H. O. grading requirements, but having plasticity indexes as high as this one, are therefore considered definitely unsatisfactory for use as base-course materials.

CONCLUSIONS

Based on the results of tests described in this report, it is concluded that:

1. Mixtures meeting the grading requirements of the A. A. S. H. O. specification for type B–1 base-course materials should also conform to the plasticity index requirement for best results under traffic even though the mixtures contain less than 40 percent of material passing the No. 10 sieve.

2. A material meeting the grading requirements and having a definite and measurable plasticity index within the specification limits is to be preferred to absolutely nonplastic materials having similar gradings and is decidedly superior to those having appreciably higher plasticity indexes.

3. Although the material having a plasticity index of 9 gave fairly satisfactory service except in the latter portion of the tests run with high water elevation, the possibility of the occurrence of conditions approaching complete saturation at some time during the life of a base course makes the present A. A. S. H. O. specification limit of 6 for the plasticity index a desirable if not a vitally necessary requirement.

TESTS OF CONCRETE CONTAINING WAYLITE

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by W. F. Kellermann, Senior Materials Engineer

A VARIETY OF MATERIALS are used as lightweight aggregates for concrete and are sold under different trade names. Probably the best known and most widely used material is known commercially as Haydite, an artificial aggregate made by burning shale or clay. Other lightweight aggregates are made by different processes. One of these is made by processing slag with steam in a centrifuge, the resulting product being known as Waylite.

When the tests reported herein were initiated, Waylite was being marketed both as a fine powder (practically 100 percent passing a No. 200 sieve), and as fine and coarse aggregate. It was decided to study both products, the first by using finely ground Waylite dust as a replacement for a portion of the portland cement in the mix, and the second by comparing some of the properties of concrete made with fine and coarse Waylite aggregate with those of comparable concrete made with natural materials.

Three series of tests were run. Series A was planned to determine the effect of substituting various percentages of Waylite dust for portland cement on the crushing and flexural strengths of concrete. In series B the effect of using Waylite fine and coarse aggregate on the unit weight and on the crushing and flexural strengths of concrete was determined by comparison with results obtained with natural aggregates. Series C was to determine the effect of using Waylite dust to replace portland cement on the resistance of concrete to attack by sulphates.

Natural river sand and crushed limestone were used as aggregates in series A and C and as aggregates for comparison with Waylite aggregate in series B.

All of the strength specimens in series A were cured in moist air and tested at 28 days. In series B, continuously moist-cured specimens were tested at 28, 180, and 360 days and, in addition, one group of specimens was cured the first 180 days in moist air and the remaining 180 days in laboratory air. This made it possible to compare the strengths of specimens cured the entire period in moist air with the strengths of specimens subjected to both moist air and laboratory air curing. The specimens used in series C were cut from beam specimens remaining after tests for flexural strength in series A.

For series A and C, four base mixes (mixes without Waylite dust) were designed with different water-cement ratios and a constant slump (2.5 to 3 inches). Water-cement ratios were selected as follows: 0.70, 0.80, 0.90, and 1.00, by volume. The corresponding cement factors ranged from 6.3 to 4.4 sacks of cement per cubic yard. For each base mix, three additional mixes were made in which Waylite dust was used to replace cement in the following percentages by weight: 25, 32, and 40. The substitution of Waylite dust for cement in the amounts used did not appear to affect the slump or ease of placement of the concrete in any way.

In series B data were obtained on the basis of both a comparable cement content and a comparable water-cement ratio. For the first condition two mixes for

TABLE 1.—*Chemical analysis of cement and Waylite dust*

Cement	Percent	Waylite dust	Percent
Silica	20.25	Silica	37.60
Alumina	6.70	Titanium oxide	.72
Ferric oxide	3.05	Alumina	11.82
Lime	62.70	Ferrous oxide	.32
Magnesia	3.68	Manganous oxide	1.72
Sulphuric anhydride	1.77	Calcium oxide	42.85
Loss on ignition	1.43	Calcium sulphide	3.17
		Magnesia	1.81
Tricalcium silicate	47	Insoluble residue	.38
Tricalcium aluminate	13		

TABLE 2.—*Grading and physical properties of aggregate and Waylite dust*

Sieve analysis	Natural sand	Crushed limestone	Waylite		
			Fine aggregate	Coarse aggregate	Dust
Percentage retained on:					
1½-inch sieve					
¾-inch sieve		60		4	
⅜-inch sieve		90		53	
No. 4 sieve	5	100		89	
No. 8 sieve	25	100		99	
No. 16 sieve	38	100	31	100	
No. 30 sieve	55	100	65	100	
No. 50 sieve	85	100	87	100	
No. 100 sieve	96	100	93	100	
No. 200 sieve					2.9
Fineness modulus	3.04	7.50	2.76	6.45	
Physical properties:					
Bulk specific gravity	2.59	2.75	1.90	1.10	[1] 2.86
Absorption (percent)	.8	.32	8.3	9.8	

[1] Apparent specific gravity.

each aggregate combination were designed, one containing six sacks and the other seven sacks of cement per cubic yard. For the second condition, an additional mix was designed with natural aggregates having the same water-cement ratio as the 6-sack Waylite concrete.

Chemical analyses of the cement and Waylite dust are shown in table 1. Physical tests of the aggregates are given in table 2 while the mix data are shown in tables 3 and 4. The effect on the weight of the concrete of using Waylite fine and coarse aggregate is shown in the last column of table 4.

The concrete was mixed in a laboratory mixer with a capacity of 1¾ cubic feet and of the pan and paddle type, the pan being removable for cleaning. Test specimens were standard 6-inch by 12-inch cylinders and 6- by 6- by 21-inch beams. The absorption of the Waylite aggregate was very high, which is, of course, typical of aggregates of this type. For this reason all aggregates were immersed in water for 48 hours prior to mixing the concrete. However, the Waylite dust used in series A and C was not immersed as it was used as a substitute for cement.

All beam specimens were tested on an 18-inch span by the third point loading method, A. S. T. M. designation C 78.

The specimens used in series C to study the effect of Waylite dust upon the resistance of the concrete to alkali were 6-inch cubes sawed from the 6- by 6-inch beam specimens remaining after the strength tests of series A. Two cuts were made on each half beam so

TABLE 3.—*Mix data and unit weight of fresh concrete for series A and C*

Mix No.	Waylite replacement by weight [1]	Proportions by dry weight: Cement	Proportions by dry weight: Waylite	Aggregate: Fine [2]	Aggregate: Coarse [3]	Cement factor, base mix	Water-cement ratio, base mix	Total water content	W_c [4]	Weight of fresh concrete
	Pct.	*Lbs.*	*Lbs.*	*Lbs.*	*Lbs.*	*Sacks per cu. yd.*		*Gals. per cu. yd.*		*Lbs. per cu. ft.*
1(a)	none	94	none	188	331	6.3	0.70	32.7	0.162	152.8
1(b)	25	70.5	23.5	188	331			32.7	.162	152.8
1(c)	32	63.9	30.1	188	331			32.7	.162	152.8
1(d)	40	56.4	37.6	188	331			32.7	.162	152.8
2(a)	none	94	none	221	389	5.4	.80	32.5	.161	152.4
2(b)	25	70.5	23.5	221	389			32.3	.160	151.2
2(c)	32	63.9	30.1	221	389			32.5	.161	152.0
2(d)	40	56.4	37.6	221	389			32.7	.162	153.2
3(a)	none	94	none	250	442	4.9	.90	32.9	.163	153.2
3(b)	25	70.5	23.5	250	442			32.5	.161	151.6
3(c)	32	63.9	30.1	250	442			32.7	.162	152.0
3(d)	40	56.4	37.6	250	442			32.7	.162	152.0
4(a)	none	94	none	278	490	4.4	1.00	33.1	.164	152.0
4(b)	25	70.5	23.5	278	490			32.9	.163	151.6
4(c)	32	63.9	30.1	278	490			32.9	.163	151.2
4(d)	40	56.4	37.6	278	490			32.9	.163	151.2

[1] Waylite dust used to replace a portion of the cement in the percentages shown.
[2] Natural river sand with a fineness modulus of 3.04.
[3] Crushed limestone, maximum size 1½ inches.
[4] Volume of water per unit volume of concrete.

TABLE 4.—*Mix data and unit weight of fresh concrete for series B*

Mix No.	Proportions by dry weight: Cement	Aggregates: Fine	Aggregates: Coarse	Type of aggregate: Fine	Type of aggregate: Coarse	Actual cement factor	Water-cement ratio	Slump	Weight of fresh concrete
	Lbs.	*Lbs.*	*Lbs.*			*Sacks per cu. yd.*		*Ins.*	*Lbs. per cu. ft.*
1	94	198	349	Natural sand.	Limestone	6.0	0.74	2.1	152
2	94	155	300	do	do	7.0	.67	3.2	154
3 [1]	94	278	490	do	do	4.4	1.00	2.1	152
4	94	122	122	Waylite	Waylite	6.0	1.04	1.6	93
5	94	104	104	do	do	7.1	.85	1.2	97

[1] Mix designed to have approximately the same water-cement ratio as mix No. 4, containing Waylite aggregates.

as to expose aggregate on two of the six surfaces. When 42 days old the cube specimens were completely immersed in a 10-percent alkali solution of 5 percent magnesium sulphate and 5 percent sodium sulphate. During the 14-day interval between 28 and 42 days the specimens were kept continuously wet. Periodic weighings were made after immersion in alkali solution from which the losses in weight were determined.

DISCUSSION OF SERIES A

Results of strength tests of series A are given in table 5 and in figures 1 and 2. The flexural strengths shown in table 5 and plotted in figure 1 show how the percentage of Waylite dust in the mix affected the strength of concretes of varying cement content. Water-cement ratio as used in the discussion of series A refers only to the water-cement ratio of the base mix of each group, mix containing no Waylite dust. For total water content of mixes containing Waylite dust see table 3. It will be observed from figure 1 that in concrete for which the base water-cement ratio was 0.70, mixes No. 1(a) to 1(d), inclusive, the flexural strength

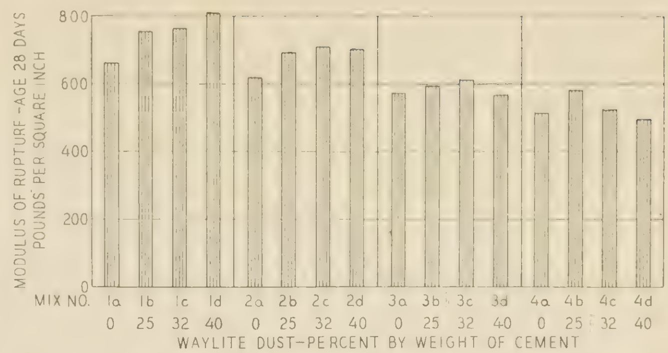

FIGURE 1.—EFFECT OF USING WAYLITE DUST AS A REPLACEMENT FOR PORTLAND CEMENT ON THE FLEXURAL STRENGTH OF CONCRETE, SERIES A.

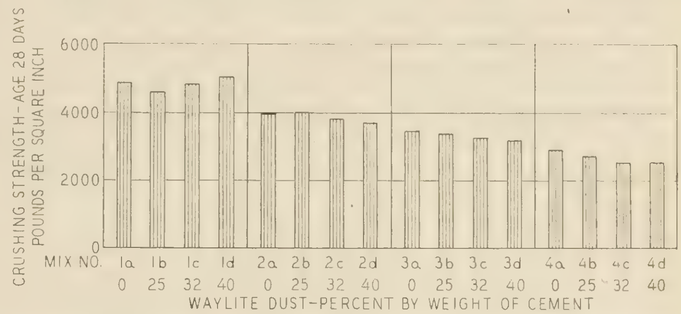

FIGURE 2.—EFFECT OF USING WAYLITE DUST AS A REPLACEMENT FOR PORTLAND CEMENT ON THE CRUSHING STRENGTH OF CONCRETE, SERIES A.

increased as the percentage of Waylite dust increased, the maximum increase being 23 percent with 40-percent replacement. In the case of the group for which the water-cement ratio of the base mix was 0.80, mixes No. 2(a) to 2(d), inclusive, the maximum increase in strength was 15 percent with 32-percent replacement. In the case of the next group, mixes 3(a) to 3(d), inclusive, the maximum increase in strength of 7 percent was also found with the 32-percent replacement, whereas in the case of the last group, mixes 4(a) to 4(d), inclusive, the 25-percent replacement gave the highest strength, a 13-percent increase over the straight portland cement mix. In this instance the 40-percent replacement was slightly lower in strength than the straight portland cement mix, the 32-percent replacement showing about

TABLE 5.—*Results of strength tests of concrete at 28 days for series A* [1]

Mix No.	Waylite dust replacement by weight	Water-cement ratio of base mix	Modulus of rupture	Modulus of rupture ratio [2]	Crushing strength	Crushing strength ratio [2]
	Percent		*Pounds per square inch*	*Percent*	*Pounds per square inch*	*Percent*
1 (a)	None	0.70	661	100	4870	100
1 (b)	25		755	114	4590	94
1 (c)	32		763	115	4830	99
1 (d)	40		810	123	5070	104
2 (a)	None	.80	620	100	3980	100
2 (b)	25		696	112	4020	101
2 (c)	32		715	115	3820	96
2 (d)	40		705	114	3710	93
3 (a)	None	.90	575	100	3470	100
3 (b)	25		596	104	3400	98
3 (c)	32		614	107	3270	94
3 (d)	40		572	99	3200	92
4 (a)	None	1.00	518	100	2910	100
4 (b)	25		585	113	2740	94
4 (c)	32		526	102	2510	86
4 (d)	40		500	97	2530	87

[1] Specimens continuously moist cured until tested. Each value is the average of three tests.
[2] Strength for base mix taken as 100 percent.

TABLE 6.—*Results of strength tests of concrete for series B* [1]

Mix No.	Actual cement factor	Type of aggregate	Modulus of rupture at age indicated in days				Crushing strength at age indicated in days			
			28	180	360	360 [2]	28	180	360	360 [2]
	Sacks per cu. yd.		*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
1	6.0	Natural sand and limestone.	692	766	744	682	4,400	5,140	5,340	5,910
2	7.0	do	688	794	761	736	5,100	5,820	5,640	6,160
3 [3]	4.4	do	560	616	566	618	3,220	3,880	3,440	3,780
4	6.0	Waylite	354	374	347	136	2,140	2,310	2,820	2,390
5	7.1	do	438	432	410	154	2,720	2,840	2,980	2,920

[1] Specimens continuously moist cured until tested unless otherwise noted. Each value is the average of two tests.
[2] Moist curing 180 days followed by 180 days curing in air. Specimens tested dry.
[3] Mix designed to have approximately the same water-cement ratio as mix No. 4, containing Waylite aggregates.

the same strength. It may be stated that the replacement of part of the portland cement by Waylite dust had a beneficial effect upon the flexural strength of the concrete at 28 days, this effect being more marked for concretes with low water-cement ratios (high cement factors).

It will be observed from the data that the amount of Waylite dust giving the maximum beneficial effect varied with the water-cement ratio, the lower the water-cement ratio (richer the mix), the greater the percentage of Waylite dust permissible. The flexural strength of the concrete was not adversely affected by a replacement of cement by Waylite dust up to 32 percent by weight. For concrete in which the base water-cement ratio is 0.90 or less, this percentage may be increased to 40 percent. In studying these results it should be noted that the tests were made at 28 days, no strength data being obtained for other ages.

Crushing strength results for series A are also shown in table 5 and are plotted in figure 2. The use of Waylite dust as a replacement did not have the same beneficial effect upon the crushing strength of the concrete as is shown for flexural strength. For example, for the mixes in which the base water-cement ratio was 0.8 or higher, the strength of the concrete containing Waylite dust was in most cases slightly lower than the corresponding straight portland cement concrete, the percentage of reduction increasing as the amount of Waylite dust used as a replacement was increased. In the case of the group for which the base water-cement ratio was 0.70, the strength of the concrete containing the Waylite dust was approximately the same as that of the straight portland cement concrete.

DISCUSSION OF SERIES B

The results for series B are tabulated in table 6 and are shown graphically in figures 3 and 4. Figure 3 shows the flexural strengths and figure 4 shows crushing strengths. These data show, for both types of concrete (that is, the concrete containing natural sand and crushed limestone and the concrete containing Waylite aggregate), the flexural and crushing strengths after 28, 180, and 360 days' continuous moist curing as well as corresponding strengths after curing 180 days in moist air followed by 180 days in laboratory air.

The flexural strengths of the limestone concrete and the Waylite concrete can be compared by studying the individual panels of figure 3. For example, the second panel gives the flexural strength of the 6-sack limestone concrete, while the fourth panel gives the strength of

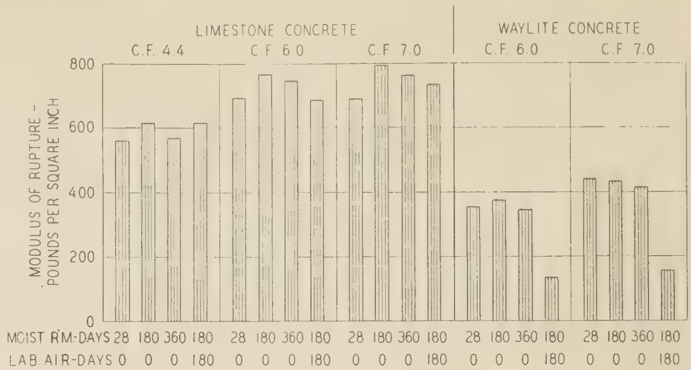

FIGURE 3.—EFFECT OF AGE AT TEST AND METHOD OF CURING ON THE FLEXURAL STRENGTH OF CONCRETE CONTAINING NATURAL AGGREGATES AND WAYLITE, SERIES B.

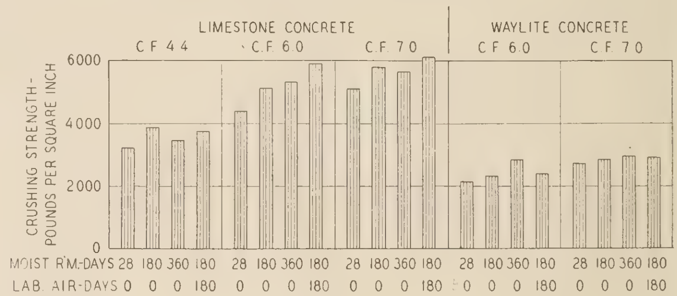

FIGURE 4.—EFFECT OF AGE AT TEST AND METHOD OF CURING ON THE CRUSHING STRENGTH OF CONCRETE CONTAINING NATURAL AGGREGATES AND WAYLITE, SERIES B.

the 6-sack Waylite concrete. The moist-cured concrete containing the Waylite aggregates developed only about one-half the strength of the limestone concrete. The same general trend applies to the 7-sack mixtures.

Comparing the two concretes on an approximately equal water-cement ratio basis (that is, 4.4-sack limestone concrete with 6-sack Waylite concrete), the Waylite concrete developed about 60 percent of the strength of the limestone concrete. These differences are no doubt due to the physical characteristics of the Waylite aggregate. Being of a cellular structure and very light in weight, it does not have the strength of a normal limestone such as was used in these tests as the basis of comparison. The strength of the concrete stored 360 days in moist air was higher than that of comparable concrete stored 180 days in moist air followed by 180 days in laboratory air. For the 6- and 7-sack limestone concrete, this decrease in strength averaged about 6 percent, while for the Waylite concrete the specimens cured in laboratory air for 180 days following 180 days in moist air and tested dry gave only about 40 percent of the strength of those cured wet for 360 days and tested wet.

The marked reduction in flexural strength obtained in these tests as a result of intermittent curing agrees very closely with results previously obtained in a series of tests of Haydite concrete.[1] In these tests Haydite specimens cured 180 days in the moist room followed by 180 days in air gave but 36 percent of the strength of similar specimens cured 360 days continuously wet. Specimens in the same series made of natural aggregates and cured under identical conditions gave 90

[1] The Effect of Curing Conditions on Strength of Concrete Test Specimens Containing Burnt Clay Aggregates, by W. F. Kellermann, PUBLIC ROADS, vol. 18, No. 3, May 1937.

TABLE 7.—*Results of sulphate exposure tests of series C* [1]

Mix No.	Waylite dust replacement by weight	Water-cement ratio, base mix	Weight loss after period of immersion indicated: 26 months	39 months	52 months	98 months
	Percent		*Percent*	*Percent*	*Percent*	*Percent*
1 (a)	None	0.70	6.4	17.2	100.0	100.0
1 (b)	25		5.8	16.6	22.8	35.5
1 (c)	32		4.1	14.8	19.0	[2] 22.8
1 (d)	40		3.9	6.4	12.0	[2] 22.6
2 (a)	None	.80	21.8	100.0	100.0	100.0
2 (b)	25		13.2	25.2	34.8	100.0
2 (c)	32		12.0	17.6	27.4	62.4
2 (d)	40		6.8	17.6	25.1	35.4
3 (a)	None	.90	34.6	100.0	100.0	100.0
3 (b)	25		26.6	75.7	100.0	100.0
3 (c)	32		19.2	35.1	50.8	100.0
3 (d)	40		11.0	21.4	31.6	51.0
4 (a)	None	1.00	63.6	100.0	100.0	100.0
4 (b)	25		46.6	100.0	100.0	100.0
4 (c)	32		35.3	57.7	100.0	100.0
4 (d)	40		31.0	45.6	78.4	100.0

[1] Each value is the average of two tests except as noted.
[2] One test only.

percent of the strength of continuously moist-cured specimens.

In general, the data shown in figure 3 indicate that the strength of the Waylite concrete was about one-half of that obtained with comparable concrete (same cement content) made with natural aggregates when the test specimens were cured continuously moist. For specimens tested at 360 days, which were subjected to 180 days of moist curing followed by 180 days of air curing, the strength of the Waylite concrete was only about one-fifth of the strength of the concrete made with the natural materials. The limestone used in these tests is of a nature that tends to produce high-strength concrete, particularly in flexure.

Crushing strength data are shown in table 6 and plotted in figure 4, the arrangement being the same as in figure 3. For the continuously moist-cured specimens the results agree very closely with those obtained in flexure, the Waylite concrete giving about half the strength of the limestone concrete for equal cement content and about two-thirds for equal water-cement ratio. However, specimens cured moist for 180 days followed by 180 days' curing in air did not show the retrogression in strength that was found with the flexure specimens. This applies both to the Waylite concrete and to the limestone concrete. In fact, the limestone concrete subjected to the air curing gave about 10 percent higher strength than did the specimens cured continuously moist.

SERIES C WITH EXPOSURE TO SULPHATE ACTION

Periodic examination was made of the cube specimens from series A which had been immersed in sulphate solution, and the losses in weight, based on the wet weight of the specimens at the time of immersion, were determined. These losses, computed on a percentage basis, are given in table 7. They are also plotted in figure 5 in a manner to bring out the effect of richness of mix and also the effect of varying the quantity of Waylite dust used as a replacement. The water-cement ratios for the base mixes, mixes containing no Waylite dust, and the percentages of replacements were, of course, the same as in series A (see table 3).

The bar diagrams in figure 5, which extend to 100 percent, are broken for the reason that in no case did complete disintegration occur at exactly 39, 52, or 98 months but at some time between these periods and the time previous weight determinations were made.

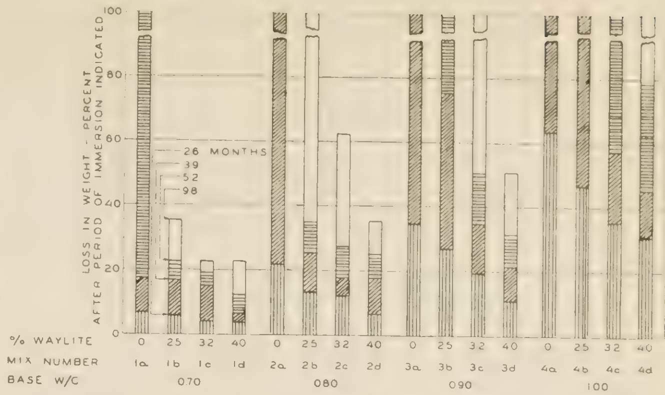

FIGURE 5.—EFFECT OF USING WAYLITE DUST AS A REPLACEMENT FOR PORTLAND CEMENT ON RESISTANCE OF CONCRETE TO ALKALI ACTION, SERIES C.

Considering the resistance of the base concrete as affected by water-cement ratio, it is evident that the durability was markedly decreased by increasing the water-cement ratio. The straight portland cement concrete showed losses after 26 months' immersion varying from 6.4 percent for a water-cement ratio of 0.70 to 63.6 percent for a water-cement ratio of 1.00. After 39 months' immersion, all of the plain concretes with water-cement ratios of 0.80 or higher had completely disintegrated while at 52 months the plain concrete with a water-cement ratio of 0.70 had completely disintegrated.

The use of Waylite dust as a replacement for a portion of the portland cement had a decidedly beneficial effect in these tests. Referring again to table 7 and to figure 5, it will be noted that, for any given base water-cement ratio, improvement in resistance to the action of the sulphate solution resulted from the use of the Waylite dust. For mixes 1 (a) to 1 (d), inclusive, the losses in weight after 26 months' immersion varied from 6.4 percent for the 0-percent replacement to 3.9 percent for the 40-percent replacement. This group had a low water-cement ratio of 0.70. In contrast, for the same immersion period, the group in which the base water-cement ratio was 1.00 showed losses ranging from 63.6 percent for the 0-percent replacement to 31.0 percent for the 40-percent replacement. The same general trends hold for the other immersion periods.

A study of the weight losses after over 8 years' (98 months) immersion furnishes interesting comparisons. In the lowest water-cement ratio group none of the mixes containing Waylite dust showed complete disintegration, the maximum loss in weight being 35.5 percent for the 25-percent replacement. In the next higher water-cement ratio group the mix containing 25 percent Waylite showed complete disintegration while the mix with 40 percent Waylite showed practically the same weight loss as that found with the 25-percent replacement in the group having a base water-cement ratio of 0.70, mix No. 1(b). In the group having 0.90 base water-cement ratio the weight loss of the mix containing the largest amount of Waylite was 51.0 percent with all other mixes showing complete disintegration. In the case of the group in which the base water-cement ratio was 1.00, all mixes showed complete disintegration at 98 months.

(*Continued on p. 250*)

TIRE WEAR AND COSTS

ABSTRACT AND CONCLUSIONS RESULTING FROM STUDIES AT IOWA ENGINEERING EXPERIMENT STATION

VALUABLE DATA ON TIRE PERFORMANCE are presented in bulletin 161, Tire Wear and Cost on Selected Roadway Surfaces, by R. A. Moyer, research professor of civil engineering, and Glen L. Tesdell, research assistant, of the Iowa Engineering Experiment Station at Ames, Iowa. The work reported was done in cooperation with the Public Roads Administration. The following is an abstract of the report and a full presentation of the summary and conclusions.

This study of automobile tires is part of a general investigation of vehicle operating costs, and of the roughness (in terms of riding comfort) and slipperiness of various types of roadway surface. The 10 sections of highway of various types, on which studies were made, varied in length from 20 miles to 320 miles. The sections or routes were located in four States—Iowa, Kansas, Missouri, and Wyoming—and were selected because they were considered as reasonably representative of the types of surface on the main highways of the United States. The scope of the investigation is indicated by the fact that it involved the use of eight automobiles with which detailed observations were made during 450,000 miles of carefully controlled driving.

Tires were the subject of special study because their cost is influenced to a greater extent by the type and condition of the roadway surface than is any other item of vehicle operating cost. The study provided data indicating the effect of each of 15 factors on the rate of tire wear, and of 12 additional factors contributing directly to tire carcass failure. It was determined that tire wear is influenced directly by: Car speed; rate of braking and accelerating; type of roadway surface; tire inflation; wheel alinement and wheel balance; atmospheric temperature; tire switching or "rotation" from one wheel position to another around the car; highway curves and grades; type, grade, and age of the tire; mechanical condition of the car; and the driving habits of the operator. Factors which contributed directly to tire carcass failures were: Neglected cuts, punctures, stone bruises, rim bruises, and fabric breaks; heat, fatigue, overload, and incorrect inflation; tread separation, deterioration of the rubber and fabric from oil and water, and damage from tire chains; and certain types of abnormal wear. The results of the tire wear measurements conducted by the Iowa State Highway Commission to determine the relative resistance to wear of various makes of 100-level and 90-level tires and of synthetic rubber tires supplement the data obtained in this investigation, thus making it possible to present a summary of all of the major factors that contribute to tire wear and tire failures. The term 100-level designates tires of the grade regularly used by car manufacturers as original equipment on new cars; the price of these tires is taken as a base (100 percent =100-level).

The studies indicated that, primarily, speed determined the rate of tire wear and that the 35-mile speed limit, imposed as a war measure in the interest of tire conservation, was based on sound premises. On the basis of the average wear rate observed on concrete pavements and bituminous surfaces in Iowa and Kansas, a life of 56,500 miles for automobile tires may be expected, with due attention to tire maintenance, if the car speed does not exceed 35 miles per hour; whereas at 65 miles per hour the life expectancy would be only about 18,700 miles.

It was found that variations in car speed affected tire wear to a greater extent than any other single factor. Stop-and-go studies, simulating car usage in cities, showed an extremely high rate of wear. It was found that when the wheels of a car are out of alinement the continuous braking effect which is created may cause tires to wear at a rate as high as 10 times the normal rate.

The investigation disclosed that there is a wide variation in tire costs on the various types of road surface, and a sharp rise in the costs as the number of stops per mile is increased. Improvements in tires and in the construction of highways have yielded big dividends to the motorist, but much improvement still is possible as is evident from the fact that the tire costs for travel on gravel surfaces are about double those for travel on concrete pavements.

The United States, in devoting its energies to the vigorous prosecution of World War II, has been confronted with a rubber shortage, and in consequence the supply of tires for civilian use has been sharply curtailed. The findings of this investigation, indicating so clearly the measures necessary for tire conservation, were summarized in nontechnical form in a pamphlet, Tire Wear and Tire Failures on Various Road Surfaces, published by the Public Roads Administration in 1943.

The results of the study show that highway engineers can aid in tire conservation by minimizing, within the limits of safety, the abrasiveness of surface treatments, and by keeping roads and streets in good repair to prevent tire bruises and punctures which may lead to blowouts. Careful attention by the vehicle owner to such measures as repairs of minor tire injuries, correct inflation, and elimination of exposure to oil and excessive sunlight or heat can contribute still more. These measures are of course equally efficacious in peacetime or in wartime. The report provides information that will be useful in highway planning and traffic control, and in the selection of maintenance methods and materials.

SUMMARY AND CONCLUSIONS

The significant data and conclusions of this investigation are as follows:

1. Results of tire wear measurements are reported for more than 2,000,000 miles of tire travel on selected sections of the major types of road surface in Iowa, Kansas, Missouri, and Wyoming.

2. Variations in car speed caused greater differences in the rate of tire wear (loss of tread rubber) than did variations in braking and acceleration, type and condition of road surface, tire inflation pressure, or any of the other factors investigated.

3. The rate of tire wear on concrete pavements at a nominal speed of 65 miles per hour was four times that at a nominal speed of 25 miles per hour.

4. The greatest change in tire wear with change in speed was observed when operating on the most abrasive surfaces, for which wear rates at 65 miles per hour

were in some cases as much as four times those at 35 miles per hour.

5. The least variation of wear with speed was observed when operating on bituminous surfaces sealed with thin coatings of asphalt or covered with moderately soft limestone chips held in place by asphalt. The rate of wear on these surfaces at 65 miles per hour was approximately double that at 35 miles per hour.

6. The higher rates of tire wear observed at the higher speeds were due to the greater amount of bouncing and slipping of the tires, the increased traction required at the rear wheels, and the increase in brake applications and braking time.

7. In stop-and-go tests, simulating city traffic conditions, the rate of wear was seven times that in normal driving on rural pavements at a speed of 25 miles per hour.

8. The average rate of wear of tires used regularly on city streets at the customary speeds was two to three times that of tires used on rural pavements at 45 miles per hour.

9. The rate of tire wear on dry pavements was approximately double that on the same pavements when wet.

10. When wheels are out of alinement, forward motion of the car produces a continuous braking effect which, in extreme cases, may result in tire wear at 10 times the normal rate.

11. Rate of tire wear during the first 500 miles of use was three times the rate after tires had been driven 3,000 to 6,000 miles.

12. Rate of tire wear was affected only slightly by variations in atmospheric temperature. On gravel surfaces the rate of wear was greatest in the summer months, but on dry concrete pavements and bituminous surfaces the rates of wear were greatest during the winter.

13. The rate of wear for the tires on rear wheels was from 130 to 200 percent of the rate on front wheels. As a general average, the tires on the right wheels wore 10 percent faster than those on the left wheels.

14. Switching or "rotating" all tires, including that on the spare wheel, from one wheel position to the next around the car every 2,000 miles is recommended as a means of equalizing tire wear.

15. Tire wear on curves at speeds that caused the tires to "scream" was more than 10 times the wear at speeds that did not result in sidewise skidding.

16. The rates of wear of tires of the 100-level grade made by various manufacturers, and normally supplied on new cars in prewar days, did not vary from the average by more than 10 percent.

17. The rate of wear of two synthetic rubber tires, on one car, exceeded the rate for the two regular 100-level tires by approximately 30 percent.

18. Increase in the rate of wear of tires, because of age, was less than 25 percent for ages up to 3 years. Effect of age on wear was twice as great for rear tires as for front tires.

19. The average life of 10 tires operated on concrete pavements was 36,651 miles. On gravel surfaces the average life for 20 tires was 23,160 miles. All of the test tires were used more miles per year than are those on the average passenger car and, therefore, under more favorable conditions as to age.

20. Records of car owners and of tire manufacturers indicate that in 1940 the average life of tires in the United States was approximately 22,000 miles.

21. The longest life of a 100-level tire in the wear tests made by the Iowa State Highway Commission was 70,000 miles, and the shortest life 20,000 miles. One of the synthetic rubber tires had a life of 36,000 miles.

22. Operation of a vehicle equipped with a set of tires with recapped treads made of 95 percent reclaimed rubber, indicated a life after recapping of 14,000 miles if speed were restricted to 35 miles per hour.

23. A life of 60,000 miles is a reasonable expectancy for 100-level tires with one recapping of reclaimed rubber, and restriction of speed to 35 miles per hour.

24. Many thousands of tire-miles are lost when tires become unserviceable because of avoidable carcass failures long before the treads are worn smooth. Among avoidable damages are abnormal tread wear; carcass weakness resulting from neglected cuts and punctures, stone bruises and fabric breaks, and loosening of inside cords by running on soft or flat tires; overload, incorrect inflation, and rim bruises; and tread separation, deterioration from oil, and long exposure to sunlight or heat.

25. In 132,000 miles of driving on gravel surfaces, there were 98 punctures as compared with a single puncture in the same distance on concrete pavements and one puncture in 72,000 miles on bituminous surfaces.

26. The heat, impact, and fatigue effects of operation on rough, loose, corrugated gravel surfaces resulted in tread cracking and blowouts. Approximately 30 percent of the original (new-to-smooth) tread-groove depth was still remaining when these tires became unserviceable because of carcass failures.

27. Continuous operation at 65 miles per hour on concrete pavements, with no change in atmospheric temperature, caused the pressure in 6.00–16 passenger car tires to increase 4.6 p. s. i. (pounds per square inch) in the rear tires and 3.5 p. s. i. in the front tires. For continuous operation under similar conditions the tire pressure was 1.5 p. s. i. higher at 65 miles per hour than at 35 miles per hour for both the front and rear tires.

28. The tire pressure rise when driving on gravel surfaces was 0.25 p. s. i. lower than that on concrete pavements for corresponding speeds and atmospheric temperatures.

29. An increase of 1.0 p. s. i. in tire pressure was observed for each 20° F. rise in atmospheric temperature.

30. A normal loss in tire pressure of 1 to 2 p. s. i. per week was observed during the winter, and of 2 to 4 p. s. i. during the summer.

31. An inflation pressure of 30 to 32 p. s. i. "cold" is recommended to obtain the greatest service from 6.00-16 tires.

32. The practice of decreasing the inflation pressure by letting air out of tires in hot weather is not recommended, unless pressures are checked and adjusted carefully and frequently.

33. The temperatures in the treads of 6.00–16 tires in continuous operation at 25 miles per hour on dry concrete pavements, averaged 35° F. higher than the atmospheric temperatures in summer and 55° F. higher in winter. When operating under similar conditions at 65 miles per hour the tread temperatures averaged 75° F. higher than the atmospheric temperatures in summer and 95° F. higher in winter.

34. Tread temperatures averaged about 40° F. lower on wet surfaces than on the same surfaces in the dry condition.

35. By making suitable allowance (an increase of 3 to 5 percent) for the changes in tire volume due to changes in tire pressure, the tire pressures for various tire-air temperatures can be computed by applying the laws relating to pressure and temperature of gases.

36. The costs for 6.00–16 tires and tubes operated on concrete pavements in Iowa and Kansas averaged 0.18 cent per car-mile; for bituminous surfaces in Kansas the average cost was 0.19 cent per car-mile; and for gravel surfaces in Iowa it was 0.37 cent per car-mile. The average speeds to which these costs apply were 42½, 43, and 37 miles per hour, respectively.

37. In 1910 the average life of tires was less than 3,000 miles and the average cost of a tire and tube was about $45. Thus, the average cost for tires and tubes in 1910 was about 6 cents per car-mile, or 33 times the average cost of the tires and tubes used on concrete pavements in this study.

38. The tire costs in the stop-and-go driving on concrete pavements averaged 0.61 cent per car-mile, or more than three times the corresponding cost for average cross-country driving.

39. Highway engineers can aid in tire conservation by minimizing the abrasiveness, within the limits of safety, when repairing or resurfacing roadways. Roads and streets should be kept in good repair, free from pieces of metal and glass that can puncture tires. Holes with sharp edges, which cause cuts and bruises that may result in blowouts, should be repaired promptly.

40. Compliance with restrictions of the wartime tire conservation program is necessary to keep civilian motor vehicles in operation until adequate supplies of new tires are available. The major items in the conservation program include limiting speed to 35 miles per hour, recapping of tires at the proper time, eliminating all nonessential travel, and periodic inspecting of tires. Other measures recommended to car owners are: Start and stop slowly, reduce speed on sharp curves and steep hills, check tire inflation each week, and keep loads within the rated capacity of the tires.

(*Continued from p. 247*)

CONCLUDING STATEMENT

These tests emphasize the importance of moisture condition at time of test upon the flexural strength of concrete containing aggregates of the type represented by Waylite. The results indicate that the moisture content of the concrete at the time of test has a far greater effect when aggregates of this type are employed than when normal aggregates are used and corroborate the results of previous investigations.

The use of Waylite as fine and coarse aggregate in concrete resulted in a marked decrease in both crushing and flexural strength as compared to the results obtained with concrete containing the natural aggregates used in these tests. The decrease was somewhat greater when the comparisons were made on an equal cement-factor basis than when based on a comparable water-cement ratio.

The use of Waylite dust as a replacement for a portion of the portland cement resulted in an increase in flexural strength and a slight decrease in crushing strength. Its use also markedly increased resistance to the destructive action of a sulphate solution consisting of 5 percent Na_2SO_4 and 5 percent $MgSO_4$. The tests emphasize the beneficial effect, from the standpoint of sulphate resistance as well as strength, of using a low water-cement ratio.

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