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Transportation Research Record 1300

Contents

Foreword

This Record contains information on the construction and performance of highway and airfield asphalt pavements and highway surface treatments. It also contains information on the construction and performance of railroad trackbeds that incorporate a layer of hot-mix asphalt. The information in this Record should be of interest to state and local materials, construction, and maintenance engineers, as well as contractors and material producers.

Brown and Cross studied premature rutting of asphalt pavements. The primary concern of their investigation was the relationship among the density of the pavement after densification by traffic, the mix-design density, and the density of laboratory-compacted samples during construction. Martinez and Bayomy compared the method for determining maximum theoretical specific gravity used by the Texas State Department of Highways and Public Transportation with the ASTM D-2041 test method. The results revealed that the Texas method developed a reasonable approximation for maximum theoretical specific gravity during the mix-design phase. Sharma and McIntyre studied reflective cracking and tenting in recycled asphaltic overlays in Wisconsin. Gourdon and Gallenne described new measuring techniques that have made it possible to study the effect of a French-built asphalt finisher's controls and settings on its behavior. Rose and Hensley described current construction procedures and long-term performance of railroad trackbeds that incorporate a layer of hot-mix asphalt as either an underlayment or an overlayment. Button et al. conducted field and laboratory investigations of asphalt pavement intersections to determine the primary causes of premature failure and suggested changes in pavement design and construction procedures that can be used to prolong intersection-pavement service life.

Brown et al. described the investigation of asphalt pavement bleeding on an Interstate overlay project in Illinois. Ahlrich described the investigation of an unstable asphalt airfield pavement that exhibited significant deformation and depressions under parked aircraft.

Gandhi et al. studied the polishing characteristics of different aggregate sources in Puerto Rico using a British polishing wheel. They found that carbonate rocks polish more easily than gravels and noncarbonate rocks. After a follow-up study on in-service pavements, they made recommendations with respect to aggregate specifications for adequate skid resistance. Kandhal and Motter developed criteria for accepting precoated aggregates for seal coats and surface treatments. Their research involved evaluating the adhesion of aggregates precoated to varying degrees so that the optimum precoating requirement could be established and developing an end-result type test for accepting precoated aggregates. Roque et al. evaluated the adequacy and recommended improvements in existing seal-coat design procedures, quality control procedures, and seal-coat performance measuring techniques. They also recommended improvements in predicting seal-coat life and the appropriate use of seal coats as a maintenance technique. Davis et al. described a study to develop laboratory test methods for chip seals to determine the performance of different binder-aggregate systems. Roque et al. evaluated the effect that specific construction, traffic, and materials variables have on the performance of bituminous seal coats. Shuler described research to develop systems for applying chip seal coats to high-traffic-volume asphalt pavements. He reported on the successful construction of the first of four full-scale experiments—the methods used for calibration of equipment, traffic control, design and construction processes—and on the several types of chip-seal treatments placed.

Comparison of Laboratory and Field Density of Asphalt Mixtures

Elton R. Brown and Stephen A. Cross

The objective of this paper is to investigate the relationships among the measured density of an asphalt mixture in the mix design, during quality control (QC) of the mixture (laboratory compaction of a field-produced mix), after initial compaction (after construction and before traffic), after densification by traffic (ultimate density), and after recompaction. Of primary concern is the relationship among density after traffic, mix-design density, and density of laboratory-compacted samples during construction. Eighteen different pavements from six states were sampled. Thirteen were rutting prematurely; five were performing satisfactorily. Construction history, including mix-design data, QC or quality assurance data or both, traffic data, and laboratory data of the physical properties of the pavement cores were analyzed from each site. The results show that in-place air-void contents below 3 percent greatly increase the probability of premature rutting and that the in-place unit weights of the pavements after traffic usually exceed the mix-design unit weight, resulting in low air voids and hence premature rutting.

Density, or in-place unit weight, is an important component of a properly designed and constructed asphalt pavement. Selection of the proper compaction level during the mix-design phase is critical for proper pavement performance. The Asphalt Institute (*1*) recommends that the mix-design density closely approach the maximum density of the pavement under traffic. The Marshall mix-design method as originally developed by the U.S. Army Corps of Engineers at the Waterways Experiment Station (*2*) in the late 1940s was based on the evaluation of samples compacted to a relative density that approximated the density developed by a number of repetitions of a selected aircraft. The original method called for compacting samples to 50 blows per side for tire pressures up to 100 psi and 75 blows per side for pressures greater than 100 psi. The Marshall method has been adapted to highway use with 50 blows per side for medium traffic and 75 blows per side for heavy traffic (*1*).

In recent years, studies have shown that typical truck tire pressures are approaching 120 psi (*3*) and that higher truck tire pressures and increased truck traffic have led to an increase in premature rutting (*4*). The problem could very well be that the mix-design density is being exceeded by the in-place density. This excess density in the field results in low in-place air voids. The relationship between low air voids and rutting is well established in the literature (*5–7*).

The objective of this paper is to investigate the relationships among the measured density of the mixture in the mix design, during quality control (QC) of the mixture (laboratory compaction of a field-produced mix), after initial field compaction (after construction and before traffic), after densification by traffic (ultimate density), and after recompaction. Of primary concern is the relationship among density after traffic, mix-design density, and density of laboratory-compacted samples during construction.

Eighteen of the 30 pavements sampled in a National Center for Asphalt Technology (NCAT) rutting study were selected for study. The 18 pavements were those for which traffic data, mix-design data, and QC or quality assurance (QA) data or both were available. Thirteen had rutted prematurely. The ages of these rutted pavements ranged from 1 to 6 years at the time of sampling. Five of the 18 pavements were identified by the various states as performing satisfactorily (Sites 4, 8, 10, 18, and 24). These five pavements ranged in age from 5 to 16 years at the time of sampling.

TEST PLAN

The test plan for the rutting study is shown in Figure 1. A complete listing of the overall test plan can be found elsewhere (*5*). The field testing consisted of obtaining 4- and 6-in.-diameter cores, measuring rut depths, and, in a majority of the rutted pavements, viewing the pavement layers in a trench cut across the traffic lane. In general, 11 to 12 cores were obtained on 1-ft intervals across the traffic lane at each site. The 4-in. cores were saved for further testing. The 6-in. cores were tested, and the relevant results are reported herein.

Rut-depth measurements were obtained using a 12-ft elevated straightedge to establish a horizontal reference line. The distance from the straightedge to the pavement surface was then recorded to the nearest 1/16 in. at 1-ft intervals over the core locations. Rut-depth measurements at each core location and measurements of each core allowed the determination of the relative elevation of each pavement layer. The maximum rut depth at the surface was determined by measuring the vertical distance between a straight line connecting high points on opposite sides of the rut and the low point near the middle of the rut. Rut depths and the traffic information are presented in Table 1.

Laboratory tests were conducted to characterize the material and mixture properties. The 6-in. cores were first measured to determine the layer thickness of each core. Next, the cores were sawed into their respective pavement layers, and the bulk specific gravity was determined (ASTM D2726) for each layer. The bulk specific gravities were evaluated across the pavement lane for each layer to determine the average in-place unit weight and the standard deviation of the

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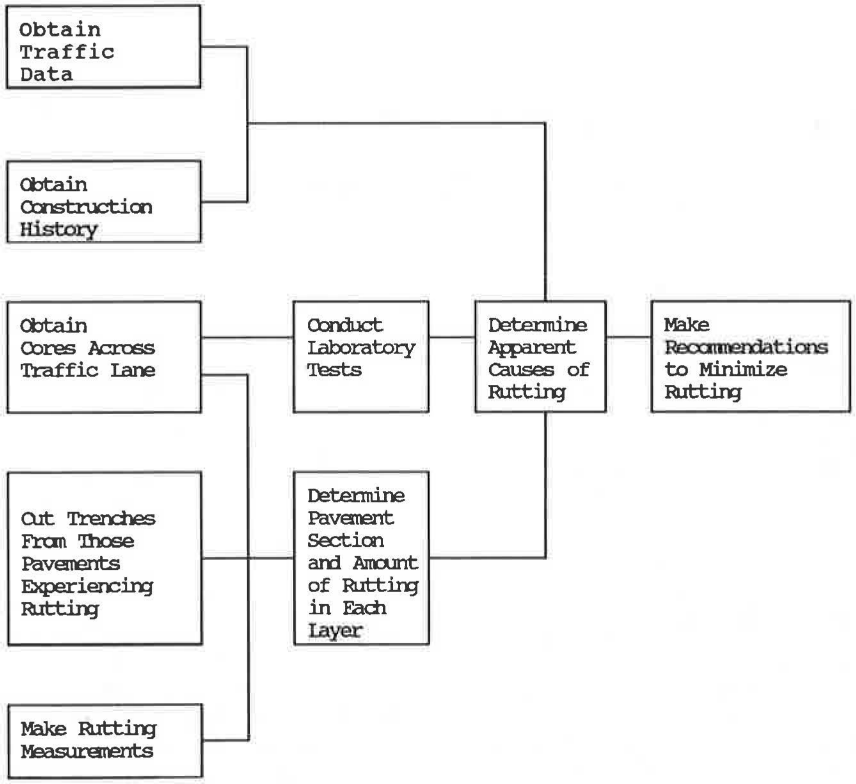

FIGURE 1 Test plan.

measured unit weights. These data were used to determine the 80th-percentile in-place density. Two cores were then selected, and the maximum theoretical specific gravity was determined (ASTM D2041). From the average maximum theoretical specific gravity, the average and 20th-percentile in-place air-void contents were determined. Previous research at NCAT (*5*) showed the 20th and 80th percentiles to be reasonable to use for comparison of in-place air voids and density after traffic with air voids and density of recompacted samples. The two cores were then extracted to determine the asphalt content (ASTM D2172).

The remaining 6-in. cores were reheated, broken up, and recompacted using two compactive efforts: 75 blows per side with the manual Marshall hammer (standard compactive effort) and 300 revolutions on the Gyratory Testing Machine (GTM) at 120 psi and a 1-degree angle. The processes produce samples that have densities approximately equal to the mix after several years of heavy traffic. Hence recompacted densities are approximately equal to mix-design densities if the materials are the same and the proper procedures are used. The recompacted samples were tested for unit weight, air-void content, Marshall stability, and flow. The average results of the tests performed on the 6-in. cores are presented in Table 2; the average results of the recompaction analysis are presented in Table 3.

Construction history and mix-design information for the pavements evaluated were provided by the various states. The data reported are all of the data available to NCAT at the time this report was prepared. The mix-design information relevant to this study is summarized in Table 4; the construction history data are presented in Table 5.

ANALYSIS OF TEST RESULTS

Design of asphalt mixtures by the Marshall method is based on the assumption that the laboratory-compacted test samples will approximate the density of the mixture in service after several years of traffic. If the mix-design density is too low, rutting could develop as a result of low air voids caused by pavement densification under traffic. The purpose of this study is to evaluate the physical characteristics of asphalt pavements, mainly density and air voids, during the various stages of the pavement's life and relate those characteristics to rutting. The stages investigated were mix design, construction, after construction and before traffic, after traffic, and laboratory recompaction.

An attempt was made to relate the rut depth of the pavement to the density and void properties of the mixtures. However, some scatter, which is caused by several factors, exists in the data. Some of the major factors contributing to the scatter include varying amounts of traffic, different aggregate properties of the mixtures, and the temperature of the pavement surface when traffic was first applied. These factors are not addressed in this report. To help alleviate the problem of different traffic loadings on the various layers of pavements, the analysis was performed on mixtures of the same layer in the pavement structure. In addition, only data from original pavement layers or the latest overlays were used in the analysis. This was done to remove the effects of various surface preparation techniques, such as milling before overlaying, on the relationship between rut depth and mixture properties. Open graded friction courses were present on 5 of the 18 pavements selected for analysis (Sites 2, 3, 5, 18, and 20).

TABLE 1 SUMMARY OF RUT-DEPTH CALCULATIONS AND TRAFFIC

SITE	LAYER	MIX TYPE	AVERAGE LAYER THICKNESS (in)	MAXIMUM SURFACE RUT DEPTH (in)	MAXIMUM RUT IN EACH LAYER (in)	TOTAL 18-kip ESAL's (millions)	TRUCK TRAFFIC (%)
1	1	SURFACE	2.4318	1.5000	1.0000	11.80	50
1	2	BINDER	2.0682	1.5000	0.0000	11.80	50
1	3	SAND	7.9659	1.5000	0.5000	11.80	50
2	N/T	OGFC	0.8000	0.8958	0.1667	2.05	20
2	1	SURFACE	1.2750	0.8958	0.4583	2.05	20
2	N/T	OLD PVM'T	14.2750	0.8958	0.2708	2.05	20
3	N/T	OGFC	0.6932	0.3750	0.1250	3.12	22
3	1	SURFACE	1.5682	0.3750	0.2500	3.12	22
3	2	BINDER	2.4306	0.3750	0.0000	3.12	22
4	1	SURFACE	1.1818	0.2500	0.0000	2.74	12
4	2	BINDER	2.2045	0.2500	0.1000	2.74	12
4	3	BINDER	2.5000	0.2500	0.0000	2.74	12
4	4	BASE	2.2614	0.2500	0.0250	2.74	12
4	5	BASE	4.8068	0.2500	0.1250	2.74	12
5	N/T	OGFC	0.7676	0.6250	0.1250	5.25	41
5	1	SURFACE	1.2045	0.6250	0.3125	5.25	41
5	2	BINDER	1.5511	0.6250	0.1250	5.25	41
5	N/T	OLD PVM'T	9.0057	0.6250	0.0000	5.25	41
6	1	SURFACE	1.4271	0.5750	0.2000	2.00	23
6	2	BINDER	1.9115	0.5750	0.3750	2.00	23
7	1	SURFACE	1.5710	0.3439	0.1563	4.81	34
7	2	BINDER	1.6080	0.3439	0.1563	4.81	34
7	3	LEVEL	1.1534	0.3439	0.0313	4.81	34
8	1	SURFACE	1.2500	0.4000	0.2000	13.34	34
8	2	BINDER	1.8177	0.4000	0.2000	13.34	34
10	1	SURFACE	0.7955	0.1250	0.0125	2.72	21
10	2	BINDER	1.7216	0.1250	0.0000	2.72	21
10	3	BASE	2.4773	0.1250	0.0000	2.72	21
10	4	BASE	1.2102	0.1250	0.1125	2.72	21
11	1	SURFACE	1.0966	0.5500	0.2500	0.68	16
11	2	BINDER	1.3523	0.5500	0.1250	0.68	16
11	3	BINDER	2.5739	0.5500	0.1125	0.68	16
11	4	BASE	3.3693	0.5500	0.0250	0.68	16
11	5	BASE	3.2443	0.5500	0.0375	0.68	16
12	1	SURFACE	1.7212	1.4500	0.5000	0.31	5
12	2	BINDER	3.0192	1.4500	0.9500	0.31	5
12	3	LEVEL	1.0139	1.4500	0.0000	0.31	5
13	1	SURFACE	1.5962	1.6563	0.8125	0.30	12
13	2	BINDER	2.4896	1.6563	0.8438	0.30	12
18	N/T	OGFC	0.8580	0.2000	0.0000	1.55	21
18	1	SURFACE	1.7898	0.2000	0.1500	1.55	21
18	2	SURFACE	2.1136	0.2000	0.0500	1.55	21
18	N/T	OLD SUR	1.8281	0.2000	0.0000	1.55	21
19	1	SURFACE	1.5280	0.3900	0.2250	0.26	3
19	2	SURFACE	1.7216	0.3900	0.0125	0.26	3
19	3	BINDER	2.7500	0.3900	0.1525	0.26	3
20	N/T	OGFC	0.8409	0.3167	0.0417	0.38	19
20	1	SURFACE	1.4091	0.3167	0.0000	0.38	19
20	2	SURFACE	2.1932	0.3167	0.2750	0.38	19
20	N/T	OLD SUR	5.7273	0.3167	0.0000	0.38	19
22	1	SURFACE	2.0375	0.5000	0.3250	4.40	50
22	2	BINDER	2.7938	0.5000	0.1750	4.40	50
23	1	SURFACE	1.4205	0.5858	0.3024	3.30	40
23	2	BINDER	1.4432	0.5858	0.1667	3.30	40
23	N/T	OLD SUR	2.0208	0.5858	0.1167	3.30	40
24	1	SURFACE	1.2750	0.3150	0.0712	5.30	9
24	2	BINDER	2.6438	0.3150	0.2437	5.30	9

N/T = Not Tested

TABLE 2 IN-PLACE DATA (NCAT CORES)

SITE	LAYER	ASPHALT CONTENT (%)	VTM AVG (%)	VTM 20th PCT'L (%)	UNIT WEIGHT AVG (pcf)	UNIT WEIGHT 80th PCT'L (%)
1	1	7.8	1.5	0.7	150.1	151.2
1	2	4.0	2.3	0.9	151.1	153.2
2	1	6.0	4.0	3.6	144.9	145.6
3	1	5.2	6.3	5.7	142.9	143.9
3	2	4.8	3.9	3.3	147.0	148.0
4	1	5.6	4.3	3.1	145.3	147.1
4	2	4.3	3.6	3.2	148.1	148.8
5	1	6.8	3.8	3.1	146.6	147.7
5	2	6.5	3.6	2.8	147.6	148.8
6	1	4.8	5.4	4.6	144.7	146.0
6	2	5.4	4.0	3.4	146.4	147.3
7	1	5.3	3.2	2.2	147.3	148.9
7	2	4.7	3.8	3.1	148.1	149.1
8	1	4.5	3.2	2.1	149.7	151.4
8	2	4.2	4.0	3.0	151.4	153.1
10	1	6.8	6.1	5.1	139.5	141.0
10	2	4.3	11.6	10.9	137.2	138.2
10	3	4.5	13.0	12.5	134.7	135.5
11	1	6.3	4.1	2.7	145.6	147.7
11	2	5.2	4.1	2.4	147.0	149.6
11	3	4.4	10.0	8.5	139.2	141.4
12	1	6.5	1.9	1.3	145.3	146.2
12	2	5.0	4.7	3.6	147.4	149.0
13	1	6.2	4.9	3.5	146.6	148.7
13	2	4.1	8.3	6.4	148.7	151.9
18	1	4.3	6.9	5.2	142.8	145.5
18	2	4.7	5.2	4.0	144.2	146.0
19	1	5.7	1.4	0.9	151.1	151.9
19	2	5.3	3.7	4.2	146.8	147.6
19	3	5.1	6.0	6.9	142.7	144.0
20	1	5.6	2.1	1.8	149.3	149.7
20	2	5.2	3.6	2.5	148.1	149.7
22	1	5.2	2.0	1.5	155.4	156.2
22	2	5.9	2.2	1.9	151.8	152.3
23	1	5.0	2.7	1.8	151.6	153.0
23	2	5.0	4.3	3.7	149.5	150.4
24	1	6.3	2.8	1.4	158.8	161.1
24	2	4.5	2.0	1.5	156.7	157.5

The friction courses were not evaluated because of their porous nature and their small effect on rutting.

Air Voids and Rutting

Figure 2 shows the relationship between air voids recompacted to 75 blows per side with the manual Marshall hammer and the total rut depth at the surface expressed as rut depth per the square root of traffic in million equivalent 18-kip single axle loads (ESALs) for the mixtures in Layer 1. An analysis of rutted pavements has shown that rut depth divided by the square root of million ESALs is a good way to quantify rate of rutting. A rate of rutting of less than 2×10^{-4} in. per square root of total ESALs has been shown as a good separation between good and poor performing pavements (*8*). There is enough scatter in the data to make the correlations poor (R-square = 0.12). However, the correlation does show a trend of lower recompacted air voids associated with higher rut depths and higher traffic. The same plot is shown in Figure 3 for the Layer 1 mixtures recompacted on the GTM. The R-square value is nearly identical to the 75-blow samples, and

TABLE 3 RECOMPACTION DATA

		GTM 300			75 BLOWS	
SITE	LAYER	VTM (%)	UNIT WEIGHT (pcf)	GSI	VTM (%)	UNIT WEIGHT (pcf)
1	1	0.6	151.1	1.37	1.8	149.3
1	2	3.2	149.7	1.01	5.6	146.0
2	1	2.4	147.4	1.29	3.1	146.3
3	1	6.1	143.3	1.00	6.1	143.2
3	2	2.1	149.5	1.07	3.1	148.0
4	1	2.9	147.3	1.04	3.7	146.1
4	2	2.2	150.3	1.13	2.8	149.3
5	1	2.3	148.9	1.27	1.7	149.8
5	2	1.1	151.4	1.37	1.2	151.3
6	1	2.9	148.4	1.08	2.8	148.7
6	2	1.8	149.8	1.43	1.6	150.0
7	1	2.1	149.0	1.04	2.2	148.9
7	2	1.3	151.9	1.39	1.8	151.1
8	1	2.3	151.1	1.07	2.7	150.5
8	2	3.2	152.7	1.12	3.7	152.0
10	1	5.7	140.1	1.00	6.0	139.7
10	2	8.9	141.3	1.00	9.1	141.0
10	3	9.3	140.5	1.00	9.6	140.1
11	1	2.4	148.1	1.37	2.6	147.9
11	2	2.2	149.9	1.28	1.9	150.4
11	3	4.1	148.3	1.03	3.6	149.1
12	1	1.0	146.7	1.63	1.1	146.5
12	2	1.6	152.1	1.53	2.2	151.2
13	1	1.9	151.2	1.43	1.8	151.2
13	2	2.8	157.6	1.33	2.8	157.6
18	1	4.3	146.7	1.02	4.1	147.1
18	2	1.7	149.7	1.50	1.6	149.7
19	1	1.2	151.5	1.36	0.7	152.3
19	2	2.5	149.5	1.50	2.4	149.6
19	3	2.8	149.0	1.25	2.5	149.4
20	1	0.8	151.4	1.53	1.5	150.3
20	2	1.4	151.4	1.44	1.9	150.6
22	1	1.3	157.2	1.47	1.3	156.6
22	2	0.8	153.9	1.72	0.6	154.3
23	1	1.7	153.3	1.41	1.8	153.0
23	2	2.4	152.5	1.32	2.5	152.4
24	1	1.8	160.4	1.53	1.3	161.2
24	2	1.2	157.9	1.67	0.8	158.6

the same trend is evident. It can be seen that the rut depth generally increases with a decrease in recompacted air-void content. The relationship between air voids and rutting is well documented in the literature (*5–7*).

Figures 2 and 3 show an important relationship between recompacted air-void content and the probability of rutting. For the Layer 1 mixtures shown in Figure 2, the chance of having a rate of rutting greater than 0.20 in. per square root of million ESALs is 69 percent (9 of 13) if the 75-blow recompacted air-void content is 3 percent. Only 1 of the 5 sites with air voids greater than 3 percent had a rate of rutting significantly above 0.2; its recompacted air-void content was only slightly greater than 3 percent. The Layer 1 mixtures recompacted in the GTM show similar results: 67 percent (10 of 15) of the sites had rut depths greater than 0.20 in. per square root of million ESALs with less than 3 percent air voids, and none of the sites had a rutting rate significantly greater than 0.2 with recompacted air voids greater than 3 percent. From these data it can be seen that mixtures should be designed to have air-void contents greater than 3 percent, preferably around 4 percent.

Pavement Densification and Traffic

Figures 4 and 5 show the relationship between traffic, expressed as total equivalent 18-kip single axle loads in millions,

TABLE 4 MIX-DESIGN DATA

SITE	LAYER	ASPHALT CONTENT (%)	VTM (%)	UNIT WEIGHT (pcf)	BLOWS PER SIDE	SIEVE SIZE (Percent Passing) 3/4	1/2	3/8	#4	#8	#16	#30	#50	#100	#200
1	1	6.3	6.0	141.1	50	100	97	90	67	56	--	35	--	14	6.0
2	1	5.8	5.4	143.7	50	100	98	93	68	57	--	34	--	10	5.0
3	1	5.4	3.9	145.5	50	100	98	93	70	52	--	35	--	7	4.0
3	2	4.2	--	--	50	96	78	--	41	32	--	--	--	--	--
4	1	6.0	5.4	144.4	50	100	98	90	68	56	--	29	--	11	6.0
4	2	4.8	3.2	150.5	50	85	68	--	43	35	--	--	--	--	--
5	1	6.2	3.8	145.8	50	100	98	94	68	54	--	33	--	13	6.0
5	2	5.2	3.6	149.8	50	95	70	--	40	32	--	--	--	--	--
6	1	4.8	4.2	146.3	50	100	99	90	60	44	34	25	10	7	5.6
6	2	5.3	3.2	147.3	50	100	99	84	57	41	32	24	9	6	5.0
7	1	5.0	4.1	147.2	50	100	99	89	70	58	48	36	18	8	6.0
7	2	4.8	3.5	148.1	50	100	95	90	67	49	37	28	13	6	4.7
8	1	4.8	5.4	147.4	50	100	99	88	60	45	36	23	13	8	5.8
8	2	4.3	6.8	148.3	50	100	98	90	78	61	51	40	26	14	8.6
10	1	7.0	7.3	135.4	50	100	100	99	73	63	49	33	19	10	2.0
10	2	6.5	5.9	139.9	50	100	90	84	69	52	33	17	8	5	3.0
11	1	6.5	4.1	142.7	50	100	100	96	71	53	40	24	16	11	8.0
11	2	4.4	7.4	142.9	50	100	96	87	67	52	34	24	16	10	6.0
11	3	3.5	5.1	142.9	50	98	90	84	71	58	41	27	18	12	8.0
12	1	6.5	3.0	144.3	75	100	100	92	62	42	31	25	17	9	5.0
12	2	4.8	3.2	150.4	75	77	64	53	37	25	20	18	12	6	4.0
13	1	6.4	3.1	148.9	75	100	100	92	65	45	26	16	10	9	5.0
13	2	4.3	3.8	153.2	75	73	63	54	44	30	17	10	7	5	4.0
18	1	5.8	3.5	147.7	50	100	93	79	53	39	28	22	14	9	6.1
18	2	5.8	3.5	147.7	50	100	93	79	53	39	28	22	14	9	6.1
19	1	6.4	3.5	146.3	50	100	88	78	55	38	27	21	14	10	6.2
19	2	6.0	4.8	145.0	50	100	84	72	54	44	35	28	19	10	5.2
20	1	5.8	3.6	146.4	50	100	89	75	53	38	28	21	14	10	5.9
20	2	5.8	3.6	146.4	50	100	89	75	53	38	28	21	14	10	5.9
22	1	5.8	2.6	152.6	50	100	97	84	55	42	--	--	19	--	7.6
22	2	6.7	2.7	151.1	50	100	98	88	61	46	--	--	15	--	6.9
23	1	5.3	3.5	150.8	50	100	97	81	51	37	--	--	18	--	7.9
23	2	6.0	2.6	150.7	50	99	--	68	--	47	--	--	16	--	6.9
24	1	6.7	2.1	159.9	50	100	100	100	99	80	54	36	26	20	15.2
24	2	4.5	2.4	156.4	50	--	84	--	53	40	--	--	16	--	9.8

"--" = Data Not Available

and pavement densification, expressed in air voids. The air-void content is the 20th-percentile in-place air-void content of the pavement at the time of sampling. The traffic is the total estimated equivalent 18-kip wheel loads applied to the original pavement or the last overlay for overlaid pavements. The figures show a reduction in air voids, or pavement densification, with an increase in traffic.

A straight-line regression analysis was used to develop the correlations between densification and traffic. The relationship is poor, with an R-square of 0.08 for Layer 1 and 0.11 for Layer 2 (Figures 4 and 5). A good correlation, however, would mean that traffic alone and not mix properties controlled rutting.

A somewhat more useful methodology for investigating the relationship between traffic and pavement densification is to use both the in-place data and the recompaction data. By dividing the in-place unit weight by the recompacted unit weight, an idea of the relative amount of densification obtained for a particular mixture can be established. By plotting this value against the traffic, an estimate of the amount of traffic necessary to reach the recompacted density can be made. The pavement layers were recompacted using both the GTM and the manual Marshall hammer with 75 blows per side. The data show that 75-blow compaction (for recompacted samples) produces a density equal to that expected after 5.4 million ESALs for the top layer and equal to that

TABLE 5 QC DATA

SITE	LAYER	ASPHALT CONTENT AVG (%)	INITIAL IN-PLACE VTM AVG (%)	INITIAL IN-PLACE AVG UNIT WEIGHT (pcf)	LAB COMPACTED VTM AVG (%)	LAB COMPACTED AVG UNIT WEIGHT (pcf)
1	1	6.1	---	---	---	---
1	2	---	---	---	---	---
2	1	5.8	---	---	3.6	---
3	1	5.4	---	---	4.5	144.9
3	2	---	---	---	---	---
4	1	6.0	---	---	---	---
4	2	4.8	---	---	---	---
5	1	6.2	---	---	3.8	145.1
5	2	5.2	---	---	---	---
6	1	5.2	8.0	141.6	---	---
6	2	5.3	5.8	144.1	---	---
7	1	5.2	6.9	142.2	---	---
7	2	4.7	6.4	142.7	---	---
8	1	4.8	---	---	---	---
8	2	---	---	---	---	---
10	1	---	---	---	---	---
10	2	---	---	---	---	---
10	3	---	---	---	---	---
11	1	5.5	---	---	1.3	149.1
11	2	5.5	---	---	3.6	147.7
11	3	---	---	---	---	---
12	1	6.4	4.7	141.4	---	---
12	2	4.7	4.7	149.0	---	---
13	1	6.1	6.1	143.8	---	---
13	2	4.3	4.5	152.1	---	---
18	1	5.8	5.6	144.3	3.53	147.7
18	2	6.1	5.6	144.3	3.48	147.70
19	1	---	6.7	143.0	---	---
19	2	---	7.4	142.3	---	---
19	3	---	7.4	142.3	---	---
20	1	5.7	3.7	146.7	2.56	148.88
20	2	5.8	3.7	146.7	2.48	148.69
22	1	---	---	---	---	---
22	2	---	---	---	---	---
23	1	---	---	---	---	---
23	2	---	---	---	---	---
24	1	---	---	---	---	---
24	2	---	---	---	---	---

"---" = Data not Available

expected after 6.3 million ESALs for the second layer. The data also show that the GTM compaction produces a density equal to that expected after 9.1 million ESALs for the top layer and after 8.63 million ESALs for the second layer. The data for both the GTM and 75-blow recompacted samples are presented in Table 3, and the results of the plots are shown in Figures 6–9.

Mix Design, In-Place, and QC/QA Mix Properties

The relevant mix-design information for the pavements evaluated in this study are presented in Table 4. Of the 18 sites investigated in this study, 16 were designed using a 50-blow Marshall mix design. A 75-blow Marshall mix design was used in two sites. Most of the pavements investigated were high-volume roads, for which a 75-blow Marshall mix design should be used. The use of 50-blow mix designs on the majority of these pavements could be a major cause of the rutting that has been observed. The 2 sites in this study for which the 75-blow mixes were used rutted severely (approximately 1.5 in.). The poor performance of these 75-blow mixes could be related to the high Gyratory Shear Index (greater than 1.3), low recompacted air voids (1.0 to 2.8 percent), and low mix-design

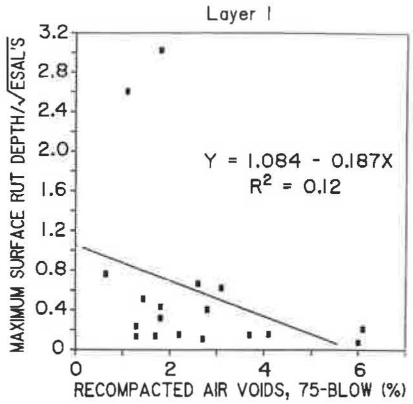

FIGURE 2 75-blow recompacted air-void content versus maximum surface rut depth divided by the square root of traffic in million ESALs.

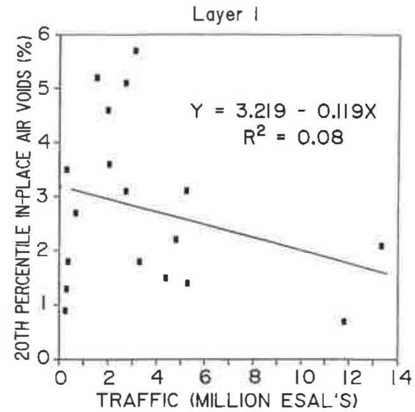

FIGURE 4 Traffic versus 20th percentile in-place air-void content for Layer 1 mixtures.

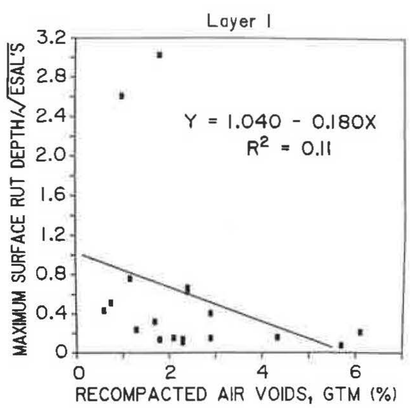

FIGURE 3 GTM recompacted air-void content versus maximum surface rut depth divided by the square root of traffic in million ESALs.

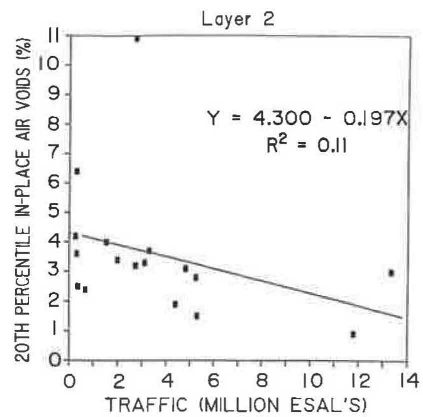

FIGURE 5 Traffic versus 20th percentile in-place air-void content for Layer 2 mixtures.

air-void contents (3.0 to 3.8 percent). The mix-design air-void contents for the mixes in Layer 1 ranged from a high of 7.3 percent to a low of 2.1 percent (Table 4). Two of the Layer 1 mixes were designed with an air-void content of less than 3 percent with 50-blow compaction, and two were designed with an air-void content of 3.0 and 3.1 percent with 75-blow compaction. The design air-void contents for the Layer 2 mixes ranged from 7.4 to 2.4 percent. Three of the Layer 2 mixes were designed with air-void contents of less than 3 percent with 50-blow compaction.

The QC data supplied by the various states are summarized in Table 5. QC data were available for 15 of the sites. At the time of preparation of this report, the mix-design information from Sites 22–24 was available; however, the construction data were not available. The laboratory-compacted data represent testing performed on samples of the mix that were obtained from either the plant or the roadway, returned to the laboratory, and compacted to duplicate the mix design.

One of the most important observations that can be made with regard to construction testing is the lack of data. Construction history data from asphalt cores were available from 14 of 15 sites, but these data are incomplete for many of the sites and pavement layers. The data from the asphalt cores represented extractions for asphalt content, gradation analysis, and unit weights to check initial compaction. Of the 14 sites with in-place density data, 6 contained extraction and gradation analysis data only, 6 contained both extraction and gradation analysis data and unit weight and air-void data, and 2 contained only unit weight and air-void data. Laboratory-compacted samples as a part of QC/QA procedures were used for only 5 of the 15 sites; this represented only 8 of 32 mixtures evaluated. Additional information was either not obtained or not available in the project files. Probably the most important test that can be conducted during QC/QA is to compact plant-mixed material in the laboratory and determine and evaluate the air voids of the laboratory-compacted mixture.

The results of the testing performed on the 6-in. cores from each project are summarized in Tables 2 and 3. The results of the testing performed on the pavement cores after traffic loadings, referred to as in-place data, are presented in Table 2. The results of the recompaction analysis performed on the cores from each site are presented in Table 3. The in-place cores (Table 2) show 16 of the 38 mixtures with in-place air voids below 3 percent with 10 of the low air-void contents occurring in Layer 1. This indicates that the in-place density was higher than the mix-design density or that something in

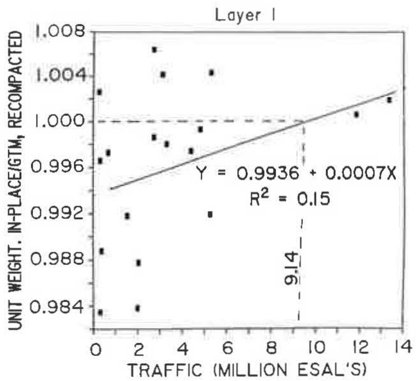

FIGURE 6 Traffic versus ratio of in-place to GTM-recompacted unit weight for Layer 1 mixtures.

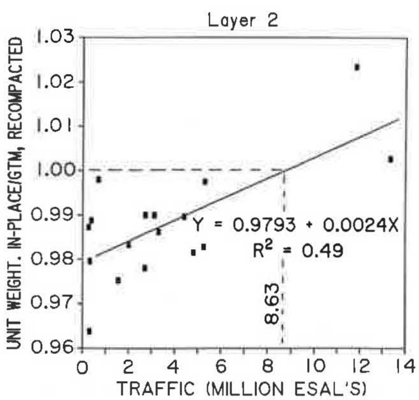

FIGURE 7 Traffic versus ratio of in-place to GTM-recompacted unit weight for Layer 2 mixtures.

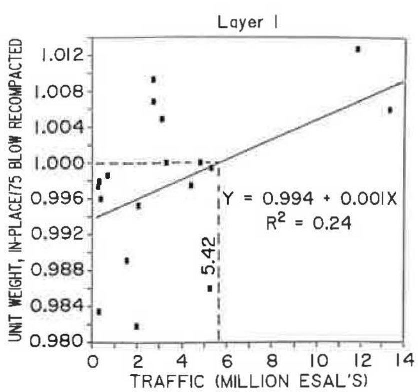

FIGURE 8 Traffic versus ratio of in-place to 75-blow-recompacted unit weight for Layer 1 mixtures.

the mix had changed. Thus the mix-design compactive effort was likely too low, probably because of 50-blow compaction or other causes, or because something in the mix, such as the amount of fines, had changed after its design.

The recompaction data include data from both the 75-blow Marshall hammer and the GTM. For the GTM, 30 of 38 mixtures and for the Marshall hammer, 27 of 38 mixtures had air-void contents below 3 percent. These low voids are typical for rutted pavements. The above data show that mixtures exceed the mix-design densities after traffic and that these high densities and low air voids lead to premature rutting.

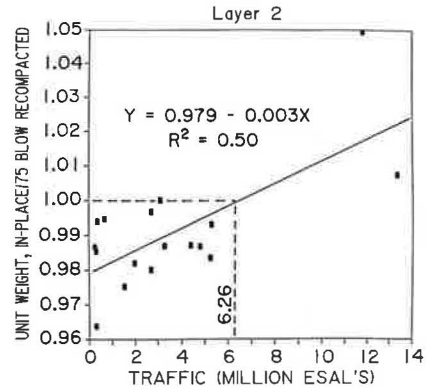

FIGURE 9 Traffic versus ratio of in-place to 75-blow-recompacted unit weight for Layer 2 mixtures.

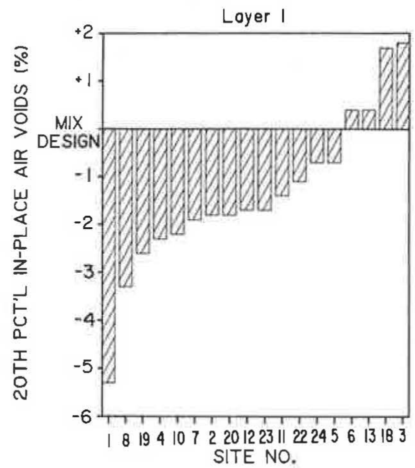

FIGURE 10 Comparison of 20th-percentile in-place air-void contents with their mix-design air-void contents for Layer 1 mixture.

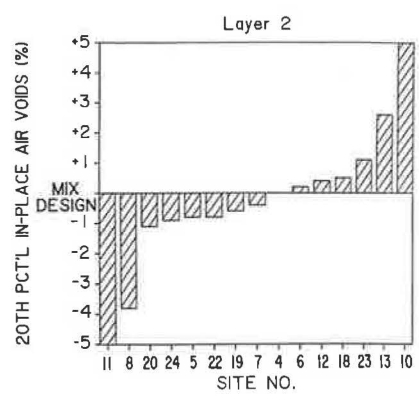

FIGURE 11 Comparison of 20th-percentile in-place air-void contents with their mix-design air-void contents for Layer 2 mixtures.

The differences between the in-place air-void contents at the 20th percentile and the mix-design air-void content are shown in Figures 10 and 11 for Layers 1 and 2, respectively. Fourteen of 18 mixtures (78 percent) for Layer 1 and 8 of 15 (53 percent) for Layer 2 showed the in-place air-void content to be lower than the mix-design air-void content. The same was true for the unit weight at the 80th percentile (Figures 12 and 13), with 78 percent of the mixtures from Layer 1 and 53 percent of the mixtures from Layer 2 exceeding the mix-design unit weight. This indicates that the mix-design compactive effort, especially for the near surface mixtures, is too low for the current level of traffic. In many cases the in-place air voids are 1 to 3 percent lower than the mix-design air voids. Because mixes are typically designed to have 4 percent air voids and rutting is expected to be a problem at 3 percent air voids, these lower in-place air voids are a major problem.

The data show that the in-place unit weight exceeds the mix-design unit weight and that the in-place void content is below the mix-design content. In an attempt to verify that the mix-design compactive effort is indeed low and that the voids are not being overfilled by adding asphalt cement to facilitate compaction, an analysis of variance (ANOVA) was performed on the asphalt contents reported in the mix design, in the QC data, and from extractions performed on the 6-in. cores (in-place) from each site. The results of the ANOVA are presented in Table 6. The analysis showed no significant difference among the means of the asphalt contents of the mix design, QC, and in-place values with a confidence interval of 75 percent.

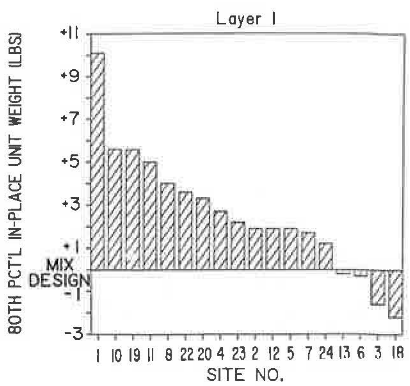

FIGURE 12 Comparison of 80th-percentile unit weights with their mix-design unit weights for Layer 1 mixtures.

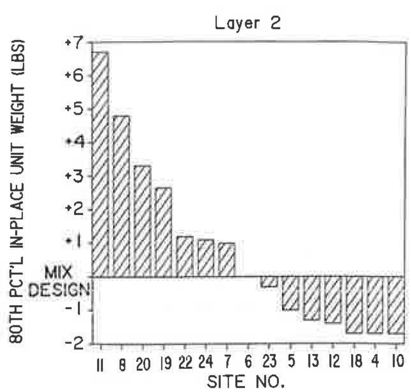

FIGURE 13 Comparison of 80th percentile in-place unit weights with their mix-design unit weights for Layer 2 mixtures.

TABLE 6 RESULTS OF ONE-WAY ANOVA ON ASPHALT CONTENT

SOURCE	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARES	F-VALUE
Total	93	13.0462		
Model	2	0.5790	0.2895	2.11
Error	91	12.4672	0.1370	

To statistically show that the mix-design density is exceeded by the in-place density, an ANOVA was performed on the unit weights obtained from the mix design, the in-place data, and the recompaction (GTM and 75-blow) data. The results are presented in Table 7. The ANOVA showed a significant difference at the 95-percent confidence level in the means of the unit weights from the above data sets. Duncan's multiple range test was performed with alpha = 0.05 to determine the rank and significant differences between the means; the results are presented in Table 8. Duncan's test ranked the means from highest to lowest as GTM, 75-blow, in-place after traffic, and mix design with a significant difference between each group of means except GTM and 75-blow recompacted.

The difference between the initial in-place air-void content and the mix-design air-void contents for Layers 1 and 2 are shown in Figures 14 and 15. All of the initial in-place air-void contents were above the mix-design void content as they should be. However, data from Site 20 indicates the initial in-place density to be very close to the mix-design density, which results in low in-place air voids after traffic. Figure 10 shows that for Site 20 the in-place air-void content after traffic is

TABLE 7 RESULTS OF ONE-WAY ANOVA ON UNIT WEIGHT

SOURCE	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARES	F-VALUE
Total	145	476.9889		
Model	3	213.3884	71.1295	38.32
Error	142	263.6005	1.8563	

TABLE 8 RESULTS OF DUNCAN'S MULTIPLE RANGE TEST

Model	Duncan's* Grouping	Mean	Number Observations
GTM	A	1.186	37
	A		
75 Blow	A	0.881	37
In-Place	B	-0.241	37
Mix Design	C	-1.931	35

* Means with the same letter are not significantly different.

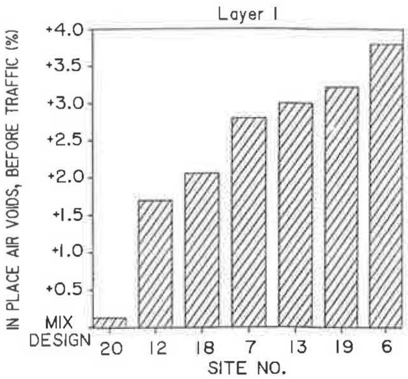

FIGURE 14 Comparison of QC air-void contents before traffic with their mix-design air-void contents for Layer 1 mixtures.

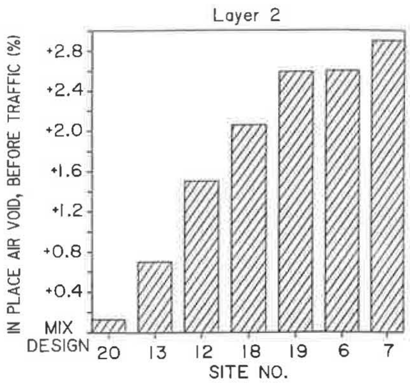

FIGURE 15 Comparison of QC air-void contents before traffic with their mix-design air-void contents for Layer 2 mixtures.

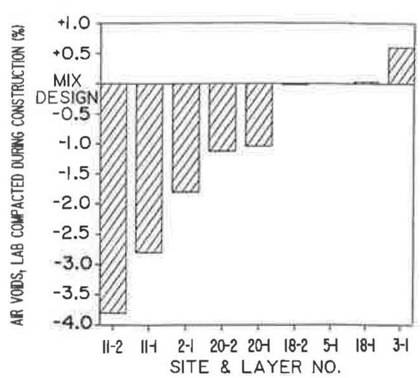

FIGURE 16 Comparison of laboratory-compacted air-void contents with their mix-design air-void contents.

indeed approximately 2 percent lower than the mix-design value. The rut depth at Site 20 was only 5/16 in., due in part to the low level of traffic (0.38 million ESALs). The results of the limited data available on the QC laboratory-compacted samples are shown in Figure 16. Five of the 9 mixtures had air-void contents significantly below the mix-design value, and 4 were within 0.5 percent of the mix-design value. From this information it can be seen that five of the mixtures should have been modified to raise the air-void content in order to minimize rutting.

CONCLUSIONS

On the basis of the data obtained in this study the following conclusions are warranted.

1. In-place air-void contents greater than 3 percent are necessary to decrease the probability of premature rutting throughout the life of the pavement. The voids in laboratory-compacted samples are used to estimate the ultimate void content of the mixture.
2. An in-place air-void content of less than 3 percent greatly increases the probability of premature rutting.
3. Compaction using 300 revolutions of the GTM set at 120 psi and a 1-degree angle gives sufficient design density and void content for up to 9 million ESALs.
4. Compaction using 75 blows per side with the manual Marshall hammer gives sufficient design density and void content for up to 6 million ESALs.
5. Construction QC documentation is not adequate on many paving projects. Samples of asphalt mixtures from the mixing plant should be compacted in the laboratory during construction to verify that the air voids are within an acceptable range. If they are not within an acceptable range, adjustments to the mix should be made.
6. A 50-blow Marshall mix design was used for most of the pavements evaluated in this study. A 75-blow Marshall mix design should be used for mixtures to be exposed to high traffic volumes to ensure adequate voids throughout the life of the pavement.
7. The in-place unit weight of the pavement after traffic usually exceeded the mix-design unit weight, resulting in low air voids and hence premature rutting.

RECOMMENDATIONS

On the basis of the data obtained from this study the following recommendations are made.

1. Samples of the field-produced mixture should be compacted using the specified mix-design compactive effort to ensure that the mix has acceptable air voids and other properties. If there is a significant difference between the field-produced samples and the mix design, modifications to the field-produced mix must be made.
2. Efforts must be made to ensure that the mix design produces a density approximately equal to the in-place density after several years of traffic. The results of this study show that this is not the case. For heavy-duty pavements with significant truck traffic, such as most Interstate highways, it is recommended that either a 75-blow Marshall mix design or the GTM be used. For the Marshall mix design, compaction should be performed with either the manual Marshall hammer or another hammer calibrated to give the same density as the manual hammer.

3. Pavements should be designed to ensure 4 percent air voids in-place after several years of traffic to help prevent premature rutting. Mixes with design air voids much less than 4 percent are likely to rut.

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Selection of Maximum Theoretical Specific Gravity for Asphalt Mixture Design

D. FRED MARTINEZ AND FOUAD M. BAYOMY

The volume of air has been cited in technical literature as one of the most important performance indicators of asphalt paving mixtures. Air-void content is not measured directly; instead it is calculated mathematically by comparing the bulk specific gravity of a mixture with the maximum theoretical specific gravity of the mixture at the same asphalt content. The test method developed by Rice (ASTM D-2041) is the most accepted means of determining maximum theoretical specific gravity. Nevertheless, agencies have used and continue to use other methods and calculations to determine this value. The purpose of this study was to compare mixture properties determined by the Rice method with those determined by the method established by the Texas State Department of Highways and Public Transportation (SDHPT). The SDHPT procedure is often referred to as the Martin method. Data from 42 mix designs were analyzed. The analysis determined that on average, use of the Rice method to determine maximum theoretical specific gravity resulted in asphalt content about 0.35 percent higher than did use of the Martin method. The difference in selected asphalt content derived from the two methods increased as aggregate absorption increased. The difference between the two methods proved insignificant for aggregate with low absorption capacities. The results revealed that the Martin method developed a reasonable approximation for maximum theoretical specific gravity during the mix-design phase.

Performance characteristics of asphalt mixtures have been related to various asphalt mixture properties (*1–4*). Some studies indicate that the most important performance predictor is the volume of air. The effect of air voids on the mixture properties is shown in a conceptual plot in Figure 1 (*4*). Such relationships substantiate most of the experience of asphalt technologists in the last 50 years and illustrate the importance of achieving an optimum level of air voids within the range of 3 to 8 percent. Apparently, determination of air-void content is an extremely important issue. Attention is most often focused on the air-void content of trial mixtures during the mix-design phase and during plant quality control. Air voids are considered again after field compaction and after a certain amount of traffic.

According to the Asphalt Institute mix design manual (*5*), air void content is calculated by

$$Pa = 100\,(Gmm - Gmb)/Gmm$$

where

Pa = air-void content (percent of total volume),
Gmm = maximum theoretical specific gravity of the mixture, and
Gmb = bulk specific gravity of the mixture.

In this formula, *Gmm* is intended to be measured experimentally by means of the Rice method (ASTM D-2041). Determination of *Gmm* and air-void content in this manner is the most accepted procedure. Nevertheless, there are numerous variations of determining *Gmm,* and each has unique implications, especially during the mix-design phase.

The Rice procedure and the procedure used by the Texas State Department of Highways and Public Transportation (SDHPT) and other agencies in Texas were used to examine the effect of *Gmm* on asphalt content and other mixture properties. As recently as 1988, more than 23 million tons of hot-mix asphalt were placed in Texas, ostensibly by the current SDHPT procedure (*6*).

BACKGROUND OF TEXAS PROCEDURE

The procedure currently used by agencies in Texas is a variation of the method developed in the 1950s by Rogers Martin, a former employee of SDHPT (*7*). The Martin procedure, applied during mix design, improves the determination of *Gmm* by molding specimens at higher-than-normal asphalt contents to simulate a mixture without voids. At the asphalt content at which the mixture is considered saturated (i.e., zero voids), the effective specific gravity of the combined aggregates in the molded specimen is determined by the following formula:

$$Gse = (100 - Pb)/[(100/Gmb) - (Pb/Gb)]$$

where

Gse = effective specific gravity of combined aggregates,
Pb = percent asphalt by weight of mix,
Gb = specific gravity of the asphalt binder, and
Gmb = bulk specific gravity of the mixture assumed to represent zero air voids.

After determination of *Gse* in this manner, *Gmm* is determined at other asphalt contents in the mix design by

$$Gmm = 100/[(Ps/Gse) + (Pb/Gb)]$$

where *Ps* is the percent aggregate by weight of mixture and *Gb* is the asphalt specific gravity.

With the exception of the saturation approach for determining *Gmm,* the density/voids analyses of trial mixtures of

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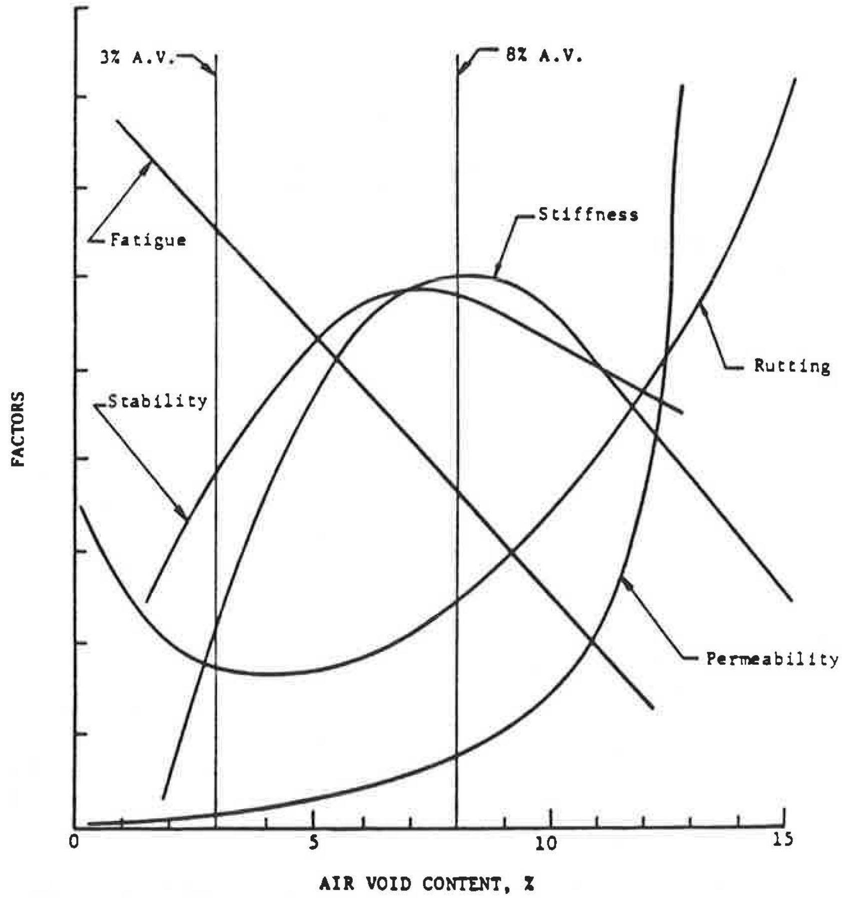

FIGURE 1 Relationships between mixture properties and air-void content (*4*).

the Martin procedure are exactly the same as those outlined in the Asphalt Institute design manual (*5*).

Although density/voids analyses do not take into account the method of compaction, it should be noted that the Texas SDHPT uses a gyratory shear compactor to mold mix-design specimens. The standardized process (*8–10*) requires all trial mix specimens to be compacted so that the specimens achieve a constant resisting pressure of 150 psi.

Another interesting aspect of the Martin approach is the manner by which it accounts for absorption of asphalt by aggregates. In order to best simulate project conditions, loose trial mix specimens are oven-cured to facilitate absorption. The standard Martin procedure specifies that loose samples be placed in an oven at 250°F for 2 hr.

During the early 1980s, Texas SDHPT personnel and various other asphalt technologists in Texas began experimental use of the Rice procedure (ASTM D-2041). Such use was initiated on the basis of in-place density specifications being promoted in Texas and elsewhere. In this application, *Gmm* determined by the Rice procedure was used as a reference to determine the air-void content of hot-mix asphalt after field compaction. Because of this practice, it seemed logical that the air-void content determined during mix design should use the same reference (*Gmm*) as determined by the Rice procedure. Because previous mix-design experience in Texas was within the framework of the Martin procedure to determine *Gmm*, three key questions needed to be answered:

1. How would the use of the Rice method affect density/voids analyses?
2. Would the use of the Rice method result in fundamental changes in asphalt content selection?
3. How does aggregate absorption capacity affect questions 1 and 2?

LABORATORY STUDY

In order to resolve these questions, asphalt contents selected from density/voids analyses were compared using the Rice and Martin methods. A typical design plot illustrating the approach is shown in Figure 2. The two air-void curves shown in Figure 2 were determined on the basis of *Gmm* determined from the Rice and Martin procedures.

A data base containing 42 mix designs was evaluated. The mixes were placed on a wide variety of facilities in southeast Texas ranging from low-volume city and county streets to interstate highways. They were all dense-graded surface courses with approximately 100 percent passing the 1/2-in. sieve.

Mix designs were performed for the various aggregate blends and types. The aggregate blends consisted of various combinations of sandstone or limestone, limestone screenings,

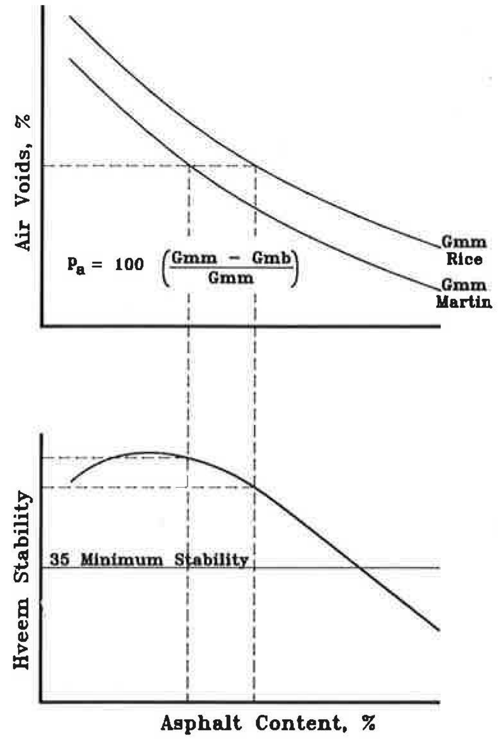

FIGURE 2 Selection of optimum AC content.

and siliceous sand. Normally, the inclusion of sandstone implied that the mixture was intended to be nonpolishing. Trial blends were computed using various percentages of the individual aggregates to satisfy agency gradation requirements. An example of these computations is presented in Table 1.

Aggregate gradations for the 42 designs were similar. The principal difference between the aggregates was their absorption properties. Therefore, mixtures were grouped into nine categories on the basis of their water and asphalt absorption capacity. Table 2 presents the mix groups and their coding. The mix code is designated by *Mwa*, where *w* refers to the water absorption level and *a* to the asphalt absorption level.

The Martin method was used with one exception. During preparation of trial mixtures, an additional sample was prepared at each asphalt content for determination of *Gmm* using the Rice method. The overall procedure is shown in Figure 3. The procedure was used to determine optimum binder content for the 42 mix designs. Optimum binder content was selected at the project-specified air-void content, normally 3 to 5 percent.

MIX DESIGN RESULTS

Effect of Absorption

Aggregates and their combinations used in this study exhibited a general relationship between water and asphalt ab-

TABLE 1 PROPERTIES OF AGGREGATE BLENDS AND JOB MIX FORMULA

	Aggr. 1			*Aggr. 2*			*Aggr. 3*			*Aggr. 4*			*Job Mix*	*SDHPT*
Sieve Size	% Ret.	Sp. Gr.	(%) Abs	% Ret.	Sp. Gr.	(%) Abs	% Ret.	Sp. Gr.	(%) Abs	% Ret.	Sp. Gr.	(%) Abs	Formula (% Ret.)	Item 340 Type "D"
1 1/2 – 1"	-	-	-	-	-	-	-	-	-	-	-	-	-	-
1 – 1/2	-	-	-	-	-	-	-	-	-	-	-	-	-	-
1/2 – 3/8	24.2	2.486	2.7	0.2	-	-	-	-	-	-	-	-	6.5	0 – 15
3/8 – No.4	59.8	2.452	3.3	52.3	2.590	1.3	0.6	-	-	0.3	-	-	35.6	21 – 53
No.4 – No.10	12.4	2.440	3.4	42.8	2.575	1.7	14.9	2.538	2.6	1.5	-	-	21.7	11 – 32
Total No.10	96.4	-	-	95.3	-	-	15.5	2.495*	3.5*	1.8	2.600*	0.9*	63.8	54 – 74
No.10 – 40	0.8	-	-	2.8	-	-	45.0	2.792**	-	20.4	2.789**	-	12.3	6 – 32
No.40 – 80	0.3	-	-	0.2	-	-	12.2	-	-	64.1	-	-	15.5	4 – 27
No.80 – 200	0.7	-	-	0.2	-	-	8.7	-	-	12.2	-	-	4.2	3 – 27
Pass No.200	1.8	-	-	1.5	-	-	18.6	-	-	1.5	-	-	4.2	1 – 8
TOTAL	100	-	-	100.0			100.0			100.0			100.0	

Notes :

Aggr.1 – Sandstone
Aggr.2 – Type D-F Limestone
Aggr.3 – Limestone Screening
Aggr.4 – Field Sand

Job Mix Formula:

27% Aggr.1
37% Aggr.2
15% Aggr.3
21% Aggr.4

TOTAL 100

* Results are for fractions passing #10 and retained on #80 sieve
** Results are for fractions passing #80 sieve

TABLE 2 DESIGNATION OF MIX GROUPS

Aggregate Asphalt Absorption Level (%)	Aggregate Water Absorption Level (%)		
	Low (l) < 2.0 %	Medium (m) 2.0 - 2.5 %	High (h) > 2.5 %
Low(l) < 1.0 %	Mll	Mml	Mhl
Medium (m) 1.0 - 1.5 %	Mlm	Mmm	Mhm
High (h) > 1.5 %	Mlh	Mmh	Mhh

sorption. A low level of water absorption tended to correlate with low asphalt absorption. Conversely, aggregates with high water absorption exhibited high asphalt absorption.

Analysis of absorption characteristics of all mixtures resulted in the distribution of mixes within various groups (Table 3). The frequency of occurrence of each group is shown in Figure 4.

This grouping indicates that mix group *Mmm* (medium water and medium asphalt absorption) was the most likely to occur. As expected, no mix fell within the group *Mhl*, and only one mix was found in each of the groups *Mlh* and *Mmh*.

During the early phases of this study, the effect of aggregate absorption on *Gmm* determined by the Rice procedure was observed to be an interesting phenomenon. In the Rice procedure (ASTM D-2041), there is no guidance on this effect. Because more than half of the aggregate showed medium-to-high asphalt absorption potential, it seemed reasonable that the *Gmm* measured by the Rice procedure might be affected. In this sense, the measured *Gmm* would vary for the same mixture depending on when the sample was tested and its cure time.

This hypothesis was tested for a mixture in the *Mmm* category. Figure 5 shows the result. Loose samples were mixed and placed in an oven according to the Martin procedure. Using the Rice procedure, *Gmm* was determined for the samples at 1, 2, 3, 4, and 5 hr. An additional sample was tested immediately after mixing and cooling. Use of *Gmm* within the range shown would result in as much as 2 percent difference in the calculated air-void content. Based on this experience, all loose mixes used to determine Rice's *Gmm* were oven-cured for 2 hr at 250°F.

Effect of Selected Specific Gravity

For each of the 42 mix designs, asphalt content was determined at 5 air-void contents (1–5 percent) according to Texas agency specifications. It should be noted that asphalt content is normally selected in the range of 3 to 5 percent air voids. In some cases, a value of asphalt content at 5 percent air voids was not available, so no comparison was made (i.e., no extrapolation was used).

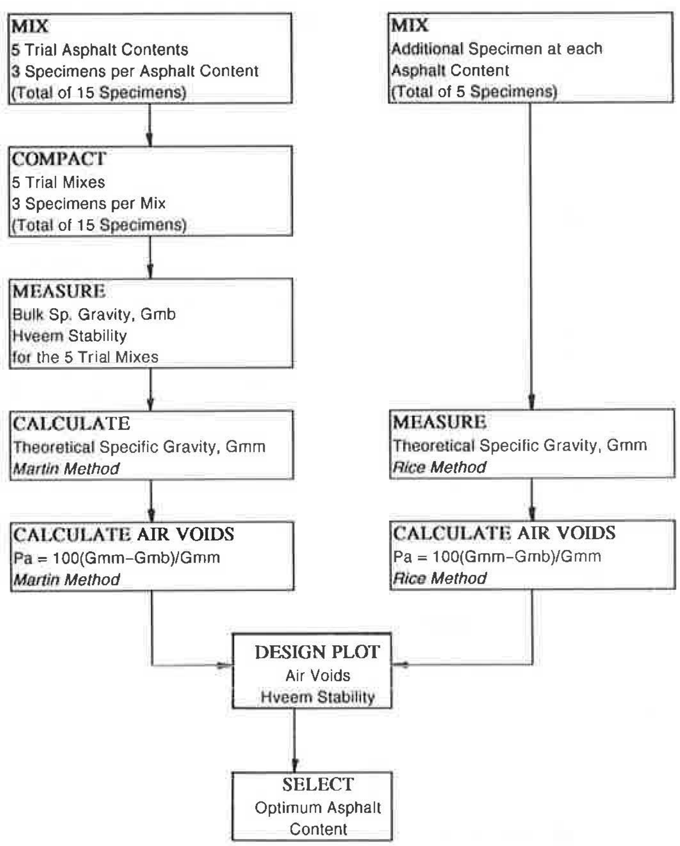

FIGURE 3 Texas method of mix design modified for Rice method.

TABLE 3 FREQUENCY OF OCCURRENCE OF MIX GROUPS

Aggregate Asphalt Absorption Level (%)	*Aggregate Water Absorption Level (%)* Low (l) < 2.0 %	Medium (m) 2.0 - 2.5 %	High (h) > 2.5 %
	[Mll]	[Mml]	[Mhl]
Low(l) < 1.0 %	N = 7 F = 16.7 %	N = 6 F = 14.3 %	N = 0 F = 0.0 %
	[Mlm]	[Mmm]	[Mhm]
Medium (m) 1.0 - 1.5 %	N = 3 F = 7.1 %	N = 11 F = 26.2 %	N = 6 F = 14.3 %
	[Mlh]	[Mmh]	[Mhh]
High (h) > 1.5 %	N = 1 F = 2.35 %	N = 1 F = 2.35 %	N = 7 F = 16.7 %

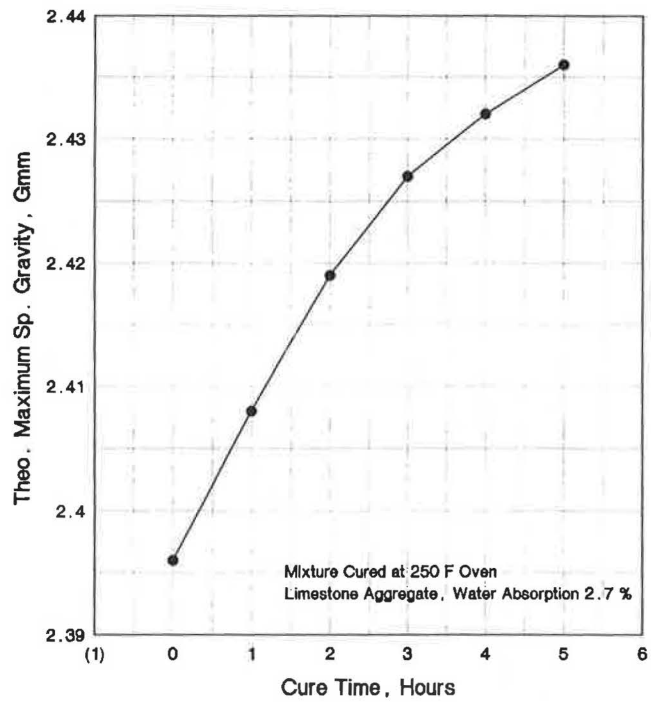

FIGURE 5 Influence of curing time on *Gmm* determination (Rice method).

Results for each cell in Table 2 were grouped to determine the average values of asphalt content at different air-void contents for all mixtures within the same group. The results for all groups are presented in Table 4 and shown in Figure 6. The average difference in asphalt content values between the two methods are shown in Figure 7.

DISCUSSION OF RESULTS

Use of the Rice *Gmm* resulted in higher asphalt contents than those selected from the Martin *Gmm*. When analyzed as a whole, all 42 mix designs at the 5 air-void contents showed that, on average, the Rice *Gmm* resulted in 0.35 percent more asphalt cement (Table 5).

The only exception to this trend was for groups *Mlh* and *Mmh*. Because these groups only consisted of two mixes, no definitive statements can be made regarding trends for those mixtures within the scope of this study.

The results in Figure 7 show that the differences between the Rice and Martin methods are relatively small. To determine the statistical significance of the difference between asphalt contents determined by the two methods, *t*-tests were conducted (*11*). The null hypothesis (H_o) was that there is no significant difference between asphalt content determined

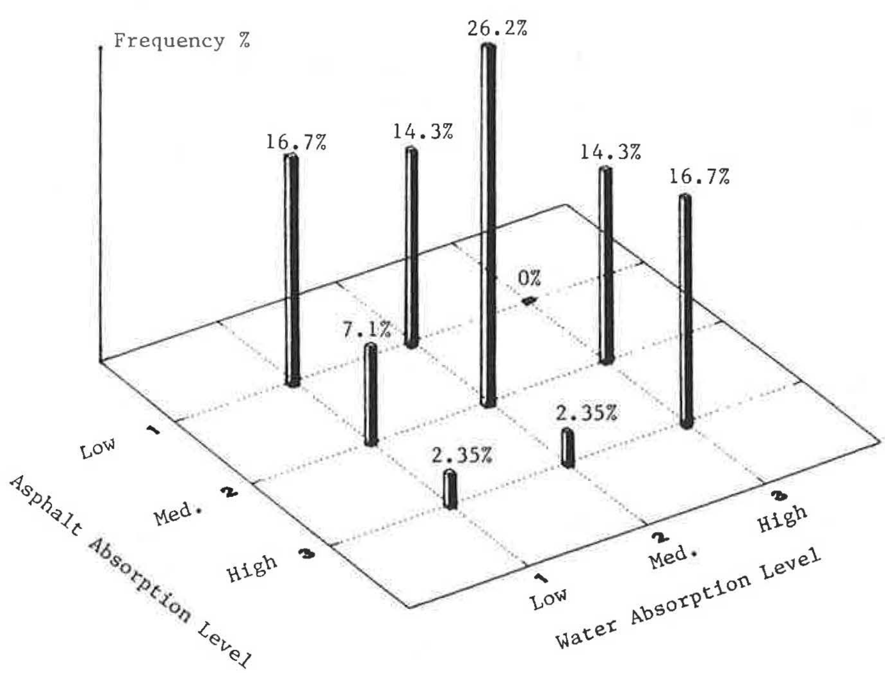

FIGURE 4 Frequency of occurrence of specific mix groups.

TABLE 4 SUMMARY OF AVERAGE ASPHALT CONTENT BASED ON DIFFERENT *Gmm*

Air Voids	AC% Based on: Bulk	Martin	Rice	Difference in AC% M-B	R-B	R-M	AC% Based on: Bulk	Martin	Rice	Difference in AC% M-B	R-B	R-M	AC% Based on: Bulk	Martin	Rice	Difference in AC% M-B	R-B	R-M
	Mll (N = 7)						**Mml** (N = 6)						**Mhl** (N = 0)					
1 %	5.46	5.94	6.60	0.49	1.14	0.66	5.82	6.25	6.88	0.43	1.07	0.63	NO MIXES ARE FOUND IN THIS GROUP					
2 %	5.11	5.50	5.91	0.39	0.80	0.41	5.45	5.80	6.12	0.35	0.67	0.32						
3 %	4.73	5.17	5.36	0.44	0.63	0.19	5.12	5.48	5.65	0.37	0.53	0.17						
4 %	4.53	4.77	4.90	0.24	0.37	0.13	4.75	5.00	5.27	0.25	0.52	0.27						
5 %	4.30	4.66	4.58	0.36	0.28	-0.08	4.70	4.73	4.95	0.03	0.25	0.22						
	Mlm (N = 3)						**Mmm** (N = 11)						**Mhm** (N = 6)					
1 %	5.47	6.33	7.07	0.87	1.60	0.73	5.05	6.11	6.68	1.06	1.64	0.57	5.50	6.82	7.24	1.32	1.74	0.42
2 %	5.17	5.97	6.33	0.80	1.17	0.37	4.74	5.65	5.92	0.91	1.18	0.27	5.13	6.23	7.04	1.10	1.91	0.81
3 %	4.87	5.53	5.77	0.67	0.90	0.23	4.52	5.17	5.41	0.65	0.89	0.24	4.80	5.75	6.47	0.95	1.67	0.72
4 %	4.90	5.23	5.20	0.33	0.30	-0.03	4.30	4.85	4.95	0.55	0.65	0.11	5.10	5.37	5.85	0.27	0.75	0.48
5 %	4.55	4.45	4.80	-0.10	0.25	0.35	4.17	4.47	4.63	0.31	0.46	0.15	4.80	4.98	5.42	0.18	0.62	0.43
	Mlh (N = 1)						**Mmh** (N = 1)						**Mhh** (N = 7)					
1 %	6.10	7.50	7.80	1.40	1.70	0.30	5.30	6.70	6.30	1.40	1.00	-0.40	5.30	7.16	7.44	1.86	2.14	0.29
2 %	5.90	6.90	7.40	1.00	1.50	0.50	5.00	6.00	5.80	1.00	0.80	-0.20	4.93	6.40	6.71	1.47	1.78	0.31
3 %	5.60	6.50	7.20	0.90	1.60	0.70	4.70	5.70	5.30	1.00	0.60	-0.40	4.58	5.97	6.31	1.39	1.73	0.34
4 %	5.10	6.10	5.90	1.00	0.80	-0.20	4.30	5.30	4.90	1.00	0.60	-0.40	4.20	5.51	5.86	1.31	1.66	0.34
5 %	4.80	5.80	5.50	1.00	0.70	-0.30		5.00	4.70			-0.30		5.16	5.37			0.21

NOTE: B= bulk, M= Martin, R= Rice.

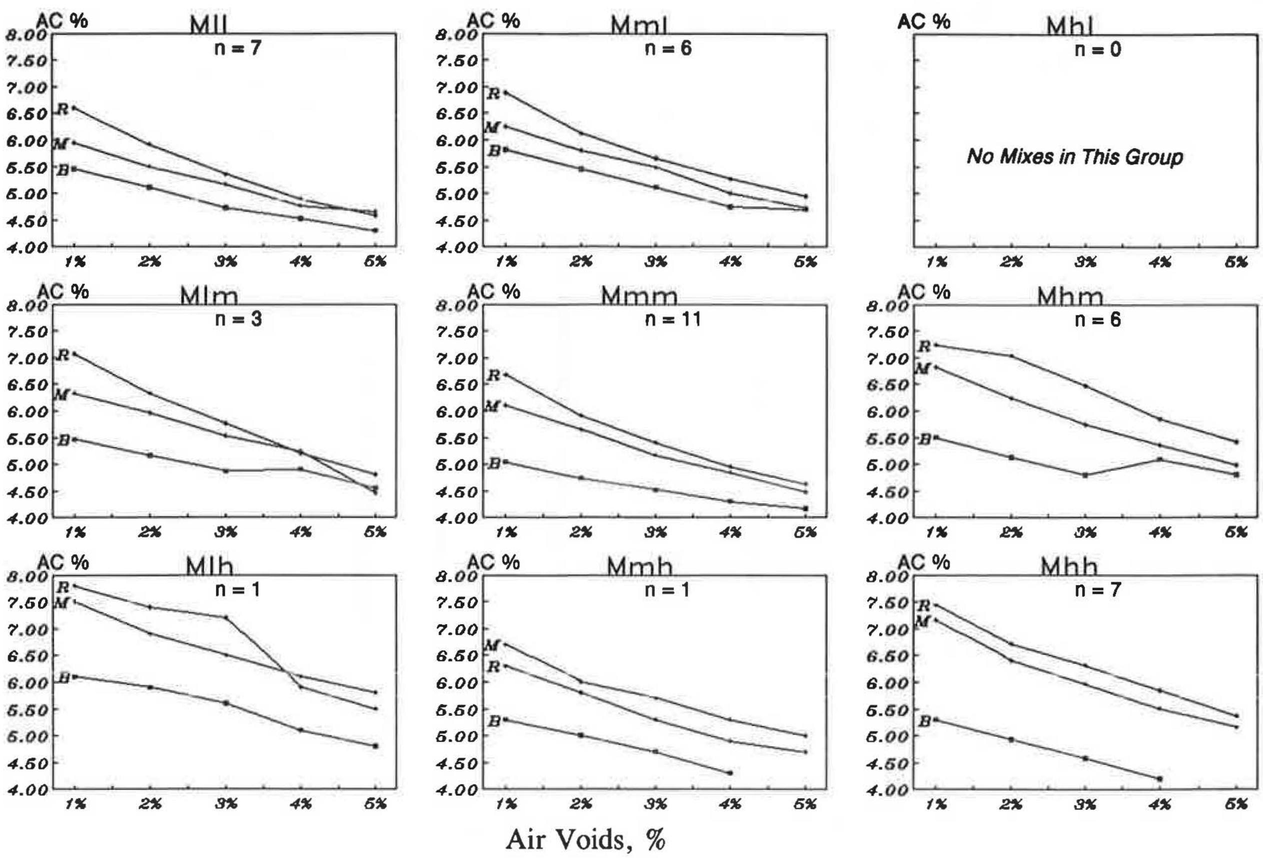

FIGURE 6 Average asphalt percent at different air voids for all mix groups (*B* = bulk, *M* = Martin, and *R* = Rice).

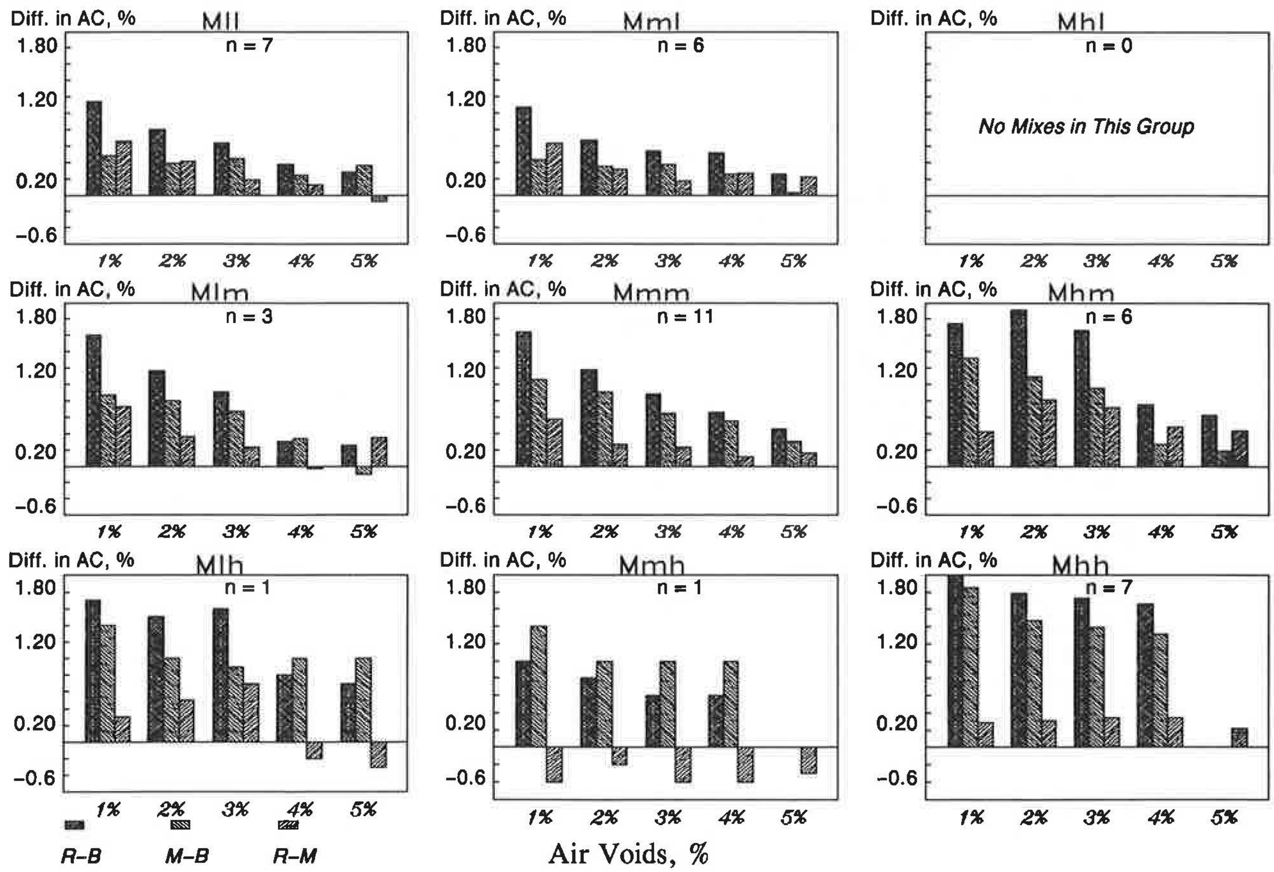

FIGURE 7 Comparison between AC percent determined for different *Gmm* methods (*B* = bulk, *M* = Martin, and *R* = Rice).

by either the Rice or Martin method. Results presented in Table 6 indicate the H_O cannot be rejected for all groups at a 1-percent significance level. Thus at a 1-percent significance level, both methods would be expected to yield a similar asphalt content at a designated level of air voids.

When significance levels were increased to 2 and 5 percent, chances for rejecting H_O became higher for mix groups with absorption characteristics. This indicates that whenever the asphalt absorption of aggregates is relatively high, results from the Rice and Martin methods tend to deviate significantly, whereas results tend to be closer for low-absorption mixtures.

No experimental work was performed to determine the fundamental reason for the observed differences. However, examination of technical literature (*10,12*) yielded a related

TABLE 5 OVERALL AVERAGE OF THE DIFFERENCE IN AC CONTENT

Parameter	Rice – Bulk	Martin – Bulk	Rice – Martin
N	167	174	201
Overall Average Diff. in AC %	1.204	0.874	0.354
Overall Std. Dev. Diff. in AC %	0.791	0.658	0.582

TABLE 6 STATISTICAL TEST ON THE SIGNIFICANCE OF THE DIFFERENCE IN AC CONTENT DETERMINED BY RICE AND MARTIN METHODS

Parameter for t-test	*Designated Mix Group*					
	[Mll]	n=7	[Mml]	n=6	[Mhl]	n=0
Air Voids, %	4%	3%	4%	3%		
Difference in AC %	0.13	0.19	0.27	0.17		
t =	1.213	1.518	2.793	2.193		
t (cr) (1% S.L) =	4.032 A	3.707 A	4.032 A	4.032 A	No Mixes Available, t-test not applicable	
t (cr) (2% S.L) =	3.365 A	3.143 A	3.365 A	3.365 A		
t (cr) (5% S.L) =	2.571 A	2.447 A	2.571 R	2.571 A		
	[Mlm]	n=3	[Mmm]	n=11	[Mhm]	n=6
Air Voids, %	4%	3%	4%	3%	4%	3%
Difference in AC %	-0.03	0.23	0.11	0.24	0.48	0.72
t =	-0.184	0.819	2.794	2.277	2.185	2.090
t (cr) (1% S.L) =	63.657 A	9.925 A	3.169 A	3.169 A	4.032 A	4.032 A
t (cr) (2% S.L) =	31.821 A	6.965 A	2.764 R	2.764 A	3.365 A	3.365 A
t (cr) (5% S.L) =	12.706 A	4.303 A	2.228 R	2.228 R	2.571 A	2.571 A
	[Mlh]	n=1	[Mmh]	n=1	[Mhh]	n=7
Air Voids, %					4%	3%
Difference in AC %					0.34	0.34
t =					2.925	3.618
t (cr) (1% S.L) =	Only ONE Mix, t-test not applicable		Only ONE Mix, t-test not applicable		3.707 A	3.707 A
t (cr) (2% S.L) =					3.143 A	3.143 R
t (cr) (5% S.L) =					2.447 R	2.447 R

Null Hypothesis (Ho) : There is no Difference Between AC% Determined by Either Rice Martin Method

Decision : Reject Ho if | t | > t (cr) A: Accept (Do not reject) R: Reject

comment. In the discussion following a 1957 paper (*13*), McLeod was asked to comment on the veracity of the Martin approach. He said:

> Experience tends to indicate that it would be very difficult, if not impossible, to expel all the air from some paving mixtures in spite of an appreciable excess of bitumen. If it is assumed that these mixtures contain no air, the values obtained for the effective specific gravity of the aggregate and for the bitumen absorbed by the aggregate will be in error by the amount of the entrapped air that actually remains in these compacted rich mixes.

McLeod's comment seems to offer the most plausible explanation for the observed differences: the volume of air that could not be displaced by increasing asphalt contents.

During placement of some of these mix designs, the impact of *Gmm* on quality control was accentuated. A serious failing of the Martin approach is that it only reflects mixture properties during the design phase. Properties of mixtures normally vary during construction. Use of this method will not account for normal fluctuation during production. Such variation in the specific gravity and absorption characteristics of aggregates will not be detected using an assumed *Gmm* from the mix design. The Rice method, performed on plant-produced materials, will detect this type of variation.

CONCLUSIONS

The Martin approach in determining *Gmm* is fundamentally sound. It rigorously addresses the issue of asphalt absorption during the mix-design phase. Because it only reflects mixture properties during the phase, however, the Martin method lacks applicability during plant quality control.

The Rice method resulted in about 0.35 percent more asphalt than did the Martin method. For mixtures with low absorption characteristics, the difference between the two methods was insignificant. The difference becomes more pronounced for mixtures with high absorption characteristics. The most likely cause of the difference is that it is impossible to saturate all air voids in a compacted specimen. The volume of air remaining in an assumed saturated sample will create an equal error in subsequent volumetric calculations.

The *Gmm* determined from the Rice method is dependent on time and absorption. Use of the oven-curing step inherent in the Martin approach was necessary to achieve accurate and precise values of *Gmm* from the Rice method.

The study revealed that caution should be exercised when mixes include absorptive aggregates. The aggregates should be permitted to absorb free asphalt before the test is conducted.

In 1989 and 1990, the Texas SDHPT began an extensive update of design procedures and specifications. During this process, it became apparent that use of the Rice method is desirable. SDHPT staff indicated that they believed the Rice method to be more precise as a mixture test. In addition, use of the Rice method to determine *Gmm* in the design phase was consistent with its use as a reference for in-place density control.

Many SDHPT districts and various cities and counties already have begun to use the Rice method. A phase-in period for all SDHPT districts will end in 1993. This will end formal use of the Martin method, which has been used by most agencies in Texas for more than 40 years.

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Reflective Cracking and Tenting in Asphaltic Overlays

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The effects of reflective cracking on the performance of recycled asphaltic pavement overlays were investigated. Two 1/2-mi sections for full removal of the existing asphaltic pavement and replacement with recycled material on the existing crushed gravel base course were selected. The pavement performance after partial removal of the existing asphaltic pavement and placement of a recycled overlay was compared with the performance after complete removal and replacement with recycled asphaltic pavement. Severe tenting (called tenting because of the tent-like structure produced at transverse cracks) developed during the freeze months of January, February, and March. The Present Serviceability Index (PSI), an indicator of ride quality, deteriorated from 3.4 in October 1985 to 1.7 in February 1986 (on a scale of 0 to 5, 5 being the best). In an attempt to solve this serious tenting problem, three different treatments were applied to the pavement. Retrofit edge drains were installed in 1987 to remove infiltrated water from the pavement structure. Retrofit transverse drains or transverse interchannel flow (TIC) drains were installed in 1988 directly under the transverse cracks to remove the infiltrated water. Crack sealing was performed in 1988 to prevent water from entering the pavement through the transverse cracks. Reflective crack study results in the 7-year period 1981 to 1988 indicate 58 percent more transverse (reflective) cracks occurred in the partial removal and recycled overlay sections than in the full removal and replacement sections. The performance, as measured by PSI, deteriorated at a faster-than-normal rate, which appeared to be increasing. The retrofit edge drains and retrofit TIC drains were found to be ineffective in solving the tenting problem. Transverse crack sealing appeared to be somewhat successful.

Construction of a recycled asphaltic overlay project, which called for 100,000 tons of recycled asphaltic paving, was begun in District 7 in Rhinelander, Wisconsin, in April 1981. Major steps in the project were cold milling the top 3½ in. of the 6-in. existing asphaltic concrete pavement and replacing it with a 4-in. overlay of recycled material. After the cold-milling operation had started, the chief construction engineer examined the milled surface and became concerned about the reflective cracking that occurred soon after completion of the overlay. The remaining mat (placed in 1956) had many relatively wide transverse cracks. This concern about reflective cracking and its possible effect on pavement performance led to the suggestion of a study to evaluate the effects of reflective cracking on the performance of recycled asphaltic overlays.

In May 1981 the Engineering Research Advisory Committee, now the Council on Applied Research, agreed that two ½-mi sections for full removal of the existing 6-in. mat and replacement with 5 in. of recycled material on the existing crushed gravel base course be constructed. This would allow a comparison of the pavement performance after partial removal and overlay with the performance after full removal and replacement.

After construction of the project, severe tenting (called tenting because of the tent-like structure produced at transverse cracks) developed during the freeze months of January, February, and March. In 1985, the District Materials Section began monitoring the effects of tenting in the reflective crack test sections, using both the road meter and the California-type profilograph. The road meter measures the ride quality of the pavement. The Present Serviceability Index (PSI) deteriorated from 3.4 in October 1985 to 1.7 in February 1986. In an attempt to solve this serious tenting problem, three different treatments were applied to the pavement: retrofit edge drains, retrofit transverse interchannel flow (TIC) drains, and crack sealants.

PROJECT DESCRIPTION

This 22-mi project is located on US-51 between Mercer and Hurley in Iron County. Except for two short grading sections totaling approximately 1 mi, the remainder of the 22 mi consists of an old 20-ft-wide portland cement concrete (PCC) pavement covered with 9 in. of gravel and 6 in. of asphaltic concrete. The asphaltic concrete consists of two 3-in. mats, one placed when the gravel lift was built in 1956, the second in 1969. The grading sections had the same pavement structure except that the 9-in. gravel lift was placed on the prepared earth subgrade instead of the old PCC pavement.

The top 3½ in. of the 6-in.-thick, 22-ft-wide existing bituminous concrete pavement was cold milled and replaced with a 30-ft-wide, 4-in. overlay of recycled asphaltic pavement. The old 2½ in. asphaltic pavement remained after milling and carried 2-way traffic during construction.

PURPOSE AND SCOPE

The objectives of this study were to monitor and evaluate the pavement performance over a period of several years. The effects were determined of reflective cracking in sections in which the existing asphaltic concrete was partially removed and a recycled asphaltic overlay was placed. The pavement performance after partial removal of the existing asphaltic pavement and placement of a recycled overlay was compared with the performance after complete removal and replace-

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ment with recycled pavement. Four typical pavement test sections were established as part of the reflective crack research study, and results are based on evaluations of the performance of these four sections.

TEST PROCEDURES

The four test sections were monitored and evaluated by means of a transverse crack survey; a California profilograph profile; material durability evaluation; and the Present Serviceability Index (PSI).

Transverse Crack Survey

A crack survey was conducted in July 1982 after removal of the upper 3½ in. of asphaltic concrete and before the placement of the 4-in. recycled asphaltic overlay. This initial survey was made on only sections 1 and 3 (partial removal and replacement sections). The survey was conducted by District 7 personnel and consisted of measuring and plotting each transverse crack to the nearest foot on a crack diagram. Beginning in November 1983, and each succeeding year through 1988, a crack survey was performed in all four test sections. All transverse cracks were again measured and plotted to the nearest foot on a crack diagram overlay. Transverse cracks that appeared in the overlaid surface of sections 1 and 3 were considered reflective cracks if they were within 1 ft of the original crack in the overlaid pavement surface. The cracks in sections 2 and 4 (full removal sections), were not reflective cracks. No crack survey was conducted in 1985.

California Profilograph Survey

The profilograph is a mobile testing instrument designed to register and record deviations in a pavement surface. The Profile Index (PI) is defined as "inches per mile in excess of the 0.2-in. blanking band." As the pavement roughness increases, the numerical value of the PI increases.

A survey was conducted November 1983 through November 1988, and in March 1987, February 1988, and 1989. No survey was conducted in 1985. The PI was determined by averaging the two northbound and southbound wheel ruts.

Material Durability Evaluation

Cores were taken at transverse cracks to investigate the deterioration. Eight-in. diameter cores were taken in the following sections at the following locations:

1. Pavement Section 1 (partial removal) at Station 108 + 0, 9 ft to the right of the centerline and
2. Pavement Section 2 (full removal) at Station 95 + 17, 12 ft to the right of the centerline and 12 ft to the left of the centerline.

Present Serviceability Index

Users assess the condition of a pavement largely by ride quality. Serviceability is quantified by means of the PSI, which is measured in Wisconsin by the road meter. PSI values range from 0 (impassable road) to 5 (perfect road). Selection of the lowest allowable PSI or Terminal Serviceability Index (P_T) is based on the lowest index that will be tolerated before rehabilitation, resurfacing, or reconstruction becomes necessary. An index of 2.5 or higher is suggested for design of major highways; 2.25 is suggested for highways with lower traffic volumes.

Road meter inventory PSI results are available on sections 1–4 for the following months: August 1981, November 1983, November 1984, November 1986, March 1987, November 1987, February 1988, November 1988, and February 1989.

EXPERIMENTAL PAVEMENT TREATMENT SECTIONS

TIC Drains

Slotted pipes were installed in September 1988 directly under the transverse crack at the interface of the asphaltic pavement and the gravel base course from Station 762 to Station 794. Twelve drains were installed from Station 762 to Station 794.

The intent of the slotted pipe was to drain infiltrated water from the area of a transverse crack (through the use of a mini-French drain) in an effort to prevent tenting during winter frost.

Two methods of installing the pipe were used. The retrofit method uses a pneumatic piercing tool to advance a hole (tunnel) from the edge of the asphaltic pavement to the centerline. The tool is backed out and a PVC slotted pipe is inserted. The trench method uses a wheel trencher to cut a 3-in. slot along the transverse crack about 12 in. deep. A slotted PVC pipe is installed in the trench and backfilled with washed limestone chips to the bottom of the existing asphaltic pavement and then backfilled with asphaltic material to match the adjacent pavement thickness.

The PVC drains slope and flow onto the inslope. The Wisconsin Department of Transportation (WisDOT) Applied Research Section installed a tipping-bucket flow meter to measure flow at one location. The performance of the TIC drain system was evaluated by a California-type profilograph and by a tipping-bucket flow meter.

A profilograph survey was conducted by WisDOT in February 1988 before the installation of the slotted pipe at transverse cracks, and another profile was taken after the pipes were installed in February 1989. Both profiles were taken at the time of maximum tenting. The effectiveness of the slotted pipe system was evaluated by comparing the magnitude of the tents at transverse cracks in 1988 with the magnitude of the tents in 1989.

Retrofit Edge Drains

In July 1987 two different types of edge drains were placed on US-51 from Station 452 to Station 552. Six test sections were established to monitor the performance of the different edge drain sections. Test sections 1 and 6 were control sections and did not have edge drains. Test section location, type of edge drain, and other information pertinent to the test section follow:

- Section 1: stations 60 to 166 (Control Section 1), three moisture-temperature cells in a 9-in. (Grade 2 crushed aggregate) dense graded base course with existing 6-in. asphaltic concrete overlay.
- Section 2: longitudinal edge drains right and left with 18-in.-deep geocomposite edge drain fabric wrapped, aggregate-filled trench with 4-in. perforated PVC pipe discharge to slope outfalls, three moisture-temperature cells in a 9-in. (Grade 2 crushed aggregate) dense graded base course with existing 6-in. asphaltic concrete overlay, 6-in. jointed concrete plain pavement.
- Section 3: stations 477 to 502, longitudinal edge drains right and left 30-in. deep, geocomposite edge drain fabric wrapped, aggregate-filled trench with 4-in. perforated PVC pipe discharge to slope outfalls, three moisture-temperature cells in a 9-in. (Grade 2 crushed aggregate) dense graded base course with existing 6-in. asphaltic concrete overlay, 6-in. jointed concrete plain pavement.
- Section 4: stations 502 to 527, longitudinal edge drains right and left 18 in. deep, flowed edge drained fabric wrapped, 4-in. perforated PVC pipe discharge to slope outfalls, three moisture-temperature cells in a 9-in. (Grade 2 crushed aggregate) dense graded base course with existing 6-in. asphaltic concrete overlay, 6-in. jointed concrete plain pavement.
- Section 5: stations 527 to 552, longitudinal edge drains right and left 30-in. deep, plowed edge drain fabric wrapped, 4-in. perforated PVC pipe discharge to slope outfalls, 6 moisture-temperature cells in a 9-in. (Grade 2 crushed aggregate) dense graded base course, 9 to 15 in. sand and gravel interlayer, 6-in. jointed concrete plain pavement.
- Section 6: stations 552 to 770 (Control Section 2), three moisture-temperature cells in a 9-in. (Grade 2 crushed aggregate), dense graded base course, 6-in. jointed concrete plain pavement.

Performance evaluation of the edge drains is based on data from the moisture-temperature cells in the gravel base course, daily tipping-bucket outfall and rain data at Station 508 + 15, results of permeability analysis of base course samples, and ride quality of the pavement as measured by the PSI and PI. PSI and PI data from 1986 and 1987 before the installation of the edge drains are available for comparison.

Transverse Crack Sealant

Three test sections were established to determine if tenting could be reduced by sealing transverse cracks to prevent the entrance of infiltrating water. The test sections were as follows: (*a*) stations 67 to 96, (*b*) stations 505 to 510, and (*c*) stations 775 to 800.

A profile of the transverse cracks and a PI of the above sections were established with a California-type profilograph in February 1988 before filling the cracks.

The transverse cracks were sealed with a rubberized hot asphalt sealant in September 1988; the cracks were not cleaned. Profiles were once again taken in February 1989 to determine the effectiveness of crack sealing in reducing tenting.

The sealant was applied again in July 1989. This application was much more thorough. Each crack was routed ¾ in. wide and deep and cleaned and dried with a hot air heat lance before sealing. Profile measurements were taken with a profilograph in February 1990 to evaluate the effectiveness of sealant in reducing tenting.

FACTORS AFFECTING PAVEMENT PERFORMANCE

Material Durability

Both the existing material and the material in the overlay will affect the life of the pavement. Most overlay design procedures do not address specific material requirements. It is assumed that both the existing and overlay material will be constructed of durable material and that the proper specifications will be used to ensure this. The existing materials should first be investigated to determine that they have not deteriorated. An overlay designed for a 20-year life placed on an existing pavement that is constructed on materials that will fail in 7 years as a result of durability problems will not reach its design life no matter how well it is designed.

Climate

An overlay that would perform satisfactorily in a warm, dry climate may not perform as well in a cold, wet climate. This is particularly true when reflective cracks are a significant problem. Climates that have extreme heat, extreme cold, continuous moisture, and many freeze-thaw cycles require special consideration.

A "lake effect micro climate" is located in the Hurley, Wisconsin, area and extends from Michigan west to Birch Hill on US-2, west to Upson on STH 77, and south to Pine Lake on US-51.

Because of the continuous precipitation and many freeze-thaw cycles between November and April, this climate may have adverse affects on pavement performance and may require special consideration.

DISCUSSION OF RESULTS

Reflective Crack Research

This part of the research was limited to pavement test sections 1–4. The performance was evaluated through a comparison of the partial removal sections with the full removal sections. Section 1 was compared with Section 2, which is over gravel base course. Section 3 was compared with Section 4, which is over gravel base and old PCC.

Performance of the partial removal and overlay section was evaluated from data collected from the transverse crack survey, the profilograph survey, the material durability investigation, and the pavement serviceability survey.

Transverse Crack Survey

Results of the transverse crack survey for Test Section 1 (partial removal and replacement) compared with Test Section 2 (full removal) is shown in Figure 1. Both sections were constructed over gravel base. Figure 2 shows the rate of reflective cracking, which could only occur in Test Section 1 (partial

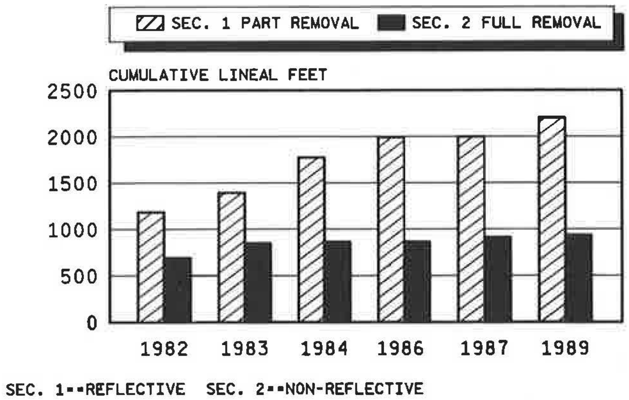

FIGURE 1 Transverse crack counts for test sections 1 and 2 (over gravel base).

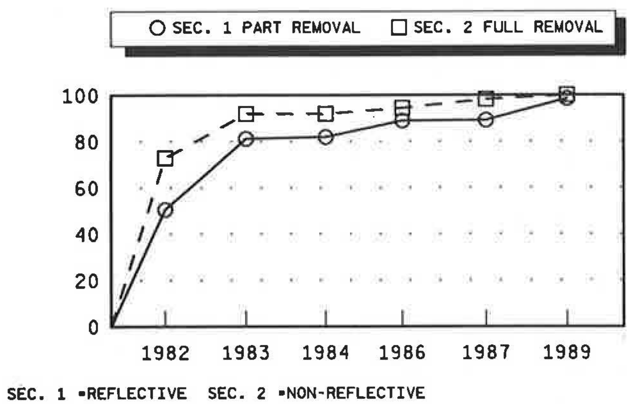

FIGURE 2 Rate (percent) of transverse cracking over gravel base, 1981–1989.

removal and replacement). No reflective cracking could occur in the full removal sections because there were no cracks to reflect.

The data show that there was 6,663 linear ft of transverse cracks in Section 1 in the old asphalt surface after the milling operation and before the placement of the recycled asphaltic overlay in July 1981. By 1989, 8 years after construction, 2,164 linear ft, or 32 percent, had reflected through the 4-in. recycled asphaltic overlay. Figure 2 shows that of the total number of cracks that reflected in the first 8 years, 52 percent occurred in the first year, and an additional 29 percent occurred in the second year, totaling 81 percent in the first 2 years. This indicates that the majority of the reflective cracking takes place in the early years of the overlays.

The comparison of the amount of reflective cracking in Section 1 with the amount of cracking (nonreflective) in Section 2 (full removal) shows that there was 915 lineal ft of cracking in Section 2; 58 percent more cracks occurred in Section 1. Figure 2 shows that of the total cracks that occurred in Section 2 in the first 8 years, 73 percent occurred in the first year, and an additional 17 percent in the second year, for a total of 90 percent in the first 2 years.

Results of the transverse crack survey for Test Section 3 (partial removal and replacement) compared with Test Section 4 (full removal) are shown in Figure 3.

Figure 4 indicates the rate (percent) of transverse cracking that occurred in 8 years. Both sections were constructed over an old PCC and gravel interlayer.

The reflective cracking that occurred in Section 3 shows that there was 4,425 linear ft of transverse cracks in the old asphaltic surface after the milling operation and before the placement of the recycled asphaltic overlay in July 1981. In 1988, 8 years after construction, 1,395 linear ft, or 32 percent, had reflected through the 4-in. recycled asphaltic overlay. Figure 4 shows that of the total cracks that reflected in Section 3 in the first 8 years, 56 percent occurred in the first year and an additional 17 percent during the second year, totaling 73 percent in the first 2 years.

The comparison of the amount of reflective cracking in Section 3 with the amount of cracking in Section 4 shows that 825 linear ft of cracking occurred in Section 4; 59 percent more cracking occurred in Section 3. Figure 4 shows that of the total cracks that occurred in the first 8 years in Section 4, 69 percent occurred in the first year, and an additional 16 percent occurred in the second year, for a total of 85 percent in the first 2 years.

Lastly, the cracking that occurred in sections 1 and 2, which were constructed over a gravel base, and sections 3 and 4, which were constructed over an old PCC with a gravel interlayer were compared. The data show that Test Section 3 (over concrete) had 4,425 linear ft of transverse cracks in the ex-

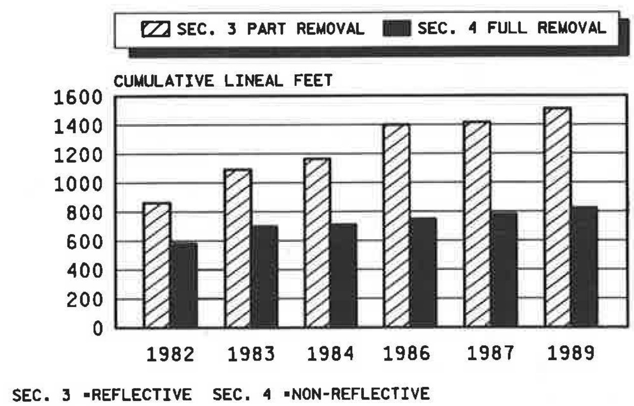

FIGURE 3 Transverse crack counts for test sections 3 and 4 (over old PCC with gravel interlayer).

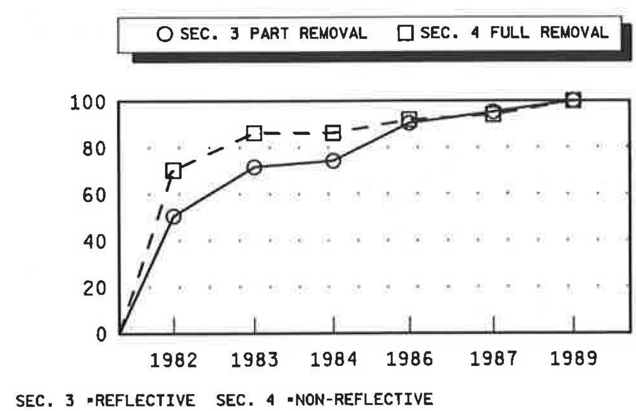

FIGURE 4 Rate (percent) of transverse cracking over old PCC with gravel interlayer, 1981–1989.

isting asphaltic pavement after milling and before placement of the asphaltic overlay. Section 1 (over gravel base only) had 6,663 linear ft, or 51 percent more, existing transverse cracks than Section 3. After 8 years, the recycled asphaltic overlay in Test Section 3 had 1,510 linear ft of transverse cracks. Section 1 had 2,164 linear ft, or 43 percent more, transverse cracking. In the full removal sections, 825 linear ft of transverse cracking occurred in Test Section 4 (over concrete), and 915 linear ft, or 11 percent more, transverse cracks occurred in Test Section 2 (over gravel base).

California-Type Profilograph Survey

Figure 5 shows the PI for test sections 1–4. The profiles were taken in November 1983, 1984, 1986, 1987, and 1988. The PIs shown in Figure 5 are the averages of the profiles taken in the two northbound and the two southbound wheel ruts. The November 1983 profiles were taken after the subgrade was frozen; thus, the results are not reliable and will not be used.

Results of the profilograph survey for Test Section 1 (partial removal and replacement) compared with Test Section 2 (full removal) are shown in Figure 5. Both sections were constructed over a gravel base. The data indicate that Test Section 1 had a lower initial PI (was less rough) than Section 2. The PI deteriorated at the same rate from November 1984 to 1987. In 1987, Section 1 continued to deteriorate at about the same rate, but the rate of deterioration in Section 2 leveled off or decreased.

Results for Test Section 3 (partial removal and replacement) compared with Test Section 4 (full removal) also are shown in Figure 5. Both sections were constructed over an old PCC with a gravel interlayer. The data indicate that Test Section 3 had a slightly higher initial PI (was rougher) than Section 4. The PIs deteriorated at the same rate from November 1984 to 1987. In 1987, Section 3 continued to deteriorate at about the same rate, but the rate of deterioration in Section 4 leveled off or decreased.

Both sections 1 and 2, constructed over the gravel base, have PIs about 10 points lower than those of sections 3 and 4, which were constructed over the PCC. Therefore, sections 1 and 2 are smoother than sections 3 and 4.

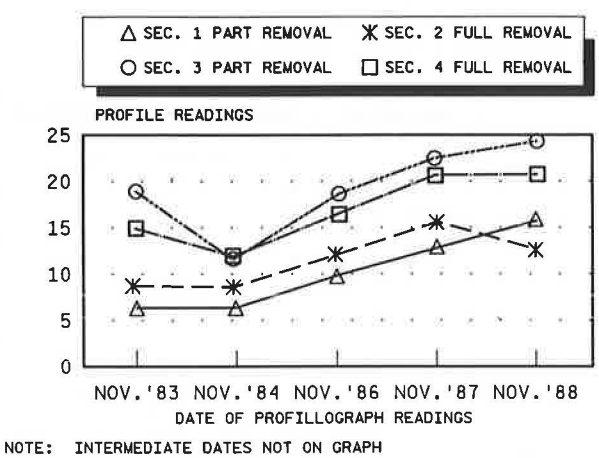

FIGURE 5 PI for test sections 1–4, 1983–1988.

Material Durability Investigation

Cores were taken to investigate the deterioration of the recycled asphaltic material at the transverse cracks. Eight-in.-diameter cores were taken in Test Section 1 (partial removal and overlay) and Test Section 2 (full removal). Both sections were constructed over a dense graded gravel base course, which had a permeability range of 0 to 30 ft per day.

Figures 6 and 7 are photographs of cores of transverse cracks in Test Section 2. The most important result of this investigation is the V-shaped deterioration at transverse cracks that occurred at the interface of the asphaltic pavement and the gravel base. Figures 6 and 7 show cores taken at Station 95 + 17. The core in Figure 6 was taken on the high side of a superelevated curve. It shows poorly developed V-shaped deterioration, whereas the core in Figure 7 was taken on the low side and shows a fairly well-developed V-shaped deterioration. This deterioration could be the result of pumping at the transverse cracks. The core of the high side of the superelevated curve would have less water, and therefore less pumping and less deterioration.

Figure 8 is a photograph of a core of transverse cracks in Test Section 1 at Station 108 + 10. Note the large well-developed V-shaped deterioration. The deterioration developed in the old asphaltic pavement placed in 1956, and was subjected to pumping for 32 years.

PSI Survey

PSI data may not be reliable for two reasons: test sections are short, and the extremes in temperature affect the results. Reliable PSI data from test sections 1 and 2 are presented below.

Test year	*PSI*
1980 (before construction)	3.0
1981	4.1
1982	4.1
1984	3.9
1986	3.5
1988	3.3

Reliable PSI data from test sections 3 and 4 are presented below.

Test year	*PSI*
1980 (before construction)	3.4
1981	4.0
1982	3.9
1984	3.7
1986	3.4
1988	3.2

Evaluation of Tenting

A comparison was made to show how different pavement sections react to tenting. The four sections are combined into two groups on the basis of similar pavement components. The two groups are as follows: Group 1—US-51, sections 1 and 3, partial removal and asphaltic overlay over gravel base (dense graded); Group 2—US-51, sections 2 and 4, full removal and replacement with recycled asphaltic pavement over gravel base (dense graded).

FIGURE 6 Core of transverse crack taken from high side of superelevated curve in Section 2.

FIGURE 7 Core of transverse crack taken from low side of superelevated curve in Section 2.

FIGURE 8 Core of transverse cracks in Section 1.

Research data indicate that the above sections are structurally adequate according to WisDOT design criteria. The climate in all sections is locally severe because of lake effect moisture and many freeze-thaw cycles. Subgrade soils for all groups are similar.

In the comparison of transverse cracks, the number per 500 ft was used as a standard. Data indicate 12 transverse cracks per 500 ft in Group 1, partial removal and replacement, and 6 transverse cracks per 500 ft in Group 2, full removal. Asphaltic cement penetration, which is normally considered to have an effect on transverse cracking, was not evaluated. The magnitude of the tents at transverse cracks as shown on the profilogram are about the same for both groups. The ride quality as measured during the most severe tenting period (February) for both groups was as follows: the PSI was 1.3, and the PI was 80.

Experimental Pavement Treatment Sections

Retrofit TIC Drain

Table 1 shows the location, description of the type of installation, and performance as measured by the profilograph. The performance is measured by the changes in the magnitude of the scallops in tenths of an inch from February 1988 before pipes were installed to February 1989 after pipes were installed.

The magnitude of most of the scallops have increased significantly, except for pipe numbers 4–6, which decreased slightly. In the performance evaluation it was observed that from February 1988 to February 1989 the average PI decreased significantly on all test sections.

No positive results were recorded from the tipping-bucket flow meter. A tipping bucket was installed in late fall; freezing temperatures and deep snow prevented reliable results.

Retrofit Edge Drains

A comprehensive evaluation of the edge drain study was made by Sharma.

Transverse Crack Sealant Performance

Table 2 shows that performance, as measured by the profilograph, was significantly better in February 1989 after the cracks were sealed. From February 1988 to February 1989, the PIs decreased on an average of 23 in./mi. These data were obtained from the profile index on 2 mi of test section. Even considering the overall decrease in the PI, test sections 1 and 2 performed better in February 1989 after being sealed.

CONCLUSIONS

1. The overlay sections (partial removal and replacement) developed 58 percent more transverse cracks than the full removal sections.
2. Most of the transverse cracks (70 to 90 percent) appeared in the first 2 years after construction.
3. Substantially less transverse cracks have developed in both the overlay and full removal test sections, which were constructed over the old PCC pavement with a gravel interlayer. Data indicate about 50 percent less transverse cracks in the milled pavement surface before overlaying (1981), 43 percent less transverse cracks in the recycled overlay pave-

TABLE 1 SLOTTED PIPE PERFORMANCE

	LOCATION & DESCRIPTION	WHEEL RUTS			
		NBout	NBin	SBout	SBin
No. 1	Sta. 749+63 Lt. & Rt.				
	Retro. .010" slot wo/sock	I-3.5	I-3.0	I-2.0	I-1.0
No. 3	Sta. 750+84 Lt. & Rt.	0.0	0.0	0.0	0.0
	Retro. .020" slots wo/sock	Note: Ave. scallops is 1 inch			
No. 4	Sta. 751+47 Rt.				
	Retro. .020" slots w/sock	I-0.5	D-2.0	D-2.5	D-1.0
No. 5	Sta. 752+36 Lt. & Rt.				
	Retro. .020" slots w/sock	I-1.5	D-1.5	I-2.0	D-3.5
No. 6	Sta. 752+72 Rt.				
	Retro. .020" slots wo/sock	D-1.0	D-1.0	0.0	D-1.5
No. 2	Sta. 753+64 Rt.				
	Retro. .020" slots w/sock	I-0.5	0.0	I-0.5	D-1.0
No. 8	Sta. 755+25 Rt.				
	Trench (3"x10") HDPE w/sock	I-1.5	I-0.5	I-3.5	I-1.5
No. 9	Sta. 756+10 Rt. & Lt.				
	Trench (3"x10") HDPE w/sock Rt.				
	Trench (3"x8") HDPE w/sock Lt.	I-2.5	I-3.0	I-3.5	I-0.5
No. 10	Sta. 756+37 Rt. & Lt.				
	Trench (3"x10") HDPE wo/sock				
	Trench (3"x8") HDPE w/sock Lt.	I-5.5	I-5.5	I-7.0	D-1.5
No. 11	Sta. 758+17 Lt. & Rt.				
	Trench (3"x10") .020 slots				
	PVC w/sock Rt.				
	Trench (3"x8") .020" slots				
	PVC w/sock Lt.	I-7.0	I-8.5	I-4.5	I-1.0
No. 12	Sta. 760+24 Rt.				
	Trench (3"x10") .020 slots				
	PVC w/sock Rt.	0.0	D-1.5	0.0	D-1.0
No. 7	Sta. 761+60 Rt.				
	Retro. .010" slots				
	PVC wo/sock Rt.	I-2.0	I-2.0	D-1.0	I-0.5

NOTE: All trenched pipes have solid PVC from edge of pavement to outlet.
Outlets have rodent protection.

All trenches are 3"x10" on the right. All trenches are 3"x8" on the left

I = Increase D = Decrease

TABLE 2 TRANSVERSE CRACK SEALANT PERFORMANCE

	Avg PI		
Test Section	February 1988	February 1989	Change
1 (stations 67 to 96)	80.1	35.4	44.7 decrease
2 (stations 505 to 510)	81.2	46.9	34.3 decrease
3 (stations 775 to 800)	54.8	58.9	4.1 increase

NOTE: PIs are the average PI of the two northbound and the two southbound wheel ruts.

ment sections 7 years after construction (1988), and 11 percent less transverse cracks in the full removal and replacement sections.

4. The overlay sections and full removal sections had deteriorated at about the same rate (2.5 in./mi/year) from 1984 to 1987. From 1987 to 1988, the overlay sections continued to deteriorate at the same rate, whereas the full removal sections did not deteriorate.

5. The average PSI for all four sections deteriorated from 4.1 in 1981 to 3.2 in 1988, a loss of 0.13 PSI a year.

6. The severe deterioration (V-shaped void) at the interface of the transverse crack and gravel base in Section 1 (partial removal and overlay) was caused by pumping. The deterioration was less severe in the full removal section because the section had been exposed to pumping for only 7 years.

7. Tenting is a severe problem on US-51 during the months of December, January, and February. The ride quality of sections 1–4 during the winter and summer months is compared in the following table.

	Summer	*Winter*
PI	18.6	80
PSI	3.2	1.3

8. TIC drains appear to be ineffective in solving the tenting problem.

9. Performance as measured by profilograph surveys was significantly better in two of three test sections where cracks were sealed. Sections were resealed in July 1989 and reevaluated in February 1990.

RECOMMENDATIONS

1. The transverse cracks should be investigated as part of the pavement performance evaluation in the selection of the type of pavement rehabilitation. If core samples show severe structural deterioration (V-shaped voids at the interface of the transverse crack and the gravel base) the asphaltic pavement should not be overlaid. The existing pavement should be completely removed and either relaid cold and compacted as a base course, or relaid as a recycled asphaltic material.

2. The performance of the test sections should be monitored for the next few years to determine the rate of pavement deterioration.

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Operation and Automation of Finishers

JEAN LOUIS GOURDON AND MARIE-LINE GALLENNE

New measuring techniques that have made it possible to study the effect of a finisher's controls (height of tow points) and settings (frequencies of vibrators and tampers, operating speed, etc.) on its behavior (thickness, levelling, and precompaction) are described. The laws governing this behavior are stated, and two models that account for its main features are presented. Finally, possible consequences of this knowledge for the future development of the finisher are considered, particularly the automation of its operation.

Opposite to its functions (self-extended screed, high-compaction power screed, etc.), the adjusting and running methods of the finisher (Figure 1) have evolved little during last 10 years, irrespective of the economic and technical stakes (the amount of materials and the security and comfort of roadways). An important effort has therefore been made to at least correctly set the problem, that is to evaluate more precisely the effect of the finisher's controls on pavement's characteristics. The means used, results obtained, and some possible developments are discussed.

EXPERIMENTAL STUDY

Means of Measurement

The main difficulty in such a study is to collect numerous, accurate, and self-connected measurements about, on one hand, the road characteristics before and after laying (such as levelling, capacity, etc.), and on the other, the settings and the commands of the finisher.

Density Measurements

An extensive range of equipment that uses radioactive isotopes is used in France for density measurements. This equipment may be used to measure either the mean density of various thicknesses or density gradients (double probe). The following equipment is used to measure the mean density of various thicknesses: variable-depth point gamma densitometer (GPV); fixed-depth gamma densitometer (GF); small moving shoe, fixed gamma densitometer (PSM-GDF); and mobile gamma densitometer (GDM).

Topometric Measurements

Usual optical methods are unusable before compaction of the course or to reach such high-rate levelling as one or more measurement per second. Two methods are used to exceed these limits:

- For site use, the Saint-Brieuc Regional Laboratory has developed a moving target (the Laserographe) that can be used to survey one or two longitudinal profiles, with at least one point per meter; the method is precise (accurate to within approximately 2 mm). The information is received and processed by a computer.
- At the LCPC spreading test track at Nantes (*1*) (shown in Figure 2), an absolute reference is provided by 2 parallel 150-m-long concrete rails that flank a 8.5-m-wide track, on which a rigid gantry equipped with ultrasonic rangefinders travels.

These rangefinders (Figure 3), derived from those used in cameras by eliminating the device that translates the output into discrete form and by adding temperature correction microprocessors, provide a measurement (without contact) of the distance to the nearest surface (between 0.3 and 2 m), precise to within 0.3 mm. One rangefinder per meter of width is generally used. A thumbwheel switch triggers the acquisition of each range measurement every 5 or 10 cm. The measurements are displayed on a screen.

For both methods, programs able to plot the surfaces surveyed and process them for analysis (evenness, etc.) or for design assistance in pavement work (new construction, maintenance) are being developed.

Measurements on Finisher

Figure 4 shows typical finisher instrumentation. Proximity sensors are used to measure angles of rotation (and thus distances) and, with the help of a clock, speeds of rotation. The speed of the finisher and the distance traveled are measured on the solid-tired wheels supporting the hopper, which have a constant diameter and, because they are not driven, do not spin. For this purpose, one of the wheels is fitted with a disc having a number of studs, and a proximity sensor detects their passage. The precision is 1 percent or better for both distance and speed. For the frequencies of vibrators and tampers, the proximity sensor detects a metal nipple bonded or welded to the drive shaft. The precision is close to 1.5 Hz. For the distance from the tow points to the ground or the chassis, the rangefinders are used. To measure the thickness laid down, either plunger potentiometers or rangefinders mounted on the ends of the screed are used. The precision is within a few millimeters. The position of the sensors, however, is not always ideal for measuring purposes and exposes them to some damage. Consequently other approaches are being developed. Finally, to measure the temperature, a thermocouple

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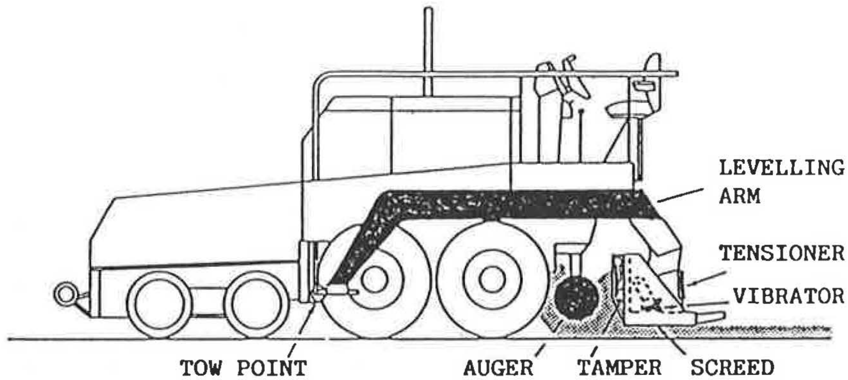

FIGURE 1 A finisher and its spreading implements (tow point, screed) and vibration adjustment units (tamper, vibrator).

FIGURE 2 Spreading of rolled asphalt on the LCPC test track at Nantes and recording of the level of the course by the measuring carriage.

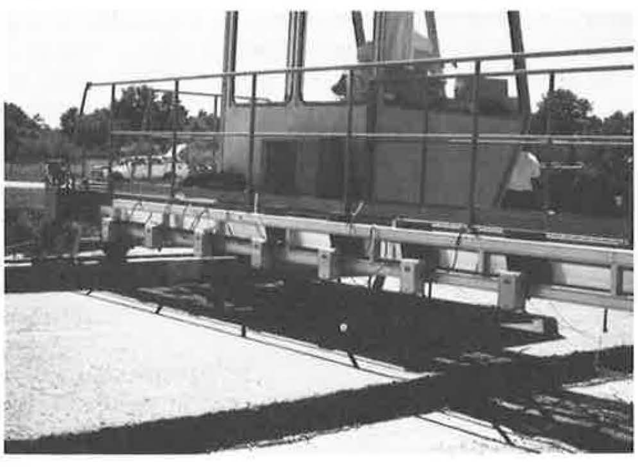

FIGURE 3 Detail of measuring carriage and ultrasonic rangefinders used to survey the course (possibility of 20 measurements per meter of longitudinal profile on each of 10 longitudinal profiles).

protected by a sheath and immersed in the material at the front of the screed is used.

The signals are displayed in real time and stored on a computer. A number of procedures have been tested. The signals may be digitized when they reach the computer or when they leave the sensors. The latter approach limits interference and

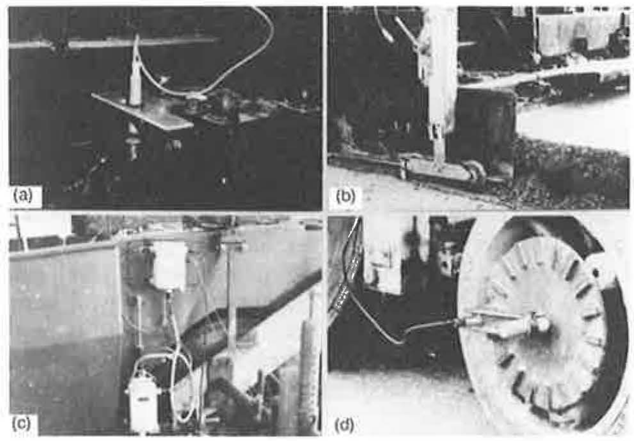

FIGURE 4 Typical finisher instrumentation: *(a)* frequency of vibration of screed—one proximity sensor directed at a metal nipple on the vibrator drive shaft; *(b)* thickness—a potentiometer on a bogie measures the displacement of the trailing edge with respect to the ground; *(c)* level of tow points—an ultrasonic rangefinder measures the displacement of the tow point—with respect to the ground (bottom housing); a potentiometer and its processing and display unit; and *(d)* operating speed—a wheel with studs detected by a proximity sensor.

makes it possible, using a suitable data bus and software, to connect the sensors in a series on a single cable. The computer may be either a special device or a standard microcomputer with a screen; the latter has the advantages of being relatively inexpensive and of being able to process and display the information.

On the spreading test track, the microcomputer—used for measurements both on the pavement and on the equipment—is in the gantry control cab. The system has now proven itself, for both the control and the analysis of the tests.

On site, these systems have been tested by the Laboratoire Régional de l'Est Parisien at Melun. Because all of the wiring is on the outside, there is no protected location for the computer, and the system was considered to be a little too vulnerable. It is believed that, as in the case of mixing plants, only when the finisher includes the necessary openings and protected locations, making the systems sufficiently reliable from the start, will it be possible to test their true value.

Yet in 1990, about 70 km of highway construction sites including slip-form paving as finisher paving were successfully surveyed by applications teams using these methods.

Experimental Results

The finishers may operate "à vis calées" (left alone self-leveling screed), that is, with the position of each tow point fixed with respect to the tractor, or guided in one of several ways. Two examples follow:

- Imposed transverse slope: one tow point is fixed and the other slaved to a transverse pendulum; and
- Double guidance: the trajectory of each tow point is controlled (nearby structure, wire or beam of light, smoothing beam), or that of only one, with the other slaved to an imposed cant.

The behavior of the "guided" finisher can be understood only with reference to that of the finisher "à vis calées," and so the studies whose conclusions are presented below were begun on the latter. These investigations were carried out between 1983 and 1985 at the LCPC, Nantes, and at the Rouen CER, on two finishers, one after the other. The first, of an older type, had a mechanical transmission and tampers, whereas the second, of a more recent type, had a hydrostatic transmission and a Blaw-Knox combined screed. Two materials were used: a reusable artificial mix (6 to 10 cm thick) and a slag-stabilized continuously graded aggregate (18 to 22 cm thick).

Settings Studied

The heights of the tow points can be adjusted separately while in operation, within a range imposed by the travel of the actuating cylinders. The travel of the tampers and the unbalance of the vibrators can be adjusted while stopped; their frequency can be adjusted while in operation. The operating speed can be adjusted while in operation. The screed adjustment angle, which is not represented in any physical form, combined with the height of the tow points, determines the range of variation of the thickness laid down. It is adjusted when stopped.

Definition, Factors, Variations of the Steady State

The steady state is characterized by the thickness and precompactness obtained when the settings and the characteristics of the material laid down are invariant. The precompaction factors are the tampers, the vibrators, the operating speed, and the weight of the screed. The factors governing thickness E, for a given material and given settings of the precompaction factors, are the height HB of the tow points and setting angle ß (Figure 5).

Effect of *HB* and ß It can be seen that the variation in thickness is a fraction R of the variation in tow point height. The value of R depends on the characteristics of the finisher, on the material, and on the precompaction factors. Under the experimental conditions, R ranged from 0.5 to 0.7. The current interpretation of this is as follows: to a given setting of the precompaction factors there corresponds a constant increase of the density of the material between "infeed" and "outfeed." This entails a linear dependency between thickness and angle of incidence, and thus between thickness HB and ß. Further knowledge of the value of R for a variety of machines and materials should in the future facilitate the initial adjustment of the finisher.

Influence of Tampers In the course of the tests, their travel was 4 mm. Their frequency ranged from 0 to 25 Hz. It was found that a variation in tamper frequency caused a variation in the same direction in thickness and, to a lesser extent, in precompaction. A possible explanation is that the tampers act primarily on the compactness at the screed infeed. If this

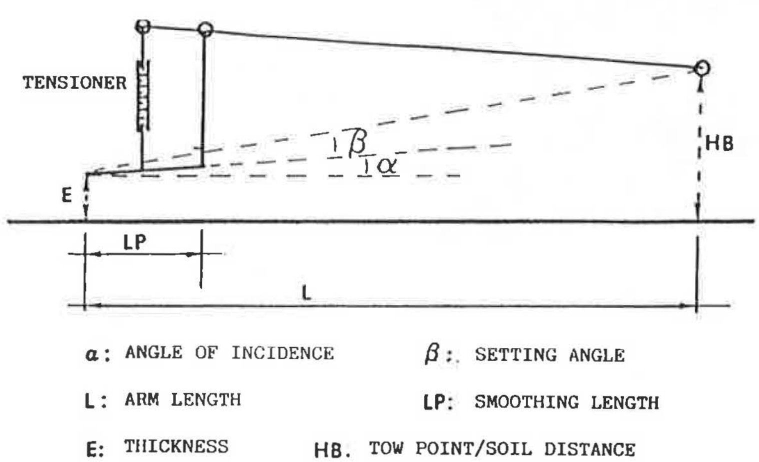

FIGURE 5 Diagram of levelling screed-arm assembly. The setting angle (ß) is fixed in operation and can be adjusted using the rear tensioners when the finisher is stopped. The angle of incidence (α) can be varied in operation by changing the tow point height (*HB*) or the thickness.

compactness increases, the compaction of the material by the pressure of the screed decreases and the thickness laid down increases. With the finisher without vibrators, stopping the tampers caused the thickness to decrease 35 percent (3.5 cm over 10 cm).

Influence of Vibrators The moments of eccentric were varied from their nominal value (0.08 mkg/m of screed) to twice this value, and their frequency was varied from 0 to 75 Hz. Variations in these two parameters caused a variation, in the same direction, of the precompaction and a smaller variation, in the opposite direction, of the thickness. One possible interpretation is that the vibrators act on the difference in compactness of the material between the screed infeed and outfeed without significantly altering its compactness at the screed infeed.

Influence of Operating Rate The speed ranged from 1 to 3 m/min during the spreading of the slag-stabilized continuously graded aggregate and from 3 to 6 m/min with the artificial mix. It was found that a variation in speed causes a variation in the opposite direction of the precompaction and, to a lesser extent, of the thickness. Current attempts at interpretation are based on the energy of compaction transmitted to the material and variations of this energy as a function of speed.

Total Effects To conclude, the ranges of variation of the thickness and precompaction for all of the settings mentioned above combined are as follows:

- Slag-stabilized continuously-graded aggregate: 1.72 < density < 2.12 (modified Proctor density: 2.16).
- Artificial mix: 1.68 < density < 1.95 (gyratory compaction test density 2.18); 9.20 < thickness < 14.2 cm (with constant tow-point height).

It can be seen that these variations are not minor.

Evolution of Equilibrium

When any one of the parameters mentioned above is changed in operation, the new equilibrium thickness is reached gradually. It is highly probable that, when it exists, the change in precompaction is also gradual. Figure 6 shows how the thickness changes when the tamper frequency changes. Figure 7 shows how the thickness changes when the tow-point height is suddenly increased.

Modeling

Dynamic Model

Work on modeling the behavior of the finisher was conducted concurrently with the experimental investigation. The point of departure was an article by Tom Shelley published in 1980

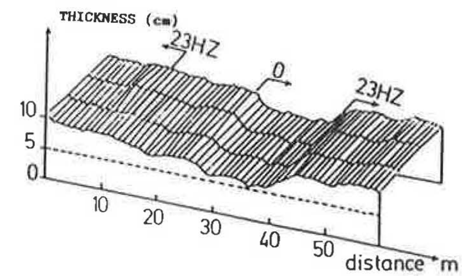

FIGURE 6 Thickness versus tamper frequency for a PF 90 DC finisher with no vibration of the smoothing screed.

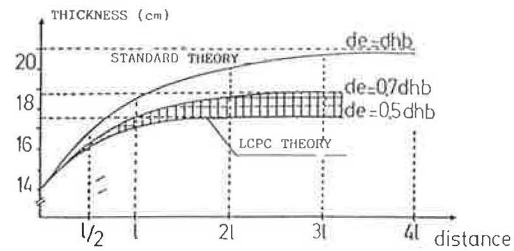

FIGURE 7 Thickness with a 70-mm change in tow-point height in the course of spreading; thickness is stable 2 LB after the command and changes by 50 to 70 percent of the change in tow-point height.

(*2*) that proposed a steady-state equilibrium model based on the following assumptions:

- No horizontal flow of the material under the screed (conservation of mass in vertical sections);
- One-to-one relation between the compactness c of the material and the pressure p to which it is subjected, in the form $c = A/(B - \log p)$;
- The resultant of the pressure of the material on the screed is equal to the weight of the screed.

To adapt this model to the case of any tow point trajectory and any underlying course, its assumptions were completed and extended to produce the current FORTRAN program, which includes two factors that are constant in the course of execution (arm length Lb and screed smoothing length Lp) and six that can vary in the course of execution (values A and B characterizing the law of compaction of the material, the compactness Ce of the material fed to the screed, the effective weight Pt of the screed—including the action of the vibrators—and the heights Zs of the underlying course and Zb of the tow points).

This model is limited because it is rather slow (30 to 50 m/min), and it is difficult to translate the actual parameters (speed, tamper frequency, etc.) into the parameters of the model. Laboratory tests will be used in part to overcome this difficulty. However, the model does accurately account for the following main experimental results:

- $d(E)/d(HB)$ between 0.5 and 0.7 with the current equipment and materials,
- Sensitivity of the profile of the course laid down to irregularities of the underlying course, and

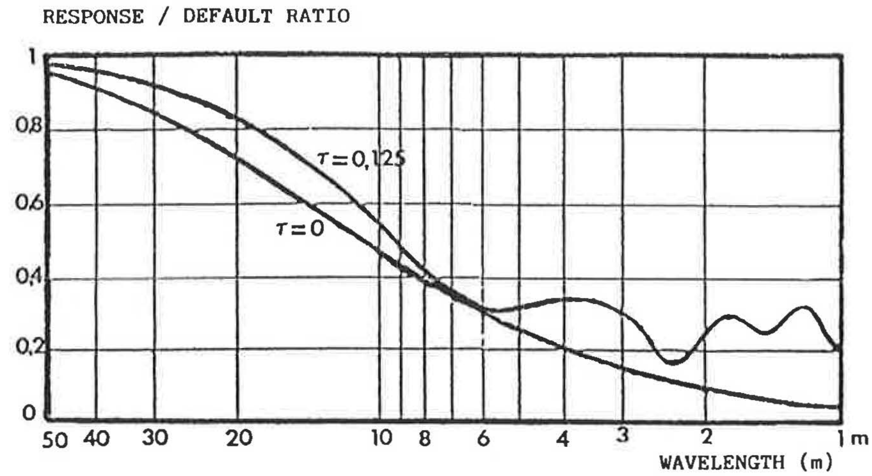

FIGURE 8 Finisher transfer function versus reciprocal of defect lengths according to the standard theory (τ = 0) and the research results (τ = 0.125).

- Gradual change of thickness following sudden changes to settings.

It therefore provides a general, but, of course, still perfectible, framework for interpreting the experimental results.

Geometric Model

The main result yielded by the foregoing model is a roughly constant relative reduction τ of thickness between the screed infeed and outfeed for a given material, finisher, and settings. This result may be used as the starting point for a simplified model in which the physical phenomena appear only indirectly, as factors modifying τ. When τ is constant, it is possible in this way to calculate the transfer function of the finisher operating with fixed settings, which has the following form (Figure 8) for common values of *Lb* and *Lp*.

The standard description of finisher behavior (*3,4*) can be recreated by setting τ equal to 0 and *Lp* equal to 0. The standard theory would therefore seem to be an optimum that overestimates actual finisher performance, and the model points the way to approaching this optimum: feeding a material that is already compacted ($\tau = 0$) to a finisher having a narrow screed ($Lp = 0$).

Such a model can also be used for many other applications, such as estimating the final evenness of a surfacing from that of the underlying course and the characteristics and operating mode of the finisher; comparing, for a given construction case, various ways of using a finisher; and forecasting the probable benefits of new ways of using a finisher before trying them out.

PROSPECTS

Operators' Information

A lot of the measurement techniques that have been developed during this study, although perfectible, are already usable and used on site. Integrating the measurement devices into the finisher during its manufacture would supply excellent reliability and would help operators better master their work.

Automation

The next step on this way would consist in linking together the different settings. On this point, the studies are just starting up. Yet it appears that a control system using as feedback a real-time measurement of the average thickness of the mat and which design would have taken into account the "geometric" model would be of great interest.

Laying Results Forecasting

The mathematical models, after more complete evaluation, would find other and numerous uses, such as estimating the evenness of a pavement, knowing the profile of the underlying course on one hand, the characteristics and the operating mode of the finisher on the other; comparing, for a given site or given type of site, various ways of using a finisher; and estimating the possible benefits of new ways of using a finisher before trying them out.

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Performance of Hot-Mix-Asphalt Railway Trackbeds

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Current construction procedures and long-term performance of railroad trackbeds that incorporate a layer of hot-mix asphalt (HMA) as an underlayment or overlayment are highlighted. Both techniques are applicable for new construction or rehabilitation of existing trackbeds. Specifically addressed herein are the following: (*a*) optimum HMA mixture design guidelines, (*b*) long-term HMA mixture characterization in the trackbed environment, (*c*) typical construction and rehabilitation procedures, (*d*) documented performances of selected installations, (*e*) economic benefits and comparisons with conventional designs, (*f*) structural design standards, and (*g*) discussion of the inferred advantages and relative applicability of HMA trackbeds. HMA trackbeds, which currently number in the hundreds in this country, are performing extremely well under widely varying traffic, roadbed and environmental conditions. The initial economics of using HMA is attractive, and indications are that the long-term savings in maintenance and operating costs can be substantial when compared with conventional construction and rehabilitation techniques. It is anticipated that the use of HMA trackbeds will increase in this country and throughout the world during the coming years as railroads resume a greater role in the transportation system. This will require the highest quality trackbeds for efficient and economical operations of intercity freight and the expected expansion of inter- and intracity passenger systems.

During 1989, U.S. Class I railroads hauled a record revenue ton-miles of intercity freight. Intermodal loadings also set a new record, and the National Railroad Passenger Corporation (Amtrak), the nation's intercity passenger network, recorded new highs (*1*). Current proposals for high-speed passenger systems to link major cities include new dedicated lines estimated to comprise 66,000 km (41,000 mi). Design and construction of new intracity light rail, including expansion of existing systems, is under way in more than 20 major U.S. cities. Expansion of existing and new construction of yards and terminals to accommodate the increased freight and passenger traffic is expected to increase during the coming years. To accommodate this increased tonnage and patronage, the railroads and transit agencies must develop and maintain quality track systems. It is imperative that such systems have minimum interferences and temporary speed restrictions from maintenance activities.

Efforts in the United States to develop applications of hot-mix asphalt (HMA) as a premium quality integral trackbed material began sporadically during the late 1960s (*2*). Renewed interest began during the early 1980s, and since then, HMA has been applied in hundreds of marginal to poor trackbeds. This construction includes new trackbeds, yards, terminals, and loading facilities. HMA has also been installed as a solution to specific trackbed instability problems. These specific projects have included the rehabilitation of high-maintenance turnouts, crossings, bridge approaches, hump tracks, tunnel floors, highway crossings, and loading facilities where conventional procedures had failed.

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Two methods are used to incorporate HMA in trackbeds (Figure 1). The method more widely accepted by the railroad industry is known as HMA underlayment. This method involves placement of an HMA mat directly on new subgrade or reconstructed old roadbed with a layer of ballast placed between the HMA and the ties. This represents little change from normal track construction practices, because the HMA layer merely serves as a subballast in place of a granular subballast. The HMA overlayment method involves placing an HMA mat in a similar manner, except no ballast is used between the HMA mat and ties. Cribbing aggregate is placed between and at the end of the ties to restrain track movement.

The locations of the primary test sites involve a wide range of traffic, roadbed, and climatic conditions across the country. Pertinent data for the projects discussed herein are presented in Table 1.

DESIGN AND CONSTRUCTION PRACTICES

Design Options

Structural design options include both HMA underlayment and overlayment. The underlayment serves as a subballast and does not require close grade control because the layer of ballast can be used as a leveling course for the track. The overlayment requires the ties to be placed directly on the HMA, using no ballast. The HMA for this method should be placed within 0.8 mm (0.03 in.) of the profile grade. Smoothness criteria for ties should be of the same general tolerance as the HMA mat. Small knots or humps will be flattened out by the loads because the HMA mat is soft and plastic. When the wood ties are placed directly on the fresh HMA mat and load is applied, there is usually a slight leveling effect. The ties adhere to the fresh HMA mat, therefore it is not necessary to apply a tack coat to the mat.

Mix Design Criteria

Several types of mixes have been used since the inception of the research in 1968. Using conventional highway mixes, initial test sections were constructed with satisfactory performance. In the early 1980s a low modulus mix (plastic) was

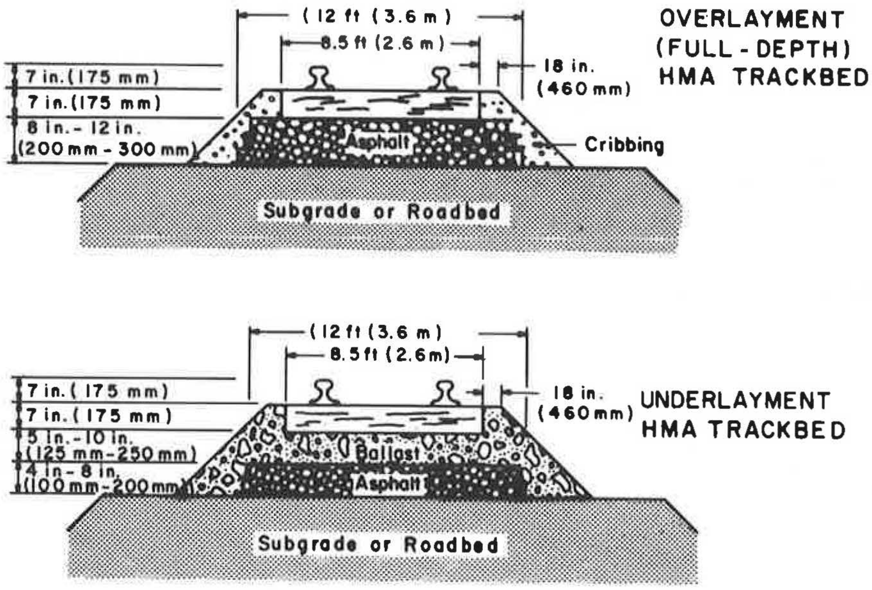

FIGURE 1 Typical overlayment and underlayment sections.

used. The Marshall mix properties considered ideal for both underlayment and overlayment are shown in Table 2.

HMA base course mixes have been effective in meeting the above criteria and minimizing the required asphalt content, because of the lower voids in mineral aggregate. The mixture developed specifically for trackbed applications is a slight variation of ASTM D-3515. In areas where this type mix was not available, normal HMA surface course mixes have been used, with good success, by increasing the asphalt cement to achieve the low voids range. A suggested aggregate gradation master range for both underlayment and overlayment mixes is shown in Table 3.

Normal construction practices should be followed to achieve the desired finished product in the asphalt underlayment or

TABLE 1 INSTRUMENTED HMA TEST TRACKBEDS

Location (Railroad)	Cleveland (RTA)	New Mexico (ATSF)	Ravenna, KY (CSX)	Oklahoma City Flynn Yard (ATSF)	Conway, KY (CSX)
Type of Facility	High Speed Commuter Transit	Slow Speed Branch Line	Slow Speed Yard Main	Slow Speed Yard Lead	High Speed Mainline
Traffic (million gross tons per year)	Passenger (3)	Unit Coal (3)	Unit Coal (8)	Mixed Freight (10)	Unit Coal Intermodal Mixed Freight (40)
Year Constructed	1968	1969	1981	1982	1983
Section Length	Two 1000-ft Underlayments	Three 700-ft Underlayments	Two 500-ft Overlayments	One 532-ft Underlayment	Two 1000-ft Underlayments
HMA Width	10 ft	16 ft	12 ft	12 ft	12 ft
HMA Thickness	5 in. WB 4 in. EB	2 1/2, 5, 7 1/2 in.	8 & 12 in.	8 in.	5 & 8 in.
Ballast Thickness	12 in.	10 in.	--	8 in.	5 in.

Note: 1 in. = 25.4 mm, 1 ft = 0.305 m, 1 ton = 0.91 tonne

TABLE 2 MARSHALL MIX DESIGN CRITERIA

Property	Range
Compaction (blows)	50
Stability, lb. (min.)	750
Flow, 1/100 in.	15 - 25
Percent Air Voids	1 - 3
Percent Voids Filled	80 - 90
In-place Density	92 - 98%*

* Percent of maximum theoretical density based on ASTM D-2041
1 lb = 4.5 N, 1 in. = 25.4 mm

TABLE 3 GRADATION RANGE FOR RAILWAY MIXTURES

Sieve Size	Percent Passing
1- 1/2 in. (37.5 mm)	100
1- in. (25.0 mm)	90-100
3/4 in. (19.0 mm)	---
1/2 in. (12.5 mm)	70-90
3/8 in. (9.5 mm)	---
No. 4	40-65
No. 10	25-45
No. 40	10-26
No. 80	6-18
No. 200	3-8
Percent AC-10, 20 or 30* Asphalt Cement	4-8

* Based upon total weight of mixture

overlayment. Resulting compaction of the finished mat should be the normal minimum of 92 percent maximum theoretical, based on the Rice method (ASTM D-2041). Most of the field densities have ranged from 93 to 99 percent in the monitored research work.

Construction and Rehabilitation Procedures

The majority of the HMA trackbeds placed to date have utilized conventional highway paving construction techniques. For existing trackbeds, the track first must be removed and the underlying material excavated to the desired grade. The HMA mix is hauled by dump truck from a hot-mix plant and is either spread using a standard highway asphalt paver or dumped from the trucks and spread with a dozer blade (only used for short sections). The mix is normally placed in 100-mm (4-in.) lifts, although lifts 150 mm (6 in.) in thickness can be adequately compacted. Compaction is achieved with a standard roller, preferably a steel-wheel vibratory type. It is desirable to obtain a well-compacted mat with minimum air voids.

Immediately after compacting the HMA mat, the track is rebuilt or dragged back on the HMA mat using rubber-tired equipment. After the rails are joined, the ballast or cribbing aggregate is distributed using conventional on-track unloading and spreading equipment. For the underlayment procedure, the track is pulled to provide the specified ballast thickness below the ties. Either No. 24 or No. 4 ballast is generally used. The ballast or cribbing aggregate fills the crib areas between the ties and provides a 0.30- to 0.45-m- (1- to 1.5-ft-) wide shoulder.

Cranes can be used to lift crossing panels, turnouts, and crossovers. Snaking techniques are applicable for longer sections of track. Adequate space must be available to facilitate removal and replacement of the track and provide access for the HMA paving operation.

An HMA underlayment was successfully placed in 1986 under a raised track without removing the rail and ties on the Santa Fe Railway's mainline near Cassoday, Kansas (*3*). The first step involved a single-pass 350-mm (14-in.) undercutting of the trackbed. Three track slewers were used to elevate the track structure 750 mm (30 in.) above the newly cut roadbed. The HMA was delivered by dump trucks to a modified road widener positioned on the adjacent service road. A system of double augers distributed the HMA under the raised track to a strike-off blade and screed. Following was a plate compactor to densify the 150-mm- (6-in.-) thick, 260-m- (850 ft-) long HMA mat. The track was immediately lowered onto the HMA to permit uninterrupted train traffic. Subsequently 200 mm (8 in.) of ballast was added.

Structural Designs

It is recommended that the HMA mat extend 0.5 to 0.6 m (1.5 to 2.0 ft) beyond the ends of the ties. This normally requires a 3.4- to 3.7-m- (11- to 12-ft-) wide mat on single track installations. The mat will necessarily extend proportionally wider on turnouts, crossovers and other special features of the track.

A rational and practical structural design procedure, KENTRACK, was developed for determining the required thickness of HMA for overlayments and the thicknesses of ballast and HMA for underlayments. The design is based on two failure criteria: to limit the horizontal tensile strain (fatigue) at the bottom of HMA and to limit the vertical compressive stress (permanent deformation) on the top of subgrade.

The thickness of the HMA mat depends primarily on three factors: (*a*) resilient modulus or California bearing ratio of the relative subgrade (or old roadbed) support, (*b*) the amount of train traffic, expressed as the number of million gross tons per year, and (*c*) the climatic region.

By the use of the computer program, charts were developed to determine the horizontal tensile strain and the vertical compressive stress in HMA trackbeds under various combinations of subgrade resilient modulus, HMA modulus, HMA thickness, and ballast thickness, if any. Detailed information on the development and applications of KENTRACK is provided elsewhere (*4–7*).

For the underlayment section, the recommended minimum thickness of ballast is 125 mm (5 in.) so that conventional roadbed maintenance equipment can be used when required for routine track adjustments. The required ballast thickness increases as the traffic level increases and as the subgrade support quality decreases. Typical ballast thicknesses are from 125 to 250 mm (5 to 10 in.). HMA mat thicknesses are from 100 to 200 mm (4 to 8 in).

For the overlayment section, the recommended HMA mat thickness ranges from 150 to 450 mm (6 to 18 in.). Unless the subgrade is classified as good or excellent, it is not feasible to use overlayments and maintain reasonable HMA thicknesses. It may be more economical to improve the subgrade quality before placing the HMA mat. HMA mat thicknesses for the sections under study range from 200 to 300 mm (8 to 12 in.).

LONG-TERM HMA MIXTURE CHARACTERIZATION

Five projects that had undergone several years of weathering in the trackbed environment were chosen for characterization studies of the HMA. Pertinent data for these projects are presented in Table 1. Mat temperatures were monitored for a year, and HMA cores were periodically taken for laboratory evaluations.

Temperature Variations

In typical highway applications, HMA undergoes relatively large temperature variations. This affects the properties of the HMA mat, primarily the stiffness. HMA in a trackbed is not exposed to ambient temperature because of the insulating effects of the cribbing and ballast, which attenuate temperature fluctuations in the HMA layer. The range between winter and summer temperatures is reduced; thus the stiffness of the HMA layer remains nearly constant throughout the year. The other factor in aging is low air voids that prevent the flow of air and moisture, which is responsible for most of the aging process. This reduces the potential for cracking in the HMA mat and increases the fatigue life. Typically, the top of an HMA underlayment system is 330 to 480 mm (13 to 19 in.) below the surface, whereas the top of the HMA overlayment lies 180 mm (7 in.) below the surface.

Temperature distributions within the track structure were measured throughout the year with thermistors embedded at various depths at two different sites. Winter measurements were taken after prolonged cold weather and at ambient temperatures near −10°C, (14°F). Summer measurements were taken during August at ambient temperatures approaching 32°C (90°F). Annual temperature measurements for those two projects are presented in Table 4. The underlayment had less temperature fluctuation, as expected, because of the thicker cover. The range in temperature extremes was considerably less than typical for HMA highway applications. Temperature gradients within the HMA layers were minimal, having a maximum value of 2.8°C (5°F) over the section thickness. No freezing temperatures were recorded within or 100 mm (4 in.) below the HMA layer.

HMA Core Analyses

HMA cores were extracted from trackbeds and evaluated in the laboratory to determine the aging effects of cribbing and ballast on the HMA trackbed layer. The laboratory evaluations involved extracting and recovering the asphalt cement for viscosity and penetration tests. Dimensions, density, voids, and dynamic modulus values of the compacted cores were also determined.

Extraction test results and asphalt core analyses for four HMA trackbed projects are presented in Table 5. The New Mexico and Cleveland data are of particular interest because these two trackbeds had been in service for about 15 years at the time of testing. The dynamic modulus values averaged about 1.04×10^6 kN/^{2} (1.5×10^5 psi), which is typical for the more recently placed Conway and Flynn Yard HMA mixes.

Absolute viscosity of the recovered asphalt from the New Mexico cores averaged 1,300 poises at 140°F (60°C), and the standard penetration averaged 72. Some loose chunks of compacted mat lying along the track were also evaluated. They represented a portion of the HMA mat removed during the installation of track scales in 1979, which had been subjected

TABLE 4 ANNUAL TRACKBED TEMPERATURE VARIATIONS

Location And System	Range From Winter to Summer: Average Within HMA Layer	Range From Winter to Summer: 100 mm (4 in.) Below HMA
Conway, KY Underlayment	5°C - 23°C (41°F - 74°F)	7°C - 21°C (44°F - 70°F)
Ravenna, KY Overlayment	2°C - 27°C (35°F - 80°F)	3°C - 25°C (37°F - 77°F)

TABLE 5 MIX EXTRACTION TESTS AND CORE ANALYSES FROM HMA TRACKBEDS

	New Mexico (1969)*		Cleveland (1968)	Conway, Kentucky (1983)			Flynn Yard (1982)		
	Trackbed Cores After 14 Years	Chunks**	Trackbed Cores After 16 Years	Trackbed Cores After 1 Day	Trackbed Cores After 2 Years	Trackbed Cores After 7 Years	Trackbed Cores After 2 Months	Trackbed Cores After 3 Years	Trackbed Cores After 7 Years
Extraction Results									
Maximum Aggregate Size, in.	1	1	1	1	1	1	1	1	1
Percent Passing No. 200 Sieve	9.3-10.1	10.3	3.7-6.2	3.8-5.3	4.6-5.9	5.1-8.2	7.0	6.0-6.5	--
Asphalt, % by Weight of Total Mix	6.9-7.3	6.5	4.1-4.2	4.8-4.9	4.5-4.8	4.9-5.5	5.7	5.5-5.6	--
Recovered Asphalt									
Viscosity, 140 F (60 C), P	1060-1610	7525	6800-10,540	4400-4410	6250-14,060	5590-24,890	3870	3490	2495
Viscosity, 275 F (135 C), cST	270-310	550	610-710	530-540	610-840	--	580	700-730	471
Penetration, 77 F (25 C), 100g, 5s	62-82	25	35-42	49-51	28-42	25-45	50	57-58	59
Core Analyses									
Height, in.	2 5/8-7 5/8	--	4-7	4 1/4-8 3/8	4 3/4-8 1/2	4 1/2-8 1/4	6-9 1/4	9 1/2-10 1/2	8-9
Air Voids, %	3.1-4.7	--	9.6-15.2	7.0-10.1	6.9-13.2	3.5-10.9	0.9-2.7	0.9-2.3	--
Dynamic Modulus, psi x 10^5 @ 1 hz, 77 F (25 C)	0.95-1.27	--	1.09-1.79	0.84-1.71	--	--	1.45-1.63	1.51	1.25-1.75
Density, lb/ft³	136-139	--	131-140	141-146	139-146	144-151	149-151	149-150	--

*Date Constructed
**Loose chunks of discarded HMA picked up along the track, representing part of HMA removed and exposed during installation of scales four years prior.
Note: 1 in. = 25.4 mm, 1 psi = 6.9 kN/m²

to atmospheric weathering for the previous 4 years. The absolute viscosity for the weathered sample was 7,500 poises, and the standard penetration was 25. An 85–100 penetration asphalt was reportedly used in the original construction. Obviously, the HMA hardened little in the trackbed environment; however, once exposed to the atmosphere, it hardened rapidly.

Higher viscosity and lower penetration values were obtained from the Cleveland cores, indicating a harder asphalt cement. This may be partly because of the higher air voids and lower asphalt content in the compacted mat, which would promote faster hardening than the New Mexico mat, which contains lower air voids and higher asphalt content. Also, a harder grade asphalt cement may have been used initially in the Cleveland mix.

Recovered asphalt and dynamic modulus tests on the more recently constructed Flynn Yard installation indicate minimal, if any, hardening or deterioration of the HMA mix after 7 years. The Conway installation was constructed with higher voids, resulting in aging similar to that of the Cleveland section. The higher viscosity on aging has not affected the dynamic modulus range, when compared with the lower viscosities at Flynn Yard. The performance has been equally good.

The ballast at the sites was clean and free of contamination, and the HMA cores were close to design thicknesses. The cores appeared to be in excellent condition, and the dynamic modulus test results confirmed the observations.

On the basis of long-term data, it appears that the insulated trackbed environment reduces weathering and hardening of an HMA mix relative to applications where the mixture is exposed. The reduced levels of oxidation and temperature fluctuations and consistent dynamic modulus values should ensure a long fatigue life for the HMA mat.

TRACKBED PERFORMANCE TESTING

Several of the HMA trackbeds have been subjected to periodic instrumental tests and measurements. Adjacent control sections employing conventional ballasted track have been evaluated for comparison purposes. The performances of all the HMA sections have been excellent. Following is a summary of the various tests and measurements.

Trackbed Moisture

Extensive studies have been made at the Flynn and Conway sites to evaluate the long-term waterproofing characteristics of HMA trackbeds. Samples of the roadbed material underlying the HMA mat were obtained after removal of HMA cores, and moisture tests were conducted (see Table 6).

Moisture contents of the compacted fine-grained, (CL, A-6) cohesive red clay soil underlying the HMA mat at the Flynn Yard after 7 years was 13 to 17 percent, slightly lower than the 17 percent prevailing moisture content of the soil during construction. The optimum moisture content for the soil is 18 percent.

TABLE 6 SUBGRADE/ROADBED MOISTURE CONTENTS

In-Place Moisture Tests (%) After Coring				
Year/Location	1982	1985	1987	1989
Flynn Yard Subgrade	17.4 avg. (as constructed)	16.8-18.5	15.6-17.7	13.1-16.9
Year	1983	1985	1990	
Conway Roadbed	>20% (as constructed)	10.7-23.4 Avg. = 18.4%	9.8-20.9 Avg. = 13.8%	

Similar results were obtained from subbase samples taken from the Conway test installations after 2 and 7 years of service. The underlying material is markedly different from that obtained at the Flynn Yard. It is a more granular mixture of fine soil, cinders, coal, and ballast that represents the existing 70-year-old roadbed. At the time the HMA was placed, this material was quite wet, with moisture contents in excess of 20 percent. The high moisture content also was caused by the high absorption qualities of the cinders and coal. After 7 years, the average moisture content was 14 percent, slightly lower than the as-constructed average moisture content.

Piezometers were installed at two locations just under the HMA mat at the Conway test site. Readings were taken periodically under test train and revenue train operations for a 3-year period following construction. No pore water pressures were recorded, indicating pore water pressures were not developing even under 100-car unit coal trains.

These results indicate that an HMA layer overlying either a fine-grained compacted soil or a granular mixture of old roadbed materials will maintain a moisture level in the underlying material. This waterproofing and membrane effect will provide consistent load-carrying capability from the underlying material while preventing intrusion of subgrade into the ballast and subsequent fouling and pumping. These factors are considered to be primary benefits of HMA trackbeds.

Long-Term Track Settlement

Top-of-rail elevations were established at five test installations soon after construction. Elevation changes along the test sections at 15-m (50-ft) intervals were periodically measured using conventional leveling techniques. No significant changes in elevation occurred, even after several years of service.

In new track construction on fills and embankments, some deep-seated settlements are likely to occur regardless of whether HMA is used. However, if infiltration of surface water is a contributing cause of the settlement, then obviously an HMA layer will reduce this settlement because of its waterproofing characteristics.

Static Track Modulus

The relative stiffness or rigidity of the track structure under static loading conditions was evaluated using the track modulus procedure. Modulus values were calculated from deflections obtained under known loads using beam on elastic foundation principles. A loaded 91-tonne (100-ton) hopper car was used for the heavy load [approximately 147-kN (33,000-lb) wheel load] and an empty 91-tonne (100-ton) car was used for the light load [approximately 35-kN (8,000-lb) wheel load]. Deflections at the base of rail and tie plate were recorded using linear scales and a transit.

Track modulus values, accumulated to date for the Conway and Flynn Yard underlayment installations, are presented in Table 7. As noted, the values tend to stabilize, after a period of time, at about 17 N/mm/mm (2,500 lb/in./in.), which corresponds to a deflection of 4.6 mm (0.18 in.) under a loaded 91-tonne (100-ton) car on heavy rail and wood ties. These values are within the desirable range for wood tie and heavy rail track to provide the optimum track stiffness and flexibility. Little variation in modulus values for different HMA thicknesses was noted. It is anticipated that HMA trackbeds will maintain an optimum stiffness level for a longer period of time and be less affected by such factors as variations in rainfall and water-table level than the typical ballast track.

Track Geometry

Establishing and maintaining track geometry is important for safe and efficient train operations. Track geometry vehicles, which continuously record the major track geometric parameters (alignment, gage, surface, and elevation), are routinely used by the railroad industry for periodic monitoring of track conditions. Results identify defects requiring immediate corrective action and provide data for long-term maintenance planning.

Several mainline HMA trackbeds have had track geometry tests conducted at 6-month intervals. No detectable changes in geometry have occurred. This is a significant finding because the sites were specifically chosen on the basis of historically documented high maintenance costs and associated trackbed irregularities caused by poor-quality trackbed support and drainage problems.

ECONOMIC EVALUATIONS

The widespread use of any new trackbed structure ultimately depends on a favorable comparative analysis of its long-term

TABLE 7 STATIC TRACK MODULUS VALUES

	Track Modulus lb/in./in.
Flynn Yard built Aug. '82	
Oct. '82	3500 - 4500
Aug. '83	2500 - 3300
June '84	2500 - 2900
June '85	2200 - 3200
Aug. '87	2600 - 3070
June '89	2450 - 2800
Conway built June '83	
Nov. '83	2170 - 2840
May '84	1700 - 2310
Nov. '85	2260 - 2560

1 N/mm/mm = 150 lb/in./in.

cost effectiveness relative to conventional structures. Even though the new or modified structure may cost more initially, if cost savings accrue from reduced maintenance and increased operating efficiency, the new technology is justified and is a good investment.

The prices typically quoted by HMA paving contractors represent the cost and placement of all materials under the actual conditions. For example, consider an HMA mix that will compact to a density of 2240 kg/m^3 (140 lb/ft^3) and is placed 100 mm (4 in.) thick and 3.7 m (12 ft) wide. This would require 850 kg per track meter (0.28 tons per track ft). Assuming the in-place cost is \$33/tonne (\$30/ton), the cost would be \$27.50 per track meter (\$8.40 per track foot), or \$7.50 per square meter (\$0.70 per square foot) for the 100-mm (4-in.) lift.

The cost of obtaining and placing HMA in a trackbed varies depending on the following factors: the cost of the HMA in the local area, the length (time) of haul to the site, the size (tonnage) of the project, the availability and cooperation of local contractors, and the ease of delivery access and construction maneuverability. The \$33/tonne cost (\$30/ton) used in the subsequent example would represent average conditions for a fairly large tonnage project.

The cost comparisons can be viewed from three categories: initial construction, long-term maintenance, and long-term operation.

Initial Construction Costs

New track construction represents an ideal condition for HMA trackbed installation because a prepared subgrade is available for placing the asphalt mat with conventional paving equipment before placing the ballast, ties, and rail. The cost of the HMA can be partially or totally offset by the elimination of geotextile and the replacement of subballast with a thinner HMA mat. In situations in which the traffic is not heavy and a new roadbed requires costly upgrading, stabilization, or extensive subsurface drainage improvements, it is possible that an HMA trackbed system may even represent a lower initial cost, because it can be satisfactorily placed on low-quality support without extensive roadbed preparation. However, for heavy-haul trackbeds, the subgrade may have to be marginally improved to a certain stiffness, so that the HMA will not fail by fatigue cracking.

Table 8 presents an idealized relative cost comparison of new track construction using conventional ballast-geotextile compared with an HMA underlayment installation. The two installations are considered to be equivalent initially, with regard to quality and load-carrying capability. It is assumed that all grade and drain activities are complete and the subgrade has been finished to final grade.

The new trackbed construction costs are essentially the same for the conventional and HMA sections. The inferred assumption is that a combined 300-mm- (12-in.-) thick HMA-ballast section is equivalent, or superior, to a combined 355-mm (14-in.) subballast-ballast-18-oz (0.60-kg) geotextile section.

Long-Term Maintenance Costs

Current techniques for rehabilitation of existing trackbeds with HMA require removing the existing track, excavating the fouled ballast-subballast-soil mixture, paving with HMA, and replacing the track. The paving process involves a small percentage of the total effort. However, if rehabilitation of conventional trackbeds can be done without removing the track, the use of HMA is more difficult to justify. Further modifications of paving equipment for efficiently placing HMA under a raised track without removing the track in conjunction with an undercutting or sledding operation would greatly decrease the time and expense.

Table 9 provides an actual cost comparison developed during 1988 on a CSX Transportation mainline for the rehabilitation and renewal of a No. 10 turnout in conjunction with a bridge approach and 90 m (300 ft) of track using conventional ballast-geotextile and ballast-HMA installations. The turnout and underlying material were badly deteriorated and had to be replaced with new materials.

The two costs compare favorably. If the turnout had not required renewal and could have been rehabilitated in place,

TABLE 8 IDEALIZED NEW TRACK CONSTRUCTION COSTS PER TRACK MILE

Materials/Labor/Equipment	Conventional	HMA Underlayment
New 136-lb Carbon Rail and Other Track Materials	\$231,200	\$231,200
New Wood Ties, 3,017 @ \$31.25	94,300	94,300
8-in. Ballast (3,500 tons) @ \$7.00/ton	24,500	24,500
Surface & Align, 2 lifts	10,000	10,000
8 Field Welds @ \$55	440	440
6-in. Subballast,(4,750 tons, 30 ft. wide) @ \$8.00	38,000	----
18-oz Geotextile @ \$2.25/yd^2	15,840	----
4-in. HMA (1,480 tons, 12 ft wide) @ \$30/ton	----	44,400
Engineering/Supervision, 5% of \$400,000	20,000	20,000
Total Cost	\$434,280	\$424,840

1 lb/yd = 0.5 kg/m, 1 in. = 25.4 mm, 1 oz/yd^2 = 34 g/m^2, 1 ft = 0.305 m, and 1 ton = 910 kg

TABLE 9 NO. 10 TURNOUT AND 90 m (300 ft) OF TRACK REHABILITATION AND RENEWAL COSTS, 1988

Items		Conventional (Estimated)		HMA Underlayment (Actual)
New Turnout (Metal & Ties)		\$16,775		\$16,775
Remove Old Turnout, Track & Excavation		11,310		11,310
Replace New Turnout & Track		5,220		5,220
Welds		570		570
Surface & Align		1,000		1,000
18-oz Geotextile		900		---
Ballast & Unloading	(12in.)	10,000	(8-in.)	8,700
5-in. HMA 120 tons @ \$28/ton		---		3,360
Total		\$45,775		\$46,935

Note: 1 in. = 25.4 mm, 1 oz/sq yd = 34 g/m^2, 1 ton = 910 kg

the percentage increase in the combined costs for removing the turnout and using HMA would have increased. However if the service life of the turnout and ballast are substantially increased by using an HMA section, the extra costs of using HMA should be recovered within a short time.

In addition to the cost comparisons for the turnout presented here, similar cost comparisons have been made for railroad crossings, crossovers, ladder tracks, bridge and tunnel approaches, highway crossings, and short sections of regular track. All of these are typically high-maintenance areas, and the material costs are generally small compared with the removal and replacement costs.

According to recent data published by the Association of American Railroads, for Class I railroads, the annual maintenance-of-way expenditures for heavy tonnage tracks exceed \$6,200/km (\$10,000/mi). This includes normal ballasting, surfacing, renewing ties and rails, and other track maintenance.

As mentioned previously, more than 100 HMA trackbed installations have been built in the United States since 1981. They are being closely monitored by the various railways involved. In addition, 10 or so installations built during the 1960s and 1970s have been evaluated. To date no significant maintenance activity has been required on any of the installations and the relative serviceability of the installations remains excellent.

The advantages of a quality roadbed structure with regard to out-of-face and spot maintenance costs are grouped as follows:

- Decreased ballast applications and surfacing cycles,
- Decreased ballast cleaning and replacement,
- Decreased tie and plate wear,
- Decreased rail and other track materials wear and fatigue, and
- Decreased special trackwork replacements.

The use of HMA trackbeds has not been sufficiently widespread to conclusively produce quantitative data that would support the aforementioned items. However, the favorable test data and results and performance evaluations obtained to date indicate that HMA trackbeds should reduce track maintenance costs.

Long-Term Operating Costs

Maintaining a quality roadbed structure will reduce operating costs by improving the operating efficiency of train movements. The advantages include:

- Increased speed and safety of operations due to good track geometry,
- Decreased train resistance and fuel consumption,
- Decreased rolling stock wear and repair,
- Increased tonnage ratings for similar motive power,
- Decreased operational interferences from maintenance activities, and
- Decreased number of slow orders and other restrictions.

The HMA trackbeds, which have been subjected to periodic track geometry tests, have not exhibited any degradation of track geometric parameters. Obviously, no slow orders or operational interferences from maintenance activities have occurred because no maintenance has been required.

Track stiffness tests and observed vertical displacements of tracks under moving loads indicate that HMA trackbeds deflect slightly less under load applications and rebound less between track loadings than conventional ballasted trackbeds. The increased stiffness and viscoelastic properties of the HMA account for these facts. The results should be decreased train resistance and fuel consumption, increased tonnage ratings for equivalent motive power, and decreased wear and repair costs for rolling stock.

SUMMARY AND CONCLUSIONS

Application Considerations

New line construction and passing track extensions represent ideal application conditions because the exposed subgrade is available for placing the HMA mat with conventional paving equipment before placing the ties and rails. Conventional highway construction procedures are not applicable for paving long sections of in-service single track because sufficient track time normally is not available for removing, excavating, paving, and rebuilding the track. Further modifications and optimization of equipment for placing HMA under a raised track without removing the track, in conjunction with an undercutting or sledding operation, will greatly decrease required track time.

Removing the track and paving short sections of in-service track, turnouts, crossovers, bridge and tunnel approaches, and crossings exhibiting poor soil, bad drainage, or subgrade pumping conditions can be accomplished with minor disruption to traffic. The use of an HMA base under highway crossings can provide an economical means of obtaining adequate support for the combined highway and railroad loadings, thus reducing costly repairs to crossing surfaces. HMA is proving most advantageous in these areas. Where high quality subgrade and adequate drainage conditions exist, the economic benefits of using HMA for reducing annual maintenance cost and improving levels of service are not likely to be as pronounced.

Rapid transit and high-speed passenger lines require substantial track structures to maintain accurate track geometry. Use of HMA in these track structures is appropriate, as evidenced by the Cleveland project. Where light rail lines are placed in-street (i.e., "paved track"), asphalt overlayment design, wherein ties sit directly on a smooth, stable asphalt layer, will reduce or eliminate the need for costly ballast adjustments, which require tearing up the street pavement and taking the street out of service.

The use of HMA is equally adaptable to the construction of intermodal yards. Heavy trucks and unloading equipment require substantial structural sections. A particular advantage is the waterproofing characteristics of the HMA and the positive drainage systems that can be incorporated in the design of the unloading area.

Findings

1. The primary finding, based on the comparative performance of conventional sections and HMA sections, is that the HMA is superior to all other sections. HMA core samples indicate that the ballast keys into the low modulus mix, thus providing a stable system for tangent and curve sections.
2. The HMA sections have not required reballasting, alignment, or any other track adjustment or periodic maintenance during the 7-year study period. Most of the control sections have required or needed periodic maintenance.
3. The HMA sections have extended the life of the ballast; however, because no reballasting has been required, ultimate ballast life on the HMA system at this time cannot be assigned. It is believed that ballast life will be increased using HMA systems. Severe pumping of the subgrade and ballast wear on conventional systems were the primary causes of the fouled ballast.
4. Subgrade (roadbed) moisture contents under the HMA mat have stabilized during the 7-year study period, at or near the optima, thus providing a uniform subgrade support for the service life of the trackbed.
5. Reduced wear of the track components was noted at heavy tonnage crossings indicating the HMA system can extend the service life.
6. Reduced deflections and constant optimum track stiffness of an HMA trackbed can provide a uniform and high-speed rail system to meet increased service requirements.
7. Properties from recovered asphalt cement from the underlayment systems indicated minimal aging of the asphalt cement when covered with ballast if the recommended low-air-void content is achieved. This should provide the trackbed many years of service without replacement of the mat.
8. Cost analyses indicate that HMA systems can be installed economically at no more than 2 to 7 percent increased initial cost and in some cases less than conventional cost. In most all cases, the HMA system has more than "paid-out" in the study period.
9. Several more years will be needed to properly assess the total life-cycle performance of the HMA system because of its potential long service life.

10. The underlayment procedure appears to have general applicability for heavy-haul freight lines, whereas the overlayment procedure is limited to specific applications. Both procedures appear applicable for passenger and transit lines.

Conclusions

The primary benefits of the HMA layer are to improve load distribution to the subgrade, waterproof and confine the subgrade, and confine the ballast, thus providing consistent load-carrying capability for a trackbed even on subgrades of marginal quality. The waterproofing effects are particularly important because the impermeable HMA mat essentially eliminates subgrade moisture fluctuations, which effectively improves and maintains the underlying support. Additionally, the resilient HMA mat provides a positive separation of ballast from the subgrade and thereby eliminates subgrade pumping without substantially increasing the stiffness of the trackbed. The resultant stable trackbed has the potential to provide increased operating efficiency and decreased maintenance costs, which should result in long-term economic benefits for the railroad and rail transit industries.

All of the HMA test tracks and specific problem-solving installations are performing extremely well. The increased cost of using HMA is most often minimal, and indications are that at many sites the long-term savings may be substantial when compared with conventional construction, maintenance, and rehabilitation techniques. Additional improvement and optimization of the field construction procedures represent activities of continuing interest.

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Study of Bituminous Intersection Pavements in Texas

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Intersection-approach pavements often experience extreme forms of distress long before the tangent segments of the same pavement and long before the design life of the pavement is attained. Field and laboratory investigations of asphalt concrete intersection-approach pavements were conducted to determine the primary causes of premature failure and in order to suggest changes in materials specifications, pavement design, and construction procedures that can be used to prolong the service life of intersection pavement. The primary mode of failure of the intersections studied was rutting; in some cases shoving and flushing also occurred. The leading materials-related cause of pavement failure was asphalt content in excess of the design value. Most of the mixtures studies contained relatively high percentages of natural (uncrushed) sand and low voids in the mineral aggregate. Modifications in materials specifications, laboratory test techniques, design procedures, and construction methods are suggested to provide a margin of safety to minimize early failures. The potential for significant economic benefits appears promising if intersection approaches are designed and constructed to accommodate the special stresses to which they are subjected.

Standard pavement structural design methods and asphalt mixture-design procedures were developed for pavements with moving traffic without regard for high, repetitive shear stresses, such as those generated by decelerating and accelerating heavy vehicles at certain pavement locations. Traffic loading, often expressed as passages of an 18-kip equivalent single axle load (ESAL), as determined from the AASHO Road Test, are used in the calculation of damage factors to estimate design life of a pavement. By definition, the ESALs are applied by freely rolling tires, which principally apply a vertical load to the pavement. The only horizontal load in the pavement is the force component generated by the vertical load.

Asphalt concrete (AC) pavements are typically designed and built as if the complete paving project was a tangent section. For this reason, nontangent segments of a pavement, such as intersections, curves, approaches to railroad crossings, bus terminals, and steep grades, often experience extreme forms of distress long before the tangent segments of the pavement and long before the design life of the pavement is attained. As a result, maintenance or rehabilitation or both of the specially stressed segments are required early in the pavement's service life, which is costly in materials and labor and to the user.

The initial phases of the problem as described above were addressed in a recent study (*1*). The analysis was limited to intersections surfaced with AC. The overall purpose of the study was to identify techniques that can be employed in a cost-effective manner to design and build specially stressed portions of pavements that will exhibit performance equivalent to the tangent sections.

The findings of this study (*1*) indicate that existing technology can be used to design and construct pavements of adequate strength and stability to withstand the special stresses associated with intersection approaches. The full report recommends changes in existing Texas State Department of Highways and Public Transportation (SDHPT) materials specifications, laboratory test procedures, and asphalt mixture-design methods in order to decrease the probability of premature failure of intersection pavements. It suggests that alternatives other than standard dense-graded asphalt mixtures should be considered for construction of intersection-approach pavements because these standard mixtures are neither designed to withstand the special stresses applied at intersections nor have they proved to be successful in intersection applications. Initial costs of implementing improved intersection designs will be significantly more than those encountered in normal practice. However, use of the improved designs or paving materials or both may result in significant cost savings during the service life because spot-maintenance and associated user costs will be reduced. This will be particularly true for high-volume roadways.

The AASHTO guide (*2*) states, "The designer will need to concentrate on some aspects of design which are not always covered in detail in the Guide." There is a need to analyze the horizontal shear forces unique to certain portions of pavement systems and develop design procedures, specifications, and materials acceptance criteria that can be used to prolong pavement service life and reduce maintenance and rehabilitation activities in these specially stressed portions of pavement.

A rational approach for intersection mixture design to increase the probability of success is presented in this paper. Results of this work may be implemented to provide adequate structures in other specially stressed segments of pavements such as bus terminals, steep vertical and horizontal curves, and even airport runways and taxiways.

FIELD INVESTIGATION

A questionnaire was distributed among all the Texas highway districts in order to locate unsuccessful AC intersections. Unsuccessful intersections were defined as those that exhibited significant problems related to premature pavement distress, such as rutting or corrugations or both. Visual inspections

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were performed on about 20 unsuccessful intersections, and 8 were selected for sampling and further study. Some consisted of a series of thin overlays that would have been difficult to analyze from a materials standpoint.

Successful intersections were defined as those that were exposed to reasonably heavy traffic and exhibited less than 0.25 in. of rutting and insignificant corrugations or flushing or both after 4 or more years of service. Visual inspections were performed on approximately 30 successful intersections; 6 were considered for sampling and further study. Most of the intersections that were reported to be successful actually exhibited significant distress or they experienced low traffic levels and were eliminated from the study. A sufficient number of "good" intersection approaches that had been last overlaid more than 4 years previously were not found. Therefore, some of the good intersections selected for study had been overlaid less than 4 years before this evaluation but were subject to heavy traffic.

It was found that many districts have implemented an intersection maintenance program in which basically all intersections in the district exposed to significant amounts of traffic received regular maintenance every other year at the minimum. Although the program is performing well in maintaining intersection quality, it did cause some difficulty in locating candidate intersections for this study.

As previously indicated, some intersections were sampled and tested, whereas others were given a more cursory study. When possible, mixture-design data, materials properties, typical sections, and a sampling of daily construction reports were obtained. Rutting was found to be the primary mode of distress in all unsuccessful intersection approaches. A summary of the intersections selected for study is given in Table 1.

Sampling and Testing Program

Rut depths were measured on the approach side of the intersections from the cross street and back until the measurements became less than 0.125-in. Twenty-five cores 4 in. in diameter were obtained from the rutted areas of selected intersections. At the approach side of the intersections, five cores across the pavement, in and between the wheelpaths, were drilled in order to ascertain the profile of the transverse cross section of the pavement. Cores were drilled in accordance with this scheme at each of 5 different locations to obtain a total of 25 cores. The cores were conveyed to the laboratory, where the surface layer portions were carefully separated by sawing and were later tested in an attempt to identify the possible causes of pavement distress.

In some instances, the cores were found to consist of a series of up to 8 thin (less than 1 in.) layers of AC pavement. Mixture testing of these cores was not performed. In these cases, only limited testing and visual inspection was performed. Laboratory test results are described by district in the following sections for each of the intersections analyzed.

TABLE 1 SUMMARY OF SELECTED INTERSECTIONS

Location	Identification	Traffic, ADT,	Age of Pavement of Last Overlay	Rut Depth, in.	Other Distress
District 8 Abilene	SH 36 @ Judge Ely*	4,000	6½ yr	<0.25	None
District 10 Tyler	Loop 323 @ FM 756	38,000	5 mo	0.75-0.9	Flushing
	Loop 323 @ Mackim		5 mo	0.5-0.9	Flushing
	Loop 323 @ SH110		5 mo	<0.25	None
District 15 San Antonio	Toepperwein @ IH35*	12,000	2 yr	0	None
	Judson @ IH35*	12,000	5 yr	<0.10	Slight Flushing
	Coliseum @ IH35*	10,000	5 yr	0.05	None
District 18 Dallas	FM2170 @ SH5	18,800	4 yr	0.25-1.0	Shoving
	SH66 @ Rowlett*	14,000	3 yr	<0.2	None
District 19 Atlanta	US259 @ SH11	8,000	8 yr	0.13-1.0	None
	US67 @ FM989	6,700	9 yr	0.3-0.9	Shoving
	US59 @ FM989*	19,000	8 yr	0	None
District 20 Beaumont	US96 @ FM1013	10,000	6 yr	0.25-2.5	Shoving
	US190 @ US96	10,100	2 yr	0.13-1.0	Shoving

* Indicates good intersections

Test Results

District 10

Cores were collected from 3 intersections (2 rutted and 1 nonrutted) along Loop 323. Rut depths up to 0.875 in. were measured only 5 months after the mixture was placed. The air-void content (2 to 5 percent) was relatively low for a pavement of this age. Voids in the mineral aggregate (VMA) appeared acceptable because they were within the criteria specified by the Asphalt Institute (*3*), which recommends a minimum of 16 percent VMA for a mixture containing ⅜-in. maximum particle size. However, low air-void content with VMA that is within specified limits is an indicator of excess asphalt. This excess asphalt decreases the internal friction of the mixture, making it unstable under slow-moving or stationary traffic loads, particularly during hot weather on a newly placed pavement.

All the Loop 323 cores were extremely tender at higher test temperatures and, as a result, collapsed when the resilient modulus test at 104°F was attempted. Stiffness of the mixtures from the rutted sites was consistently lower than that from the nonrutted sites as evidenced by resilient modulus at 77°F and lower. In addition, Hveem stability was much lower (23 versus 55) for the cores obtained from the rutted sites. Marshall stability and flow of the rutted and nonrutted sections were not much different.

The aggregate gradation showed a notably high percentage of sand-sized particles, as indicated by a hump in the gradation curve at the No. 40 sieve (more than 30 percent passing). The aggregate system was composed of 100 percent crushed sandstone. The coarse aggregate was angular and rough in texture. However, on examination under a microscope, the fine aggregate was found to consist of a high percentage of individual sand particles that appeared to be mostly subangular, glassy, and nonporous. The sandstone was not well cemented and, on crushing and handling, a significant portion reverted to the original individual sand particles.

Asphalts were extracted from selected cores from the 3 locations, and penetration and viscosity were measured at 77°F and 140°F, respectively. The results indicated satisfactory materials. Measurements of asphalt content showed that the mixtures from the rutted intersections contained about 0.5 percent more asphalt than the optimum, whereas the nonrutted mix contained the optimum asphalt content (6 percent).

The major contributor to failure of this intersection mixture was the excess asphalt content, which created the low void content. The glassy, nonabsorptive character of the aggregate with excess sand sizes and the relatively low filler content (filler/asphalt ratio less than 0.5) made the mixture sensitive to asphalt content, and therefore increased the propensity for permanent deformation problems.

District 20

Rut depths at the intersection approach of US-96 at FM 1013 in Kirbyville measured 0.75 to 2.5 in. Nearest the intersection, where the vehicles halted, a ridge had developed, particularly alongside the outer edge of the outside wheelpath. Rut depths at the approach of US-190 at US-96 in Jasper measured 0.13 to 1 in. The pavements more than 250 ft away from the intersections appeared to be in good condition and had rut depths less than 0.125 in.

After examination of the cores, it was concluded, by matching layer profiles with the rut depths measured, that the pavement consisted of a succession of overlays, each of which had experienced various degrees of rutting. The layers within the cores were approximately 1-in. thick, and thus too thin to accommodate most of the standard mixture tests.

Only the uppermost overlay was tested. Data showed that air voids in the wheelpaths (2 percent) were less than half those outside the wheelpaths at both intersections. The voids in the mineral aggregate were, however, within the range specified by the Asphalt Institute.

The aggregates blended to produce these mixtures consisted of more than 30 percent natural, uncrushed sand. On the basis of results of sieve analyses, both mixtures were generally composed of aggregate significantly smaller than that specified by the design. It is recognized that the coring operation reduced the measured aggregate size, but not to the extent shown here, especially in the smaller size ranges. In addition, the gradation curve exhibited a notable hump at the No. 40 sieve, indicating an excess of sand, and thus a mixture relatively weak in shear strength and sensitive to a slight excess of asphalt. On examination under a microscope, the fine aggregate was found to be mostly subangular to subrounded, showing smooth to polished surfaces and a nonporous siliceous character. The gradations measured do not correspond well to the design gradations.

Extraction tests showed that the mixture from Kirbyville contained 0.8 percent more asphalt than the design content, whereas the mixture from Jasper contained an amount near the design content.

A combination of high field sand content and overall small aggregate size produced a mixture susceptible to plastic flow. This problem was compounded at Kirbyville by the excess asphalt. In time, traffic further densified the in-place mixtures to a low void level, which further decreased its shear strength in the wheelpath, and failure occurred as a result of rutting.

District 18

Rut depths in the intersection of FM 2170 and SH 5 were a maximum of 1 in. A second location (SH 66 at Rowlett Street) showed no signs of significant distress. Mix-design data for these pavements were not available. Both pavements were composed of a series of thin (less than 1 in.) overlays placed during a period of several years. Therefore, only a few tests were performed on the pavement cores.

Air voids in the uppermost pavement layer were low (1 to 2.5 percent) for the rutted intersection and acceptable (5 to 7 percent) for the nonrutted intersection. The gradation of the surface layer in the nonrutted intersection was coarser (½-in. maximum size) than that of the rutted intersection (⅜-in. maximum size). The presence of the larger stones in the surface layer may have been a significant factor in its resistance to plastic deformation; the composition of the subsequent layers and other factors such as traffic and subgrade were quite similar. In the minus No. 40 sieve sizes, the aggregates in the surface layers of both pavements were largely subrounded, smooth textured, and nonporous. Figure 1 shows that the gradation of the rutted mix was very near the maximum density line.

District 19

Rut depths ranged up to 1 in. for the intersection of US-259 and SH 11, and up to 0.9 in. for the intersection of US-67 and FM 989. Researchers originally understood that these pavements were about 2 years old; it was later determined that they were considerably older. Because a significant amount of data had been generated by that time, the pavements were included in the study. The intersection of US-59 and FM 989 exhibited no rutting or other forms of distress.

The top two layers of the two rutted intersection approaches exhibited significantly lower air-void contents than the upper layer of the nonrutted intersection approach (Table 2). Air voids in the wheelpaths of the rutted pavements were extremely low (about 1.5 percent) compared with the wheelpath of the nonrutted pavement (5.4 percent) and were also quite low outside the wheelpaths.

Asphalt contents in the uppermost layers were, on the average, higher for the rutted sections than for the nonrutted section. In the rutted portion of US-67 at FM 989, the measured binder content in the uppermost layer exceeded the design value by 1.2 percent. In the nonrutted pavement, the measured binder content was 0.9 percent less than the design value.

The aggregates in the upper layer of all three sections were crushed sandstone and field sand. Material in the plus No. 40 sizes from the top layers of all three intersections was angular and rough. However, the minus No. 40 material in all mixtures tested (both layers where applicable) was subrounded, smooth, and nonporous. The plus No. 40 material in the second layer of the rutted intersections was mostly pea gravel and was also subrounded, smooth, and nonporous. The pea gravel layer of FM 67 at FM 989 was measurably thicker in the cores from outside the wheelpath than in those from the wheelpath, which indicates that plastic flow (rutting) had occurred and may still have been occurring in this layer.

District 15

Three excellent intersection pavements on relatively high traffic-volume facilities were found in San Antonio (Table 1). Two of these pavements had been in service for 5 years, and 1 had been in service for 2 years. They showed no visible signs of distress. Each of the pavements was placed as new construction in two 1-in. lifts. Mixture-design data for these two mixes are presented in Table 3.

These mixtures were composed of 74 percent crushed limestone and 26 percent natural sand. It should be pointed out, however, that the sand was of exceptionally good quality in that the particles were angular to subangular and well graded. Typical gradations for the mixture are shown in Figure 2. The quantity of minus No. 200 sieve size material was comparatively low (about 3 percent). Asphalt contents were also comparatively low. However, asphalt film thickness in the mixtures at Toepperwein and Judson were calculated to be more than 9 microns, which is normally considered adequate for protection against moisture and oxidation.

District 8

An intersection that exhibited excellent performance after 6 years in service in Abilene was reported by District 8 personnel. It is located on SH 36 at Judge Ely Street and is exposed to average daily traffic of about 4,600 vehicles (Table 1). The surface mix was a ⅜-in. maximum size hot-mix asphalt

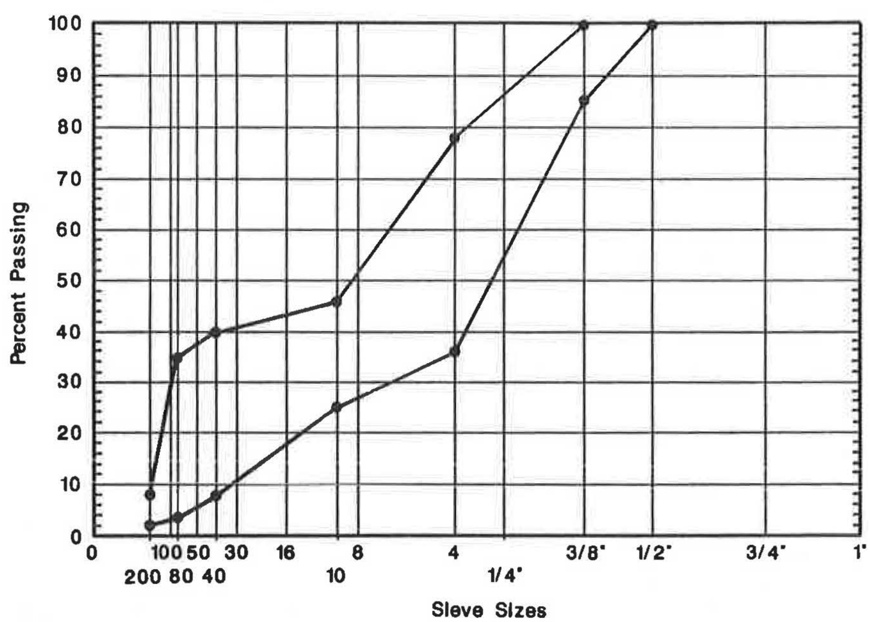

FIGURE 1 Gradation of extracted core for surface mix on intersection approach on FM 2170 at SH 5 in District 18 (sieve sizes raised to 0.45 power).

TABLE 2 PROPERTIES OF UPPER LAYERS IN CORES FROM INTERSECTIONS IN DISTRICT 19

	Location				
	US 259 @ SH 11		US 67 @ FM 989		US 59 @ FM 989
Rutting	Yes: 1.0-in.		Yes: 0.9-in.		No: 0-in.
Layer	1	2	1	2	1
Air Voids in Wheelpath, in.	1.5	1.4	1.8	1.4	5.4
Air Voids outside Wheelpath, in.	2.9	-	2.8	3.2	6.5
Asphalt Content, percent	5.7	5.9	6.9	4.7	5.1
Design Asphalt Content, percent	5.8	-	5.7	5.2	6.0
Hveem Stability for Mix Design	47	-	41	36	45
Aggregate Blend, percent					
Crushed Stone	58	-	65	-	60
Pea Gravel	-	-	-	50	-
Sand	25	-	25	35	20
Crusher Screenings	17	-	10	15	20

TABLE 3 MIX-DESIGN DATA FOR GOOD INTERSECTION PAVEMENTS IN DISTRICT 15

	Location			
Design Data	Toepperwein/Judson at IH 35		Coliseum Rd at IH 35	
Layer Identification	1 (Surface)	2	1 (Surface)	2
Specification Item	340	340	340	340
Mix Type	D	D	D	D
Aggregate Blend, percent				
Crushed Limestone	36	33	36	33
Crusher Screenings	7	7	7	7
Crushed Gravel	-	-	29	33
Crushed Sandstone	29	33	-	-
Field Sand	28	27	28	27
Absorption, percent	<1	<1	<1	<1
Minus # 200, percent	3.0	3.0	3.8	3.1
L. A. Abrasion, percent	30	30	30	30
Asphalt Source	Exxon	Exxon	Exxon	Exxon
Asphalt Grade	AC-20	AC-20	AC-20	AC-20
Asphalt Content	5.0	4.5	4.3	4.5
Avg. Specimen Density, percent	96.5	96.5	97.1	96.7
Initial Avg. Field Voids, percent	6.4 - 8.0	6.4 - 8.0	-	-
Average Hveem Stability	46	46	40's	40's
VMA, percent	15.0	15.0	13	15.0

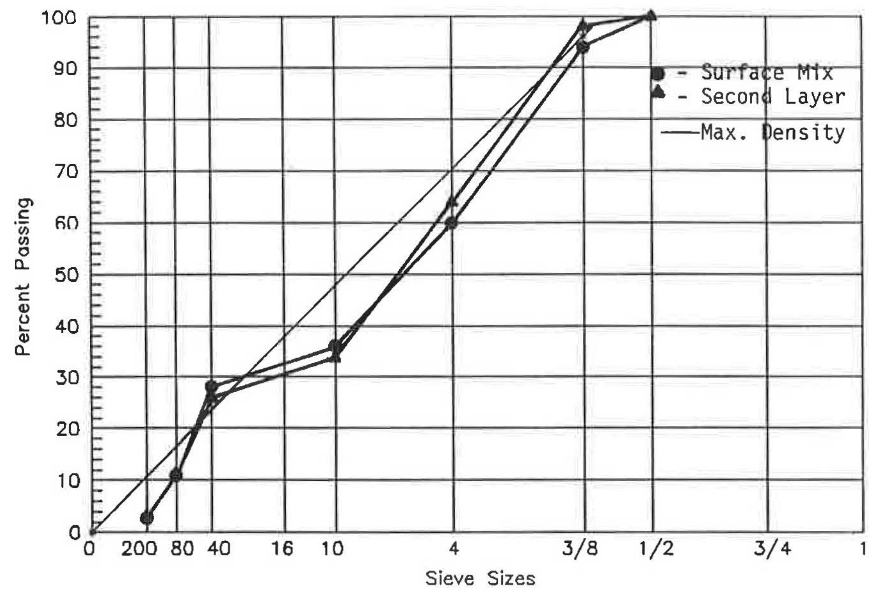

FIGURE 2 Design gradation for surface mixture and second layer on intersection approaches at Toepperwein and Judson at IH 35 in District 15 (sieve sizes raised to 0.45 power).

concrete (HMAC) overlay placed in a single 1½-in. lift. Visual inspection revealed no signs of plastic deformation, flushing, or other forms of distress.

The aggregate was composed of 88 percent crushed limestone and 12 percent field sand. The filler (minus No. 200) content was 4.0 percent. The design asphalt content was 6.2 percent, which yielded an average Hveem stability of about 51 percent. Field air voids after initial compaction were about 6 percent. The angularity of the coarse aggregate and the low field-sand content are partially credited with the satisfactory performance of this intersection pavement.

APPLICATION OF FINDINGS TO INTERSECTION ENGINEERING AND CONSTRUCTION

Although design engineers have no control of traffic volume, traffic loads, or environmental factors, adequate construction quality control as well as properly designed paving mixtures and structural systems are well within their jurisdiction. A well-designed asphalt paving mixture that is correctly mixed and placed can withstand the shear and compressive stresses of heavy traffic at intersection approaches and should exhibit adequate resistance to deformation when temperatures and wheel loads are at the peak. The following paragraphs offer suggestions designed to provide a margin of safety to minimize premature failures of specially stressed intersection pavements.

HMAC Specifications

Existing Texas SDHPT specifications for fine-graded HMAC surfaces allow and possibly encourage the use of gap-graded mixes (Figure 3). These mixtures are characterized by the hump in the gradation curve near the No. 40 sieve and a relatively flat slope between the No. 40 and the No. 10 sieves. This indicates a deficiency of material in the No. 40 to No. 8 sieve size range and an excess of material passing the No. 40 sieves. Mixtures of this type, particularly when the fines are composed primarily of natural sand, are termed "critical" because they lack resistance to plastic deformation, tend to rapidly loose stability if the asphalt content exceeds optimum, and become tender and shove during hot weather. One method of improving the aggregate grading specification to yield tough intersection mixes would be to lower the upper limit of the total percentage of material allowed to pass the No. 40 and 80 sieves. According to Chastain and Burke (*4*), in 1957, less than 20 percent of highway agencies allowed more than 37 percent passing the No. 40 sieve and more than 40 percent of them required less than 32 percent passing the No. 40 sieve.

The 0.45 power gradation chart, as used in this report, is particularly useful in evaluating aggregate gradations. A straight line, plotted from the origin of the chart to the percentage point plotted for the largest sieve with material retained, represents the gradation of maximum density. Aggregate gradation should be examined on the 0.45 power chart as a routine procedure during mixture design. When a plant inspector becomes accustomed to using this chart, it may help the inspector to recognize gradation problems early and make the necessary adjustments before large quantities of the mix are placed.

Although it is well known that rounded, smooth-textured siliceous gravels and sands generally produce AC mixtures subject to plastic deformation and moisture damage, existing Texas SDHPT HMAC specifications do not limit the use of these natural aggregate particles. The specification requires that a minimum of 85 percent of the particles retained on the No. 4 sieve have at least 2 crushed faces (primarily to address skid resistance). This is certainly a positive move regarding the coarse aggregate, but there is no limitation placed on the fine aggregate (minus No. 4). The quality and quantity of fine

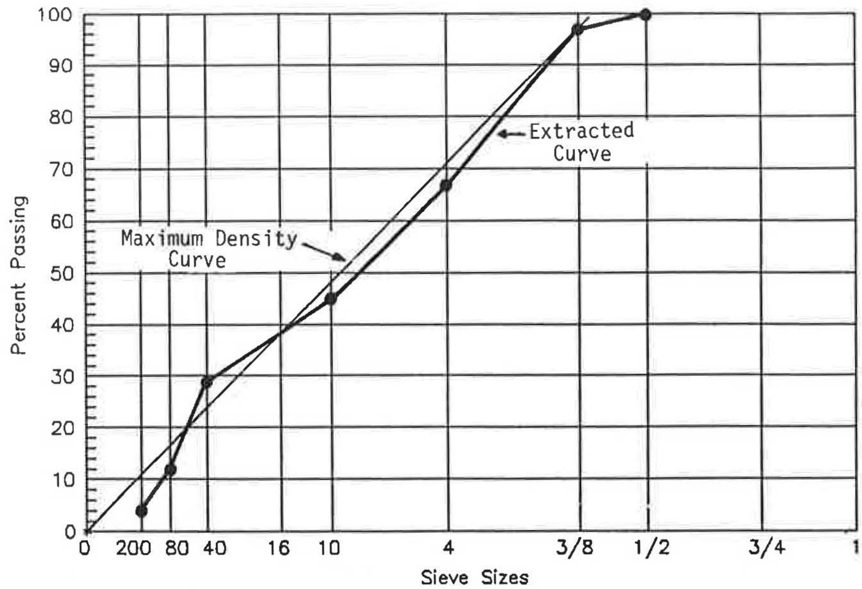

FIGURE 3 Aggregate specification limits for Texas SDHPT fine-graded asphalt concrete surface mixture (sieve sizes raised to 0.45 power).

aggregate is critical because it greatly influences the amount of asphalt a mixture can tolerate and the volume of air in the compacted pavement (*5–7*). Use of excessive quantities of poor quality natural sand is indirectly addressed in the specification by the requirement of a minimum Hveem stability. Experience, however, has shown that mixtures with satisfactory Hveem stability may yield unsatisfactory performance as surface courses on approaches to intersections that carry more than 7,000 vehicles a day. Evidence of this was demonstrated by the 2-year routine maintenance program for intersection pavements practiced in several districts. To provide a margin of safety, the natural aggregate particle content of mixtures to be applied at intersection approaches should not exceed about 15 percent (*5,8*). The quality of natural aggregate varies widely and should be considered by allowing special provisions to exceed the maximum limit when "sharp" natural sands with demonstrated good performance are used.

To meet gradation requirements with limited use of natural sand, it is usually necessary to replace these particles with "manufactured sand" (crusher screenings with limited minus No. 200). Texas currently has no specification for washed screenings, which has caused difficulties on occasion. For example, District 17 requisitioned washed screenings, but the material that was delivered contained only 3 percent less minus No. 200 material (15 instead of 18 percent) than the stone screenings usually received. A reasonable specification for washed screenings should require near 100 percent passing the No. 4 sieve and limit the amount passing the No. 200 sieve to less than 6 percent.

A target value for VMA should be obtained through the proper distribution of aggregate gradation to provide adequate asphalt film thickness on each particle and accommodate the design air-void system (*8*). Current Texas SDHPT specifications for HMAC do not require a minimum VMA. Recommended minimum VMA for various nominal maximum particle sizes have been developed by McLeod (*9*). These values are based on compaction using the Marshall hammer. Optimum values of VMA using the Texas gyratory compactor need to be established. On the basis of findings from a recent study sponsored by the National Cooperative Highway Research Program (*10*), it is reasonable to expect that acceptable VMA requirements using the gyratory compactor may be about 0.5 percent lower than those developed using the Marshall hammer.

Another item that is critical to mixture performance that is not addressed in the Texas SDHPT specifications is the ratio of filler (minus No. 200 aggregate) to asphalt. This ratio is computed by dividing the weight percent or mass of filler by the weight percent or mass of asphalt, respectively, and should range between a minimum of 0.6 and a maximum of 1.2 (*8*). Mixtures containing preponderantly absorptive aggregates will need less filler than mixtures composed primarily of nonabsorptive aggregates. Theoretically, absorptive aggregates will selectively absorb the lighter, more mobile components (lower viscosity) of the asphalt more deeply into the aggregate, leaving, in effect, a harder grade material to act as binder. In such cases, it may be advisable to design at the lower limit of filler content to ensure adequate mixture flexibility. (When using highly absorptive aggregates, improvements in mixture quality may be gained by specifying an asphalt one grade softer than usual to provide for loss of the low viscosity materials due to absorption. Research has not been performed to establish the critical level of absorption above which a softer asphalt should be used.)

Finally, incorporation of some or all of the above recommended changes in the Item 340 specification will result in a substantial increase in the Hveem stability. As a measure to further ensure that the mixture will withstand the special stresses applied at intersection approaches, the minimum required Hveem stability should be raised to a value near 40. A value of 37 is recommended by the Asphalt Institute (*3*) and the Federal Highway Administration (FHWA) (*8*) for traffic volumes exceeding 1 million equivalent single axle loads during the design life.

Methods of Testing

In the search for possible reasons for the excess asphalt found in some paving mixtures, standard Texas test methods were investigated. Design of hot bituminous mixtures in Texas requires the use of test methods Tex-205-F (mixing) and Tex-206-F (compaction) for specimen preparation. These test methods specify a mixing temperature of 275°F and a compaction temperature of 250°F, regardless of the grade or viscosity-temperature relationship of the asphalt cement. Examination of 1988 data for AC-20 asphalts used in Texas revealed that the viscosity may range from 6 to 14 St at 250°F and from 2.8 to 6.8 St at 275°F. On the basis of the experience of the authors, it is believed that this range of viscosities will significantly affect density of the compacted specimens. Higher viscosity will, of course, result in higher air voids. Because optimum asphalt content is selected at 97 percent density (or 3 percent air voids) by the Texas method, it follows that the harder asphalts (at compaction temperature) will require higher asphalt contents. Now, because the materials under discussion are all AC-20s, the viscosity range at high pavement service temperatures (e.g., 140°F) is comparatively small (1,610 to 2,280 St, based on 1988 Texas asphalt data). Therefore, in service, the higher asphalt content required by the design procedure may be detrimental to resistance to plastic deformation of the mix. Furthermore, when modified asphalts, which often have significantly lower-than-usual temperature susceptibilities (or much higher viscosities at the compaction temperature), are used, the standard design procedure may require a binder content in excess of that desirable for optimum performance.

The potential for these standard test methods to produce mixes with excess asphalt should be investigated. If it is determined that the risk is unacceptable, the test methods should be modified to require mixing and compaction at some preselected viscosity instead of the constant temperatures. Guidelines for the Marshall design procedure (*3*, AASHTO T245-82) recommend a mixing temperature that provides 170 cSt and a compaction temperature that provides 280 cSt. Asphalt viscosity at compaction temperatures when the Texas gyratory compactor is used may not be as critical as viscosity when the Marshall hammer is used, but this needs to be verified.

Design Considerations

In contrast to the current empirical pavement design procedures in which one is unable to determine if a paving mixture of specific strength parameters is capable of sustaining vertical and horizontal loads of varying magnitude, a mechanistic design method may provide a rational approach to design of intersection pavements capable of withstanding the applied vertical and horizontal loads (*1*).

The octahedral shear stress ratio (OSR) concept (*11*) can be used to evaluate the potential of an AC overlay, over existing AC or portland cement concrete (PCC), to rut or deform under traffic. This ratio is based on the principle of octahedral shear stress, which is a scaler or numerical representation of the stress state at any point within the pavement cross section. This scaler quantity is calculated from the three normal and six shear stresses acting at a given point within the pavement. Because materials fail in different modes on the basis of the conditions of loading and temperature and because a number of failure criteria exist, selecting the proper failure mode and criterion by which to judge failure potential is of great importance. It makes sense that the potential of an AC pavement to deform, shove, or rut at an intersection should be evaluated on the basis of a shearing failure criterion.

The procedure is summarized in the following steps:

1. Compute the maximum octahedral shear stress in the AC overlay under the climatic, structural, and loading conditions involved. Ameri-Gaznon and Little (*11*) accomplished this for the majority of conditions that will normally be encountered.
2. Measure the octahedral shearing strength of the AC material used in the overlay at the same state of stress at which the maximum octahedral shearing stress occurs within the pavement (Step 1). This can be accomplished by following the procedure for testing and analysis outlined elsewhere (*1*).
3. Compute the ratio of the maximum octahedral shear stress developed within the AC overlay to the octahedral shear strength of the AC material used in the overlay at the same stress state that occurs in the overlay at the critical point.

When the OSR is high, the potential to deform excessively is high. When the OSR is low, the potential to deform is low. Theoretically, an OSR equal to unity represents incipient failure. However, limited dynamic creep data have shown that conditions favoring excessive deformation can result at OSRs of 0.65 in highways subjected to normal loading conditions. Although it is currently not possible to identify an OSR that represents a selected quantification of deformation, the OSR is an excellent way to compare the relative potential of various mixtures of AC to resist permanent deformation in specific structural and climatic categories.

Tougher Asphalt Mixtures

It is possible to substantially reduce plastic deformation of the pavement by using larger nominal maximum size aggregates that are mixed with harder grade of asphalt (e.g., AC-30) or modified asphalt binder. Davis (*12*) states the largest stone size should be two-thirds the pavement thickness. Large crushed aggregate generally require less energy to produce and less asphalt and are, therefore, less expensive. Research has shown that certain polymer additives will produce a significant increase in asphalt viscosity at high pavement service temperatures while having little affect on viscosity at low pavement service temperatures (*13,14*). Higher-than-usual compaction energy may be required for these mixtures.

Plant mix seals or open-graded friction courses are quite resistant to rutting. These mixtures may provide a viable alternative to the usual fine-grained dense-graded mixtures for overlays or reconstruction of intersection approaches. Additional benefits provided by plant mix seals include improved surface friction and resistance to hydroplaning and reduced glare at night, which are important factors to consider at intersections.

Stone-filled mixtures (*15*), briefly described by Button and Perdomo (*5*), should also provide excellent service on intersection approaches. Stone-filled mixtures essentially consist of a small top-size, dense-graded asphalt-concrete mix combined with about 45 percent (by total weight of mix) of a larger single-sized stone of about ¾ in. for surface courses. A stone matrix is formed by the large stones, and the voids between are filled with the fine-grained asphalt mix. The bridging effect of the large stones resists plastic deformation and further densification under traffic in a manner similar to the open-graded mixes.

PCC

An alternative approach to eliminate plastic flow of the pavement surface materials at intersections is the application of PCC (*5*). Generally, the major portion of load-carrying capacity of pavements surfaced with PCC is provided by the slab itself. This is in contrast to the flexible pavement, wherein the strength of the pavement is provided by the thick layers of the subbase or base or both (*2*).

Construction Sequence

An efficient and possibly cost-effective approach to alleviate permanent deformation at critically stressed pavement sections is to employ a sequential construction technique. In this approach, the intersections and other critical areas that receive a higher concentration of vehicle maneuvering are constructed first with a preselected mixture that is designed to conform with the intensity of the traffic and applied vertical and horizontal loads. Once construction of these areas is completed, construction of the tangent sections may begin; the normal mixture that is compatible with the type of traffic to which those sections are exposed should be used. On occasion, it may be advantageous to let bids separately for construction or overlaying of intersections and work on connecting tangent sections.

Intersection Geometries

Traffic monitoring was conducted at several intersections to estimate the distance from the intersection at which braking force is first applied to reduce vehicle speed and then further applied to bring a vehicle to a complete stop. As mentioned previously, rut depths were measured along the intersection approaches. This information was used to estimate the average length of the damaged zone of typical intersection approaches, and thus to estimate the length of approach that should receive specially designed pavements. Evidence indicates that the typical length of an intersection approach that should receive special treatment ranges from 100 to 250 ft, depending on the amount and speed of traffic, traffic control methods, and the average length of the queue that forms during stoppages.

Economic Considerations

The potential for significant economic benefits appears promising when intersection approaches are engineered specifically to accommodate the special stresses to which they are subjected. Cost comparisons of these alternatives on both a first-cost and life-cycle basis are of interest to the engineer and should be considered when the optimum rehabilitation alternative for a particular intersection approach is selected. An example to illustrate the potential savings follows.

On the basis of the findings in this study, it seems reasonable to assume that an improperly designed intersection will need to be maintained by overlaying or milling or both every 2 years. As a basis for comparison, assume an intersection approach consisting of four 12-ft lanes 150 ft in length will be (*a*) overlaid with 1 in. of asphalt maintenance mix every 2 years or (*b*) designed and built with special hot mix to serve without maintenance for 10 years. The approximate costs of the materials, equipment, and labor for the two alternatives are presented in Table 4. It can be seen from this oversimplified example that a savings of $4,920 per intersection approach can be realized every 10 years when an approach pavement is built during initial construction or rehabilitation to withstand special stresses. If pavement user cost was considered at an intersection, it would be significant because traffic flow on at least two different thoroughfares is interrupted when maintenance or rehabilitation activities are required.

Additional benefits that can be gained by considering the special stresses associated with intersections during pavement design and construction include (*a*) improved driver safety as a result of the good condition of the pavement surface (no rutting or flushing and adequate surface friction in wet weather) and (*b*) no buildup of maintenance mix, which is often of lower quality than hot mix.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

1. The most common form of distress associated with failure of AC intersection-approach pavements was plastic deformation manifested in the form of rutting. Shoving and flushing were evident in some cases. In all cases investigated, rut depths more than 250 ft from the intersection were practically negligible. Ruts always become progressively deeper nearer the intersection. This combination of findings clearly indicates that the slower the traffic and the greater the frequency of horizontal forces (deceleration and acceleration), the greater the damage to AC pavements.

2. The leading cause of intersection pavement failure related to AC materials was binder in excess of that required by the optimum mixture design. It also appears that on occasion asphalt content is arbitrarily increased to facilitate compaction.

3. Most of the mixtures studied contained relatively high percentages of natural (uncrushed) sand. The smooth, rounded, nonporous, glassy character of these fine aggregates causes the mixture to be sensitive to asphalt content and weak in shear strength, which thus imparts a higher propensity for permanent deformation. Approximately 30 percent minus No. 40 sieve size material, which was largely field sand, was found in all the problem intersections. [State specifications for fine-graded surface mix (⅜-in. maximum size) allow up to 40 percent passing the No. 40 sieve.]

TABLE 4 PAVEMENT TREATMENT ALTERNATIVES AND COST COMPARISONS

Maintenance Alternative:

One-inch thick level-up course of asphalt mix placed by maintenance forces every 2 years for 10 years. Assume 1 day required to perform maintenance each time.

Materials -	42 tons HMCL at $20/ton	$ 850
Equipment -	2 dump trucks at $30 ea/day	60
	1 sign truck at $30/day	30
	1 steel wheel roller at $20/day	20
	1 distributor truck at $30/day	30
	1 grader at $50/day	50
	Total Equipment	$ 190
Labor -	1 crew leader at $100/day	100
	2 maint. operators at $80 ea/day	160
	3 maint. workers at $65 ea/day	195
	1 flagman at $50/day	50
	Total Labor	$ 505
	TOTAL DAILY COST	$ 1,545

Assume 4 repetitions of the above maintenance activity will be performed in 10 years.

TOTAL 10 YEAR COST	$ 6,180

Ten-Year Design Alternative:

During construction, apply 3 inches of special hot mixed asphalt concrete (HMAC) designed to perform satisfactorily without maintenance for 10 years.

Materials -	Additional Cost of 126 tons of Special HMAC, $10/ton	$ 1,260

Savings = $ 6,180 - $ 1,260 = $ 4,920 per intersection per 10 years

4. Aggregate gradations appeared to be dense (low VMA) for some intersection pavements that experienced early failure. Dense aggregate gradations leave little room for asphalt binder, and the mixture may become unstable with a slight excess of asphalt. This is particularly true for fine-graded asphalt mixtures.

5. Air-void contents obtained from almost all the rutted intersection pavements were comparatively low (less than 3 percent), particularly in the wheelpaths. This indicates that either the mixture designs were too dense or that they were overcompacted during construction such that additional densification by traffic caused the mixtures to become unstable soon after construction and exhibit plastic flow (rutting or shoving or both).

6. Several districts had established a routine 2-year maintenance program, wherein most intersection approaches in the district with significant traffic received treatment every other year. This is an indicator of the severity of the problem of pavement service life at intersections.

7. The potential for significant economic benefits appears promising when intersection approaches are designed and constructed specifically to accommodate the special stresses to which they are subjected.

8. A rational approach for design of asphalt mixtures for intersections using the OSR appears capable of providing suitable mixtures. This procedure needs verification.

Recommendations

1. Reduce the allowable quantity of sand-sized (minus No. 10 to plus No. 200) particles in asphalt mixtures to be used on intersection approach pavements.

2. Limit the natural (uncrushed) sand content of mixes to be used on intersection pavements to about 15 percent. Special provisions should be allowed for "sharp" natural sands that have demonstrated good performance wherein they may exceed the specified value.

3. Institute a specification for voids in the VMA considering that the gyratory compactor generates a specimen that simulates final density after significant traffic. Optimum VMA values for gyratory compacted specimens may be slightly lower than those proposed by FHWA and the Asphalt Institute.

4. Require a minimum Hveem stability of 40 for mixes to be applied on the surface of intersection approaches that have high traffic volumes. This is an indirect method of ensuring good aggregate quality.

5. Use of comparatively large maximum-size aggregate or asphalt modifiers or both to increase viscosity at higher pavement service temperatures may offer cost-effective alternatives to prolong intersection pavement life. Options include dense-graded large-stone mixes (⅝- and ⅞-in. maximum size), stone-filled mixes, and plant mix seals. The National Asphalt Pavement Association recommends a maximum aggregate size of 3 in. or up to two-thirds the pavement layer thickness, whichever is smaller, for heavy-duty mixes.

6. Specify constant asphalt viscosities during mixing and compaction instead of constant temperatures for standard test methods. Use of the mixing temperature of 275°F and the compaction temperature of 250°F for hard or modified asphalts with the standard Texas mix-design procedure may result in excess binder content, which could lead to rutting or flushing.

7. Consider the use of PCC for intersection approaches for which economic analyses of the alternatives indicate its appropriateness.

8. Employ a sequential construction technique in which all intersection approaches within the project are built or overlaid before the remainder of the job with a special, tough mix to accommodate the special stresses.

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Evaluation of Pavement Bleeding on I-55 in Illinois

E. R. BROWN, STEPHEN A. CROSS, AND JAMES G. GEHLER

Shortly after construction fat spots began to appear throughout the project on a 3-in. hot-mix asphalt (HMA) overlay of an existing portland cement concrete pavement. The fat spots appeared to occur at the end of truck loads. After time the fat spots developed into potholes and the asphalt appeared to be stripped from the aggregate at the bottom of the potholes. Some rutting and shoving also developed at the fat spots. The objective of this study was to evaluate the HMA and to determine potential causes of the fat spots. The test plan included inspecting the pavement visually and obtaining core samples from fat spots, adjacent to fat spots, and from random locations throughout the project. Rut-depth measurements were also obtained. The cores were tested for asphalt content, gradation, void content, and slag content. Several of the asphalt mix layers were split into top and bottom halves, and the asphalt content and gradation of each half were compared. The asphalt cement from several cores was recovered, and the viscosity and penetration were determined. The results of this study indicated that the most likely cause of the fat spots was contamination of the HMA with some solvent (probably diesel fuel) during the placement operation.

The existing pavement surface on I-55 near Collinsville, Illinois, was overlaid from 1985 to 1987 with 3 in. of asphalt mix. This work, finished in 1987, was performed under two contracts. The asphalt mix that experienced problems was placed in 1987.

Shortly after construction, personnel of the Illinois Department of Transportation (DOT) noticed fat spots throughout the project. Most of the spots appeared to occur at the end of truck loads. Over time these spots developed into potholes, and the asphalt appeared to be stripped from the aggregates at the bottom of the potholes. Some rutting and shoving also developed at the fat spots.

The objective of this study was to evaluate the asphalt mix placed on I-55 and to determine potential causes of the fat spots. The study included an inspection of the roadway to develop a detailed test plan. A test plan was developed that included cutting a trench through one of the fat spots, drilling cores in and adjacent to these spots as well as randomly throughout the project. The asphalt samples were evaluated to determine aggregate gradation, asphalt content and properties, and mix properties. The test data were analyzed to identify possible causes of the localized bleeding problems.

FIELD INSPECTION

The inspection of the pavement on I-55 in May 1989 verified that a number of fat spots existed throughout the project. The distance between the fat spots varied, but it appeared that the problem was generally associated with some segregation that typically occurs at the end of truck loads. In some places it appeared that the asphalt in these fat spots had migrated to the pavement surface. In these places it appeared that the asphalt had been stripped from the underlying aggregates. The test plan was set up to determine whether stripping had occurred or if some other problem had caused the fat spots. The area of pavement adjacent to these fat spots had become rough in some cases, and it appeared that the roughness would increase with time.

TEST PLAN

A test plan was developed for I-55 to determine the cause of localized bleeding. Samples of the asphalt mix were taken in the fat spots, adjacent to the fat spots, and at random locations from station 916+25 to station 937+50 (Figure 1). A total of thirty 4-in. cores was taken for testing. A trench approximately 2 ft wide and approximately full width was excavated at station 934+70, at a location that had bleeding spots, to determine if the problem could be identified by viewing the side of the trench.

Typical cores were taken at 300-ft. intervals to evaluate the average asphalt mix properties. Additional cores were taken in bleeding areas at stations 923+47, 936+10, and 936+93. One core was taken inside the bleeding area and one core was taken immediately adjacent to the bleeding area. The asphalt content and aggregate gradation of the samples in the bleeding areas were compared with that from adjacent areas and with that from the typical cores.

The cores in the bleeding areas and adjacent to the bleeding areas were sawed into a top half and a bottom half. The asphalt content and aggregate gradation were determined for each half in order to determine whether the asphalt cement and perhaps some fines had migrated from the bottom of the top layer to the surface.

Rut measurements were taken at the trench; the results are plotted in Figure 2. The rut depth at this location was approximately 0.6 in. A typical localized bleeding area is shown in Figure 3.

There was some concern about the amount of slag that was used in the asphalt mixture. Because slag has a high absorption, a high variability in slag content would adversely affect the optimum asphalt content. The amount of slag was measured by visually separating the slag particles from the limestone particles on individual sieve sizes during the aggregate gradation tests. The slag particles were weighed for each sieve

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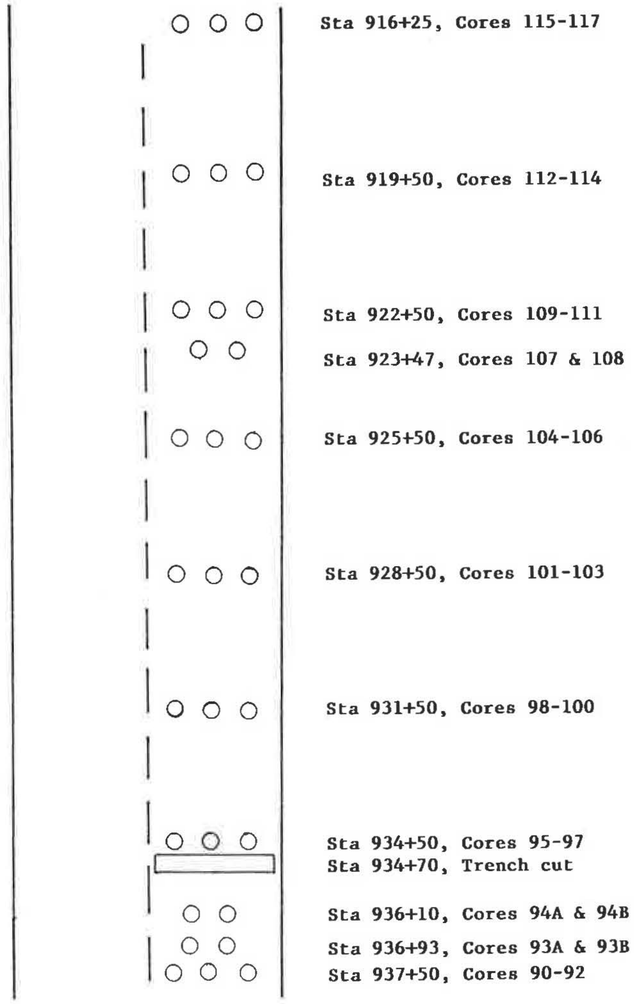

FIGURE 1 Layout of test section—northbound lanes of I-55 near Collinsville, Ill.

size and compared to the weight of the nonslag particles. The percentage of slag larger than the No. 16 sieve was determined in this way. Because this is not a standard test, the accuracy is questionable, but it was one method of estimating the slag content. The material smaller than a No. 16 sieve could not be separated visually, and hence it was not considered in the slag content.

TEST RESULTS AND DISCUSSION

The pavement section (Figure 2) shows that two layers of asphalt mixture were placed over an existing layer of concrete. The total thickness of asphalt mixture was approximately 3 in. Figure 2 shows that rutting had occurred in the surface and binder course. The rutting was more severe at the locations that had localized bleeding than in the other areas. The major concerns at the time of inspection were the rutting problem in general and the ravelling and potholes that were developing at the localized fat spots.

An inspection of the pavement adjacent to the trench cut through a bleeding spot did not indicate the causes of the problem. This inspection did show that the problem was confined to the top layer of asphalt mixture and that the lower half of the layer had less asphalt cement than the top half.

The results of slag content tests are presented in Table 1. The measured slag content ranged from a low of 28.4 to a high of 31.6. The job mix formula required 39.3 percent slag between the No. 4 and No. 16 sieves. The test used to measure slag is not a standard test; therefore the accuracy of the test is unknown. Because the measured amount of slag is consistent at each of the stations it is doubtful that variation in slag content caused the bleeding problem.

The aggregate gradation of the surface course is reasonably consistent at the various sampling locations. The average gradation of the surface course does deviate from the JMF on the No. 4 and No. 8 sieves (Table 1). After evaluation of the individual gradation it is believed that the gradation has had no effect on rutting or the localized bleeding problems. It is possible that segregation of the mix at particular points could be a problem. That is discussed later.

The measured in-place asphalt content is approximately 1 percent lower than the designed asphalt content (Table 1). This lower asphalt content could be the result of testing error, mix modifications made during construction, or failure of the contractor to meet the job mix formula. The relatively low in-place voids (4.6 percent average) show that the asphalt content actually used is not too low. Regardless of the reasons for an asphalt content lower than design, the asphalt content

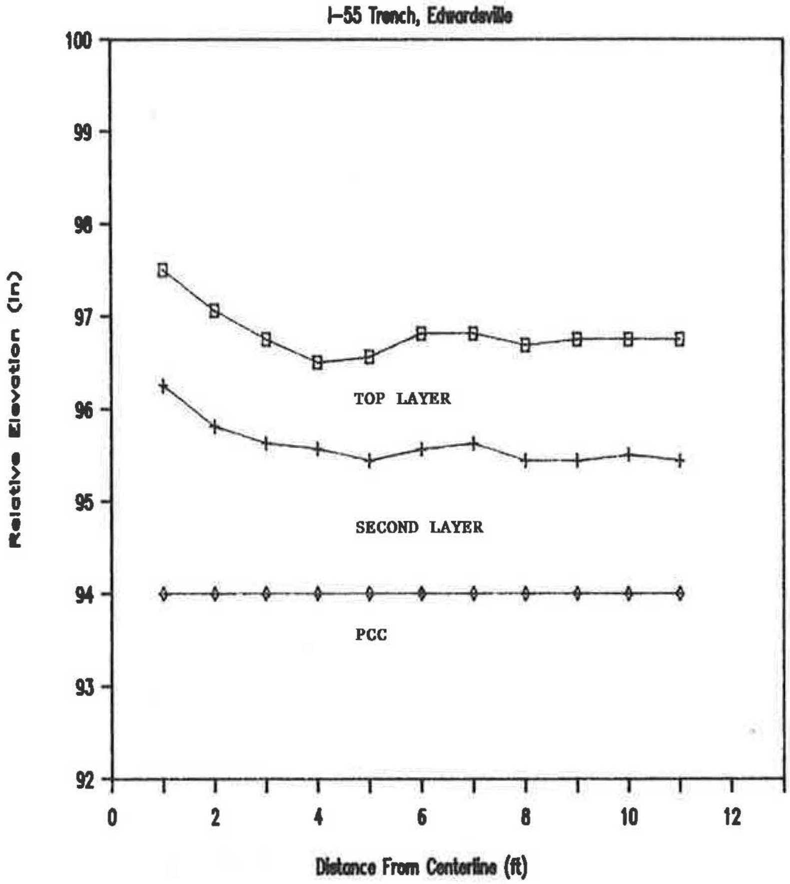

FIGURE 2 Trench cut at Station 934+70, northbound lane of I-55 near Collinsville, Ill.

FIGURE 3 Typical localized bleeding area, northbound lanes of I-55 near Collinsville, Ill.

actually measured on the final product appears to be acceptable on the basis of the in-place voids.

Samples of the asphalt mix in Layers 1 and 2 were recompacted with a Marshall hammer and with a Gyratory Testing Machine. The compaction results (Table 2) show that the voids were low in Layer 1 and satisfactory in Layer 2. The low voids in Layer 1 are likely one cause for rutting (*1–3*). The Gyratory Shear Index (GSI) values in layers 1 and 2 are 1.2 and 1.3, respectively. A mix with a GSI greater than 1.1 may have a tendency to rut; a mix with a GSI greater than 1.3 is almost certainly going to rut (*1*). The low voids and high GSI help explain the rutting problem but do not explain the bleeding problem at localized areas.

Test results from the mix obtained from localized bleeding areas are presented in Table 3. The average asphalt content is 5.4 percent higher in the top half of the asphalt cores than in the bottom half (8.8 percent in top half and 3.4 percent in bottom half). The average aggregate gradation is approximately 3 percent finer in the top of the cores than that in the bottom. This indicates that the asphalt and fines likely migrated from the bottom of the top layer to the surface.

Test results on material taken adjacent to the bleeding areas are presented in Table 4. These results show that the average gradations for the top half and bottom half are essentially the same. The average asphalt content is actually slightly higher in the bottom half of the cores, which is opposite of that shown in the bleeding area. A slightly higher asphalt content in the bottom half of the core could be the result of normal variability in test results, or there could be some scientific reason for it being lower. For example, the bottom half of the layer could include some tack coat material, which would increase

TABLE 1 PROPERTIES OF RANDOM SAMPLES OF ASPHALT MIX (TOP LAYER)

Property	JMF	916+25	919+50	922+50	925+50	928+50	931+50	934+50	937+50	Average
Rice Gravity	---	2.376	2.376	2.380	2.406	2.442	2.397	2.413	2.402	2.399
Bulk Gravity	---	2.271	2.304	2.292	2.279	2.298	2.298	2.257	2.305	2.288
Voids in Total Mix	---	4.4	3.0	3.7	5.3	5.9	4.1	6.5	4.0	4.6
Slag Content	39.3	29.3	29.8	28.4	31.2	30.5	30.8	30.7	31.6	30.3
Asphalt Content	7.3	6.7	6.5	6.2	6.3	5.6	6.5	6.1	6.4	6.3
Aggregate Gradation Sieve Size										
1/2 inch	100	100	100	100	100	100	100	100	100	100
3/8 inch	98	98	98	97	97	96	97	94	98	97
No. 4	52	60	64	59	63	50	56	54	54	58
No. 8	30	37	41	38	40	30	36	34	33	36
No. 16	20	25	28	26	27	22	25	24	24	25
No. 30	13	17	18	18	18	15	18	17	16	17
No. 50	9	11	12	11	11	11	12	11	11	11
No. 100	7	7	8	8	8	7	9	8	8	8
No. 200	5.3	4.8	6.4	5.8	5.8	5.4	6.3	5.8	5.5	5.7

TABLE 2 PROPERTIES OF RECOMPACTED MIX

Compactive Effort	Layer No.	Voids in Total Mix	Marshall Stability	Marshall Flow	Gyratory Shear Index
Marshall 75 Blows	1	2.2	3888	12	---
GTM (120 psi, 1 degree angle, 300 revolutions)	1	2.1	3165	14	1.2
Marshall 75 Blows	2	3.6	3194	12	---
GTM (120 psi, 1 degree angle, 300 revolutions)	2	3.1	3033	13	1.3

TABLE 3 PROPERTIES OF TOP HALF AND BOTTOM HALF OF SAMPLES TAKEN FROM BLEEDING AREAS (TOP LAYER)

Property	Top Half				Bottom Half			
	936+93	936+10	923+47	Average	936+93	936+10	923+47	Average
Asphalt Content	8.8	9.4	8.3	8.8	3.0	2.3	4.9	3.4
Aggregate Gradation Sieve Size								
1/2 inch	100	100	100	100	100	100	99	100
3/8 inch	96	97	100	98	98	99	97	98
No. 4	52	62	70	61	46	58	71	58
No. 8	31	39	44	38	26	33	45	35
No. 16	23	28	30	27	19	22	31	24
No. 30	17	22	20	20	14	15	21	17
No. 50	12	17	13	14	9	10	14	11
No. 100	9	14	10	11	6	6	10	7
No. 200	6.8	11.6	7.3	8.6	4.2	4.6	7.3	5.4

TABLE 4 PROPERTIES OF TOP HALF AND BOTTOM HALF OF SAMPLES TAKEN ADJACENT TO BLEEDING AREAS (TOP LAYER)

Property	Top Half				Bottom Half			
	936+93	936+10	923+47	Average	936+93	936+10	923+47	Average
Asphalt Content	5.6	6.2	6.1	6.0	6.1	6.7	6.5	6.4
Aggregate Gradation Sieve Size								
1/2 inch	100	100	100	100	100	100	100	100
3/8 inch	96	98	100	98	98	100	98	99
No. 4	51	59	64	58	53	56	64	58
No. 8	30	35	39	35	32	34	39	35
No. 16	21	24	26	24	23	24	27	25
No. 30	15	16	17	16	16	16	18	17
No. 50	11	12	11	11	11	11	12	11
No. 100	8	8	8	8	8	8	8	8
No. 200	5.8	6.4	5.8	6.0	6.0	5.9	5.9	5.9

the asphalt content. It is obvious that little or no migration has occurred outside the bleeding areas, but significant migration has occurred inside these areas.

A summary of the test results obtained from samples taken at random, inside bleeding areas, and adjacent to bleeding areas is presented in Table 5. These data show that the overall gradation and asphalt content on random samples, samples from bleeding areas, and samples adjacent to bleeding area are approximately the same. The major difference is the higher amount of material passing the No. 200 sieve in the bleeding areas. This indicates that the mixture initially placed contained the correct gradation and asphalt content but after compaction or traffic or both the asphalt cement and fines migrated from the bottom of the top layer to the surface.

The asphalt was recovered from the bleeding area and adjacent to the bleeding area to determine if there were differences in asphalt properties between the two areas. It is obvious from Table 6 that there are significant differences. The properties of the asphalt from the bleeding areas are considerably softer than the properties of asphalt adjacent to bleeding areas. In fact the asphalt binder from bleeding areas was softer than the original asphalt would have been. For instance, the viscosity of 958 is close to that for an AC-10, 460 is close to that for AC-5, and 322 is close to that required for an AC-2.5. The original asphalt cement on this project was an AC-20. Hence the viscosity of the asphalt recovered from the in-place mix should be significantly higher than the results indicated.

The low viscosity of the recovered asphalt cement indicates some type of contamination. Because the viscosity of the asphalt cement recovered from samples taken adjacent to bleeding areas is reasonable, the contamination must have occurred after mixing. It is suspected that the contamination either came from the use of diesel or some other unacceptable release agent to coat truck beds or from some spillage of one of these materials on the binder course before overlaying. Diesel fuel is the most likely contaminant because it evaporates slowly, and thus would keep the viscosity of the asphalt low for a long period of time. The specific cause of the contamination problem was not identified.

TABLE 5 SUMMARY OF ASPHALT CONTENTS AND GRADATION AT VARIOUS LOCATIONS IN TOP LAYER

Property	JMF	Average of Random Samples	Bleeding Areas			Adjacent to Bleeding Areas		
			Top Half	Bottom Half	Combined	Top Half	Bottom Half	Combined
Asphalt Content	7.3	6.3	8.8	3.4	6.1	6.0	6.4	6.2
Aggregate Gradation Sieve Size								
1/2 inch	100	100	100	100	100	100	100	100
3/8 inch	98	97	98	98	98	98	99	98
No. 4	52	58	61	58	60	58	58	58
No. 8	30	36	38	35	36	35	35	35
No. 16	20	25	27	24	26	24	25	24
No. 30	13	17	20	17	18	16	17	16
No. 50	9	11	14	11	12	11	11	11
No. 100	7	8	11	7	9	8	8	8
No. 200	5.3	5.7	8.6	5.4	7.0	6.0	5.9	6.0

TABLE 6 SUMMARY OF ASPHALT PROPERTIES IN BLEEDING AREAS AND ADJACENT TO BLEEDING AREAS

Property	Bleeding Areas			Adjacent to Bleeding Areas		
	936+93	936+10	923+47	936+93	936+10	923+47
Viscosity (140°F, Poises)	958	460	322	10003	5430	2575
Penetration (0.1 mm)	290+	169+	350+	30	36	151

SUMMARY

Small localized bleeding areas were identified on I-55 a short time after placement. The bleeding areas developed into potholes in some cases and increased rutting. A test pattern was developed to determine causes of the bleeding problem. After completion of tests it was obvious that the bleeding was a result of migration of the asphalt cement and filler from the bottom of the surface course to the top of the course. Tests also showed that the asphalt cement recovered from the bleeding areas had much lower viscosities than expected, even lower than the original viscosity. It was concluded from this that the bleeding was caused by contamination with some solvent, probably diesel fuel. Further investigation, which included conversations with the contractor and state DOT personnel and a review of the construction records, did not identify the exact cause(s) of contamination.

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Investigation of Unstable Asphalt Concrete Airfield Pavement

RANDY C. AHLRICH

The U.S. Army Engineer Waterways Experiment Station (WES) was requested by the Middle East/Africa Projects Office (MEAPO) in April 1989 to provide technical assistance in analyzing an unstable asphalt concrete (AC) airfield pavement. An AC overlay had been constructed on an aircraft parking apron and taxiway. The overlay had exhibited significant deformation and depressions under parked aircraft. The performance of the unstable AC pavement was unacceptable. The Materials Research and Construction Technology Branch of the Geotechnical Laboratory at WES was requested to perform laboratory tests on AC specimens to determine asphalt cement, aggregate, and AC mixture properties. The purpose of this analysis was to evaluate the in-place AC material for compliance with specifications, determine possible causes for pavement failure, suggest procedures to avoid problems in the future, and recommend options for the repair of the airfield pavement. The laboratory evaluation of the AC material indicated that the poor performance was due to an improperly designed and produced AC mixture. Several factors contributed to the unstable mixture: (*a*) the aggregate gradations were consistently out of specification and were gap-graded, (*b*) the amount of natural sand in both the surface course and intermediate course was extremely high, and (*c*) the mix designs for the surface-course and intermediate-course mixtures did not meet the minimum requirements of the specification.

The U.S. Army Engineer Waterways Experiment Station (WES) was requested by the Middle East/Africa Projects Office (MEAPO) in April 1989 to provide technical assistance in analyzing an unstable asphalt concrete (AC) airfield pavement in Egypt. An AC overlay had been constructed on an airfield parking apron and taxiway. The AC overlay had exhibited significant deformation and depressions under normal aircraft traffic. The performance of the unstable AC was unacceptable.

The Materials Research and Construction Technology Branch of the Geotechnical Laboratory was requested to perform laboratory tests on AC specimens to determine asphalt cement, aggregate, and AC mixture properties. The purpose of this analysis was to evaluate the in-place AC materials for compliance with specifications, determine possible causes for the pavement failure, suggest procedures to avoid these problems in the future, and recommend options for the repair of this airfield pavement.

MEAPO also requested technical assistance during the repair and construction of the airfield pavement in February 1990. A visit to the construction site was required in order to monitor the production and construction of the pavement. Additional WES laboratory testing was conducted to evaluate new materials and to determine why the WES laboratory data did not agree with the field laboratory data.

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LABORATORY ANALYSIS OF IN-PLACE PAVEMENT MATERIAL

Sample Description and Preparation

In June 1989, WES received four slab samples of AC from Egypt. The samples were approximately 2- × 2-ft in size and 4.5 to 6.5 in. thick. Each pavement sample consisted of a surface course layer, an intermediate course layer, and a single bituminous surface treatment. Two pavement samples (S-3 and S-4) had a fuel-resistant sealer coating on the surface course layer.

Because of the unstable condition of the AC pavement, it was decided that all pavement samples would be evaluated and that both surface-course and intermediate-course material would be tested. Before any testing, the surface course and intermediate course layers were separated. All loose material that had been broken off the slab samples was discarded. The surface treatment that was attached to the bottom of the intermediate course and the fuel-resistant sealer on the surface course were removed and discarded before the evaluation.

The first step in evaluating the in-place material was to determine the field density of the AC layers. Because of the condition of the samples, cores could be taken only from one slab sample (S-4). For the other three samples, segments or chunks of the material were weighed in air and water to determine density values. Field density values for the surface course and intermediate course are presented in Tables 1 and 2, respectively.

The next step in preparing the AC material was to trim and remove all cut edges from the samples. This was accomplished by heating the cut edges and removing at least ¾ in. of material with a hot spatula. This procedure was performed to ensure that the aggregate gradation was not affected by the sampling technique and that a true representative sample was evaluated. After this preparation was completed, the materials representing each of the eight samples were tested.

Laboratory Tests

Of the eight samples evaluated, four were surface course materials and four were intermediate course materials. A complete evaluation of each sample included extractions, asphalt

TABLE 1 SURFACE COURSE FIELD DENSITY ANALYSIS

Sample	No.	Thickness (in)	Specific Gravity	Density (pcf)
S-1	1	2 1/4	2.237	139.6
	2	2 1/4	2.275	141.9
	3	2 1/4	2.274	141.9
	AVG	2 1/4	2.261	141.1
S-2	1	2 1/8	2.196	137.0
	2	2 1/4	2.154	134.4
	3	2 1/8	2.220	138.5
	AVG	2 1/8	2.190	136.6
S-3	1	2 1/2	2.289	142.8
	2	2 1/2	2.330	145.4
	3	2 1/2	2.308	144.0
	AVG	2 1/2	2.309	144.1
S-4	1	2 1/2	2.375	148.2
	2	2 1/2	2.359	147.2
	AVG	2 1/2	2.367	147.7

TABLE 2 INTERMEDIATE COURSE FIELD DENSITY ANALYSIS

Sample	No.	Thickness (in)	Specific Gravity	Density (pcf)
S-1	1	2 3/4	2.230	139.2
	2	2 3/4	2.224	138.8
	3	2 3/4	2.235	139.5
	AVG	2 3/4	2.230	139.2
S-2	1	2	2.201	137.3
	2	2	2.224	138.8
	3	2	2.231	139.2
	AVG	2	2.218	138.4
S-3	1	3	2.247	140.2
	2	3	2.261	141.1
	3	3	2.249	140.3
	AVG	3	2.252	140.5
S-4	1	2 1/2	2.299	143.4
	2	2 1/2	2.292	143.0
	AVG	2 1/2	2.295	143.2

recoveries, and recompaction studies. Four asphalt extractions (ASTM D2172), two aggregate gradations (ASTM C136 and C117), and one Abson recovery (ASTM D 1856) were conducted on each sample.

Extractions and recoveries were run on prepared material from each sample. Technical-grade solvents and a two-stage extraction procedure using a high-speed centrifuge were used to optimize the results of the procedure. The aggregates obtained from this extraction procedure were used to run aggregate gradation, specific gravity, fractured face, and natural sand count tests. The results of these tests are summarized in Tables 3 and 4. The asphalt cements recovered from the Abson recovery procedure were used to run the penetration, viscosity, specific gravity, and ductility tests. The results of these tests are presented in Table 5. The aggregate gradations from the in-place material are compared with the specified gradation band and the job mix formula (JMF) supplied by the contractor in Figures 1–8.

The remaining AC material for each sample was then used for a recompaction study. This material was reheated to approximately 250°F and used to recompact 7 Marshall specimens by applying 75 blows on each side with a hand hammer. Two additional samples were compacted using the Corps of Engineers gyratory testing machine (GTM) using 200 psi, 30 revolutions, and 1-degree gyration angle, which is equivalent to a 75-blow hand-hammer compactive effort. This gyratory compaction was used to check for flushing of the specimen, which indicates excess asphalt cement in the mix or an unstable mix. Of the seven recompacted Marshall specimens, four were used to run the standard Marshall mix test (MIL-620A, Method 100), and three were used to run the retained stability Marshall mix test (MIL-STD 620A, Method 104). The results of the recompaction study and Marshall mix tests are presented in Tables 6 and 7.

DISCUSSION OF LABORATORY RESULTS

Field Density

The field density results presented in Tables 1, 2, 6, and 7 show inconsistent results between cored samples and chunk samples. The percent compaction values were determined using the field density and recompacted density values. The average field compaction for cored samples (S-4) for both the surface course and intermediate course material is above the specified 98 percent minimum compaction requirement. The average field compaction for the chunk samples (S-1, S-2, S-3) is below the minimum compaction requirement. The compaction values for the chunk samples ranged from 94.3 to 96.9 percent. Because field core specimens, which are more reliable than chunk samples, were not available for all samples, a true indication of the in-place density could not be determined.

Aggregate Analysis

The sieve analysis results presented in Tables 3 and 4 and shown in Figures 1–8 indicate that all samples have aggregate gradations that do not meet specifications. The primary prob-

TABLE 3 SURFACE COURSE AGGREGATE ANALYSIS

Sieve Size	Specified Limits*	JMF	S-1	S-2	S-3	S-4
1 in.	---	---	100	100	100	100
$^3/_4$ in.	100	100	93.5	97.4	98.6	97.7
$^1/_2$ in.	82-96	95.3	84.4	90.7	91.9	91.7
$^3/_8$ in.	75-89	77.7	75.4	81.2	83.9	81.7
No. 4	59-73	58.1	52.8	56.1	61.6	58.1
No. 8	46-60	57.4	44.1	49.0	48.7	44.4
No. 16	34-48	50.0	40.5	45.5	44.2	39.1
No. 30	24-38	39.0	33.1	35.1	36.0	31.6
No. 50	15-27	16.0	15.0	12.9	14.6	14.4
No. 100	8-18	8.0	6.8	4.3	6.2	6.7
No. 200	3-6	6.0	4.8	2.7	4.2	4.9
% Fractured Faces (Coarse)			90.8	93.1	97.0	95.9
(Fine)			100	99.6	99.8	98.8
Natural Sand Count (%)			35.4	40.9	38.5	34.4
Specific Gravity (+ No. 4)			2.73	2.69	2.74	2.72
(- No. 4)			2.54	2.56	2.63	2.61

* Percent passing

TABLE 4 INTERMEDIATE COURSE AGGREGATE ANALYSIS

Sieve Size	Specified Limits*	JMF	S-1	S-2	S-3	S-4
1 in.	---	---	100	100	100	100
$^3/_4$ in.	100	100	96.6	93.2	96.8	96.7
$^1/_2$ in.	73-91	83.1	84.5	81.7	88.1	87.0
$^3/_8$ in.	63-81	63.4	76.1	70.4	78.2	77.6
No. 4	45-63	42.3	53.4	47.5	54.2	55.5
No. 8	32-50	41.9	43.4	38.9	43.0	44.1
No. 16	23-41	39.4	41.0	35.9	38.4	40.3
No. 30	15-33	29.6	33.4	29.8	29.8	33.0
No. 50	10-24	14.0	10.8	12.4	11.3	13.7
No. 100	7-17	8.6	3.7	6.0	3.6	6.1
No. 200	3-7	6.8	2.6	4.4	2.2	4.5
% Fractured Faces (Coarse)			83.1	86.8	92.4	94.1
(Fine)			98.3	99.6	100	100
Natural Sand Count (%)			37.2	30.7	34.2	35.5
Specific Gravity (+ No. 4)			2.67	2.63	2.65	2.67
(- No. 4)			2.54	2.63	2.63	2.62

* Percent passing

TABLE 5 RECOVERED ASPHALT CEMENT ANALYSIS

Test	S-1	S-2	S-3	S-4
		Surface Course		
Penetration (100g, 5sec, 77°F)	37	29	32	43
Viscosity (abs, 140°F, P)	7439	16,495	6287	4059
Viscosity (kin, 275°F, cSt)	690	989	649	538
Specific Gravity	1.036	1.036	1.036	1.033
Ductility (5 cm/min, 77°F, cm)	54	---	68	---
		Intermediate Course		
Penetration (100g, 5sec, 77°F)	32	28	35	37
Viscosity (abs, 140°F, P)	15,036	30,417	5731	8634
Viscosity (kin, 275°F, cSt)	902	1155	618	761
Specific Gravity	1.033	1.030	1.036	1.036

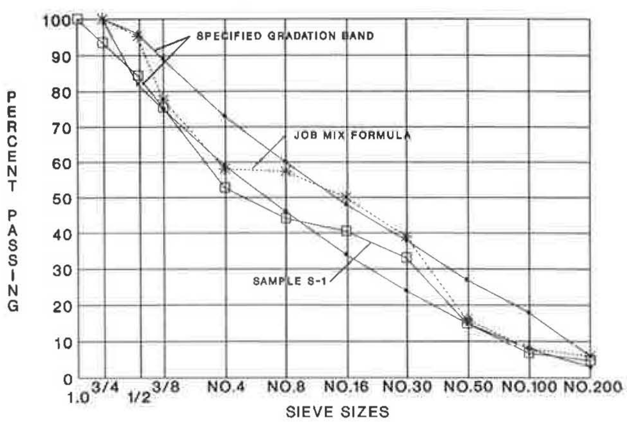

FIGURE 1 Surface course aggregate gradation (Sample S-1).

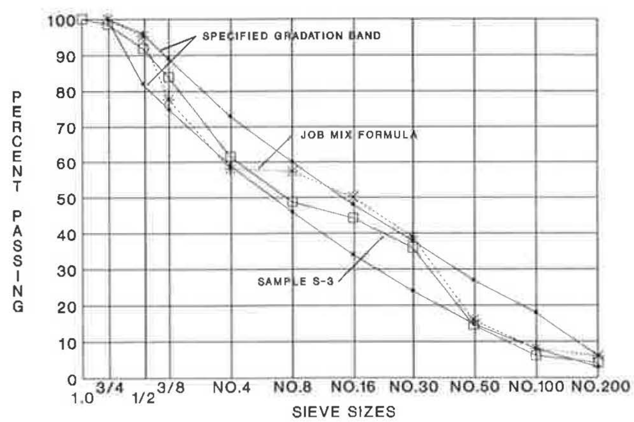

FIGURE 3 Surface course aggregate gradation (Sample S-3).

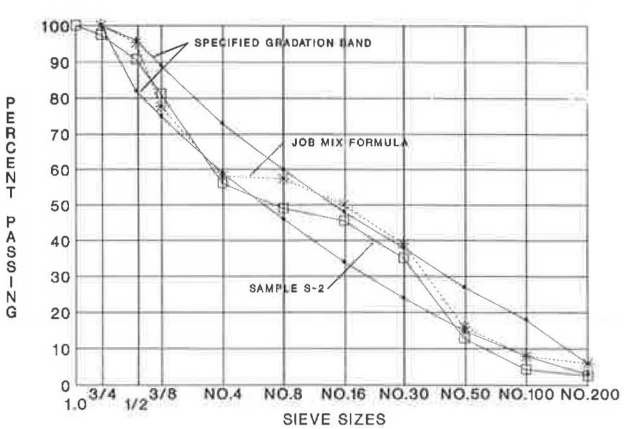

FIGURE 2 Surface course aggregate gradation (Sample S-2).

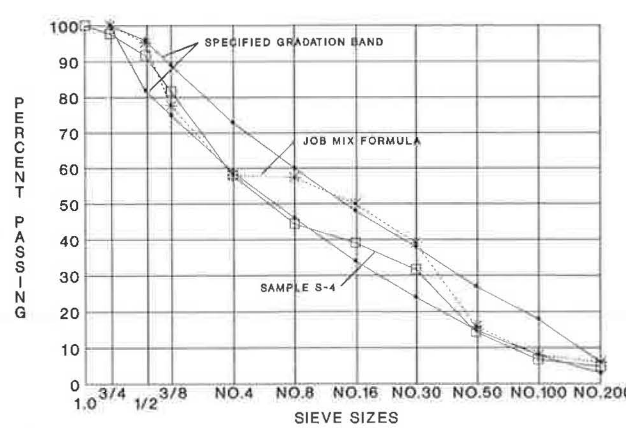

FIGURE 4 Surface course aggregate gradation (Sample S-4).

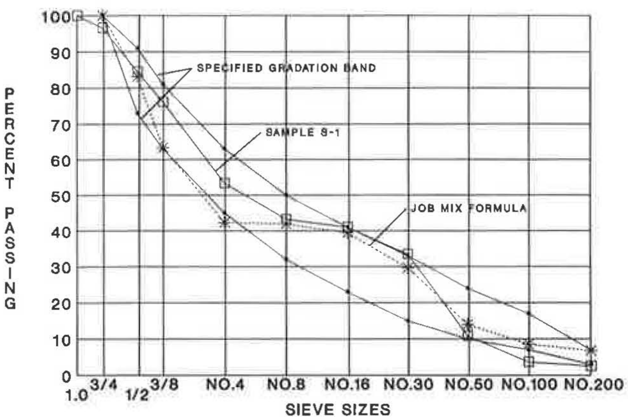

FIGURE 5 Intermediate course aggregate gradation (Sample S-1).

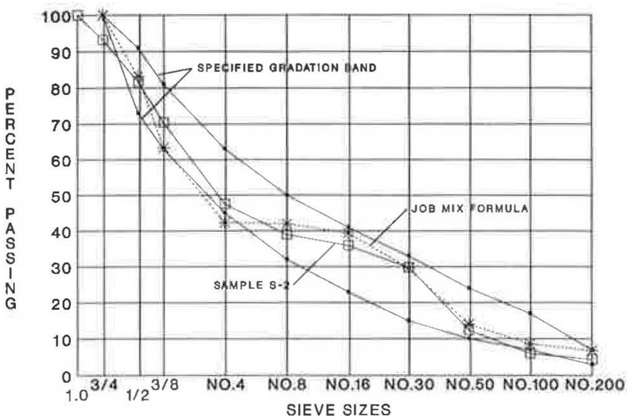

FIGURE 6 Intermediate course aggregate gradation (Sample S-2).

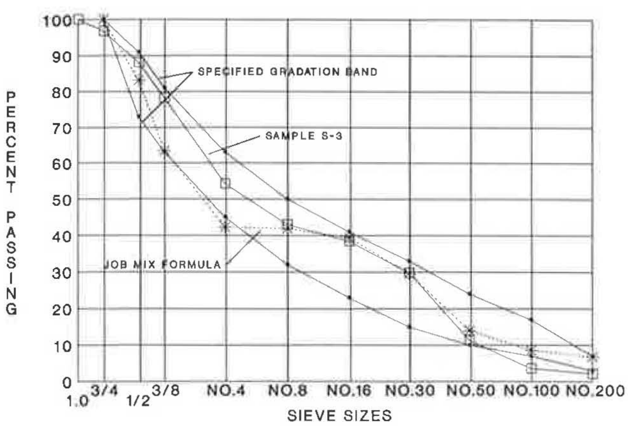

FIGURE 7 Intermediate course aggregate gradation (Sample S-3).

lem with the aggregate gradation for both the surface course and intermediate course mixtures is that the gradations are gap graded instead of well or dense graded. Normal gradations for high-tire-pressure pavements are dense graded and do not vary from the upper to lower limits of the specified gradation limits. AC mixtures that have gap-graded gradations generally tend to be less stable than dense-graded materials.

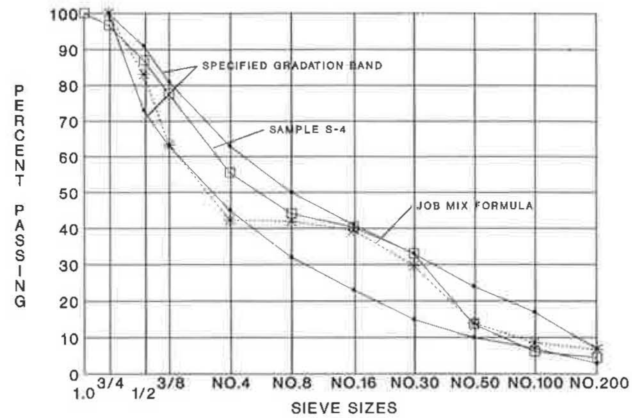

FIGURE 8 Intermediate course aggregate gradation (Sample S-4).

The sieve analyses for the surface course material indicate that coarser-than-specified material (plus ¾ in.) is in the AC mixture and that there is a shortage of material passing the No. 4, No. 8, No. 50, and No. 100 sieves. The sieve analyses for the intermediate course also indicate that coarser material has been added to the mixture and that there is a shortage of No. 100 material. The shortage of the finer-sized material will decrease the stability of the AC. As a whole, the aggregate gradations are not acceptable for heavy-duty airfield pavements.

The natural sand content was determined by visually observing the aggregate particles smaller than the No. 4 sieve under a microscope. The percentage of natural sand was calculated by determining the number of sand particles versus crushed aggregate particles.

The amount of natural sand in the aggregate gradation was extremely high. The natural sand count for the surface course material ranges from 34.4 to 40.9 percent, whereas the intermediate course gradation has sand counts between 30.7 and 37.2 percent. The maximum amount of natural sand allowed by the contract specifications is 15 percent. Too much natural sand is a primary cause of unstable or tender AC mixes.

Asphalt Cement Analysis

The test results for the recovered asphalt cement are presented in Table 5 and indicate that this material has typical values for a 60 to 70 pen asphalt cement that has been recovered from an AC mixture. The recovered penetration of the asphalt cement varied from 29 to 43 for the surface course and 28 to 37 for the intermediate course. The typical initial percent loss for penetration values in an AC mixture is 40 to 50 percent. These penetration values are in or near that range. The ductility test was also conducted on recovered asphalt cement from two surface course samples; the results were 54 to 68 cm. Both of these values exceed the minimum ductility requirement for an aged asphalt cement (ASTM D 946).

AC Mixture Analysis

The laboratory test results for the recompacted AC mixtures are presented in Tables 6 and 7. The results indicate that the

TABLE 6 SURFACE COURSE MIXTURE ANALYSIS

	Specs (JMF)	S-1	S-2	S-3	S-4
Asphalt Content (%)	4.8	4.8	5.1	5.2	5.3
Stability (lbs)	1800 min	1892	1301	1842	1855
Flow (0.01 in)	16 max	9	10	9	10
Voids Total Mix (%)	3 - 5	2.0	4.4	3.2	2.2
Voids Filled (%)	70 - 80	84.7	72.2	78.9	84.8
Retained Stability (%)	75 min	85.1	100	78.1	85.9
Gyratory Flushing	---	YES	YES	YES	YES
Recompacted Density (pcf)	---	149.7	144.8	149.1	149.5
Theoretical Density (pcf)	---	152.6	151.4	154.1	152.9
Field Density (pcf)	---	141.1	136.6	144.1	147.7
Percent Compaction (%)	98 min	94.3	94.3	96.7	98.8

TABLE 7 INTERMEDIATE COURSE MIXTURE ANALYSIS

	Specs (JMF)	S-1	S-2	S-3	S-4
Asphalt Content (%)	4.75	5.0	4.2	4.8	4.7
Stability (lbs)	1800 min	1373	2186	1334	1909
Flow (0.01 in)	16 max	9	9	9	9
Voids Total Mix (%)	5 - 7	4.7	5.8	5.4	5.9
Voids Filled (%)	50 - 70	70.3	62.1	66.7	64.2
Retained Stability (%)	75 min	81.4	97.5	81.2	77.6
Gyratory Flushing	---	YES	YES	YES	YES
Recompacted Density (pcf)	---	143.7	145.2	145.0	144.6
Theoretical Density (pcf)	---	150.8	154.1	153.3	153.7
Field Density (pcf)	---	139.2	138.4	140.5	143.2
Percent Compaction (%)	98 min	96.9	95.3	96.9	99.0

AC mixtures are inconsistent and do not meet specifications in many instances. The aggregate blends of all samples have a water absorption below 2.5 percent and are therefore considered nonabsorptive. Each surface course sample has at least one AC mixture property requirement that is not met. As mentioned previously, all aggregate gradations are out of specification. Samples S-1 and S-4 have low voids in the total mix (2.0 and 2.2 percent, respectively) and high voids filled with asphalt (84.7 and 84.8 percent, respectively). The asphalt content values are generally higher than the JMF recommended value, especially in samples S-3 and S-4. The stability value for Sample S-2 is extremely low and unacceptable. The reported stability values for samples S-1, S-3, and S-4 are above the minimum 1,800 lb, but these values are misleading. Recompaction stability values are usually higher than the stability of the mixture when it was placed because the asphalt material has been reheated, causing the asphalt cement to harden. The hardened asphalt material causes the stability values to be high. On the basis of these data, the surface course material would not meet the requirements of the specifications and was not suitable for an airfield pavement.

The Marshall properties of the intermediate course were acceptable, except for the stability values. Samples S-1 and S-3 had low stability values—1,373 and 1,333 lb. Samples S-2 and S-4 were above the 1,800-lb minimum requirement, but because these samples had been heated and reheated, the stability values were considered to be higher than the actual stability value when it was produced. On the basis of these data, the intermediate course material would not meet the requirements of the specifications.

INSPECTION OF CONSTRUCTION SITE

During February 1990, MEAPO requested technical assistance during the repair and continued construction of the airfield pavement. The purpose of the visit was to monitor the production and construction of an additional portion of the airfield pavement. MEAPO requested that a complete inspection be conducted of the quarry, asphalt plant, and testing laboratory.

The aggregate quarry site was visited in order to observe the crushing operation. The material being crushed was similar to a pit-run gravel that was excavated from the existing terrain by a front-end loader. The size of the unprocessed material ranged from small boulders to fine aggregates. Approximately 50 percent of the material being processed was estimated to be minus No. 4 material (natural sand). Located at the beginning of the crushing operation was a grizzly, a device designed to discard all material smaller than 3 in. The grizzly was not functioning properly because it was partially blocked, and the feed rate was too high for the short 6-ft grizzly. Both of these problems allowed uncrushed minus 3-in. material to pass through the crushing operation without being fractured. All stockpiles contained uncrushed particles. It was estimated that between 20 and 40 percent of the material being processed was smaller than the No. 4 sieve and was not crushed. It was suggested that the minus No. 4 material be removed from the aggregate before crushing to ensure that natural sand and uncrushed materials were not contaminating the crushed stockpile. This could be accomplished by screening the material before placing it in the jaw crusher or ensuring that the grizzly functioned properly.

The asphalt plant was visited in order to observe plant operations. The asphalt plant was a Barber Green batch plant that was approximately 2 years old and had a capacity of 4 tons per batch. The aggregate stockpiles at the asphalt plant were contaminated with fine material. The handling of these materials was inconsistent with good stockpile management methods. The loader operator was picking up natural fine material off the underlying ground surface and mixing this material with the aggregate stockpiles. The cold feed bins were being overfilled by the loader operator, which resulted in material spilling over into the adjacent bins. The AC material was produced at an extremely high mixing temperature of 350°F.

The testing laboratory was also inspected. The field laboratory was located adjacent to the parking apron. The laboratory was set up to run gradations, asphalt contents, and Marshall tests. The Marshall stability and flow tests were observed, along with the compaction of several samples. The laboratory compaction of the AC material was normally conducted with a mechanical hammer that had not been calibrated to correlate with a hand hammer. Marshall specimens were compacted according to MIL-STD 620 with a hand hammer during this visit. The main problem observed with AC testing was the placement of the uncompacted AC mixture in the oven, where the material was reheated for several hours before laboratory compaction.

LABORATORY ANALYSIS ON EFFECT OF HEATING AC SPECIMENS

During this entire project, laboratory data from the field laboratory indicated that the stability of the AC mixtures was acceptable, whereas the stability values determined at WES were always lower than the field laboratory results. After observing no major errors in the field laboratory operations, it was determined that the differences in stability values were caused by the difference in heating and mixing of the AC material. Standard laboratory mixing and compaction temperatures used at WES were lower than the temperatures used to produce the AC material at the asphalt plant.

The sensitivity of the original asphalt cement had been questioned because the laboratory tests indicated that 60 to 70 pen asphalt did not meet the requirements of ASTM D946. The asphalt cement testing indicated that this material had the potential to lose lighter fractions. The asphalt cement had a large weight loss and a large decrease in penetration after the thin film oven test. This indicated that the material had a tendency to harden significantly when subjected to heat.

A laboratory study was conducted to determine the effect of excessive heating by leaving the AC material in the oven for an extended time before compaction. Several asphalt mixtures consisting of labstock limestone aggregate and the original 60 to 70 pen asphalt cement from Egypt were mixed at 250°F and 350°F and stored in an oven at each temperature before compaction. The cure time in the oven before compaction varied from 0 min to 24 hr. The results of this study are presented in Table 8 and shown in Figures 9 and 10. The stability values increased tremendously with extended time in the oven, especially at the higher temperature. The increase in stability at 250°F was 35.3 percent at 4 hr and 92.9 percent at 24 hr. The increase in stability at 350°F was 288.4 percent at 4 hr. This increase in stability indicated that the asphalt cement was sensitive to heat and age hardening.

SUMMARY

The performance of the AC has been unacceptable: normal aircraft traffic has caused depressions and deformation on the parking apron. On the basis of the test results, an evaluation of the mix designs, and an inspection of the construction site, it appears that the poor performance of the AC was due to an improperly designed and produced AC mixture. Several factors that contributed to this improper mixture are discussed below.

- Aggregate gradation: the aggregate gradations were consistently out of specification and were gap graded. Gap-graded aggregate gradations should not be used for heavy-duty airfield pavements because these materials are less stable and have the potential to rut and deform.
- Natural sand: the amount of natural sand in both the surface course and intermediate course materials was extremely high. Tender mixes often result from the use of an excessive amount of natural sand. The excessive amount of rounded sand particles acts like ball bearings, causing the mixture to be unstable. The excess natural sand is a major contributor to the instability of these mixtures.
- Mix designs: the JMF for both the surface-course and intermediate-course mixtures did not meet specifications. The aggregate gradations for both mixtures did not meet the specified limits of the contract. The amount of natural sand used in the JMF for the surface course material was 52.5 percent. Thirty-five percent natural sand was used in the JMF for the

TABLE 8 EFFECT OF HEATING ASPHALT CONCRETE SPECIMENS

Time (min)	Stability 250 F * (lbs)	Percent Increase (%)	Stability 350 F * (lbs)	Percent Increase (%)
0	2184	---	2107	---
30	2181	---	---	---
60 (1HR)	2143	---	3069	45.6
120 (2HR)	2658	21.7	4633	219.9
240 (4HR)	2954	35.3	6076	288.4
1440 (24HR)	4213	92.9	---	---

* ASPHALT CONCRETE SPECIMENS WERE MIXED AND CURED IN OVEN PRIOR TO COMPACTION AT 250 F AND 350 F. 75 BLOW HAND HAMMER WAS USED TO COMPACT ALL SPECIMENS.

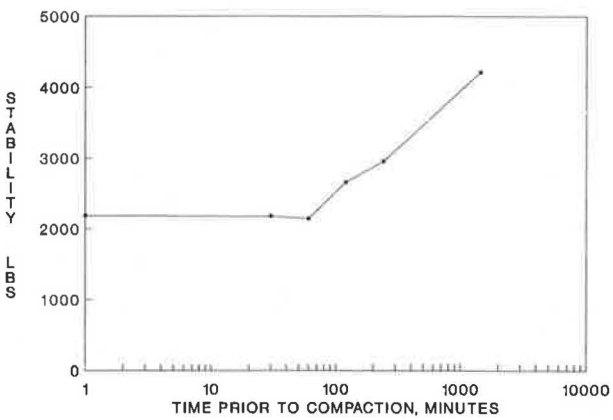

FIGURE 9 Cure time versus Marshall stability (250°F compaction temperature).

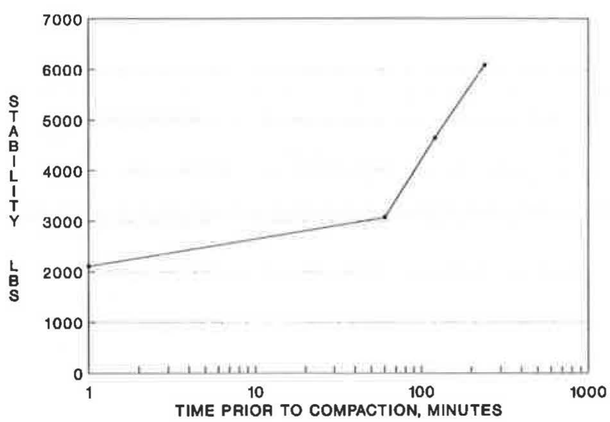

FIGURE 10 Cure time versus Marshall stability (350°F compaction temperature).

intermediate course. Both of these values exceed the limit specified in the contract specifications. The mix designs produced for this project are not acceptable for a heavy-duty AC pavement.

- Asphalt cement: the original asphalt cement tested and evaluated from these samples indicates that this material was sensitive but was not the cause of the pavement deformation. However, the 60 to 70 pen asphalt cement was sensitive to heat and was affected significantly when exposed to high temperatures for an extended amount of time. The hardening of this asphalt cement increased the stability values tremendously and produced misleading field laboratory results.
- Site visit: the aggregate quarry and asphalt plant were operating in an insufficient manner to produce high-quality materials for an airfield pavement. The quarry was not functioning properly, and a large percentage of the uncrushed material was allowed to pass through the crushing operation, thus contaminating the crushed stockpiles. The handling procedures at the asphalt plant further contaminated the stockpiles through the use of improper stockpile management methods.

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Polishing of Aggregates and Wet-Weather Accident Rates for Flexible Pavements

PODURU M. GANDHI, BENJAMIN COLUCCI, AND SRINIVAS P. GANDHI

A high wet-weather accident rate on highways is a general indication of the low skid-resistance of the pavement surface. The polishing of aggregates under the vehicle tires decreases the microtexture, which is an important component of the skid resistance of the pavement. The rate of polishing depends on the mineralogical characteristics and the texture of the aggregates. A detailed laboratory study was conducted on different aggregate sources in Puerto Rico and their polishing characteristics by means of a British polishing wheel. It was found that carbonate rocks polish more than gravels and noncarbonate rocks. In a follow-up study, field data were collected on accident rates, pavement surface friction, and macrotexture of in-service flexible pavements. Samples also were collected from selected pavements to determine polish resistance and carbonate content of the recovered aggregates. Statistical analyses indicated that the relationship between the wet/dry accident ratio and pavement surface friction was statistically significant. Carbonate content of the aggregate showed better correlation with pavement surface friction than polished stone values, although both had significant influence. On the basis of the results, recommendations were made with respect to specifications of aggregates to ensure adequate skid resistance.

The high rate of accidents on highways in the United States and Puerto Rico is of concern to the driving public as well as to highway engineers. The costs of accidents run into billions of dollars in addition to loss of life. The Highway Safety Act passed by the United States Congress in 1966 requires that every state have a program of highway design, construction, and maintenance to increase highway safety (*1*). Although accidents result from many causes that involve vehicle, driver, pavement, and their interaction, the highway engineer is responsible for building as much safety as possible into the pavement. A significant portion of the total number of accidents is caused by the skidding of vehicles on wet pavement surfaces.

The research reported here is related to the improvement of skid resistance of pavements through the use of polish-resistant aggregates. It was conducted in two phases (Figure 1). In the first phase, aggregate samples were collected from a number of quarries in Puerto Rico, and their polishing characteristics and other properties were evaluated (*2*). In the second phase, information was collected on accident experience on highways, pavement surface friction, and macrotexture, as well as properties of aggregates, such as polish resistance and acid solubility (*3*). On the basis of the study, recommendations were made regarding specifications for aggregates used in asphalt pavements to ensure adequate skid resistance.

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REVIEW OF PHASE 1 RESEARCH

Research during the first phase consisted of an island-wide inventory of aggregate sources and laboratory evaluations of physical and mineralogical characteristics of aggregates. The details are described in the following sections.

Aggregate Inventory

A field sampling team of two technicians and a geologist from the Puerto Rico Highway Authority was organized. The geologist visited most of the quarries and selected representative samples of rocks from which the aggregates are processed. In addition, the team collected about 50 lb of ¾- to ¼-in. representative aggregate samples from the stockpiles of the quarries. A total of 55 samples was collected from various parts of the island. These samples represent the active quarries and gravel extraction sites on the island that supply materials for road and bridge construction.

Laboratory Evaluations

The laboratory tests on each aggregate sample included specific gravity and absorption (ASTM C127), Los Angeles abrasion test (ASTM C131), acid insoluble residue (ASTM D3042), and polish value (ASTM D3319 and ASTM E303). The polish test was conducted in two parts. Initially, the stone samples were subjected to accelerated polishing by means of the British wheel. The state of polish reached by each sample was then evaluated by measuring the surface friction with the British Portable Tester. The result was expressed as polished-stone value (PSV) instead of polish value (PV), as specified in the ASTM procedure, because the auxiliary scale was used to correct for the smaller rubber slider. The auxiliary scale represents the actual friction value, and PSV refers to the readings from this scale.

Geological Evaluations

The following information was obtained for each rock sample:

- Macroscopic description (color, texture, hardness, and rough visual identification of the rock);
- Microscopic description (petrographic analysis of thin sections);
- Mineral composition (relative amounts of different minerals observed); and
- Rock name (identification of rock).

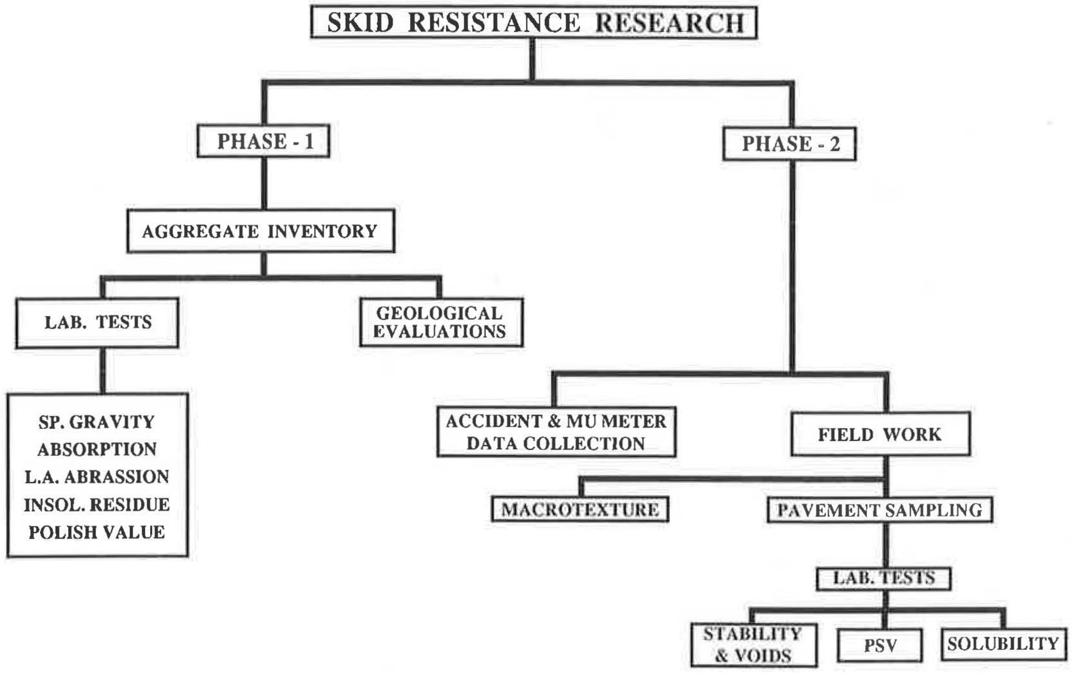

FIGURE 1 Flow chart of research activities.

Detailed tables of this information are available elsewhere (*2*). Aggregates in this study fall into two general categories: igneous and sedimentary. The igneous rocks, including those in gravels of alluvial deposits, consisted of volcanic breccia (mostly andesitic), andesite, volcanic sandstone, volcanic silt stone, granodiorite, granophyre, and diorite. The sedimentary rocks were mostly limestone except for one sample each of calcareous mudstone, conglomeratic mudstone, and mudstone.

River gravels were generally heterogeneous, with several types of rock in each sample. It was not possible to make thin sections of all rock types in the gravel samples. Even in the samples that were analyzed, only two or three thin sections of the most abundant rock types present in the sample were made.

PSVs

The PSVs in the 55 aggregate samples tested ranged from 39 to 59. The aggregates were classified into the following three groups: (*a*) carbonate rocks (22 samples), (*b*) noncarbonate rocks (13 samples), and (*c*) river gravels (20 samples). The term "gravel" is mainly a particle-size classification that may have different types of rocks and minerals. In this study, all gravels were from river beds and contained only noncarbonate rocks. The distribution of PSVs for each group is shown in Figure 2. Noncarbonate rocks and gravels as a group had higher mean PSVs than did carbonate rocks. Statistical analyses showed that there are significant differences in PSVs between these three groups of aggregates. The mean PSVs of noncarbonate rocks and gravels were found to be significantly higher than those of carbonate rocks, but the differences between noncarbonate rocks and gravels were not significant.

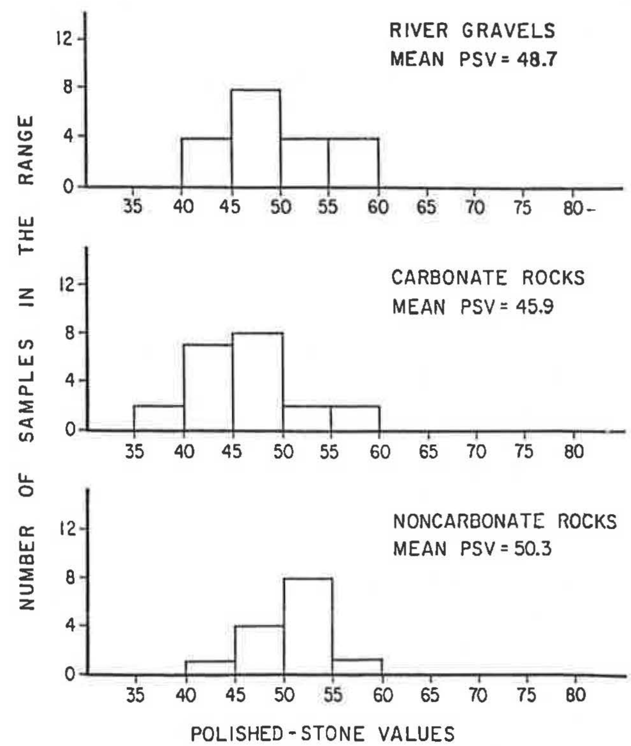

FIGURE 2 Distribution of PSVs in the three groups of aggregates.

Effect of Blending on PSVs

Tests also were conducted to study the effect of blending of aggregates with different polish characteristics. Five blends were tested. For each blend, two aggregate samples with high and low polish values were selected from nearby locations.

Specimens for the polish test were prepared by arranging the particles of the two aggregates alternately, using approximately 50 percent of each. The data obtained from these specimens are presented in Table 1. In general, it can be seen that the PV of the blend is much higher than the value of the softer component. Statistical analysis showed that this improvement is significant at a confidence level of 1 percent. This indicates the possibility of the use of some of the softer rocks in combination with others, instead of outright rejection of them. However, these findings are preliminary, and performance of these blends must be verified by experimental test sections.

Effect of Mineral Composition on PSVs

Results of this study showed that polishing of aggregates does not depend entirely on mineral composition but that other factors, such as texture of the rock (grain size, shape, and grain-to-grain relationship), degree of alteration, cementation, nature of cementitious material, nature of impurities present, porosity, and the like, can have considerable influence on polishing.

The following general observations can be made on the basis of mineralogical analysis:

- Among noncarbonate rocks, breccias, consisting of angular, coarse, usually heterogeneous rock and mineral fragments, showed high polish resistance with PSVs of 50 or higher. Andesites had PSVs close to 50. Serpentine showed a low PSV of 41, which was expected because it is a soft mineral.
- Several crushed river gravels had PSVs of 50 or higher, which can be attributed mainly to their heterogenous character and the rough surface texture of the crushed particles. However, some crushed gravels that had smooth textured particles or soft minerals had PSVs much lower than 50.
- Hard mineral particles embedded in a softer matrix can produce high polish resistance. As the matrix wears out, the relatively hard particles projecting up from the surface give high frictional resistance.
- Most of the carbonate rocks in this study were pure limestones. However, a wide variation exists in the nature of polishing of these aggregates. Dense limestones showed low PSVs, whereas porous limestones generally showed higher values. The porosity imparts a rough texture to the rock as it wears out.

On the basis of this research, the Puerto Rico Highway Authority specified a minimum PSV of 48 for aggregates used in surface courses of asphalt pavements. However, it was found later that it was difficult to get aggregates with a PSV of 48 or higher in some parts of the island. The Highway Authority became interested in determining whether the history of accidents on highways in Puerto Rico justifies this value. This led to the second phase of the research, which involved collection of accident data and pavement friction data on in-service pavements.

PHASE 2 RESEARCH

The principal objective of the second phase of research was to determine the relationship between the accident rate on

TABLE 1 POLISH DATA FOR UNBLENDED AND BLENDED SAMPLES

Sample No.	Classification	Location	PSV
12	Mudstone	Hormigueros	59
14	Limestone	Cabo Rojo	39
12-14	Blended		56
5	Andesitic Breccia	Bayamón	54
8	Limestone	Dorado	44
5-8	Blended		54
19	Gravel	Ponce	52
54	Limestone	Ponce	42
19-54	Blended		46
28	Breccia	Carolina	56
29	Gravel	Carolina	42
28-29	Blended		52
38	Gravel	Arecibo-Utuado	53
39	Limestone	Arecibo-Lares	41
38-39	Blended		52

wet pavements and pavement friction and to recommend aggregate specifications to control skidding on flexible pavements. The research tasks included collection of accident and pavement friction data, macrotexture determination, and laboratory tests on aggregates recovered from samples of in-service pavements.

Accident Data Collection

The Traffic Safety Commission furnished data on accidents that occurred in 1986. The analysis was limited to pavement sections in rural areas. The specific fields of information that were analyzed included location, date, type and severity of accident, alignment, pavement condition, and type of traffic control. The dBase III Plus computer program was used for the sorting and preliminary analysis of the accident data. Reports were generated by pavement condition (wet or dry) and for pavements without other defects in order to determine pavement segments most likely to be associated with true skidding problems.

Pavement Friction Data

Pavement friction data were obtained from the Public Works Department. The department uses a mu-meter for the evaluation of pavement friction. The test is conducted over a wetted pavement surface at a constant speed of 40 mph. The current testing procedure provides a continuous graphical record of the friction coefficient along the length of the test section and prints an average value for the section, which is referred to as the mu (μ) value. Figure 3 shows the mu-meter in operation at one of the pavement test sites. The mu-meter data were one-time only readings for each lane of the section that were taken during 1987 and 1988. Some older data consisted of multiple readings for each lane. Recent data consisted of graphical output for each lane. From these data, a computerized mu-meter data base was developed using dBase III Plus. The accident and mu-meter data bases were merged for conducting the statistical analyses.

Field Data Collection

A total of 20 pavement sections was selected for the field work. The data included general information about the site, geometric characteristics, surface macrotexture, level of polishing of the surface aggregates, pavement overall condition, and information relevant to other features, such as number of entrances, fixed-object location, and skid marks in the vicinity of the site. The field work was concentrated on the measurements of the surface macrotexture and the cutting and removal of representative pavement samples. The macrotexture of the pavement sections was measured using the sand patch method (ASTM E 965-83). The sand patch test was performed on four locations along the wheelpath, and the average texture depth was determined. For getting enough material for subsequent laboratory tests a 2- × 2-ft pavement slab was cut from the wheelpath of each section (Figure 4).

Laboratory Test on Field Samples

The laboratory procedures performed on the pavement samples are described below.

British Pendulum Number of Slabs (BPN)

The British Pendulum was used to determine the average friction value of the 2- × 2-ft flexible-pavement slabs. This gave another measure of pavement friction in addition to the measurements from the mu-meter. BPN readings correspond to the wheelpath, whereas in the mu-meter test, both wheels cannot be kept simultaneously on the wheelpath. Also, it was not determined whether the operators of the mu-meter were keeping even one wheel on the wheelpath during the test run. A wooden base was prepared and filled with sand to serve as a foundation to the 2- × 2-ft pavement sample for BPN determination (Figure 5). The testing procedure consisted of taking readings at four positions with five repetitions at each position. The average BPN values for the 20 test sections are presented in Table 2 along with other data. The average BPN values ranged from 39 to 63 with a standard deviation of 5.5. Two-thirds of the test sections were in the range of 50 to 60.

FIGURE 3 Mu-meter in operation.

FIGURE 4 Typical pavement sample.

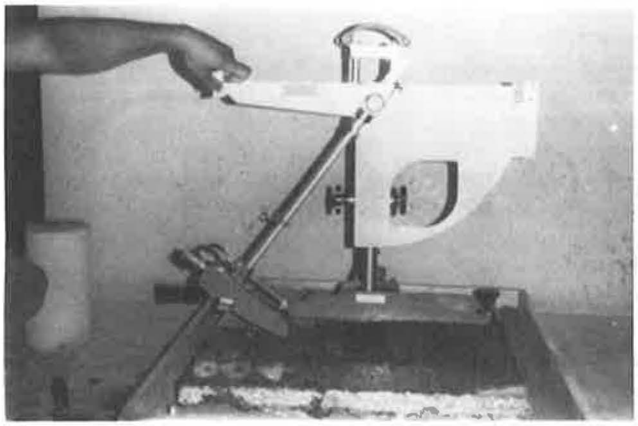

FIGURE 5 Testing pavement slab using British Portable Pendulum.

Mix Characterization Tests

Four cores (4 in. in diameter) were drilled from each of the 20 asphalt concrete blocks. The asphalt surface layer was separated from the cores and subjected to several characterization tests, such as Marshall stability, flow, voids, asphalt content, and aggregate gradation. The proportion of filler (passing the No. 200 sieve) and coarse fraction (retained on the No. 16 sieve) was determined from a gradation analysis.

PSV Determination

Coarse aggregate used in the mixes was recovered by extraction, and 4 test specimens (briquets) were prepared using the aggregate fraction passing the ⅜ in. sieve and retained on the No. 4 sieve. The aggregate particles were placed in the molds, and an epoxy resin was used for bonding the particles. Typical specimens are shown in Figure 6. The PSV was determined using the procedures explained earlier. The equipment used for polishing the specimens is shown in Figure 7, and the British Pendulum Tester used to determine the PSV is shown in Figure 8. The average PSV for the 20 pavement sections shown in Table 2 ranged from 40 to 55 with a standard deviation of 3.77. It may be noted that 75 percent of the test sections had a PSV of less than 48, which is the minimum value specified by the Puerto Rico Highway Authority for surface course mixes.

Acid Insoluble Residue Test

The acid insoluble residue test was performed on the recovered aggregate in accordance with ASTM D 3042-86, except that a 250 g sample was used instead of 500 g. Table 2 presents the percent of material soluble in hydrochloric acid for the 20 pavement test sections. The solubility ranged from 1.7 to 95.8 percent. In most samples, the insoluble material (residue) was river gravel, which was probably blended with the carbonate aggregates. Thus the amount of residue does not actually represent the residue within the carbonate aggregates. For this reason, instead of reporting the insoluble residue as the test requires, solubility of the aggregate in the acid, which indicates the percentage of carbonate rock in the sample, was calculated and reported as such.

TABLE 2 MU-METER AND LABORATORY DATA FOR PAVEMENT SECTIONS

Sec.	Average Mu	BPN	PSV	TD (in.)	Sol. (%)
01	26	53	41	0.02	95.8
02	44	53	48	0.02	13.8
03	46	62	49	0.03	24.4
04	20	53	42	0.02	83.2
05	25	53	47	0.01	37.1
06	20	39	43	0.02	75.0
07	54	63	55	0.03	1.7
08	28	57	45	0.01	61.7
09	30	56	45	0.02	76.7
10	40	53	44	0.02	30.1
11	50	60	49	0.06	13.9
12	42	56	52	0.02	1.9
13	53	55	45	0.01	1.9
14	38	54	40	0.01	16.1
15	49	49	45	0.04	11.4
16	33	61	53	0.05	5.1
17	46	55	46	0.02	5.3
18	44	53	46	0.02	4.6
19	44	58	44	0.03	1.7
20	46	45	46	0.01	3.1

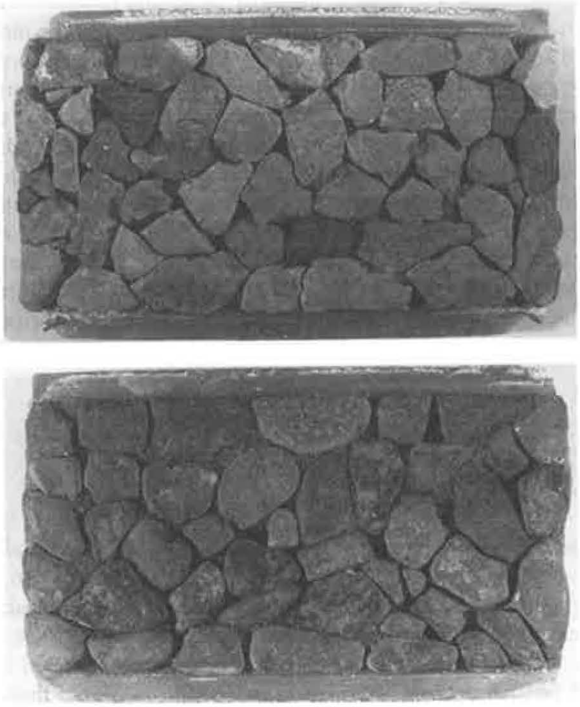

FIGURE 6 Typical specimens for polish test.

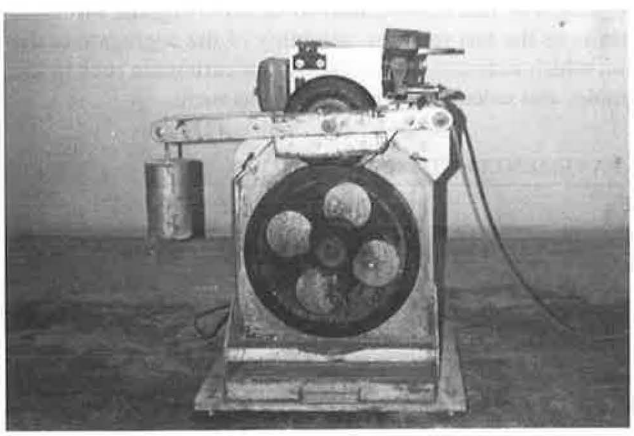

FIGURE 7 British accelerated polishing machine.

Statistical Analysis of Accident and Mu-Meter Data

Regression analysis were made to relate the wet-pavement accident rate, wet/dry (W/D), and wet/total (W/T) accident ratios to pavement surface friction values. The matching of accident and mu-meter data produced a set of 44 observations for these analyses. The independent variables considered were μ avg and μ min. The μ avg corresponded to the average μ value for all the lanes, and μ min is the lowest μ value among the lanes. Logarithmic transformations were also made on the independent variables μ avg and μ min. Furthermore, because the wet/dry accident ratio showed better response as a dependent variable, it was decided to pursue the statistical analysis with the wet/dry accident ratio as the dependent or response variable. As models having μ min as a variable were generally

FIGURE 8 PSV determination using the British pendulum.

better than μ avg, only models with μ min are reported below; μ min is referred to as μ hereafter. For each regression model, correlation coefficients (r) and p-values are shown in parentheses. The p-value represents the observed level of significance. If it is equal to or less than 0.01, the contribution by the variable to the model is considered highly significant.

By using μ or its transformations as the independent variable, the following models were obtained:

$$W/D = 1.0230 - 0.016\ \mu$$
$$(r = 0.504,\ p = 0.0006) \quad (1)$$

$$W/D = 2.559 - 1.366\ \text{Log}\ (\mu)$$
$$(r = 0.530,\ p = 0.003) \quad (2)$$

$$\text{Log}\ (1 + W/D) = 0.80 - 0.431\ \text{Log}\ (\mu)$$
$$(r = 0.585,\ p = 0.008) \quad (3)$$

The correlation coefficients were low because several other factors, which are difficult to quantify, affect accidents. However, the low p-values indicate that pavement surface friction has a significant influence. Several researchers (*4–6*) had reported considerable scatter in the data while relating accident rates to pavement surface friction.

Statistical Analysis of Laboratory Data

Accident Ratio Correlations

An attempt was made to statistically correlate the data obtained from the laboratory tests, such as BPN, PSV, and the acid solubility (Sol) to wet/dry (W/D) accident ratios. The resulting models are described below:

$$W/D = 0.192 + 0.006\ \text{Sol}$$
$$(r = 0.57,\ p = 0.011) \quad (4)$$

$$W/D = 2.128 - 0.038\ \text{PSV}$$
$$(r = 0.418,\ p = 0.075) \quad (5)$$

$$W/D = 0.499 - 5.54 \text{ TD}$$
$$(r = 0.228, p = 0.35) \tag{6}$$

$$W/D = 0.913 - 0.01 \text{ BPN}$$
$$(r = 0.163, p = 0.51) \tag{7}$$

The relationship between the wet/dry accident ratio and solubility was more significant compared with PSV. It was surprising that BPN correlated so poorly with the W/D ratio. It was expected that BPN values may be more representative of the wheelpaths than mu values because both mu-meter wheels cannot be placed on the wheelpaths.

Correlation of Mu Values to Pavement Characteristics

Regression models relating pavement surface friction to various characteristics such as BPN, texture depth (TD), and aggregate characteristics such as PSV and solubility were tried. Solubility gave the best correlation, as shown by the following models:

$$\mu = 47.034 - 0.288 \text{ Sol}$$
$$(r = 0.836, p = 0.0001) \tag{8}$$

$$\mu = -23.0 + 1.325 \text{ PSV}$$
$$(r = 0.472, p = 0.041) \tag{9}$$

Multiple regression models including TD in addition to PSV and solubility did not significantly improve the predictive significance of the models in this study. However, it can be seen from Tables 2 and 3 that the pavement surface friction values are generally greater than 40 if the texture depth is equal to or greater than 0.03 in.

Discussion of Data Analyses

The relationship between wet/dry accident ratios and mu values was found to be statistically significant, although correlation coefficients were not very high. This is because several other variables related to driver, vehicle, and weather conditions not included in the models may also have considerable influence. Also, accident data and mu-meter data may not be highly reliable because the data were obtained from existing records and the authors had no control on the data collection.

The analyses also showed that the carbonate content of aggregates as measured by acid solubility has a significant inverse or negative relationship to pavement surface friction. The models having solubility term generally gave higher correlation coefficients. Although the correlation coefficients for models incorporating PSV were not high, the models were statistically significant.

The determination of PSV is more complicated and time-consuming, and equipment problems cause even more delays. Hence it may be better to place more emphasis on the solubility test instead of the polish test and require polish tests only for such major highways as expressways.

An examination of Figure 9 relating mu values to solubility indicates that mu values are low (less than 40) when solubility is higher than about 25 percent. This suggests that for primary highways a limit of 25 percent of carbonate rock in the mix is a reasonable value to specify.

From Figure 10, it also can be seen that for a pavement surface to have a mu value of 40, which is generally specified, the aggregate PSV should be around 48; the Puerto Rico Highway Authority currently specifies this value. However, it must be pointed out that the correlation between value and PSV is not high in this study. Nevertheless, it can be inferred that the currently specified PSV of 48 is reasonable, especially for such high-speed facilities as expressways and for high-risk locations on major highways. However, a range of PSV in the specification is needed because of the scarcity of high PSV aggregates in some regions.

From the average values summarized in Table 3 it can be seen that the wet-pavement accident experience is almost three times higher for pavements that have aggregates with PSVs of less than 45. However, there is little difference between the accident ratios of pavements with PSVs of 45 to 47 and those with 48 or higher. This suggests that for most highways the PSV specification for aggregates can be reduced to 45 from the current 48. For expressways and high-risk areas it would be desirable to keep a minimum PSV of 48.

TABLE 3 RELATIONSHIP OF PSV TO ACCIDENT RATIOS, SOLUBILITY, AND MU VALUES

PSV Range	No. of Pavement Sections	Average Values for the Sections		
		W/D	Sol.(%)	μ
< 40	0	-	-	-
40-44	6	0.75	50	31
45-47	8	0.24	25	40
48 or higher	6	0.23	10	45

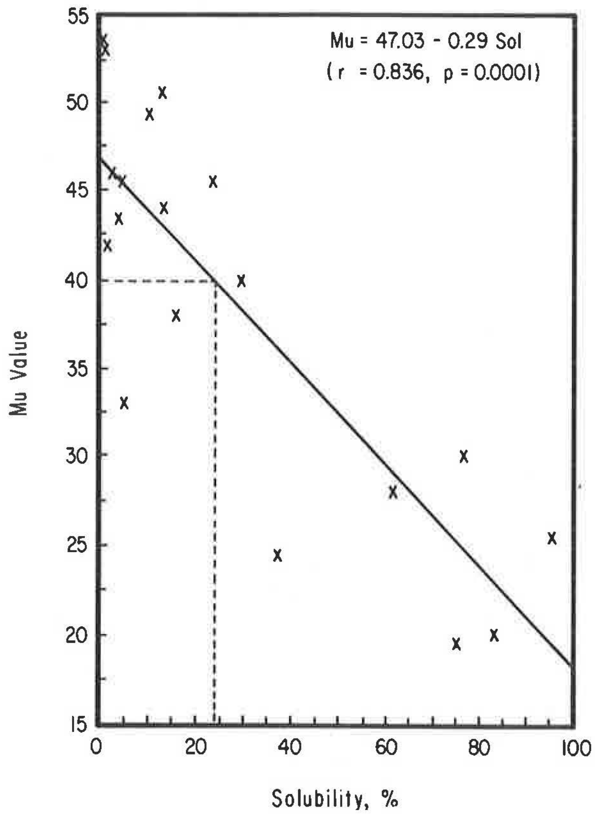

FIGURE 9 Relationship of mu to acid solubility.

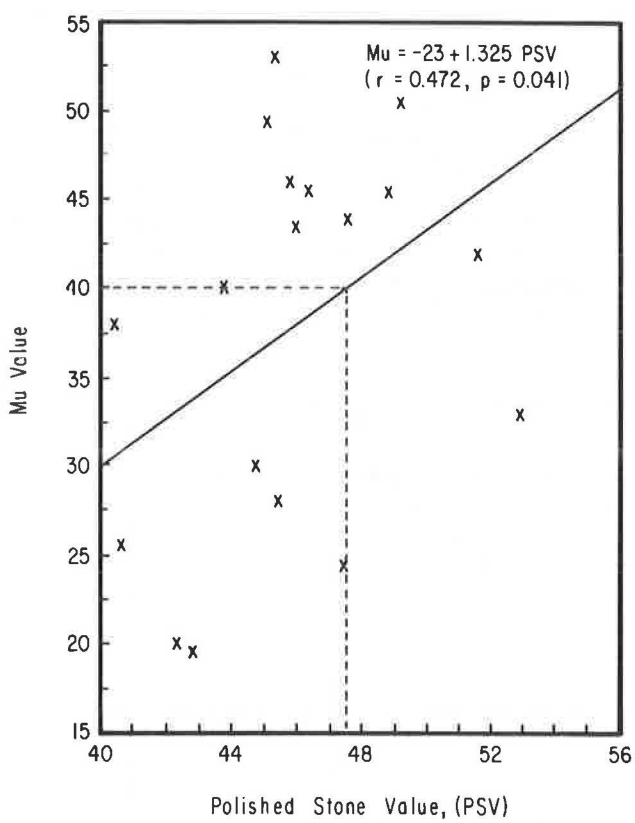

FIGURE 10 Relationship of mu to PSV.

It can be also seen from Tables 2 and 3 that aggregates that have acid solubility of 10 percent or less ensure higher pavement friction. However, reasonable pavement surface friction (mu value of 40) can be obtained if acid solubility does not exceed a value of 25. Values of acid solubility or carbonate content of aggregate that do not exceed 10 or 25 percent, depending on the highway, may be specified as alternatives to the polish value specification or in combination with it at the discretion of the highway authority.

These research findings are in accordance with the practices in the United States and several European countries. British specifications, for example, require the PSV of aggregates to be 40 to 75, depending on traffic and risk conditions, and a minimum texture depth of 0.04 in. (*7*). French specifications require a PSV of 45 or higher to classify an aggregate as good and a texture depth of 0.03 in. or higher for moderate- to high-speed facilities (*7*). Pennsylvania gives an excellent rating to gravels with less than 10 percent carbonate particles and a high rating to gravels having 10 to 25 percent carbonate particles (*8*). New York requires that limestones contain 20 percent chert particles or 10 percent sand-sized insoluble residue (*9*). AASHTO guidelines (*10*) recommend the use of either the Acid Insoluble or Polish test for evaluating aggregates and also the consideration of field experience. Thus, a requirement of a minimum PSV from 45 to 48 and maximum carbonate content of aggregate from 10 to 25 percent will be in accordance with the accepted national and international practice.

CONCLUSIONS

1. The relationship between the wet-pavement accident experience on highways (wet/dry ratio) in Puerto Rico and pavement friction (mu values) was found to be statistically significant, although the correlation coefficients were not high.
2. Among the aggregate properties that affect friction, PSVs and acid solubility (carbonate content) were statistically significant. However, the carbonate content gave a better correlation than PSV.
3. When texture depth was included as an additional variable in the models with either solubility or PSV, the correlation coefficients increased slightly. The pavement surface friction values were generally higher than 40 when the texture depths were equal to or greater than 0.03 in.

RECOMMENDATIONS

On the basis of this study, the following recommendations are made for conditions in Puerto Rico:

1. PSV may be required only for major highways (expressways and primary highways). A minimum value of 48 may be specified for expressways and high-risk areas of primary highways. The value may be reduced to 45 for low-risk areas.
2. Carbonate content of aggregates also may be included in the specifications to ensure adequate skid resistance. A maximum value of 10 percent for expressways and high-risk areas and 25 percent for primary highways is recommended. The polish-value specification may be replaced with a limit

on carbonates for secondary highways because the polish test is more time consuming and difficult to perform.

3. Because the macrotexture also influences the pavement surface friction, it is preferable that such high-speed facilities as expressways have a texture depth not less than 0.03 in. This may be used as a general guideline (and not a requirement) for mix designs and construction practices.

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Criteria for Accepting Precoated Aggregates for Seal Coats and Surface Treatments

PRITHVI S. KANDHAL AND JOHN B. MOTTER

One of the most common causes of seal coat failures is the presence of dust on the cover aggregate, which prevents good adhesion between the applied bituminous binder and the aggregate. Precoating the aggregate with a thin film of bituminous binder usually solves the dust problem and provides good adhesion. This research was undertaken (*a*) to evaluate the adhesion of aggregates precoated to varying degrees so that the optimum precoating requirement can be established, and (*b*) to develop an end-result type test in lieu of the subjective visual test for accepting precoated aggregates. Five AASHTO No. 8 aggregates of different mineralogical compositions and absorptive characteristics were used. These aggregates were precoated with MC-30 cutback asphalt to varying degrees (from a salt-and-pepper effect to 90 percent or more coating). The Pennsylvania Aggregate Retention Test developed in this study was used to evaluate the effect of precoating on aggregate retention loss. Immediate adhesion of the cover aggregate with the bituminous binder was best obtained at 90 percent or greater precoating. The agreement (reproducibility) between different evaluators who made subjective visual evaluations of the percent precoating was also by far the best at a level of 90 percent or more. Of the three end-result type tests attempted, dry gradation test of the precoated aggregate was determined to be most appropriate with an acceptance criteria of 0.5 percent maximum minus 200 (dust). It has been recommended to use AC-20 asphalt cement as a precoating material in lieu of MC-30 cutback asphalt, because it can be mixed at higher temperatures in a hot-mix asphalt (HMA) plant, does not need any curing, and will cause better aggregate retention.

The Pennsylvania Department of Transportation (PennDOT) is responsible for the maintenance of 43,000 mi of roadway. PennDOT's projected maintenance program for 1987 included placing seal coat applications over 5,000 mi of roadway requiring more than 14 million gal of emulsified asphalt. One of the most common causes of seal coat failures is the presence of dust on the cover aggregate, which prevents good adhesion between the aggregate and the applied bituminous binder. Precoating the aggregate with a thin film of bituminous binder usually solves the dust problem and provides good adhesion. PennDOT recommends the use of precoated aggregates in seal coats and surface treatments on roads carrying average daily traffic (ADT) of more than 1,500 vehicles.

The current PennDOT specifications require that "at least 90 percent of the total visible area of the aggregates shall be coated with a bituminous film—any thin, brownish, translucent areas will be considered full coated." Questions have been raised about the minimum degree of precoating required and its subjective determination. Some people believe that a lesser degree of coating (even a salt-and-pepper effect) will be as effective as 90 percent coating. A need was felt to develop an end-result type test in lieu of the subjective visual test for accepting the precoated aggregates for department work.

The objectives of this research were (*a*) to evaluate the adhesion of aggregates precoated to varying degree so that the optimum precoating requirement could be established, and (*b*) to develop an end-result type test in lieu of the subjective visual test for accepting precoated aggregates.

REVIEW OF LITERATURE AND CURRENT PRACTICES

PennDOT uses a rational design method (*1,2*) to establish the application rates of bituminous binder and cover aggregate. This was done in 1975 to have a uniform practice throughout the state and to minimize failures resulting from improper application rates. However, the specification allowed up to 2 percent of minus 200 material (dust) in the cover aggregate. This was considered excessive for applications on high-volume roads and, therefore, specifications for precoated aggregates were developed in 1980 based on the experience in other states and overseas (particularly in Australia, New Zealand, and the United Kingdom).

Literature Review

Before commencing this research, a review of literature on precoated aggregates was conducted. The existing literature on this subject was summarized in 1968 in Special Report 96 of the Highway Research Board (*3*). It was mentioned that "one cannot overemphasize the importance of the physical condition (dusty) of the cover aggregate, the success or failure of a particular surface treatment might well depend solely upon the condition of the cover material." Research of Benson and Gallaway (*4*) indicated that for the presence of 1 percent dust there was a loss in aggregate retention of 12 percent by weight per unit area. One method of dealing with the dust problem is washing and drying the aggregate by mechanical means before application, which solves the problem almost entirely. The other methods include coating the aggregate with either a bituminous material or a kerosene film before application. Precoating with a bituminous material al-

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most guarantees good adhesion. Longer life experienced with precoated aggregates justifies the increased cost.

Harris (*5*) recommended the use of precoated aggregates in 1955, mentioning that "on heavily travelled roads, the trend is definitely towards the use of precoated aggregates, and probably as much as one hundred thousand tons will be used in Texas during 1955." Parr (*6*) reported on a surface treatment project more than 33 mi long in Michigan, which gave 17 years of service without maintenance. The aggregate was precoated with an SC-1 oil (approximately 1 percent). Although the cost is something to be considered, the long life of the surface treatment more than paid for the extra cost of precoating.

The Asphalt Institute (*7*) recommends that precoating the aggregate with a thin film of asphalt usually solves the dust problem and provides good adhesion of the asphalt to the aggregate. The aggregate is run through a hot-mix asphalt (HMA) plant dryer, cooled to under 200°F, then mixed in the pugmill with about 1 percent MC-70 to coat each particle thoroughly. The small amount of asphalt does not change the aggregate from a free flowing material, which can still be applied with aggregate spreaders. The precoating adds to the cost of the aggregate, but the additional cost is often justified by the better results obtained.

The Transport and Road Research Laboratory (TRRL) of the United Kingdom also suggests the use of precoated chippings to overcome the problem of dust (*8*). Compared with dry, uncoated chippings, coated chippings adhere immediately to the binder, especially when the binder viscosity is relatively high. The most common technique is to heat the chippings to between 220°F and 300°F and to coat them with more normal grades of tar or bitumen. The high temperature hardens the coating and makes the chippings easy to handle. A binder content of about 0.75 to 1 percent by weight of the chippings is suitable.

McLeod (*9*) stated that rapid development of good adhesion between cover aggregate and bituminous binder is highly desirable. To achieve this, the National Roads Board of New Zealand requires that all cover aggregates on state roads be washed to remove dust and then precoated. Precoating is done by a special cationic emulsion or cutback asphalt at the rate of 1 gal/yd^3 of chips. In Australia, to promote adhesion, stone chips are often similarly precoated with diesel fuel oil at the rate of 1 to 2 gal/yd^3 of cover stone or with 1 percent of MC-30 or MC-70.

Precoated sandstone cover aggregate was used successfully on Interstate 81 in Pennsylvania in FHWA Demonstration Project 55 (*10*). The seal coat job was completed in August 1980 using MC-30 cutback as a precoating material, and CRS-2 (Pennsylvania Designation E-3) emulsified asphalt as the application binder.

Epps et al. (*11*) prepared a field manual on design and construction of seal coats. They stated that "precoated aggregates are more expensive than untreated aggregates but have been utilized to reduce the effect of a dusty aggregate, to reduce automobile glass damage due to flying stone and to promote bond with asphalt."

Current Practices

A questionnaire was sent to all 50 United States and various highway agencies in Australia, New Zealand, and the United Kingdom. Australia and New Zealand are considered to be leading countries in obtaining most successful seal coat jobs in the world.

The questionnaire and the summary of the responses of 44 responding United States and other foreign countries on general seal coat practices such as most commonly used aggregate gradation and applied bituminous materials, tests and specifications for minus 200 material (dust) in the cover aggregate are given elsewhere (*12*). Only six states (Illinois, Oregon, Pennsylvania, Texas, Utah, and Virginia) use precoated aggregates.

MATERIALS

Aggregates

Five AASHTO No. 8 (PennDOT 1B) cover aggregates of different mineralogical compositions and absorptive characteristics were used in this study. Table 1 presents the sources and properties of these aggregates. Table 2 presents the specified and as-received gradations. Two gradations (graded and single-size) shown in Table 2 were used in the study for all five aggregates. These gradations were held constant to eliminate gradation as a variable. The gradation of AASHTO No. 8 aggregate was based on the average of 425 samples of aggregates in Pennsylvania.

Precoating Bituminous Material

PennDOT specifications permit the use of MC-30 and MC-70 cutback asphalts and AC-20 asphalt cement as precoating bituminous materials. MC-30 cutback asphalt (AASHTO M82), which is most commonly used in Pennsylvania, was used. The test properties of MC-30 such as kinematic viscosity at 140°F, distillate volumes at various temperatures, percent asphalt, and asphalt residue viscosity at 140°F are provided elsewhere (*12*).

Application Bituminous Materials

PennDOT specifications permit the use of AASHTO RS-2 (PA E-2) and CRS-2 (PA E-3) emulsified asphalts and AC-2.5 asphalt cement as the application bituminous material in seal coats. However, CRS-2 (PA E-3) emulsified asphalt, the most commonly used in Pennsylvania, was used in this study. The test properties of this cationic emulsion such as Saybolt Furol viscosity at 122°F, percent asphalt, and residue penetration are provided elsewhere (*12*).

TEST PROCEDURES

Tests on Aggregates

Bulk specific gravity and percent water absorption were determined in accordance with AASHTO T85. Flakiness index measures the tendency of an aggregate particle toward particle flatness, and it represents the percentage by weight of flat particles having a least dimension smaller than 60 percent of the mean size (*13*).

TABLE 1 SOURCES AND PROPERTIES OF AGGREGATES USED

	Aggregate Number				
	1	2	3	4	5
Producer	New Enterprise Stone & Lime Co. Ashcom	Obtained from Dist. 5-0	Wyoming Sand & Stone Co. Eaton Twp.	Columbia Asphalt Corp. Bloomsburg	State Aggregates Clifford
Type	Limestone	Limestone	Gravel	Siltstone	Sandstone
Bulk Sp. Gr.	2.795	2.758	2.559	2.678	2.639
% Water Absorption	0.31	0.97	1.95	1.69	1.66
Flakiness Index	27.2	54.2	15.9	28.0	16.0
Median Size, Inch	0.26	0.26	0.26	0.26	0.26
Average Least Dimension, in.	0.185	0.15	0.20	0.185	0.20
Particle Index (ASTM D3398)	15.9	---	12.3	14.7	13.9

TABLE 2 GRADATION OF AGGREGATES

Sieve	Specification	Aggregate Gradation as Received					Gradation Used in Study	
		1	2	3	4	5	Graded*	Single-Size
% Pass.								
1/2"	100	100	100	100	100	100	100	100
3/8"	85 - 100	94	84	91	93	97	90	100
No. 4	10 - 30	27	15	26	30	27	18	0
No. 8	0 - 10	4.2	6.0	3.0	2.8	5.1	2.5	0
No. 200	0 - 2.4	0.2	1.4	0.6	0.4	1.0	Variable	Variable

* Based on the average of 425 samples of aggregates in Pennsylvania.

Median size and average least dimension (ALD) of the aggregates were also determined according to the procedures given in the Asphalt Institute's Manual Series No. MS-19. These parameters and flakiness index are generally used for designing seal coats and surface treatments.

The particle index, which is a quantitative measure of aggregate particle shape and texture characteristics, was also measured in accordance with ASTM D3398.

Incorporation of Varying Dust Contents

Before the precoating phase of this study, it was believed necessary to study the effect of varying dust contents in the uncoated aggregate on aggregate retention. The aggregates were thoroughly washed with water to eliminate the minus 200 (dust) material completely. Then, varying amounts of dust (1, 2, 3, 4, and 5 percent by weight of the dry aggregate) were added to the aggregate. Water was added to the clean aggregate-dust mixture and thoroughly mixed to disperse the dust uniformly in the wet mixture. The mixture was then dried to constant weight. This procedure was used to simulate, as much as possible, the naturally occurring dust coatings on mineral aggregates.

Precoating Procedures

Aggregates containing 3.0 percent dust (establishment of this threshold value is discussed later) were precoated with MC-30 cutback asphalt to obtain the following five conditions:

1. No coating,
2. Salt-and-pepper effect,
3. Less than 50 percent coating,
4. More than 50 percent (but less than 90 percent) coating, and
5. More than 90 percent coating.

Any thin, brownish, translucent areas were considered to be coated. The percentage of coating was based on the total visible area of the precoated aggregate material. Individual particles were not considered.

A mechanical mixer was used to mix the aggregate and MC-30 cutback asphalt. Both materials were mixed at ambient temperature. Mixing time was approximately 6 min. Different percentages of MC-30 were used to obtain the required precoating conditions from a salt-and-pepper effect to 90 percent or greater coating. This required varying the percentage of MC-30 (by weight of the aggregate) from 0.4 to 1.1. All samples were cured in a flat pan for 2 days and were considered to be free flowing. Figure 1 shows an aggregate with five different precoating conditions.

Pennsylvania Aggregate Retention Test

This simple test method was developed by trial and error during this study. The testing equipment needed is available in most highway materials testing laboratories. The equipment consists primarily of 8-in. sieves, 8-in. pans, a sieve shaker (sifter), rubber pads, a compression machine, and a balance.

The procedure is described below:

1. Application of bituminous material: The emulsified asphalt (CRS-2) was poured on the back side of an 8-in. separator pan to obtain an application rate of 0.25 gal/yd^2. The emulsion was applied at 140 ± 5°F, and its weight was 36.8 g to give the desired application rate in an 8 in.-diameter pan.

2. Application of cover aggregate: It was established by trials that 300 g of aggregate is sufficient to obtain a single particle layer in the 8-in. diameter pan. This corresponds to 17.4 lb/yd^2. The aggregate is applied in the field by a chip spreader, which is difficult to simulate in the laboratory. However, an attempt was made to mechanize the process to minimize the variation in applications.

A Mary Ann laboratory sieve shaker (or sifter) was used. It can take an unclamped stack of 8-in.-diameter × 2-in.-deep standard laboratory sieves and pans. These are laid on a pair of 45°-inclined, rubber-covered, power-driven rollers, which revolve the stack. The pan bottom rests on a free-wheeling turntable. The aggregate is tumbled, mixed, and passed as it is carried up on the revolving inclined screen wire. To encourage clearing the openings, the sieve frames are tapped laterally (from below) by hardwood-faced aluminum hammers. These cam-cocked and spring-thrown hammers are pivoted on a nylon sleeve bearing.

For this study, the sieve shaker was inclined at an angle of 60° instead of 45°. Attempts to make it more to vertical were unsuccessful because the unclamped sieve stack fell out.

The pan containing applied emulsion was placed at the bottom of 5 inverted ½-in. sieves. A retainer or collar (sieve with no screen) was placed on the top. Figure 2 shows the complete assembly and feeding of aggregate from the top. The screen mesh in each ½-in. sieve was rotated 45° from the adjacent top or bottom sieve so that 2 consecutive sieve meshes did not have the same orientation.

After the sieve assembly was placed on the shaker and the shaker was turned on, 300 g of aggregate was poured into the retainer at the top. After 1 min, the pan containing emulsion and applied aggregate was removed and tapped to spread the aggregate evenly on the emulsion film.

3. Compaction and curing: Within 15 min, the pan was covered with a 7½-in.-diameter × ¾-in.-thick Neoprene bearing pad (of 50 durometer hardness) and placed under a compression machine to apply a load of 2,000 lb for 5 sec. This is equivalent to a pressure of 40 to 50 psi, which is normally used in pneumatic-tired rollers for seal coats. After compaction, the bearing pad was removed and the pan containing emulsion and aggregate was cured at the ambient temperature for 23 to 25 hr. The weight of pan + emulsion + aggregate was obtained after curing.

4. Initial retention loss: After the 24-hr curing, the pan containing the seal coat was inverted to allow the aggregate particles (which did not develop initial adhesion to the binder) to fall. These aggregate particles were weighed to determine the initial loss in grams. The percentage of initial loss is determined as follows:

$$\text{Percent initial loss} = B/A \times 100$$

where A is the weight of total aggregate (300 g), and B is the initial loss in grams.

5. Knock-off loss: After the initial loss was determined, the pan containing emulsion and aggregate was placed upside down at the top of the five ½-in. sieves (used for filling only), and a pan was placed at the bottom of the assembly to collect the knock-off loss. This complete assembly was placed in the Mary Ann sieve shaker described earlier and subjected to the shaking and tapping action for 5 min. The knock-off loss of the aggregate, which was collected in the bottom pan, was weighed (C). The percentage of knock-off loss was determined as follows:

$$\text{Percent knock-off loss} = [C/(A - B)] \times 100$$

where C is the knock-off loss in grams. It is realized that this knock-off test does not simulate the action of traffic in dislodging the aggregate from the seal coat. Nonetheless, it was used to give comparative results for uncoated aggregates containing varying dust contents and the precoated aggregates with different conditions of precoating.

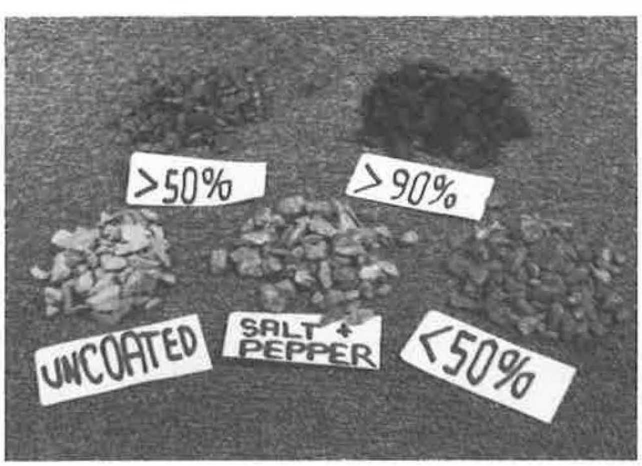

FIGURE 1 Typical five precoating conditions.

FIGURE 2 Complete assembly for applying aggregate.

6. Total loss: The total loss (initial loss + knock-off loss) was calculated as follows:

Percent total loss = $D/A \times 100$

where D is the total loss in grams ($B + C$). It should be noted that three aggregate retention tests were run for each sample type and the results averaged.

TEST RESULTS AND DISCUSSION

Aggregate Test Results

As mentioned earlier, the five aggregates (AASHTO No. 8 size) had different mineralogical compositions and absorptive characteristics. Tables 1 and 2 present the properties of the aggregates used. Limestone, gravel, siltstone, and sandstone aggregates ranged in water absorption from 0.31 to 1.95 percent. The flakiness index ranged from 15.9 percent (Aggregate 3, gravel) to 54.2 percent (Aggregate 2, limestone). The National Association of Australian State Road Authorities specifies 35 as the maximum permissible flakiness index for surface treatment. The median size of 0.26 in. was same for all aggregates because the same gradation was used. The ALD determined from the median size and flakiness index ranged from 0.15 to 0.20 in. The particle index ranged from 12.3 (Aggregate 3, gravel) to 15.9 (Aggregate 1, limestone).

Effect of Dust Contents on Aggregate Retention

Before precoating all aggregates, it was believed necessary to establish the dust content to be used consistently throughout the study. Therefore, varying amounts of dust (1, 2, 3, 4, and 5 percent) were added to the aggregates as described earlier.

The single size (⅜ in., No. 4) gradation instead of the total (½ in., No. 8) gradation was used to obtain better and more consistent results. The Pennsylvania Aggregate Retention Test described earlier was used.

Figure 3 shows the plots of percent dust content versus percent knock-off loss for all aggregates. The following trends were observed in this figure:

1. The rate of increase in knock-off loss with increasing dust contents (slope of the percent dust content versus percent knock-off loss line) becomes significantly greater after about 3 percent dust content in most cases. Therefore, this was considered a threshold value for all practical purposes and was used before precoating in the next phase of this study.
2. Most states specify a maximum of 2 percent (or 2.4 percent rounded off to 2) dust for unwashed aggregates. This appears to be reasonable for low-volume roads, particularly if the cost of washing or precoating the aggregate is high.
3. No correlation was observed between the percent knock-off loss and percent water absorption or particle index of the aggregate. However, a good relationship was observed when the flakiness indices of the aggregates and the corresponding aggregate retention losses were ranked (*12*). It shows the trend that the aggregate retention loss increases with increasing values of the flakiness index. It is quite possible that the flaky (flat) particles did not get pressed down well into the bituminous binder when compressed with the Neoprene bearing pad (pneumatic-tired roller in the field) because of the surrounding protruding cubical particles. Therefore, when the percentage of flat particles in the sample (or flakiness index) increases, the corresponding retention loss also increases.

Effect of Degree of Precoating on Aggregate Retention

All aggregates containing 3.0 percent dust contents were precoated to obtain five different conditions as described earlier in precoating procedures. Ten evaluators made subjective visual determinations of the percentage of coating on all aggregates for three conditions: less than 50 percent, more than 50 percent (but less than 90 percent), and more than 90 percent. The data are given in Table 3. It should be noted from the average data that it was difficult to achieve the condition of less than 50 percent precoating in actual practice because then the precoated aggregate tended to border on the salt-and-pepper effect. The average observed coating obtained for this condition actually ranged from 45 to 54 percent. This can reasonably be considered about 50 percent, although the tables will indicate it to be less. Table 3 presents the mean and standard deviations of percent coating on 15 precoated aggregate samples observed by 10 evaluators. It is quite evident

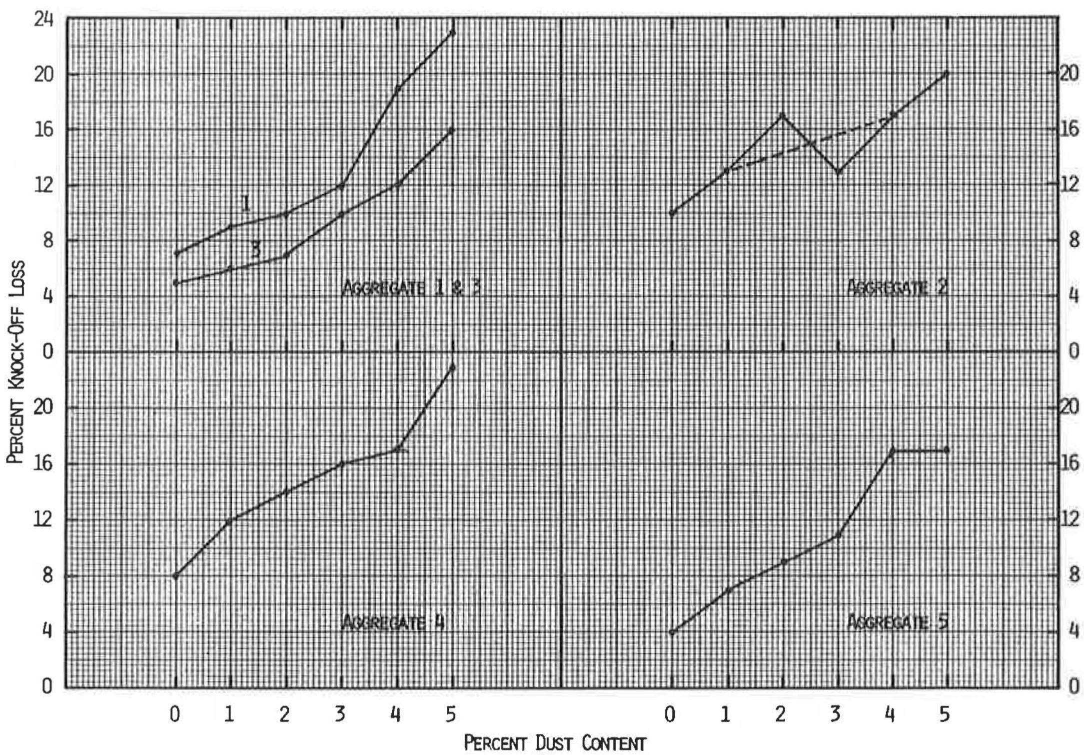

FIGURE 3 Percent dust content versus knock-off loss (all aggregates/one size).

from the data that the standard deviation decreases as the percentage of coating increases. In other words, the agreement between different evaluators becomes increasingly better when the percentage of coating is increased from 50 to 90 or more, the best agreement being for 90 percent or greater coating. It should be noted that the current PennDOT specifications require 90 percent or greater precoating and few, if any, problems have been experienced in judging this specified minimum percentage of precoating. It has been recognized by ASTM on the basis of cooperative tests that only at 95 percent level can a reasonable degree of the reproducibility be obtained when rating the same sample by visual estimation.

TABLE 3 SUBJECTIVE EVALUATION OF PERCENT COATING (10 EVALUATORS)

Evaluator	Percent Coating By Observation														
	Aggregate 1			Aggregate 2			Aggregate 3			Aggregate 4			Aggregate 5		
	<50	>50	90+	<50	>50	90+	<50	>50	90+	<50	>50	90+	<50	>50	90+
1	60	80	95	50	90	98	50	80	98	50	80	98	60	90	98
2	50	75	99	35	75	99	40	65	99	35	65	99	80	90	99
3	40	70	98	40	80	95	30	50	98	30	50	92	40	80	97
4	50	75	95	60	80	98	50	70	98	50	75	98	60	75	98
5	60	75	98	55	85	98	65	85	99	55	75	97	50	75	97
6	55	70	98	60	80	99	50	80	99	45	80	99	55	85	99
7	50	80	100	60	90	100	55	75	97	45	70	94	50	85	98
8	40	60	97	40	70	98	40	55	99	40	65	97	40	70	98
9	50	70	96	30	60	98	50	60	97	40	65	96	40	65	95
10	65	75	100	65	70	96	75	85	98	60	70	92	60	75	93
$\bar{X}$	52	73	98	50	78	98	50	70	98	45	70	96	54	79	97
Std. Dev.	8.2	5.9	1.8	12.4	9.5	1.4	12.8	12.6	0.8	9.1	9.0	2.7	12.5	8.4	1.9

Note: It was difficult to obtain less than 50 percent coating because then it approached salt & pepper effect.

ASTM specifies 95 percent level in ASTM D 1664-80, Coating and Stripping of Bitumen-Aggregate Mixtures, as a go-no-go test because the precision is not satisfactory for applications at lower levels.

All five aggregate precoated to different degrees were subjected to the Pennsylvania Aggregate Retention Test. Table 4 presents the aggregate retention loss data in percentages. Figure 4 shows the plots of percent precoated surface versus percent initial retention loss. The following observations were made:

1. Considering the percent initial loss, the 90 percent or greater precoating is by far the best. This means that immediate adhesion of the cover aggregate with the bituminous binder is best obtained with 90 percent or greater precoating. The primary function of precoating is to obtain immediate adhesion as discussed earlier in the literature review.

2. Increasing the percentage of precoating decreased the initial aggregate retention loss. A salt-and-pepper condition is better than uncoated aggregate, and so forth.

3. Initial aggregate retention loss was reduced by as much as 80 percent when the uncoated aggregate (containing 3 percent dust) was precoated with 90 percent or more coating.

It should be noted that 90 percent or greater coating gave poor results in the knock-off test. More than 50 percent coating gave the best results for Aggregates 3, 4, and 5. The precoating on Aggregate 2 did not help at all. It is suspected that the MC-30 cutback asphalt used as the precoating material in this study did not cure completely in two days. Although the initial adhesion was immediate and good, it appears that the cutback asphalt film around the aggregate was too soft (thus weakening the bond between the aggregate and emulsion residue) for the severe knock-off test. Therefore, it

TABLE 4 EFFECT OF PRECOATING ON AGGREGATE RETENTION LOSS (PERCENT)

	Coating	Aggregate Number 1	2	3	4	5
% Initial Loss						
$= \frac{B}{A} \times 100$	No Coating	21	34	18	21	18
	S. & P.	18	30	14	17	12
	Less than 50%	15	24	10	11	11
	More than 50%	13	15	10	10	9
	90% +	4	10	4	3	4
% Knock-Off Loss						
$= \frac{C}{A - B} \times 100$	No Coating	12	13	10	16	11
	S. & P.	10	19	10	15	7
	Less than 50%	10	17	7	10	5
	More than 50%	11	19	6	8	4
	90% +	15	27	9	13	8
% Total Loss						
$= \frac{D}{A} \times 100$	No Coating	31	43	26	34	27
	S. & P.	26	44	22	29	19
	Less than 50%	23	37	17	21	16
	More than 50%	22	31	15	17	13
	90% +	18	31	13	16	12

Notes: 1. A = Wt. of total aggregate (300 grams), B = Initial loss in grams, C = Knock-off loss in grams, and D = Total loss in grams (B + C).

2. S. & P. = Salt & Pepper effect.

3. All aggregates (uncoated and precoated) contained 3.0% dust.

4. Above results are based on an average of 3 tests. Therefore, the initial loss and the knock-off may not add up exactly to the total loss.

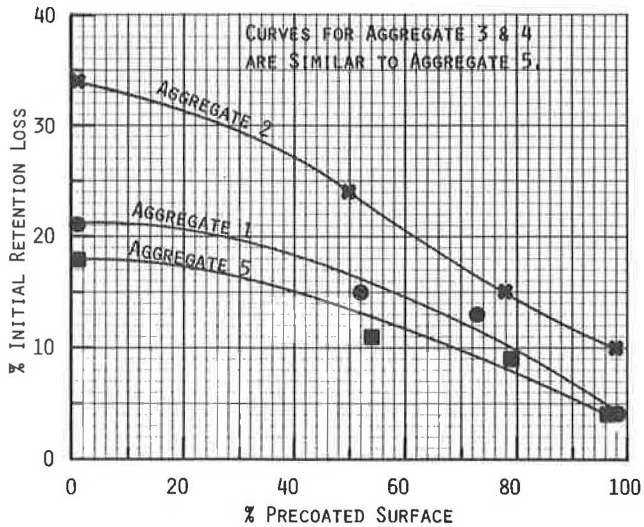

FIGURE 4 Percent coating versus initial retention loss.

is recommended to use AC-20 asphalt cement as the precoating material because it can be mixed with the aggregate at high temperatures in an HMA plant and does not require any curing. If MC-30 or MC-70 cutback asphalt must be used, the surface coating must cure completely.

It is also known that the rate of setting (breaking) of applied emulsified asphalts is slower with precoated aggregates compared with uncoated aggregates. The latter absorb water readily from the emulsified asphalt and thus accelerate the breaking process. This has been documented by TRRL in Road Note No. 39 (*14*), thus "the use of lightly-coated chippings when bitumen emulsions are used may lead to delay in the break of emulsion." Most experience in the past with the precoated aggregate in the United States and abroad has been with asphalt cements and cutback asphalts as application bituminous materials. Texas does not recommend the use of precoated aggregate with emulsions because the latter breaks and cures slowly (responses to questionnaire on durable asphalt emulsion seal coats, 1987). Undoubtedly, traffic control will be required for longer periods of time when precoated instead of uncoated aggregates are used with emulsified asphalts. Nonetheless, the importance of obtaining immediate and good adhesion that results from the use of precoated aggregates cannot be ignored.

If the total aggregate retention loss (initial loss and knock-off loss) is considered (Table 4), the benefits of precoating are apparent, and the 90 percent or more precoating is still the best.

Single Size Versus Total Gradation

So far, the reported data and discussion pertained to the single size (⅜ in., No. 4) aggregate gradation. Because AASHTO No. 8 aggregate (½ in.) is used by PennDOT for seal coats and surface treatments, it was necessary to verify whether similar results are obtained when the total gradation is used. This was attempted on Aggregates 4 and 5. The detailed data are given elsewhere (*12*).

These two total aggregates (½ in., No. 8) were also precoated to different degree and were evaluated by 9 observers. Similar to the single size aggregates, the standard deviation of the observed percent coating decreased as the coating increased. Again, the best reproducibility was obtained at precoating levels of 90 percent or greater.

Dust contents varying from 0 to 5 percent were also attempted on the total gradation of Aggregates 4 and 5. The comparative test data (single size versus total gradation) are given elsewhere (*12*). The following observations were made:

1. Aggregate retention loss (initial as well as knock-off) increased with increasing dust contents.
2. The rate of increase in knock-off loss with increasing dust contents became significantly greater after about 3 percent dust content in both cases.
3. As expected, the aggregate retention loss at all dust content levels was greater for the total aggregate compared with the single size. The former contains additional smaller particles that tend to fill the voids between large particles and thus may not get effectively embedded into the applied binder.

The above observations are similar to the observations of uncoated single size aggregates reported earlier. The following observations were made based on the test data obtained on precoated total gradations of Aggregates 4 and 5:

1. Considering the percent initial loss, the 90 percent or greater precoating is by far the means that immediate adhesion of the cover aggregate with the bituminous binder is best obtained with 90 percent or greater precoating of the total aggregate similar to the single size aggregate.
2. Unlike single size aggregates, the total aggregates had higher initial retention loss when precoated less than 50 percent compared with uncoated aggregates. Only when the percent coating exceeded 90 percent was a drastic reduction in the retention loss realized. Increasing the precoating of Aggregate 4 from 76 to 97 percent reduced the initial retention loss by about 50 percent. This observation is important because the results of this study are to be applied to the total aggregate (AASHTO No. 8) used by PennDOT and not to the single size aggregate. It appears that increased precoating is required to effectively bind the dust to the graded aggregate particles. This will be more evident when the end-result test data are discussed later.

End-Result Tests

It has been demonstrated in the previous sections that the subjective visual evaluation test is suitable and reasonably reproducible for 90 percent or greater precoated aggregates. However, the following end-result type tests were attempted on the precoated total aggregate (½ in., No. 8) to eliminate the subjective evaluation for final acceptance:

1. Dry gradation test: Because the precoated aggregates were free flowing, the samples were subjected to dry gradation test using ½-in., ⅜-in., No. 4, No. 8, No. 16, No. 30, No. 50, No. 100, and No. 200 sieves (sieving time was 10 min). This was attempted in order to quantify the presence of unbound

fine material (passing No. 8) and dust (passing No. 200) in the sample. The dry gradation data for all five aggregates precoated to different degree are given elsewhere (*12*).

It was evident from the data that a substantial amount of fine material (passing No. 8) and dust (passing No. 200) remains unbound (or loose) until the precoating level of 90 percent or more is reached. This unbound dust is quite likely to fall off in the chip spreader during field operations and interfere with the initial adhesion of the precoated aggregate to the bituminous binder. The previously discussed laboratory test data using the Pennsylvania Aggregate Retention Test support this observation. It was noted that 90 percent or greater precoating reduces the unbound dust (minus 200) to less than 0.1 percent, which ensures the development of good initial adhesion. If the dry gradation is used as an acceptance (or referee) test, it appears reasonable and practical to establish 0.5 percent maximum minus 200 as the acceptance criterion.

2. Wash test: The precoated aggregate samples were subjected to a wash test (with and without detergent) to determine the minus 200. Because the water containing detergent (sodium tripolyphosphate) started to strip the coating, the wash tests were performed under running tap water only. Two sieves (No. 16 and No. 200) were used. The unbound dust contents obtained by wash test and dry gradation test were comparable (*12*). Again, 90 percent or greater precoating reduced the dust content by the wash test to 0.3 percent.

After running several wash tests, it was concluded that the reproducibility of this test may not be satisfactory because it involves physical manipulation (stirring) of the sample by the operator and because of the likelihood of some partial stripping.

SUMMARY AND CONCLUSIONS

One of the most common causes of seal coat failures is the presence of dust on the cover aggregate, which prevents good adhesion between the applied bituminous binder and the aggregate. Precoating the aggregate with a thin film of bituminous binder usually solves the dust problem and provides good adhesion. This research was undertaken (*a*) to evaluate the adhesion of aggregates precoated to varying degrees so that the optimum precoating requirement could be established, and (*b*) to develop an end-result type test in lieu of the subjective visual test for accepting precoated aggregates.

Five AASHTO No. 8 aggregates of different mineralogical compositions and absorption characteristics were used. Two gradations, single-size (3/8 in., No. 4) and total (1/2 in., No. 8), were used. MC-70 cutback asphalt and CRS-2 (PA E-3) emulsified asphalt were used as the precoating and application bituminous materials, respectively.

The Pennsylvania Aggregate Retention Test was developed for this study to evaluate the initial adhesion loss and knock-off loss. Uncoated aggregates with 0 to 5 percent dust contents were also evaluated. Precoating of aggregates was varied from a salt-and-pepper effect to 90 percent or greater coating. On the basis of the preceding review of literature, test results, and discussions the following conclusions were drawn:

1. The rate of increase in knock-off loss with increasing dust contents in uncoated aggregates was generally observed to be significantly greater for more than 3 percent dust content. Therefore, 3 percent is considered to be a threshold value for all practical purposes.
2. A good relationship was observed between the flakiness indices of the aggregates and the corresponding aggregate retention losses. The latter increase with increasing values of the flakiness index.
3. Increasing the percentage of precoating decreased the initial aggregate retention loss. This loss was reduced by as much as 80 percent when the uncoated aggregate was precoated with 90 percent or more coating.
4. Considering the percent initial retention loss, the 90 percent or greater precoating was observed to be by far the best. This means that immediate adhesion of the cover aggregate with the bituminous binder is best obtained with 90 percent or more precoating.
5. Use of AC-20 asphalt cement (in lieu of MC-30 cutback asphalt) as a precoating material is recommended because it can be mixed with hot dry aggregate in a HMA plant, does not need any curing, and will cause better aggregate retention. If MC-30 or MC-70 cutback asphalt must be used it should be ensured that the coating has cured completely before the precoated aggregate is used.
6. Effects of dust content and extent of precoating on the aggregate retention loss were similar for the two gradations [3/8 in., No. 4 (single size) and 1/2 in., No. 8 (total)]. However, the corresponding retention losses were greater in the latter gradation, as expected, because it contained additional smaller particles.
7. Ten observers made 150 subjective visual examinations of the precoated aggregate samples. The agreement between different evaluators becomes increasingly better when the percentage of coating is increased from 50 to 90 or more, by far the best agreement (reproducibility) being for 90 percent or greater coating. The current PennDOT specifications require 90 percent or greater coating. Few, if any, problems have been experienced in judging this specified minimum coating except during the first 2 years when the precoated aggregate specifications were introduced in 1980.
8. Two simple end-result type tests: dry gradation test and wash test were attempted on the precoated aggregates in this study. The dry gradation test was determined to be more appropriate with an acceptance criteria of 0.5 percent maximum minus 200 (dust).

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Mr. Thomas Schmidt obtained most of the test data with the assistance of Mr. Morter. Edward Macko prepared the illustrations. Karen Siegfried and Mary Kathryn Gaylor assisted in the preparation of the manuscript. Messrs. Davis, Koehler, Casner, Nicholas, Ollinger, Lubold, Chilek, and Merrill assisted in reviewing the work plan and the research report.

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Bituminous Seal Coats: Design, Performance Measurements, and Performance Prediction

REYNALDO ROQUE, MATTHEW THOMPSON, AND DAVID ANDERSON

A field study was conducted at the Pennsylvania Transportation Institute's Pavement Durability Research Facility at Pennsylvania State University to determine the effects of specific construction, traffic, and material variables on the performance of bituminous seal coats. As part of the study, the adequacy of existing seal-coat design procedures, quality control procedures, and seal-coat performance measuring techniques was evaluated. The focus of this paper is on the latter evaluation; the effects of the other variables are reported elsewhere. The evaluations were based on actual field measurements and led to numerous recommendations for improvements in seal-coat design methods, equipment calibration, measurement and evaluation of seal-coat performance, prediction of seal-coat life, and the appropriate use of seal coats as a maintenance technique. The recommendations are reported herein. Finally, a definitive pattern of seal-coat macrotexture degradation was identified under this closely monitored field experiment. This finding was used to develop a prediction model for seal-coat life. Aggregate wear rates and embedment rates were measured on two surfaces under closely monitored traffic loading conditions. The wear and embedment rates were used to illustrate the potential of the seal-coat-life prediction model to evaluate the effects of different variables on expected seal-coat life. On the basis of the deficiencies observed in the existing design procedures, updated design charts that use more objective methods of evaluating the existing pavement surface are proposed, as are potential methods for rating the surface.

Seal coats are one of the most efficient and cost-effective methods available to state highway departments to rehabilitate and increase the skid resistance of highway pavements (*1*). However, results of surveys in Pennsylvania and elsewhere indicate that premature failure of seal coats is a common occurrence (*2*). These failures may be caused by improper design and construction procedures, substandard materials, or simply use of seal coats in cases in which some other form of maintenance may be more suitable.

Comparisons of aggregate and emulsion application rates predicted by different seal-coat design procedures indicate that a great deal of uncertainty is involved in seal-coat design. Roque et al. (*2*) showed that, for the same surface and aggregate, the emulsion application rate predicted by seven design procedures ranged from 0.19 to 0.30 gal/yd^2. These comparisons appear to indicate that much improvement can be made in categorizing surface hardness and texture and in including wear and embedment characteristics of the aggregate in the design process.

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Although a great deal of research has been devoted to analyzing the factors that affect seal-coat performance and seal-coat design procedures, no one has attempted to develop a model to predict seal-coat life. Only Marais attempted to incorporate the concept of a design life into a design process (*3*). However, Marais did not present a model to predict seal-coat life using known or measured properties of the materials and pavement surfaces. Most highway agencies predict the performance life of a seal coat on the basis of previous experience. Average seal-coat life expectancies range from 3 to 10 years (*4*). These averages are used to establish statewide maintenance budgets but are of little predictive value for individual projects. For example, McLeod and Nagi reported on seal coats performing well after 10 to 15 years, whereas others failed after only 1 or 2 years (*5,6*). If highway engineers are to make informed decisions about seal-coat maintenance within the framework of pavement management systems, then prediction models for seal-coat life must be developed.

DESCRIPTION, MATERIALS, AND COLLECTION OF FIELD DATA

Experiment Description

The following four variables were included in the experiment:

1. Pavement condition: worn ID-2 or new ID-2 leveling;
2. Emulsion rate: high or low;
3. Number of roller passes: 1 or 3; and
4. Traffic control (delay before application of traffic): 2, 8, or 24 hr.

Twenty-four test sections, each approximately 50 ft long, were used to accommodate each of the 24 variable combinations at the Pavement Durability Research Facility. The ID-2 mixture is a typical surface course mixture used by the Pennsylvania Department of Transportation (PennDOT); the maximum aggregate size is ⅜-in.

Traffic

Traffic at the Pavement Durability Research Facility was started August 8, 1988, and continued through August 18, 1988. Traffic consisted of a tractor pulling two single-axle trailers loaded to the legal axle limit of 22,400 lb per axle. Two two-person crews operated the truck for approximately 16 hr a day. A total of 73,400 truck passes was applied during the monitoring period.

Materials

A single aggregate source and emulsion were used. The aggregate was a heterogeneous, siliceous, glacial gravel produced at the Fairfield Township operation of Lycoming Silica Sand Company. The aggregate met the grading requirements for a Pennsylvania IB stone (AASHTO 8) that is to be used for seal-coat work (*7*). The percentage of material passing the No. 200 sieve was less than 1 percent. This aggregate met all other PennDOT specification criteria. Data for the aggregate gradation (sieve analysis) were as follows:

	Percent Passing	
Sieve Size	*IB*	*Single-Sized*
½ in.	100	100
⅜ in.	89	92
No. 4	21	8
No. 8	8	4
No. 16	4	3.5
No. 30	4	not applicable
No. 200	1	1

Data for the hydrometer analysis (the percent finer than the given size expressed as a percent of the total aggregate) were as follows: .025 mm, .55; .008 mm, .29; .001 mm, .13. Additional data were as follows: flakiness index (average least dimension), .2 in.; Los Angeles abrasion (percent wear), 30 (the maximum is 40); crush count (percent crushed faces), 94; bulk specific gravity, 2.62; and absorption, 2.15 percent.

A standard E-3 (ASTM CRS-2) emulsion was used in the construction of all test sections. The properties of the base asphalt cement used to manufacture the E-3 emulsion satisfied the requirements for an AC-10 asphalt cement. Routine tests performed on the emulsified asphalt indicated that all PennDOT specifications were met. An extensive series of conventional and nonconventional tests was performed on both the aggregate and the emulsion used in the project. The results of these tests can be found elsewhere (*2*).

Preconstruction Evaluation

The rut depths and surface texture of the pavement sections were evaluated before seal-coat construction. Measured rut depths ranged from 0.1 to 1.05 in. for the sections tested. The surface texture of the worn and leveled sections was evaluated visually. All of the surfaces were categorized into one of the five categories listed in the PennDOT Seal Coat Design Method (*7*). For design purposes, the worn ID-2 wearing surfaces were classified as smooth, nonporous surfaces (Category 2). The leveled surfaces, though not oxidized, were classified as slightly pocked, porous, and oxidized surfaces (Category 4).

Documentation of Construction Activities

During construction, Pennsylvania Transportation Institute personnel documented the following activities:

- Emulsion application rate;
- Aggregate application rate;
- Quantity of whip-off aggregate;
- Environmental conditions before, during, and after construction (including air temperature, pavement temperature, relative humidity, cloud cover, and wind conditions);
- Emulsion application temperatures;
- Time between emulsion and aggregate application;
- Time between aggregate application and rolling;
- Number of roller passes; and
- Time between rolling and application of traffic.

All construction activities and equipment calibration were under the control of PennDOT personnel. No attempt was made to alter the normal construction techniques, and, to the maximum extent possible, the experimental aspects of the project were designed to minimize any disturbance of normal construction procedures.

The emulsion application rate was determined with two different methods: the Standard Recommended Practice for Determining Application Rate of Bituminous Distributors (ASTM D 2995) and a procedure whereby fabric patches were placed on the pavement. The patch method, though simple and rapid, has not been standardized. The procedure was performed as follows:

1. A 2- × 2-ft, preweighed geotextile patch was placed on the pavement surface before the application of the emulsion. The fabric—4 oz/yd^2 nonwoven needle-punched polypropylene (Petromat L17540)—was nailed to the pavement at its corners with ½-in. roofing nails.
2. Immediately after the application of the emulsion but before the spreading of the aggregate, the fabric was carefully removed and placed in a preweighed plastic trash bag.
3. The trash bag containing the emulsion-soaked fabric was returned to the laboratory, opened, and placed in an oven at 140°F (± 5°F) for 24 to 48 hr to evaporate the water.
4. The asphalt-soaked fabric and the trash bag were weighed, and the quantity of emulsion in gallons per square yard was calculated using the water content of the emulsion and the specific gravity of the emulsion.

The Standard Recommended Practice for Determining Application Rate of Bituminous Distributors (ASTM D 2995) can be used to measure the transverse uniformity of the emulsion application rate, but it is more tedious to perform. Neither the ASTM method nor the geotextile patch procedure is suitable as a quality control test because of the turnaround time required to obtain the test results.

All measurements obtained with these methods on each of the seal-coat sections tested can be found elsewhere (*2*). On the basis of detailed statistical analyses performed on the emulsion rate measurements obtained, replicate samples appear to be necessary to obtain reliable results from either of these methods. The geotextile method described herein was found to be much less cumbersome and is therefore recommended over the cotton pad method. The geotextile method was determined to be sufficiently repeatable for use as a routine test procedure for checking emulsion application rates as long as three or more determinations are made for each test (*2*).

Aggregate Application Rate and Whip-Off

The aggregate application rate was determined in triplicate by the following method:

1. A 22- × 22-in. pan was placed between the wheelpaths of the pavement immediately after the emulsion was applied.
2. After the chip spreader passed over the pan, the pan was moved to the side of the pavement.
3. The collected aggregate was transferred to a preweighed bucket and dried in an oven at 140°F (± 5°F) for 24 hr.
4. The dried aggregate was weighed, and the aggregate application rate in pounds per square yard was calculated.

The aggregate not captured by the emulsion film (and susceptible to whip-off under traffic) also was measured for each test section by the following method:

1. The test was conducted approximately 20 to 50 min after the aggregate was rolled, when the bulk of the water in the emulsion had evaporated.
2. A 1-yd² template was placed between the wheelpaths of the test section.
3. All loose chips within the template area were collected by carefully brooming the pavement surface and were placed in a plastic bag for transport to the laboratory.
4. The aggregate was dried in an oven at 140°F (± 5°F) for 24 hr and weighed, and the aggregate whip-off in pounds per square yard was calculated.

The actual measurements of aggregate application rate and estimated whip-off can be found elsewhere, along with a detailed statistical analysis of these measurements (*2*). The results indicated that the variability between target and applied aggregate rates was significant. Although the variability is not good from the standpoint of construction, it did allow the research team to evaluate the effect of amount and gradation of aggregate retained on seal-coat performance and the reasonableness of the assumption of 10 percent whip-off for design purposes. The measurement techniques themselves were found to be relatively simple and accurate and are recommended as routine test procedures for checking aggregate application rates.

DESIGN OF SEAL COATS

The emulsion and aggregate application rates were determined using the procedure described in PennDOT Bulletin 27 (*7*). The PennDOT procedure uses the existing pavement condition, spread modulus (D_{50}) of the aggregate, average daily traffic (ADT), and absorption capacity of the aggregate as the variables necessary to calculate the application rates. Aggregate whip-off for this project was assumed to be 10 percent. The design for the Pavement Durability Research Facility was based on the following data:

- D_{50}: 0.268 in.;
- Loose unit weight: 90.4 lb/ft³;
- ADT: >2,000 vehicles/day;
- Absorptive aggregate: yes;
- Bitumen type: emulsion;
- Surface condition:
  –Worn ID-2: Category 2 (smooth, nonporous surface) and
  –New ID-2 leveling: Category 3 (lightly-pocked, porous, and oxidized surface); and
- Whip-off: 10 percent.

On the basis of these data, the following emulsion application rates were determined for the surfaces at the Pavement Durability Research Facility:

Surface	*Emulsion Application Rate (gal/yd²)*
Worn ID-2	0.27
New ID-2 leveling course	0.35

EVALUATION OF PERFORMANCE MEASURING TECHNIQUES

The following techniques were used to monitor the performance of the seal-coat sections at regular intervals:

- Sandpatch method,
- Skid resistance,
- Visual evaluations,
- Stereophotographs, and
- Geotextiles.

Table 1 includes a summary of the parameters that were obtained and the frequency, number, and location of measurement for each of these techniques. The table also presents a summary of the advantages and disadvantages of each technique in evaluating the performance of seal coats as determined from this project.

For this experiment, the mean texture depth (MTD) provided the most effective indication of seal-coat performance. This parameter proved to be sensitive and consistent in indicating the relative performance of the sections over time. This point is illustrated in Figure 1, which shows MTD as a function of time for three test sections: one with low MTD, one with intermediate MTD, and one with high MTD. Figure 2 shows that a similar trend in MTD over time was observed for all 24 test sections. The figure shows the MTD over time for all the test sections at specific times during the experiment. These consistent patterns were not detected by any of the other measuring techniques. The visual ratings were clearly affected by lighting conditions and other external factors so that no consistent pattern was observed over time or between evaluators. The skid resistance test was simply not sensitive to the texture changes taking place during the course of the experiment. A more detailed comparison of the measurements can be found elsewhere (*2,8*).

PREDICTION OF SEAL-COAT LIFE

The detailed measurements obtained during this experiment indicate that it may be possible to develop a model to estimate the life of seal coats that are well constructed and do not suffer from excessive chip loss. As shown in Figures 1 and 2,

TABLE 1 EVALUATION OF PERFORMANCE MEASUREMENTS FOR SEAL COATS

Technique	Parameter Obtained	Frequency	Number Per Section (location)	Advantages	Disadvantages	Remarks
Sand Patch	Mean Texture Depth (MTD)	Monthly	4 (outer wheel path)	Objective, sensitive	Small test area; distress mode not identified	May become less sensitive as macrotexture is reduced.
Skid Resistance	Skid Number (SN)	Monthly	5 (wheel path)	Covers entire section length, microtexture and microtexture may be evaluated	Lacks sensitivity early, affected by temperature and contamination, distress mode not identified	Becomes more sensitive as macrotexture is reduced.
Visual Examination	Three performance ratings: overall; bleeding; aggregate retention	Monthly	3 Evaluators (entire section)	Covers entire section; identifies failure mode	Subjective, lacks sensitivity, affected by lighting and environment	Unsuitable for ranking the sections or for detailed analysis.
Stereophotos	None	Monthly	1 (outer wheel path)	Visual record of changes with time at one location	Small test area.	Unsuitable for detailed analysis, since no parameter is obtained.
Geotextiles	None	---	---	---	Appear to affect performance	Did not work (could not be recovered from sections

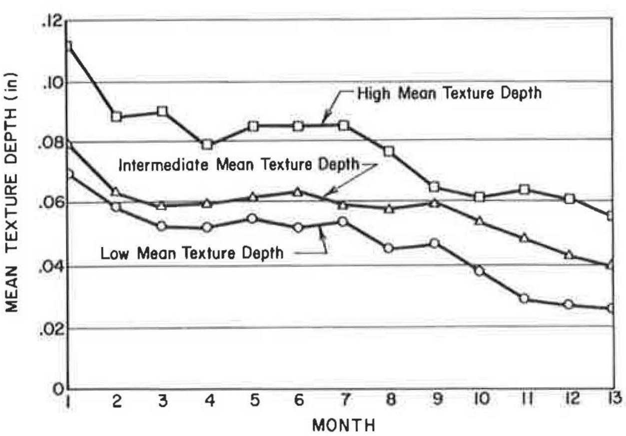

FIGURE 1 MTD as a function of time for low, intermediate, and high MTD sections.

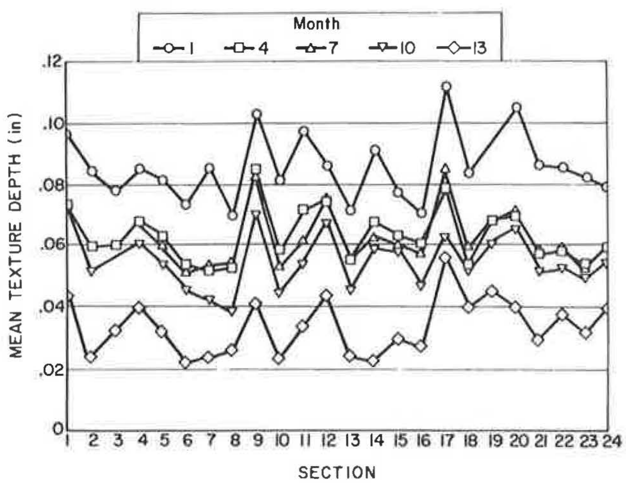

FIGURE 2 MTD at different times.

a consistent pattern was observed for the reduction in the MTD over time for all the test sections. It was found that during the warm months, the rate of reduction in MTD was greater than in the cool months. A generalized version of the pattern is illustrated in Figure 3, which shows MTD as a function of time or wheel passes. It was concluded that during cooler months, the underlying surface is sufficiently stiff to prevent aggregate embedment, so that the reduction in MTD may be attributed solely to aggregate wear. During warmer months, the underlying pavement surface stiffness decreases, thereby allowing aggregate embedment. It is assumed that the rate of wear is the same during cool and warm months.

Based on these findings, a simple model was developed to estimate seal-coat life by predicting the number of design wheel passes it will take for a seal coat to reach some terminal MTD. The model essentially predicts the seal coat's MTD as a function of time or wheel passes by using the equation shown

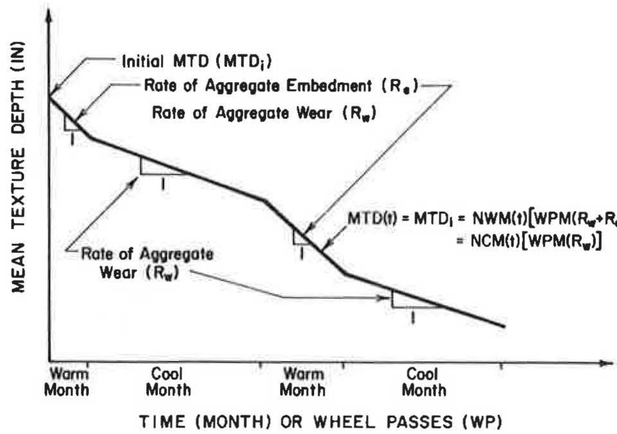

FIGURE 3 MTD as a function of time or wheel passes.

in Figure 3. The basic elements needed to estimate seal-coat life are as follows:

- The wear rate of the aggregate under traffic loading,
- The stiffness characteristics of the underlying surface as related to the aggregate embedment rate (primarily at warmer temperatures), and
- An estimate of traffic volume and distribution.

Using these elements, it is proposed that the following equation can be used to estimate seal-coat service life:

$$\mathrm{MTD}(t) = \mathrm{MTD}_i - \mathrm{NWM}(t)\,[\mathrm{WPM}(R_w + R_e)] - \mathrm{NCM}(t)\,[\mathrm{WPM}(R_w)] \quad (1)$$

where

$\mathrm{MTD}(t)$ = Mean texture depth (in.) at time t,
MTD_i = Initial mean texture (in.),
NWM = Number of warm months,
WPM = Number of loaded wheel passes per month,
R_w = Rate of wear during warm and cool months (in./20,000 wheel passes),
NCM = Number of cool months, and
R_e = Rate of embedment during the first three years of warm months (in./20,000 wheel passes). (Note that $R_e = 0$ for $t > 36$ months.)

This equation is based on several assumptions. First, the initial MTD must be measured or estimated. MTD measurements can be obtained after construction, but it is recommended that the measurement be taken 1 month after construction. If MTD measurements are not available, the initial MTD can be estimated as a percentage of the average least dimension (ALD) of the seal-coat aggregate. McLeod's design procedure assumes that the binder will have a height equal to 50 to 70 percent of the ALD (*5*). Therefore, the initial MTD will be 30 to 50 percent of the ALD. An average value of 40 percent can be used. In this model, only truck tires (80 to 100 psi) are assumed to produce aggregate wear and embedment.

Finally, it is assumed in this equation that embedment is a gradual process and is considered to reach an equilibrium after 3 years (*3*). For seal coats placed on asphalt concrete surfaces, it appears that the aggregate will cease to embed at some point. No one has attempted to quantify the amount of time or level of embedment at which embedment will no longer continue. Marais suggested that 3 years is an appropriate time period but noted that this time is affected by traffic volume. The MTD data from this experiment did not indicate that the seal coats reached an embedment equilibrium point. Thirteen months of accelerated traffic were applied to the test sections.

Determination of Aggregate Wear and Embedment Rates

Aggregate wear and embedment rates were measured under closely controlled field conditions at the Pavement Durability Research Facility. The rates were obtained for one aggregate on the two surfaces used in this investigation by converting the MTD and traffic data into the units required by Equation 1. The wear rate was computed as the loss in MTD per wheel pass during cool months, whereas the embedment rate was computed as the loss in MTD per wheel pass during warm months minus the calculated wear rate. The wear and embedment rates were determined for each surface type by finding R_w and R_e as defined in Figure 3, by using regression analysis on actual plots of MTD versus wheel passes for each traffic section. The following values were obtained:

- Worn ID-2: wear rate = 0.0080 in. (MTD) per 20,000 wheel passes; embedment rate = 0.0050 in. (MTD) per 20,000 wheel passes.
- Leveled ID-2: wear rate = 0.0080 in. (MTD) per 20,000 wheel passes; embedment rate = 0.0052 in. (MTD) per 20,000 wheel passes.

Additional details on the computations can be found in work by Thompson (*8*). As expected, the wear rate was identical for both surfaces because the same aggregate was used for both. The embedment rates were also nearly equal, which indicates that both surfaces had about the same stiffness.

Analyses Using the Model

A computer program based on Equation 1 was developed on spreadsheet software to estimate seal-coat life. A simplified flowchart of the program is shown in Figure 4. The following input is required:

- Highway ADT, percentage of trucks, and number of axles per truck. The program uses the three input parameters to determine the number of wheel passes per month (WPM).
- Initial MTD after construction. This value can be measured or estimated.
- The date of construction. This establishes a starting point from which the number of warm and cool months can be determined.
- The rate of MTD reduction resulting from aggregate wear (occurs during both warm and cool months), and the rate of MTD reduction resulting from aggregate embedment (occurs during warm months only).
- MTD failure criteria. This is the value of MTD that the user defines as seal-coat failure (MTD_f).

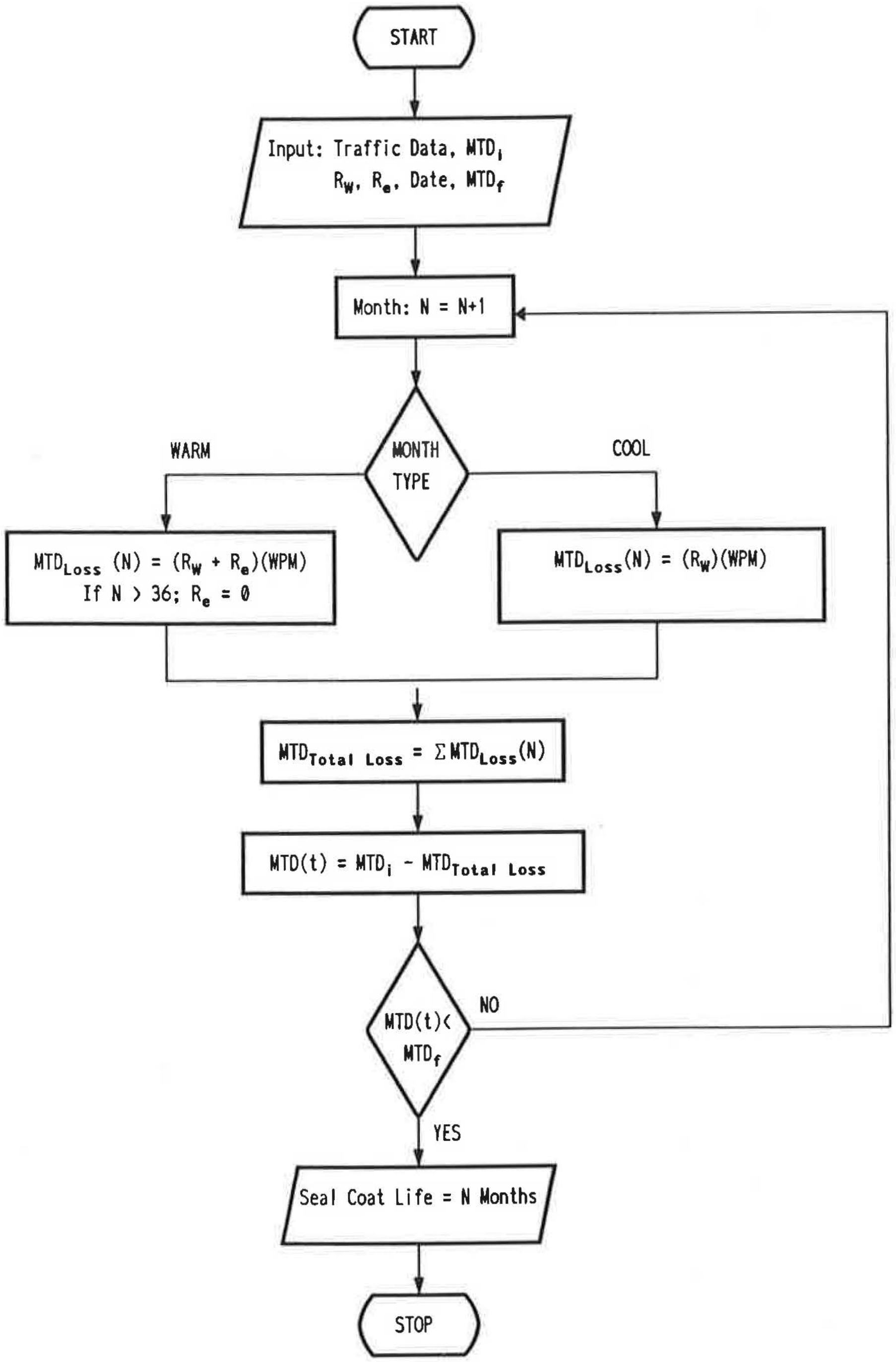

FIGURE 4 Flow chart of computer program to predict seal-coat life.

The program uses the data above to estimate the amount of MTD reduction each month. It then sums these reductions in MTD until the remaining MTD is less than the failure criteria. The program counts the number of iterations required to reach the failure criteria and uses that number to determine the following information:

- The predicted number of wheel passes to failure,
- The predicted life of the seal coat in months, and
- The predicted date of failure on the basis of the construction date.

Although not validated, this program provides a means by which to predict differences in expected seal-coat life caused by variations in aggregate characteristics, underlying surface stiffness, and emulsion application rates. Sabey recommended an MTD failure criterion of 0.025 in. as the texture depth below which a seal coat will no longer have adequate skid resistance (*9*). As mentioned earlier, if measured values are not available, the initial MTD of a well-constructed seal coat can be estimated as approximately 40 percent of the average least dimension of the aggregate used. Aggregate wear rates can be measured as was done in this investigation and eventually determined from relationships to laboratory tests such as the ones proposed by Marais and shown in Table 2 (*3*). Similarly, aggregate embedment rates would need to be measured as in this investigation and eventually determined from relationships to surface hardness. The results of this investigation indicated that the embedment rates for the two surfaces tested were not significantly different.

Several analyses were performed to illustrate the potential of the model. An ADT of 2,000 with 10 percent trucks was assumed with an initial MTD of 0.085 in. and an August construction date. In one analysis, the effect of aggregate wear rate was compared. The embedment rates measured for the field test sections in this experiment were used for the analysis. The wear rate of the aggregate in this experiment was 0.0008 in. per 20,000 wheel passes. Its Los Angeles abrasion loss was 30 percent. Using Marais's relationships (Table 2), a wear rate of 0.0006 in. per 20,000 wheel passes was determined. Using the model, a difference in expected life of approximately 1 year was computed. Similar comparisons can be made for the effect of underlying surface stiffness, emulsion application rate, and traffic (*8*).

At present, of course, the accuracy of the predictions is uncertain. Therefore, the model is simply proposed as a method to objectively evaluate the effects of different factors on a relative basis. Only systematic measurements and experience will validate the model.

ADJUSTMENTS TO EXISTING DESIGN CHARTS

Most existing design procedures compute the emulsion application rate by simply using a visual evaluation of the surface. An evaluation of visual ratings as performed in this investigation indicated that the rating obtained by two different evaluators, or even by the same evaluator on two different days, may be very different. Also, visual ratings do not give a true indication of the surface hardness. Given the importance of determining proper emulsion application rates in producing adequate seal coats, a better, more objective method of rating the existing surface is needed.

The sandpatch test, which measures MTD, was found to be reliable and consistent for measuring surface texture. It is therefore recommended to obtain an objective measure of the texture of the pavement from which adjustments in the amount of emulsion required to fill the voids in the existing pavement can be made. McLeod's binder adjustment factors for surface texture were used to determine the amount of emulsion required for a known change in texture (*10*). A conversion factor of 1 gal/yd^2 = 0.20 in (MTD) was determined. This conversion factor was used to convert PennDOT's existing visual classification to MTD measurements, as shown below.

- Category 1 (flushed asphalt surface):
  - –McLeod's Adjustment Factor = −0.06 gal/yd^2;
  - –Calculated MTD = not applicable.
- Category 2 (smooth, nonporous surface):
  - –McLeod's Adjustment Factor = 0.00 gal/yd^2;
  - –Calculated MTD = 0.000 in.
- Category 3 (slightly porous, oxidized surface):
  - –McLeod's Adjustment Factor = 0.03 gal/yd^2;
  - –Calculated MTD = 0.006 in.
- Category 4 (slightly pocked, porous, and oxidized surface):
  - –McLeod's Adjustment Factor = 0.06 gal/yd^2;
  - –Calculated MTD = 0.012 in.
- Category 5 (badly pocked, porous, and oxidized surface):
  - –McLeod's Adjustment Factor = 0.09 gal/yd^2;
  - –Calculated MTD = 0.018 in.

Thus, if MTD measurements of the existing surface are obtained, it is possible to enter the design chart by using the equivalencies shown above.

Adjustments for surface stiffness would still have to be incorporated. This would require more work to relate surface hardness measurements (as described earlier) to the amount of aggregate embedment. However, the authors feel that the establishment of such relationships is well worth the effort because it would take much of the guesswork out of seal-coat design and construction and would eventually provide the capability to obtain better estimates of seal-coat life. A design example as it might be performed with the proposed measurements can be found elsewhere (*8*).

TABLE 2 ESTIMATED DEGRADATION AND WEAR UNDER CONSTRUCTION ROLLING AND TRAFFIC (10-YEAR LIFE) (*3*)

Los Angeles Abrasion Value	Degradation and wear of stone mat (mm x 10^{-2})									
	Equivalent traffic (vpd/lane)									
% Loss	>4,000	4,000	3,000	2,000	1,000	800	600	400	200	100
34 - 27	100	92	86	78	66	66	58	52	44	37
26 - 22	90	86	80	72	60	58	54	48	40	34
21 - 15	80	78	74	68	56	54	50	46	38	32
14 - 10	75	72	68	62	52	48	46	42	36	30
9 - 4	70	68	62	56	48	46	42	38	32	28

CONCLUSIONS AND RECOMMENDATIONS

On the basis of the findings of this investigation, several conclusions were drawn related to different phases of a seal-coat operation.

Given that emulsion and aggregate application rates were found to be one of the most critical factors governing the performance of a seal coat (*2*), both the distributor and the chip spreader must be properly calibrated. The geotextile method used in this investigation and described herein was determined to be sufficiently repeatable for use as a routine test procedure for checking emulsion application rates as long as three or more determinations are made for each test (*2*). Use of a 1-yd^2 pan to measure aggregate application rates was found to be suitable for calibration purposes and for construction control.

The existing method of visually rating pavement surfaces for seal-coat design purposes appears to be inadequate. A more objective method for rating pavement surfaces for seal-coat design purposes should be developed to include both surface texture and surface hardness. Mean texture depths as measured by the sandpatch test should be used to characterize the surface texture. The equivalency factors presented earlier can be used for this purpose. Additional work must be done to determine relationships that incorporate a measurement of the surface hardness.

A mean texture depth measurement should be used along with a visual rating to evaluate the in-service performance of seal coats. Deficiencies of obtaining only a visual rating were evident, and the MTD measurement may also be used to estimate the remaining life of a seal coat using the model developed in this investigation.

It appears that the service life of well-constructed seal coats can be estimated for the aggregate and surfaces used in this investigation using the prediction model developed herein. Wear and embedment rates for other aggregates and surfaces should be measured and relationships developed with laboratory tests and field measurements of surface hardness.

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Correlation of Laboratory Tests to Field Performance for Chip Seals

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The Nevada Department of Transportation constructed 44 chip-seal test sections between Yerington and Wabuska, Nevada. One of the purposes of these test sections was to correlate laboratory and field testing with pavement performance. The laboratory testing consisted of the Vialit-time series and the Vialit-temperature series. Field testing included only the Vialit-time series. Pavement performance was based on pavement evaluations and percent reflective cracking. The pavement evaluations recorded overall condition, aggregate retention, aggregate embedment, and bleeding. After a comparison of the field and laboratory testing with pavement performance three conclusions were drawn: (*a*) the field Vialit-time series testing did not correspond to laboratory Vialit-time series testing because of the variation in curing temperature of the field samples; (*b*) the laboratory Vialit-temperature series can detect the effects of aggregate gradation on different binders; and (*c*) aggregate retention of the sample cured at 0°F is a good indicator of overall chip-seal performance. Ratings of 8.0 or greater are likely for overall condition and aggregate retention if the percent aggregate retention for the laboratory sample cured at 0°F is greater than 60 percent.

The three main factors that determine the success of a chip seal are weather conditions during construction, construction practices, and selection of materials. Weather conditions, although they cannot be controlled, can in many cases be anticipated. Chip sealing in cool weather might affect the selection of binder or require much tighter control of the construction sequence.

Construction practices can alter the performance of almost any binder. Careful attention must be paid to the uniformity of the binder application, the time between the binder and aggregate applications, and the sequencing and timing of the rollers.

However, even if construction practices are perfect, some binder-aggregate systems still will not perform adequately. Many choices of materials are available from conventional emulsions and viscosity grade binders to a myriad of modified binders. Each of these materials has desirable properties, such as low application temperatures (emulsions), quick setting time (emulsions, viscosity grades), and increased resistance to thermal cracking (modified binders). Variables associated with selecting aggregates include gradation (uniformly sized stone versus graded), prewet, or precoated.

The purpose of the joint research of the Nevada Department of Transportation (NDOT) and the University of Nevada, Reno (UNR) conducted in July 1989 was to attempt to develop laboratory test methods for chip seals to determine the performance of different binder-aggregate systems.

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RESEARCH PROGRAM

The research program had one main objective: to correlate laboratory testing with field test results and performance of chip seals. Laboratory testing included Vialit testing versus both time and temperature. The field testing included taking Vialit field samples during construction. Pavement performance was based on pavement evaluations conducted 3 months and 11 months after construction and percent reflective cracking.

Forty-four experimental chip-seal test sections were placed on US-95 (Alternate) near Yerington, Nevada, between July 17 and July 21, 1989. Nine product suppliers provided the following: one unmodified emulsion, two unmodified viscosity grade asphalt cements, and six modified viscosity grade asphalt cements. The binder and aggregate quantities were designed to vary (two levels). Two aggregate gradations were used with each of the binder quantities. The same source of aggregate was used for both gradations.

Vialit sample plates that were placed on the pavement and subjected to the actual construction sequence of spraying, aggregate application, and rolling were used to monitor the material that each binder was capable of retaining at various test times.

MATERIALS

The physical properties of the binders were obtained from the producers, and the aggregate properties reported below were determined by the UNR laboratory.

Binders

The field test sections consisted of two control and seven experimental binders from various companies. The control binders were LMCRS-2H (emulsion) and ASTM D3381 Table 3 AR 2000 (viscosity grade). Other binders used were the following:

- ASTM D3381 Table 2 AC10,
- EVA-modified AC10,
- Kraton-modified AR 1000,
- AC20R,
- AR1000 modified with crumb rubber,

- AC10R, and
- AR 4000 modified with a styrene-based polymer.

The physical properties of these binders, as supplied by the producers, are presented in Table 1.

Aggregates

Two gradations of aggregates were used during the course of this project: the standard NDOT 705.03.05 and the Texas Item 302 No. 4 (*3,4*). The gradations are presented in Table 2. Most of the aggregates used were precoated with 0.5 percent AR 4000 and stockpiled for at least a week before construction.

There was one major source for all the aggregates used in the construction of these test sections with one exception—the uncoated Texas gradation used in only four of the LMCRS-2H control sections. Inadvertently, all of the Texas gradation aggregate prepared was precoated. Therefore, the uncoated Texas gradation aggregate from the previous year's chip-seal test sections was used. The physical properties of the aggregates can be found in Table 2.

CONSTRUCTION OF TEST SECTIONS

Test sections were constructed according to the data presented in Table 3. A total of 11 test sections was constructed in this field experiment. To provide easier notation, each section was assigned a number that also designated the order of construction. For example, the first binder applied was assigned the number one, the second applied was given the number two, and so on. Sections 10 and 11 were duplicates of sections 1 and 2 (i.e., they were control sections).

Each test section was separated into four subsections: A, B, C, and D. (Figure 1). The viscosity grade asphalt cement subsections labeled A and B were constructed with the Nevada gradation precoated aggregate. The C and D subsections were constructed with the Texas gradation precoated aggregate. The A and B emulsion subsections were constructed with uncoated Nevada gradation aggregate; uncoated Texas gradation aggregate was used for test sections C and D. Aggregate spread rates varied among sections and subsections from 13 to 30 lb/yd^2.

TABLE 1 ASPHALT CEMENT PROPERTIES

Test	AC10	EVA Modified	Kraton Modified AR 1000	AC20R	AR 1000 Modified w/Crumb Rubber	AC10R	Liquid Styrene Modified AR 4000
Viscosity							---
140F, P	---	---	---	---	---	1000	---
275, cSt	250	418.8	550	325	---	---	
Penetration							
77°F, 100g 5 sec	80	62	110	---	---	83	---
Ductility							
77°F, cm	75	---	---	---	---	---	---
39.2°F, cm	---	9	145	25	---	26	---
45°F	---	32	---	---	---	---	---

NOTE: Results supplied by producers.

TABLE 2 AGGREGATE PROPERTIES

Test	Specification		Project Aggregate	
	Nevada	Texas	Nevada	Texas(Pre)
Bulk Specific Gravity	NA	NA	-----	2.785
Bulk Specific Gravity, SSD	NA	NA	2.689	2.825
Apparent Specific Gravity	NA	NA	-----	2.902
Adsorption Capacity, %	NA	NA	1.180	1.443
Theoretical Maximum	NA	NA	NA	2.748
Sieve Analysis (% Passing)			---	---
	100	100	99.7	94.5
5/8"	100	98-100	77.4	61.0
1/2"	50-80	65-85	32.6	43.5
3/8"	0-15	0-5	3.6	0.8
#4	0-5	---	---	---
#8	---	0-1	1.7	0.6
#10	---	---	---	---
#16	---	---	---	---
#30	---	---	---	---
#50	---	---	---	---
#100	0-2	---	---	---
#200				
Board Test (lb/yd^2)	NA	NA	15	18
Unit Weight Rodded	NA	NA	50.0	47.6
Shoveled	NA	NA	45.5	45.0
Average Least Dimension	NA	NA	0.31	0.32

NOTE: Information provided by UNR, except aggregate gradation of Texas precoat, which was provided by Exxon.

TABLE 3 SUMMARY OF BINDER AND AGGREGATE QUANTITIES FOR SUBSECTIONS

Product	Subsection	Binder Quantity (gal/yd^2)	Aggregate Quantity (lb/yd^2)
AR 2000 (2)	A C	0.36 0.46	21 25
AC20R (3)	A C	0.35 0.43	22 28
AR 1000 w/crumb rubber (4)	A C	0.60 0.69	30 30
AC10 (5)	A C	0.38 0.39	24 22
EVA Modified AC10 (6)	A C	0.36 0.36	18 23
Kraton Modified AR 1000 (7)	B C	0.36 0.36	20 21
Liquid Styrene Modified AR 4000 (8)	B C	0.33 0.34	20 23
AC10R (9)	A B	0.40 0.36	20 20
AR 2000 (10)	A C	0.36 0.36	20 20

A binder quantity selected as optimum by the producer was used for all subsections labeled A and C. Subsections labeled B and D were constructed with an increased binder quantity, relative to each binder's original design quantity.

FIELD TESTING AND PERFORMANCE

The field performance was based on Vialit samples from actual construction, pavement evaluations 3 months and 11 months after construction, and percent reflective cracking.

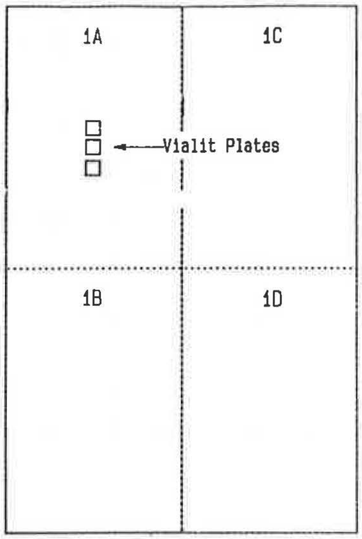

FIGURE 1 Typical test section.

Field Vialit Test

The Vialit test, described in detail in previous work, was used to evaluate aggregate retention. A brief synopsis of the test is as follows: a chip seal is simulated on a 7- × 7-in. steel plate and is allowed to cure for various time intervals (typically 10 min, 30 min, 2 hr, 5 hr, and 24 hr). The plate is then weighed, inverted for 10 sec, weighed, and inverted while a steel ball is dropped three times on the back of it. The plate is weighed again, and the material retained is calculated.

The Vialit plates were placed on the pavement 6 ft from and parallel to the center line (Figure 1). These plates proved to be heavy enough to allow construction to proceed as usual with one exception: rollers were slowed to half their normal speed to prevent the metal plates from flipping up and damaging the samples. After the last roller and before the brooms had passed, the plates were picked up and tested (Figure 2). Figure 3 shows the testing apparatus.

Samples were then tested after the appropriate time interval. The test times were limited to 10 min, 30 min, and 2 hr for viscosity grade binders because of the large quantity of test sections being placed and the limited mobile storage for test plates. The 2-hr limit was chosen on the basis of previous research by UNR that indicated results for viscosity grade did not change after this time interval.

FIGURE 2 Removal of Vialit plates for testing.

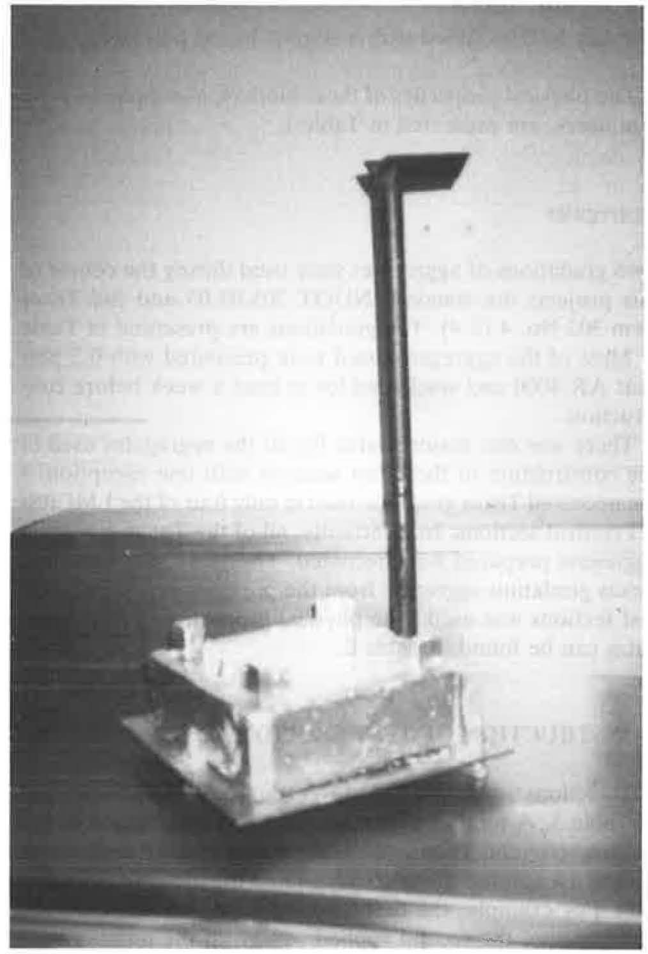

FIGURE 3 Vialit test apparatus.

Field Evaluations

Field evaluations were conducted 3 months after construction (October 1989) and 11 months after construction (May 1990). The evaluations were made at the same location both times. The evaluators were Jon Epps (UNR), Ken Davis (NDOT), and representatives from each of the participating suppliers.

To save time, evaluations were recorded in only two of the four subsections per binder. One subsection was located where the Texas gradation was used and the other was located where the Nevada gradation was used. The subsections evaluated were chosen as the most representative subsections for each binder and gradation. Evaluation sites were located near the field testing sites.

Overall condition, aggregate retention, and bleeding were rated on a scale of 1 to 10, 10 being best. Aggregate embedment was assessed by the percentage of embedment of an average-sized aggregate.

Percent Reflective Cracking

The percent reflective cracking was calculated as the number of feet of cracking 1 year after construction divided by the number of feet of cracking before chip-seal construction. These quantities were determined from preconstruction and postconstruction crack maps.

Preconstruction crack maps covered 24- × 100-ft sections (i.e., both the northbound and southbound lanes) and were recorded every 1,200 ft for the length of the project. Three to four crack maps are located in each binder section. Moderate to severe cracking was recorded at the south end of the project, whereas slight to moderate cracking was noted at the north end. None of the cracks at the north end were sealed just before construction; cracks at the south end of the project were sealed several months before construction. Postconstruction crack maps were recorded in the same locations and cover the same distance as the preconstruction crack maps. The postconstruction maps were recorded by UNR personnel June 25 to 28, 1990.

LABORATORY TESTING

The purpose of the laboratory testing was to establish a criterion for accepting or rejecting binders to be used for future chip seals. The testing consisted of the laboratory Vialit (both over time and various temperatures).

Actual binder and aggregate quantities used to construct each test section were determined, and these quantities were used to prepare the laboratory Vialit plates for each test section. The binder quantity was found by averaging the tank stabbing, and the aggregate spread rate was calculated from the average of the pan tests for each subsection. The laboratory Vialit-time series is the same test used in the field (described above). Plates 7 × 7 in. were prepared in the laboratory and then tested at 10-min, 30-min, and 2-hr intervals for the hot binders (Figures 4–8).

Because hot binders behave relatively the same over time, a more severe test was needed to differentiate between binders. This test was the temperature series Vialit. Five samples for each section were prepared as before and cured for 24 hr

FIGURE 5 Aggregate application box over sample plate after application of binder.

FIGURE 6 Preparing to drop aggregate on binder.

FIGURE 4 Preparation of sample plate (application of binder).

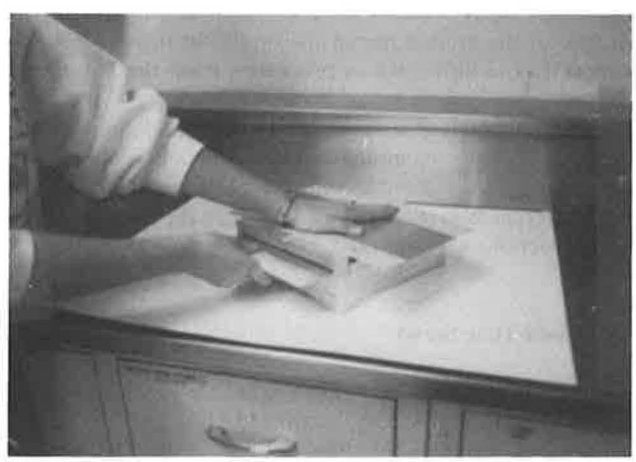

FIGURE 7 Metal plate being pulled out so that chips fall evenly onto binder surface.

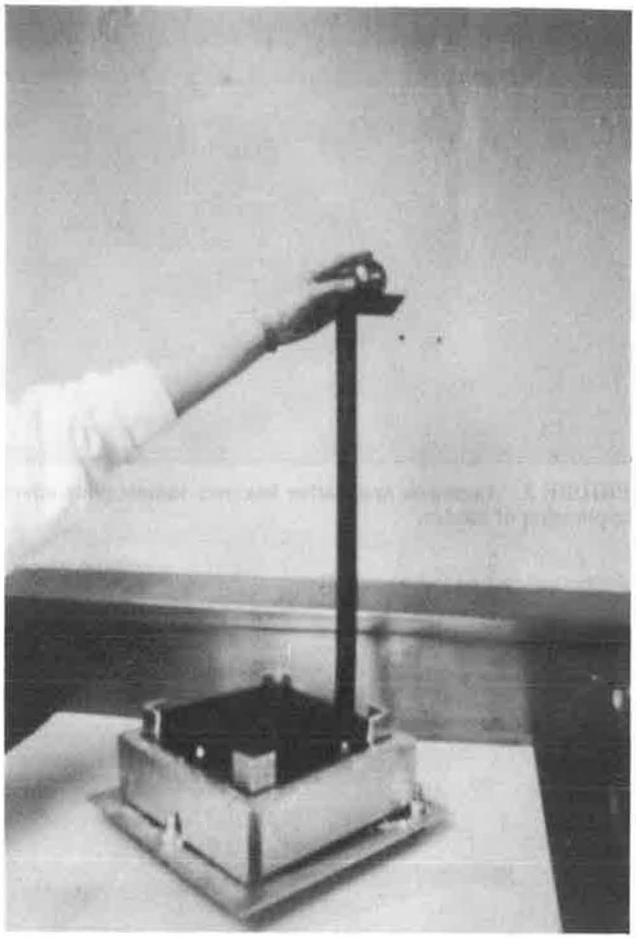

FIGURE 8 Preparation of cured specimen testing.

at 77°F. The first sample was then conditioned to 0°F, the second to 32°F, the third to 50°F, the fourth to 77°F, and the fifth to 104°F; each sample was tested after 24 hr.

ANALYSIS OF TEST RESULTS

The LMCRS-2H sections were not analyzed in this report because of the limited use of the binder in this project and because of the difference in properties from the hot-paving binders.

In the data analysis some binders exhibited definite differences between subsections in laboratory and field results. These subsections are discussed individually. Other binders did not show differences in subsections; therefore the average of the two subsections was analyzed.

Field Vialit-Time Series

Field testing information was not available for the AR 2000 section (2) and the AC20R section (3A). All of the binder sections had aggregate retention between 60 and 100 percent except one. AR 2000 (10) had aggregate retention between 25 and 100. The AR 1000 modified with crumb rubber (4) section, the styrene-based polymer modified AR 4000 (8) section, and the AC10R (9) section showed some decrease in aggregate retention with time. This was caused by the change in curing temperature during the day. For example, a field sample may have been completed at 10:00 when the temperature was 85°F and tested 2 hr later when the temperature was 100°F. Therefore, the aggregate retention for the 2-hr test would probably be less than the aggregate retention after 10 min. Figures 9 and 10 show typical aggregate retention versus time.

Pavement Evaluation 3 Months After Construction

The overall condition of the sections ranged from 6.0 to 9.4, and aggregate retention ranged from 6.0 to 9.7. Little bleeding was detected in any of the sections; ratings varied from 8.4 to 10. Aggregate embedment fluctuated between 25 and 66 percent in the wheelpaths and from 20 to 52 percent between the wheelpaths. Results are presented in Table 4.

Pavement Evaluation 11 Months After Construction

Section 4, an AR 1000 modified with crumb rubber, had the best overall condition with ratings of 9.2 and 9.3. Other sections had moderate to good overall condition with ratings from 7.0 to 8.7. Two of the binders showed significant drop in overall condition. They were the AC 10R (9C) and the AR 2000 (10A and 10C) with ratings of 5.4, 3.9, and 3.8, respectively.

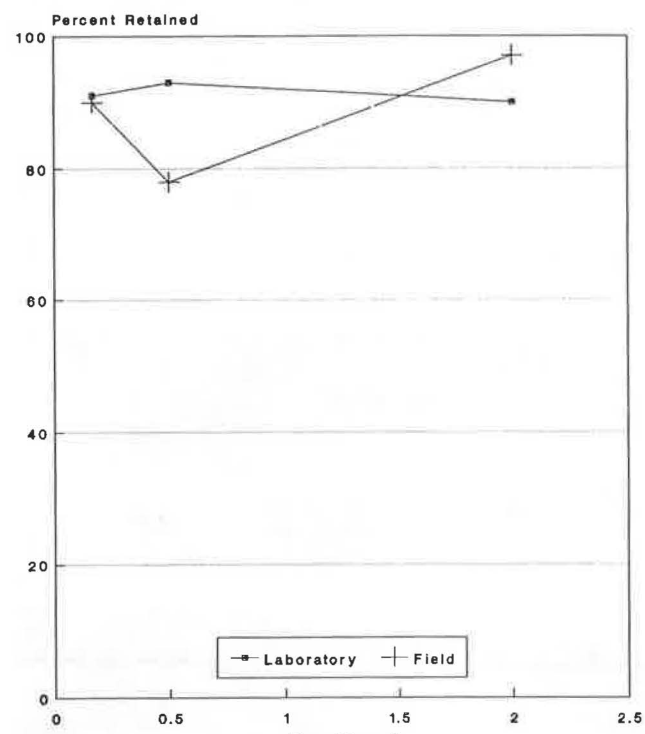

FIGURE 9 Percent retention of laboratory and field testing (styrene-based modified AR 4000, Section 8B).

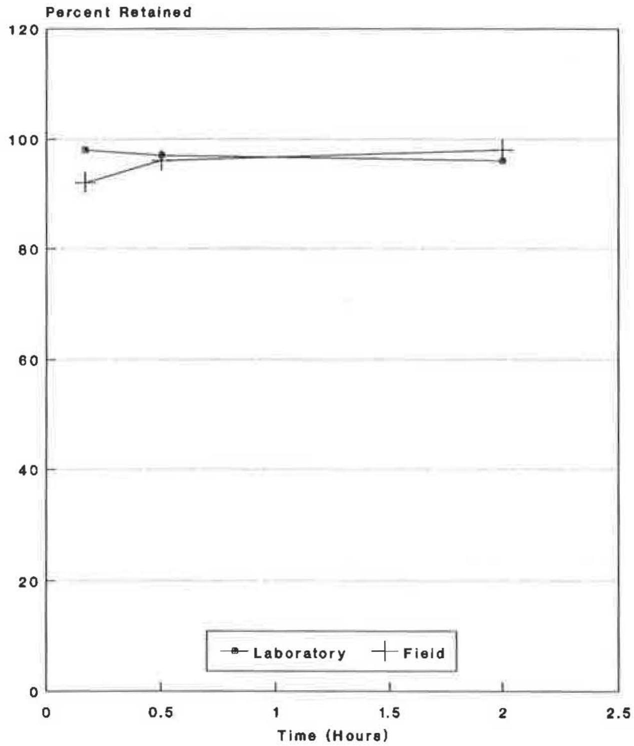

FIGURE 10 Percent retention of laboratory and field testing (styrene-based modified AR 4000, Section 8C).

TABLE 4 SUMMARY OF PAVEMENT EVALUATION 3 MONTHS AFTER CONSTRUCTION

Overall Condition	Aggregate Retention	Bleeding	Aggregate Embedment	Section	Product
9.4	9.4	10	36	7B	Kraton modified AR 1000
9.4	9.2	9.9	37	6A	EVA modified AC10
9.3	9.7	9.8	60	4	AR 1000 modified w/crumb rubber
9.2	9.0	9.9	32	7C	Kraton modified AR 1000
9.0	9.1	9.9	30	9B	AC10R
8.8	9.4	9.5	42	2	AR 2000
8.7	8.2	9.9	28	6C	EVA modified AC10
8.6	9.1	9.7	25	3A	AC20R
8.6	8.3	9.9	42	5A	AC10
8.0	7.7	9.7	33	5C	AC10
7.7	7.2	9.9	25	9C	AC10R
7.3	6.6	9.9	30	10	AR 2000
7.0	6.6	9.9	34	8B	Liquid styrene modified AR 4000
6.0	6.0	9.9	25	8C	Liquid styrene modified AR 4000

Aggregate retention followed the same ranking in performance as discussed in the overall condition ratings. These values ranged from 6.3 to 3.1 in the wheelpath and from 5.3 to 2.9 between the wheelpaths.

Aggregate embedment in the wheelpaths was 67 percent for Section 4, whereas sections 1–3 had moderate aggregate embedment in the wheelpaths with ratings ranging from 43 to 51 percent. Sections 5–11 had values between 23 and 41 percent. Between the wheelpaths aggregate embedment was slightly less than the values above.

Bleeding for all sections ranged from 8.3 to 9.6 between the wheelpaths and from 8.6 to 9.5 at the center line. The results are presented in Table 5.

Comparison of Pavement Evaluations

All of the binder sections looked fairly good after 3 months, but after the winter weather there were definite changes in 2 of the binders. The styrene-based polymer modified AR 4000 (8C) declined in overall condition from 6.0 to 2.4. Its aggregate retention also dropped from a 6.0 to a 2.9. The bleeding and aggregate embedment had little change. The average aggregate embedment was about 24 percent for this section. AR 2000 sections 10A and 10C also decreased in overall condition from 7.5 to 3.9 and from 7.0 to 3.8, respectively. Aggregate retention dropped from 7.8 to 4.8 in Section 10A and from 6.6 to 4.4 in Section 10C.

AR 2000 sections 2A and 2C, however, did not show significant drops in either the overall condition or the aggregate retention. Sections 2A and 2C had greater embedment depth, 38 and 51 percent, than sections 10A and 10C, which had 32 and 29 percent. The greater embedment probably increased the performance for both the overall condition and aggregate retention in sections 2A and 2C.

No significant changes occurred in the other sections between the 3- and 11-month evaluations for overall condition, aggregate retention, bleeding, or aggregate embedment. The relationship between the 3- and 11-month pavement evaluations is shown in Figure 11.

Percent Reflective Cracking

In each section the pre- and postconstruction crack maps closest to the evaluation site were used to determine the percent reflective cracking. These crack maps were located less than 400 ft from the evaluation site.

The least amount of cracking was found in Section 4 (AR 1000 modified with crumb rubber), which had 0 percent. Sections 5A and 5C (AC10) had slightly more cracking with values of 10 and 13 percent. Next were sections 2 and 3A (the AR 2000 section and the AC20R section) having 13 and 19 percent reflective cracking. Values of 19 and 51 percent were recorded in the Kraton-modified AR 1000 section (7C) and the Kraton-modified AR 1000 (7B). Moderate reflective cracking was found in the AC10R section (9C) and the EVA-modified sections (6A and 6C), which had corresponding percentages of 58, 75, and 83. The AC10R section (9B) had 90 percent reflective cracking, and the AR 2000 section (10) had 99 percent. Excessive reflective cracking was recorded in sections 8C and 8B (AR 4000 modified with a styrene-based polymer) with 100 and 113 percent reflective cracking (Table 6).

Laboratory Vialit-Time Series

Most of the binders had similar results over time with usually greater than 85 percent retention. The AR 2000 section (10) had slightly lower retention for subsection C after 10 min, but

TABLE 5 SUMMARY OF PAVEMENT EVALUATION 11 MONTHS AFTER CONSTRUCTION

Overall Condition	Aggregate Retention	Bleeding	Aggregate Embedment	Section	Product
9.3	9.3	9.4	67	4	AR 1000 modified w/crumb rubber
8.7	9.9	9.6	36	7B	Kraton modified AR 1000
8.6	8.9	8.9	44	3A	AC20R
8.1	8.1	9.5	41	7C	Kraton modified AR 1000
8.1	8.4	9.1	39	5A	AC10
8.1	8.2	9.3	33	9B	AC10R
7.9	8.5	9.4	39	6A	EVA modified AC10
7.7	7.6	9.0	43	2	AR 2000
7.3	8.1	9.3	36	8B	Liquid styrene modified AR 4000
7.3	8.0	9.1	37	5C	AC10
7.1	7.5	9.5	27	6C	EVA modified AC10
5.4	5.8	9.3	29	9C	AC10R
3.9	4.4	9.0	29	10	AR 2000
2.4	3.1	8.6	23	8C	Liquid styrene modified AR 4000

TABLE 6 PERCENT REFLECTIVE CRACKING IN EACH SECTION

% Reflective Cracking	Section	Product
0	4	AR 1000 modified with crumb rubber
10	5A	AC10
13	5C	AC10
13	2	AR 2000
19	3A	AC20R
19	7C	Kraton modified AR 1000
51	7B	Kraton modified AR 1000
58	9B	AC10R
75	6A	EVA modified AC10
83	6C	EVA modified AC10
90	9A	AC10R
99	10	AR 2000
100	8C	Liquid styrene modified AR 4000
113	8B	Liquid styrene modified AR 4000

all other tests were above 85 percent retention. Figures 9 and 10 show typical aggregate retention versus time.

The AR 2000 section (2) had aggregate retention greater than 95 percent for the 10-min, 30-min, and 2-hr tests. However, the AR 2000 section (10) had lower percent aggregate retention. These values ranged from 75 to 98 percent after 10 min, 98 percent after 30 min, and 90 to 95 percent after 2 hr. Asphalt and aggregate spread rates for these sections were similar. The AR 2000 section (2) had binder spread rates of 0.36, 0.37, and 0.46 gal/yd^2 and aggregate spread rates of 21, 25, and 21 lb/yd^2. Both subsections of the AR 2000 (10A and 10C) had binder spread rates of 0.36 gal/yd^2 and aggregate spread rates of 20 lb/yd^2.

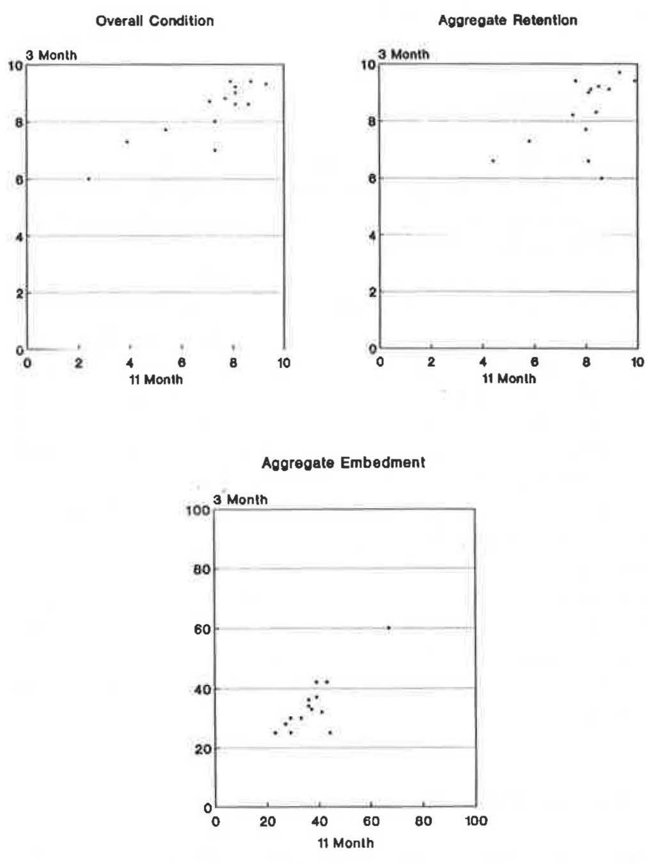

FIGURE 11 Comparison of 3-month and 11-month overall condition, aggregate retention, and aggregate embedment.

Laboratory Vialit-Temperature Series

The AR 1000 modified with crumb rubber section (4) and the EVA-modified AC10 (6A) had the best aggregate retention at 0°F with values of 98 and 92 percent, respectively. Aggregate retention percentages for the AC10 section (5A), the Kraton-modified AR 1000 section (7B), and the AC10R section (9B) were 88, 87, and 82. In the AC20R section (3A) aggregate retention dropped to 75 percent, and in the Kraton-modified AR 1000 (7C) aggregate retention was 67 percent. Aggregate retention for the AC10 section (5C) was fairly good at 60 percent, whereas the EVA-modified AC10 section (6C) was lower at 43 percent. The AR 2000 section (10), the styrene-based AR 4000 section (8B), and the AC10R section had aggregate retentions of 36, 34, and 33 percent, respectively. The least aggregate was retained in the AR 2000 section (2), which had 8 percent aggregate retention, and the styrene-based AR 4000 section (8C), which had 5 percent retention. The best and worst cases are shown in Figures 12 and 13.

Aggregate retention seems to be affected by the different gradations. The subsections constructed with the Nevada gradation (A and B subsections) have greater aggregate retention than the subsections of the same binder constructed with the Texas gradation (C and D subsections). Because the Texas gradation is more of a one-sized gradation than the Nevada gradation, more embedment depth is needed with the Texas gradation for adequate aggregate retention (Table 7).

CORRELATION BETWEEN LABORATORY AND FIELD TESTING

Laboratory Versus Field Vialit-Time Series

Field testing results are not available for the AR 2000 section (2) and the AC20R section (3). Some sections had less than

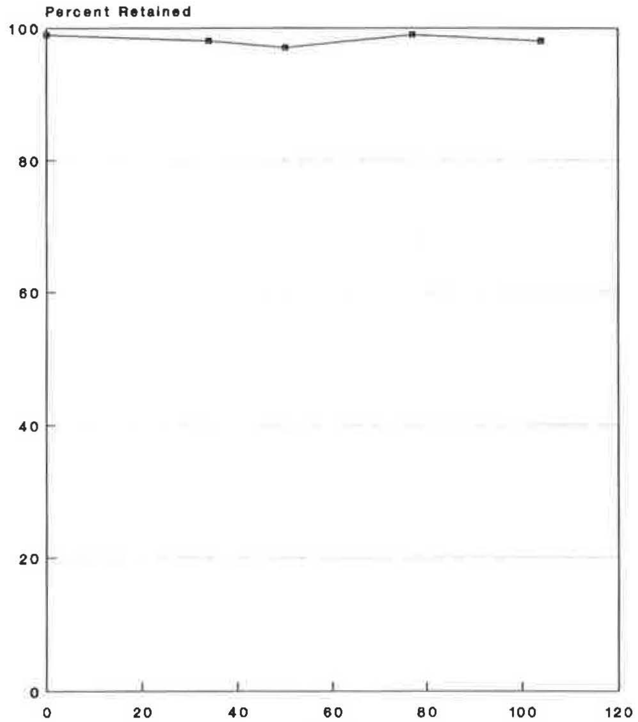

FIGURE 12 Percent retention of laboratory Vialit-temperature series (AR 1000 modified with crumb rubber, Section 4).

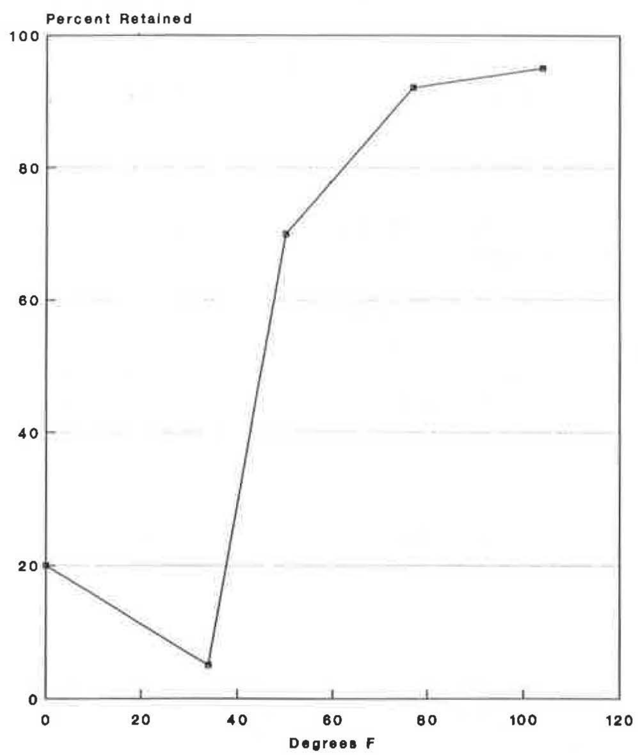

FIGURE 13 Percent retention of laboratory Vialit-temperature series (styrene-based modified AR 4000, Section 8C).

TABLE 7 LABORATORY VIALIT-PERCENT AGGREGATE RETENTION AT 0°F FOR EACH SECTION

% Aggregate Retention at 0°F	Section	Product
98	4	AR 1000 modified with crumb rubber
92	6A	EVA modified AC10
88	5A	AC10
87	7B	Kraton modified AR 1000
82	9B	AC10R
75	3A	AC20R
67	7C	Kraton modified AR 1000
60	5C	AC10
43	6C	EVA modified AC10
36	10	AR 2000
34	8B	Liquid styrene based AR 4000
33	9C	AC10R
8	2	AR 2000
5	8C	Liquid styrene based AR 4000

TABLE 8 COMPARISON OF OVERALL CONDITION AND PERCENT AGGREGATE RETENTION AT 0°F FOR EACH SECTION

Overall Condition	% Retention at 0°F	Section	Product
9.3	98	4	AR 1000 modified with crumb rubber
8.7	87	7B	Kraton modified AR 1000
8.6	75	3A	AC20R
8.1	88	5A	AC10
8.1	67	7C	Kraton modified AR 1000
8.1	82	9B	AC10R
7.9	92	6A	EVA modified AC10
7.7	8	2	AR 2000
7.3	34	8B	Liquid styrene modified AR 4000
7.3	60	5C	AC10
7.1	43	6C	EVA modified AC10
5.4	33	9C	AC10R
3.9	36	10	AR 2000
2.4	5	8C	Liquid styrene modified AR 4000

10 percent difference in aggregate retention between laboratory and field testing, with the higher values from the laboratory testing. These sections were the Kraton-modified AR 1000 section (7B and 7C), the styrene-based modified AR 4000 section (8C), the AC10R section (9C), and the AR 2000 section (10). The other binders had between 10 and 35 percent difference in aggregate retention between laboratory and field testing. These were the AR 1000 modified with crumb rubber section (4), the AC10 section (5), the EVA-modified AC10 section (6), the styrene-based modified AR 4000 section (8B), and the AC10R section (9B).

Some sections, such as the styrene-based modified AR 4000 section (8B), had little difference in aggregate retention between laboratory and field for the 10-min test. At 30 min there was a large difference in aggregate retention between laboratory and field. At 2 hr there was little difference in aggregate retention between laboratory and field testing (Figures 9 and 10). This was because of the large fluctuation of curing temperatures in the field. A sample made at 11:00 when the ambient temperature was 90°F and tested 30 min later after being stored in an automobile at 100°F will have less aggregate retention than a sample tested after 10 min.

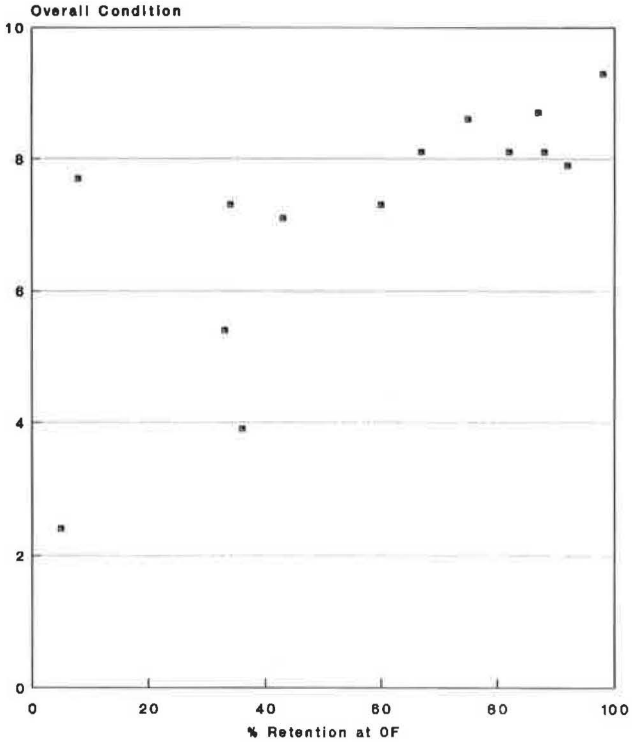

FIGURE 14 Correlation between percent retention at 0°F and overall condition.

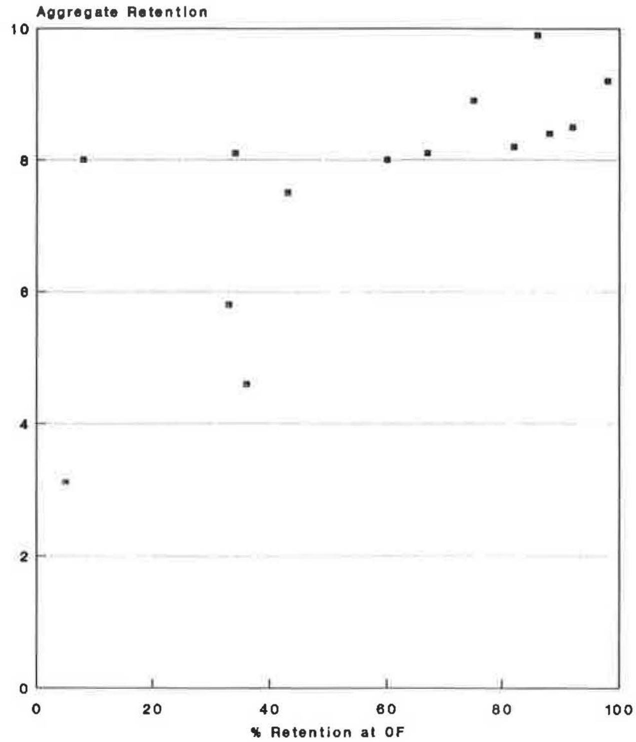

FIGURE 15 Correlation between percent retention at 0°F and aggregate retention.

TABLE 9 COMPARISON OF AGGREGATE RETENTION AT 11 MONTHS FROM PAVEMENT EVALUATIONS AND PERCENT AGGREGATE RETENTION AT 0°F FROM VIALIT TEST

Aggregate Retention at 11 months	% Retention at 0°F	Section	Product
9.9	86	7B	Kraton modified AR 1000
9.2	98	4	AR 1000 modified with crumb rubber
8.9	75	3A	AC20R
8.5	92	6A	EVA modified AC10
8.4	88	5A	AC10
8.2	82	9A	AC10R
8.1	67	7C	Kraton modified AR 1000
8.0	8	2	AR 2000
8.0	60	5C	AC10
7.5	43	6C	EVA modified AC10
8.1	34	8B	Liquid styrene modified AR 4000
5.8	33	9C	AC10R
4.6	36	10	AR 2000
3.1	5	8C	Liquid Styrene modified AR 4000

Laboratory Vialit-Temperature Series Versus Overall Condition

Aggregate gradation influenced the overall condition rating in some binders. Sections constructed with the Nevada gradation had better overall condition ratings than those constructed with the Texas gradation. The Texas gradation is more of a one-sized aggregate than the Nevada gradation. It is believed that greater embedment depths (thus greater binder quantities) are needed to prevent aggregate loss in the Texas gradation sections.

There is a correlation between the percent retention at 0°F and the overall condition. To achieve an overall condition rating of 8.0 or better the aggregate retention at 0°F from the laboratory Vialit test should be greater than 60 percent (Table 8 and Figure 14).

Laboratory Vialit-Temperature Series Versus Aggregate Retention

Table 9 and Figure 15 show the relationship between aggregate retention and percent retention at 0°F for the upper values of aggregate retention. For aggregate retention values of 8.0 or better, aggregate retention at 0°F determined from the laboratory Vialit should be greater than 60 percent.

Laboratory Vialit-Temperature Series Versus Percent Reflective Cracking

The AR 2000 section (2) and the AR 2000 section (10) had the same performance in the laboratory but different performance in the field. Aggregate embedment depth for the AR 2000 section (2) was about 47 percent, whereas the AR 2000 section (10) was about 30 percent. This is probably the reason that the AR 2000 section (2) had considerably less reflective cracking than the AR 2000 section (10) (Table 10).

Effect of Material, Design, and Construction Variables on Seal-Coat Performance

REYNALDO ROQUE, DAVID ANDERSON, AND MATTHEW THOMPSON

A field study conducted at the Pavement Durability Research Facility of the Pennsylvania Transportation Institute and on Pennsylvania Route 64 to determine the effect of specific construction, traffic, and materials variables on the performance of bituminous seal coats is described. The condition of the existing surface (worn or leveled), emulsion application rate, rolling patterns, time between construction operations and opening to traffic, and polymer modification were among the study variables. Accelerated traffic was applied to the sections for 1 year, and the performance, skid resistance, visual evaluations, and mean texture depth were documented. Design and construction variables were found to diminish the other study variables. Conclusions are presented that relate to the different phases of a seal-coat operation, including surface preparation, materials selection and specification, seal-coat design, construction procedures, and quality control.

The application of a seal coat is recognized as one of the most efficient and economical maintenance techniques used to extend pavement life. During 1987, the Pennsylvania Department of Transportation (PennDOT) applied seal coats to more than 5,000 mi of roadway, requiring more than 14 million gal of asphalt emulsion. Specifications, policies, guidelines, and a seal-coat design method have been developed by PennDOT for district maintenance personnel (*1,2*). A number of seal-coat design procedures have been developed (*3*). More recently, procedures have been published by McLeod and Holmgreen (*4–6*). The procedure adopted by PennDOT has been largely patterned after the one developed by McLeod (*7*). In spite of this information and extensive training, the service life of some seal coats has been shorter than desirable, often resulting in a severe loss of skid resistance through flushing of the surface (*2*). Epps et al. wrote a review of the factors that influence the longevity of seal coats (*8*).

Although considerable attention has been given to design procedures, much less has been given to materials, construction, and traffic control variables. Therefore, this study was initiated to investigate the effect of these variables, as well as the effect of design variables on seal-coat performance. The study was conducted at the Pavement Durability Research Facility of the Pennsylvania Transportation Institute (PTI) and on a section of Pennsylvania Route 64 near the Pennsylvania State University. The test sections were monitored during construction and for 1 year after construction to evaluate their relative performance. An extensive laboratory testing program also was conducted to characterize the aggregate and binders used in the study.

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EXPERIMENT DESIGN

The research plan was divided into two experiments: a primary construction-related experiment and a secondary material-related experiment. Elements of each experiment were incorporated both at the Pavement Durability Research Facility and on Route 64. A description of each experiment is given below.

Primary Construction Variable Experiment

Four variables were included in this experiment:

1. Pavement condition (two levels): worn ID-2 or new ID-2 leveling;
2. Emulsion rate (two levels): high or low;
3. Number of roller passes (two levels): 1 or 3;
4. Traffic control (delay before application of traffic) (three levels): 2, 8, or 24 hr.

Twenty-four test sections, each approximately 50 ft long, were used to accommodate each of the 24 variable combinations at the Pavement Durability Research Facility. The ID-2 mixture is a typical surface course mixture used by PennDOT; the maximum size of the aggregate is ⅜ in.

Secondary Material Variable Experiment

Three variables were included in this experiment: emulsion type, aggregate gradation, and age of leveling course. A summary of the characteristics of each of the 14 test sections in this experiment is presented in Table 1.

Route 64 Experiment

Four variables were introduced in 18 test sections on Route 64: emulsion type, number of roller passes, existing pavement surface condition, and time of traffic control. Each test section, which was approximately 2,250 ft long, is described in Table 2.

TRAFFIC

Traffic at the Pavement Durability Research Facility was started in August 1988 and continued through August 1989. It consisted of a tractor pulling two single-axle trailers loaded to the

TABLE 10 COMPARISON OF PERCENT REFLECTIVE CRACKING AND PERCENT RETENTION AT 0°F FROM VIALIT TEST FOR EACH SECTION

% Reflective Cracking	% Retention at 0°F	Section	Product
0	98	4	AR 1000 modified with crumb rubber
10	88	5A	AC10
13	8	2	AR 2000
13	60	5C	AC10
19	75	3A	AC20R
19	67	7C	Kraton modified AR 1000
51	87	7B	Kraton modified AR 1000
58	33	9C	AC10R
75	92	6A	EVA modified AC10
83	43	6C	EVA modified AC10
90	82	9A	AC10R
99	36	10	AR 2000
100	5	8C	Liquid styrene modified AR 4000
113	34	8B	Liquid styrene modified AR 4000

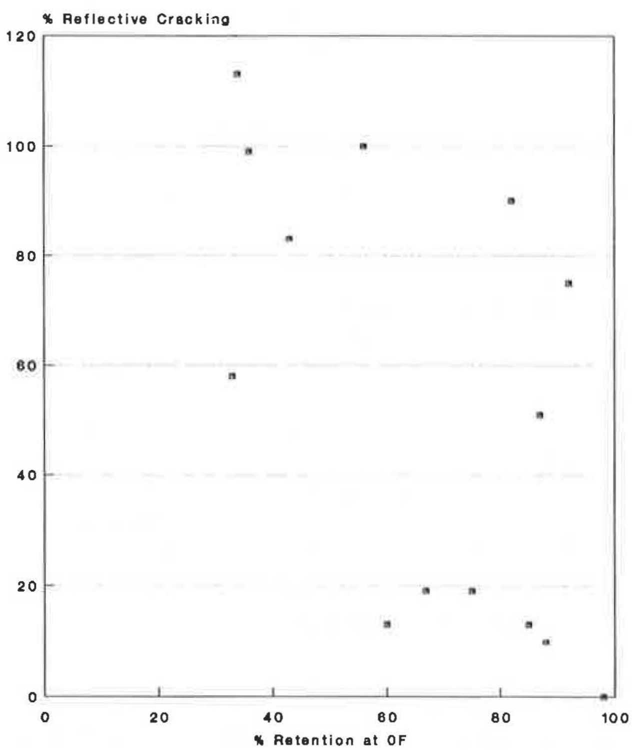

FIGURE 16 Correlation between percent retention at 0°F and percent reflective cracking.

There is no correlation between aggregate retention at 0°F and percent reflective cracking. However, there is also no correlation between overall condition and reflective cracking (Figures 16 and 17). These comparisons suggest that reflective cracking is not just a function of the binder-aggregate system but may be dependent on the brittleness of the binder.

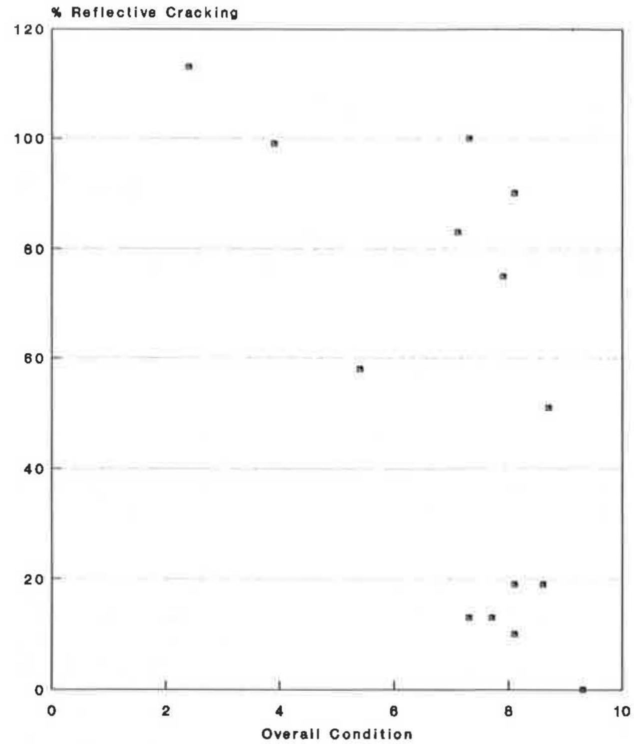

FIGURE 17 Correlation between overall condition and percent reflective cracking.

CONCLUSIONS

1. The field Vialit-time series testing did not correspond to laboratory Vialit-time series testing because of the variation in curing temperature of the field samples.
2. The laboratory Vialit-temperature series can detect the effects of aggregate gradation on different binders.
3. Aggregate retention of the sample cured at 0°F is a good indicator of overall chip-seal performance. Ratings of 8.0 or greater are likely for overall condition and aggregate retention if the percent aggregate retention for the laboratory sample cured at 0°F is greater than 60 percent.

REFERENCES

1. *Standard Specifications.* Nevada Department of Transportation, Carson City, 1984.
2. *Standard Specification for Construction of Highways, Streets, and Bridges.* Texas State Highway and Public Transportation Department, Austin, 1972.

Publication of this paper sponsored by Committee on Characteristics of Bituminous-Aggregate Combinations To Meet Surface Requirements.

TABLE 1 SUMMARY OF CHARACTERIZATION FOR EACH TEST SECTION OF SECONDARY MATERIAL VARIABLE EXPERIMENT AT PAVEMENT DURABILITY FACILITY

Section No.	Emulsion[1] Type	Aggregate Gradation	No. of Roller Passes	Pavement Condition
S-1	E-3	Single-Size	1	Worn ID-2
S-3	E-3	Graded	3	Worn ID-2 Leveling[2]
S-4	E-3	Graded	1	Worn ID-2 Leveling
S-5	E-3	Graded	1	New ID-2 Leveling
S-6	E-3	Graded	1	New ID-2 Leveling
S-7 (S-5)[3]	Neoprene	Graded	1	New ID-2 Leveling
S-8	E-3	Graded	1	New ID-2 Leveling
S-9 (S-6)	SBR	Graded	1	New ID-2 Leveling
S-10	E-3	Graded	1	New ID-2 Leveling
S-11 (S-8)	SBS 1	Graded	1	New ID-2 Leveling
S-12 (S-10)	SBS 2	Graded	1	New ID-2 Leveling
S-13	E-3	Single-Size	3	New ID-2 Leveling
S-14	E-3	Single-Size	1	New ID-2 Leveling

[1] E-3 is an ASTM CRS-2 emulsion
SBR is a styrene-butadiene-copolymer-modified emulsion (2.8 percent)
SBS-1 is a styrene-butadiene-styrene-modified emulsion from manufacturer 1 (2.8 percent)
SBS-2 is a styrene-butadiene-styrene-modified emulsion from manufacturer 2 (3.0 percent)
Neoprene is a neoprene-modified emulsion (2.8 percent)

[2] Worn ID-2 Leveling was placed in 1987 with no traffic until after the seal coat was applied.

[3] Numbers in parentheses correspond to the control for the sections that were constructed with modified binders.

TABLE 2 SUMMARY OF CHARACTERISTICS OF SEAL-COAT TEST SECTIONS CONSTRUCTED ON ROUTE 64

Section No.	Emulsion Type	No. of Roller Passes	Traffic Control (hours)	Existing Pavement Condition
Control North	AC-10 Control	1	2	3-Year Seal Coat
Control South	AC-10 Control	1	4	3-Year Seal Coat
1a	Neoprene	1	2	3-Year Seal Coat
1b	Neoprene	2	2	3-Year Seal Coat
1c	Neoprene	2	4	Thin Overlay
1d	Neoprene	1	4	Thin Overlay
2a	SBR	1	4	3-Year Seal Coat
2b	SBR	2	4	3-Year Seal Coat
2c	SBR	2	2	Thin Overlay
2d	SBR	1	2	Thin Overlay
3a	SBS	1	4	Thin Overlay
3b	SBS	2	4	Thin Overlay
3c	SBS	2	2	3-Year Seal Coat
3d	SBS	1	2	3-Year Seal Coat
4a	SBS	1	2	Thin Overlay
4b	SBS	2	2	Thin Overlay
4c	SBS	2	4	3-Year Seal Coat
4d	SBS	1	4	3-Year Seal Coat

legal axle limit of 22,400 lb per axle. Two 2-person crews operated the truck approximately 16 hr a day. A total of 73,400 truck passes were applied during the monitoring period.

The traffic on Route 64 consisted of trucks and automobiles. The average daily traffic (ADT) for Route 64 was reported by PennDOT as 2,600 vehicles per day per lane. The traffic consisted of 9 percent trucks.

EMULSIONS

A standard E-3 (ASTM CRS-2) emulsion was used to construct all test sections except the polymer-modified sections. The emulsion met all of PennDOT's specification requirements. The properties of the base asphalt cement used to manufacture the E-3 emulsion satisfied the requirements for an AC-10 asphalt cement. The four modifiers used in the secondary experiment and on Route 64 are described in Table 1. A comprehensive set of conventional and nonconventional laboratory tests was performed on all of the emulsions used on the project, as well as on the emulsion residues. The results of the tests may be found elsewhere (*8,9*).

AGGREGATE PROPERTIES

The aggregate supplied for this project, selected by PennDOT personnel, is a heterogeneous siliceous, glacial gravel produced at the Fairfield Township operation of Lycoming Silica Sand Company. The results of tests performed on the aggregate, including gradations for both the graded material and the single-sized stone, follow.

	Percent Passing	
Sieve Size	*Graded*	*Single-Sized*
½	100	100
⅜"	89	92
#4	21	8
#8	8	4
#16	4	3.5
#30	4	–
#200	1	1

Additional data are as follows:

- Hydrometer analysis (percent finer than given size expressed as percent of total aggregate:
  - –.025 mm, .55;
  - –.008 mm, .29; and
  - –.001 mm, .13;
- Flakiness index (average least dimension): .2 in.;
- Los Angeles abrasion (percent wear): 30 (the maximum is 40);
- Crush count (percent crushed faces): 94;
- Bulk specific gravity: 2.62; and
- Absorption: 2.15 percent.

The aggregate meets the grading requirements for a Pennsylvania IB stone (AASHTO 8) that is to be used for seal-coat work. All other PennDOT specification criteria were met by this aggregate.

PRECONSTRUCTION EVALUATION AND SEAL-COAT DESIGN

The rut depths and surface texture of the pavement sections at the Pavement Durability Research Facility and on Route 64 were evaluated before seal-coat construction. At the Pavement Durability Research Facility, the rut depths for the worn sections that did not receive a leveling course ranged from 0.1 to 1.05 in. Rut depths generally ranged from ¼ to ½ in. on Route 64. The surface texture of the worn and leveled sections was evaluated visually. All of the surfaces were categorized into one of the following five categories:

- Category 1: flushed asphalt surface;
- Category 2: smooth, nonporous surface;
- Category 3: slightly porous, oxidized surface;
- Category 4: slightly pocked, porous, and oxidized surface; and
- Category 5: badly pocked, porous, and oxidized surface.

These are the categories listed in the PennDOT Seal Coat Design Method (*7*).

The emulsion and aggregate application rates for all surfaces were determined using the procedure described in PennDOT Bulletin No. 27 (*7*). The PennDOT procedure uses the existing pavement condition, spread modulus (D_{50}) of the aggregate, ADT, and absorption capacity of the aggregate as the variables necessary to calculate the application rates. Aggregate whip-off for this project was assumed to be 10 percent. A summary of the seal-coat designs used in this project is presented in Table 3.

DOCUMENTATION OF CONSTRUCTION ACTIVITIES

During the construction at the Pavement Durability Research Facility and Route 64, PTI personnel documented the following activities:

- Emulsion application rate;
- Aggregate application rate;
- Quantity of whip-off aggregate;
- Environmental conditions before, during, and after construction (including air temperature, pavement temperature, relative humidity, cloud cover, and wind conditions);
- Emulsion application temperatures;
- Time between emulsion and aggregate application;
- Time between aggregate application and rolling;
- Number of roller passes; and
- Time between rolling and application of traffic.

All construction activities and equipment calibration were under the control of PennDOT personnel. No attempt was made to alter the normal construction techniques, and, to the maximum extent possible, the experimental aspects of the project were designed to minimize any disturbance of normal construction procedures. Obviously, a large amount of data was obtained, making it impossible to report even a summary of the measurements herein. Complete documentation of the various methods used to obtain the measurements, as well as the measurements themselves, can be found elsewhere (*8*).

TABLE 3 SUMMARY OF SEAL-COAT DESIGN

Experiment	Aggregate	Surface	Emulsion Application Rate (gal/yd²)	Aggregate Spread Rate (lb/yd²)
Pavement Durability Research Facility[1]		Worn ID-2	0.27 (low)	22
		New ID-2	0.35 (high)	22
	Single-sized stone[2]	Worn ID-2	0.27 (low)	20
		New ID-2	0.35 (high)	20
Route 64[3]	Graded	Worn ID-2	0.30	18
		Worn seal coat	0.30	18

[1]Determined using PennDOT design procedure and the following assumptions:

D_{50}:	0.268 in
Loose Unit Weight:	90.4 lb/ft³
ADT:	>2,000 vehicles/day
Absorptive Aggregate:	Yes
Bitumen Type:	Emulsion
Surface Condition:	
Worn ID-2:	Category 2, smooth, nonporous surface
New ID-2 Leveling:	Category 3, lightly-pocked, porous, and oxidized surface
Whip-off:	Use 10 percent

[2]Aggregate spread rate for the single-sized stone was selected on the basis of engineering judgment.

[3]All values selected by PennDOT personnel on the basis of local experience.

PERFORMANCE INDICATORS

The following techniques were used to monitor the performance of the seal-coat test sections at regular intervals:

- Sandpatch method (ASTM E965-83): four per section in the outer wheelpath; monthly.
- Skid resistance (ASTM E274-85): five per section in the wheelpaths; monthly.
- Visual evaluations (Texas State Department of Highways and Public Transportation Method): three evaluators per section assigned an overall rating, and ratings for bleeding and aggregate retention; monthly.
- Geotextile pads: Several were placed in each test section to evaluate aggregate loss with time; method failed (textiles could not be recovered and seemed to affect performance).

A detailed explanation and evaluation of each of these techniques can be found elsewhere (*8,10*). The mean texture depth was found to give the best indication of expected seal-coat life and an excellent parameter for comparing well-constructed seal coats on a relative basis (*8,10*).

DATA ANALYSIS: PRIMARY CONSTRUCTION VARIABLE EXPERIMENT

Visual observations during 13 months of accelerated traffic at the Pavement Durability Research Facility indicated the following:

- On the basis of the general appearance of the seal-coat sections at any given time, the various seal-coat sections clearly performed differently (i.e., some sections definitely outperformed others).
- The macrotexture of all the sections decreased with time and traffic, apparently as a result of aggregate wear and embedment.
- All sections maintained adequate macrotexture and skid resistance throughout the experiment.
- Little chip loss was observed in the wheel tracks of any of the sections.
- Chip loss caused by snowplows was severe between the wheel tracks of all the sections. It should be noted that traffic was highly channelized at the Pavement Durability Research Facility so that there was no traffic between the wheel track to help set the aggregate in that area.
- The first failures at the Pavement Durability Research Facility were caused by shoving of the leveling course mixtures during hot weather. The failures were at least partially caused by a lack of bond between the leveling course and the original surface. Although some of these sections were repaired, these types of failures eventually led to the stoppage of traffic.
- No failures resulted from a complete loss of texture or skid resistance. However, some sections no doubt were close to losing skid resistance, others appeared to have excellent texture, and a number of sections were somewhere between.

As illustrated in Figure 1, the mean texture depth (MTD) measurements for all the test sections showed a consistent and similar relationship when plotted as a function of time.

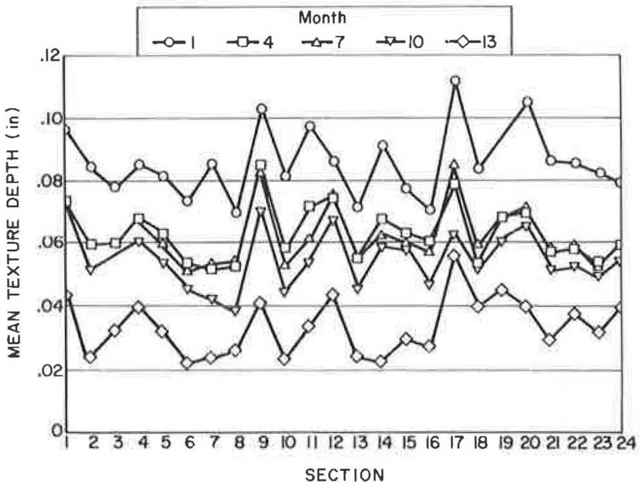

FIGURE 1 Mean texture depths at different times: primary experiment.

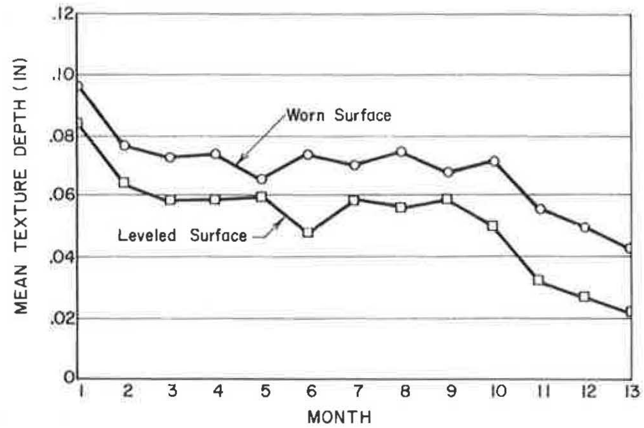

FIGURE 2 Mean texture depth as a function of time for worn and leveled surfaces, low emulsion application rate: typical.

This figure, which shows a comparison of the MTD measurements for all sections at different times, indicates that MTD dropped fairly sharply for all sections after Month 1, but after Month 4, the MTD reduced more slowly. Another significant drop in MTD was observed for all sections after Month 10. The sharp drops occurred during hotter months when the aggregate embedment rate was higher.

Because of the consistency of the MTD measurements and the significant differences observed in MTD between the different sections, a more detailed analysis of the MTD measurements was performed to evaluate the effects of the design and construction variables on seal-coat performance. Because higher MTD implies greater macrotexture, and greater retained macrotexture results in prolonged skid resistance, it appeared reasonable to use MTD as a performance criterion. This is particularly valid for the seal-coat sections in this experiment because they exhibited a minimal amount of chip loss in the wheel tracks. The analysis is presented next.

Effect of Surface Type

Analysis of the MTD measurements indicated that for the lower emulsion application rates used in this investigation, the seal coats constructed on leveled surfaces had consistently lower retained MTD than did seal coats on worn surfaces. This is illustrated in Figure 2, which shows a comparison of MTD versus time between a seal-coat section constructed on a leveling course and one constructed on a worn surface with all other factors held constant. The same trend was evident in similar comparisons for the five other pairs of sections constructed with low emulsion rates and for which the only difference was surface type. This result was expected because the low emulsion application rate was the design rate for the worn surface and considered too low for the leveled surface. Measurements obtained during construction indicated that the lower emulsion rates used in this investigation resulted in lower initial aggregate retention on the leveled surfaces than on the worn surfaces, which partly explains the lower MTDs.

Comparisons made between seal-coat sections on leveled and worn surfaces but constructed using high emulsion application rates indicated that there was no clear difference in retained MTD. Approximately the same amount of aggregate was retained on both surfaces when the higher emulsion rate was used.

The highest retained MTDs were obtained on worn surfaces. In other words, the retained MTD was generally higher on worn surfaces, even when the initial chip retention was comparable for both types of surfaces. This is probably because leveled surfaces are softer than the older worn surfaces and, therefore, allow more embedment and reorientation of the aggregate.

These results seem to indicate that a leveling course should not be applied before a seal coat unless it is absolutely necessary to overcome a severe rutting problem. Leveling courses require additional emulsion for adequate chip retention and, on the basis of the results obtained, do not appear to offer any benefits in terms of seal-coat performance.

Effect of Emulsion Application Rate

This variable appeared to control the retained mean texture depth for both the worn and the leveled surfaces for the range of emulsion rates used in this investigation. In the case of worn surfaces, even the lowest emulsion application rates used were sufficient for adequate initial chip retention. Additional emulsion on the worn surfaces served only to reduce the macrotexture of the seal-coat surface. For leveled surfaces, it was found that the amount of aggregate retained initially was related to the emulsion application rate used. The lower emulsion application rates were not enough for adequate initial chip retention on some of the leveled surfaces.

Figure 3 shows a comparison of MTD versus time between two sections constructed on worn surfaces: one with a high emulsion application rate, the other with a low application rate. All other factors were constant for these sections. The figure shows that the section with the lower emulsion application rate had the higher retained MTD. Similar comparisons between seal coats on worn surfaces and for which the only difference was the emulsion application rate showed the same trend. Statistical analyses performed on MTD measurements on worn surfaces not only confirmed that emulsion application rate had a significant effect on retained MTD, but also in-

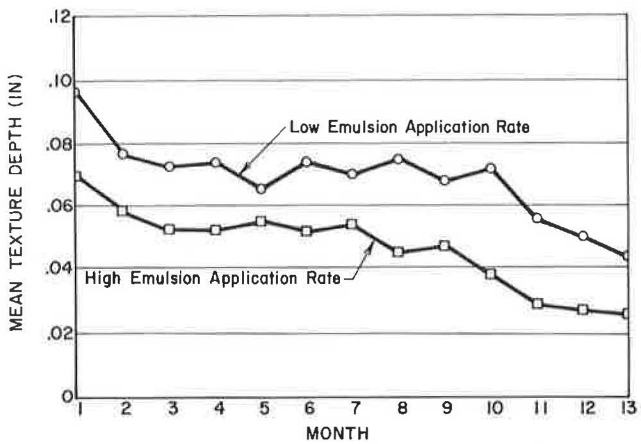

FIGURE 3 Mean texture depth as a function of time for low and high emulsion rates: worn surfaces, Month 13.

dicated that the effect of this factor overwhelmed the effect of all other factors in this experiment. Regression analyses were performed using MTD as the dependent variable and the following factors as independent variables:

- Emulsion application rate (measured),
- Number of roller passes,
- Traffic control (actual time after compaction before traffic was allowed on the seal coats),
- Weather during construction,
- Rut depth of the existing surface, and
- Interactions between these factors.

A forward stepwise multilineal regression procedure was used to analyze the MTD data for each month. For each month, the analyses indicated that the measured emulsion application rate was the only factor necessary to account for the observed differences in retained MTD between the seal-coat sections constructed on worn surfaces. The following regression model was obtained by analyzing the MTD measurements from Month 8:

$$\text{MTD} = 0.096 - 0.125*\text{EAR} \qquad R^2 = 0.71 \qquad (1)$$

where MTD is the mean texture depth (in.) and EAR is the measured emulsion application rate (gal/yd^2).

A better model was not obtained, even when other factors were included. Apparently, for the range of emulsion application rates applied on the existing surfaces (0.23 to 0.41 gal/yd^2) in this investigation, the effect of emulsion application rate overwhelmed the effects of all other factors.

This finding appears to indicate that the lower emulsion application rates were the most appropriate for the worn surfaces tested. The lower rates resulted in adequate chip retention and the highest levels of retained MTD. It is unclear whether emulsion application rates lower than those used would be more appropriate for these surfaces. Because the lower emulsion application rate used in this investigation corresponded to the design emulsion rate for the existing surface (according to PennDOT's design procedure), this finding also indicates that there appears to be no evidence to change the existing design procedures for the worn surfaces and the aggregate tested in this investigation.

A clear relationship was not found between the emulsion application rate and the retained MTD on leveled surfaces. However, separate regressions performed using aggregate retained as the dependent variable indicated that the amount of aggregate initially retained on the leveled surfaces was related to the emulsion application rate. The amount of aggregate retained initially as determined from measurements during construction was used as the dependent variable, and the following factors were included as independent variables in the regression:

- Emulsion application rate (measured),
- Number of roller passes,
- Weather during construction,
- Rut depth of the existing surface, and
- Interactions between these factors.

A forward stepwise multilineal regression analysis was used, and the following regression model was obtained:

$$\text{AGG} = 10.8 + 21.1*\text{EAR} + 2.0\,TC3 \qquad R^2 = 0.72 \qquad (2)$$

where

AGG = lb/yd^2 of aggregate retained on the leveled surface;
EAR = measured emulsion application rate (gal/yd^2); and
*TC*3 = a dummy variable corresponding to the hotter, dryer, and windier construction day.

For the range of emulsion application rates used on the leveled surfaces (0.23 to 0.41 gal/yd^2), the lower emulsion rates apparently were not sufficient for adequate initial chip retention. Greater initial chip retention should result in greater retained MTD and improved long-term performance. Therefore, the higher emulsion rates used in this investigation apparently were most appropriate for the leveled surfaces. Because, according to PennDOT's design procedure, the higher emulsion application rate used corresponded to the design emulsion rate for the rougher, leveled surface; this finding also indicates that no evidence to change the existing design procedure for the leveled surfaces and the aggregate tested in this investigation is apparent.

Other Factors

The number of roller passes seemed to have no effect on the retained MTD of the seal coats constructed on either worn or leveled surfaces. No relationships were found, either from basic plots or from regression analyses, between the number of roller passes and the retained MTD of the seal-coat sections.

The level of traffic control also seemed to have no effect on the retained MTD of the seal-coat sections constructed on either worn or leveled surfaces. Again no relationships were found, either from basic plots or from regression analyses, between traffic control and the retained MTD of the seal-coat sections.

Of the other factors considered in the analysis (weather, rut depth, aggregate retained, and in-place aggregate gradation), none were found to have a significant effect on the retained MTD of the seal coats on existing surfaces or leveled surfaces. As mentioned earlier, the amount of aggregate retained on the leveled surfaces was found to be related to weather and emulsion rate. Apparently, a greater amount of aggregate was retained on the hotter, dryer, windier construction day. This is logical, because these conditions are conducive to the emulsion breaking faster. This would also appear to indicate that aggregate retention may be reduced in areas that are shaded.

DATA ANALYSIS: SECONDARY MATERIAL VARIABLE EXPERIMENT

Both the visual ratings and the measurements of mean texture depth reflected the lack of variation in performance among the sections in the modifier experiment. After 13 months of accelerated traffic the visual evaluators observed little difference in chip retention or bleeding and flushing among these sections. Also, the difference between the sections with the highest and lowest retained MTD after 13 months was only 0.017 in., compared with a difference of 0.034 in. observed in the primary experiment.

Statistical analyses performed on the retained MTD after 13 months of traffic confirmed that for each of the four modifiers, no significant difference existed between the MTD of the modified section and the MTD of its corresponding control section. *T*-tests were performed using an error term determined from the control sections.

In general, all of the MTD versus time relationships for the modified sections closely resembled those observed for the sections in the primary experiment. This may indicate that the pattern of aggregate wear and embedment for the modified sections was the same as for the unmodified sections. This also appears to indicate that the modifiers used in this investigation did not help to prevent embedment during hot weather.

On the basis of retained MTD measurements, no evidence is apparent to indicate that the single-sized stone sections outperformed comparable sections constructed with graded aggregate. Figure 4 shows a comparison of MTD versus time between comparable single-sized stone and graded aggregate sections constructed on worn surfaces. There is no discernible difference between the two. The results shown for the graded aggregate sections in Figure 4 were the average of the MTD measurements for sections 1 and 4 from the primary experiment.

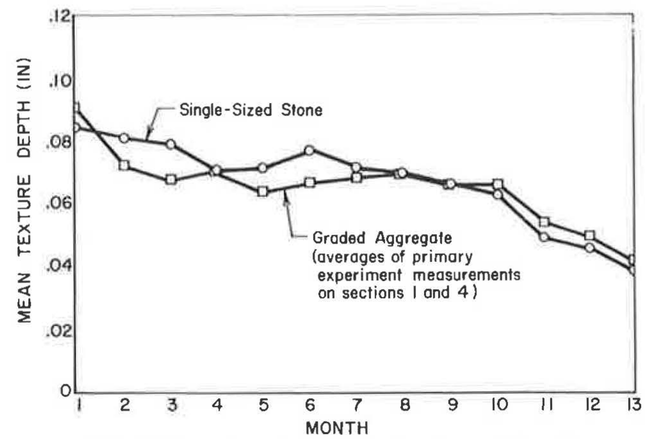

FIGURE 4 Mean texture depth as a function of time for single-sized stone section and control sections on worn surfaces.

DATA ANALYSIS: ROUTE 64 EXPERIMENT

The results obtained on Route 64 confirmed the findings of the experiments at the Pavement Durability Research Facility. Analyses of MTD measurements indicated that roller passes had no effect on retained MTD. Comparisons of MTD versus time among sections constructed using one and two roller passes and for which all other factors were constant clearly indicated that roller passes had no effect on retained MTD.

Finally, the relationship of MTD versus time for the modified and control sections on Route 64 exhibited the same trend that was observed for similar sections at the Pavement Durability Research Facility. This again indicates that the modifiers were unsuccessful in preventing the embedment during hot weather.

SUMMARY AND CONCLUSIONS

On the basis of the findings of this investigation, a number of conclusions are presented relating to each of the different phases of a seal-coat operation.

Surface Preparation

Worn surfaces should not be leveled before applying a seal coat unless rutting or surface roughness or both are so severe that they cause a safety or maintenance problem.

For the materials and surfaces tested in this investigation, it was found that rut depths up to 1 in. had no influence on the constructibility or performance of seal coats. Also, seal coats constructed on worn surfaces resulted in higher retained mean texture depths and longer expected life.

Materials Selection and Specifications

1. Good seal coats can be produced using aggregates and emulsions that meet PennDOT's existing specification (3/8-in. maximum size graded aggregate) if emulsion application rates and aggregate application rates are closely controlled in the field.
2. For the aggregate used in this investigation, there seemed to be no advantage in using single-size stone over the graded aggregate.
3. On the basis of the results of this study, polymer-modified emulsions are not warranted. However, their consideration should not be discontinued, particularly in which cases in which the need for early chip retention and conditions such as intersections and corners are encountered.

Design

PennDOT's design charts give reasonable estimates of the most appropriate emulsion application rates for the graded aggregate tested in this investigation.

Construction

1. When 8-ton pneumatic rollers are used, no more than one roller pass needs to be specified for proper seal-coat compaction.
2. No more than 2 hr of traffic control needs to be specified after construction before a seal coat is open to traffic.
3. It was unclear whether traffic control of less than 2 hr can be allowed. Therefore, for lack of more detailed information, 2-hr traffic control should be the minimum allowed before opening a seal coat to traffic.

Quality Control

Emulsion application rate and amount of aggregate retained were found to be the most important factors governing seal-coat performance. Therefore, both the distributor and the chip spreader must be properly calibrated.

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High-Traffic Chip-Seal Construction: The Tulsa Test Road

SCOTT SHULER

Chip-seal coats are used to extend pavement service life by reducing water and air infiltration and improving skid resistance. Application of chip seals is usually limited to low-traffic volume facilities because of unknown cost-effectiveness, vehicle damage caused by stones, inattention to proper principles, traffic disruption, and inappropriate design procedures. The causes of the problems that discourage use of chip-seal coats on high-traffic-volume pavements were investigated in this research project such that wider use of this potentially cost-effective construction process can developed. The first of four experimental field projects designed to demonstrate the potential use and effectiveness of chip-seal applications on high-traffic-volume asphalt concrete pavements is described. The successful construction of the first of these full-scale experiments and the several types of chip-seal treatments placed are documented. Methods used for calibration of equipment, traffic control, design, and construction processes essential for successful application of chip seals on high-traffic pavements are described. In addition, a promising new version of a laboratory test is described as a method of predicting adhesive qualities of chip-seal binders in advance of construction.

A full-scale test pavement was constructed by maintenance personnel of Division 8, Oklahoma Department of Transportation (DOT) from October 11 to 13, 1989 between 46th and 56th streets on US-169, Mingo Valley Expressway, in Tulsa, Oklahoma. A schematic of the test road location is shown in Figure 1.

The experimental sections were placed in the north- and southbound lanes of US-169 starting at 56th street and proceeding south in the northbound lane on October 11. The work was completed by starting at 46th street and proceeding north in the southbound lane. All work was performed from 9:00 a.m. to 4:30 p.m. each day.

Traffic on US-169 was measured by Oklahoma DOT at the rate of 37,300 average daily traffic (ADT) for four lanes. This volume is more than double the minimum criteria of 7,500 vehicles per day per direction on four lanes established as high traffic for this research project.

Weather three weeks before construction consisted of snow showers and cool to cold daily and nighttime temperatures. However, fair to warm temperatures were forecast for the week of October 10, and the decision to build the experiment was made because of the "realistic" conditions, that is, placing a chip seal on a distressed asphalt pavement immediately before winter in an attempt to forego further deterioration and more costly repairs. Daytime temperatures during construction varied from 75°F to 90°F. Nighttime temperatures ranged from 45°F to 55°F.

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EQUIPMENT CALIBRATION

Spray Nozzles

Special spray nozzles were fabricated by the research team for use on the experimental test sections. Three sizes of nozzles were supplied for installation in the pressure distributor used on the project. They were standard Roscoe No. 2 nozzles and standard nozzles machined to provide 20 percent and 30 percent increase in volume. Machining was accomplished so that spray width remained equal for all nozzles. These nozzles were placed in the spray bar so that the higher volume was applied outside and between the wheelpaths. Machining was accomplished in Brownwood, Texas, with the help of the Texas State Department of Highways and Public Transportation (SDHPT). The Brownwood district provides nozzles of the type fabricated for this experiment to contractors during construction of chip seals and believes that this practice is largely responsible for the success of the chip-seal program in that part of Texas. Details regarding the fabrication of the special nozzles are discussed in a recent paper by Martin (*1*). Following is a summary of how the nozzles were fabricated for this study.

A set of Roscoe No. 2 nozzles was purchased. The nozzles were grouped according to spray width and checked for volume output using the apparatus shown in Figure 2. Ten nozzles from the same spray width group were placed in the apparatus and volume output measured and averaged. A single nozzle was selected closest to the average volume output of the group of 10 nozzles and used as a reference nozzle. It was desired to create a set of 45 nozzles with known char-

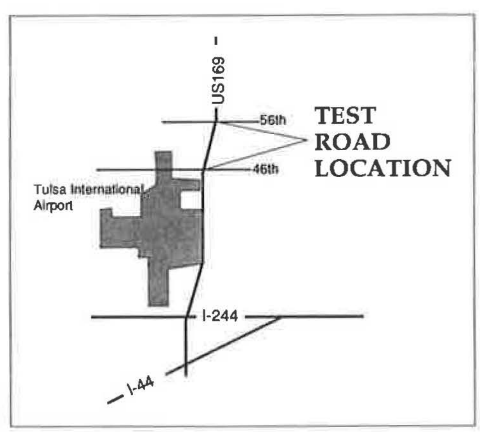

FIGURE 1 Area map of Tulsa test road.

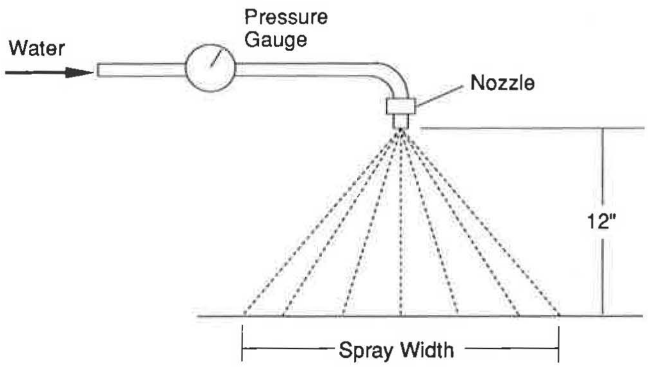

FIGURE 2 Spray nozzle calibration apparatus.

acteristics to produce a potential spraying capability of 15 ft. Each of 45 nozzles with the same spray width were then compared with the reference nozzle for volume output. Any nozzle deviating more than 10 percent from the average volume of the 45 nozzles was discarded. After the set of 45 nozzles with equal spray volume was obtained, 25 of the nozzles were selected for volume modification. These modified nozzles were placed in the spray bar for use outside the wheelpath areas.

Nozzle modification must be done so that an increase in spray volume occurs without a change in spray width. This is done by cutting the v-shaped groove in the nozzle deeper. This process must be done by trial-and-error at first until the amount of cutting can be related to the change in volume output of the nozzle.

The volume output of water was compared for each nozzle in the manner described above to verify that the machined nozzles produced 20 and 30 percent more output than the standard nozzles. Results of this laboratory evaluation were used to determine which nozzles provided accurate enough volume output for use in field tests. Results of the analysis are shown in Figure 3.

Uniformity of spray width was measured for each nozzle. This information is important so that difficulties such as "streaking," "roping," or "drilling" during spray application are reduced. Also, the spray width obtained during calibration can be used to determine correct spray bar height in the field. Spray bar height should be adjusted to produce a minimum of three overlaps from adjacent nozzles.

The equation for determining spray bar height based on laboratory calibration is as follows:

$$H_q = qCN/W \cos \theta$$

where

H_q = spray bar height for q overlaps (in.),
C = nozzle calibration height (in.),
N = nozzle spacing in distributor spray bar (in.),
W = spray width during calibration (in.), and
θ = nozzle angle in distributor spray bar.

This relationship has been used to generate a convenient graph for checking spray bar height given various calibration spray widths, desired overlaps, and two nozzle angles as shown in Figure 4.

The calibration procedure used to derive the relationship for spray bar height and to generate Figure 6 is based on laboratory calibration using water as the spray medium. Water should be less viscous than asphalt materials used in chip-seal construction. This may affect volumetric output of the nozzles but has little, if any, effect on spray width.

The nozzles were positioned in the spray bar of the pressure distributor so that the 20- and 30-percent oversize nozzles were located in areas outside the wheelpaths of the pavement lane to be sprayed. Distribution of traffic across a typical two-lane pavement was measured, and these data were used to determine positioning of the special oversize nozzles in the spray bar. The distribution of traffic and the corresponding nozzle positions are shown in Figure 5.

The spray bar was positioned at a height to produce three overlaps of the spray pattern, as shown in Figure 5. Because of the overlapping pattern a transition between the 20- and 30-percent oversize and standard nozzles occurs in conjunction with the transition of traffic within the lane.

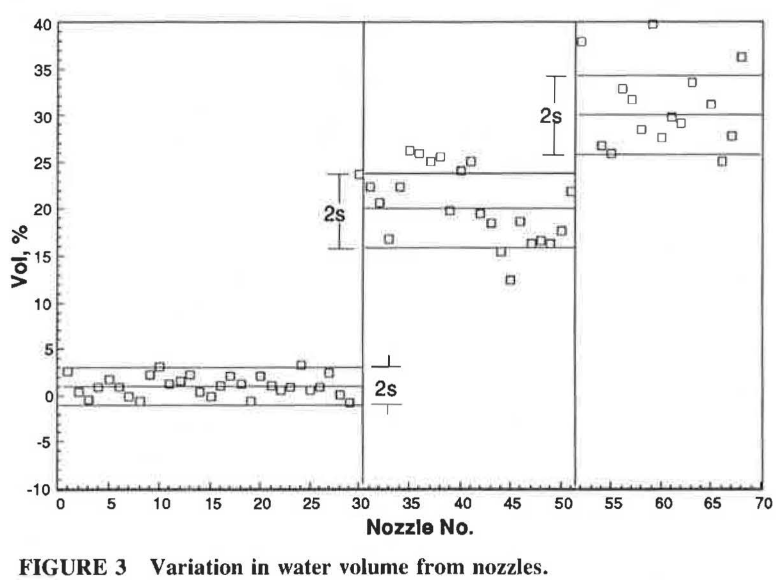

FIGURE 3 Variation in water volume from nozzles.

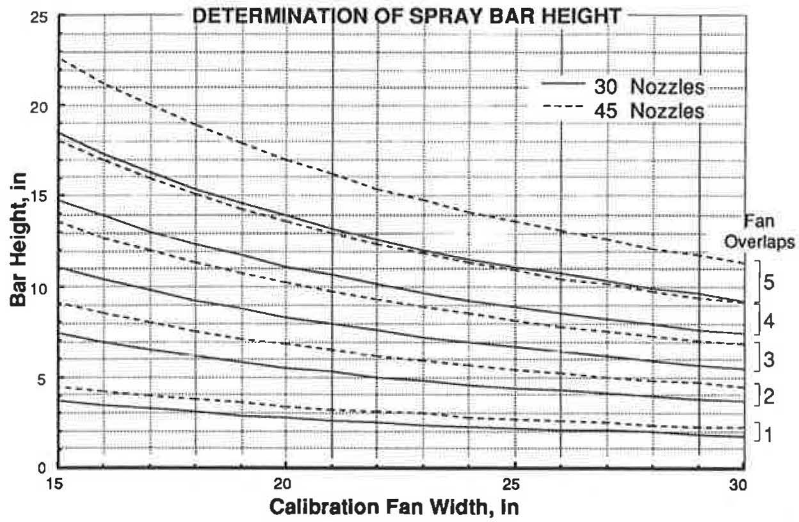

FIGURE 4 Distributor spray bar height determination.

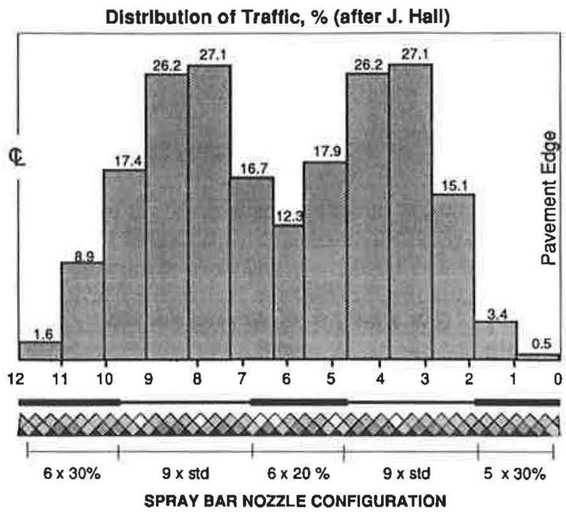

FIGURE 5 Spray bar nozzle position in full-scale field test (data from personal communication with J. Hall).

Pressure Distributor Output

The pressure distributor was calibrated before construction to determine volumetric output of asphalt from each of the spray bar nozzles. Nozzles were selected for use in the distributors based on volumetric accuracy as discussed earlier. Determination of asphalt volume output from each spray nozzle for each pressure distributor was determined as follows: sample containers were placed under each spray nozzle to collect asphalt during discharge from the spray bar; asphalt was sprayed into the containers, and each was weighed. Results of this testing are shown in Figure 6.

According to District 23 Texas Highway Department personnel, variation in volume of 10 percent from the target volume desired for each nozzle group is satisfactory to achieve desired results. Results of testing shown in Figure 6 indicate this variation was exceeded for two nozzles on the left and one nozzle on the right side of the bar. However, it was felt that on heating the bar during spray operations these nozzles would be within the tolerances suggested. Notice the trend to lower volume output in the nozzles located at the edges of the bar.

Aggregate Spreader Adjustment

Two weeks before construction of the test sections the aggregate spreader was inspected and adjusted for lateral spread uniformity. This operation consisted of accompanying the Division 8 maintenance personnel during chip-seal construction of SH127 and observing the appearance of the chips after spreading. Adjustments were made to gate openings on the spreader until a uniform appearance was achieved laterally across the pavement. After construction was completed at this location and uniform spread had been accomplished, the spreader was parked until it was needed for construction of the test sections.

EXPERIMENTAL TREATMENTS

Independent variables to be evaluated in the full-scale experiment included aggregate type, gradation, treatment type (whether single or double application of aggregate), and traffic control. Each was varied in accordance with the following outline:

- Aggregate: natural limestone and synthetic expanded shale;
- Gradation: ⅝-in. and ⅜-in. nominal;
- Treatment types:
  - –First course natural ⅝ in. and second course synthetic ⅜ in.,
  - –Double course natural ⅜ in. over ⅝ in.,
  - –Sandwich natural ⅜ in. over ⅝ in.,
  - –Single course natural ⅝ in.,
  - –Single course synthetic ⅝ in., and
  - –Double course synthetic ⅜ in. over ⅝ in.; and
- Traffic control: pilot car at 15 mph for 1 hr and at 25 mph for 1 hr.

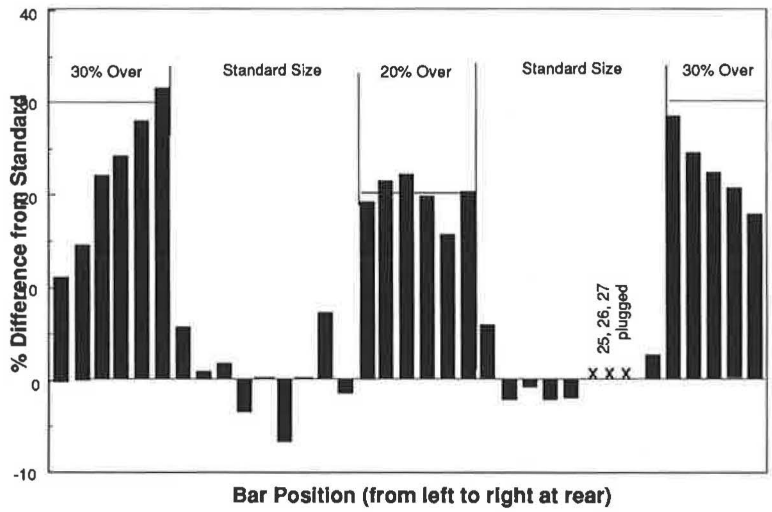

FIGURE 6 Asphalt output from pressure distributor.

The natural limestone was a hard, durable mineral aggregate that is commonly used by Oklahoma DOT for chip-seal operations. The synthetic lightweight expanded shale is locally produced and had never been used by Oklahoma officials for this type construction before. It was desired to experiment with the lightweight aggregate in hopes of reducing the potential for vehicular damage.

The single course applications were both constructed using the larger ⅝-in. aggregate. The larger aggregate has the ability to produce a better seal because of the increased quantity of binder required to hold the aggregate. Also, larger aggregate results in more margin for error in the field because of the increased binder quantity required. However, it was unknown whether the larger aggregate could be retained by the asphalt binder under the high traffic conditions, potential snowplow operations and metal tire studs in the winter.

Single and double application treatments were designed and constructed as conventional multiple chip seals with the exception of the "sandwich seal." The sandwich seal was constructed as follows:

1. The surface was swept clean of dust and debris.
2. ⅝-in. aggregate was placed at approximately 80 percent of the single course application rate; aggregate was placed on dry pavement in loose condition.
3. Asphalt emulsion was applied to the loose aggregate at approximately 90 percent of the design rate for conventional double-application seal.
4. ⅜-in. aggregate was spread over coated first course stone at the design application rate for single course treatment.
5. Conventional rolling by pneumatic equipment was performed when the surface could accept construction traffic.
6. A pilot car was used to control traffic.
7. The section was opened to traffic.

This procedure was developed on the basis of experience gained by the author in constructing similar treatments with the help of the California Department of Highways near Susanville, the New Mexico Department of Highways north of Albuquerque, and through discussion with others (personal communication with Gayle King and Marvin Exline of Elf Asphalt).

Pilot cars were operated to guide traffic over the completed seals immediately after final rolling was complete. It was felt that slow-moving vehicular traffic would benefit the new surface by kneading the aggregate together. The optimum speeds to operate the pilot cars were selected on the basis of past experience with motorist patience (lower speed, 15 mph) and on potential for damage to the new chip seal (higher speed, 25 mph).

MATERIALS

Aggregates

Two types and size gradations of aggregates were used in the test sections. They consisted of a natural crushed stone and a synthetic lightweight aggregate produced by Chandler Materials of Tulsa. Both types of aggregate were produced to meet the Oklahoma DOT gradation requirements for No. 2 and No. 3 chip-seal aggregates as presented in Table 1.

Additional physical tests of the aggregate chips included unit weight, specific gravity, board test (spread rate), flakiness index, and L.A. abrasion.

Results from the other four laboratory tests completed for each aggregate source are presented in Table 2.

These tests were conducted as input information for two chip seal design procedures. The estimated theoretical quantities calculated from the designs were then used for comparison to actual quantities used in the field.

Asphalt Emulsion

The emulsion was a cationic type designated CRS-2S modified using a styrene block copolymer and special processing. Properties of the emulsion are as shown in Table 3.

TABLE 1 AGGREGATE GRADINGS

	3/8" (No. 2)			5/8" (No. 3)		
Sieve Size	Natural	Synthetic	Spec	Natural	Synthetic	Spec
5/8"				100	100	100
1/2"	100	100	100	86	86	90-100
3/8"	95	95	90-100	40	40	40-75
1/4"	31	31		2	2	0-15
4	2	2	0-25	1	1	
8	2	2	0-5	1	1	0-5
80	2	2		1	1	
200	1.6	1.6	0-2	1	1	0-2

TABLE 2 PHYSICAL PROPERTIES OF AGGREGATE CHIPS

Test	3/8" Natural	5/8" Natural	3/8" Synthetic	5/8" Synthetic
Bulk Specific Gravity, ASTM C127	2.559	2.555	1.457	1.542
Unit Weight, Rodded, pcf, ASTM C29	93.0	94.2	52.3	50.4
Board Test, psy	21.2	24.6	17.1	14.3
Flakiness Index, Ref 1, 2	30.3	19.2	2.6	9.1
L. A. Abrasion, loss %	16	16	26	26

TABLE 3 PHYSICAL PROPERTIES OF ASPHALT EMULSION

Test	Test Result
Viscosity, SFS, 122F	379
Demulsibility, %	68.2
Sieve, %	0.01
Residue	
Penetration, 77F, dmm	137
Softening Point, F	116
Viscosity, 140F, P	1437
Ductility, 4C	25
Residue, %	69

CHIP-SEAL DESIGNS

Properties of each of the four aggregates were used to estimate the design emulsion and aggregate application rates by two recognized procedures. Detailed descriptions of the two procedures referred to here as the McLeod design (*2–4*) and the Texas design (*5–7*) have been well documented in the literature, and therefore will not be described here. A summary of the material quantities recommended by each design method is presented in Table 4 for natural aggregate applied as a single seal and in Table 5 for synthetic aggregate applied as a single seal.

The emulsion quantities shown in Tables 5 and 6 have been adjusted for pavement surface condition, temperature (60°F), emulsion water, and maximum traffic allowed for each design. Notice that the two procedures can differ substantially for estimates of emulsion quantity, but agree fairly well for aggregate quantity.

Differences in the way that binder quantities are estimated in each design explain the variation in the results shown in Tables 4 and 5. For example, the residual binder is estimated in the Texas design on the basis of more than 1,000 vehicles per day and adjusts binder content upward for traffic volumes below this level. The McLeod design, however, estimates residual binder based on zero traffic and adjusts binder content downward for traffic levels above this value.

Although both designs agree relatively well for emulsion shot rates with the natural aggregate, significant differences for estimated emulsion rates appear for the two designs when the synthetic aggregates are used, as shown in Table 5. The Texas procedure accounts for the higher absorption probable for many synthetic aggregates, which raises the recommended emulsion application quantities above those suggested by the McLeod procedure.

Another difference in emulsion rate estimates occurs because the McLeod design accounts for all asphalt residue present in the emulsion. That is, the emulsion quantity is simply the residual rate corrected for all water. However, the Texas procedure accounts for emulsion residual providing a higher meniscus on aggregates than does asphalt cement, and depending on the season during construction, the difference between the residual rate and the emulsion rate should be reduced by the amounts shown in Table 6.

TEST ROAD CONSTRUCTION

Construction of the experimental test sections began October 11, 1989, in the northbound driving lane of US-169. Construction proceeded in the direction opposite to traffic to eliminate the need to turn aggregate trucks around.

The six treatments described previously were placed on US-169 in the arrangement shown in Figure 7.

Materials Application Quantities

Materials application rates were measured for every asphalt distributor and aggregate truck load. Emulsion shot rates were obtained by measuring the change in volume for the distributor after each shot using the dipstick supplied with the truck and comparing this to the length of shot obtained from project stationing.

Other methods of obtaining shot rates have been evaluated (*8*) and may offer adequate accuracy as well. One method

TABLE 4 NATURAL AGGREGATE SINGLE CHIP-SEAL DESIGNS

3/8" Surface>	Slight Flush		Normal		Slight Ravel	
Design Method>	Texas	McLeod	Texas	McLeod	Texas	McLeod
Emulsion Rate Inside Wheelpaths, gsy =	0.25	0.18	0.29	0.22	0.33	0.27
Aggregate Spread Rate, lbs/sq yd	21.2	17.1	21.2	17.1	21.2	17.1

5/8" Surface>	Slight Flush		Normal		Slight Ravel	
Design Method>	Texas	McLeod	Texas	McLeod	Texas	McLeod
Emulsion Rate Inside Wheelpaths, gsy =	0.29	0.30	0.33	0.34	0.37	0.39
Aggregate Spread Rate, lbs/sq yd	24.6	25.6	24.6	25.6	24.6	25.6

TABLE 5 SYNTHETIC AGGREGATE SINGLE CHIP-SEAL DESIGNS

3/8" Surface>	Slight Flush		Normal		Slight Ravel	
Design Method>	Texas	McLeod	Texas	McLeod	Texas	McLeod
Emulsion Rate Inside Wheelpaths, gsy =	0.54	0.27	0.58	0.32	0.62	0.36
Aggregate Spread Rate, lbs/sq yd	17.1	14.0	17.1	14.0	17.1	14.0

5/8" Surface>	Slight Flush		Normal		Slight Ravel	
Design Method>	Texas	McLeod	Texas	McLeod	Texas	McLeod
Emulsion Rate Inside Wheelpaths, gsy =	0.51	0.30	0.55	0.35	0.59	0.39
Aggregate Spread Rate, lbs/sq yd	14.3	18.3	14.3	18.3	14.3	18.3

TABLE 6 CORRECTION FOR RESIDUAL ASPHALT

Season of Construction	Emulsion vs Residual Reduction Factor*
Spring	0.60
Summer	0.55
Autumn	0.70
Winter	0.90

* This reduction is not a straight multiplier but rather a percentage of the difference between emulsion rate and residual asphalt. For example, the cooler the weather, the application will be closer to the total residual available in the emulsion.

involves placing preweighed geotextile fabric pads on the pavement in front of the asphalt distributor, allowing the asphalt distributor to spray the pads, and weighing the pads after asphalt application to determine asphalt spray rates. The method outlined by ASTM D2995 follows such a procedure.

Much difficulty has been documented in the measurement of asphalt spray quantities with fabric pads including relatively small sample sizes. Therefore, such techniques were not evaluated.

Volumetric measurements of distributor volume can also be performed using the external gauge supplied with most distributors. However, inaccurate determinations of volume can occur when the external gauge is used because of broken or improperly functioning gauges or foaming in asphalt emulsions. Therefore, this equipment should only be used as an indication of distributor volume and not for determination of accurate asphalt application quantities.

Aggregate trucks were weighed after loading, and the distance required to place a given truck load was obtained from project station distances.

Other methods of measuring aggregate spread rates have also been documented (*8*). The technique often involves placement of sample containers in front of the aggregate spreader to collect chips as they are spread. These containers must be designed to fit under the spreader equipment as it passes so that a representative quantity of aggregate can be collected. The contents of the boxes are weighed and the aggregate quantity obtained. Although no apparent difficulty has been reported with estimating aggregate spread quantities by this procedure, it can be cumbersome if large, representative samples are obtained. Therefore, this technique was not used in lieu of the method of weighing each truck and measuring aggregate spread length.

Materials Application Rates: Actual Versus Design

The actual application rates compared with application rates determined from both the Texas and McLeod design methods are shown in Table 7. The shaded area of the table is provided as an indication of the design that most closely matches the application quantities judged to be appropriate in the field.

Design rates presented in Table 7 were obtained from Tables 4 and 5. Substrate pavement surface condition was considered to be "normal" for both designs. Second course asphalt emulsion application rates were determined by considering the substrate pavement surface condition (first chip application) to be that of a slightly raveled surface, that is, having moderate surface "hunger." First and second course emulsion rates for double application treatments do not appear to match when Tables 4 and 5 are compared with Table 7. This is because,

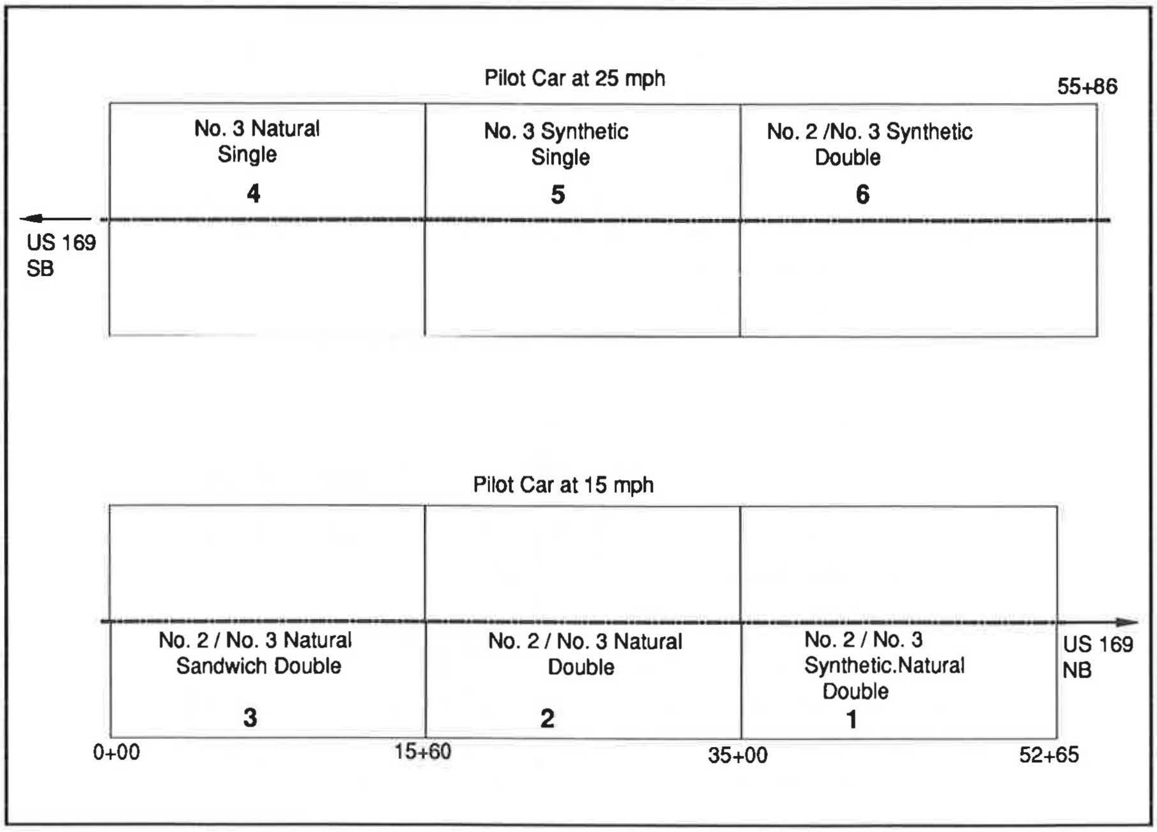

FIGURE 7 Placement of experimental treatments on Tulsa test road.

in accordance with the design procedures, the application of emulsion for the first aggregate course is decreased by an amount that is compensated for during application of the second aggregate course. Note that both design methods estimate the appropriate actual aggregate application relatively well. Although there was agreement between the two procedures for emulsion quantities for the natural aggregate, significant discrepancies occurred for the synthetic aggregate. The Texas design allows for higher embedment when lightweight synthetic aggregate is used, whereas the McLeod procedure does not discriminate between synthetic lightweight and natural aggregate emulsion rates.

TABLE 7 COMPARISON OF DESIGN AND ACTUAL MATERIALS RATES ON TULSA TEST ROAD

		Emulsion			Aggregate		
Section	Description	Texas Design	McLeod Design	Actual	Texas Design	McLeod Design	Actual
1	Double* 3/8"	0.62	0.36	0.42	17.1	14.0	14.4
	Nat/Synth 5/8"	0.33	0.34	0.42	24.6	25.6	26.2
2	Double 3/8"	0.41	0.34	0.24	21.2	17.1	18.0
	Natural 5/8"	0.25	0.23	0.36	24.6	25.6	26.5
3	Double** 3/8"				21.2	20.0	21.1
	Natural 5/8"	0.56	0.57	0.48	20.0	20.0	18.9
4	Single Natural 5/8"	0.33	0.34	0.48	24.6	25.6	29.1
5	Single Synthetic 5/8"	0.55	0.35	0.50	14.3	18.3	23.7***
6	Double 3/8"	0.76	0.45	0.56	17.1	14.0	14.5
	Synthetic 5/8"	0.41	0.26	0.51	14.3	18.3	19.6

* Single 5/8" natural failed during over night trafficiking. Second course of 3/8" synthetic applied next day.

** "Sandwich Seal". Design rates are from past experience of author, and are not related to Texas or McLeod procedures. Emulsion rate is based on 90% of Texas double seal rate. Aggregate rate for 3/8" second course based on Texas 3/8" rate. Aggregate rate for 5/8" first course based on 80% of Texas 5/8" rate.

*** Too high. Adjusted on first course of section 6.

Field Adjustments to Planned Experiment

Test Section 1

Section 1 was originally planned as a single treatment of ⅝-in. natural stone. The design emulsion shot rate for this section as shown in Tables 3 and 6 was 0.33 gal/yd² for the Texas procedure and 0.34 gal/yd² for the McLeod procedure. These shot rates were considered too low by the field personnel responsible for construction, and the actual shot rate was increased to 0.42 gal/yd². However, this increased rate was still not adequate for the conditions, and after approximately 16 hr of traffic the surface began to lose a significant amount of aggregate.

To help prevent further raveling of the surface a second application of smaller ⅜-in. synthetic chips was placed over the remaining ⅝-in. chip seal. The emulsion application rate used in the second application was decreased (0.42 gal/yd²) from that estimated by the Texas procedure (0.62 gal/yd²) and increased slightly from that suggested by the McLeod procedure (0.36 gal/yd²).

Test Section 3

The sandwich seal first course aggregate application rate was too high for several hundred feet at the beginning of the section. The excess aggregate prevents asphalt emulsion from penetrating to the substrate pavement, which causes a potential for disbonding of the new chip seal from the original surface. Disbonding occurred in several isolated locations, which had to be resealed.

The setting time for the emulsion in this section was significantly longer than that required for other double application seals in the experiment. This was evidently caused by the high quantity of emulsion sprayed at one time for the sandwich seal. Although the other double application seals also had high binder contents, the emulsion was sprayed in two stages, allowing the first application to set before the second application was applied.

Traffic Control

Pilot cars were used to control traffic speed on the project. Some difficulty was experienced initially in getting traffic to follow the pilot cars onto the new chip seal surface. Because the passing lane had not been sealed, drivers had a tendency to drive on the smoother asphalt concrete surface and avoid the new chip seal. However, a plan was developed to force traffic onto the chip seal surface so that an evaluation of the beneficial effects could be judged. Speed of the pilot cars was reduced to 15 mph for 45 minutes, but traffic would not follow at this speed. Drivers passed the pilot cars on the shoulders and the median. The pilot car speed was subsequently increased to 25 mph. This speed solved the problem of impatience on the part of the motorists and did not have a detrimental effect on the chip-seal surface. It is believed the slow-moving traffic had a beneficial effect in helping embed chips better than the pneumatic rollers.

Scheduling completion of construction on the chip seal during the lowest volume traffic period (before the 4:00 p.m. rush hour) is believed to have contributed to the lack of complaints and vehicular damage documented.

Return of Traffic to Facility

The construction sequence required approximately 2 hr before traffic could return to the new chip seal behind the pilot vehicles. Pilot vehicle trafficking was conducted for 45 min for the 15 mph sections and 1 hr for the 25 mph sections. Traffic was returned to the pavement without pilot vehicles after these periods. Some sweeping was required the day after construction to remove loose aggregate dislodged by traffic.

Complaints and Damage Claims

The highway department received six telephone calls the day after construction of the first three seals. Each of the calls was regarding the noise generated by the seals and the change in texture of the pavement surface at the transition between the sealed surface and the smoother underlying asphalt concrete.

No calls or claims of vehicular damage were received.

SUMMARY AND CONCLUSIONS

1. An experimental chip seal was constructed on a 4-lane divided pavement with 38,000 ADT using 2 types and gradations of aggregate resulting in 6 experimental test sections approximately 1 mi long. No vehicular damage claims were received by the agency constructing the experimental treatments.
2. Two chip seal designs were used before construction to estimate the actual application rates of emulsion and aggregate. Comparison of the design material quantities with actual rates considered appropriate for the conditions indicates that one method was better at estimating aggregate spread quantities and neither design adequately estimated proper quantities for asphalt emulsion application rate. Both design methods estimated approximately the same emulsion rates for the natural aggregate, but disagreement occurred when emulsion rates for the synthetic aggregate were estimated.
3. Variable volume nozzles were fabricated for use during construction. Spray volumes were adjusted so that approximately 30 percent more binder was placed outside the wheelpaths and approximately 20 percent more binder was sprayed between the wheelpaths.
4. Traffic control using pilot vehicles traveling at 25 mph to keep traffic on the new chip-seal surface is considered mandatory for 1 hr following final rolling. This trafficking appears to benefit the early performance of the seal by improving initial embedment and voids reduction. Operation of the pilot vehicles at 15 mph was too slow and resulted in motorists attempting to pass the pilot vehicles. Scheduling construction to begin after and end before rush hour is believed to have contributed to the overall cooperative nature of motorists.

5. Adhesion testing was done using a modified Vialit procedure. Results of the testing indicate that adhesive strength of emulsions is dependent on setting rate and that setting rate depends largely on ambient temperature and wind conditions, as might be expected. Therefore, use of the Vialit test for field evaluations may be suspect unless ambient conditions can be controlled.

6. A modification to the Vialit test using glass marbles in place of mineral aggregates appears promising for laboratory evaluation of adhesive properties of emulsion chip-seal binders. Repeatability of the test is greatly improved compared with results obtained using mineral aggregates. Because the test does not use actual project aggregates, however, utility for estimating binder adhesive qualities for specific aggregates is low. However, it would be possible to compare results of new binders with a known standard for relative evaluations.

7. Correlation of the field Vialit test and the modified laboratory version appears promising. Comparison of results for an HFE-100S used on a full-scale experiment in New Mexico indicates relatively good agreement between a new accelerated laboratory version of the Vialit test and results obtained during actual field evaluations.

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